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Mechanistic behaviour under traffic load of a clayey silt modified with lime

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The behaviour under traffic loads of a soil modified with lime was researched through a laboratorial mechanistic approach. A clayey silty soil from Cebollatí, eastern Uruguay, and a commercial calcic lime were used. Cyclic triaxial compression tests were carried out on the soil treated with different lime contents, curing periods and dry densities in order to study its resilient behaviour. The fatigue life was investigated through cyclic indirect diametrical tensile tests by varying the lime content and curing period. A model of resilient modulus as power function of the deviator stress was found, independent of the lime content and curing period and dependent on the compaction effort. The model is similar to that observed in the natural soil, which means that the treated soil behaves as modified material. Nevertheless, the material is susceptible to fatigue cracking. The fatigue life, expressed as the number of load cycles up to break, is an inverse power function of the initial tensile strain, being most sensitive to that parameter. It was found that the fatigue life can be better explained when expressed as function of the ratio between the initial tensile strain and the tensile stain at break.

Keywords: pavements; soil modification; mechanistic approach; resilient modulus; fatigue life

1. Introduction

Soil treatment with lime is a practice widely used to produce pavement materials. The addition of lime to reactive fine-grained soils improves the strength, stiffness, durability and long-term performance under traffic loading (Gnanendran & Piratheepan, 2010; Mallela, Von Quintus, & Smith, 2004; Núñez, 1991; Núñez, Lovato, Malysz, & Ceratti, 2005).

There are essentially two forms of improvement: modification and stabilisation (Little, 1999). Modification occurs due to cationic exchange and flocculation, resulting in reduction of plasticity and swelling, and improvement of texture and workability, although it can produce structural improvements such as significant bearing capacity. It begins immediately after lime addition and generally develops in a few minutes or hours. Stabilisation takes place when the proper amount of lime is added to a soil to develop a significant level of long-term strength gain through pozzolanic reactions, and develops over time and can take months, or even years, until the new material reaches the final properties.

Lime-treated materials are also divided into modified and stabilised according to its 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

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a sufficiently enhanced elastic modulus and tensile strength to have a practical application in stiffening the pavement. Nevertheless, as reported by several authors (Behak, 2013; Behak & Núñez, 2014; Jameson, 2013; Nahlawi, Chakrabarti, & Kodikara, 2004; Wallace, 1998), modified materials may develop tensile strength and tensile strain; therefore they may be susceptible to fatigue cracking. Jameson (2013) concluded that further research is required to improve the characterisation of modified materials to reduce the risk of fatigue.

Thompson (1970) defined a lime-soil mixture as acceptable for a structural base if the unconfined compression strength (UCS) exceeded about 1050 kPa, while some transport agencies of United States require minimal UCS values for subbase and base layers between 700 and 1400 kPa (Little, 1999). The Austroads Guide to Pavement Technology of 2006 suggests for lime modification of pavement layers a 28-day UCS between 0.5 and 1.5 MPa, and for lime stabilisation UCS greater than 1.5 MPa (Jameson, 2013). A maximum design modulus of 1000 MPa is normally adopted for modified materials.

The development of mechanistic-empirical pavement design methods has motivated the need to obtain the elastic or resilient modulus of soils and materials, as well as the characterisation of its fatigue life. However, only a few studies 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, Maina, & Kannemeyer, 2007). The laboratory characterisation of cemented materials for pavements of low-volume roads have been recently completed by the Austroads (Gonzalez, Howard, & De Carteret, 2010), concluding that it is necessary to do a substantial revision in the thickness design method for such materials. Although essential, the laboratorial characterisation of pavement materials presents some limitations, being the scale effect one of them. To avoid this, full-scale pavement test sections and accelerated pavement testing devices are used.

A global research on the performance of three full-scale pavement test sections with wearing courses made of a clayey silty soil modified with 3%, 4% and 5% of lime was conducted on a low-volume road in Cebollatí, eastern Uruguay (Behak, 2011, 2013; Behak & Núñez, 2014). As part of the research, a laboratorial study of the elastic response under cyclic loads and fatigue of the lime-modified soil was carried out, with the purpose of establishing mechanistic models. In this context, this paper aims at presenting the development, results and conclusions of this last study.

2. Mechanistic behaviour of cemented materials

2.1. *Recoverable strain*

Hveem (1955) established that progressive cracking of asphaltic mixtures is due to resilient deformation of the underlying layers. The resilient modulus (M_r) of pavement materials is an essential parameter for mechanistically based pavements design and it is generally considered as an appropriate measure of stiffness for pavement materials (Ahmed & Khalid, 2011). The M_r is defined as the ratio of deviator stress to recoverable strain under repeated loading.

Many nonlinear empirical models have been suggested to represent the M_r of soils and unbound materials (Seed, Chan, & Lee, 1962). Generally, the M_r of fine soils depends on deviator stress (σ_d), as is given in Equation (1).

$$M_r = k_1 {\sigma_d}^{k_2}, \tag{1}$$

where k_1, k_2 are regression constants.

Meanwhile the M_r of coarse materials strongly depends on the confining pressure (σ_3) (Dunlap, 1963), and models as function of bulk stress have been proposed (Seed et al., 1962; Uzan, 1985; Witczak, 2001).

Little (1999) reported that the long-term effects of stabilisation of soils with lime increases 10 times, or even more, the M_r of the untreated soil. Little and Yusuf (2001) obtained M_r values between 210 and 400 MPa for a moderate to highly plastic soils treated with lime after five days curing at 38°C, and values between 260 and 415 MPa for four Mississippi soils stabilised with 4% and 6% of lime, after seven days curing at 40°C, previously socked in water for 24 h.

The resilient modulus increases with increasing lime content, due to the generation of cementitious products. Elkady, Al-Mahbashi, and Al-Shamrani (2015) reported that the M_r of an expansive clay modified with lime increased up to a stabiliser content of 4%, but decreased for higher lime contents. On the other hand, Solanki, Khoury, and Zaman (2009) found a quite small variation among the M_r of a CL-ML soil stabilised with 3%, 6% and 9% of lime. Puppala, Mohammad, and Allen (1996) observed that the M_r of a silty clayey soil stabilised with 4% of lime was not highly affected by different compaction degrees.

A dependency of M_r on both deviator and confining stresses has been found (Elkady et al., 2015; Puppala et al., 1996; Solanki et al., 2009). Elkady et al. (2015) proposed a three-parameter model (Equation (2)) and Puppala et al. (1996) proposed a model with M_r depending on the octahedral stress state (Equation (3)).

$$M_r = k_1 \left(\frac{\theta}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a} + 1\right)^{k_3}, \tag{2}$$

$$\frac{M_r}{\sigma_{adm}} = k_1 \left(\frac{\sigma_{oct}}{p_a}\right)^{k_2} \left(\frac{\tau_{oct}}{p_a}\right)^{k_3}, \tag{3}$$

where θ = bulk stress = $\sigma_1 + 2\sigma_3$; σ_{adm} = admissible normal stress; σ_{oct} = octahedral normal stress = $(\sigma_1 + \sigma_2 + \sigma_3)/3$; and τ_{oct} = octahedral shear stress = 1/3 $\sqrt{(\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_3 - \sigma_1)^2}$.

2.2. *Fatigue*

The main distress mechanisms of pavement with stabilised layers are fatigue cracking, permanent deformation and thermal cracking (Gnanendran & Piratheepan, 2010). Fatigue is the response bearing without formation of significant cracks under cyclic loads and environmental conditions and it is caused by successive bendings of pavement layers resulting in crack formation and its propagation (Crockford & Little, 1987; Little & Yusuf, 2001).

Fatigue life is commonly expressed as the number of load applications to fail the stabilised material at a particular stress or strain level at the bottom of the stabilised base layer (Gnanendran & Piratheepan, 2010). Typically, a fatigue life relationship is shown by plotting the initial tensile stress or strain at the bottom of the stabilised layer versus the number of load cycles to failure.

According to the South African Mechanistic Design Method (SAMDM), cemented materials may exhibit two failure modes, namely effective fatigue (cracking) and crushing (De Beer, 1990). The critical parameters for cemented materials are the maximum tensile strain (ε) at the bottom of the layer controlling the effective fatigue life, and vertical compression stress (σ_v) on top of the cemented layer controlling crushing life (Theyse, De Beer, & Rust, 1996). Effective fatigue models are expressed as the number of load applications (N) as function of the tensile strain (ε) and tensile strain at break (ε_b) for soils stabilised with cement (Equation (4)). Regression parameters are defined according to road category.

$$N = 10^{a\left(1-\frac{\varepsilon}{b\cdot\varepsilon_b}\right)}, \tag{4}$$

where a and b are parameter models, as defined below.

$a = 6.72, 6.84, 6.87, 7.06$ for Road Category A, B, C, D, respectively,
$b = 7.49, 7.63, 7.66, 7.86$ for Road category A, B, C, D, respectively.

The Austroads Guide to the Design of Roads Pavements adopted the SAMDM model for fatigue life of cemented materials (Gonzalez et al., 2010). However, the effective fatigue life is estimated as power function of ε_o in microstrains ($\mu\varepsilon$). Pretorius (1970) presented the model showed in the Equation (5), where the high exponent (20.3) indicates that ε_o remarkably affects the fatigue life of soils stabilised with cement.

$$N = \left(\frac{142}{\varepsilon_o}\right)^{20.3}. \tag{5}$$

Baran and Aubrey (1978) considered that the model of the Equation (5) is valid for materials with a modulus larger than 10,000 MPa, thus defining an allowable strain value of 72 $\mu\varepsilon$ for $N = 10^6$ cycles.

Angell (1988) reported a relationship for cemented materials with modulus of 2000, 5000 MPa and higher than 10,000 MPa (Equation (6)), with K values of 259, 244 and 152 corresponding to moduli of 2000, 5000 MPa and higher than 10,000 MPa, respectively.

$$N = \left(\frac{K}{\varepsilon_o}\right)^{12}, \tag{6}$$

where K is a constant value depending on the material stiffness.

The Austroads Pavement Design Method of 1987 and 1992 adopted similar models but with exponent equal to 18 and K values of 280, 200 and 150 for moduli of 2000, 5000 and 10,000 MPa, respectively (Jameson, 2010). Nowadays, the model for soils treated with cement is expressed as 12th-power function (Jameson, 2010) (Equation (6)), with K values of 440, 350, 310, 260 and 240 for moduli of 2000, 3500, 5000, 10,000 and 15,000 MPa. However, Gonzalez et al. (2010) found exponents ranging from 14 to 26, with an average of approximately 22, for 12 different soils stabilised with cement.

3. Materials and methods

3.1. *Soil*

A brown fine soil was used in the research. It was characterised by means of grain-size analysis (ASTM D422), Atterberg limit tests (ASTM D4318) and specific gravity tests (ASTM D854). Based on the results presented in Table 1 the soil can be defined as clayey silt. X-ray diffraction (XRD) showed that the predominant clayey minerals are montmorillonite and illite. As per the Unified Soil Classification System (USCS) (ASTM D2487) the soil is clay of high compressibility.

The UCS of the soil compacted at maximum dry density (MDD) and optimum moisture content (OMC) of modified Proctor test (AASHTO T180, 1986) is 214 kPa.

Table 1. Characteristics of the soil.

Property	Value
Passing #200 (0.074 mm)	68%
Sand	32%
Silt	46%
Clay	21%
Liquid limit	51%
Plastic limit	19%
Plasticity index	32%
Specific gravity	2.75
USCS	CH

3.2. *Lime*

A commercial calcic lime, with some 66% of calcium oxide (CaO), 5% of magnesium oxide (MgO) and traces of silica and ferric oxide, was used. It was a fine lime with 100% passing sieve #10 and 93% sieve #200; the silt fraction constituting the 91% of the total dry mass.

3.3. *Soil-lime specimens*

The study of the elastic strain behaviour under cyclic loads of clayey silty soil treated with lime was carried out on compacted specimens with 10.1 cm diameter and 20.2 cm height. The influence of lime content (3%, 4%, 5% and 7% regarding the soil dry weight), curing periods (28 and 90 days) and two compaction energies (standard and modified of Proctor test, AASHTO T99, 1986; AASHTO T180, 1986, respectively) were analysed. The specimens were compacted at OMC corresponding to both energies. Table 2 summarises the experimental program of cyclic triaxial compression tests, including the denomination of materials. The OMC are the mean values of the specimens of each lime content.

Fatigue tests were carried out on cylindrical specimens of 10 cm in diameter and 6.24 cm in height compacted at MDD and OMC of Proctor modified energy. As shown in Table 2, two lime contents were considered: 3% and 5%. The specimens of soil treated with 3% of lime were cured just for 150 days, while those with 5% of lime were cured by either 28 or 150 days.

Specimen preparation began with manual mixing of the dry soil with the corresponding amount of lime up homogenisation. Thereafter, water was added and the mixtures were manually homogenised once more. Specimens were immediately compacted into metal moulds using a Proctor test hammer. Then, they were extracted from moulds, wrapped in PVC film and stored at room temperature during the established period.

UCS of clayey silty soil treated with 3%, 4% and 5% of lime, cured for 28 days, were determined following AASHTO T208 (1986). Specimens of 3.76 cm in diameter and 7.66 cm in height were compacted at MDD and OCM of modified Proctor test into a mini-Harvard mould by kneading, immediately after mixing. Specimens of each lime content were compacted in triplicate.

It is shown in Table 2 that UCS remarkably increased when the soil was treated with 3–5% of lime, for both curing periods. However, the 28- and 90-days UCS values are quite similar for the three lime contents (mean values of 0.81 and 1.54 MPa, respectively), so that the treated soil UCS can be considered independent of the lime content. The 28-days UCS is 3.8 times the UCS of natural soil but it increases 1.9 times between 28 and 90 days. Although the UCS increases when lime is added to soil, according to the Austroads Guide to Pavement Technology of 2006 (Jameson, 2013), the studied material may be defined as modified for all lime contents.

Table 2. Test program and soil-lime specimens used in the research.

Test	Designation	Lime (%)	Time (days)	Compaction energy	MDD (kN/m^3)	OMC (%)	UCS (MPa)
Cyclic triaxial compression	Soil-SE	0	0	Standard	16.3	14.7	—
	Soil-ME			Modified	17.6	15.3	0.21
	S3L-28-SE	3	28	Standard	15.7	15.3	—
	S3L-28-ME			Modified	17.8	14.8	0.83
	S3L-90-SE		90	Standard	15.5	15.8	—
	S4L-90-ME			Modified	17.7	14.5	1.51
	S4L-28-SE	4	28	Standard	15.6	14.6	—
	S4L-28-ME			Modified	17.9	14.8	0.83
	S4L-90-SE		90	Standard	15.3	15.3	—
	S4L-90-ME			Modified	17.7	15.0	1.51
	S5L-28-SE	5	28	Standard	15.6	14.2	—
	S5L-28-ME			Modified	17.9	14.7	0.76
	S5L-90-SE		90	Standard	15.4	15.2	—
	S5L-90-ME			Modified	17.7	14.7	1.60
	S7L-28-SE	7	28	Standard	15.7	14.2	—
	S7L-28-ME			Modified	17.8	14.8	—
	S7L-90-SE		90	Standard	15.2	15.0	—
	S7L-90-ME			Modified	17.7	14.7	—
Fatigue	S3L-150	3	150	Modified	18.0	14.5	—
	S5L-28	5	28		17.6	15.6	—
	S5L-150		150		17.9	15.7	—

3.4. *Cyclic triaxial compression tests*

The resilient behaviour of the modified soil was studied through cyclic triaxial compression tests following AASHTO TP46-97 (2004) Method. Combinations of principal stress states are applied, with the axial deviator stress (σ_d) being cycled while the confining pressure (σ_3) is kept constant. Loading cycles with frequency of 1 Hz are applied with a pulse of 0.1 s followed by a rest period of 0.9 s. Table 3 describes the applied sequences, where the Sequence 0 corresponds to the initial conditioning. The M_r at each stress state was calculated averaging the values of the last five cycles.

3.5. *Cyclic indirect diametrical tensile tests*

The fatigue behaviour of the soil modified with lime was studied through cyclic indirect diametrical tensile tests. The specimens were tested in the controlled stress mode, and the applied stresses were percentages of the split tensile strength of each combination (lime content and curing period). The initial tensile strains (ε_o) were externally measured with LVDT, as is shown in Figure 1, and were computed as the mean values of three readings made every 10 load cycles after 50 conditioning load cycles (that is at 60, 70 and 80 cycles).

Loading cycles were applied, at a frequency of 1 Hz, with a pulse of 0.1 s followed by a rest period of 0.9 s. The numbers of cycles (N) were recorded by an external timer and tests were carried out up to either the specimen breaking or up reaching 10^6 cycles.

4. Results and discussion

4.1. *Resilient behaviour*

Fitting curves after statistic analysis of M_r of soil compacted at standard and modified energies as function of deviator stress normalised by atmospheric pressure (σ_d/p_a) are depicted in Figure 2.

Table 3. Stress states applied in the cyclic triaxial compression tests.

Sequence	σ_3 (kPa)	σ_d (kPa)	Cycles (N)
0	27.6	48.3	1000
1	55.2	27.6	100
2	41.4	27.6	100
3	27.6	27.6	100
4	13.8	27.6	100
5	55.2	48.3	100
6	41.4	48.3	100
7	27.6	48.3	100
8	13.8	48.3	100
9	55.2	69.0	100
10	41.4	69.0	100
11	27.6	69.0	100
12	13.8	69.0	100
13	55.2	96.6	100
14	41.4	96.6	100
15	27.6	96.6	100
16	13.8	96.6	100

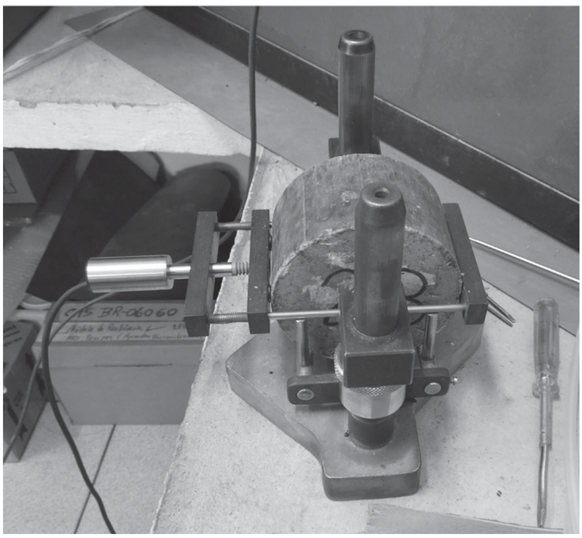

Figure 1. Strain measurement device used in fatigue life tests.

The M_r of soil decreases with increasing of σ_d/p_a for both compaction energies, which is in accordance with the behaviour of fine-grained soils (Seed et al., 1962).

The M_r decreased from 285 to 42 MPa and from 360 to 80 MPa, in specimens compacted at standard energy and modified energy, respectively, which means a M_r decrease of 85% and 77%, respectively, for an increasing of σ_d/p_a of 3.7 times. For a given deviator stress, the M_r of soil compacted at modified energy is higher than that of compacted at standard energy.

Models of M_r as potential function of σ_d/p_a were found for specimens of the soil compacted at standard and modified energies as are respectively presented in Equations (7) and (8). The k_2 exponents show that the M_r sensibility related to the deviator stress variation decreases as the

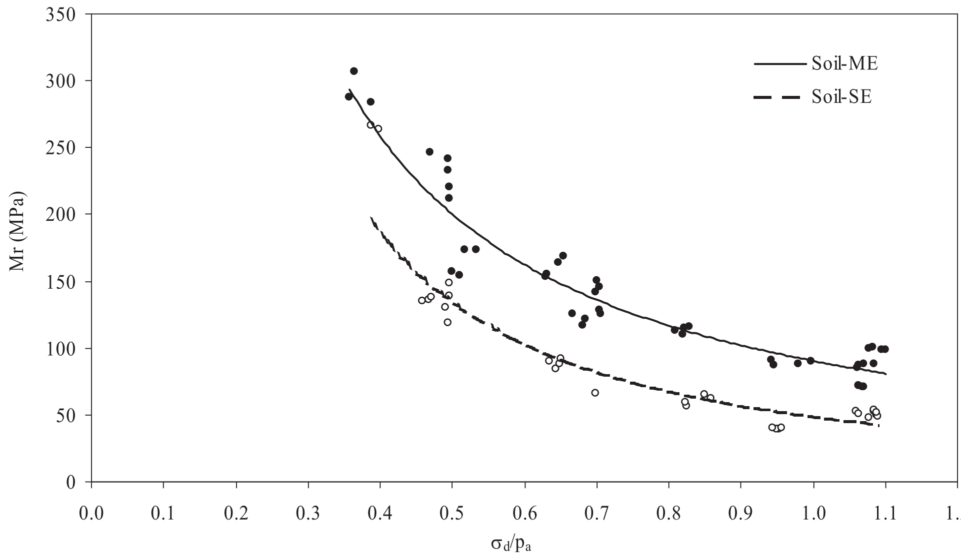

Figure 2. Resilient modulus of soil compacted at standard and modified efforts as function of normalised deviator stress.

compaction energy increases.

$$M_r = 48.5 \cdot \left(\frac{\sigma_d}{p_a}\right)^{-1.47} \quad (R^2 = 0.90), \tag{7}$$

$$M_r = 90.4 \cdot \left(\frac{\sigma_d}{p_a}\right)^{-1.15} \quad (R^2 = 0.91). \tag{8}$$

Fitting curves of M_r of soil treated with different lime contents and curing period as function of σ_d/p_a, for standard and modified energies, are respectively shown in Figures 3 and 4. For a given compaction energy and deviator stress, the M_r of soil is improved when treated with lime. Such as behaviour was due to the cementation generated by the products resulting from the alkaline reactions between the lime calcium ions and the silica and alumina present in the soil. M_r increase was best seen in specimens compacted with modified energy.

Globally, the resilient behaviour of treated soil is similar to that of the natural soil for all percentages, curing periods and compaction energies. That is, the M_r only depends on σ_d/p_a and decreases with increasing deviator stress, with a rate that reduces as the stress level increases. In accordance with the analysis done with UCS results, the resilient modulus results allows classifying the soil treated with lime as a modified material, as defined by the Austroads Guide to Pavement Technology (Jameson, 2013).

For any lime content cured and curing periods, when σ_d/p_a varied from 0.3 to 1.0, M_r varied from 550 to 210 MPa in specimens compacted with the modified energy and from 360 to 100 MPa in specimens where the standard energy was applied. These values are similar to those found by Little and Yusuf (2001) and Puppala et al. (1996), but lower than those of Solanki et al. (2009).

No variation trend of M_r with lime content, for both curing periods and compaction efforts, was observed. In spite of the curves of soil modified with 7% of lime compacted with standard energy being above the others, at several stress states that mixture M_r is lower than those of soil modified with 3%, 4% and 5% of lime. This may be due to the fitting process rather than

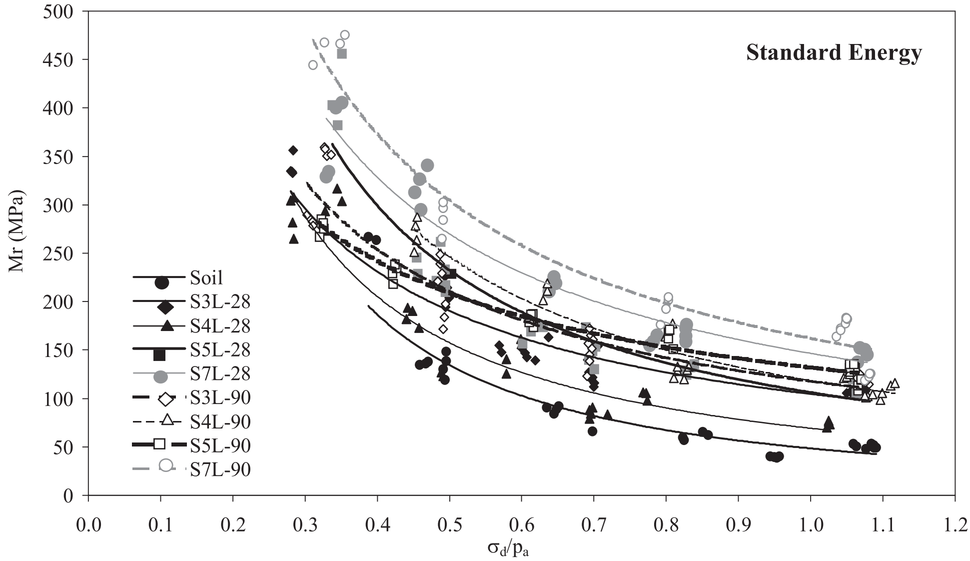

Figure 3. Resilient modulus of soil and soil treated with different lime contents and times compacted at standard energy as function of normalised deviator stress.

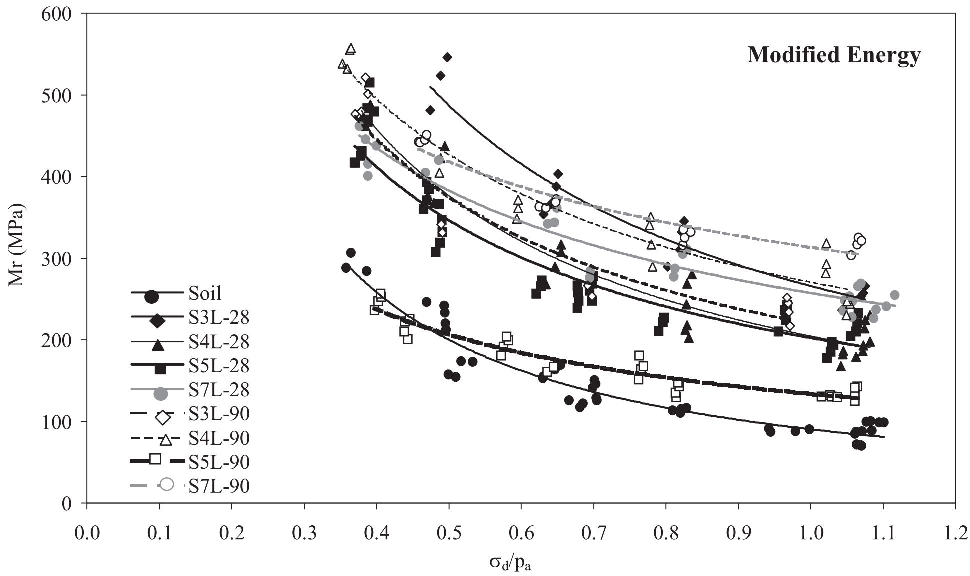

Figure 4. Resilient modulus of soil and soil treated with different lime contents and times compacted at modified energy as function of normalised deviator stress.

a trend of M_r increase with lime. Lime increases above 3% lead to little significant increases in M_r, which is in agreement with the results found by Solanki et al. (2009).

Models of M_r for each curing period, after statistical analysis of all points of soil modified with the different lime contents, are shown in Equations (9)–(12). Though the M_r of the modified soil cured by 28 days is 1.1–2.4 and 1.4–2.7 times higher than that of the natural soil, compacted with the standard and modified energies, no significant improvement is observed between 28 and 90 days. The k_1 coefficients in Equations (10) and (12) are scarcely higher (1.08 and 1.07 times) than those of Equations (9) and (11), while the k_2 exponents are practically the same for both compaction energies. The M_r behaviour with lime contents is consistent with the UCS behaviour previously analysed and it may be concluded that the resilient behaviour of the clayey silt modified with lime is practically independent of the lime content and curing period beyond 3% of lime and 28 days.

$$\text{28 days, Modified Energy:} \quad M_r = 248 \cdot \left(\frac{\sigma_d}{p_a}\right)^{-0.61} \quad (R^2 = 0.92), \tag{9}$$

$$\text{90 days, Modified Energy:} \quad M_r = 269 \cdot \left(\frac{\sigma_d}{p_a}\right)^{-0.65} \quad (R^2 = 0.94), \tag{10}$$

$$\text{28 days, Standard Energy:} \quad M_r = 117 \cdot \left(\frac{\sigma_d}{p_a}\right)^{-0.81} \quad (R^2 = 0.92), \tag{11}$$

$$\text{90 days, Standard Energy:} \quad M_r = 125 \cdot \left(\frac{\sigma_d}{p_a}\right)^{-0.82} \quad (R^2 = 0.91). \tag{12}$$

The statistical analysis of all points of soil modified with the different lime contents and curing period, for a given compaction energy, lead to the curves of M_r behaviour as function of σ_d/p_a depicted in Figure 5. The M_r of modified soil depends on the compaction energy and it is nearly twice higher in specimens compacted with the modified energy.

Equations (13) and (14) relate of M_r to the σ_d/p_a ratio with different k_1 and k_2 values depending on the compaction energy (standard and modified, respectively). In those models M_r does not depend on lime content and curing period. The k_2 parameters show that the sensibility of the

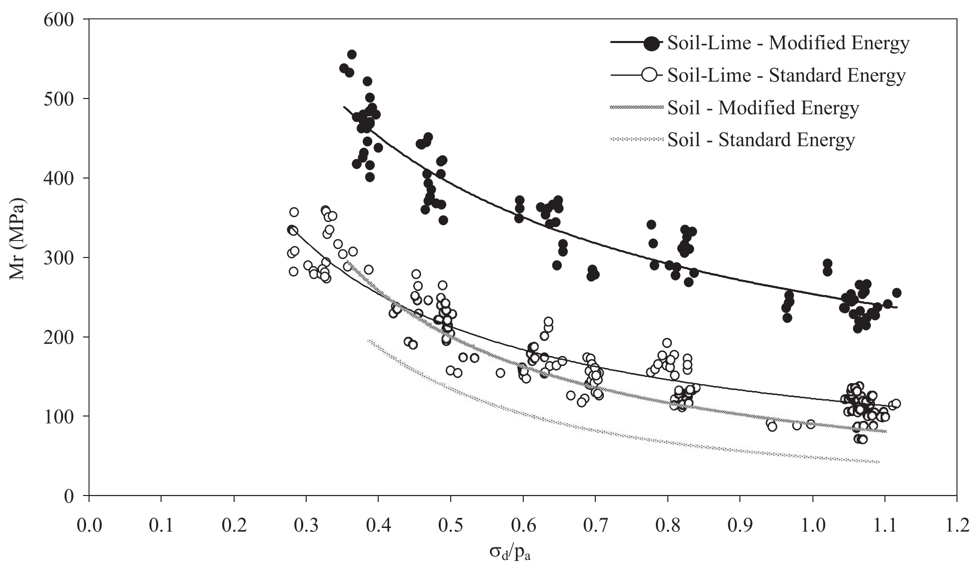

Figure 5. Resilient modulus of soil and soil modified with lime as function of normalised deviator stress contents and compaction energy.

M_r to the deviator stress variation decreases as the compaction energy increases.

$$M_r = 122 \cdot \left(\frac{\sigma_d}{p_a}\right)^{-0.80} \quad (R^2 = 0.91), \tag{13}$$

$$M_r = 254 \cdot \left(\frac{\sigma_d}{p_a}\right)^{-0.63} \quad (R^2 = 0.90). \tag{14}$$

The stiffness of the natural soil compacted at modified energy and the modified soil compacted with standard energy are similar; nevertheless it is worth to note that the compaction degree reached in the modified soil specimens was quite low (84% of MDD at the modified energy). Although cementitious reactions occur, low compaction degrees can produce materials as deformable as natural soils with high compaction degrees.

The models here reported differ from those proposed by other researchers (Elkady et al., 2015; Puppala et al., 1996; Solanki et al., 2009), since it was not observed any relationship between M_r and the confining pressure. The cementation due to pozzolanic reactions produces an increase in the cohesion of the treated soil, which further reduces the dependence of M_r on confining pressure.

4.2. *Fatigue life*

As shown in Figure 6, all specimens failed before the number of loads (N) had reached 10^6 cycles. This proves that lime-modified materials are susceptible to fatigue cracking (Jameson, 2013).

The relationship between the fatigue life of the soil modified with 3% of lime cured by 28 days and those of specimens with 5% of lime, cured by 28 and 150 days, expressed as the logarithm of N and the initial tensile strain (ε_o) in microstrains (μm) are presented in Figure 7. The fatigue life behaviour is similar to that of cemented materials (Gnanendran & Piratheepan, 2010; Gonzalez et al., 2010; Theyse et al., 1996); N linearly decreases as the ε_o increases for both lime contents and curing periods.

A remarkable increase in the ε_o corresponding to any given N is observed when the curing period increases from 28 to 150 days. For instance, considering a $N = 10^5$ cycles, ε_o is 27 μm for the S5L-28 mixture and it is 94 and 125 μm for the S5L-150 and S3L-150 mixtures (3.6 and 4.7 times higher, respectively). The fatigue life is less dependent on lime content and, moreover, the ε_o of S3L-150 is 1.3 times greater than that of S5L-150. It may be concluded that the

Figure 6. Break failure observed in fatigue life tests of clayey silt modified with lime.

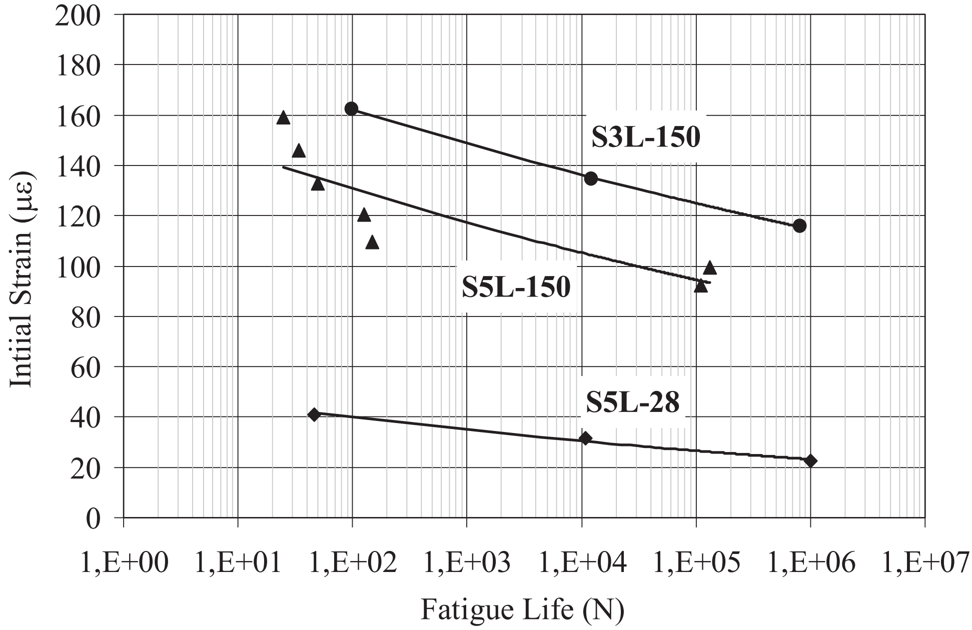

Figure 7. Fatigue life of soil modified with lime as function of the initial tensile strain.

development of cementation with longer curing period increases the fatigue of the lime-modified soil. Accordingly, Behak and Núñez (2014) reported that the split tensile strength of that lime-modified soil increases with curing period up to 180 days and does not depend on the lime content varying from 3% to 5%.

Models of the Equations (15)–(17) show that the fatigue life (N) of S5L-28, S5L-150 and S3L-150 mixtures may be expressed as inverse power function of the initial strain, in microstrains (ε_o), as previously proposed by the Austroads Guide to the Design of Roads Pavements (Angell, 1988; Baran & Aubrey, 1978; Jameson, 2010; Pretorius, 1970).

$$N = \left(\frac{53}{\varepsilon_o}\right)^{17} \quad (R^2 = 0.99), \tag{15}$$

$$N = \left(\frac{177}{\varepsilon_o}\right)^{16} \quad (R^2 = 0.77), \tag{16}$$

$$N = \left(\frac{192}{\varepsilon_o}\right)^{26} \quad (R^2 = 0.99). \tag{17}$$

The strain damage exponent of Equation (17) is higher than that adopted by Austroads for cemented materials (Jameson, 2010), but the exponents found for the soil modified with 5% of lime (Equations (15) and (16)) are similar to that proposed for soils stabilised with cement by the Austroads Pavement Design Guide 1992 (Jameson, 2010), which is a 18th-power function. The exponent corresponding to the soil modified with 3% of lime is equal to the maximum value of the range found by Gonzalez et al. (2010) when stabilising 12 different soils with cement. The mean exponent value of the 3 models is 20, similar to that obtained by Pretorius (1970) (20.3) and lower than that reported by Gonzalez et al. (2010) (22).

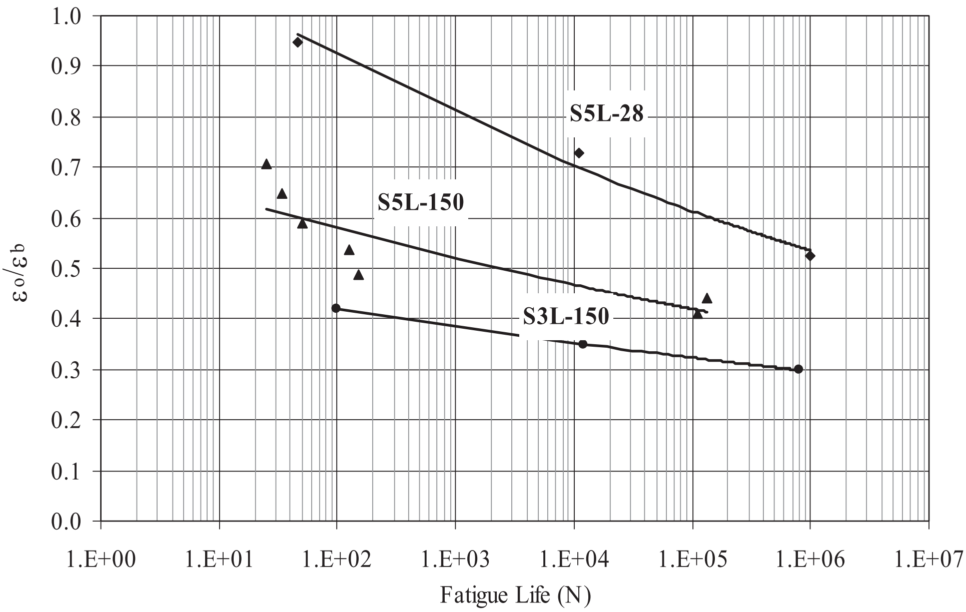

Figure 8. Fatigue life of soil modified with lime as function of the initial tensile strain-tensile strain at break ratio.

The high exponents show that the lime-modified soil fatigue life remarkably depends on the initial tensile strain. That is, the fatigue life sharply decreases with a small increase in tensile stress.

The fatigue behaviour of this lime-modified soil is as sensitive as that of the cemented materials (Angell, 1988; Baran & Aubrey, 1978; Gonzalez et al., 2010; Jameson, 2010; Pretorius, 1970), or even more. The treatment of soil with lime produces less stiff materials than those treated with cement. Besides, the mixtures tested in this research behave as modified materials. So, it could be supposed that the fatigue life of this lime-treated soil would be longer than those of cemented materials. However, further research must be carried out in order to check strain damage exponents here presented.

The fatigue life of the lime-modified soil can be better explained when it is analysed as a function of ratio between the initial tensile strain (ε_o) and the tensile stain at break (ε_b), as shown in Figure 8. The tensile strains at break were assumed as the ones measured in the split tensile stress strength tests of S5L-28, S5L-150 and S3L-150 mixtures (Behak, 2013; Behak & Núñez, 2014). Figure 9 presents the fitting curves of ε_o as functions of the tensile stress normalised by the atmospheric pressure (σ_t/p_a), applied in fatigue life tests.

Models of fatigue life as function of the ratio between the initial stain and the normalised break strain for S5L-28, S5L-150 and S3L-150 are presented in Equations (18)–(20), respectively. The strain damage exponents in the three models vary from 10 to 20 and are similar to those found by Gonzalez et al. (2010). However those researchers analysed the cemented materials' fatigue life through 4-point flexural beam tests.

$$N = 5 \cdot \left(\frac{\varepsilon_b}{\varepsilon_o}\right)^{20} \quad (R^2 = 0.94), \tag{18}$$

$$N = \left(\frac{\varepsilon_b}{\varepsilon_o}\right)^{13} \quad (R^2 = 0.79), \tag{19}$$

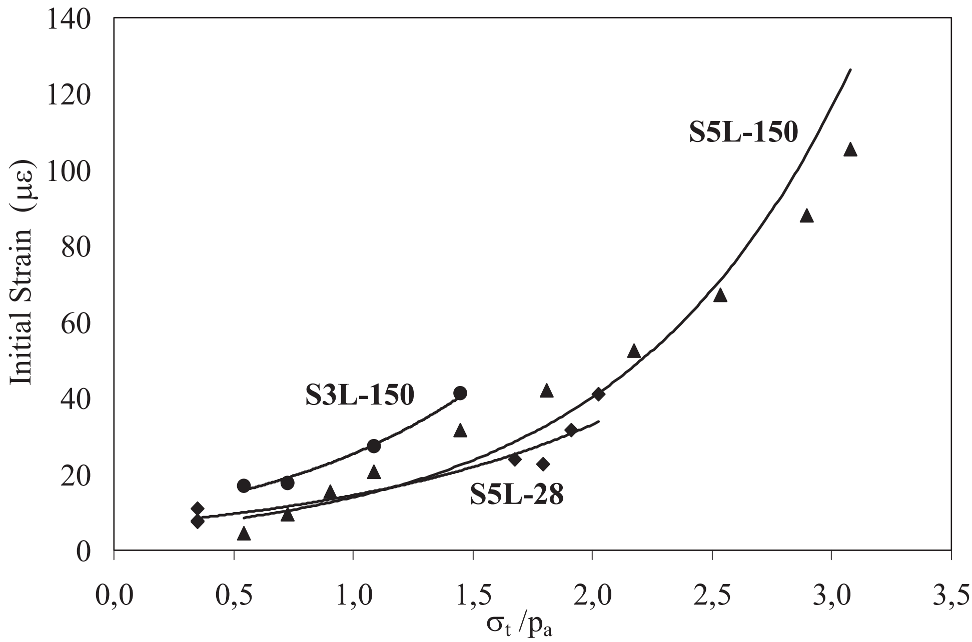

Figure 9. Initial tensile strain as function of the normalised tensile stress applied in fatigue life tests.

$$N = \left(\frac{\varepsilon_b}{\varepsilon_o}\right)^{10} \quad (R^2 = 0.84). \tag{20}$$

The consumption of fatigue life of pavements with cemented materials starts as soon as commercial traffic begins. Therefore, the actual fatigue life of such materials shall be the resultant between the fatigue improvement over the time and the fatigue consumption by the traffic and environmental action. Besides, the traffic at the beginning of the pavement service life can cause micro-cracks in cemented materials (Behak, 2013; Behak & Núñez, 2014; Jameson, 2013), which reduces the risk of early fatigue. These facts, and the convenience of reducing test times, led to the conclusion that models of fatigue life of soils modified with lime cured by 28 days suffice for pavement design. Specifically considering the lime-modified soil discussed in this paper, the model to be adopted would be that corresponding to 5% of lime and 28 days (Equation (18)).

5. Conclusions

The behaviour under traffic load of soil treated with lime was analysed through a mechanistic-empirical approach. A clayey silty soil from Cebollatí, eastern Uruguay was selected for the study. The recoverable strain behaviour and fatigue life of the soil treated with different lime contents and with different curing periods were modelled using laboratory tests. As a result, the following conclusions can be drawn.

The UCS of the clayey silty soil increased when treated with 3–5% of lime, although it proved to be practically independent on the lime content in the studied range. Considering the 28-days UCS and according to the Austroads Guide to Pavement Technology of 2006 (Jameson, 2013), the material can be defined as modified for all lime contents.

The resilient modulus (M_r) of soil increased when it was treated with 3–7% of lime. The alkaline reactions between the lime calcium and the silica and alumina of the soil produce cementitious products that lead to a stiffer and less deformable material. A model of M_r as a function of the normalised deviator stress (σ_d/p_a), similar to that obtained for the natural soil, was found for all lime contents, curing period and compaction energies. The recoverable strain behaviour of the clayey silty soil treated with lime was similar than that of the soil, which verifies that it is a modified material, despite there is improvement of resilient modulus. The M_r practically did not vary for lime contents above 3% and for curing periods beyond 28 days but it increased when the compaction energy increases. For the studied case, the model of M_r as function of σ_d/p_a does not depend on the lime content (higher than 3%) and of the curing period (from 28 days on), but it depends on the compaction effort.

The clayey silty soil treated with 3% and 5% of lime cured by 28 and 150 days develops tensile strength and strain, hence being susceptible to fatigue cracking. The fatigue life of the studied material, expressed as the number of load cycles up to break (N), can be modelled as an inverse power function of the initial tensile strain (ε_o) in the cyclic indirect tensile stress test. The mean stain damage exponent found is about 20, which is higher than that adopted by the Austroads for cemented materials (12) but similar to that proposed by the Austroads Pavement Design Guide 1992 (18). The fatigue life is very sensitive to the initial tensile stain, similar to that observed for cemented materials. Small variation in initial tensile strain causes significant variations in fatigue life. The initial tensile strain to reach a given number of cycles increases over time, between 28 and 150 days. The development of cementation over time increases a fatigue life gain of clayey silt modified with lime.

The fatigue life is better explained when analysed as function of the ratio between the initial tensile strain (ε_o) and the tensile stain at break (ε_b), the last tensile strain being measured in the monotonic split tensile stress strength test. Models of fatigue life as an inverse power function of the initial stain to normalised break strain ratio were found, with average strain damage exponent equal to 14, similar to those found for cemented materials through 4-point flexural beam tests.

Since fatigue tests may last too much, it is recommended to use results corresponding to mixtures with shorter curing periods when performing mechanistic pavement design. Since the actual fatigue life of stabilised materials shall be the resultant between the fatigue improvement over the time and the fatigue consumption by the traffic and environmental action, it may be considered that the fatigue life models of soils modified with lime with 28 days are enough for pavement design purposes.

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