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Chemical stabilization of tropical soils: shear strength and elastic properties characterization using triaxial test

Abstract. This paper addresses the mechanical characterization of soils of the *Zona da Mata* of Minas Gerais State, Brazil, and their mixtures with a chemical stabilizer for geotechnical highway applications using non consolidated-non drained triaxial tests. Materials and methods are as follows: (i) soils: one mature and two young gneiss residual soils; (ii) chemical stabilizer: RBI Grade 81 at the contents of 2%, 4% and 6% related to soils dry weights; (iii) soils and mixtures specimens compacted at the standard, intermediate and modified Proctor compaction efforts; (iv) mixtures specimens cured during seven days in a conditioning chamber. Conclusions of the study are as follows: (i) addition of the stabilizer to the soils and increase in its content, as well as in the compaction effort led to significant gains in the soils shear strength, mainly regarding the intercept cohesion parameter; (ii) the effect of the confining pressure in the shear strength of the tested mixtures was progressively reduced with increases in the stabilizer content and in the compaction effort; (iii) addition of the stabilizer to the tested soils probably generated internal mechanisms of cementation among the soil particles that produced more rigid soil mixtures with fragile rupture behavior, affecting significantly the soils deformability parameters.

Keywords: Tropical soils; Chemical stabilization; RBI Grade 81 chemical stabilizer; Shear strength; Geotechnical and highway applications.

6.1. Introduction

Brazil is the largest country in Latin America, covering almost half (47.3%) of the South American continent, with an area of 8,547,403 km^2. It is the fifth largest country in the world after the Russian Federation, Canada, China and United States.

Because of Brazil's territorial extension, implanting its transport system has been an enormous challenge over the years.

Since the beginning of the 1970s, the Brazilian government has prioritized funds for roadway construction and maintenance, which transport about 85% of the Brazilian population and products (BRAZIL, 2005). Even with this effort, the roadway network in Brazil is still not considered satisfactory, bearing in mind (REZENDE, 2003): (i) the need to pave more roadways; (ii) a great part of this network has serious conservation and maintenance problems; and (iii) the existence of several regions without transport roads.

Rezende (2003) reported that at the start of roadway construction in Brazil, project specifications and methodologies were adopted that existed in other countries so that at first granular materials were used to execute base layers and sub-base layers on a large scale and, as consequently, these materials are not easily found today. In Brazil, recent research comes being carried through to analyze the viability of using fine local soils and rejects including slag, fly ash, civil construction rubble, residues or byproducts from the paper and cellulose industries and milled asphalt as materials to form roadway paving layers. These studies refer to the so-called economic or low cost paving’s, that with the use of the materials mentioned have reduced construction costs satisfactorily, and the environmental impacts generated by exploitation of quarries. When these materials do not present satisfactory structural performance, one of the recommended solutions is the use of chemical stabilization techniques that aim to obtain an end product that meets the projected specific requirements.

The chemical stabilizers that have been proven to improve the physical and mechanical properties of a soil include Portland cement and lime (THOMPSON, 1966, 1969; Transportation Research Board - TRB, 1976; LIMA, 1981; FERRAZ et al., 1991; FERRAZ, 1994; ALCÂNTARA, 1995; MENDONÇA et al., 2000; SCHNAID et al., 2001; TRINDADE et al., 2003). However recently another product, RBI Grade 81, has been used successfully in chemical stabilization of soils on the European and African continents and it is the object of study in the present investigation.

As Thompson (1966) pointed out, the shear strength parameters of stabilized mixtures are essential to rational analysis and dimensioning of the structure of flexible pavements, also including in the spectrum the semi-rigid pavements dimensioned according to the dimensioning method of the Brazilian National Department of

Highways (DNER, 1996), currently the Brazilian National Department of Transport Infrastructure (DNIT). In this sense, the present study was carried out to widen the Brazilian databank in the characteristics of resistance to shearing of soils stabilized chemically for geotechnical purposes, especially for roadway applications. For this, the mixtures of three residual gneiss soils typical of the *Zona da Mata* of Minas Gerais State, Brazil, and the RBI Grade 81 chemical stabilizer were characterized technologically in order to analyze the influence of the compaction effort and stabilizer content on shear strength and elastic properties of the mixtures, represented by data from unconsolidated-undrained triaxial tests.

6.2. Soil stabilization

6.2.1. Background

According to Lima (1981) the stabilization of a soil is any artificial modification introduced in its performance to permit its use in engineering works, acquiring a quantitative character through parameters inherent to determined project criteria, such as shear strength, deformation under load action, moisture absorption. This author also stated that when dealing with roads, stabilizing a soil means giving it the capacity to resist natural erosive actions and efforts and wear induced by traffic under the most adverse conditions considered in the project. In this context, Winterkorn (1955) summarized the stabilization process in: (i) determination of soil strength and complimentary strength necessary for the determined engineering purpose; (ii) transformation of this complimentary strength required in materials and construction processes to be used, based on technical, economical or special criteria; and (iii) consideration of the durability of the work in the project period.

Silva (1968) emphasized that when attempting to order the soil stabilization techniques there is a problem related to the disparities and similarities of the means and mechanisms used, although it is of practical interest to adopt a classification criteria. From this point of view, this author refers to the following soil stabilization modalities: mechanical, chemical, electrical and thermal. Of these, those of greatest interest in roadway construction are the mechanical and the chemical, and the latter is the technique of greatest interest in the present study.

Chemical stabilization of a soil refers to the alterations produced in its mass by introducing a small quantity of chemical additive, which gives it certain engineering properties pre-established in the project. According to Silva (1968), the chemical soil stabilization can be processed through four different mechanisms: (i) substitution of water molecules and cations adsorbed by water repellent cations; (ii) establishing reinforced links among particle aggregates by adding linkers; (iii) flocculation; and (iv) dispersion. Lima et al. (1993a) reported that it is common to include chemical additives for roadway purposes that produce better structuring in the soil mass, such as hydrated lime and Portland cement. In this context, other products have also been used tentatively, such as lignin, sugar cane waste, wood tar, Kraft black liquor, lime mud, grits and DS-328, but the practical results have not yet permitted a precise definition of their potentialities (LIMA et al. 2003).

With the objective of supplying a theoretical framework for the soil chemical stabilization techniques commonly used in roadway construction in Brazil, the subsequent items present a brief reference to the use of Portland cement and lime and a synthesis of the main characteristics of RBI Grade 81.

6.2.2. Soil-cement stabilization

In terms of the historical use of soil-cement, the English engineer H. E. Brook-Bradley was pioneer according to the Portland Cement Association (PCA, 1960), and the precursor of this mixture, when it was used in treating road beds and tracks for animal traction vehicles in southern England in 1891.

In Brazil, the first works executed with a soil-cement date from 1939. According to Dama and Miranda (1996), the first experiment in soil-cement in the Brazilian roadway network was the paving of the Rio-Bahia Highway with about 1800 km of soil-cement base. Regarding research, the Brazilian Portland Cement Association (ABCP) began studies with soil-cement mixtures in 1941, extending this research not only to the area of roadway engineering but also to sectors of Brazilian civil construction.

Regarding the soil-cement doses, it can be stated that this consists in performing laboratory experiments with a certain soil type and variable ratios of soil and Portland cement and water and in the interpretation of the results of these experiments by technical economic criteria established in function of the experience. The result is the definition of the soil, Portland cement and water quantities that guarantee the mixture the minimum characteristics required of resistance and durability (LIMA et al. 1993a).

The Brazilian experience in using base and sub-base layers of soil-cement is based on the design methods of the Portland Cement Association, with adaptations by the Brazilian Portland Cement Association (ABCP, 1986), and in the technical standard NBR 12253 of the Brazilian Technical Standards Society (ABNT, 1992).

According to Lima et al. (1993a), the effect produced by Portland cement can be seen in two distinct ways, whether referring to granular or cohesive soils. In the first, the additive is destined mainly to create links in the intergranular contacts to guarantee more effective strength of the material to external solicitations, by increase in the strength part relative to cohesion; in the case of fine soils, the Portland cement grains perform as a nucleus to which the surrounding tiny particles of the soil adhere, forming regions of flocculated materials that present links derived from the cementation phenomena. The soil-cement doses are processed by the study of the soil by characterization studies, and from the mixture by unconfined compression tests. Thus every soil can be stabilized with Portland cement; however, it is a basic rule that the cement quantity used increases with the quantity of clay that technically and economically makes sandy soils more favorable to this form of stabilization.

The Brazilian National Department of Highways (DNER, 1996), presents a distinction between the terms soil-cement stabilization and soil improved with cement. The first term refers to a duly compacted mixture of soil, Portland cement (contents commonly in the 6% to 10% range) and water, and the mixtures should meet certain density, durability and mechanical strength requirements, giving as a result a cemented and highly tensile strength material. The second is a modality that is obtained by adding small quantities of Portland cement (2% to 4%) to primarily modify soil regarding its plasticity and sensitivity to water, without accentuated cementation, and the mixtures are considered as layers with flexible performance.

Some considerations regarding the chemical and mineralogical constitution of the soils should be considered in soil-cement stabilization. One of them is the presence of sulfates, because, according to Petrucci (1978) reactions may occur of these substances with calcium hydroxide and free aluminum, giving rise to calcium sulfoaluminate with expansive performance. Another is the presence of organic matter, considered by this author as a negative factor for occurrence of Portland cement hydration reactions. The third consideration, according to Alcântara (1995), concerns soils with fine fraction consisting of montmorilonitic clay that, in the stabilization process with Portland cement, may lead to the disaggregation of the mixture because of the occurrence of significant expansion of the clay minerals when in the presence of water.

6.2.3. Soil-lime stabilization

Lime has been used for more than 5000 years and it is believed that its use in road stabilization originated more than 2000 years ago with the Romans (LIMA, 1981). The Romans also used a cementation material, pozzolan, in the search for better mechanical strength characteristics of the soil and lime mixtures. Pozzolan is a silicic material that alone, presents little or no agglutination capacity. However, it reacts with lime and water to form cementation compounds, capable of agglutinating soil particles.

Generally, all the soils with fine particle size react with lime (LIMA et al., 1993b), and cationic exchanges and flocculation’s occur between these and lime that benefit their workability, plasticity and expansive characteristics. However, according to these authors, pozzalanic reactions among the soils and lime do not always occur and are influenced by the natural property of these materials, by the types and quantity of lime used, by the cure period conditions and by the compacting characteristics of the mixtures.

Regarding soil-lime stabilization mechanisms, it is known that the cation exchange reactions and flocculation are processed rapidly and produce immediate alterations in plasticity and, on a smaller scale, in shear strength of the mixture (LIMA et al., 1993b). Pozzolanic reactions can occur between the soil and lime, depending on the characteristics of the soils to be stabilized. These reactions result in the formation of several cementation compounds that increase mixture strength and durability and develop in the long-term, as reported by Diamond and Kinter (1965), Guimarães (1971), TRB (1976) and Lima (1981).

Lima et al. (1993b) pointed out that soil lime stabilization in roads involves three service modalities, namely modification of the soil by lime action; stabilization of the subgrade; and stabilization of the base and sub-base layers and reinforcement of the roadway pavement. According to these authors, the first functions are a constructive expedient in regions where the soils are naturally excessively wet for use in roadway construction; the second includes generally the in situ improvement of engineering properties of soils of the subgrade; and the third includes improvement in the excavated material engineering properties of soils that are reactive to lime action. Regarding experiments for design purposes, there are, for the soils that show appreciable strength gain with lime addition, unconfined compression of samples molded in the standard Proctor compaction effort; for situations when workability improvement is required, followed by small strength gains, unconfined compression and California bearing ratio (CBR) tests are used.

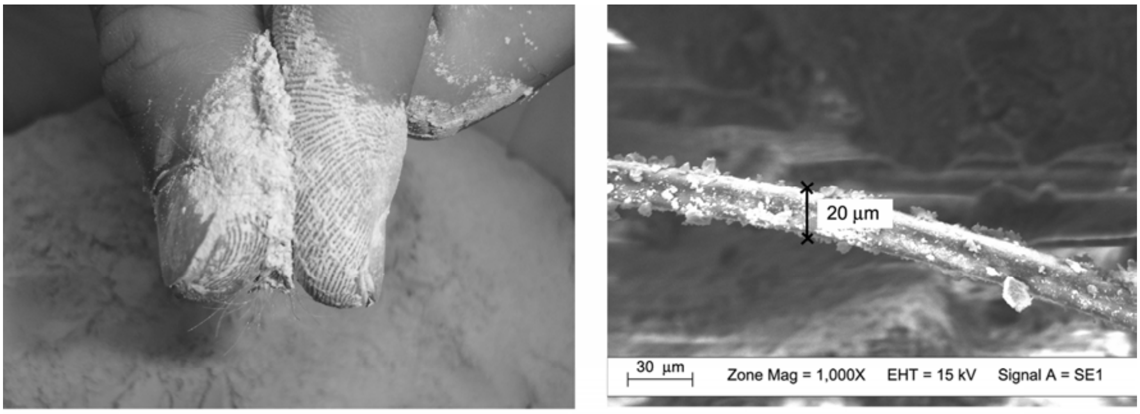
The characteristics of compaction, plasticity, volume variation and mechanical strength of a soil are influenced by lime addition as follows: the optimum soil moisture content increases with the lime content and the maximum dry unit weight decreases (TRB, 1976; CARVALHO et al., 1981; LIMA et al., 1993b); Castro and Serafim Luis (1974) and Osula (1996) reported reductions in the liquidity limit and increases in the plasticity limit; volume variations through water absorption and contractions by drying are, normally, reduced by treatments of soils with lime (MENDONÇA, 1998); and mechanical strength of reactive soils stabilized with lime, after compaction and suitable cure period, reach values greatly superior to those of compacted soils (TRB, 1976; LIMA, 1981; ALCÂNTARA, 1995; OSULA, 1996; MENDONÇA, 1998).

Similarly to the soil-cement mixtures, the Brazilian National Department of Highways uses the terminologies soil stabilized with lime and soil improved with lime (DNER, 1996). The first designation is for so-called non-reactive soils where cationic exchange reactions and carbonation reactions predominate from the addition of lime and the mixtures are considered as flexible layers; the second is applied to so-called reactive soils, where lime addition gives rise specifically to pozzolanic reactions, and the mixtures are considered as semi-rigid layers.

6.2.4. Soil-RBI Grade 81 stabilizer mixtures

RBI Grade 81 is a chemical stabilizer developed in South Africa at the beginning of the 1990s for roadway application (TECFLORA, 2001). This product is currently manufactured in Israel and Canada by the Anyway Solid Environmental Solutions Ltd. Kerem Maharal Corporation and commercialized in the form of grains, in 25 kg packs and in one ton containers that can be stored for up to 12 months. Until the end of 2003, the product was commercialized in Brazil by the TECFLORA S.A. Company (Advanced Forest Technology S.A.), head office in the city of Viçosa, Minas Gerais State, who kindly supplied part of the samples used in the present study.

According to the manufacture's information, the product consists of inorganic substances that act on the soils to produce strength to weathering action and are stable under load action and relatively waterproof (ANYWAY, 2002). It consists of gray colored odorless grains (Figure 6.1a) not self inflammable, with 12.5 pH (in the form of saturated paste) and 25 kN/m^3 grain unit weight. The main chemical elements forming the product and their respective variation range in terms of percentage are: Ca (25-45%), Si (5-20%), S (5-15%), Mg (0-10%), Fe (0-5%), K (0-5%), Al (0-5%) and Mn (0-2%) (ANYWAY, 2004). The stabilizer also presents approximately 2 cm long and 20 μm diameter natural microfibers (Figure 6.1b), randomly distributed in its mass, that create greater tensile strength and mixture flexibility, minimizing the occurrence of cracks in the stabilized pavement layer (ANYWAY, 2002).



(a) (b)

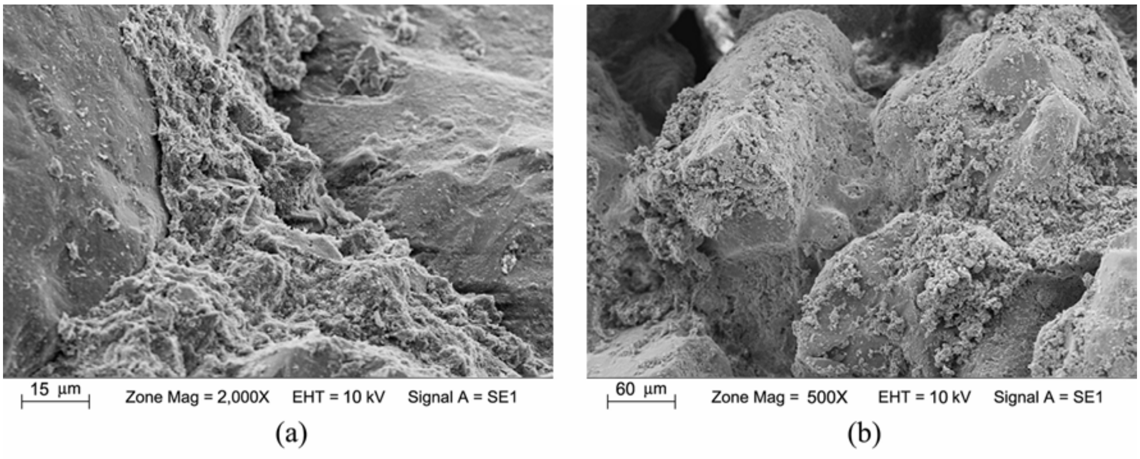
Figure 6.1. (a) photograph showing RBI Grade 81 grains; and (b) details of the microfibers present in chemical stabilizer mass obtained via secondary electrons signs produced in scanning electronic microscope (SEM).

It is known that the percentage of chemical additive to be used in the stabilization of a soil for geotechnical roadway purposes depends on several factors, especially on the type of soil to be stabilized, the function of the stabilized layer in the road pavement and the type and volume of traffic solicitation. However, regarding RBI Grade 81, the application field includes, without distinction, soils ranging from sandy to highly plastic clay (ANYWAY, 2001) and this is one of its most striking characteristics.

Regarding the evolution of mechanical strength of the RBI Grade 81 soil mixtures over time, technical information from the manufacturer states that at the end of the first 24 hours there is a substantial quantitative in the chemical reactions, that reach, at the end of seven days, after mixing and compacting, 90% and can extend for one year, and this evolution depends on the type of soil to be stabilized (ANYWAY, 2002). The mixture project, for purposes of use as layers of highway pavement, is based on the results of the CBR and unconfined compression tests for a seven-day cure time (ANYWAY, 2003). Regarding accompanying the performance of the mixture in the field, it is common to recommend specimen collection for laboratory experiments after the 28 day cure time (ANYWAY, 2003).

From the structural point of view, previous studies have suggested that the product acts as soil particle linkers, creating an interparticle cementation nuclei through hydration and pozzolanic reactions that are responsible for the transformation of the stabilized mass to a product with greater strength and rigidity (ANYWAY, 2001), thus including the potentialities of the traditional stabilizers lime and Portland cement. The cementation of the particles produced by the stabilizing agent is clearly shown in Figure 6.2 that presents images obtained from the results of SEM analysis, carried out on two soils with different textures (clayey and sandy soils).

The constructive procedures to apply the product in the field are common to other chemical stabilizers, such as Portland cement and hydrated lime, involving operations of scarification of the soil in the field, applying the product and mixture of the stabilized soil mass, application of the quantity of water to take the mixture to the field compaction moisture content established in the design, new mixing and compaction. According to the manufacturer (ANYWAY, 2003), the stabilized layer can be open to traffic shortly after compaction, which functions as a compaction agent responsible for new mechanical strength gains by the mixture.



(a) (b)

Figure 6.2. Images of soil-RBI Grade 81 samples obtained by secondary electrons signs produced in SEM: (a) clayey soil of high plasticity; and (b) sandy soil no plastic.

6.3. Experimental program

6.3.1. Materials

The present study was carried out on three soils collected in the city of Viçosa, located in the *Zona da Mata* of Minas Gerais State, Brazil, using a mature gneiss residual soil, pedologically classified as a Red-Yellow Latosol (soil 1) and two young gneiss residual soils (soils 2 and 3).

Soil 1 is widely distributed in Brazil and presents a yellow colored latosol B horizon, strong granular structure and individualized blocks, with a fairly porous aspect. It has a clay-sand-silt texture (61% clay, 14% silt and 25% sand), with 74% LL and 39% IP. It’s classified as A-7-5 (20) according to the soil classification system of the Transportation Research Board (TRB). The soil sample was collected from a cutting slope located on the right side of the highway that links the cities of Viçosa and Paula Cândido, close to the Viçosa cold mixed plant, between 4.60 and 5.20 meters counted from the top of the slope.

Soil 2 is from saprolytic layer, C horizon, abundant in a large area of Southeast Brazil, thick and presenting clay minerals of the mica group. The TBR classification is A-6 (1) and it presents a sand-silt-clay texture (15% clay, 22% silt and 63% sand), 38% LL and 13% IP. The soil samples were collected from a cutting slope situated on the left side of the roadway that links the cities of Viçosa and Paula Cândido, in front of the entrance to the Romão dos Reis neighborhood in the city of Viçosa.

Soil 3 came from a weathered profile of soils developed from pre-Cambrian gneiss and the saprolytic layer, classified by the TRB as A-2-4 (0). It's gray in color with sand-silt-clay particle size (13% clay, 17% silt and 70% sand), 29% LL and 9% IP. The sample collection location was a cutting slope located in the Vila Secundino, on the campus of the Federal University of Viçosa (UFV), in Viçosa. This soil was chosen because it complies with the Brazilian highway engineering standard design requirement for soil-cement mixtures.

The RBI Grade 81 stabilizer was used at amounts of 2%, 4% and 6% in relation to the dry soil weight.

6.3.2. Methods

6.3.2.1. Soils sampling

The samples were collected with manual tools in slopes cutting, placed in 50 kg plastic bags and identified by labels containing the soil origin data. Next, these bags were then taken to the Civil Engineering Laboratory (LEC) of the Department of Civil Engineering (DEC) at the Federal University of Viçosa (UFV), where they were air dried, the clumps removed, sieved through a 4.8 mm nominal opening and placed in plastic bags for later use. The samples were prepared for the laboratory tests according to the recommendations of the Brazilian Technical Standards Association (ABNT, 1986a).

6.3.2.2. Compaction tests

The compaction tests were carried out at the compaction efforts of the standard, intermediate and modified Proctor tests, according to the methodology recommended by ABNT (1986b), to determine the optimum moisture content (w_{opt}) and the maximum dry unit weight (γ_{dmax}) of the soils and the respective mixtures, with compaction of the mixtures between two and four hours after stabilizer addition, following the procedures common to soil-cement and soil- lime mixtures.

The specimens were compacted by dynamic process according to specifications presented in Table 6.1 using the Proctor compaction test mold, that is, a 10 cm diameter and 12.73 cm tall cylindrical mold (about 1000 cm^3). It should be emphasized that the Brazilian standard and modified Proctor compaction tests are similar to the AASHTO standard and modified compaction tests.

Table 6.1. Compaction laboratory test parameters according to the Brazilian Technical Standards Association (ABNT, 1986b)

Laboratory compaction test parameters	Applied compaction effort		
	Standard	Intermediate	Modified
Hammer	Small size (mass of 2.50 kg)	Large size (mass of 4.54 kg)	Large size (mass of 4.54 kg)
Number of layers	3	3	5
Number of blows per layer	26	21	27
Compaction effort (kN.m/m^3)	590	1290	2690

6.3.2.3. Mixtures specimens preparation for triaxial tests

The procedures used to prepare the soil-chemical additive mixtures followed the recommendations of the product manufacturer (ANYWAY, 2003), but a variation was adopted that consisted of considering the optimum times between mixing and compaction previously determined for the soils used in the present study in a research carried out by Trindade et al. (2005) based on best mixtures mechanical strength responses. In this study the authors carried out unconfined compression tests in specimens of soils 1, 2 and 3 stabilized with 4% RBI Grade 81, molded on the optimum compaction parameters (w_{opt} and γ_{dmax}) determined in the modified Proctor compaction test, adopting periods between mixing and compaction of 0, 4, 8 and 24 hours. The optimum times between mixing and compaction for each one of the soils were those associated to the highest unconfined compressive strength values: (i) four hours, for soils 1 and 2; and (ii) compaction immediately after mixing for soil 3.

The optimum times determined between mixing the soils with the additive and compaction of the mixtures specimens determined by Trindade et al. (2005) were used in the laboratory test program, where, to optimize the mechanical performances of the mixtures, the additive was added at contents of 2%, 4% and 6% compared to the dry soil weight, homogenized, water added to the stabilized mass with later homogenization, and the samples were placed in plastic bags, considering: (i) mixture specimen modeling in the optimum parameters (w_{opt} and γ_{dmax}) determined in the compaction efforts of the standard, intermediate and modified Proctor tests; (ii) mixture specimen cure in an acclimatized room under the conditions already described for a period of seven days; (iii) static unconsolidated-undrained triaxial tests.

6.3.2.4. Triaxial tests

The shear strength parameters of soils 1, 2 and 3 and their respective mixtures, after being submitted to a seven day cure time in a acclimatized chamber, were determined by carrying out unconsolidated-undrained triaxial tests on specimens molded in the optimum parameters(w_{opt} and γ_{dmax}) of the standard, intermediate and modified Proctor compaction efforts.

Initially specimens were molded identical to those used in CBR tests, that is, they were compacted by dynamic process in five equal layers using a 15.08 cm diameter and 17.78 cm tall cylindrical mold and a 6.35 cm high disk spacer, a 4.536 kg mass hammer dropping 45.72 cm (ABNT, 1987). To the standard, intermediate and modified Proctor compaction efforts were applied, respectively, 12, 26 and 55 blows. The limits for specimen acceptance were fixed as recommended by the Standard Method DNER-ME 202 of the Brazilian National Department of Highways (DNER, 1994): (i) maximum dry unit weight in the range $\gamma_{dmax} \pm 0.30$ kN/m^3 and optimum moisture content in the range $w_{opt} \pm 0.5\%$. Smaller (8.0 ± 0.2 cm tall by 3.6 ± 0.1 cm diameter) specimens were molded and used for the unconsolidated-undrained triaxial tests. Six specimens were prepared for each shear strength envelope determination, four to carry out the unconsolidated-undrained triaxial tests and two stored for possible use in case of problems during the tests.

The unconsolidated-undrained triaxial tests were carried out according to the procedures recommended by Head (1982). The specimens were submitted to 50, 100, 200 and 400 kN/m^2 confining stresses. These values were chosen because the influence of low confining stress is progressively reduced with the development of cementation reactions common in chemically stabilized mixtures (ALCÂNTARA, 1995; MENDONÇA et al., 2000). The tests were carried out until each specimen reached failure or until the axial strain reached values of 15%. The test displacement rate used was 0.5 mm/min. The axial displacements were measured using a linear variable differential transducer (LVDT) installed on the outside of the unconsolidated-undrained triaxial test chamber and the vertical axial force applied on the top of the sample by a piston was recorded by a 20 kN capacity load cell attached to it. The displacement and force readings were collected every two seconds by a data acquisition system that permitted a more realistic determination of the values of the secant deformation modulus at 1% axial strain ($Es_{1\%}$) in the soils and mixtures. The values of this parameter were presented at 1% axial strain because the data obtained were more reliable starting at this level of deformation. The values of the secant deformation modulus at the 50% failure stress level (E_{50}) were also determined. This last parameter has been commonly used to estimate the immediate settlement of shallow foundations (SOUTO SILVEIRA, 1967). It is pointed out that water pore-pressure measurements were not taken during the tests in the present study.

6.4. Results and analysis

6.4.1. Compaction parameters

Tables 6.2 to 6.4 show the values of the optimum compaction parameters of soils 1, 2 and 3 and their mixtures. It was observed that the optimal moisture content in the soils increased with growing quantities of chemical additive, for the compaction efforts used. The maximum dry unit weight decreased with increases in the contents of chemical additive. It was also observed that the increase in the compaction effort was responsible for increases in the maximum dry unit weight and reductions in the values of the optimum moisture content of the soils and their respective mixtures.

Concerning influence of RBI Grade 81 on the compaction parameters of tropical soils, there is scarce data available in the literature. One of the few references in this respect is a study by França (2003), who obtained similar results to those reported in the present study. Regarding the soils, this author mentioned lower maximum dry unit weight values and greater values of optimum moisture content of the mixtures with the increase in the additive content, and greater compaction efforts produced mixtures with higher maximum dry unit weights and lower optimum moisture contents, a performance that was confirmed by the results in the present research presented in Tables 6.2 to 6.4.

Table 6.2. Compaction test data: soil 1 and its mixtures

Amount of RBI Grade 81 (%)	Standard Proctor		Intermediate Proctor		Modified Proctor	
	γ_{dmax} (kN/m^3)	w_{opt} (%)	γ_{dmax} (kN/m^3)	w_{opt} (%)	γ_{dmax} (kN/m^3)	w_{opt} (%)
0	13.85	30.78	14.68	28.19	15.17	27.32
2	13.79	31.35	14.64	28.51	15.05	27.56
4	13.72	31.66	14.17	30.18	14.94	28.44
6	13.64	32.70	13.81	31.87	14.35	30.35

Table 6.3. Compaction test data: soil 2 and its mixtures

Amount of RBI Grade 81 (%)	Standard Proctor		Intermediate Proctor		Modified Proctor	
	γ_{dmax} (kN/m^3)	w_{opt} (%)	γ_{dmax} (kN/m^3)	w_{opt} (%)	γ_{dmax} (kN/m^3)	w_{opt} (%)
0	15.07	17.98	17.01	15.67	17.49	14.31
2	15.01	18.33	16.80	15.73	17.37	14.70
4	14.89	19.68	16.31	16.57	17.09	15.09
6	14.81	20.35	16.23	18.13	16.92	16.11

Table 6.4. Compaction test data: soil 3 and its mixtures

Amount of RBI Grade 81 (%)	Standard Proctor		Intermediate Proctor		Modified Proctor	
	γ_{dmax} (kN/m^3)	w_{opt} (%)	γ_{dmax} (kN/m^3)	w_{opt} (%)	γ_{dmax} (kN/m^3)	w_{opt} (%)
0	17.71	13.11	18.84	11.55	19.68	9.95
2	17.52	13.51	18.52	11.91	19.19	10.72
4	16.74	14.08	17.51	12.65	18.30	12.06
6	15.68	14.01	17.02	12.74	18.12	12.10

6.4.2. Shear strength and elastic properties

Regarding the stress-strain relationships, it can be stated that they are fundamental for the analysis of the structural performance of a roadway pavement that presents chemically stabilized soils in its composition. In general, the stress-strain behavior of cemented soils can be described as initially stiff, apparently linear up to a well defined yield point, beyond which the soil suffers increasingly plastic deformations until failure (THOMPSON, 1966; SCHNAID et al., 2001).

Initially it was observed from the photographs shown in Figure 6.3 that generally the performance of soils 1, 2 and 3 was significantly influenced by the addition of the chemical stabilizer under study. The untreated soil specimens presented visually a greater axial strain than the specimens of the mixture with 6% chemical additive. Furthermore, in addition to the small axial strains, the mixture specimens presented cracks along the loading axe, indicating a type of fragile failure characteristic of extremely rigid materials.

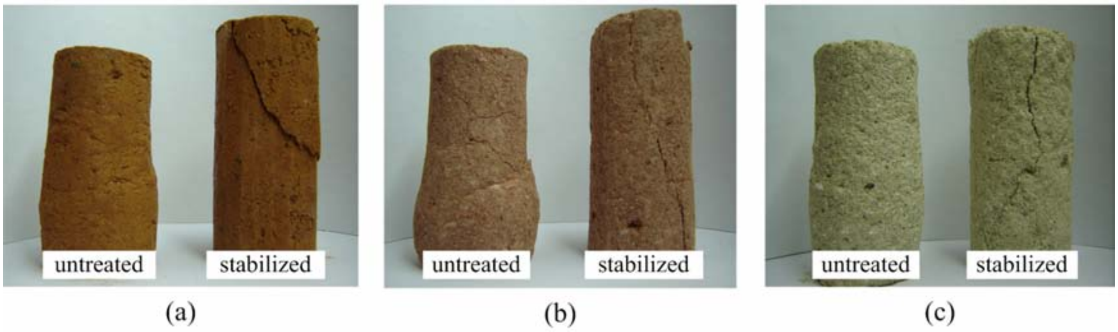


Figure 6.3. Specimens of untreated soils and its mixtures with 6% of RBI Grade 81 photographed after failure: (a) soil 1; (b) soil 2; and (c) soil 3.

Figures 6.4 to 6.6 show the results of the unconsolidated-undrained triaxial tests carried out with soils 1, 2 and 3 and their mixtures in the form of principal stress difference versus axial strain curves, and secant deformation modulus versus axial strain curves, for the standard, intermediate and modified Proctor compaction efforts. To illustrate the stress-strain behavior of the soils and mixtures, only the curves for the 200 kN/m^2 confining stress are presented, but it should be emphasized that similar performance was observed for the curves of the samples submitted to the other confining stresses.

Regarding the compaction effort, it was verified that the soils and mixtures stress-strain responses, especially the mixtures, depended on the level of applied compaction efforts (Figures 6.4 to 6.6). However, only the peak strength values of soils 1, 2 and 3 were clearly altered with the changes in both the variables analyzed in the present study, that is, the quantities of chemical additive and the different compaction efforts applied.

Figure 6.4 shows that untreated soil 1 presented stress-strain curves typical of fine soils, with ductile behavior and failure by excessive axial strain, which in the present study was the maximum axial strain adopted as criteria for closing the tests, equal to 15%. It was verified that the increase in compaction effort and the addition of 2% chemical additive accounted for not very significant changes in the elastic parameters of soil 1. An intermediate behavior was detected in the mixtures of soil 1 with 4% RBI Grade 81 and a transition was observed between the mixtures with 2% and

6% chemical stabilizer. When 6% chemical additive was added to soil 1, the stress-strain curves presented fragile failure modes and peak stress values related to smaller axial strains of the order of 1.5-4% (Figure 6.4), which is a common performance in mixtures with a significant degree of particle cementation and in line with the specialist literature (THOMPSON, 1966; TRB, 1976; ALCÂNTARA, 1995; MENDONÇA et al., 2000; CONSOLI et al., 2001).

It was possible to visualize with greater clarity from the secant deformation modulus versus axial strain curves the progressive rise in rigidity in soil 1 in function of the increase in the chemical stabilizer contents added, mainly at the deformation levels between 0.1% and 1%.

Results in Figures 6.5 and 6.6 show that the stress-strain curves of soils 2 and 3 presented similar developments. The untreated soils 2 and 3 specimens compacted at standard Proctor compaction effort failed because of excessive axial strain. On the other hand, the non-stabilized soils 2 and 3 specimens compacted at the intermediate and modified Proctor compaction efforts presented peak stress values related to the smaller axial strains, ranging from 5-8%.

Regarding the mixtures of soils 2 and 3, similarly to the mixtures in soil 1 with 6% chemical additive, it was verified that the stress-strain curves presented fragile behavior and peak stress values related to axial strains in the 2-6% range, as depicted in Figures 6.5 and 6.6.

The failure stresses in soils 1 and 2 and their mixtures were in the same range, considering all the applied confining stresses and compaction efforts. It was observed also that peak stresses in soil 3 and its mixtures reached values 20-60% greater than those determined for soils 1 and 2 and their respective mixtures.

Regarding the elastic properties of soils, Massad (1981) reported that three modulus are commonly determined through unconsolidated-undrained triaxial tests, as follows: (i) deformation modulus corresponding to the angular coefficient of the straight-line fitted to the three first points of the stress-strain curves (E_0); (ii) the deformation modulus corresponding to the angular coefficient of the straight-line that leaves from the origin of the stress-strain curve and is secant at the point where the specific axial strain reaches 1% ($Es_{1\%}$); and (iii) deformation modulus determined from the slope of the straight-line that leaves the origin of the stress-strain curve and is secant at the corresponding point at 50% failure stress (E_{50}).

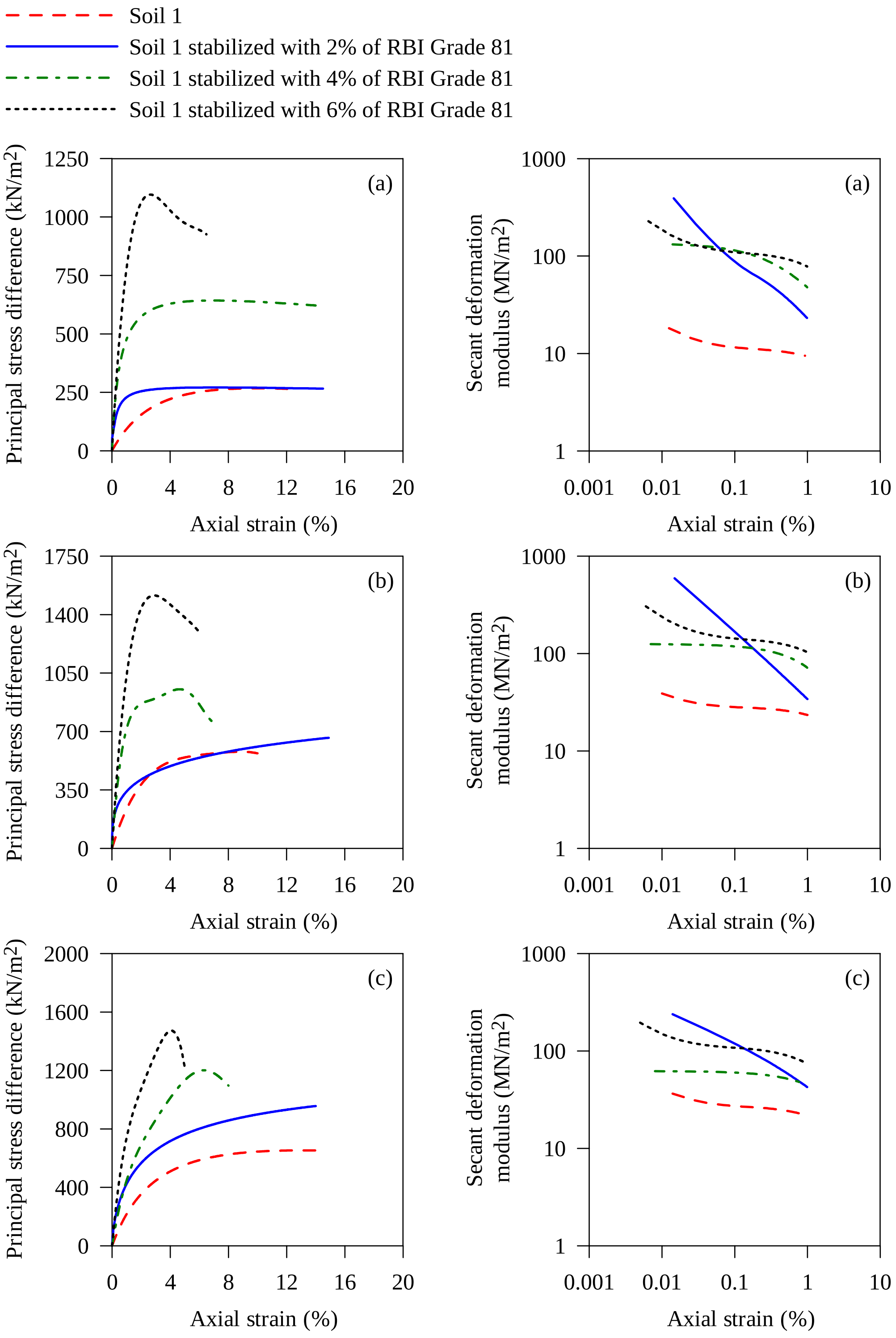


Figure 6.4. Stress-strain response and secant deformation modulus versus axial strain for soil 1 and its mixtures specimens (confining pressure 200 kN/m^2) compacted at the: (a) standard Proctor compaction effort; (b) intermediate Proctor compaction effort; and (c) modified Proctor compaction effort.

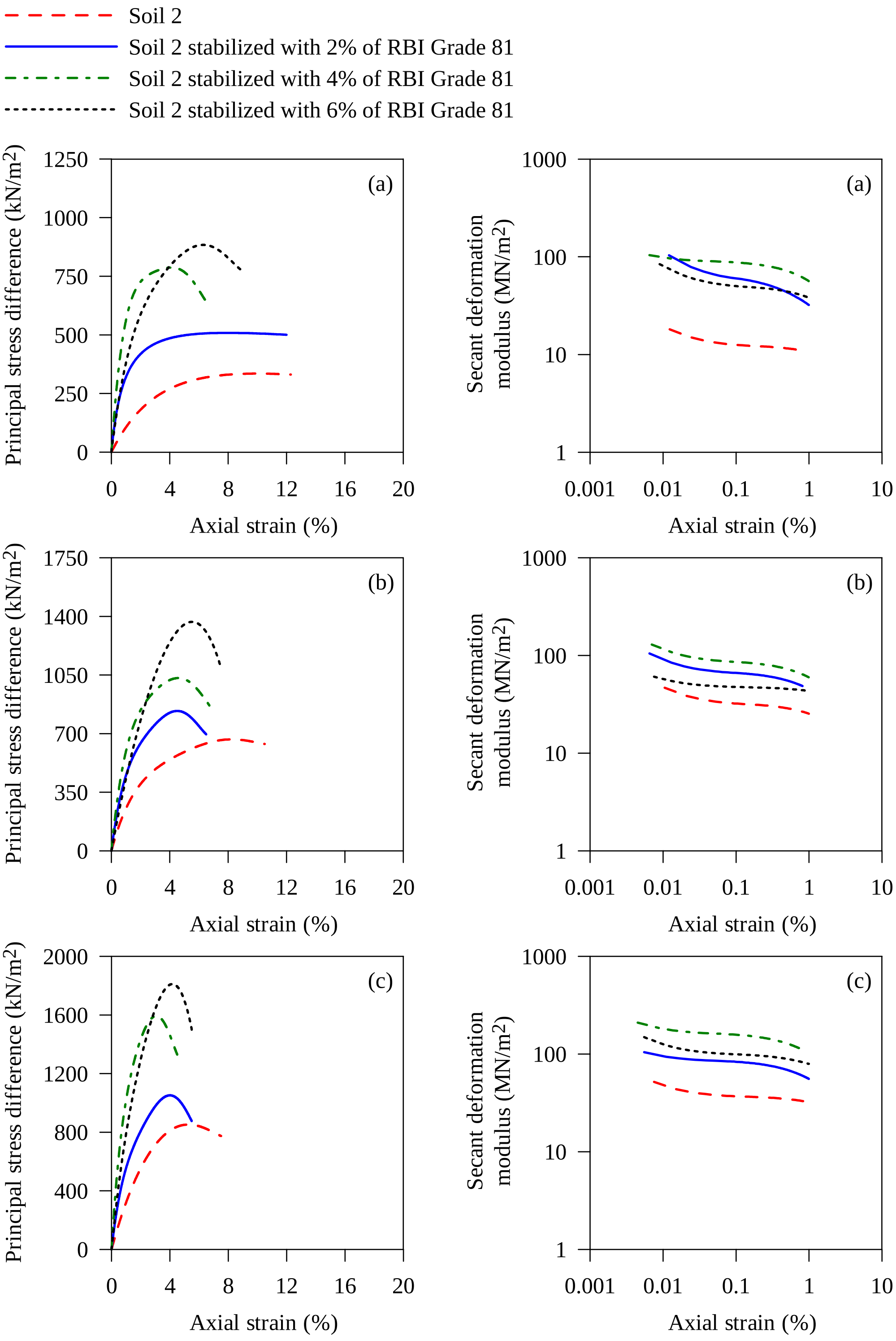


Figure 6.5. Stress-strain response and secant deformation modulus versus axial strain for soil 2 and its mixtures specimens (confining pressure 200 kN/m^2) compacted at the: (a) standard Proctor compaction effort; (b) intermediate Proctor compaction effort; and (c) modified Proctor compaction effort.

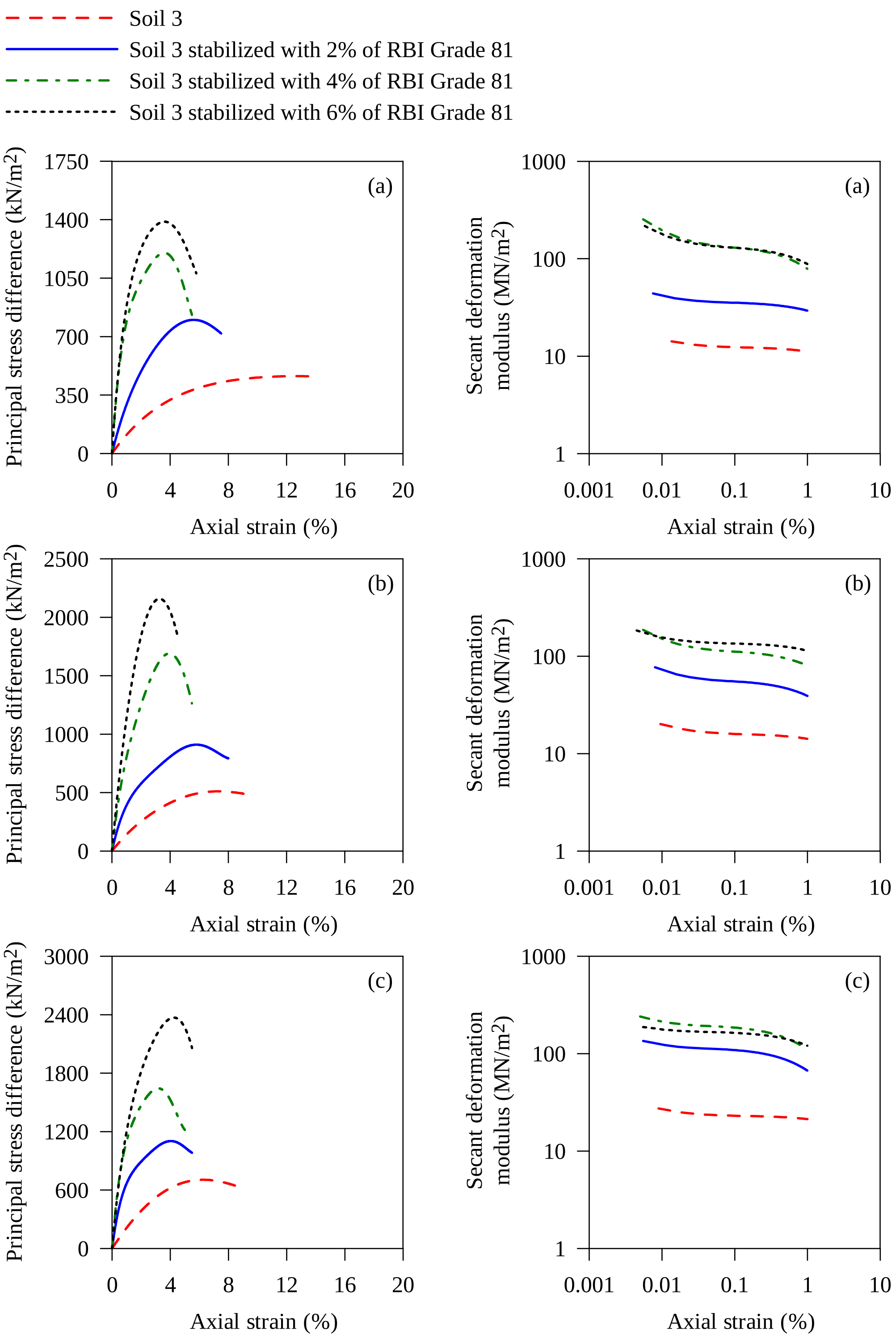


Figure 6.6. Stress-strain response and secant deformation modulus versus axial strain for soil 3 and its mixtures specimens (confining pressure 200 kN/m^2) compacted at the: (a) standard Proctor compaction effort; (b) intermediate Proctor compaction effort; and (c) modified Proctor compaction effort.

Figures 6.7 to 6.9 show the maximum, minimum and mean $Es_{1\%}$ and E_{50} values determined from specimens of soils 1, 2 and 3 and their mixtures with the chemical stabilizer analyzed, tested under the 50, 100, 200 and 400 kN/m^2 confining stresses. Analysis of the data presented in the figures showed that the chemical additive accounted for significant increases in the $Es_{1\%}$ and E_{50} values. The increases in the confining stress and the compaction effort generally also caused increases in the $Es_{1\%}$ and E_{50} values, but on a smaller scale than those produced by adding the chemical additive. Soils 1, 2 and 3 and their mixtures generally presented $E_{50} > Es_{1\%}$, that indicated that the materials under study, when they reached a stress level corresponding to 50% failure, were still in the elastic regime of strains, that is, with axial strains lower than 1%.

Figures 6.7 to 6.9 showed that the mean $Es_{1\%}$ and E_{50} values determined for soils 1, 2 and 3 were in the range 10-30 MN/m^2. In contrast, the mean $Es_{1\%}$ and E_{50} values reported for the mixtures of soils 1, 2 and 3 were 21-158 MN/m^2. The values of the deformation modulus of the mixtures studied in the present study were within ranges determined for chemically stabilized soils and presented by authors such as Thompson (1966), for soil-lime mixtures, and Consoli et al. (2001), for soil-fly ash-carbide lime mixtures.

Generally, the increase in the chemical additive quantities and the increase in compaction energy produced increases in the $Es_{1\%}$ and E_{50} values. However, two cases were observed where the performance was atypical to that usually observed. The first case was specimens from soil 1 and its mixtures that presented the highest mean $Es_{1\%}$ and E_{50} values associated to the intermediate Proctor compaction effort. It's believed that this performance may be associated to the deleterious effects caused to soil 1 and its mixtures by the applied modified Proctor compaction effort, with probable formation of plasma orientations and empty cracks, responsible for the lamellation in layers of specimens, in line with performance previously reported by Carvalho et al. (1981), who studied a purple latosol stabilized with hydrated lime. The second case concerned soil 2 mixtures, where the addition of increasing quantities of chemical additives produced increases in the mean $Es_{1\%}$ and E_{50} values up to the 4% content, and falls in the deformation modulus measured when 6% chemical additive was added to the soil. In this case there is no plausible explanation for this performance, and this behavior needs further study.

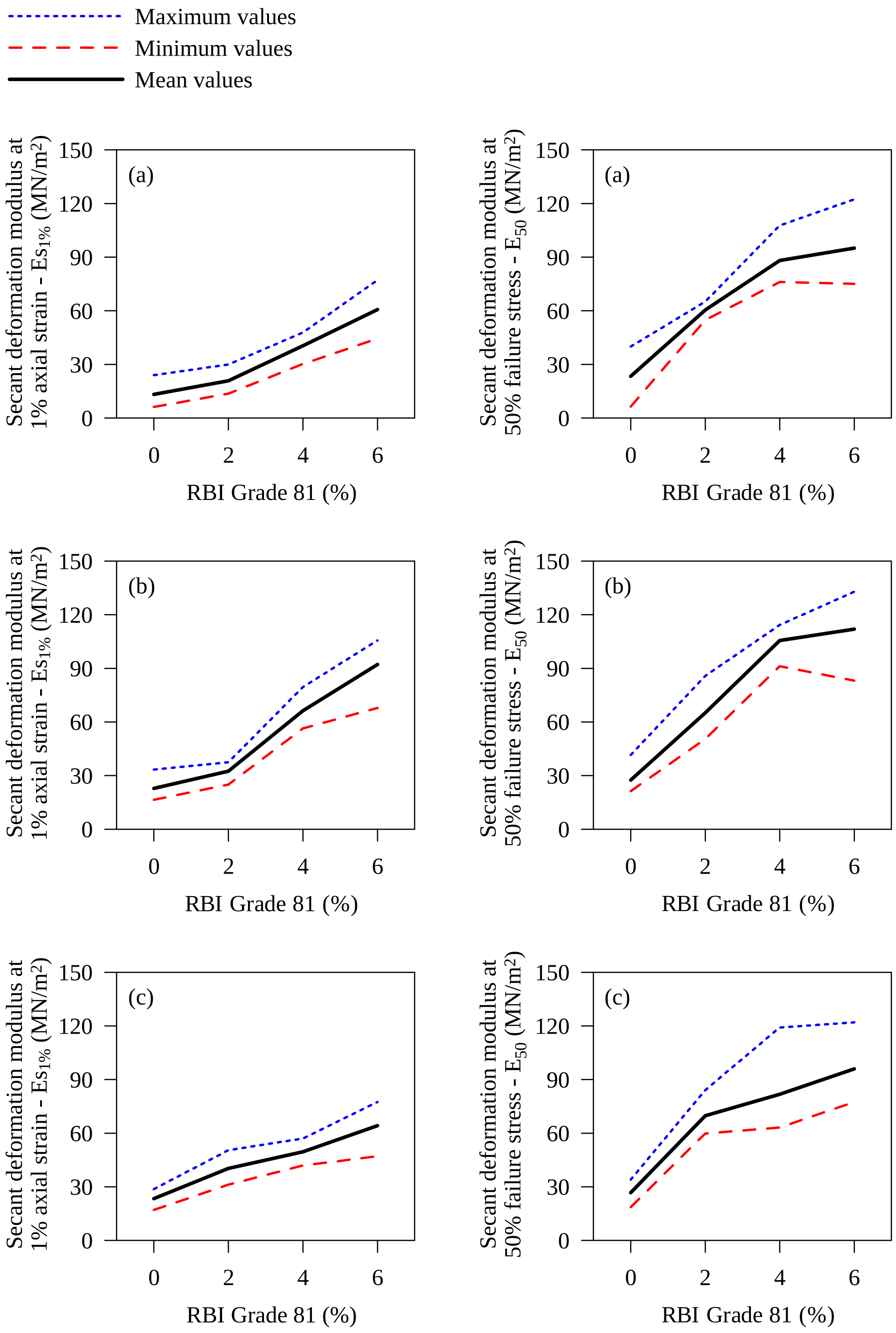


Figure 6.7. Maximum, minimum and mean values of secant deformation modulus $Es_{1\%}$ (i.e., secant modulus at 1% axial strain) and E_{50} (i.e., secant modulus at 50% failure stress level) of soil 1 and its mixtures specimens tested at different confining pressures (50, 100, 200 and 400 kN/m^2) and compacted at the: (a) standard Proctor compaction effort; (b) intermediate Proctor compaction effort; and (c) modified Proctor compaction effort.

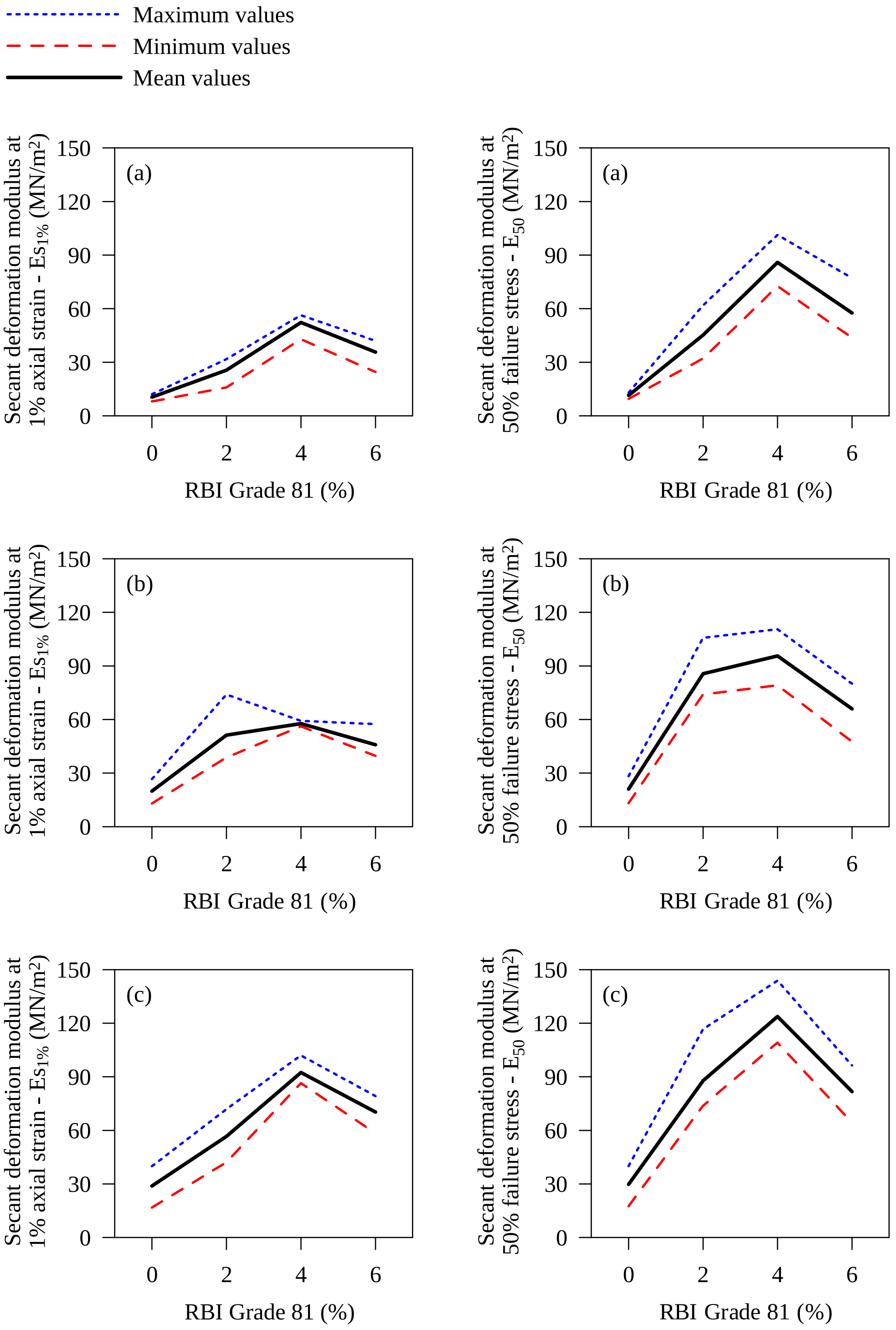


Figure 6.8. Maximum, minimum and mean values of secant deformation modulus Es$_{1\%}$ (i.e., secant modulus at 1% axial strain) and E$_{50}$ (i.e., secant modulus at 50% failure stress level) of soil 2 and its mixtures specimens tested at different confining pressures (50, 100, 200 and 400 kN/m^2) and compacted at the: (a) standard Proctor compaction effort; (b) intermediate Proctor compaction effort; and (c) modified Proctor compaction effort.

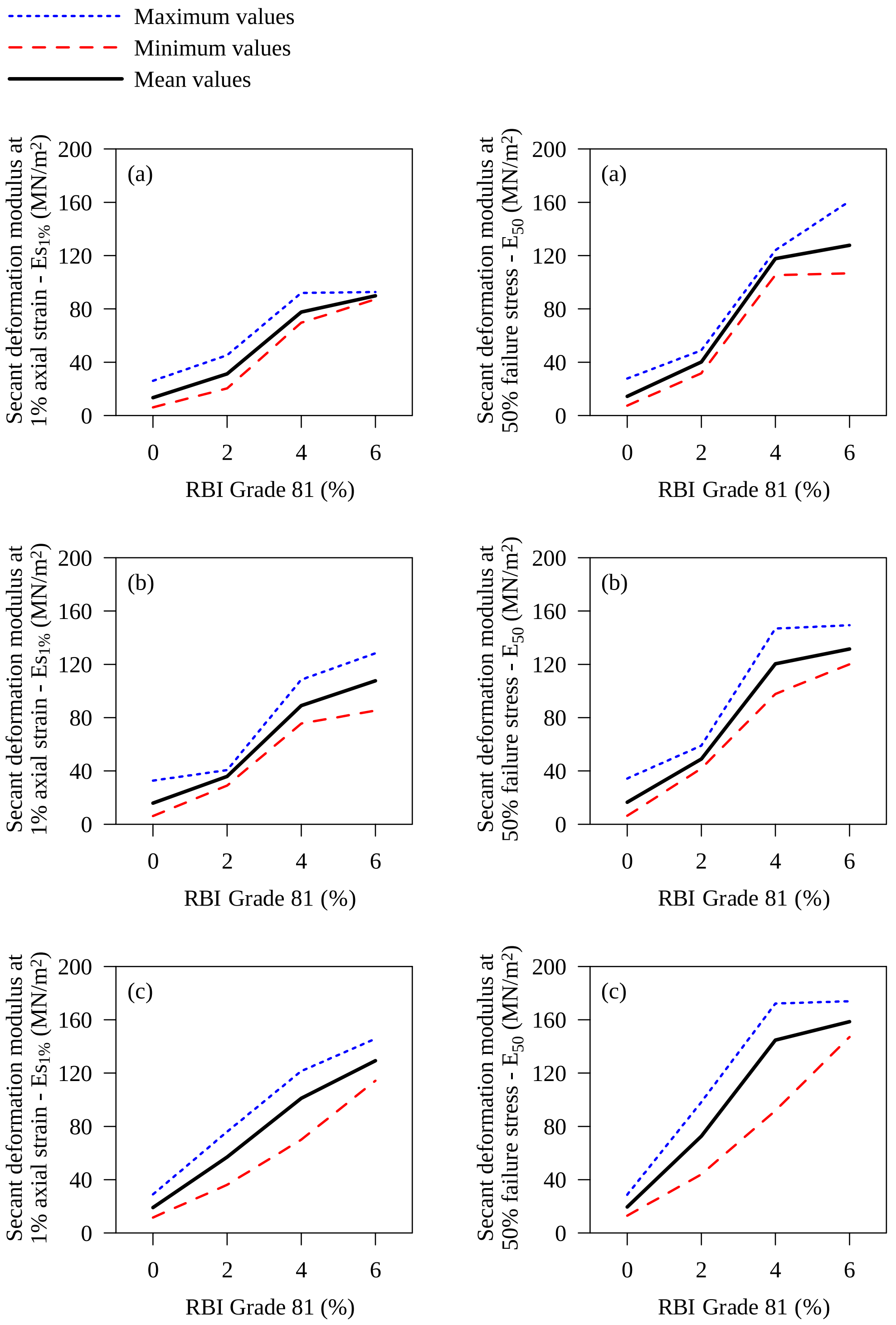


Figure 6.9. Maximum, minimum and mean values of secant deformation modulus $Es_{1\%}$ (i.e., secant modulus at 1% axial strain) and E_{50} (i.e., secant modulus at 50% failure stress level) of soil 3 and its mixtures specimens tested at different confining pressures (50, 100, 200 and 400 kN/m^2) and compacted at the: (a) standard Proctor compaction effort; (b) intermediate Proctor compaction effort; and (c) modified Proctor compaction effort.

Figures 6.10 to 6.12 present the peak strength envelopes, in terms of total stresses, of soils 1, 2 and 3 and their mixtures with RBI Grade 81 and the peak friction angle and peak cohesion intercept values determined for these materials. Analysis of these results showed that increases in chemical additive content and the compaction effort accounted for increases in the peak friction angle and peak cohesion intercept values, with more expressive increases in this last parameter.

The results in Figures 6.10 to 6.12 showed that generally the mixtures with 6% chemical additive compacted at the modified Proctor compaction effort presented the greatest peak friction angle and peak cohesion intercept values, except for soil 1 mixtures that presented a maximum peak cohesion intercept associated to the mixture with 6% chemical additive compacted at the intermediate Proctor compaction effort. The maximum peak friction angle and peak cohesion intercept values of the mixtures of soils 1, 2 and 3 were, respectively: (i) 30, 37 and 40 degrees; and (ii) 348, 284 and 407 kN/m^2.

From the data presented in the Figures 6.10 to 6.12 it was possible to determine also the percentage gains in the peak friction angle and peak cohesion intercept parameters due to the addition of the chemical additive analyzed. The greatest percentage gains in the peak friction angle values were 71%, 23% and 21% determined, respectively, for the mixtures of soils 1, 2 and 3. Regarding the peak cohesion intercept, the highest percentage gains determined for the mixtures were 415%, 410% and 2900%, respectively, for soils 1, 2 and 3.

It is believed that there are short and medium-term hydration/cementation and medium-term pozzolanic reactions and the latter account for the solubilization of some minerals present in the soil (e.g. SiO_2 and Al_2O_3) and consequently for the formation of stable cementing compounds. These compounds, together with the microfibers randomly distributed in the chemical stabilizing mass, give the mixtures higher peak cohesion intercept values. The changes in the peak friction angle values were probably associated to the alterations in the texture of the soil studied caused by fine particle flocculation and/or agglomeration.

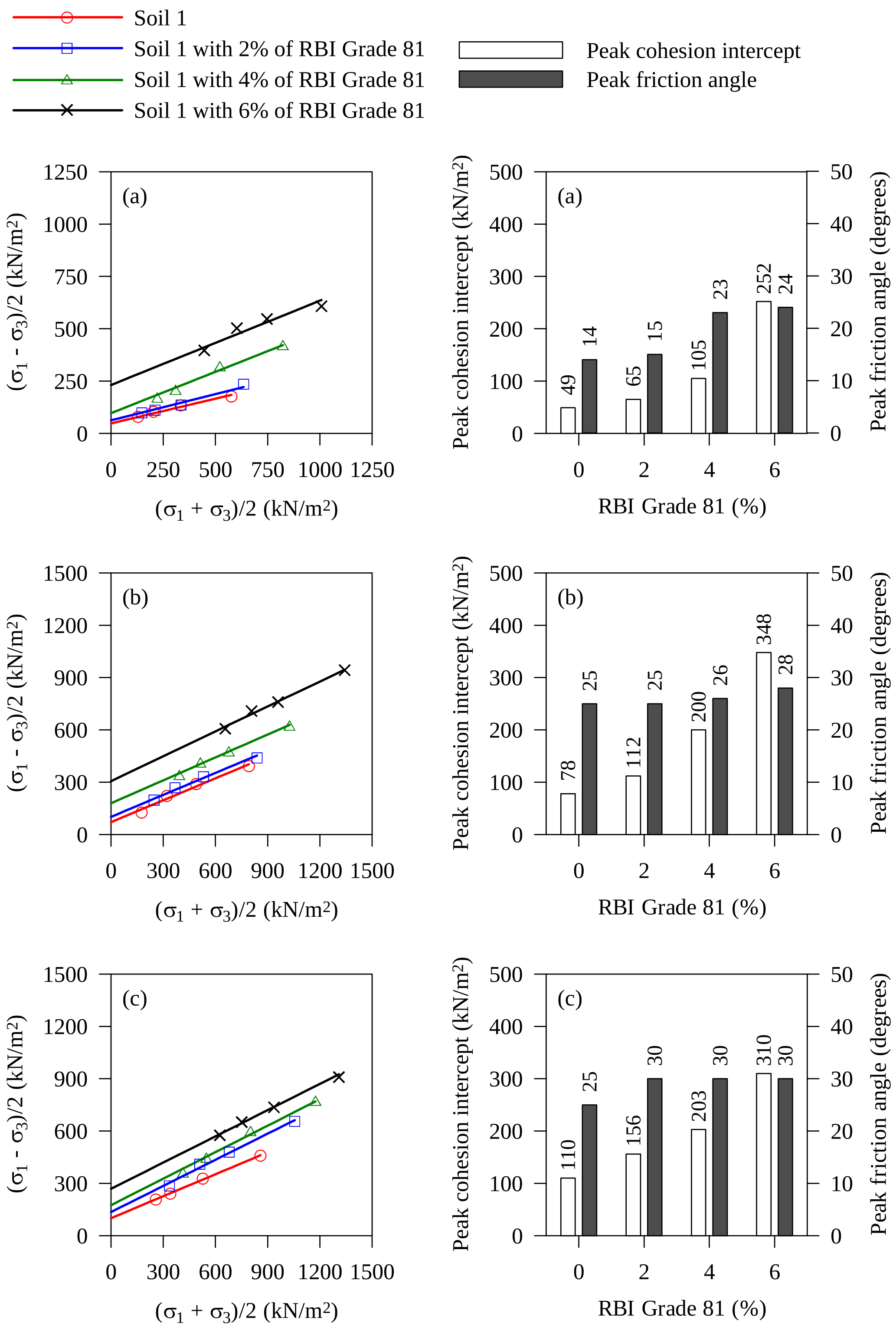


Figure 6.10. Peak strength envelopes, peak cohesion intercept and peak friction angle for soil 1 and its mixtures specimens compacted at the: (a) standard Proctor compaction effort; (b) intermediate Proctor compaction effort; and (c) modified Proctor compaction effort.

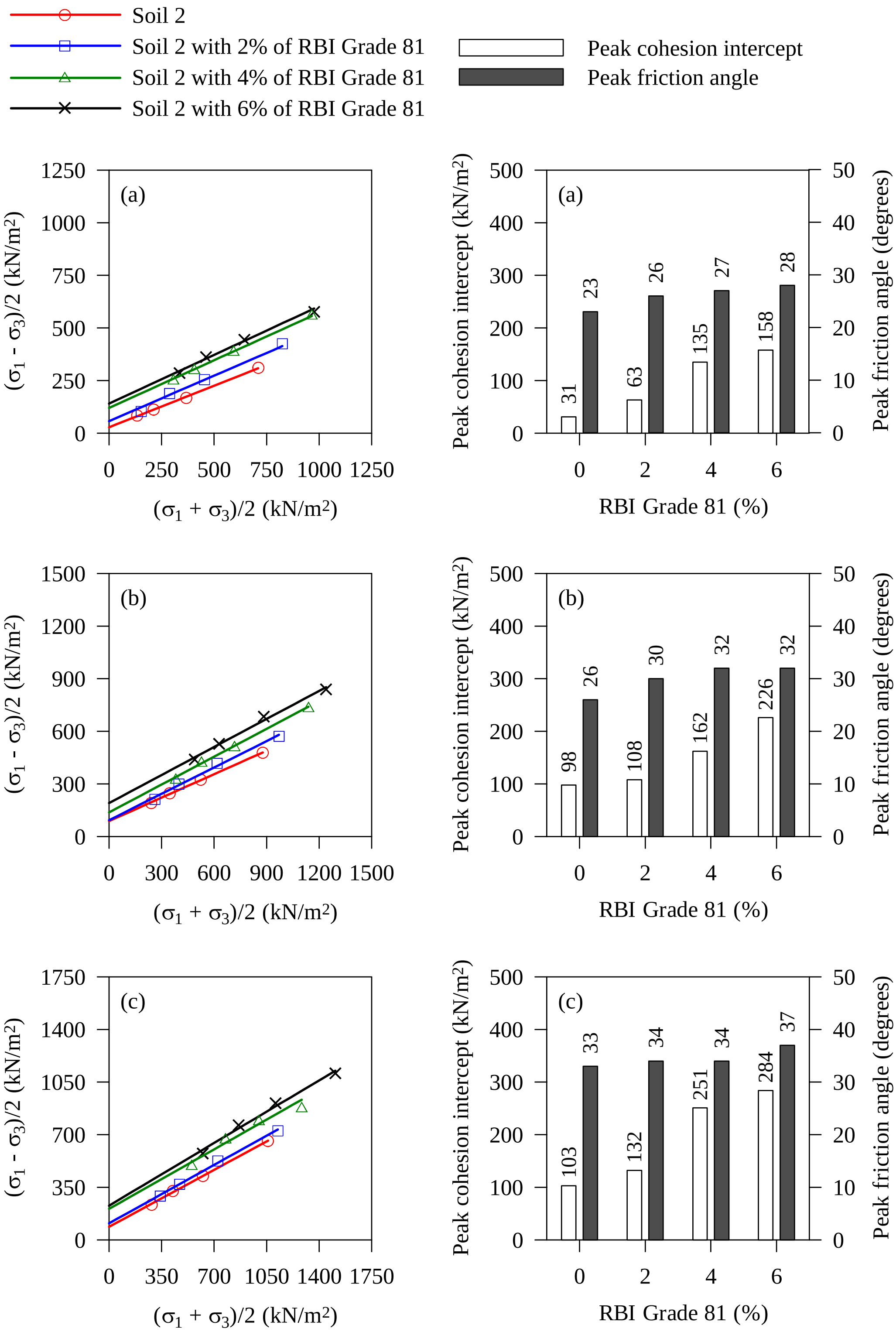


Figure 6.11. Peak strength envelopes, peak cohesion intercept and peak friction angle for soil 2 and its mixtures specimens compacted at the: (a) standard Proctor compaction effort; (b) intermediate Proctor compaction effort; and (c) modified Proctor compaction effort.

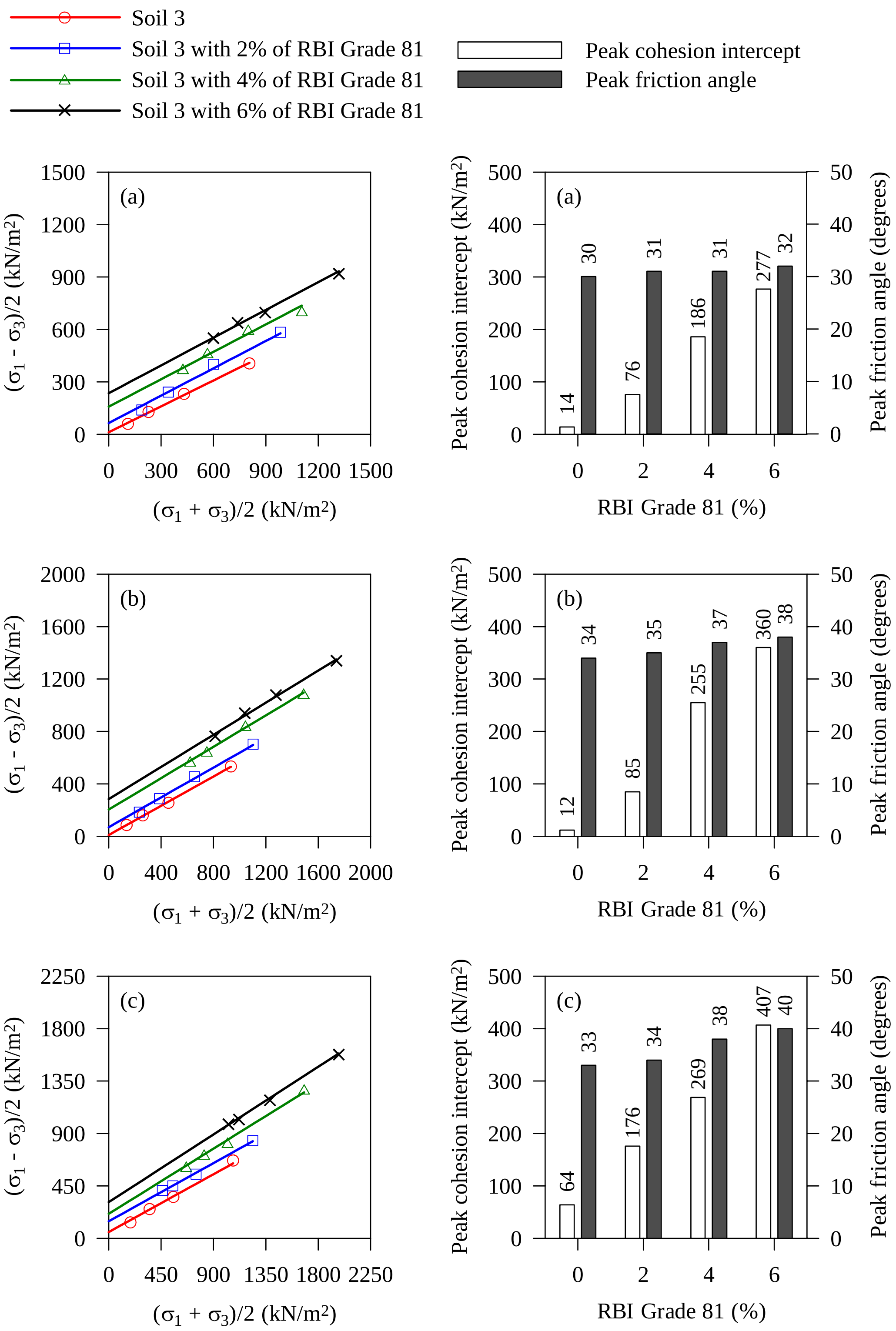


Figure 6.12. Peak strength envelopes, peak cohesion intercept and peak friction angle for soil 3 and its mixtures specimens compacted at the: (a) standard Proctor compaction effort; (b) intermediate Proctor compaction effort; and (c) modified Proctor compaction effort.

In a recent study, Trindade et al. (2006a) reported the results of unconfined compression tests carried out on specimens from soils 2 and 3 and their mixtures with 2%, 4% and 6% RBI Grade 81, considering standard, intermediate and modified Proctor compaction efforts and a seven day cure time in a acclimatized chamber, in the case of the mixtures. From these data, in the present paper regression analyses were performed to determine models for the soil RBI Grade 81 mixtures behavior that would relate the unconfined compressive strength with the peak cohesion intercept parameter, as well as with the secant deformation modulus at the level of 1% strain.

It should be pointed out that Thompson (1966), from the results of unconfined compression and triaxial tests, the latter carried out with the confining pressures of 34.5, 103.4 and 241.3 kN/m^2, established relationships among the unconfined compressive strength and the peak cohesion intercept, as well as the deformation modulus in compression (for the 103.4 kN/m^2 confining pressure) for mixtures of four typical soils from the State of Illinois, USA, with 3% and 5% hydrated lime, submitted to cure periods of 1, 2, 4 and 6 days, shown in Equations 6.1 and 6.2, as follows:

$$\mathrm{c(kN/m^2)} = 64.12 + 0.292 \cdot q_u\,\mathrm{(kN/m^2)} \quad \text{.........} \quad (6.1)$$

$$\mathrm{E(MN/m^2)} = 68.81 + 0.124 \cdot q_u\,\mathrm{(kN/m^2)} \quad \text{.........} \quad (6.2)$$

where: (c) is the peak cohesion intercept; (E) is the deformation modulus in compression determined from unconsolidated-undrained triaxial tests carried out at the 103.4 kN/m^2 confining pressure; and (q_u) is the unconfined compressive strength.

The results of the present paper are presented in Figure 6.13, which shows the relationships between the unconfined compressive strength and the peak cohesion intercept values and between unconfined compressive strength and the secant deformation modulus at the level of 1% strain values established to the tested mixtures. It was observed that the linear model, in addition to its greater simplicity, was a good fit to the obtained test data.

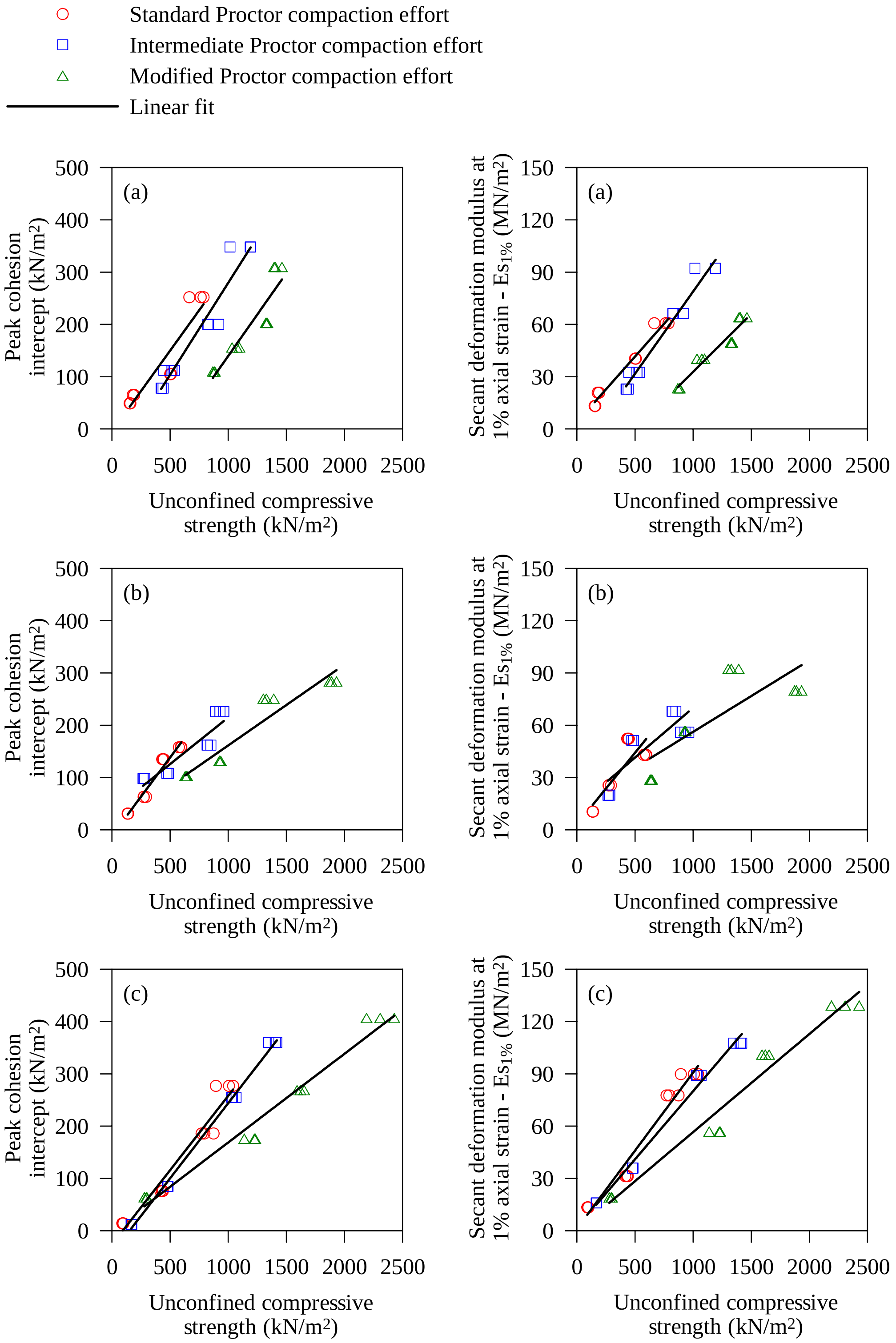


Figure 6.13. Peak cohesion intercept (c) and mean values of secant deformation modulus at 1% axial strain ($Es_{1\%}$) determined at different confining pressures (50, 100, 200 and 400 kN/m^2) versus unconfined compressive strength (q_u): (a) soil 1 mixtures; (b) soil 2 mixtures; and (c) soil 3 mixtures.

Table 6.5 shows the equations obtained from the linear regression analyses to determine the peak cohesion intercept (c), in kN/m^2, and the secant deformation modulus at 1% axial strain ($Es_{1\%}$), in MN/m^2, in function of the unconfined compressive strength (q_u), in kN/m^2. All the equation parameters were significant at levels of 0.1 and 0.001 probability by the Student t test and determination coefficients (R^2) were ≥ 0.85, with the exception of the equations to estimate the $Es_{1\%}$ values of the soil 2 mixtures that presented $R^2 \leq 0.75$. The low R^2 values detected for the equations to estimate $Es_{1\%}$ in function of q_u for the soil 2 mixtures were attributed to the atypical performance shown in Figure 6.8, where a decrease in the $Es_{1\%}$ values was observed with the addition of 6% chemical stabilizer.

Table 6.5. Regression equations adjusted to (c) and ($Es_{1\%}$) of soil-RBI Grade 81 mixtures as dependent variables of (q_u) these same materials

Parameters	Soils samples	Compaction efforts	Equations	R^2
Peak cohesion intercept (kN/m^2)	Soil 1	Standard Proctor	$\hat{c} = -4.6091 + 0.3088^{***} \cdot q_u$	0.87^{***}
		Intermediate Proctor	$\hat{c} = -72.0713 + 0.3515^{***} \cdot q_u$	0.94^{***}
		Modified Proctor	$\hat{c} = -177.1962 + 0.3170^{***} \cdot q_u$	0.85^{***}
	Soil 2	Standard Proctor	$\hat{c} = -11.2979 + 0.2994^{***} \cdot q_u$	0.96^{***}
		Intermediate Proctor	$\hat{c} = 36.5322 + 0.1783^{***} \cdot q_u$	0.86^{***}
		Modified Proctor	$\hat{c} = 6.8514 + 0.1546^{***} \cdot q_u$	0.91^{***}
	Soil 3	Standard Proctor	$\hat{c} = -26.2829 + 0.2842^{***} \cdot q_u$	0.95^{***}
		Intermediate Proctor	$\hat{c} = -43.4631 + 0.2872^{***} \cdot q_u$	1.00^{***}
		Modified Proctor	$\hat{c} = -0.7638 + 0.1696^{***} \cdot q_u$	0.98^{***}
Secant deformation modulus at 1% axial strain (MN/m^2)	Soil 1	Standard Proctor	$\hat{E}s_{1\%} = 3.9491 + 0.0753^{***} \cdot q_u$	0.98^{***}
		Intermediate Proctor	$\hat{E}s_{1\%} = -15.4012 + 0.0943^{***} \cdot q_u$	0.97^{***}
		Modified Proctor	$\hat{E}s_{1\%} = -33.6231 + 0.0664^{***} \cdot q_u$	0.94^{***}
	Soil 2	Standard Proctor	$\hat{E}s_{1\%} = 3.1415 + 0.0822^{***} \cdot q_u$	0.75^{***}
		Intermediate Proctor	$\hat{E}s_{1\%} = 12.9434 + 0.0571^{***} \cdot q_u$	0.72^{***}
		Modified Proctor	$\hat{E}s_{1\%} = 15.1882 + 0.0411^{**} \cdot q_u$	0.64^{**}
	Soil 3	Standard Proctor	$\hat{E}s_{1\%} = 1.1740 + 0.0896^{***} \cdot q_u$	0.99^{***}
		Intermediate Proctor	$\hat{E}s_{1\%} = 1.8690 + 0.0782^{***} \cdot q_u$	0.99^{***}
		Modified Proctor	$\hat{E}s_{1\%} = 0.3027 + 0.0563^{***} \cdot q_u$	0.96^{***}

** and *** significant at the 0.01 and 0.001 probability level, respectively.

Finally, it should be emphasized that the mechanical behavior of the soil mixtures described in this paper so far reflects the main changes in the stress-strain response of soils associated with their developed degree of cementation and it is qualitatively in agreement with triaxial test results reported in the specialized literature for naturally cemented and chemically stabilized soils, as reported by Thompson (1966), TRB (1976), Clough et al. (1981), Little et al. (1987), Mendonça et al. (2000), Consoli et al. (2001), Schnaid et al. (2001) and Trindade et al. (2006b).

6.5. Conclusions

- The addition of increasing of the chemical stabilizer content reduced the maximum dry unit weight values and increased the optimum moisture content values of the soils analyzed. In contrast, the increase in the compaction effort accounted for increases in the maximum dry unit weight and reductions in the optimum moisture content values of soils and respective mixtures.
- The increase in the chemical additive content and the compaction effort caused significant gains in the shear strength of the soils, with greater effect on the peak cohesion intercept parameter.
- The addition of the chemical stabilizer to the soils started a internal process responsible for the development of cementation linking among particles, causing significant changes in their stress-strain parameters and resulting in mixtures with higher rigidity and fragile failure behavior.
- From linear regression analyses it was determined equations to estimate the peak cohesion intercept (c) and the secant deformation modulus at 1% axial strain ($Es_{1\%}$) in function of the unconfined compressive strength (q_u). The equations obtained presented high R^2 values, generally greater than 0.85, and all their parameters were significant to the levels of probability of 0.01 and 0.001 by the Student t test.

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