PERFORMANCE OF LOW-VOLUME ROAD TEST SECTIONS WITH WEARING COURSE OF SILTY CLAYEY SOIL MODIFIED WITH LIME IN URUGUAY

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ABSTRACT

A research project into the structural behaviour of soil-lime as wearing course in two unsealed low-volume roads is presented. Full-scale pavement test sections with a wearing course of local silty clay soil treated with lime were constructed in 2008 and 2011 in Cebollatí, in eastern Uruguay. Mixtures of this soil with different lime contents were characterized in the laboratory through unconfined compression strength, splitting tensile strength, and resilient modulus by cyclic triaxial compression tests. The materials were classified as lime modified soil, according to the Austroads Guide to Pavement Technology. Test sections with soil modified with 3%, 4%, and 5% of lime have been monitored by means of visual observations and deflection measured by Benkelman Beam. Up to now, and despite being trafficked by heavy trucks immediately after they were constructed, rutting and cracking have not been observed on top of the soil-lime layers. The soil modified with lime has shown an acceptable performance as a wearing course for a low-volume road, which stimulates its use as a technical alternative in regions where granular materials of high quality are not available, also reducing construction and maintenance costs, and preserving non-renewable materials such as rocks and soils.

INTRODUCTION

Modification and stabilisation of soils and materials using lime is a well-established method to construct pavement layers. The addition of lime to reactive fine-grained soils improves their engineering properties, such as reduction in plasticity and swell potential, improved workability, increased strength and stiffness, and enhanced durability. In addition, lime has been used to improve the strength and stiffness properties of unbound base and sub-base materials (Mallela et al. 2004). In this way, it is possible to improve the pavement performance, increase service life and thereby reduce costs. Good quality materials for pavements are finite and increasingly scarce resources and the exploitation of new quarries is more and more limited by environmental restrictions. The planning of the rational use of such resources is an environmentally correct practice, being the modification and stabilisation of soils one of them.

The development of mechanistic-empirical design methods has motivated the need to obtain the elastic or resilient parameters of soils and materials used in pavements, as well as the characterization of its fatigue life. However, only a few researches have been performed to evaluate the behaviour of cemented materials and to determine the inputs required by such design methods (Foley 2001) and even less for soils modified or stabilised with lime. The formulation of models of resilient response of such materials is not yet concluded (Theyse et al. 2007). The laboratory characterization of cemented materials for pavements for low-volume roads has been recently completed by the Austroads (Gonzalez et al. 2010), concluding that it is necessary to do a substantial revision in the thickness design method for such materials. Although essential, the laboratory characterization of pavement materials presents some limitations. The scale effect is one of these and to avoid this problem, full-scale pavement test sections and accelerated pavement testing were undertaken.

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The concept of modification and stabilisation of soils varies from country to country and has changed over time. According to Little (1999), modification occurs primarily due to cationic exchange and flocculation, resulting in plasticity reduction, reduction in moisture holding capacity (drying), swell reduction, improved stability and the ability to construct a solid working platform. Stabilisation takes place when the proper amount of lime is added to a reactive soil to develop a significant level of long-term strength gain through a long-term pozzolanic reaction. The modification process can produce very important structural improvements such as significant bearing capacity. Based on these concepts of modification and stabilisation of soils with lime, Thompson (1966) defined the optimum lime content to stabilise a soil as one that produces unconfined compression strength (UCS) of 345 kPa after 48 h at a temperature of 49.5°C. Seddom and Bhindi (1983) established the boundary between stabilised and modified phases as corresponding to a splitting tensile strength (STS) of 120 kPa. The Australasian Association of Road Transport and Traffic Agencies (Austroads) divides lime treated materials as modified and stabilised according to their fundamental behaviour under applied loadings (Jameson 2013). Modified materials are used and evaluated in the same manner as conventional unbound flexible pavement while stabilised materials have a sufficiently enhanced elastic modulus and tensile strength to have a practical application in stiffening the pavement.

Thompson (1970) defined a lime-soil mixture as acceptable for a structural base if the UCS exceeded about 1050 kPa, while some transport agencies of United States require minimal UCS values for sub-base and base layers between 700 kPa and 1400 kPa (Little 1999). The Austroads Guide to Pavement Technology of 2006 suggests 28-day UCS values between 0.5 MPa and 1.5 MPa for lime modification of pavement layers and UCS greater than 1.5 MPa for lime stabilisation (Jameson 2013). A maximum design modulus of 1000 MPa is normally adopted for modified materials. For lime stabilisation, the use of pH testing together with 28-day UCS testing is recommended to establish the optimum lime content of a material, however, no mention is made of the strength to produce a lime modified material. The resilient behaviour of materials treated with low cement contents is similar to unbound materials which depend on the density, saturation degree and stress state (Theyse et al. 2007).

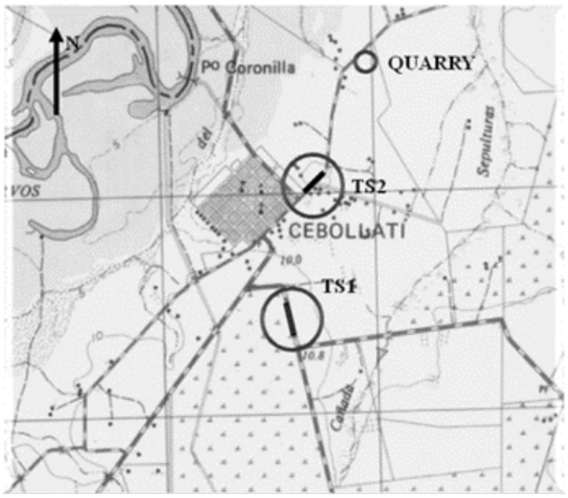

Figure 1: Locations of full-scale test sections and collection of soil.

A research into the behaviour of low-volume roads pavements with a wearing course of soil modified with lime was done through constructing and monitoring two full-scale pavement test sections in Cebollatí in eastern Uruguay, and performing laboratory mechanistic tests. The locations of test sections are shown in Figure 1. Test Section 1 (TS1) was constructed in April 2008 (Behak 2011) and consists of two sectors, each one of 50 m in length, with a 15 cm-thick wearing course of silty clayey soil treated with 3% and 5% of lime respectively. Test Section 2 (TS2), 100 m in length, was constructed in March 2011 with a 15 cm-thick wearing course of silty clayey soil treated with 4% of lime. All wearing courses were constructed over the current pavement, which is a base layer of coarse material of 10 cm thick over a subgrade of silty clayey soil. A part of the laboratory testing program conducted on mixtures of Cebollatí's silty clayey soil with lime and the tests results, including field data from monitoring of test sections, follows.

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METHODOLOGY

Materials

The soil used for the laboratory and field studies was collected from a site near Cebollatí as can be seen in Figure 1. It is located 3.5 km and 1.7 km from the first test section (TS1) and second test section (TS2) respectively. The selected soil was characterized by grain-size analysis test and Atterberg limits, and the results are shown in Table 1. According to the Unified Soil Classification System (USCS) the soil is a clay of high compressibility with greater content of silt than clay in the fine fraction (less than 0.075 mm) and a relatively high content of fine sand. Montmorillonite, illite and kaolinite are the clayey minerals which predominate.

Table 1: Physical and compaction properties of soil

Property	Value
Passing 0.075 mm (%)	68
Sand (%)	32
Silt (%)	46
Clay (%)	22
Liquid Limit	51
Plastic Limit	19
Plasticity Index	32
USCS	CH
Specific Gravity	2.75

A commercial lime was used, consisting of 66% calcium oxide, 5% magnesium oxide and traces of silica and ferric oxide. The lime is fine with 100% passing the 2.0 mm sieve and 93% the 0.075 mm sieve. The silt fraction constituted 91% of the total dry mass.

Laboratory tests

UCS tests were conducted on mixtures of soil with 3%, 4%, and 5% of lime (S3L, S4L and S5L, respectively) at 7, 14, 28, 56, 90, and 181 days, and in the natural soil as control, according to AASHTO test method T208. Specimens 3.76 cm in diameter and 7.66 cm in height were compacted at corresponding 100% maximum dry density (MDD) and optimum compaction moisture (OCM) of the modified Proctor test, into a mini-Harvard mould by kneading system immediately after the manual mixing of soil, lime and water. The mixtures for each lime content and curing time were compacted in triplicate. After compaction, the specimens of mixtures were wrapped in PVC film and they were stored in a moisture chamber at room temperature.

STS tests were carried out on mixtures of soil with 3%, 4%, and 5% of lime (S3L, S4L and S5L, respectively) at 7, 14, 28, 56, 90, and 181 days, and in the natural soil as control, in accordance with ASTM test method D6931. Specimens of internal diameter of 10.17 cm and height of 6.19 cm were compacted at corresponding MDD and OCM of the modified Proctor test, with modified Proctor effort immediately after the manual mixing of soil, lime and water. The mixtures for each lime content and curing time were compacted in triplicate. After compaction, the specimens of mixtures were wrapped in PVC film and they were stored in a moisture chamber at room temperature.

Cyclic triaxial compression tests were performed to determine the resilient modulus on cylindrical specimens of 10.1 cm diameter and 20.2 cm height, following the AASHTO TP 46 procedure. The natural soil and mixtures of soil with 3%, 4%, and 5% were tested at 28 days and 90 days. In the research was added a percentage of 7% of lime for a more wide study of

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the deformation behaviour under traffic loads of the studied material. The effect of compaction effort on the resilient modulus was also researched considering the standard and modified Proctor energies. All specimens were compacted in triplicate at OCM of the modified Proctor test and immediately after the manual mixing of soil, lime and water. After this, mixture specimens were wrapped in PVC film and after that they were stored in a moisture chamber at room temperature.

Field study

The performance under traffic loads of tested pavements was monitored with deflection measurements by Benkelman beam tests according to AASHTO T256. Two monitoring points were established in TS1: one of them in the sector with wearing course of soil with 3% of lime (TS1-S3L) and another one in the sector with wearing course of soil with 5% of lime (TS1-S5L). A third point was located, as control point, in the traditional pavement (without soil-lime layer) adjacent to TS1 (CP1). The monitoring was conducted immediately after the construction of TS1 (April 17, 2008) and 131 days (August 26, 2008), 1481 days (May 11, 2012) and 1693 days (December 5, 2012) after construction. Four monitoring points were defined in TS2 (P1, P2, P3 and P4), spaced 25 m each, and a control point was located in the traditional pavement adjacent to TS2 (CP2). Monitoring was carried out 168 days (August 18, 2011), 442 days (May 11, 2012), and 650 days (December 5, 2012) after constructing the test section.

RESULTS AND DISCUSSION

Laboratory study

Unconfined compression strength

The UCS values obtained from all tested specimens of the soil and mixtures of soil with 3%, 4% and 5% of lime (S3L, S4L, and S5L) are summarized in the Table 2 and Table 3. In this last table, the mean values, standard deviation (Std. Dev.) and coefficient of variation (cv) correspond to all mixtures for each time.

Table 2: Unconfined compression strength and splitting tensile strength values of soil

UCS (MPa)	Mean Value (MPa)	Std. Dev.	cv	STS (kPa)	Mean Value (kPa)	Std. Dev.	cv
0.20	0.21	0.02	0.09	40.3	39.0	5.0	0.13
0.20				43.2			
0.24				33.5			

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Table 3: Unconfined compression strength and splitting tensile strength values of soil-lime mixtures

Time (days)	UCS (MPa)						STS (kPa)					
	S3L	S4L	S5L	Mean Value	Std. Dev.	cv	S3L	S4L	S5L	Mean Value	Std. Dev.	cv
7	0,45	0,43	0,43	0,43	0,02	0,04	152,3	149,3	144,1	144,7	7,3	0,05
	0,42	0,42	0,44				131,7	146,8	152,3			
	0,43	0,42	0,47				145,0	146,8	134,1			
14	0,38	0,52	0,49	0,51	0,05	0,11	149,3	200,8	205,1	184,4	31,7	0,17
	0,49	0,55	0,54				145,0	199,3	203,6			
	0,50	0,55	0,52				134,4	205,7	216,3			
28	0,87	0,82	0,77	0,81	0,06	0,07	180,8	202,4	212,1	201,1	10,0	0,05
	0,89	0,87	0,79				198,0	199,6	208,7			
	0,74	0,80	0,74				196,0	213,6	198,4			
56	1,07	1,34	1,07	1,13	0,09	0,08	250,0	266,1	265,2	261,9	9,1	0,03
	1,05	1,22	1,13				253,0	270,9	262,7			
	1,08	1,12	1,09				255,0	256,4	277,9			
90	1,41	1,52	1,61	1,54	0,09	0,06	311,3	353,1	358,0	341,8	20,8	0,06
	1,45	1,58	1,57				306,4	350,4	353,0			
	1,68	1,43	1,62				335,5	367,4	341,0			
181	1,62	1,71	1,76	1,71	0,12	0,07	378,9	395,6	370,0	388,8	11,7	0,03
	1,72	1,58	1,94				387,7	402,0	395,9			
	1,60	1,62	1,86				396,8	398,0	374,0			

The evolution of the UCS of soil with lime content over time is depicted in Figure 2. An increase in UCS for all mixtures was observed over time, although the values are similar for the different lime contents at a given time. Therefore, it is possible to consider the UCS as a function of time and independent of the lime content when this varies between 3% and 5%. The UCS at 7 days increased from 0.21 MPa for the soil to an average value of the three treated materials of 0.43 MPa; this is a doubling in value. The 28-day UCS of the treated soil materials reaches an average value of 0.8 MPa, which means a gain about 4 times the value of the soil. With 90 and 181 days, the UCS average values of the materials are of 1.54 MPa and 1.71 MPa respectively, which is equivalent to a gain of 7 and 8 times of UCS compared to the soil without treatment.

The 28-day UCS value is of 0.9 MPa when it is considered a fit curve of UCS as function of time for the three lime percentages tested. This value is within the range suggested by the Austroads Guide to Pavement Technology of 2006 (Austroads 2006) to classify the studied soil-lime material as modified, with the expectation that it would behave as an unbound material and the rutting would be the most likely failure mechanism to the wearing course. However, the fact that Cebollatí's soil-lime material would be considered as modified does not mean that it is not susceptible to fatigue cracking (Jameson 2013).

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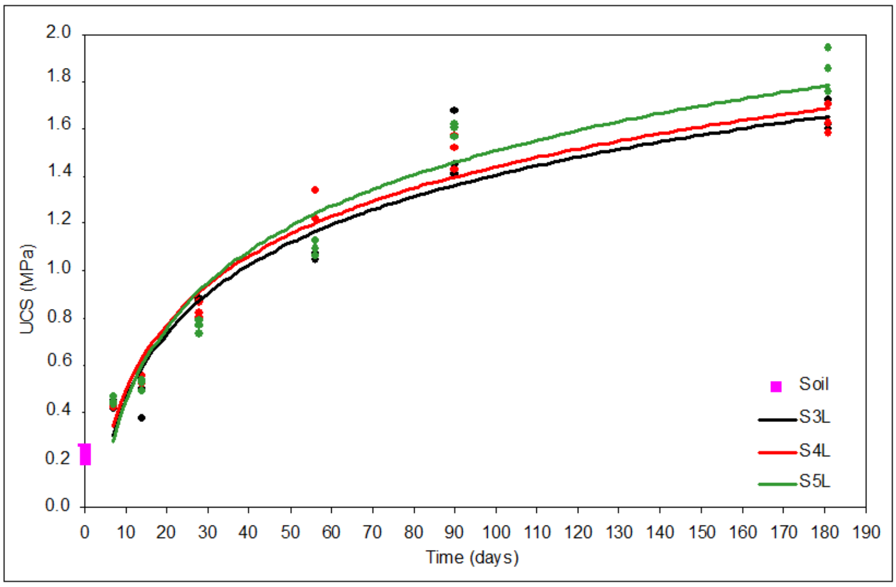

Figure 2: Unconfined compression strength as function of the lime content and the time.

Splitting tensile strength

The STS values obtained from all tested specimens of the soil and mixtures of soil with 3%, 4% and 5% of lime (S3L, S4L, and S5L) are summarized in the Table 2 and Table 3. In this last table, the mean values, standard deviation (Std. Dev.) and coefficient of variation (cv) correspond to all mixtures for each time. The evolution of the STS with the lime treatment over time is shown in Figure 3. STS increases with time for all mixtures but the values are similar for the different lime percentages at any given time. The STS of Cebollatí's soil-lime material is a function dependent of time and independent of the percentage of lime content when this varies between 3% and 5%. The development of tensile strength is indicative of formation of pozzolanic products that cement the soil due to the alkaline reactions between the silica of soil clayey mineral and calcium ions of lime. Although the soils theoretically do not have tensile strength, it is observed from Figure 3 that the tested soil presents an STS of 40 kPa because of the effect of the water suction into the unsaturated pores and a certain real cohesion. An increase of 3.7 times in the STS with respect to soil was found at 7 days, with an average value for the three treated materials of 145 kPa. The 28-day STS average value is 200 kPa, which means a gain about 5 times the value for the soil. The 90-day and 181-day STS average values of the material are of 340 kPa and 390 kPa respectively, which is 9 and 10 times greater than the STS of soil.

If the STS boundary value of 120 kPa (Seddom and Bhindi 1983) is considered to differentiate the stabilised and modified phases and the 28-day STS value of Cebollatí's soil-lime material, the tested material must be classified as stabilised. The same conclusion is drawn when taking into account the 7-day indirect tension value of 80 kPa suggested by the 1986 NAASRA Guide to Stabilization for demarcation between modified and bound material (Jameson 2013). However, it is to be noted that this conclusion is inconsistent with that obtained by the UCS tests.

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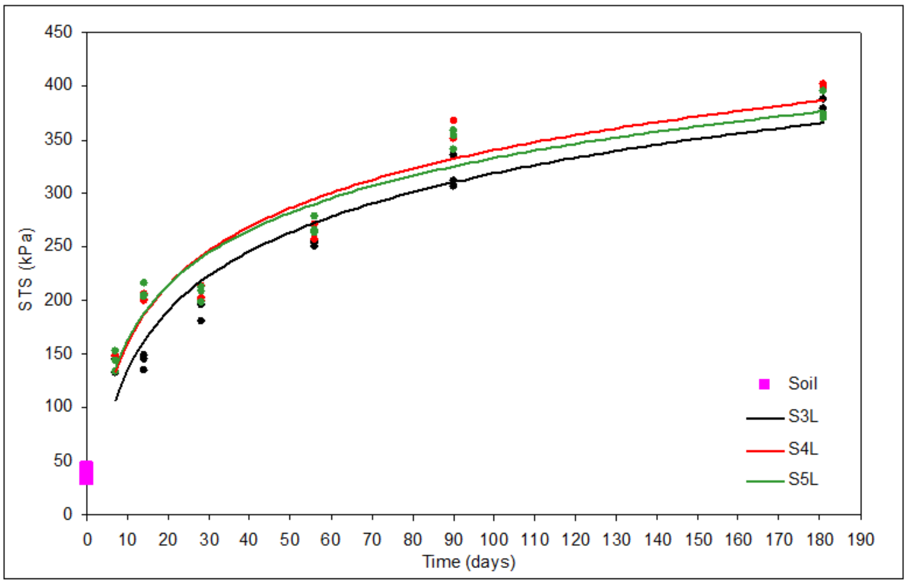

Figure 3: Splitting tensile strength as function of the lime content and the time.

Resilient modulus

The mean values of confining pressure (σ_3), deviatoric stress normalized by the atmospheric pressure (σ_d/p_a) and resilient modulus (Mr) measured for each sequence of the cyclic triaxial compression tests of the soil and mixtures with 3%, 4%, 5% and 7% of lime (S3L, S4L, S5L, and S7L) are summarized in the Table 4 and Table 5. The standard deviation (Std. Dev.) and coefficient of variation (cv) are referred to the Mr.

Table 4: Resilient modulus values of cyclic triaxial compression tests of the soil

Standard Energy					Modified Energy				
σ_3 (kPa)	σ_d/p_a	Mr (MPa)	Std. Dev.	cv	σ_3 (kPa)	σ_d/p_a	Mr (MPa)	Std. Dev.	cv
54.8	0.41	279	157	0.56	54.9	0.39	369	117	0.32
40.9	0.43	291	170	0.58	41.0	0.41	367	133	0.36
27.0	0.42	296	170	0.57	27.1	0.42	387	139	0.36
16.7	0.43	297	169	0.57	13.9	0.42	413	135	0.33
54.8	0.54	149	79	0.53	54.9	0.54	210	65	0.31
40.9	0.54	144	69	0.48	41.0	0.53	215	81	0.38
27.0	0.55	162	86	0.53	27.1	0.53	233	104	0.45
17.2	0.55	168	88	0.52	14.0	0.54	246	107	0.44
54.8	0.74	73	29	0.39	54.9	0.72	138	32	0.24
40.9	0.74	76	29	0.39	41.0	0.73	143	35	0.24
27.0	0.75	83	36	0.43	27.1	0.73	148	42	0.28
17.7	0.75	86	39	0.45	14.2	0.73	147	41	0.28
54.8	1.03	47	7	0.15	54.9	1.04	87	12	0.14
40.9	1.03	46	6	0.12	41.0	1.04	87	12	0.14
26.9	1.04	47	7	0.14	27.1	1.06	87	12	0.13
18.1	1.04	48	7	0.15	14.3	1.06	87	12	0.13

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Table 5: Resilient modulus values of cyclic triaxial compression tests of the soil-lime mixtures

Standard Energy					Modified Energy				
σ_3 (kPa)	σ_d/p_a	Mr (MPa)	Std. Dev.	cv	σ_3 (kPa)	σ_d/p_a	Mr (MPa)	Std. Dev.	cv
54.7	0.36	313	105	0.34	54.7	0.41	361	101	0.28
40.9	0.37	320	101	0.32	40.8	0.41	367	105	0.29
27.3	0.37	330	104	0.31	26.9	0.42	383	109	0.29
18.4	0.37	334	101	0.30	17.2	0.42	393	112	0.29
54.7	0.53	175	45	0.26	54.8	0.56	255	72	0.28
40.9	0.53	184	44	0.24	40.9	0.56	266	70	0.26
27.5	0.54	193	47	0.24	27.0	0.57	281	75	0.26
19.0	0.53	198	46	0.23	17.8	0.57	291	77	0.26
54.8	0.72	133	30	0.22	54.8	0.75	213	60	0.28
40.9	0.71	143	30	0.21	39.3	0.75	227	63	0.28
27.7	0.72	151	32	0.21	26.7	0.76	240	66	0.27
19.6	0.72	155	33	0.21	18.2	0.76	248	67	0.27
54.7	0.99	115	27	0.24	54.7	1.04	197	57	0.29
40.9	1.00	120	29	0.24	40.9	1.05	207	61	0.29
27.8	1.00	124	30	0.24	27.0	1.05	217	63	0.29
20.0	1.00	127	30	0.24	18.6	1.05	222	66	0.30

The behaviour of the Mr of the soil for both compaction energies after a statistical analysis of the dates resulting of the cyclic triaxial tests is shown in Figure 4. A model of variation of the resilient modulus (Mr) as function of the normalized deviatoric stress (σ_d/p_a) was found for both compaction energies. The stiffness exponentially decreases with the increase of vertical stress, with fitting curves as are presented in Figure 4. The observed resilient behaviour is the normally found for fine soils by several researchers and was the expected for Cebollatí's soil. The resilient coefficients of soil (k_1 and k_2), defined for both compaction energies, are depicted in Table 2. The Mr of soil compacted at modified energy is greater than soil compacted at standard energy for all deviatoric stress values. The coefficient k_2 of the soil compacted at standard energy is greater than soil compacted at modified energy, which means that the sensitivity of stiffness decrease with the stress increase is less when the compaction effort increases.

The results of the statistical analysis for all soil-lime materials tested (3%, 4%, 5%, and 7% of lime) for the two curing times (28 days and 90 days) are also depicted in Figure 4. Fitting curves for each compaction effort were found with a high coefficient of determination (R^2), which means that the resilient behaviour of Cebollatí's silty clayey soil treated with lime can be treated as independent of the lime content in the studied range and of the curing time between 28 days and 90 days. The low variation of the Mr with the lime content is consistent with the results determined in the UCS and STS tests but it differs with those found for curing times. In fact, the Mr increased between 28 days and 90 days but this was not significant when compared with the increase of UCS and STS values found in the same period.

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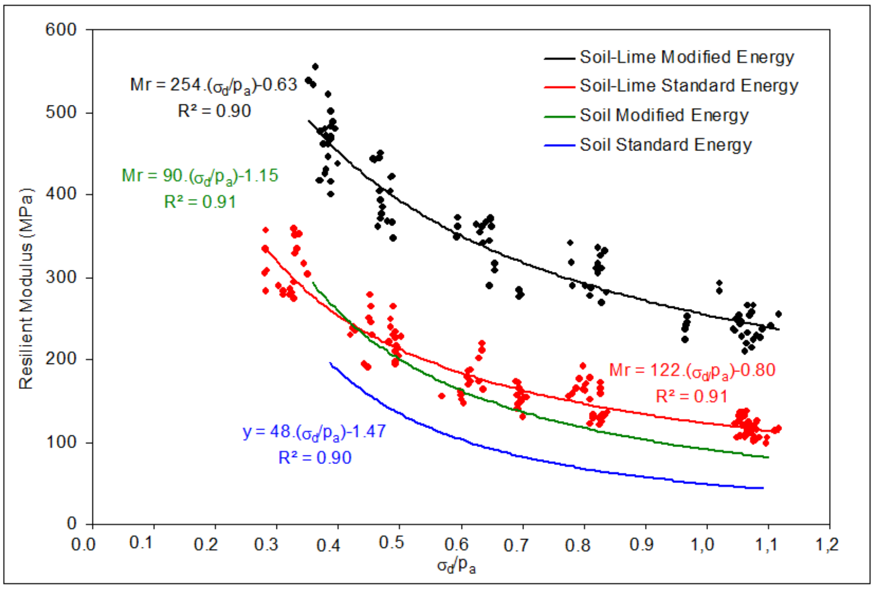

Figure 4: Resilient modulus of soil and soil-lime mixtures as function of normalized deviatoric stress and compaction energy.

The fitting curves defined for all lime contents and two curing times show an exponential decrease of the Mr values with the increase of normalized σ_d for both compaction energies. This resilient behaviour is similar to that established for the natural soil which indicates that this material behaves as unbound. This observation verifies the obtained through the UCS tests and, thereby, the studied material can be classified as modified. Nevertheless, the Mr values measured in mixtures compacted at modified energy (210 MPa - 550 MPa) are significantly less than that recommended by the Austroads Guide to Stabilization on Roadworks 1988 for modified and cemented materials (700 MPa – 1500 MPa) (Jameson 2013). The stiffness behaviour of the studied soil modified with lime can be modelled as a potential function of the normalized deviatoric stress as, follows for standard effort (1) and modified effort (2). The resilient coefficients are summarized in Table 6.

$$Mr = 122 \cdot \left(\frac{\sigma_d}{p_a}\right)^{-0.80} \qquad (1)$$

$$Mr = 254 \cdot \left(\frac{\sigma_d}{p_a}\right)^{-0.63} \qquad (2)$$

Table 6: Resilient coefficients of soil and soil modified with lime for two compaction energies

Material	K_1	k_2
Soil – Standard Energy	48	-1.47
Soil – Modified Energy	90	-1.15
Soil-Lime – Standard Energy	122	-0.80
Soil-Lime – Modified Energy	254	-0.63

The fact that the material is modified does not mean that it cannot improve the strength and stiffness of soil. The Mr values of the soil modified with lime are greater than those of the natural soil for the same compaction energy. Furthermore, the sensitivity of the stiffness under vertical stress decreases when the soil is modified with lime, as seen from the lesser k_2 values of this material with respect to those of the natural soil (Table 2). It should be noted that the Mr values

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of modified soil, when compacted at standard energy, are similar to natural soil compacted at modified energy for low values of σ_d (between 0.3 and 0.5), while they are significantly different only for the highest σ_d values. Although the stiffness of soil is improved when it is modified with lime, low compaction degrees produce a material as or more deformable as the soil without treatment but with higher compaction degrees. This draws attention to the importance of an adequate compaction in this kind of material to ensure the attempted improvement.

Field study

Monitoring of test section 1

The first test section (TS1) was constructed in April 7, 2008 during the rice harvest. The harvest period (between March and April) is the most intensely trafficked by heavy trucks. The traffic per year estimated for TS1 is about 26,000 ESALs. TS1 consists of two sectors each 50 m in length, with a 15 cm-thick wearing course of silty clayey soil modified with 3% and 5% of lime, respectively. These layers were constructed over the current 10 cm-thick base layer of coarse material, a rhyolite carried from a quarry located 80 km from the test section, and the subgrade of local silty clayey soil (Behak 2011). The TS1 was trafficked immediately after its construction, without enough curing time and when this traffic was the worst for the pavement. A thin layer, of about 1 cm thick, of rhyolite material was extended over the soil-lime layer two years after test section construction in order to reduce the dust emission of soil-lime material and to improve the roughness of the wearing course.

Table 7 summarizes the results of in-situ density testing by sand-cone tests and moisture content performed immediately after the construction of TS1. The compaction degree (CD) reached for both soil-lime layers was very low because a smooth roller was used to compaction and the compaction moisture content was higher than the OMC of these materials.

Table 7: Dry density and moisture content of wearing courses of Test Section 1

Point	Dry Density (kN/m^3)	Moisture (%)	Compaction Degree (%)
Soil-3%Lime	14.0	18	80
Soil –5%Lime	12.1	20	70

Deflection measurements with Benkelman beam were performed on April 17, 2008 (immediately after construction), August 26, 2008 (131 days), May 11, 2012 (1485 days), and December 5, 2012 (1693 days), at two points in TS1 and a control point in the current pavement (CP1). One point is representative of the sector with a wearing course of soil modified with 3% of lime (S3L) and the other point of the sector with soil modified with 5% of lime (S5L). The results of the four monitoring times by point are depicted in Figure 5. The deflection measured in CP1 was almost half that in the two points of TS1 immediately after its construction (131×10^{-2} mm in the CP1 and 264×10^{-2} mm, and 224×10^{-2} mm in TS1-S3C and TS1-S5C respectively). Without enough time for pozzolanic reactions, the soil-lime mixtures behave as fine soil, and are less stiff than the rhyolite material of the wearing course of the current pavement. Four months later (131 days), the deflection relationship is reversed and they are less in TS1-S3C (67×10^{-2} mm) and TS1-S5C (65×10^{-2} mm) than in CP1 (93×10^{-2} mm), showing that there were cementitious reactions in this period that improve the elastic deformability of the soil-lime material.

The deflections of the two points of TS1 were similar in the first and second monitoring which is in agreement with the laboratory results, mainly in the cyclic triaxial tests where the resilient modulus was shown independent of the lime content. The deflection decreased in the first 131 days 75% and 71% in TS1-S3C and TS1-S5C respectively, despite the heavy traffic suffered by the pavement immediately after its construction and the medium and low temperatures, typical of the autumn and winter of the region, occurring in the period.

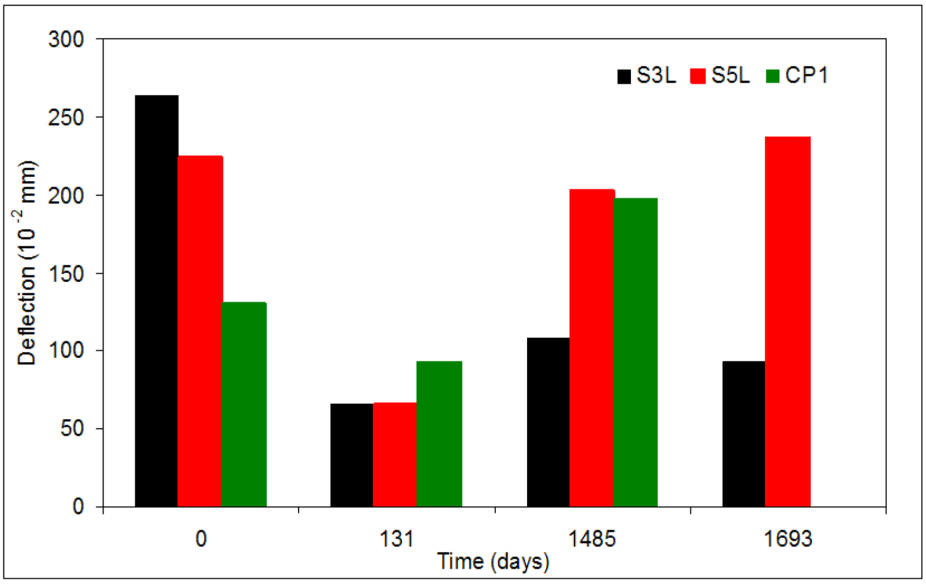

Figure 5: Deflection as function of time in fixed points of Test Section 1.

Deflections in the three points were greater at 1485 days (May 2012) with respect to the values measured at 131 days. Nevertheless, deflection values remained less than those measured immediately after the construction of the pavement. The values in TS1-S3C and TS1-S5C were respectively 41% (109×10^{-2} mm) and 90% (202×10^{-2} mm) of values measured initially. Between the second and third monitoring a thin rhyolite layer was placed over the soil-lime wearing course that resulted in a reduction of the soil-lime from 15 cm to 12 cm. Such thickness reduction implies an increase in vertical stress on the top of the subgrade that causes the decrease in its stiffness. As a result, the deflection of the pavement increased. Furthermore, the saturation of the subgrade was greater in TS1-S5C than in TS1-S3C which further affects the stiffness of soil and explains the greater deflection observed in TS1-S5C. Similar results as those observed after 1485 days for both points of the test section were observed at 1693 days.

Figure 6: Superficial condition immediately after construction and with 5 years of Test Section 1. Left: Immediately after construction. Right: 5 years after

After 5 years, the pavement of TS1 does not exhibit rutting and there is not observed cracking or crushing in the modified layers, as shown in Figure 6 where the surface is compared with the superficial condition immediately after construction. The deflection values indicate that the pavement would be in the effective fatigue life phase (Theyse et al. 1996).

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Monitoring of test section 2

The second test section (TS2) was constructed in March 3, 2011, shortly before the start of the rice harvest. This section is less trafficked than TS1, with an estimated traffic of about 10,000 ESALs per year. 100 m of 15 cm-thick wearing course of silty clayey soil modified with 4% of lime was constructed over the current 10 cm-thick base layer of rhyolite carried from a quarry located 80 km from the test section, and the subgrade of local silty clayey soil. TS2 was released to traffic immediately after its construction, without enough curing time and when this traffic was the worst for the pavement. A thin layer, about 1 cm thick, of rhyolite material was extended over the soil-lime layer one year after the test section was constructed to reduce the dust emission of soil-lime material and to improve the roughness of the wearing course. The compaction degree (CD), measured by in-situ density tests by the sand-cone method in three points, was 91%, 92%, and 101% of the MDD of modified Proctor test. These CD are low because the compaction effort of the soil-lime layer was standard rather than modified. However, they are better than those achieved in TS1, mainly due to the better control of the compaction moisture.

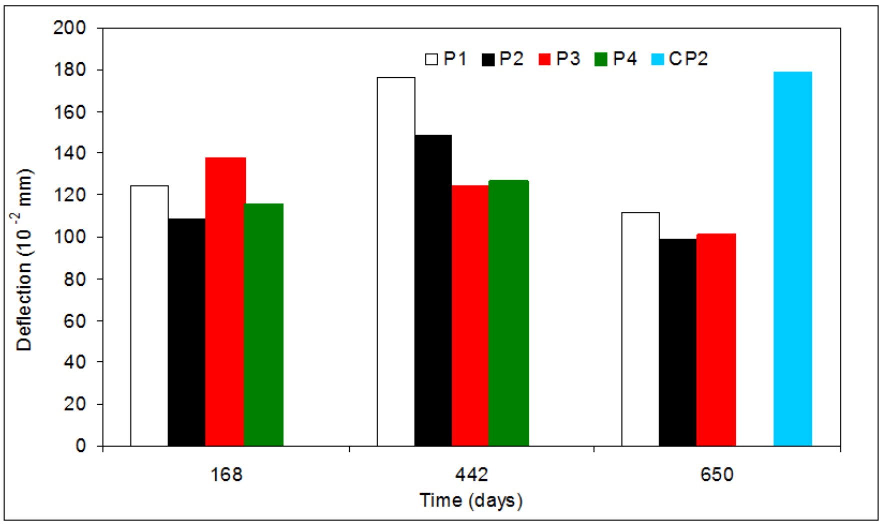

Figure 7: Deflection as function of time in fixed points of Test Section 2.

Figure 7 shows the results of the monitoring conducted at four points in TS2 (P1, P2, P3, and P4) on August 18, 2011 (168 days), May 11, 2012 (442 days), and December 5, 2012 (650 days). A control point on the adjacent current pavement (CP2) was also considered. Differences in the deflection values are observed in the four points of the wearing course, particularly between P1 and P2 at 168 days and 442 days. However, the deflections were similar at 650 days. Variations in deflections are observed for each point, but they are random. It is believed that rather than the compaction degree (CD) and the heterogeneity of pavement materials, particularly of soil modified with lime, differences in readings are due to changes in moisture content in each point. The average deflection values were of $122x10^{-2}$ mm at 168 days, $144x10^{-2}$ mm at 442 days, and $104x10^{-2}$ mm at 650 days. The average deflection for all points and monitoring times was $123x10^{-2}$ mm. This average deflection value is less than the deflection measured in the CP2 ($179x10^{-2}$ mm), which means that the modified material is stiffer than the material traditionally used in the region. Despite the material of a wearing course is modified, its stiffness improves so that it is greater than that of the coarse material.

Only 2 years after construction, the pavement of TS2 does not exhibit rutting and there is not observed cracking or crushing in the modified layers. Considering the experience of the TS1, it is possible to conclude that the pavement is in the effective fatigue life phase (Theyse et al. 1996).

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CONCLUSIONS

A research project into the behaviour of soil modified with lime as wearing course in low-volume roads pavements was undertaken with laboratory mechanistic testing and monitoring of full-scale pavement tests section performed. Two test sections were constructed in Cebollatí in eastern Uruguay with a wearing course of local silty clayey soil modified with 3%, 4%, and 5% of lime over the current pavement (base layer of coarse material and subgrade of silty clayey soil). The first trial was undertaken in April, 2008 and the second in March, 2011. Unconfined compression strength tests, splitting tensile strength tests and cyclic triaxial compression tests were conducted on mixtures of Cebollatí's soil with different lime contents, curing times and compaction effort. The results obtained in 5 years of experience lead to the following conclusions.

The unconfined compression strength (UCS) and splitting tensile strength (STS) of silty clayey soil are improved with the addition of 3% - 5% of lime. This improvement is indicative of the occurrence of pozzolanic reactions between the calcium hydroxide of lime and the silica and alumina of clayey minerals of soil, mainly through tensile strength gain, and is time dependent. The UCS and STS gain continues while calcium hydroxide free ions and silica and alumina are available to react, in presence of water, to form cementitious products. However, the unconfined and tensile strength were independent of the lime contents in the studied range. The 28-day UCS of soil treated with lime was of 0.9 MPa which means, according to the Austroads Guide to Pavement Technology of 2006, that it is a modified material. That is, the studied soil-lime material behaves as unbound material.

An improvement of resilient modulus (Mr) is observed when the silty clayey soil is treated with 3% - 7% of lime. Its stiffness is a potential function of the deviatoric stress (σ_1), with a negative exponent, and independent of the confining stress (σ_3). This behaviour is similar to Cebollatí's soil, which in turn is characteristic of cohesive soils. The soil-lime material performs as modified, which is in agreement with the conclusion made from the UCS tests. Similarly to that observed in UCS and STS tests, the Mr of soil modified with lime is independent of the lime content in the studied range but, furthermore, it is independent of the curing time between 28 days and 90 days. The stiffness of soil modified with lime is a function of deviatoric stress, independent of the lime content and curing time for a given compaction effort. The fact that the material is modified does not mean that it cannot improve its strength and stiffness.

A decrease in deflection values was measured at recording locations in the wearing courses of soil modified with 3% and 5% of lime between the first monitoring, immediately after construction of Test Section 1, and the second monitoring, 131 days after construction. Form that time up until the present, deflection values in the wearing courses of soil modified with 3%, 4%, and 5% of lime have remained almost constant for the same thickness of layers. The measured variations were due to the changes in the moisture content of the subgrade. Despite compaction difficulties, mainly in the Test Section 1, and pavements trafficked immediately after construction when the heavy truck traffic was greatest together with poor drainage conditions of the sites, the soil-lime layers improved the pavement stiffness and the pavement performance was not affected for the first few months.

The deflections in pavements with a wearing course of soil modified with 3% and 5% of lime were similar, when the moisture of the subgrade soil was similar in both monitoring points. This behaviour was in accordance with that observed in the laboratory tests, where the strength and the stiffness were independent of the lime content in the range studied.

After 5 and 2 life years, the wearing courses do not present significant rutting nor superficial cracking or crushing (Figure 6) and, according to the similar deflections values measured in the last monitoring, they are in the effective fatigue life phase. The layers of soil modified with lime contribute to the decrease of compressing vertical stresses in the top of the subgrade. As the deformability of silty clayey soil is proportional to this vertical stress, its permanent deformation is less and, thus, the development of rutting is slower and the pavement life increases. Possible micro-cracks formed at early ages in the modified soil layers would have sealed by the cementation of the pozzolanic reactions between the lime and the silica of the soil, phenomenon known as autogenous healing (Thompson and Dempsey 1969). Besides that, the early-life trafficking is likely to produce micro-cracking of modified materials and hence reducing the risk of they becoming bound (Jameson 2013) and thus extending their fatigue life.

As general conclusion, the pavements with wearing course of soil modified with 3% to 5% of lime are a well-documented alternative for low-volume roads. All test sections have a 25 mm armouring of rhyolite gravel. These pavements exceed by far the strength, stiffness and life of traditional pavements with a wearing course of coarse material traditionally and currently used the region of the study. The construction of wearing courses with local soils is an alternative with economic benefits, through reduction of construction, maintenance, and rehabilitation costs. Furthermore, it has environmental benefits due to a rationalisation of the use of non-renewable resources such as soils and rocks.

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ACKNOWLEDGEMENTS

The authors thank the Local Government of Rocha and the Local Board of Cebollatí for their continue interest and support, mainly in the construction and monitoring of test sections. Also, we are grateful with the Technology Development Program of the Inter-American Development Bank-National Science and Technology Direction for funding the research. A special acknowledgment goes to all those who contributed in different ways, in particular to Jorge A. Ceratti of Pavement Laboratory (LAPAV) of Federal University of Rio Grande do Sul (UFRGS) and his collaborators, Alvaro Rostan and Marcos Musso, professors at the Geotechnical Department of the University of the Republic. Finally, a big thank you to the laboratory technician Juan Alvez.

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