Railway embankments composed by soil and rockfill mixtures

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Abstract: Due to the newly designed Portuguese high-speed railway, as part of the future link between Lisbon and Madrid, the use of new geo-materials required a special treatment. The application of these on embankments and given the importance of the platform deformation for railways, imposed a solid study of their properties and consequent behaviour. This was achieved through a series of laboratorial tests, that allowed the definition of constitutive parameters. Consequently, the results obtained were applied to computational modelling and the expected behaviour was evaluated. A preliminary analysis and design of the railway sub-structure was also carried out.

Keywords: Embankment, Mixture, Soil, Rockfill, Deformability, Railway

1. INTRODUCTION

The common relation between every human edification is its relation with the soil. The foundations relate themselves to the structure's behaviour through the influence on deformations or stresses. When using geo-materials, which aren't produced on a controlled environment such as concrete or steel, a proper study of their characteristics is required. They also can interact actively with the environment, so they are a constant target of research.

Many methodologies have been developed along the years to predict the behaviour of geotechnical structures. The data gathering through in situ investigation, laboratory testing and field observation are current practice for studying these materials. More recently, the support of structural design through computational modelling has become essential.

This paper has the objective of carrying out laboratory testing on two materials, composed by soil-rockfill mixtures, that occur on regions to be crossed by the first Portuguese high-speed railway, to be used on the construction of embankments. There are two essential characteristics on the analysis of the problem. Firstly, the use of soil and rockfill mixtures imposes some uncertainty in predicting their behaviour, due to the soil and rockfill particles interaction. Secondly, the importance of reducing embankment deformations, through an appropriate geo-materials use, due to their proved relation to a maintenance operation and corrective measures implementation on railways.

To analyse the short and long term effects of the soil-rockfill mixture embankments, a computational modelling analysis was implemented. From this, some conclusions were drawn about the initial deformability and creep effects after compaction and material wetting. Another important analysis focuses on slope stability and on the behaviour after material wetting. A preliminary design of the railway sub-structure was also carried out, in order to provide a basic definition of the platform's constitution and the finishing of the embankments, through the capping.

2. EMBANKMENTS AS RAILWAY PLATFORM

With the increase of passenger mobility on Europe, the usual transport systems suffered an overload. Therefore, in order to make use of new transport systems such as the high-speed trains, proven profitable and environmentally friendly, the European Union developed a transport policy specifically design for this. It aspires to, in the future, create a European high-speed railway network capable of giving a real alternative land-transport solution, which could be integrated with other systems. With an average growth of 11.1% for passengers using high-speed trains since 1990 [**1**], Portuguese railway authorities found crucial to the country's future, the integration on this network, thus resulting in the new link between Lisbon and Madrid. The first section to be constructed is located between Poceirão and Caia, where the geo-materials studied on this paper can be found (Fig. 1).

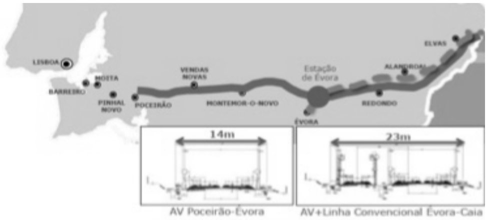

Fig. 1 – General characteristics of the first section of the high-speed line Lisbon-Madrid (www.rave.pt).

In the section in focus, the more conditioning embankments are situated between Évora and Caia, where the railway has three separate lines, two for passengers and one for cargo. The general external conditions that can limit the design of embankments are basically related to their needed height and the nature of their foundation. In the first case, previous experience [**2**] proves that maintenance works on the railway are directly correlated to their height, meaning that the deformation occurred must be corrected more frequently by the repositioning of the ballast through tamping operations (Fig. 2).

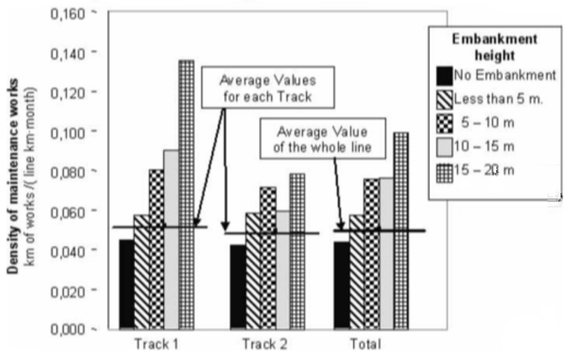

Fig. 2 – Maintenance work density on Madrid-Seville high-speed line, according to embankment height (Pita, 2006).

The second case relates to the quality of the foundation, which wasn't part of this paper, and to the declivity of the embankment's base. Typically, embankments with a steeper base, tend to register more significant deformations and also a very peculiar behaviour – differential settlements due to sliding effects. A comparative analysis was developed between two embankments with different base and height conditions.

Regarding the specificity of the application of soil and rockfill mixtures, a few important notes should be referred. According to the *Junta Autónoma de Estradas* (Portuguese state authority for roads), the definition of a soil-rockfill mixture must comply with the following rules.

1. 30 to 70% of the material must be retained on the 19 mm sieve.
2. 12 to 40% of the material must pass through the 0,074 mm sieve (percentage of the fine portion).
3. The biggest dimension of rock particles must not be superior to 2/3 of the compaction layer or 400 mm.

Although after compaction a reduction on the particle size is frequently registered, specially for those of larger dimensions, the behaviour of these materials are peculiar enough to justify their detailed characterization. The parameters for these materials are, in general, similar to those for soils and rockfill. The only important difference worth mentioning, would be the fact that these geo-materials share a shear strength law similar to the one registered for pure rockfill embankments [**3**], but deformations prone to soils. The shear strength law consists of a reduction of the shear strength with an increase off the confinement stress. This wasn't considered herein, due to the lack of material on the laboratory, given the preliminary stage of designing. To account for this situation, the shear strength evaluation was obtained for 250 kPa of confinement stress, which is the highest expected confinement stress to be registered on the embankments.

When compared to rockfill, the soils tend to register a worse behaviour, both in strength as in creep. Therefore, it was expected that the mixtures would behave slightly better than a soil of the same origin. The geo-materials studied on this paper are a Devonian Shale and a Gneiss with Migmatite (from Granitic origin), from now on referred to as *MSE1* and *MSE2*, respectively. These materials occur specially between Évora and Caia (Fig. 3), being the materials with the more application on the embankments.

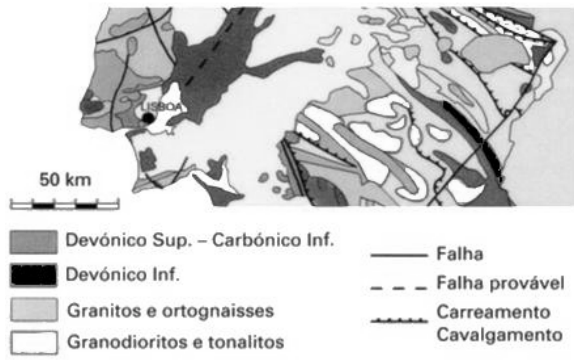

Fig. 3 – Portuguese geological chart: high-speed railway's region (*INETI*).

3. GRAIN SIZE DISTRIBUTION AND COMPACTION

In order to determine the initial conditions of the mixtures, it was essential to conduct a grain size distribution by sieving. This one was made accordingly to *LNEC*'s specification E 196-1967, which makes use of *ASTM* sieves. The sieving was also applied to the samples after all the laboratory tests, after drying. From the grading curves obtained the first conclusion taken, is that *MSE1* meets, more adequately, the definition for a soil-rockfill mixture, but *MSE2*'s grading curves are close enough to imply a behaviour similar to a soil-rockfill mixture. The second obvious conclusion was that after compaction there are some grading evolution, but not significantly enough to classify the samples as soils. Due to the larger *MSE2*'s grading, with samples wetting, the fracture of the rockfill particles explains the bigger gap between the field and post-test grading obtained for this mixture.

Regarding the optimum compaction conditions, an heavy compaction test, also known as modified *Proctor* test, was used in order to determine the maximum dry unit weight and the optimum water content. An automatic compactor was used, in order to obtain a more even sample and because of the mould's dimensions. Due to the small quantities available for each geo-material, the Hilf's Method was used for the interpretation of the compaction tests. The use of this method allowed the determination of the compaction curves resorting only to three points, sparing material for further laboratorial testing. The results obtained, used on the preparation of further samples for strength and deformability testing, are summarized on Table 1.

Table 1 – Optimum compaction conditions.

	MSE1	*MSE2*
w_{opt} (%)	9.4	6.7
$\gamma_{dry.max}$ (kN/m^3)	20.74	21.69

It was concluded that *MSE1*, due to it's schist origin, requires a higher value of water content, when compared to *MSE2*, from granitic origin. It was also expected that, based on the lower dry unit weight, *MSE1* would register a lower shear strength. This was proven not true, possibly due to the fracturing registered on *MSE2*'s particles, although the results for both mixtures were very similar in what comes to shear strength.

4. STRENGTH AND DEFORMABILITY

All civil engineering projects resorting to any material have a common requirement: the definition of strength and stiffness of those materials. Accordingly, for the geotechnical design of embankments, laboratory testing was conducted for the determination of parameters.

Firstly, a one-dimensional deformation test, an oedometer test, was performed to determine the parameters necessary to define the strains as a function of stresses. A chamber, as seen on Fig. 4, was used to determine the deformation parameters, which were: the compression index (C_C), the expansion index (C_S) and the secondary compression index or creep parameter (C_α). These values were latter adjusted by the modelling, which resorts to Young's modulus to represent deformability. Nevertheless, on Table 2 the values for the laboratory determined parameters can be found.

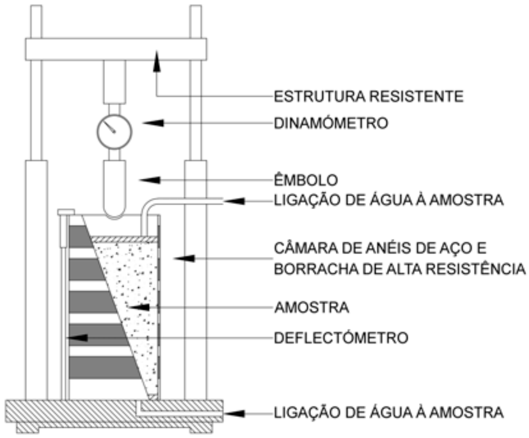

Fig. 4 – One-dimensional deformation test chamber, for soil and rockfill mixtures.

The values obtained predict a worst deformation by *MSE2*, with an improvement on its behaviour after submerging, due to the rearrangement of the material's structure by fracture. In general, *MSE2* is worst than *MSE1*.

Table 2 – Deformation parameters.

Sample		*MSE1*	*MSE2*
C_C [.10^{-5}]	Dry	4,33	5,49
	Submerged	3,37	2,61
C_S [.10^{-5}]	Dry	0,94	3,04
	Submerged	0,62	0,81
C_α* [.10^{-4}]	Dry	4,37	4,57
	Submerged	6,37	6,49
* Average value, for software application.			

In terms of strength, a triaxial compression shear test was conducted for the determination of the corresponding parameters. This one was a consolidated undrained (*CU*) test, meaning that the samples were previously consolidated (isotropic compression phase) while being simultaneously saturated and that the shear compression phase took place without drainage. The tests were made in a large dimensions chamber (Fig. 5), specially designed for geo-materials rich in rockfill particles. These materials have, typically, a high value of internal friction angle, therefore the height of the sample must be considerably higher when compared with the diameter. Otherwise, the rupture surface might cross the bottom or the top of the sample, overestimating the material's strength. It's important to mention that the test was conducted with a confinement stress of 250 kPa, in order to simulate the highest values registered on the embankments, and the saturation had the objective of reducing the shear strength in order to simulate the worst conditions for the embankment's design.

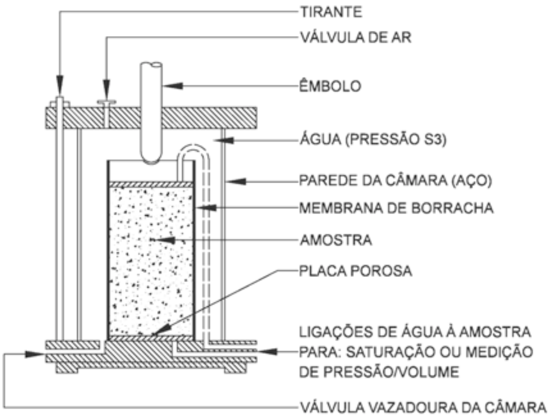

Fig. 5 – Triaxial compression shear test chamber, for soil and rockfill mixtures.

In order to define the strength of the mixtures, two parameters were defined: the peak internal friction angle and the ultimate internal friction angle (Table 3). These allow to define the stress states that, if exceeded, lead to the rupture of the geo-material. When compared with other materials, the values obtained are considered to be typical and both materials have about the same residual strength.

Table 3 – Strength parameters.

Sample	*MSE1*	*MSE2*
Φ'_P (º)	56,4	41,2
Φ'_U (º)	43,2	40,5

5. MODELLING: SOFTWARE'S CALIBRATION

As information technology advanced, naturally civil engineering gained support by these new developments. Modelling was one of the new techniques that helped engineers to quickly test materials performance before signing off any design. This paper made use of this ability, modelling two different embankments designed for the Portuguese high-speed line, resorting to both geo-materials analysed. The software used for modelling was *Plaxis 2D* for stress-strain analysis, and *GeoStudio 2007* for slope stability analysis. As a good practice on engineering, all systems must be calibrated, so the results may be, at least, plausible or as accurate as possible.

The calibration process took place in two phases, for an adequate representation of the deformation and the strength. This was obtained by a reproduction of the experimental samples with the software, and with a simulation of the tests already mentioned. The stress-strain analysis was accomplished with the initial introduction of the parameters obtained by laboratory testing and a subsequent adjustment of these, with the objective of meeting the same results (Fig. 6). Regarding strength, the software adequately simulated the laboratory results (Fig. 7), even though this was better accomplished to *MSE1* than for *MSE2*. It's important to refer that due to convergence problems related to the iterative process, the strength analysis resorted to a total stress path, instead of the laboratory obtained effective stresses path.

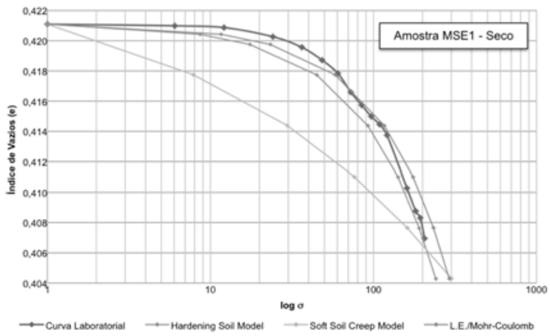

Fig. 6 – Modelling calibration regarding deformations for *MSE1*'s dry sample – simulation of the one-dimensional deformation test.

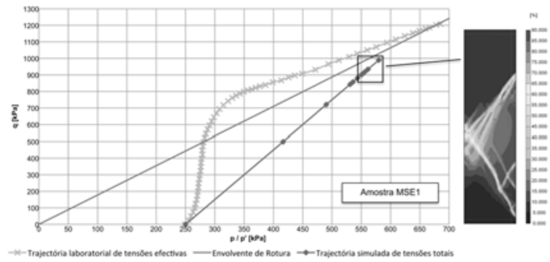

Fig. 7 – Modelling calibration regarding strength for *MSE1*'s sample – simulation of the triaxial compression shear test.

From the calibration it was possible to obtain, not only the best-fit value for the software's parameters, but also the more adequate model to apply, given the various choices the software provided. It was concluded that the Hardening Soil Model was the one that offered the best similarity to the real behaviour, even though it didn't allow the input of the secondary compression index, as the Soft Soil Creep Model would. The differentiating characteristic that made the Hardening Soil Model more adequate is the possibility of defining a hyperbolic law to link the Young's modulus with the confinement stress. The definitive parameters used on the embankment's modelling can be found in Table 4.

Table 4 – Parameters used on *Plaxis* for embankment modelling.

Hardening Soil Model					
Parameters	Units	*MSE1*		*MSE2*	
		Dry	Wet	Dry	Wet
γ_{unsat}	kN/m^3	21,41		21,21	
γ_{sat}	kN/m^3	23,83		23,78	
k_x / k_y	m/s	1,331E-09		1,854E-06	
e_{init}	-	0,4211		0,3118	
K_o^{nc}	-	0,3155		0,3506	
ν	-	0,2398		0,2596	
p_{ref}	kN/m^2	100		100	
E_{50}^{ref}	MPa	25,28	34,28	13,24	17,24
E_{oed}^{ref}	MPa	20,87	28,87	12,51	16,51
E_{ref}^{ur}	MPa	65		32	
m	-	0,05	0,25	0,25	0,4
ϕ	º	43,2		40,5	
The remaining parameters were calculated and/or suggested by the software.					

6. SHORT-TERM BEHAVIOUR

After the appropriate definition of the parameters that would better represent the mixtures real behaviour, a plane strain analysis took place, for real cross sections of the track. These sections were chosen based on the disadvantageous conditions for their behaviour, such as average height or base declivity. The cross sections represented on Fig. 8 and Fig. 9 where chosen and, even though it's not expected for both *MSE1* and *MSE2* to be used on the same region, for a comparison between the mixtures behaviour, both were used for each of the two cross sections. Note that the rockfill on the slope's base for the downhill embankment was considered necessary after an analysis with the submersion of the first two layers, considered during the construction phasing. This one has the objective of assuring stability and to contribute to a reduction of the deformations. The horizontal based embankment has a maximum height of 24 meters, while the downhill based embankment has 15 meters measured on its axis. Both have a width of 24 meters at the crest, due to the section imposed by the railway's infrastructure.

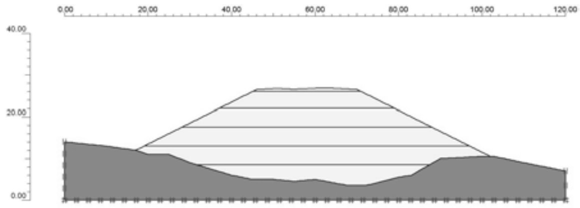

Fig. 8 – *Plaxis* simulation of the conditioning cross section for the horizontal based embankment (km 6+650).

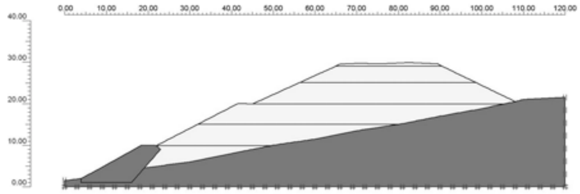

Fig. 9 – *Plaxis* simulation of the conditioning cross section for the downhill based embankment (km 18+425).

A correct designing approach would require an adequate characterization of the foundation's parameters. Due to the lack of information on the foundation associated to the chosen cross sections, as a simplification of the problem, the foundation's parameters were obtained from the ones obtained for the mixtures (Table 5), assuming that the mixtures would only be used over a foundation with a similar origin. Given the simplification, a more simple model was chosen – the Mohr-Coulomb Model – which applies a constant Young's modulus (with higher value – simulating rigid conditions) until plasticity takes place on the geo-material.

Table 5 - Parameters used on *Plaxis* for embankment's foundation modelling.

Mohr-Coulomb			
Parameters	Units	Foundation (*MSE1*)	Foundation (*MSE2*)
γ_{unsat}	kN/m^3	19	
γ_{sat}	kN/m^3	21	
k_x / k_y	m/s	1,331E-09	1,854E-06
ν	-	0,3	
E_{ref}	MPa	200	
ϕ	º	50	
The remaining parameters were calculated and/or suggested by the software.			

After the definition of the initial conditions, it was possible to obtain an average of the primary settlements registered in each embankment, resorting to *MSE1* and *MSE2* (Table 6). The process also included an analysis on the effects of submerging the first two layers after the embankment's construction, at approximately the first 8 m of the embankments (Fig. 10), from which was possible to verify an increase on the displacement's overall value. Given that all these settlements are immediate, as a direct result of the material's own weight, they can be corrected during the construction phase, not posing a direct threat to the railway's operation.

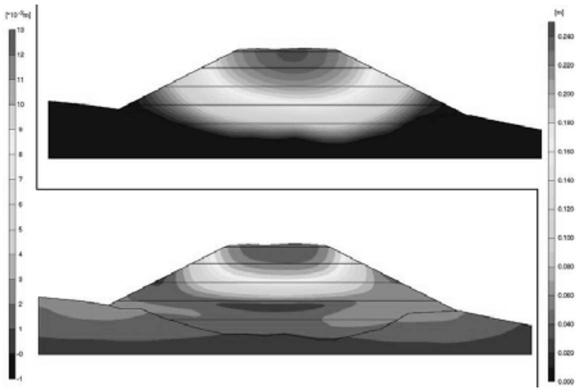

Fig. 10 – Total vertical displacements after construction (above) followed by submersion (bellow), for *MSE1*'s horizontal based embankment.

Table 6 – Average primary consolidation settlements on the embankment's crest.

Horizontal based embankment		Downhill based embankment	
MSE1	*MSE2*	*MSE1*	*MSE2*
Average settlement after construction [m]			
0,232	0,420	0,150* 0,073**	0,255* 0,126**
Average settlement after submersion [m]			
0,012	0,022	0,016**	0,019**
* Without slope-base rockfill		** With slope-base rockfill	

It was also interesting to find the relatively small impact of the high-speed trains on the embankment's internal stresses, due to an adequate stress distribution by the railway's sub-structure. The weight of the railway's infrastructure was also considered to be negligible. Although the influence of the trains was neglected on this paper, the study of these effects may be relevant, due to fatigue, associated with cyclic loading. A slope stability analysis was also conducted, leading to the conclusion that the designs provided were adequate to the use of *MSE1* and *MSE2*, even after the submersion in the conditions already described. Note that the results obtained through *GeoStudio* were confirmed by *Plaxis*, which helped to validate the results obtained (Fig. 11). The collapse is expected to happen only when safety factor (*SF*) below 1.0 occur, and the design is considered safe for *SF*'s values over 1,0 when the design parameters are used in the analysis. This value was achieved or exceeded for all dry conditions, being 1,05 the lowest value registered, after submersion of *MSE2*'s downhill base embankment.

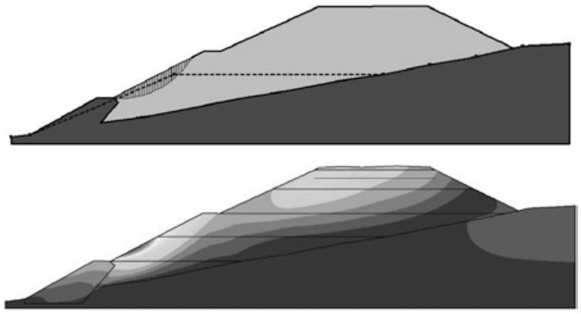

Fig. 11 – Rupture wedge on *GeoStudio* with SF=1,05 (above) and total displacements on *Plaxis* (bellow), after submersion of the first two layers.

7. LONG-TERM BEHAVIOUR

More interesting for the railway context, is the long-term analysis of the embankment's behaviour, since any settlements that may occur, after the construction works are finished, can only be corrected through the tamping of the ballast. This requires the interruption of the train traffic for maintenance operations. The referred settlements are mainly associated with the creep phenomenon, also referred as secondary consolidation. In order to control the quality of the analysis resorting to *Plaxis*, given that the Hardening Soil Model does not allow the input of C_{α}, a comparison with a displacement obtained analytically was necessary (Table 7). The same determination was made considering the possibility of submersion, of the first two layers on each embankment, during the first year of operation (Table 8). As expected, the results were considerably higher, with results spanning from a 20% to a 40% increase, approximately (for the experimental values).

Another conclusion was that the software did not represented adequately the creep behaviour for every situation and therefore it could not be trusted for a long-term analysis. Nevertheless, for *MSE1*, the results were considered to be sufficiently approximate.

Table 7 – Average secondary consolidation settlements on the embankment's crest.

Time [years]	*MSE1* Exp. [m]	*MSE1* *Plaxis* [m]	*MSE2* Exp. [m]	*MSE2* *Plaxis* [m]
Horizontal based embankment				
1	0,0173	0,0084	0,0196	0,0392
5	0,0221	0,0187	0,0250	0,0392
10	0,0241	0,0232	0,0273	0,0392
20	0,0261	0,0249	0,0296	0,0392
Downhill based embankment				
1	0,0118	0,0065	0,0134	0,0231
5	0,0150	0,0133	0,0170	0,0231
10	0,0164	0,0149	0,0186	0,0231
20	0,0178	0,0156	0,0202	0,0231
Note: the experimental settlement value was determined for the embankment's axis height.				

Table 8 – Average secondary consolidation settlements on the embankment's crest, after the submersion of two layers.

Time	*MSE1*		*MSE2*	
[years]	Exp. [m]	*Plaxis* [m]	Exp. [m]	*Plaxis* [m]
Horizontal based embankment, 2 layers submerged				
1	0,0198	0,0199	0,0279	0,0538
5	0,0252	0,0251	0,0355	0,0538
10	0,0276	0,0265	0,0388	0,0538
20	0,0299	0,0273	0,0421	0,0538
Downhill based embankment, 2 layers submerged				
1	0,0191	0,0243	0,0160	0,0426
5	0,0243	0,0293	0,0203	0,0426
10	0,0265	0,0303	0,0222	0,0426
20	0,0288	0,0308	0,0241	0,0426

Note 1: the experimental settlement values was determined for the embankment's axis height.
Note 2: the *Plaxis* settlement values include the parcel due to wetting.

8. RAILWAY'S SUB-STRUCTURE DESIGN

Although the objectives of this paper focused on the geotechnical analysis of the embankments and the characterization of two materials, it was considered relevant to introduce a preliminary design for the railway's sub-structure. This was carried out accordingly to the suggestions made by the International Union of Railways (*UIC*) on its regulation 719-R (3rd Edition, 2006). The following values were obtained for the sub-structure's constituents: 0,35 m for the capping layer thickness, 0,10 m for the sub-ballast and at least 0,30 m for the ballast. With the embankment creep, it is clear that the ballast layer will have to be thickened along the years (Fig. 12), in order to compensate for the settlement previously presented.

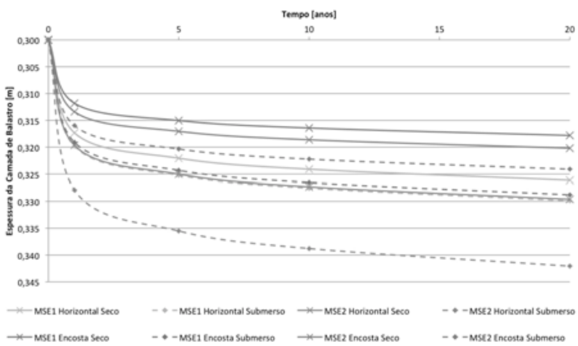

Fig. 12 – Progress of the ballast's required thickness.

9. CONCLUSIONS

The main conclusion drawn from this paper is the importance of the characterization of soil and rockfill mixtures, due to their particular nature, which can be easily influenced by any of their main constituents. Their behaviour is clearly at half way from a rockfill and a pure soil, meaning they can't be described by previous experience.

It was also proven that both materials are adequate for the desired application, having both a very good performance, even though *MSE2* revealed itself to be more deformable. Even after being submerged, both mixtures reveal an acceptable behaviour, though this may lead to stability problems. The analysis by submersion proved the importance of the drainage system and its adequate maintenance. Still, the slopes declivity was considered to be appropriate for the materials in use.

The modelling phase led to the conclusion that introducing directly the laboratory obtained parameters does not suffice for a good analysis. This conclusion is of extreme importance for geotechnical engineers, proving that for better safety on design all results must be proven adequate for behaviour's simulation. Even though the Hardening Soil Model did not represented adequately the settlements by secondary consolidation, it was considered that using the results obtained wouldn't be inappropriate, due to their conservative nature, providing safety on design.

One of the surprising results was the negligible influence of the high speed train's weight. Nevertheless, its recommended as a future development on this paper, an appropriate study of this phenomenon. In this case, the considerable height of the embankments, lead to values of stress significantly superior to the ones caused by the trains.

From the geotechnical engineering scope, the study of these materials and their interaction with any structures is extremely important to improve safety on the construction's technology and reduce the costs due to unpredicted behaviour. For railway engineering, the Portuguese high-speed project poses new challenges, which might be more easily overcome through research, one of the simplest ways of supporting the development of new transport technologies.

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