Article

Modifications for Optimization the Compaction Quality Control in Core Slate Random Filling Embankments for Linear Infrastructures

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Abstract: The construction of random fillings from the excavation of medium hardness rocks, with high particle sizes, presents limitations in compaction control. This research applies new control techniques with revised test procedures in the construction of the random fillings core, which constitutes the main part of the embankment, with the bigger volume and provides the geotechnical stability to the infrastructure. The maximum layer thickness researched was 800mm. As there are many types of rocks, this research is applied to metamorphic slates. Quality control has been carried out by applying new research associated with the revision of wheel impression test, topographic settlements and plate bearing test (PBT). A statistical analysis of the core of 16 slate random fillings has been carried out, with a total of 2250 in situ determination of density and moisture content, 75 wheel impression tests, 75 topographic settlement control and 75 PBT. The strong associations found between different tests have allowed to simplify the quality control.

Keywords: Random filling; slate rock; core; wheel impression test; topographic settlement test; plate bearing test.

1. Introduction

The Construction Embankment Technical Guide [1] recommends compaction control at stone fillings by procedure. It limits the maximum size to 800 mm for the operation of the compaction rollers. Teijón-López-Zuazo et al [2] show that in the construction of fillings with stone materials, compaction have limitations that avoid the optimal quality control.

The General Specifications for Roads and Bridges Works PG-3 [3] prescribes macro pits with a minimum surface of 1 m^2 and 1 m^3 of volume. So, is complex to obtain gradings weighing fractions of different sizes of rocks. For Teijón el al. [4], the nuclear methods for the obtention of density and humidity in situ are not adequate in random fillings for a tested thickness of 300mm when the layer thicknesses are usually 600mm. Also, the particle sizes reduce the significance of the test. The density by substitution methods such as sand are not correct, the high hollow introduces errors, limiting its application to 50mm.

The modified Proctor test is not a correct reference for the degree of compaction, since it is carried out by substituting material bigger than 20 mm, which in this case is the biggest fraction. Finally, as the plate bearing test (PBT) is a point test, to be representative the diameter of the plate must be 5 times the maximum size, which results in sizes outside the test procedure. The control of topographic settlements needs adequate references, according to Sopeña [5].

The French standard NF P98-736 [6] classifies the compactors according to their load per wheel (CR) as P1, with CR values between 25 - 40 kN, P2 for CR between 40 - 60 kN and P3 for CR > 60 kN. For Fernandez and others [7], test sections are necessary because of the limitations of the compaction control. The tests carried out in this research are shown in Table 1:

Table 1. random embankments quality control summary

Test	Limitations	Spanish procedures
topographic settlements	no reference values	on test sections
automatic online monitoring	strong influence on human behavior	not applied
pit gradings	not very practical	on test sections
wheel track testing	usually works	in compaction batches
plate bearing test	diameter of the element 5 times the maximum size	requires the diameter of the element to be 5 times the maximum size
nuclear density gauging	particle dimensions	not recommended, in compaction batches only correlated with other tests
modified proctor	replacement 70% fines	ussually reference to maximum density and optimum moisture
sand method	maximum size < 50mm	in compaction batches

The main reasons proposed in the new method are to define a new method of compaction control applying only representative tests and thresholds, and the revision of procedures in the wheel-tracking test and in the topographic settlement avoiding limitations. The statistical analysis of the main compaction trials has been carried out, obtaining correlations. Due to the large size of the explanations, the study has been particularized to the core zone, with a maximum layer thickness of 800mm. The tests carried out have been applied the revised procedures of the topographic settlement and the wheel impression test [2]. Among the different types of rock, research is focused on the family of slate rocks. Fernandez et al [7] consider that rocks with a single compressive strength below 25 MPa produce random fillings of sufficient quality, performing test sections with excellent results. Rahman et al [8] relate the compactor placed on the vibratory rollers with the calculation of the instantaneous density, finishing the compaction when the dynamic module is reached. This method can be used to locate soft spots by means of a global positioning system (GPS). Oteo [9] considers granulometry and density as the main parameters to be defined for use in the formation of fillings. Lim [10] says that Korean road specifications include a 30-centimeter layer thickness that avoids the use of rocks. There are also no specifications for quality control with stone materials. He proposes obtaining the density on site by the "water replacement method" within the inspection pit. For Sakaia et al. [11] the revision of the Road Embankment Earthquake Manual does not sufficiently consider the influence on the mechanical behaviour of soil compaction. Triaxial tests have associated the highest load deviation with the compaction degree. One-dimensional consolidation tests have allowed a linear adjustment of the compression curve. The highest dry density corresponds to the lowest compressibility, although overconsolidation can produce the collapse of the structure.

The compaction procedure for random fillings, in accordance with PG-3 [3], should define the optimum moisture content, the number of passes, the maximum layer thickness, and the machinery to be used for earthworks. Kyung-Tae et al. [12] investigate the execution of a rock embankment built by dynamic compaction performing PBT. Due to dynamic compaction, an increase in pressure can develop in the foundation. A hyperbolic model associated with the construction method was adjusted to the seat. The estimated results were compared to the settlements and the results of PBT.

Oteo [9] considers that geophysical prospecting techniques, in addition to the plastic sheet substitution method, are suitable for the control of random fills. Nuclear methods present problems such as the penetration of the emitting rod between rock fragments. It can be measured by backscattering, that is, by direct emission from the surface, although the results are not reliable because they correspond to the most compacted area. In the study of space exploration, research has been carried out to estimate soil density by means of drone drilling. Similar methods can be applied to quality control in compaction. Iai et al. [13] fit a model by obtaining the density of the raked soil by raking force. The application of the method allows the support of the drone on the Moon or Mars in addition to the Earth. Scale models were made with JSC-1a (artificial lunar regolite), obtaining relationships between the density of the soil or lunar regolite, the ripple force and the spacing and number of scarifiers. The instrumentation allows a high resolution mapping of the density of the raked site, providing an in situ calibration of the ground by remote control from the Earth. For Wu and Wang [14], the effect of the time between the layers on field compaction must be considered in the construction of filler. For longer surface exposures, moisture tends to evaporate, and test results change. With the Clegg soil impact test hammer, compacted Xiangshan sand was practical for dry density measurement. The force of compacted sand and compaction effort correlated well with the soil impact test hammer. The main factors influencing the compacted Xiangshan sand were moisture and degree of compaction. Lower compaction effort results in lower soil strength as moisture content increases. The stability of the embankments depends on the quality of the compaction of the fill. Non-destructive testing techniques have more advantages than conventional field density tests. Therefore, the use of non-destructive testing techniques in fill monitoring seems interesting in geotechnical applications. Using the Clegg impact tester, impact (Iv) values varying in compaction effort, moisture content and density were observed in the laboratory. The variations of Iv with moisture are equal to the moisture-density ratio. The Iv has a strong relationship, for each compaction effort, with the moisture-density ratio. With a simple moisture test, the dry density can be predicted using the Iv values. This allows efficient quality control compaction.

Cacciola et al [15] perform a geo-analytical investigation. The use of frequent surveys adds both costs and delays to earthwork projects. With continuous compaction control and intelligent compaction systems, they provide a real-time monitoring. This is the Real-Time Kinematic Global Positioning System. This process can be used with great benefit by ensuring the quality of the compacted soil. In addition, Liu et al. [16] have proposed an innovative process for quality control in earth rock engineering. The compaction monitoring technology integrated into the rollers has been combined with real-time global positioning kinematics, adopting the value of compaction as real time monitoring. The compaction value has decreased with the speed of the rollers, increased with the decrease of the layer thickness and increased with the increase of the dry density. Thus, the compaction value has a relation with the quality control of the compaction. Therefore, the compaction value is similar to the compaction meter value used by the geodynamic engine drive power. Therefore, it can serve as a real-time characterization, identifying the quality control of the compaction. Regression models were used with compaction value, moisture and gradation as independent variables. Rapid and continuous evaluations of the compaction quality control prevent quality defects and improve the quality of the embankment construction, traditionally controlled through compaction thickness, vibration condition, compaction passes and roller speed. These are limited point samples to represent the construction quality of the entire work area can be unreliable, with delays in rectification of problems at the paving site.

Sawangsuriya et al. [17] comment that quality control in road compaction in Thailand is based on in-situ density measurements using the sand method. Quality monitoring is basically carried out through the sand cone test, UNE 103503 [18]. This is a simple test, although it generally requires a long testing time and is a destructive procedure. A laboratory machine provides a rapid impedance-based measurement of density and moisture in electrical spectroscopy. They investigated the density and moisture results with other tests such as nuclear methods, PBT, sand method, etc. Anjan Kumar et al. [19] have proposed an alternative method by setting target values according to soil characteristics, trying to avoid test sections. By measurement of rollers and non-destructive tests

different soils were analyzed. The in-situ tests carried out were the dynamic penetrometer, the light deflectometer and the density measurement by radioactive isotopes. With the use of the intelligent compaction they establish correlations between the values of the in-situ ensembles and the measurements of the rollers, quantifying the improvement of the material at the passing of the compacting rollers.

Nazarian et al. [20] evaluate modules as a function of moisture from the portable seismic analyzer. While all sections tested with the nuclear density meter exceeded the traditional 95% maximum dry density acceptance limit of the modified Proctor test, the modules estimated with ultrasonic surface wave technologies are higher than the moisture-dependent adjusted module. Mansour and Aly [21] have adopted the Modflow program for modelling groundwater flow conditions. Using a genetic algorithm, they achieve optimization in order to minimize the number of wells. Road construction requires high water consumption for compaction. Thus, groundwater optimization contributes to future drainage projects and can be applied in construction excavations to obtain satisfactory quality control.

2. Material and Methods

This research was done on the A-66 Spanish highway, with 21 slate random fillings for a 3000000m^3 rock digging approximately. Table 2 provides examples of the tests that were conducted on the slate alluvial material during excavation, the last row showing average values.

Table 2. Examples of physical parameters for slate alluvial material identification

Ref.	# 100 (mm)	# 20 (mm)	# 2 (mm)	#0.40 (mm)	#0.075 (mm)	LL	PL	PI	d (g/cm^3)	H (%)	CBR
CC-017	100.0	56.0	29.0	20.0	14.5	29.5	21.4	8.1	2.14	6.7	25.8
CC-014	100.0	54.0	22.0	16.0	13.3	31.8	24.1	7.6	2.14	6.8	14.0
CC-015	100.0	40.0	17.0	14.0	11.5	31.9	19.4	12.5	2.05	8.8	9.3
I-09030/04	100.0	66.0	41.0	28.0	20.6	35.0	24.3	10.7	2.06	5.3	21.1
CC-011	100.0	89.0	53.0	46.0	38.4	30.3	23.4	6.9	2.10	7.5	6.6
CC-027	100.0	72.0	47.0	35.0	28.9	28.1	21.7	6.4	2.10	10.0	25.8
Averages	100.0	64.0	35.7	26.3	21.1	31.9	22.3	9.9	2.05	8.9	15.5

The analyzed soils come from the alteration of the slates, with a high percentage of coarse fraction, 64% by the 20mm sieve and 21% fine content and low plasticity by the Atterberg limits. High values of the CBR index were obtained, around 15.

For the expected engineering behavior of the rocks, field boreholes were carried out before the excavations. The codification has attempted to unify the description of rocks with its origins in geology. The tests to identify the main geomechanical parameters done on samples of metamorphic rocks, that belong to the slate family, are shown in Table 3. The description of mass rocks has been covered on weathering, description of discontinuities and fracture state logging. Since there the boreholes are methods of acquiring discontinuity data in the field, as well as method of presenting data of this type of rocks. Some areas less important, that present big difficulty, such a description of slates mixed colors or the stratigraphic names, were omitted in the research.

Table 3. Geomechanical evaluation of the rock mass (RMR determination)

depth (m)	lithologhy	weathering grade	UCS (kp/cm2)	RQD (%)	diaclase spacing (mm)	water freatic	RMR
3.40 - 7.10	slate	IV - V	7.40	90.00	0.33	almost dry	53
7.10 - 14.60	slate	III - IV	100.90	85.00	0.33	almost dry	55
14.60 - 16.00	slate	III	194.00	90.00	0.33	almost dry	55
2.20 - 4.30	shale	III - IV	30.00	0.00	0.30	slightly wet	22
4.30 - 9.00	shale	III - IV	122.00	21.00	0.13	slightly wet	38
9.00 - 10.00	shale	III - IV	30.00	0.00	0.03	slightly wet	22
3.50 - 5.80	slate	IV - V	104.70	10.00	0.03	almost dry	34
5.80 - 7.80	slate	III - IV	104.70	50.00	0.40	almost dry	46
7.80 - 8.55	grauwacke	III	44.60	50.00	0.40	almost dry	46

where:

- UCS: Unconfined Compressive Strength [kp/cm2]

- RQD: Rock Quality Designation, quality index proposed by Deere. It is the relation of the percentages between the sum of the recovered pieces from the borehole with length higher than 10 cm and the total length drilled in the maneuver. This length depends on the compactness of the ground, in this investigation between 1.5 and 3.0 m basically.

- RMR: Rock Mass Rainting, quality index of the rock, which has been calculated on the basis of other parameters such as unconfined compressive strength (UCS), the RQD evaluated previously, the spacing, condition and orientation of the discontinuities and, lastly, the presence of water.

So, the description of rocks was done with the best tool possible, that are the unaltered samples from a borehole. The nomenclature of codification and description references to the latest codes and standards (EN ISO 14688, EN ISO 14689). Thus, 4 samples have been classified by their resistance as very weak rock, $1.0 \leq UCS \leq 5.0$, other 3 values as weak rock, with resistances between $5.0 \leq UCS \leq 12.5$ [MPa]. Finally, one sample has been classified as extremely weak, $0.6 \leq UCS \leq 1.0$ and another as moderately weak rock, $12.5 \leq UCS \leq 25.0$

The degree of meteorization has been defined according to the ENV 1997-3:1999 standard, with all samples being classified in grades III and IV. Grade III corresponds to moderately weathered rocks, in which less than half of the rock material has decomposed or disintegrated into the soil. At this grade, fresh or discolored slate rock is preserved as a rock core. Grade IV, on the other hand, is reserved for rocks of the highly weathered shale family. They belong to the rocks that have decomposed in soil into more than half of the rock material. Fresh or discolored slate rock has been observed in a discontinuous manner.

In general, they are rocks and soils from the alteration of slate, with a low - medium plasticity. According to the USCS classification, they mainly belong to the GC group, classified as coarse-grained soils wrapped in a clay matrix. There are large sizes of the mother rock, with a sifting through the 20mm sieve of only 74%, and at the same time an important percentage of fines, with an average pass through the #0.08mm sieve of 21%. As there were several degrees of weathering of the parent rock, a significant number of samples were classified within the group of high plasticity (MH) silts. With these values, the digging materials corresponding to investigated slate rock masses are valid for use in foundation fill, cores and transition zones in random fillings.

The field and laboratory works were developed in order to elaborate new test procedures for a proposed compaction control in rocks. In this research, the modified compaction control tests according to Teijón-López-Zuazo et al [22] have been used, which modify the test procedures in the wheel-tracking test and in the topographic settlements. It has been proposed as a measurement criterion the settlement between the penultimate and last pass of the compacting roller, which in case of random filing core, should be less than 5mm. The measuring system has been changed from being undefined to having levelling picks distributed in 2 rows of 5 points spaced 10 m.

The study was applied to the core random fillings with slate rocks. To facilitate interpretation, the core also includes foundations and shoulders, as shown figure 1.
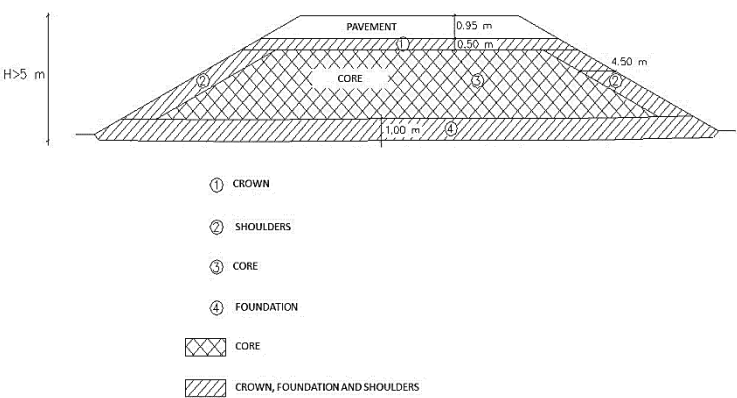
Figure. 1 Schematic diagram of random filling parts

All the tests that were used in the experiment are shown in table 3.

Table 3. Compaction tests used in the research

laboratory	2250 nuclear methods	UNE 103900 [23]
	425 modified Proctor	UNE 103501 [24]
field	75 wheel impression	UNE 103407 [25]
	75 topographic settlements	PG-3 [3]
	75 PBT	UNE 103808 [26]

The wheel tracking test is measured with a "H" dispositive, Figure 2(a). The truck should be conducted through topographic leveling pegs, as Figure 2(b).

Figure 2. (a) Measurement structure, "H" (b) Passing of the twin wheels over the peg

The results of the revised test (h) are the different measurements before and after the passing of the truck in millimeters, figure 3. The pegs reduce the possibility of extreme erroneous observations and the chances of any potential errors.

Figure 3. Topographic settlement measured after the roll pass.

The revised topographical settlement procedure was also used. The results are the settlements in millimeters between the penultimate and last roller pass. As shown in the figure 4, the first pass has easily exceeded such settlement threshold (one per cent of the thickness layer). Therefore, this control method and its limitations were thoroughly revised in the research.

The compaction degree proposed is associated with a modified Proctor compaction energy level. All the tests were performed under the same moisture conditions to prevent soil stiffness increases and noticeable dry density decreases in the plate bearing test as a result of decreases in water content to below optimum.
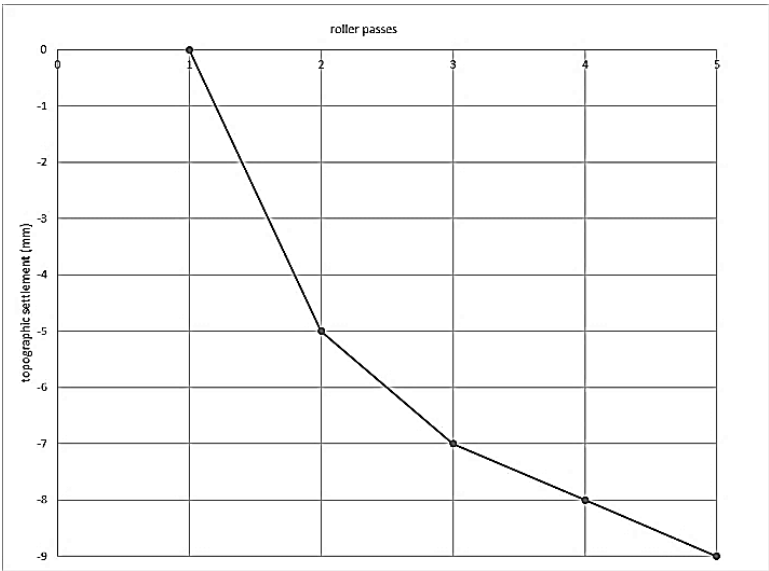
Figure 4. Topographic settlement – roller passes. 800mm section random filling.

The criteria suggested to quality control in core of random embankments were grouped in table 4.

Table 4. Specifications suggested for core random fills

Degree of compaction (%)	Settlement		Modulus		
	h (mm)	s (mm)	E_{v1} (MPa)	E_{v2} (MPa)	k (E_{v2}/E_{v1})
95.0	≤ 4.0	≤ 4.0	≥ 30.0	---	< 3.0

--- not required

In the statistical analysis, a minimum value of the determination coefficient of 0.70 was chosen to define a correlation between the variables. As a result, 2-variable linear models are better suited than multivariable models. There is no difference between dependent and independent variables. The specific schematic diagram of compaction tests relationships is shown in Figure 5.
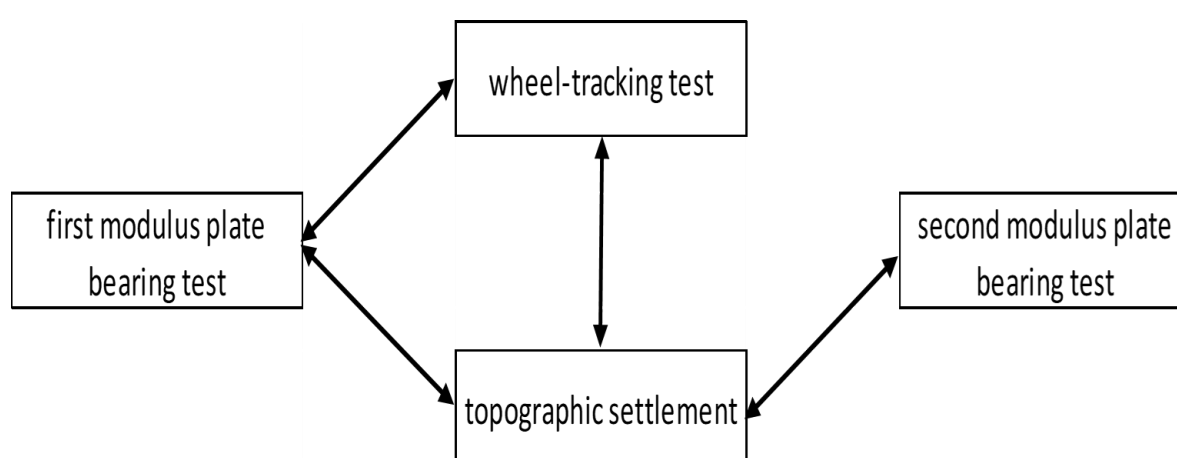
Figure 5. Schematic diagram of compaction tests relationships

3. Results

Linear correlations between 225 lots were evaluated. There was no relation between density–topographic settlement test, wheel-tracking–topographic settlement test and first–second modulus PBT (ϕ 600mm). The variables have been entered into the SPSS Statistics calculation program. An analysis of variance ANOVA shows the sums of squares and the degrees of freedom associated with each: is significant at $p < 0,05$. A multitude of non-linear models have been analyzed, although finally all the adjustments have been linear because no curve has been found that has significantly improved the adjustments.

3.1. Relation Wheel-Tracking – Topographic Settlement tests

As shown in figure 6, there is a correlation between the wheel-tracking and the topographic settlement tests. The association is directly proportional, higher values of the wheel rut corresponding to higher topographic settlements.
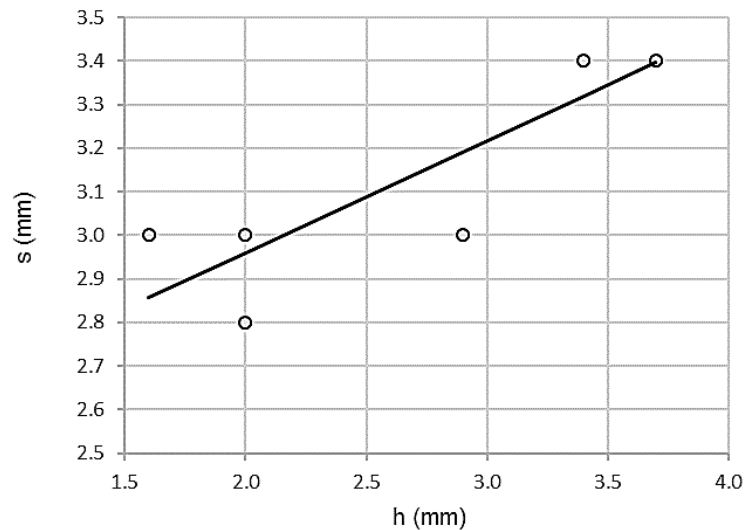
Figure 6. Scatterplot wheel-tracking – topographic settlement.

The high value of the Pearson correlation coefficient $\varrho = 0.843$ (R in table) shows a strong relationship between the wheel-tracking and the topographic settlement tests, as detailed in table 5. The coefficient of determination $R^2 = 0.710$ means a variance percentage of 71.0%. The standard error is 0.1475mm.

Table 5. Determination coefficients of wheel-tracking and topographic settlement tests.

Summary model			
R	R^2	R^2 fit	standard error
0.843[a]	0.710	0.637	0.1475

[a] Predictors: constant, h (mm)

As shown in table 6, Levene test is significant with a value of $F = 9.786$ (Hartley´s F). Consequently, the homoscedasticity criterion is not met. The variances are different. The variables, therefore, are related.

Table 6. Variance analysis wheel impression and topographic settlement tests.

ANOVA [a]					
model	sum of squares	degrees of freedom	quadratic average	F	sig.
regression	0.213	1	0.213	9.786	0.035 [b]
sampling error	0.087	4	0.022		
total	0.3	5			

[a] dependent variable: s (mm)   [b] predictors: (constant), h (mm)

Table 7 shows high t-values (Student's t test) of 11.237 and 3.128, both significant. The wheel-tracking test permits an accurate prediction of the topographic settlement values, which allows for the substitution of the compaction control procedure and vice versa.

Table 7. Linear regression coefficients in wheel-tracking and topographic settlement tests.

model	nonstandard coefficients		standard coefficients	t	sig,
	B	standard error	beta		
(constant)	2.446	0.218		11.237	0.000
h (mm)	0.257	0.082	0.843	3.128	0.035

[a] dependent variable: s (mm)

According to the coefficients, the linear fit equation for the topographic settlement and the wheel-tracking tests is:

$$s = 2.446 + 0.257\,h \qquad R^2 = 0.710 \qquad (1)$$

The function domain uses the intervals of $[2.5 \le s \le 3.5]$ and $[1.5 \le h \le 4.0]$.

3.2. Relation Wheel-Tracking Test – First PBT Modulus

As shown in figure 7, there is a high correlation between the wheel-tracking test and the first PBT modulus (ϕ 600mm), with inverse proportionality. In this case, high values for the wheel-tracking test correspond to low values for the first PBT modulus.
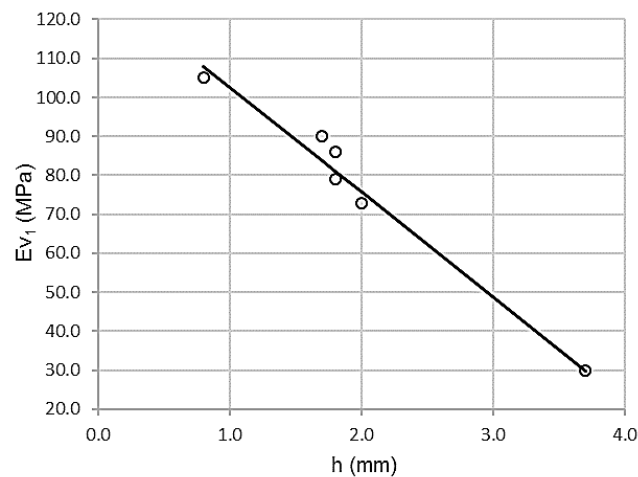
Figure 7. Scatterplot for wheel-tracking test and first modulus PBT (ϕ 600mm).

Table 8 shows a high Pearson correlation coefficient value, $\varrho = 0.990$, which is associated with low dispersion. The coefficient of determination $R^2 = 0.980$ yields a variance of 98%. The standard error is only 4.1934 MPa.

Table 8. Determination coefficients for wheel-tracking test and first PBT modulus

Summary model

R	R^2	R^2 adjusted	standard error
0.990 [a]	0.980	0.975	4.1934

[a] Predictors: constant, h (mm)

Table 9 shows the ANOVA results. Levene's test proved significant sig = 0.000 with a value of F = 199.826. Therefore, the null hypothesis of homoscedasticity is rejected and variances are significantly different.

Table 9. Variance analysis wheel impression test and first PBT modulus

ANOVA [a]

model	sum of squares	degrees of freedom	quadratic average	F	sig.
regression	3513.855	1	3513.855	199.826	0.000 [b]
sampling error	70.338	4	17.585		
total	3584.1932	5			

[a] dependent variable: Ev_1 (MPa)   [b] predictors: (constant), h (mm)

The t-test in table 10 offers high values, 32.576 and -14.136, both significant (sig = 0).

Table 10. Linear regression coefficients wheel-tracking test - first PBT modulus

Coefficients [a]

| model	nonstandard coefficients		standard coefficients	t	sig.
	B	standard error	beta		
(constant)	129.468	3.974		32.576	0.000
h (mm)	-26.291	1.904	-0.990	-14.136	0.000

[a] dependent variable: Ev_1 (MPa)

Moreover, the wheel impression test predicts the first vertical modulus of the plate bearing test. Besides the linear regression coefficients, the fit between the wheel impression test and the first modulus of the PBT (ϕ 600mm) is:

$$Ev_1 = 129.468 - 26.921 \ h \quad R^2 = 0.980 \tag{2}$$

The domain of the function between the intervals is [20 ≤ Ev_1 ≤ 110] and [0.5 ≤ h ≤ 4.5].

3.3. Relation Topographic Settlement Test – First PBT Modulus

As shown in figure 8, there is a strong correlation between the topographic settlement test and the first modulus plate bearing test (ϕ 600mm).
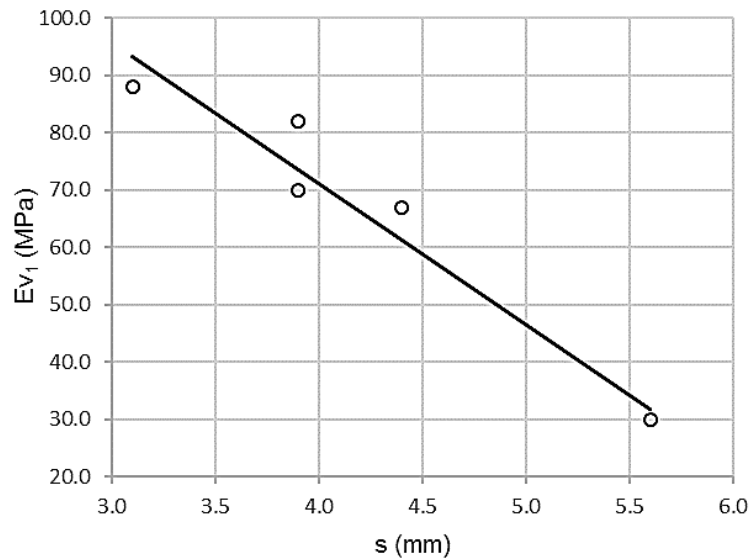
Figure 8. Scatterplot for topographic settlement test - first PBT modulus (ϕ 600mm).

Table 11 shows a high Pearson correlation coefficient value, $\varrho = 0.962$ and a low standard error of Se = 7.1343 MPa. The coefficient of determination validates a variance of 92.5%. All the parameters suggest a high correlation between both variables.

Table 11. Determination coefficients topographic settlement test - first PBT modulus

Summary model			
R	R^2	R^2 adjusted	standard error
0.962 [a]	0.925	0.9	7.1343

[a] Predictors: constant, s (mm)

ANOVA parameters are in table 12. Levene's test is significant, F = 36.847, sig = 0.009. The homoscedasticity criterion is not clearly met. Since the variables are strongly related, variances are significantly different.

Table 12. Analysis of variance for topographic settlement and first PBT modulus

ANOVA [a]					
Model	sum of squares	degrees of freedom	quadratic average	F	sig.
regression	1875.433	1	1875.433	36.847	0.009 [b]
sampling error	152.695	3	50.898		
Total	2028.128	4			

[a] dependent variable: Ev_1 (mm)   [b] predictors: (constant), s (mm)

Table 13 shows high t-values of 9.884 and -6.070, which are both significant. The topographic settlement test predicts the first PBT modulus (ϕ 600mm).

Table 13. Linear regression coefficients for topographic settlement and first PBT modulus

model	Coefficients [a]				
	nonstandard coefficients		standard coefficients	t	sig,
	B	standard error	beta		
(constant)	169.243	17.124		9.884	0.002
h (mm)	-24.549	4.044	-0.962	-6.070	0.009

[a] dependent variable: E_{v1} (MPa)

According to the linear regression coefficients, the adjustment line is:

$$E_{v1} = 169.243 - 24.549\ s \quad R^2 = 0.925 \tag{3}$$

The domain of the function uses the $[20 \leq E_{v1} \leq 100]$ and $[3.0 \leq s \leq 6.0]$ intervals.

3.4. Relation Topographic Settlement Test – Second PBT Modulus

As shown in figure 9, there is a high correlation between topographic settlement and first PBT modulus. The distribution is inversely proportional to the lower settlement values corresponding to the higher values of the second PBT modulus (ϕ 600mm).
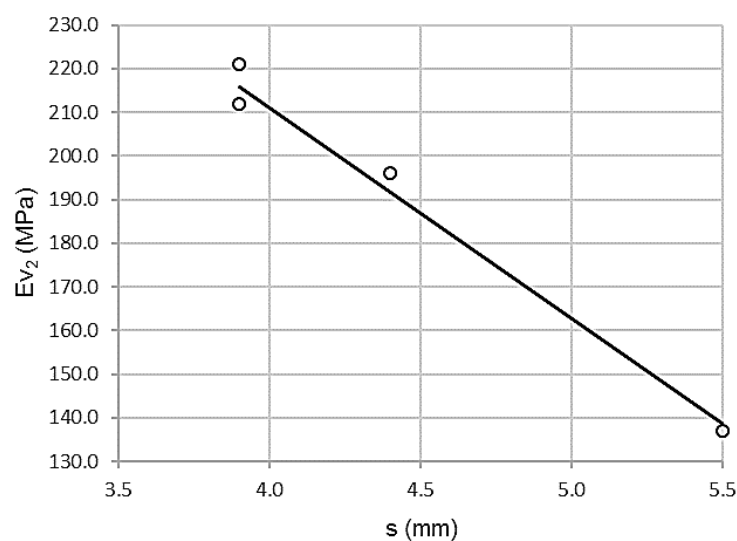

Figure 9. Scatterplot for topographic settlement test and second PBT modulus.

Table 14 illustrates a high Pearson correlation coefficient, $\varrho = 0.995$. There is a low standard error $S_e = 4.5260$ MPa and a high coefficient of determination $R^2 = 0.990$. There is low dispersion.

Table 14. Determination coefficients for topographic settlement test and second PBT modulus

Summary model

R	R^2	R^2 adjusted	standard error
0.995 [a]	0.990	0.985	4.5260

[a] Predictors: constant, s (mm)

The ANOVA analysis parameters are shown in table 15. Levene's test is significant, sig = 0.005 with F = 19.251, therefore the assumption of homoscedasticity criterion is not met, since variances are different and have a dependency relationship.

Table 15. Variance analysis for topographic settlement test and second PBT modulus

ANOVA [a]

model	sum of squares	degrees of freedom	quadratic average	F	sig.
regression	3951.860	1	3951.860	19.251	0.005 [b]
sampling error	40.970	2	20.845		
total	3992.830	3			

[a] dependent variable: Ev_2 (mm)   [b] predictors: (constant), s (mm)

Student's t test values are significant. As shown in table 16, there is a significant contribution of the topographic settlement in the second modulus plate bearing test (ϕ 600mm).

Table 16. Linear regression coefficients for topographic settlement and second PBT modulus

Coefficients [a]

Model	nonstandard coefficients		standard coefficients	t	sig.
	B	standard error	beta		
(constant)	403.329	15.493		26.420	0.001
s (mm)	-48.108	3.464	-0.995	-13.889	0.005

[a] dependent variable: Ev_2 (MPa)

The expression of the adjustment line is:

$$Ev_2 = 403.329 - 48.108 \, s \qquad R^2 = 0.985 \qquad\qquad (4)$$

The domain of the function has values between [$140 \le Ev_2 \le 240$] and [$3.5 \le s \le 6.0$].

3.5. Significance Matrix

For better understanding, a matrix of significance is shown in table 17 with the results obtained. If no relationship has been obtained, the numerical value is replaced by ns (non-significant). Some elements of the matrix are not considered because they are easily deduced.

Table 17. Slate core random fill significance matrix

	Determination coefficients (R^2)					
	d (g/cm³)	h (mm)	s (mm)	Ev_1 (MPa)	Ev_2 (MPa)	k (Ev_2 / Ev_1)
d (g/cm³)	---					
h (mm)	ns	---				
s (mm)	ns	0.710	---			
Ev_1 (MPa)	ns	0.874	0.925	---		
Ev_2 (MPa)	(*)	ns	0.990	ns	---	
k (Ev_2 / Ev_1)	(*)	ns	(*)	(*)	Ns	---

ns: nonsignificant   (*) obvious relationships

The values of the Student t test were grouped in Table 18.

Table 18. Slate core random fill significance matrix

	Student t test (t)					
	d (g/cm³)	h (mm)	s (mm)	Ev_1 (MPa)	Ev_2 (MPa)	k
d (g/cm³)	---					
h (mm)	ns	---				
s (mm)	ns	3.128	---			
Ev_1 (MPa)	ns	-14.136	-6.070	---		
Ev_2 (MPa)	(*)	ns	-13.890	ns	---	
K	(*)	ns	(*)	(*)	ns	---

ns: nonsignificant (*) obvious relationships

The *in-situ* density did not correlate with any other variable. Alternatively, with the first PBT modulus (ϕ 600 mm), the wheel-tracking and topographic settlement tests proved to have a strong relationship. A revised control method has been designed for the *in-situ* density test and the PBT.

There is a strong correlation between the revised topographic settlement test and the plate bearing test (ϕ 600mm) so the PBT can be easily replaced. With significant improvements in both the topographic settlement test and the wheel impression test, the PBT is associated with both, so the PBT (ϕ 600mm) can replace these tests in quality control.

The nuclear methods have a low efficiency, limited by a maximum test thickness of 300mm and the high variability of the materials. Therefore, the PBT (ϕ 600mm) is proposed as the most representative test to define the degree of compaction in the new control method. As this test is strongly associated with surface moisture, it should be carried out in the same area of validity as the optimum moisture obtained in the modified Proctor.

4. Conclusions

The maximum size of the random fill particles conditions the effectiveness of compaction tests such as in-situ density, modified Proctor, PBT, topographic settlements and wheel tracking test. The new procedure revises the wheel tracking test and the topographical settlement test, optimizing the results. Finally, statistical analysis allows simplification of the quality control procedure for core slate random fillings, with a maximum layer thickness of 800mm. An optimization of the compaction control system has been achieved in random fillings core, obtaining a reduction in the control time.

The contributions of the research are:

- The nuclear methods have a low efficiency, limited by a maximum test thickness of 30 cm and the high variability of the materials.
- The PBT (ϕ 300mm) provides unreliable results in the core of random fillings, with maximum sizes up to 500mm. This research has demonstrated optimal control using PBT (ϕ 600mm).
- The PBT (ϕ 600mm) is proposed as the most representative test to define the degree of compaction in the new control method on core of slate random fillings. As this test is strongly associated with surface moisture, it should be carried out in the same area of validity as the optimum moisture obtained in the modified Proctor.
- New procedures for topographical settlement control and wheel impression tests have been applied with optimal results to the core of random fillings formed by slates with maximum layer thicknesses of 800mm.
- Statistical correlations have been found between different compaction tests, which has made it possible to eliminate redundant tests, thus optimizing quality control and construction procedures.
- The wheel tracking test can be deduced from the adjustment model for values between $1.5 \leq h \leq 4$ millimeters. The limitations of the nuclear methods made it impossible to relate to other tests. Finally, the topographic seat control can be replaced for values of the PBT modules between $20 \leq Ev_1 \leq 100$ and $140 \leq Ev_2 \leq 240$.
- In the core of random fillings, including foundations and shoulders, which are formed by slates laid in layers with a maximum thickness of 800mm, it is proposed as a quality control of the compaction to carry out PBT tests (ϕ 600mm) and the in situ determination of density and moisture content by nuclear methods.

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