Mechanical behaviour of granitic residual soil involving the effect of chemical contaminants

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Abstract: The demographic and technological evolution is confronted by geo-environmental problems. When chemicals or contaminants are introduced into the soil, they affect the soil properties such as the grain size curves, the structure, compressibility and stress-strain behaviour. However, classical soil mechanic models and method to incorporate the effects of contaminants have not yet been developed. This study proposes the analysis of the mechanical behaviour of granitic residual soil when it is blended with lime and wasted lubricating oil and also when it is subject to saturation by using BETX components (petrol). These concepts can improve the use of these types of soils in geotechnical engineering works.

Résume: L´évolution démographique et technologique est confrontée par des problèmes géo-ambiental. Quand le sol est contaminé par des agents chimiques, la distribution granulométrique, la structure, la compressibilité et son conduite concernant la tension - extension sont affectées. Cependant, les modèles classiques de la mécanique des sols ne comportent pas encore convenablement ces situations. On propose dans cette étude l'analyse de la conduite du sol résiduel granitique quand on mélange du chaux et du huile de graissage usé et aussi quand le sol est exposé aux conditions de saturation avec composants BETX (essence). Ces concepts peuvent développer l'usage et connaissance de ce genre de sol quand il est contamine au niveau de la génie geo-technique.

Keywords: soil mechanics, soil-structure interaction, contaminated land

INTRODUCTION

Urban and industrial development, the use of soils in various engineering works and the increase of contamination leads to the availability of soils decreasing. The re-use of contaminated soils for building and construction calls for knowledge of their mechanical behaviour and their physical, chemical and mechanical improvement. There is a trend to improve the use of contaminated soil or else to re-use it in embankments or as dressings for embankments or in foundations or sub-foundations for roads.

Aiming at a better knowledge of the affinity between the chemical agent and the granitic residual soil, several proportions of lime and lubricating oil were blended. This process leads to a stiffness increase by developing potential links between particles. After knowing the inherent behaviour of the granitic residual soil, the artificial soils obtained were compared as to its mechanical behaviour. The concentration of components was chosen so that an exothermic reaction occurs to correct pH and neutralize heavy metals contained in the lubricating oil (Meegoda *et al.* 1996).

Degradation has its potential origin also in the underground storage tanks of petrol (gasoline), which are very common in developed countries as well as in developing countries. These storage tanks are located at petrol stations, fuels storage areas, industrial or service areas. Some of them being very degraded due to their old age. The loss of fuels takes place at the very refuelling terminals, where possibly infiltration occurs near to the foundations of the local structures, blended with rain-water and clean up water. In Portugal, more than 1250 contaminated places may exist among the existing 5000 storage tanks (Celeste Jorge 2000). These kinds of substances, which permeate into the soil or into discontinuities filled with weathered rock may affect the environment and also the mechanical behaviour of these various materials.

To know the mechanical behaviour of the granitic residual soil when subject to petrol (gasoline) and drying saturation circumstances, this soil was remoulded at maximum dry density and optimum water content. After measuring its intrinsic behaviour, the soil samples were saturated with petrol for 6 months. Afterwards, tests of compressibility and resistance were made.

NATURAL AND CONTAMINATED SOIL CLASSIFICATION

The soil, which in the present research serves as matrix to the mixture of wasted lubricating oil and lime and petrol saturation, is a granitic saprolitic soil, which basically results from the alteration of feldspar through kaolinization. The granite belongs to a calc-alkaline group and from the mineralogical point of view it contains two types of mica where biotite is predominant. The rock texture is porphyritic with mega crystals of potassium-sodic feldspar (Andrade Pais 2002).

Four groups of samples were obtained: i) Granitic natural residual soil – NS; ii) Granitic natural residual soil with various proportions of mixture, artificial soil – M5 to M20; iii) Granitic natural residual soil with 5% of waste lubricating oil – OS5 (the 5% content of oil was chosen to be added to the soil because its lubricating effect goes to high values); and iv) Granitic residual soil with petrol – NSG. Grain size characteristics are presented at Table 1 and Figure 1.

Table 1. Grain size characteristics of tested soils

Sample	Effective size-D_{10} (mm)	Liquid limit-w_L (%)	Plasticity index - IP (%)	Clay activity - At	Classification
NS	0,006-0,13	40,4-42,5	1,7-5,6	very low	SW-SM with G
M5	0,20	43,0	4,6	-	SW-SP with G
M10	0,19	44,0	11,6	-	SW-SP with G
M15	0,20	44,0	10,8	-	SW-SP with G
M20	0,20	46,5	12,7	-	SW-SP with G
OS5	0,10	30,50	11,50	-	SW with G
NSG	0,007	46	10	-	SW-SM with G

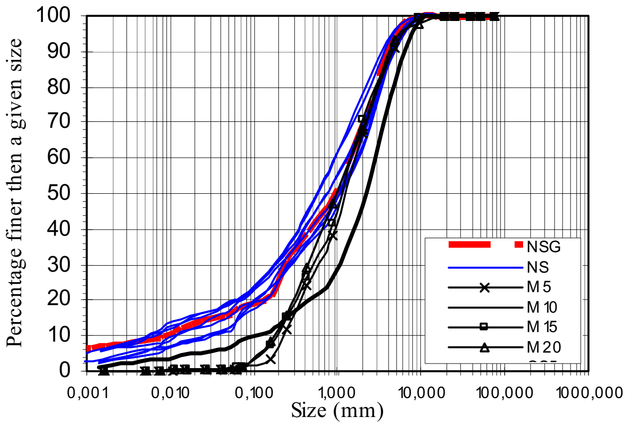


Figure 1. Grain size curves of tested soils

The classified granite residual soil belongs to group SW-SM with gravel. Clay activity is normal to low, revealing the presence of kaolinite, a low expansion clay. Liquid limit and low plasticity index, reflect the presence of mica minerals, retaining water in internal cleavage. The classified granitic soil mixture belongs to group SW-SP with gravel and the soil with gasoline is SW-SM with gravel, (ASTM D2487 1985).

The comparative granulometric curves of natural soil and M5-20 soils show that the addition of the mixture changes the particles original dimension a great deal. The thin particles of soil agglutinate themselves due to the lime and the oil effect, making flakes of bigger dimensions with lime hydrate nuclei. Results of chemical analysis of the soil did not indicate the presence of any metals or organic chemicals with concentrations exceeding the minimum allowable concentration levels for hazardous waste classification (Fabrin & Wagner 2002). As far as NSG soil is concerned, with petrol the granulometric curve is similar to the one for NS soils, with the fines percentage slightly increasing.

INTRINSIC BEHAVIOUR OF GRANITIC RESIDUAL SOIL

The granitic residual soil was disaggregated with the fingers and remoulded with a water content close to the liquid limit in a soft state and tested in the oedometer in an axis-symmetric condition with null radial deformation. Therefore, it was possible to define the intrinsic compression line, in a K_0 condition, with a compression index of C_c = 0,14. The intrinsic compression may be defined through the following equation: [$e = 0{,}7187 - 0{,}0458 \cdot \ln \sigma'_v$]. The critical state line in e-logσ'_v space was determined by estimating the final voids ratio after great deformations of the tests made in the direct shear test. The critical state lines in (e - log σ'_v) space and in (q-p') space are illustrated in Figures 1a and 1b, respectively.

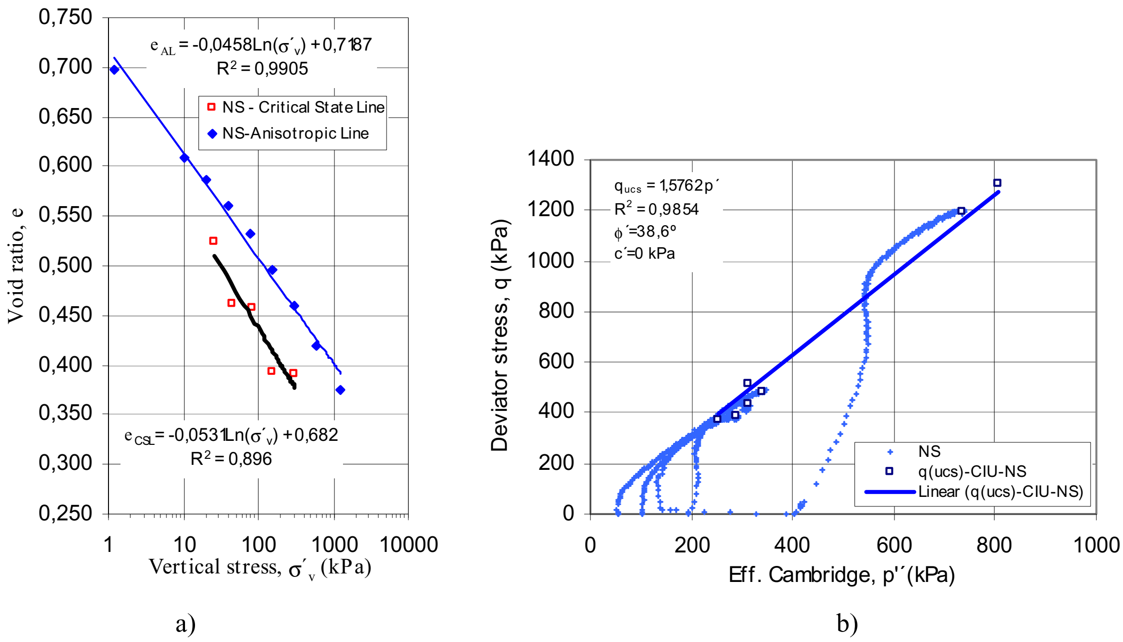


Figure 2. Line of the anisotropic behaviour and the critical state line of the granitic residual soil from Covilhã: a) in (e - log $\sigma`_v$) space; b) in (q – p') space.

MECHANICAL BEHAVIOUR OF THE CONTAMINATED SOIL

Compressibility behaviour

Oedometric trials of axis-symmetric compression were made under a K_0 condition with null radial strain and saturated elements, by using samples of 63 mm diameter and 20 mm height for the M5-20 and NS soils and also of 100 mm diameter and 40 mm height for the NS and NSG soil.

In the soils artificially "cemented" (M5-20), the initial vertical effective stress will have to consider the yielding tension of the interparticle "cement" developed, which normally is low. The initial increment of 1.2 kPa and the maximum vertical effective stress of 3067 kPa were used in this work. The following increment was double the previous one during 24 hours.

The samples of NS, M5 to 20, OS5 and NSG soils were obtained from the compaction curves with maximum dry density and optimum water content. Their physical features are summarized on Table 2. From the one-dimensional consolidation testing an increase of virtual preconsolidation stress (σ'_p*) for the soils influenced by the contaminative agents used, is noticeable. As for the M5 to 20 soils, the increase of virtual preconsolidation stress has no relation with the proportion of the mixture used, Figure 2, and Table 2.

Table 2. Characteristics of samples and one-dimensional consolidation test results of soil

Sample	Inicial void ratio -e_0	Dry unit weight - γ_d (kN/m^3)	Virtual preconsolidacion stress -σ'_p* (kPa)
NS	0,471	17,9	110-140
OS5	0,434	18,2	120
M 5	0,567	16,3	190
M 10	0,468	17,2	200
M 15	0,521	16,2	210
M 20	0,530	15,9	200
NSG	0,425	18,4	170

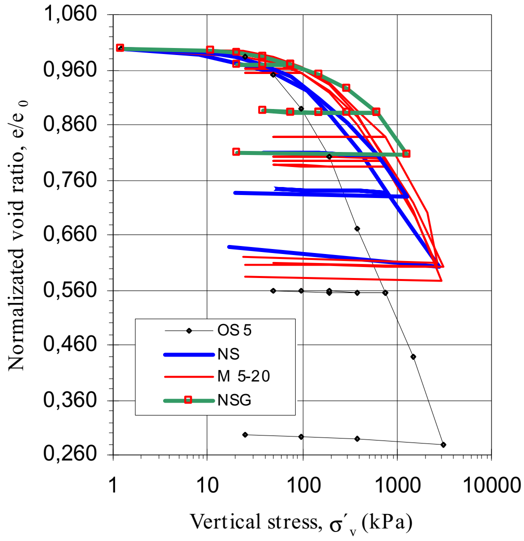


Figure 3. One-dimensional consolidation test results of natural (NS), artificial soil (M5-20) and soil with petrol (NSG)

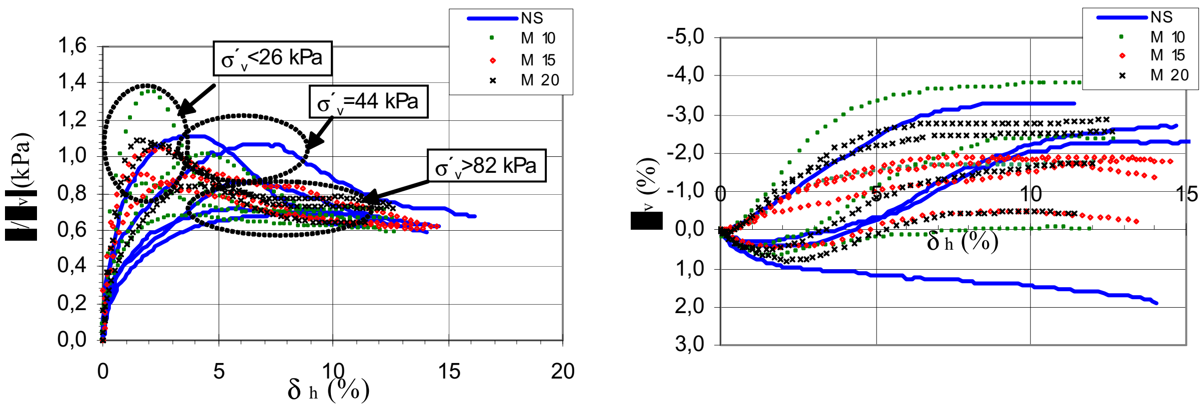
The artificial soils show higher values of compression index (C_c) and for stresses below σ'_p* than the NS soil, which means an improvement in stiffness. The OS5 soil shows an increase of compressibility and a decrease of σ'_p* due to the pure oil lubricating action.

In Figure 3, we can even see the space void ratio-effective vertical stress log (e-log σ' v) to have a tendency to become linear for high stresses (higher than the 3.1 MPa achieved), supported by the C_c variation to σ'_v. In samples with lower initial void index, the C_c value is only converging for high stresses due to the breaking of particles and also due to the great number of interpeculiar contacts (Coop, Atkinson & Taylor 1995). It is therefore convenient to define two different spaces on the index field of effective stress-voids: the space defined by the line which determines a possible loose state of casing for a remoulded soil and the space beyond that line where the soil can only exist due to its original or raised structure. Behaviour of the soil artificially structured is placed on the right side of the intrinsic behaviour curve, which emphasizes the meta-stable behaviour. For remoulded natural soils and the same vertical effective stress, the structured material can coexist with void indices as high as possible beyond the border of the stable space (Vaughan & Kwan 1984).

Shear strain behaviour

Some trials using the direct shear box were made, which were drained and consolidated according to the following increments to each sample with the vertical effective stresses of: σ'_v = 26; 44; 82 and 157 kPa. The shear box used has circular sections with 100 mm diameter and a sample height of 40 mm. The samples tested at resistance were obtained from the maximum dry unit weight and optimum water content of M10-M20 soils with no restoration time.

On the M10 to 20 soils, initially there is an increase of strength due to the ionic exchange and consequent flocculation of particles, where even the clays may take on the typical behaviour of a granular soil. The value of effective cohesion (c´) decreases and the peak friction angle (ϕ'_p) increases in accordance with the increase of mixture proportion on the soil (Figure 4).



a) b)

Figure 4. Results of direct shear tests of soil NS and soil contaminated with oil and lime: a) behaviour of normalized shear strength curves (τ/σ'_v) versus horizontal displacement (δ_h); b) vertical displacement curves (δ_v) versus horizontal displacement (δ_h).

The mixture of lime and lubricating oil used leads to a stiffness increase due to the potential connections development between particles. The addition of lime into the soil produces ionic exchange mechanisms where the Ca^{++} cation replaces the lower valency cations, giving rise to flocculation of fine particles, thus gaining some resistance and consequently improving the properties of the resulting soil (Cristelo & Jalali 2004).

The possible effective cohesion (c´) coming from the interpeculiar connections becomes hidden due to the dilating phenomenon, which basically controls behaviour, presupposing the rate between dilatancy angle (ψ) increase and c' decrease, Table 3.

Table 3. Physical and mechanical parameters obtained through direct shear test

Sample	Inicial void ratio -e_0	Dry unit weight - γ_d (kN/m3)	Failure criterion τ_{max}	
			ϕ'	c´ (kPa)
NS	0,444-0,459	17,57-18,35	41	23
M10	0,503-0513	16,92-17,14	38	26
M15	0,660-0,694	15,19-16,39	44	2
M20	0,604-0,609	15,16-15,56	47	10

A conventional triaxial test was used for the study of stress-strain behaviour of NS and GNS soils for samples of 100 mm diameter and 200 mm height obtained from the maximum dry unit weight and optimum water content of all soils. Drainage was allowed at the bottom and the top of the samples. The samples were saturated with water through a backpressure system, isotropically consolidated and cut in pressure according to the conventional courses from the constant main stress ($_3$) and under undrained condition (CIU). Effective stresses of isotropic consolidation were used for the set of samples of p'_c = 50; 100; 200 and 400 kPa.

The stress-strain-resistance behaviour of the soils is described by a set of parameters, which normally are defined at the very moment of failure or at other moments related to failure. Normally, different criteria are used depending on the kind of problem to be solved. The criteria used correspond to the maximum shear stress (q_{max}) and the shear stress corresponding to the critical/stable state (q_{cs}). The parameters of strength at failure were obtained through the involving linearization and are illustrated on Table 4.

Table 4. Physical and mechanical parameters obtained through CIU trials

Sample	Inicial void ratio -e_0	Dry unit weight - γ_d (kN/m3)	Failure criterion			
			q_{max}		q_{cs}	
			ϕ'	c´ (kPa)	ϕ'	c´ (kPa)
NS	0,386-0,402	18,5-19,0	39,0	0	38,6	0
NSG	0,376-0,395	18,8-19,1	38,4	17,3	35,7	0

With overconsolidated soils, or similar ones, great deformations are demanded to mobilize the maximum value of deviator stress, and so the corresponding points may not correspond to a sole involving element. After analysing Figure 5, we can see that the lines properly define surfaces at limit state in both cases. We can see that the failure involving the soil NSG and p'_c < 200 kPa stays clearly above the corresponding one for the NS sample. Probably this fact is connected to its state of structure (bonding) developed in the fines of the soil in question due to the action of petrol.

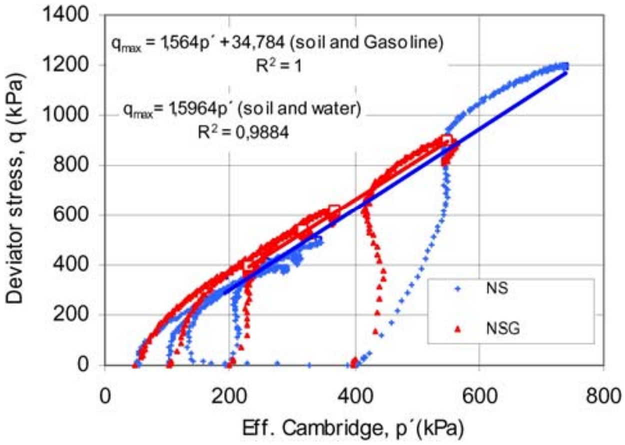


Figure 5. Undrained paths in q:p´ space for the NS and NSG soil.

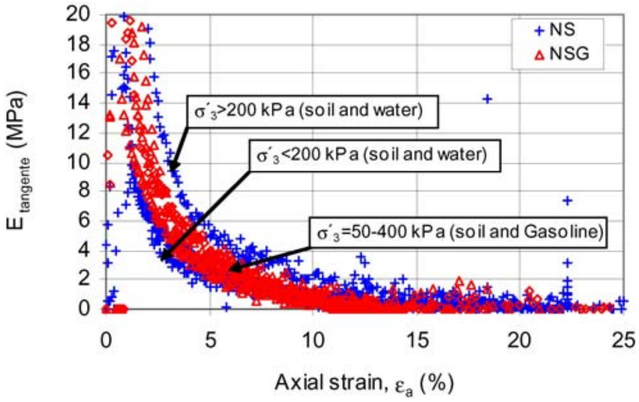


Figure 6. Change rate of Young´s modulus of elasticity extended to NS and NSG soils

The presence of petrol correlates with the maximum dilating tendency shown by the GNS sample during the cut. Regarding the samples subject to a higher confining pressure there is a reduced dilating tendency, even if they show a strong tendency to dilating after failure. The effect of both components, interparticle connections and densest state of packing take on important rôles in the mechanical behaviour during the cut, and so discussion on the maximum resistance concerning the point of higher dilatancy rate must take into consideration this reality and, if possible, by separating the effects.

As for the NSG soils, stiffness for the different samples is greater when p'_c <200 kPa but smaller for higher consolidation pressures by expected failure of the connections established, prevailing the lubricating rôle of petrol between the grains from there (Figure 6).

At the last state NSG loses mechanical properties through ϕ'_{cs} reduction and cohesion vanishing, which may cause problems for foundations in the long term or for the stability of embankments in these conditions.

CONCLUSIONS

1. The granitic residual soil used belongs to group SW-SM with G. The classified granitic soil mixture belongs to group SW-SP with gravel. The comparative granulometric curves of natural soil and M5-20 soils show that the addition of the mixture changes the particles original dimension a great deal. The thin particles of soil agglutinate themselves due to the effect of the presence of lime and oil. The same soil mixed with petrol does not undergo an expressive evolution regarding its granulometric distribution.
2. From the one dimensional consolidation testing an increase of virtual preconsolidation stress (σ'_p*) is noticeable as the percentage of mixture is increased or when the remoulded granitic residual soil is saturated with petrol. The comparative compression index (C_c) exhibits increased difficulty on stabilization for artificial soils. The soils contaminated for stresses below σ'_p* show higher C_c values than the NS and OS5 soils, which means a slight improvement on stiffness.
3. As far as M5-20 soils are concerned, the dilatancy angle (ψ) group is greater than for the natural soil used, which means that the mechanical behaviour of these soils is led by the dilatancy angle, what explains the peak friction angle (ϕ_p). The possible effective cohesion coming from the interpeculiar connections becomes hidden due to the dilating phenomenon which especially makes the control of behaviour, presupposing the relationship between the ψ increase and the c´ decrease, for the stress levels studied.
4. Finally, we have to point out that the use of lime and lubricating oil causes a slight improvement in the mechanical behaviour, making them more resistant and less compressible for stress levels below σ'_p*, even improving with time of cure. Regarding the NSG soil, there is an improvement on resistance and stiffness for consolidation

tensions p'_c<200 kPa due to the resulting interparticle connections, developing in the field of fines and reflected in the cohesion developed. However it must be emphasized that there is a loss of mechanical qualities for large deformations.

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