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Influence of Matric Suction on the Results of Plate Load Tests Performed on a Lateritic Soil Deposit

ABSTRACT: This paper evaluates the influence of soil matric suction on the results of plate load tests conducted in an unsaturated, lateritic soil deposit at a depth of 1.5 m. Soil suction was monitored during the tests with tensiometers installed at the bottom of the testing pit. Field test results show that small increases in matric suction lead to substantial increases in bearing capacity of the soil-plate system. In situ experimental loading-collapse (LC) and suction increase (SI) yield surfaces are proposed for the soil investigated. Changes in matric suction were observed to significantly influence settlement response, particularly for high levels of surcharge load. The rate of settlement shows a non-linear decreasing trend with increasing soil matric suction.

KEYWORDS: matric suction, plate load test, settlement rate, bearing capacity, lateritic soils, unsaturated soils

The performance of foundation systems on unsaturated soil deposits is considerably influenced by variations of the negative pore-water pressure (i.e., matric suction) distribution within the soil mass due to local microclimate conditions. Although significant understanding has been gained in the last few decades on the behavior of unsaturated soils, the use of unsaturated soil mechanics concepts in the interpretation of field tests has not been incorporated into the state-of-practice. As a consequence, conservative foundation engineering principles are used in practice in regions where unsaturated soil conditions prevail. For example, direct use of field test results (e.g., from plate load tests) obtained during a dry season in unsaturated soil deposits may lead to the selection of unconservatively high design parameters at the site. On the other hand, ignoring altogether the beneficial impact of matric suction on the bearing capacity of soils may lead to unnecessarily expensive foundation systems in tropical, arid climates.

This paper investigates the influence of soil suction on the results of plate load tests conducted at a depth of 1.5 m on a structured, naturally occurring lateritic soil. Ten tests were carried out under different soil suction conditions. Matric suction was monitored during plate load testing using tensiometers installed at the bottom of the testing pit up to a depth equal to one plate diameter (0.80 m), which is generally recognized as the influence zone in which significant stress variation occurs (Terzaghi and Peck 1948). The results provide insight into the influence of soil matric suction on ultimate bearing capacity and settlement rate of plate load tests performed on lateritic soils.

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Background

Although the notion that the presence of negative pore-water pressures (matric suction) in the soil influences the behavior of foundations is not new, there is only limited information reported in technical literature dealing with the quantification of this problem. A brief description of previous research on this topic regarding shallow footings is presented below.

Bearing capacity loss associated with the soil saturation has been commonly accounted for through the use of different values of unit weight of the soil (γ) that arise due to total or partial submersion. Other soil parameters such as the effective cohesion (c') and the effective internal friction angle (ϕ') are usually assumed to play a minor role. Considering the submerged unit weight of the soil is about 50 % of the moist unit weight, Terzaghi and Peck (1948) stated that the bearing capacity of shallow footings could be approximately reduced by 50 % if the water level rises from a depth equal to the footing width below the footing to the surface. Meyerhof (1955) proposed an analytical model for estimating the bearing capacity of shallow foundations based on the variations of the soil unit weight and taking into consideration the possibility of partial submersion. The groundwater table is assumed to lie between the foundation base and the lower portion of the soil failure surface. The value of unit weight of soil to be used is defined as:

$$\gamma = \gamma_{sub} + F\,(\gamma' - \gamma_{sub}) \tag{1}$$

where

γ_{sub} = submerged unit weight;
γ' = unit weight for the soil above the level of the groundwater table, and
F = factor varying from 0 (fully submerged soil) to 1 (water table at or below the failure surface).

Bowles (1996) suggests the use of γ_{sub} for bearing capacity computation in the case of full submergence. Partial submerged condition is considered to take place when the water table lies above a depth of $\{0.5\,B \tan(45 + \phi'/2)\}$, where B is the founda-

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tion width. In this case, an expression analogous to Eq 1 is provided to determine an average unit weight to be used in bearing capacity estimation.

The bearing capacity factor N_γ has also been used to account for the effects of the soil saturation on shallow foundation behavior. Using the method of characteristics, Krishnamurthy and Kameswara Rao (1975) developed charts to assess N_γ when the water table level varies from the surface to a depth equal to 12 B. The analyses were conducted for different values of ϕ' and γ.

However, accounting for bearing capacity losses due to a decrease in soil saturation only by changing γ or N_γ may not be an adequate or a rational design approach, since only the effect of positive pore-water pressures is taken into consideration, that is, the effect of negative pore-water pressures is ignored in the design approach.

In most experimental work on shallow footings, the depth of the water table has been used as a reference parameter for quantifying the foundation performance with respect to the variation of the degree of saturation within the soil mass. Meyerhof (1955) performed load tests in small-scale models using square and strip footings with $B = 25$ mm over a sand layer. Tests were carried out considering different positions of the water table. A linear increase in ultimate bearing capacity with depth was observed up to a maximum value, beyond which the ultimate bearing capacity remained unchanged. Similar tests were carried out by Steesen-Bach et al. (1987) using a 22-mm-square plate on two types of sand. A clear relationship between ultimate bearing capacity and the water table level was not obtained for a comparatively coarse sand. However, a substantial increase in bearing capacity with decreasing water level was observed for the finer sand. The ultimate bearing capacity using sand under moist condition was eleven times higher in comparison to the submerged condition.

Small-scale tests with footings under constant loading are described by Agarwal and Rana (1987). The main purpose of the research was to observe settlements of the footing as the water table raised from a depth of 1.5 B to the surface. When the water table reached a depth of B below the footing, settlements were 38 % higher than those measured using dry soil. Settlements were 95 % higher when the water table reached the base of the footing. Papadopoulos and Anagnostopoulos (1987) proposed an analytical model for settlement computation on granular soil based on elastic stress distribution and on the constrained modulus from oedometric tests. Settlement analyses for saturated and dry conditions were reported for square footings of width B ranging from 0.5 to 50 m. It was verified that the magnitude of settlements when the water table was at the level of the base of the footing was 10 to 50 % higher than when the water table was below a depth where the excess vertical stress becomes negligible. The effect of water level rise was observed to be more relevant for large footings.

Amar et al. (1987) performed field plate load tests on a silt layer using square footings, 1 m wide, placed at the ground surface. A first group of tests was performed with the water table at a depth of less than 1 m, which corresponded to a degree of saturation of approximately 90 %. Another group of tests was carried out with the water table at a depth of 2.5 m (degree of saturation of 75 %). The average ultimate bearing capacity of the second group of tests was 25 % higher than that in the first group. The settlement corresponding to the average ultimate bearing capacity was 60 % smaller in the second group.

Limited research work has been performed so far on shallow foundations in which the negative pore-water pressures of the soil were explicitly accounted for. Steesen-Bach et al. (1987) reported results of four field plate load tests with measurement of soil matric suction ($u_a - u_w$) with tensiometers. Plates of 0.1 and 0.2 m in diameter were used. The ultimate bearing capacity obtained in the test results was used to backcalculate the cohesion of the soil corresponding to each test. However, a clear trend between soil suction and the calculated cohesion was not observed.

Fredlund and Rahardjo (1993) proposed an extension of bearing capacity formulations to account for the increase in bearing capacity due to soil suction. The increase in bearing capacity is considered as an additional cohesive component due to matric suction, which can be estimated as $\{(u_a - u_w)\cdot\tan\phi^b\}$. The angle ϕ^b represents the increase in shear strength contribution due to matric suction. Rahardjo and Fredlund (1992) conducted a parametric analysis to evaluate the effect of soil matric suction on the ultimate bearing capacity for a square and a strip footing embedded in clay. The undrained shear strength of the clay was 50 kPa, and the value of ϕ^b was taken as 15^0. The initial bearing capacity for the strip and the square footing was 285 and 342 kPa, respectively. The initial bearing capacity was observed to increase by 27 % when the matric suction increased by an amount equal to the undrained shear strength.

Site Characterization

The plate load tests were carried out at the Experimental Foundation Site of the University of Sao Paulo at Sao Carlos, Brazil. The typical soil profile at the test site includes a superficial lateritic clayey sand layer (brown coluvium). A 0.3-m-thick layer of pebbles, located at a depth of approximately 6 m, separates the superficial layer from a residual soil layer, which is composed of a reddish clayey sand (saprolite). Both layers classify as clayey sand (SC) according to the Unified Soil Classification System. A clayey silt layer with fragmented and altered basaltic rock is reached at a depth of 24 m. The ground water level varies seasonally between 7 to 10 m below ground surface.

Figure 1 shows the typical soil profile at the experimental site, as well as the average results of five cone penetration tests (CPT). Measurements of the cone tip resistance, q_c, sleeve friction, f_s, and friction ratio, R_f are plotted with depth. The range of deviation of the parameters from the mean is also shown in Fig. 1.

Average grain-size information indicates that the upper soil layer, where the plate load tests were conducted, has 64 % of sand, 9 % of silt, and 27 % of clay. Figure 2 shows the moist unit weight (γ), dry unit weight (γ_d), void ratio, and Atterberg limit values (w_L, w_P, and I_P) obtained at intervals of 1 m within the surficial soil layer.

Figure 3 shows the drying portion of the soil-water characteristic curve obtained from an undisturbed specimen of lateritic soil collected at the experimental site at a depth of 2 m. These results, obtained using a pressure plate apparatus, relate the water content in the soil sample to the applied matric suction. The experimental data were fitted using the function proposed by Fredlund and Xing (1994). It can be observed that the air entry value for the soil is comparatively small (less than 1 kPa).

Field Testing Program and Instrumentation

Ten plate load tests were performed using a rigid circular steel bearing plate, placed on the ground at a depth of 1.5 m. The plate was 0.8 m in diameter and 25 mm in thickness. Two loading procedures were used: slow maintained load (SML) and quick maintained load (QML).

The tests were conducted in two distinct series. In the first series, five tests were carried out after inundating the pit for a

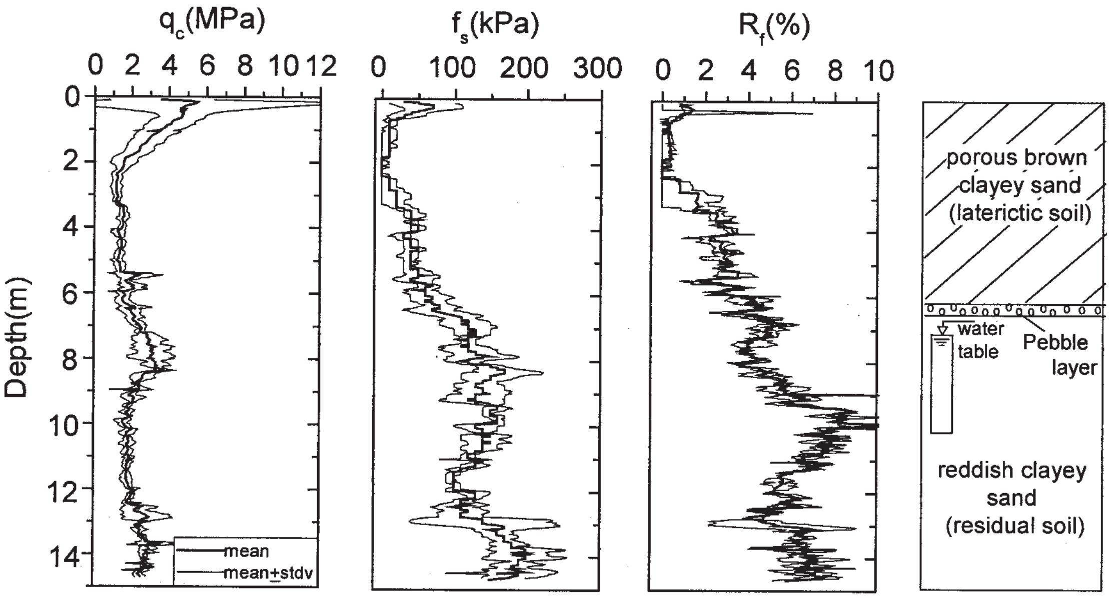

FIG. 1—*Soil profile at the experimental site and results of CPT tests.*

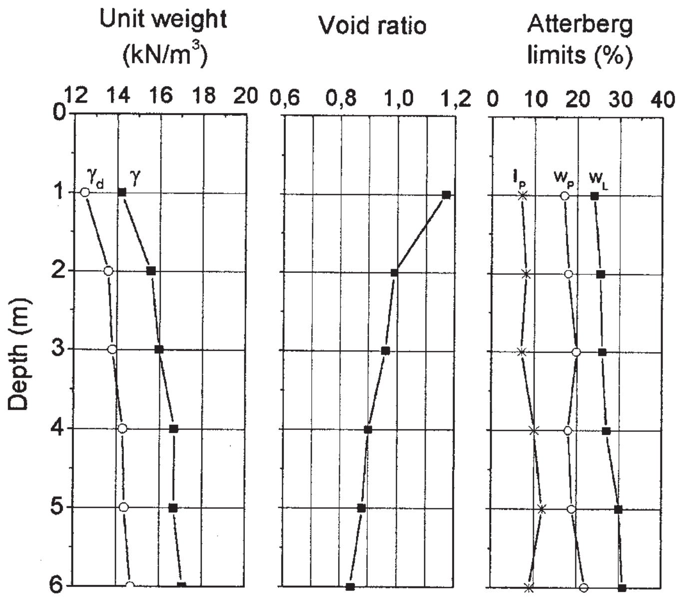

FIG. 2—*Dry and moist unit weight, void ratio, and Atterberg limits with depth within the surficial soil layer.*

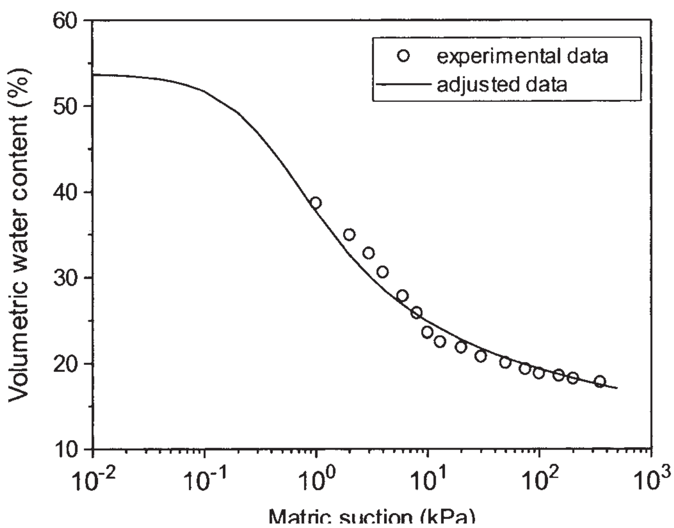

FIG. 3—*Soil water characteristic curve for lateritic soil specimen (after Machado 1998).*

period of 24 h prior to the beginning of the test (inundated tests). In the second series, the tests were performed preserving in situ water content of the soil (moist tests). A new test pit was excavated for each plate load test. All tests were carried out following Brazilian Standards for Load Tests on Shallow Foundations (NBR 6489-84) and for Static Loading Tests (MB 3472-91), which are consistent with ASTM Standard Test Method for Bearing Capacity of Soils for Static Load and Spread Footings (D 1194-72).

In the SML tests, settlements after each load increment were considered stabilized according to the following criterion:

$$s_t - s_{t/2} \leq 0.05\,(s_t - s_{ti}) \qquad (2)$$

where

s_t = settlement recorded at time t,
$s_{t/2}$ = settlement recorded at time $t/2$, and
s_{ti} = initial settlement ($t = 0$).

In the QML tests, each load increment was held constant for a period of 15 min as recommended by Fellenius (1975), and settlement readings were taken at 0, 1, 2, 3, 6, 9, 12, and 15 min.

A summary of the testing program is presented in Table 1, which indicates the designation, test type, and soil condition for each test. The first character in the test designation corresponds to the test type (S for SML and Q for QML). The second character in the designation corresponds to the soil condition during testing (I for inundated and M for moist). The last character corresponds to the test number.

Several inundated tests were conducted to evaluate the reproducibility of the test results, while several moist tests were conducted to evaluate the effect of matric suction. In order to obtain different soil suction profiles at the same site, the moist tests were conducted at different periods of the year.

TABLE 1—*Testing program summary.*

Testing No.	Designation	Test Type	Soil Condition
1	SI1	SML	Inundated
2	SI2		
3	SI3		
4	QI1	QML	Inundated
5	QI2		
6	SM1	SML	Moist
7	SM2		
8	QM1	QML	Moist
9	QM2		
10	QM3		

A general scheme of the testing setup is illustrated in Fig. 4. The load was applied in cumulative equal load increments using a hydraulic jack connected to an electric pump. Load measurements were obtained by a calibrated load cell. Metallic beams, resting on 27-m-long anchor piles, provided reaction against the applied load. Settlements were recorded using four dial gages with a resolution of 0.01 mm and a maximum stroke of 50 mm. The dial gages were set in pairs on two steel reference beams and were mounted in magnetic-articulated bases. Steel rods 32 mm in diameter were used as connecting elements between the dial gages and the bearing plate. The rods were fixed to steel cylinders welded to the plate surface. Load tests were performed trying to reach as close as possible the maximum dial gage stroke of 50 mm. The gages were reset in Tests QI2 and SM1, allowing settlement measurements greater than 50 mm.

Matric suction was monitored during the load tests using four tensiometers installed at the bottom of the pit besides the loading plate at depths of 100, 300, 600, and 800 mm. The tensiometers were installed in predrilled holes with a slightly smaller diameter than the tube diameter (20 mm). In order to facilitate a full contact of the porous element with the soil, the tensiometers were driven into the soil for a few centimeters. The tensiometers were equipped with a porous ceramic cup of 10^{-3}-m/s hydraulic conductivity and a Bourdon type vacuum gage, which was used for negative pore-water pressure measurements. Readings were taken prior to and

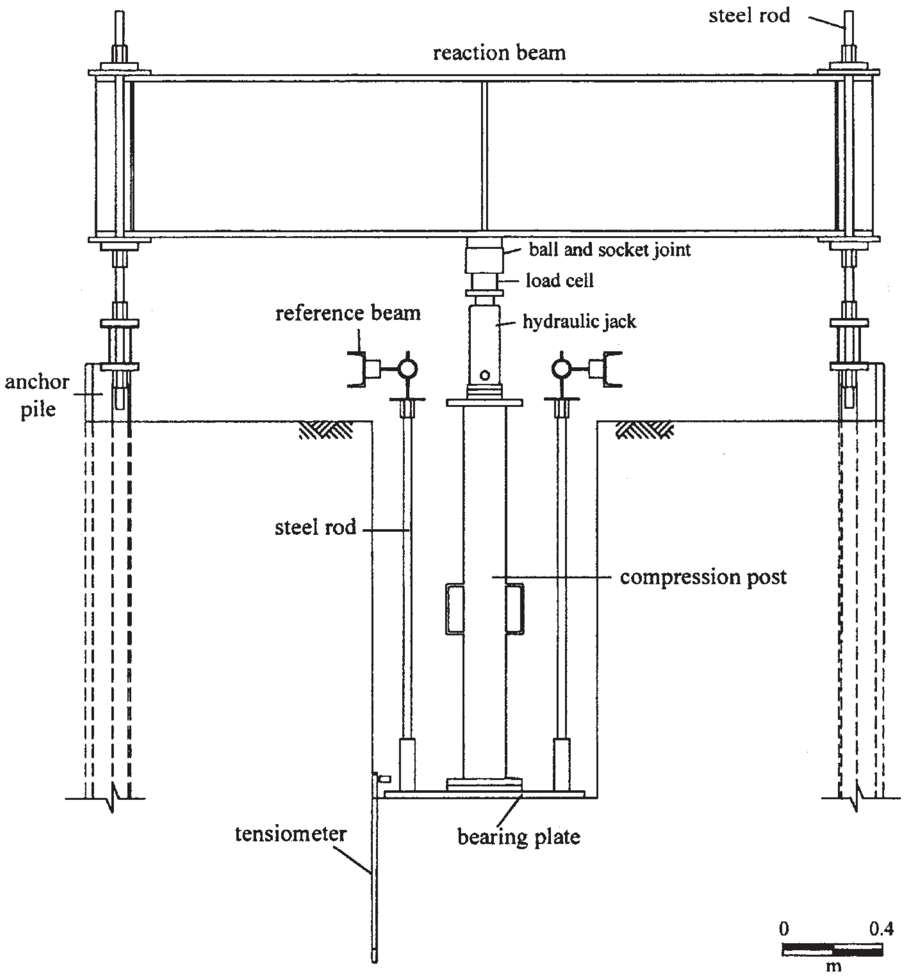

FIG. 4—*General scheme of the testing assembly and used apparatus.*

after each plate load test. Further details about the characteristics of the testing program are available in Costa (1999).

Results and Discussion

Stress-Settlement Curves and Representative Suction Values

Figures 5 and 6 exhibit the stress-settlement curves of the inundated and moist tests, respectively. The final portion of the curves in all tests shows an essentially linear increase of settlement with applied stress. Such a behavior suggests that strain hardening occurs beyond a certain stress level. In other words, additional loading does not lead to a decrease in soil stiffness. The results of Tests QI2 and SM1 showed that this yielding behavior also occurs for settlements beyond 50 mm. The shape of the curves is similar for inundated tests and for moist tests conducted at different levels of soil suction. Visual inspection at the bottom of the test pit after the end of the tests confirmed the occurrence of a punching failure mechanism (Vesic 1963). With the exception of Test SI2 for applied stress levels greater than approximately 40 kPa, Fig. 5 also shows that the stress-displacement curves of all inundated tests are within a narrow range. This indicates a good repeatability of test results and a low variability of the soil deposit in the region where the pits were excavated.

The representative suction level for each load test was defined as the average of the tensiometer readings collected at the bottom of the test pit up to a depth equal to the plate diameter. Table 2 summarizes the representative suction levels.

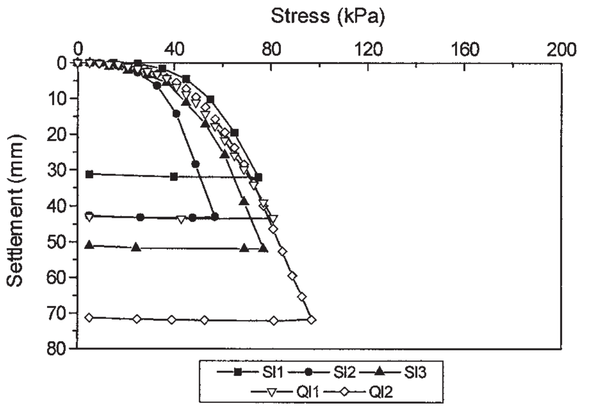

FIG. 5—*Stress-settlement curves obtained for the inundated tests.*

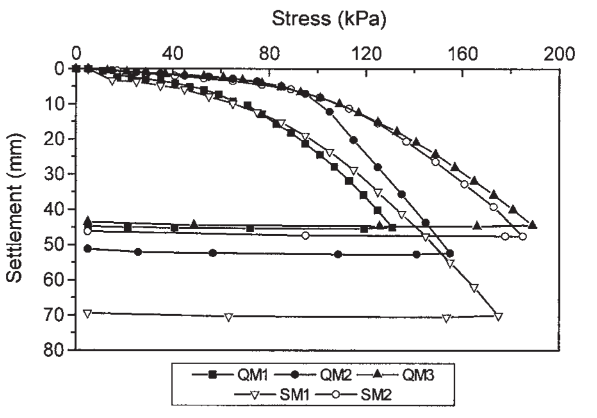

FIG. 6—*Stress-settlement curves obtained for the moist tests.*

TABLE 2—*Representative suction values for the plate load tests.*

Load Test	Measured Suction, kPa at Depth, m 0.1	0.3	0.6	0.8	Representative Suction, kPa
Inundated tests	…	…	…	…	0
SM1	13	9	11	9	10
SM2	35	30	31	29	30
QMI	24	14	12	9	15
QM2	42	22	15	11	22
QM3	38	34	27	25	31

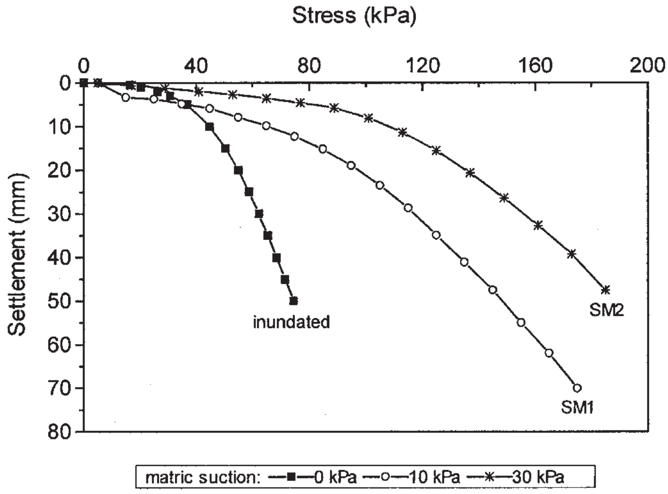

FIG. 7—*Stress-settlement curves for SML tests performed under varying suction levels.*

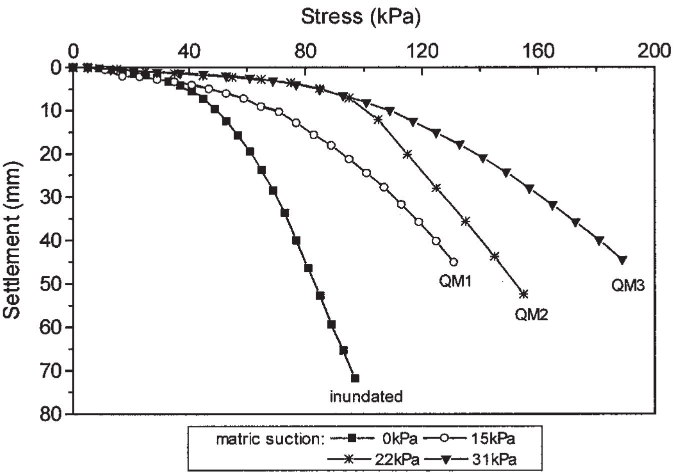

FIG. 8—*Stress-settlement curves for QML tests performed under varying suction levels.*

Influence of Soil Suction on the Soil-Plate System Response

Stress-settlement curves obtained for Tests SM1 (representative suction of 10 kPa) and SM2 (representative suction of 30 kPa) as well as the average curve obtained for Tests SI1, SI2, and SI3 (suction of 0 kPa) are shown in Fig. 7. Similarly, stress-settlement curves obtained for Tests QM1 (suction of 15 kPa), QM2 (suction of 22 kPa), QM3 (suction of 31 kPa), and the average of QI1 and QI2 (suction of 0 kPa) are presented in Fig. 8. These figures provide an assessment of the effect of matric suction on plate load test results. As observed in the figures, settlement increases considerably as suction decreases for a given stress level, that is, increasing suction values lead to a substantial increase in soil stiffness.

Bearing Capacity Analysis

Since the shape of the curves does not show a clear failure pattern, the ultimate failure stress (σ_f) was defined as the stress level that corresponds to the onset of permanent soil deformation. In other words, failure stress was defined as the yield stress of the soil. Specifically, the failure stress value for each test was estimated as the stress defined by the intersection of the tangents to the initial and final portions of the stress-settlement curve (Ismael 1996; Consoli et al. 1998).

Table 3 shows the failure stress values obtained for the moist tests (σ_f) and the average values for the inundated tests (σ_{fi}). The ratio between σ_f and σ_{fi} is also shown in Table 3. Regarding SML test results, a small increase in soil suction from 0 to 10 kPa leads to a failure stress twice as high as the failure stress corresponding to zero suction. This shows that bearing capacity in lateritic soils is significantly influenced by small soil suction variations. However, bearing capacity in soils with a denser fabric, for which general shear failure may dominate, is expected to be influenced to a lesser extent by small soil suction variations. Increase in failure stress due to soil suction was slightly smaller for QML tests in comparison to SML tests.

A prediction of the failure stress for the tested soil is also given in Table 3 for comparisons with the experimental results. The following bearing capacity formulation was used in this analysis:

$$\sigma_p = \gamma\, H\, N_q\, \zeta_q + 0.5\, \gamma\, B\, N_\gamma\, \zeta_\gamma + \{c' + (u_a - u_w) \tan\phi^b\}\, N_c\, \zeta_c \quad (3)$$

where

σ_p = predicted failure stress,
H = depth of the bearing plate,
N_q, N_c, N_γ = bearing capacity factors (estimated according to Vesic 1973), and
$\zeta_q, \zeta_\gamma, \zeta_c$ = shape factors (estimated according to Vesic 1973).

The required shear strength parameters were obtained from suction-controlled triaxial tests carried out on undeformed samples collected at a depth of 2 m at the test site (Machado 1998). The following values, reduced to account for punching failure (Terzaghi 1943), were used in the calculations: $\phi' = 20.3^0$, $c' = 0$ kPa, and $\phi^b = 10.8^0$. A soil unit weight of 15 kN/m^3 was assumed (see Fig. 2). As indicated in Table 3, Eq 3 provides highly unconservative previsions of bearing capacity for the tested system, about five times as high as the experimental results of the inundated tests and about three times as high as the experimental results of the moist tests.

TABLE 3—*Failure stress for moist and inundated tests.*

Test Type	Suction, kPa	σ_f, kPa	σ_f/σ_{fi}	σ_p, kPa[b]	σ_p/σ_f
SML	0	45[a]	1	223	5.0
	10	92	2.0	264	2.9
	30	118	2.6	347	2.9
QML	0	51[a]	1	223	4.4
	15	78	1.5	285	3.6
	22	98	1.9	314	3.2
	31	122	2.4	351	2.9

[a] σ_{fi} (average value of inundated tests).
[b] Estimated using Eq 3.

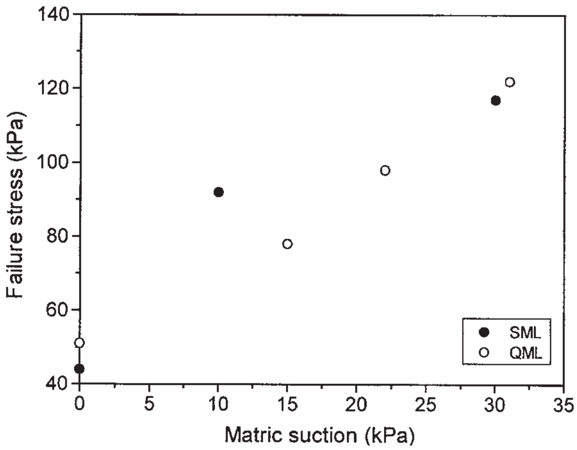

FIG. 9—*Behavior of σ_f with matric suction for the SML and QML tests.*

The increase of σ_f with matric suction is illustrated in Fig. 9. A nonlinear trend was obtained for the SML tests, while a nearly linear trend was obtained for the QML tests. However, the large background of laboratory test results has demonstrated that shear strength of different soils increases nonlinearly with matric suction (Delage et al. 1987; Escario and Saez 1987; Gan and Fredlund 1995; Teixeira and Vilar 1997). Therefore, a potential function was used to fit the failure stress results as a function of matric suction. No distinction was made regarding the test type (SML or QML). The following expression was obtained, with $R^2 = 0.92$:

$$\sigma_f = 48.1 + 6.2\, (u_a - u_w)^{0.70} \quad (4)$$

Experimental results have been used to develop unified frameworks to explain the behavior of unsaturated soils. Matric suction has been considered an independent stress variable in these models, for which the yield surfaces are defined using concepts of hardening plasticity. Based on the loading-collapse yield surface proposed by Alonso et al. (1987), Eq 4 can be used to represent the locus of stress variables σ_f and $(u_a - u_w)$. Whenever this yield surface is reached, either by a decrease in suction or an increase in applied stress, plastic deformations take place within the unsaturated soil mass.

There is also evidence that an increase in suction levels not previously reached by an unsaturated soil induces irrecoverable volumetric strains. A suction increase yield surface (SI) is also proposed for this loading scenario (Alonso et al. 1987). This surface represents an upper limit beyond which increases in suction will cause plastic deformations within the unsaturated soil mass. Tensiometers installed 0.5 m below the ground surface for a period of several years have rarely recorded matric suction values exceeding 50 kPa. Also, it is unlikely that suction in the soil mass has ever experienced suction levels above 50 kPa below a depth of 1.5 m. Consequently, a suction value of 50 kPa is assumed to define the location of the SI yield surface. Figure 10 shows the proposed in situ yield surfaces, which define the elastic (inner) and plastic (outer) regions of the tested soil-plate system.

However, it should be emphasized that a relationship such as the one represented in Eq 4 needs to be used with caution for design purposes. As previously mentioned, suction within the soil

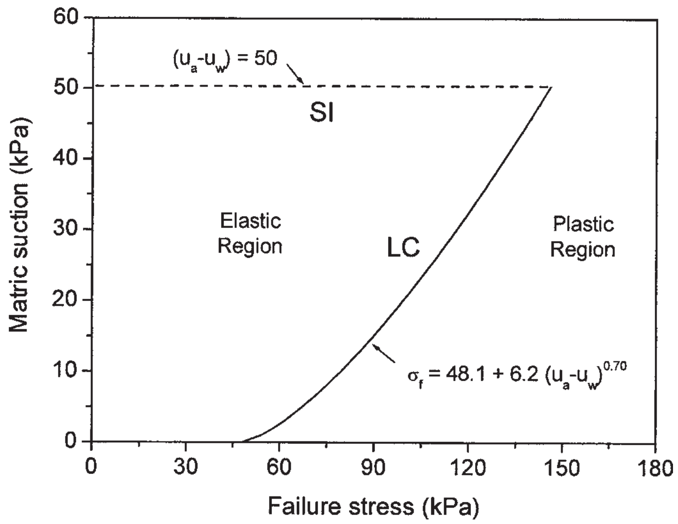

FIG. 10—*LC and SI surfaces for the tested soil-plate system.*

mass experiences seasonal variations according to the microclimate conditions of the area. Therefore, the suction value used to determine σ_f needs to be the lowest value ever recorded in the site, reduced by an appropriate factor of safety. The decision concerning the suction value to use depends upon local experience and the microclimate condition in a particular region (Fredlund and Rahardjo 1993). Specifically, a design suction value higher than zero is not recommended in the region where the load tests were conducted, because periods of intense precipitation occur throughout the year.

Settlement Analysis

The results from the plate load tests also provide insight on the settlement rate under constant loading. At each load increment stage the applied load was kept constant until settlements stabilized according to the criterion defined by Eq 2 for SML tests, and according to the Fellenius (1975) criterion for QML tests. Settlement readings were collected in predefined time intervals and were used to define settlement *versus* time plots for each loading stage.

The slope β of the settlement-time curves, plotted in a semi-logarithmic scale, is a representative parameter of the average settlement rate at each loading stage. A comparatively high value of β indicates a high settlement rate during the loading stage.

Figures 11 and 12 show the settlement rate index β as a function of the applied stress for the SML and QML tests, respectively. As shown in the figures, the settlement rate index β is highly influenced by the stress level of the loading stage and by the soil suction. Specifically, the settlement rate index β increases with increasing applied stress and with decreasing soil suction. SML inundated tests lead to higher β values than those obtained for moist tests (Fig. 11). This trend is more pronounced at higher applied stress levels. Although slightly less pronounced, a similar trend is observed for the QML tests (Fig. 12). The results shown in Fig. 12 for the QM2 test (suction of 22 kPa) were expected to lie between the results obtained for the QM1 test (suction of 15 kPa) and the QM3 test (suction of 31 kPa). However, the estimated β values for the QM2 test show some discrepancy with the expected trend and are not considered in the analyses presented below.

The relationship between β and matric suction at stress levels (σ) of 40, 80, and 120 kPa is shown in Fig. 13. Data points corresponding to zero suction and applied stress of 120 kPa were obtained by extrapolation of results in Figs. 11 and 12, as this stress level was not reached in the inundated tests. Trend lines fitting the experimental data are also shown in Fig. 13. The figure shows that the settlement rate index β decreases with increasing suction for all stress levels. The settlement rate becomes approximately independent of soil suction at the stress level of 40 kPa. As observed in Fig. 10, only elastic deformation takes place in the soil below a stress of 40 kPa regardless of suction variations. As a consequence, settlements will not be influenced by soil suction at this stress level. The same behavior can be observed for the curves corresponding to stress levels of 80 and 120 kPa for suction values above approximately 10 and 32 kPa, respectively. Plastic deformations occur in the soil for suction values below these levels, so that variations of β with suction become evident.

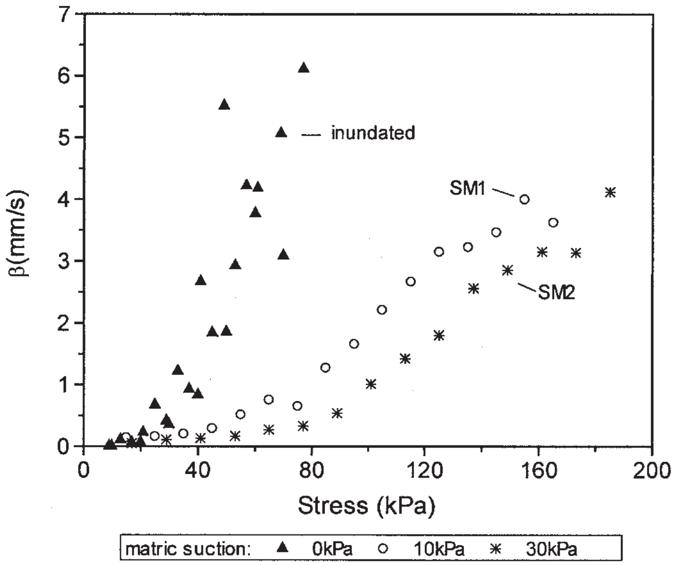

FIG. 11—*Settlement rate as a function of stress (SML tests).*

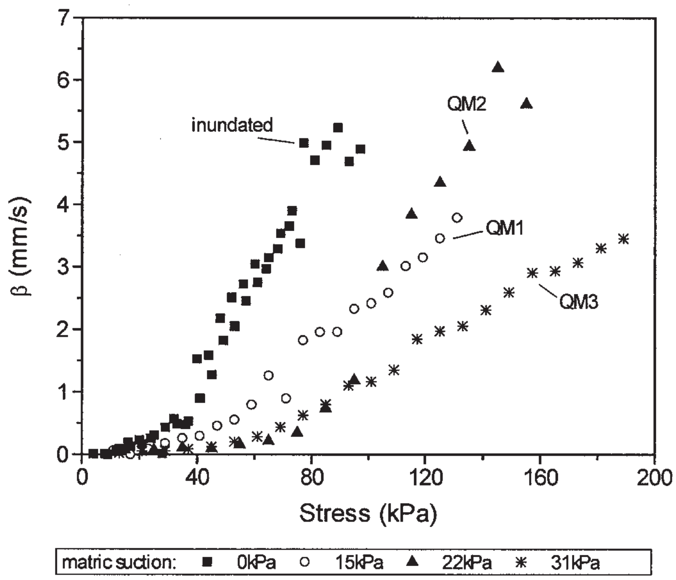

FIG. 12—*Settlement rate as a function of stress (QML tests).*

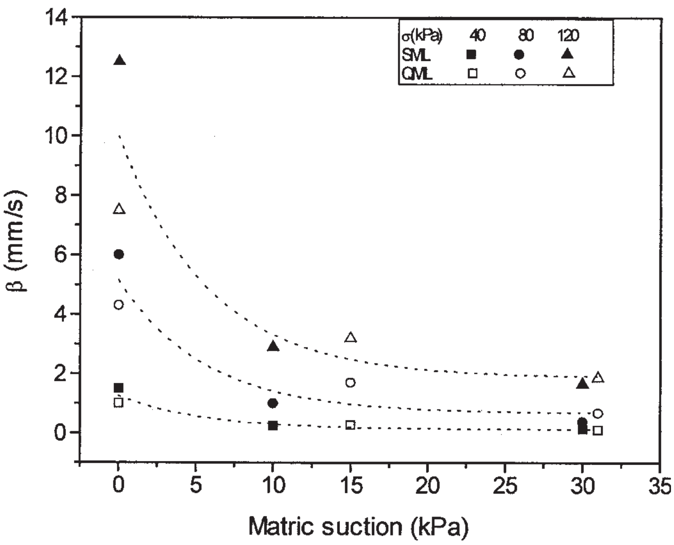

FIG. 13—*Relationship between* β *and matric suction.*

Conclusions

A testing program involving plate load tests was conducted in an unsaturated lateritic soil deposit. Ten plate load tests were performed under different soil suction conditions using a bearing plate 0.8 m in diameter placed at a depth of 1.5 m. Soil matric suction was monitored during the tests using tensiometers installed at the bottom of the testing pit.

The analysis of the results showed that the response of soil-plate systems are highly influenced by the magnitude of soil matric suction in lateritic soils. A small increase in suction from 0 to 10 kPa led to an increase of approximately 100 % in the failure stress of the soil-plate system tested herein. The response of failure stress to matric suction is proposed to be represented by a loading-collapse (LC) yield surface defined by a potential function based on in situ test results. A suction-increase (SI) yield surface, defined by suction history at the site, is also proposed. Together, both yield surfaces define the elastic (inner) and plastic (outer) regions of the tested soil-plate system.

The rate of settlement shows a non-linear decreasing response with increasing soil suction. For a certain applied stress level, it was verified that settlement rate increases with decreasing soil suction. The influence of soil suction is more significant for comparatively high applied stress levels and for comparatively low soil suction values.

Insight into the influence of soil suction is particularly relevant for proper interpretation of field test results conducted on lateritic soil deposits. Such results can be useful in the implementation of models for settlement and bearing capacity predictions that take into account the effect of soil suction.

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