Peak and residual shear strength parameters of soft clay-concrete interfaces

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ABSTRACT

The peak and residual shear strength parameters of soft clay-concrete interfaces are relevant for the analysis of soil-structure interaction problems in the lacustrine environment of Bogotá City. These parameters were evaluated using the direct shear test in consolidated-drained conditions. In these tests, interfaces with different roughnesses were implemented in the direct shear box and subjected to medium and large displacements in post-rupture cycles, to date the peak and residual strength parameters. The results are analyzed in light of a roughness coefficient (R), in order to study their influence on shear strength parameters. It was found that the peak and residual strength angles on the interfaces are almost independent of this variable, while the cohesion intercept at peak states is similar to undrained adhesion estimated using the previously defined (a) factor, for all cases. The results represent a contribution to the specific knowledge in Colombian practice, and might guide further research in this topic.

RÉSUMÉ

Le pic et résiduelle des paramètres de résistance au cisaillement de l'argile molle interfacés en béton sont pertinentes pour l'analyse des problèmes d'interaction sol-structure dans l'environnement lacustre de Bogotá City. Ces paramètres on été évalués en utilisant le test de cisaillement direct dans des conditions consolidés drainés. Dans ces tests, les interfaces avec rugosités différentes ont été mises en œuvre dans la zone de cisaillement direct et soumis à des déplacements moyennes et grandes entreprises dans les cycles post-rupture à ce jour le sommet et les paramètres de résistance résiduelle. Les résultats sont analysés à la lumière d'un coefficient de rugosité (R), afin d'étudier leur influence sur les paramètres de résistance au cisaillement. Il a été constaté que le pic et les angles de résistance résiduelle sur les interfaces sont pratiquement indépendantes de cette variable, tandis que l'interception de cohésion au niveau des États de pointe est similaire à l'adhésion non drainé estimée au moyen du préalablement défini (a) des facteurs, pour les trois cas. Les résultats représentent une contribution à la connaissance spécifique en Colombie, et pourrait orienter les recherches dans cette rubrique.

1 INTRODUCTION

The interaction between soils and structural elements is a critical problem in geotechnical engineering (Gomez, 2000). The design of structural elements in contact with ground is governed by the stresses and strains induced by the mechanical response of this geological material, of uncontrolled quality and formed by nature. In soil-structure interaction, the friction generated on interfaces is a key variable in modelling such problems.

In the lacustrine environment of Bogotá City (Colombia), the performance of many geotechnical structures depends critically of their interactions with clayey soft soils, which is part of mechanisms poorly studied.

In this environment, quantification of the friction of the interfaces between concrete (considered the most common building material in Colombia) and soft clayey soils has a great importance in the understanding and numerical simulation of urban geotechnical problems as the induced deformations by excavations, settlements of deep foundations as a function of friction loads, and the bearing capacity of pipelines, among others.

This paper presents a critical review and the analyses of the results of an experimental investigation conducted to assess the variation of shear strength parameters of concrete -soft clay interfaces as a function of concrete side roughness, using the direct shear test (after Pineda & Montejo, 2005).

2 BACKGROUND

Several authors have mentioned that a valid approach in the application of analytical methods involving the shear strength parameters of interfaces is to assume the angle of resistance as a percentage of the peak angle of the soil (equation 1):

$$\delta = (2/3)\ \varnothing \qquad [1]$$

Where δ=friction angle of the interface
$\varnothing$ = peak shear resistance angle of the soil

However it is evident from field observations that this variable (interface friction represented on –δ- angle) is largely dependent on soil type and material roughness of the geotechnical structure (Hashash & Finno, 2008).

In Colombia, normal design practice is restricted typically to the application of equation 1 in most practical cases. The reassessment of some results previously presented by Pineda & Montejo (2005), which gave a first approximation to the peak strength parameters of soft clay-concrete interfaces (occurring in at least 90% of geotechnical structures in Bogotá City), are shown below.

2.1 Peak and Residual Shear Strength of Soils

Some authors (Terzaghi & Peck 1987; Mitchell & Soga, 2005) define the peak shear strength as that resistance available in the mineral soil skeleton before permanent shear deformations occur on a preferential rupture plane. The micromechanical approach proposed by Santamarina (2001) suggests that this peak resistance in soils is lost when irreversible displacements of mineral grains occur (this approach is valid principally in granular materials, although some authors such as Burland, 1973; Holtz & Kovacs, 2010; and Leon-Resendiz, 1989; mention its applicability to normally consolidated clays also).

This peak resistance is essentially of frictional nature even in fine-grained soils (Mitchell y Soga, 2005), but in some cases, there are additional forces that provide an apparent or a true cohesion between particles (i.e cemented soils, capillary forces in partially saturated soils, van deer walls forces in silty/clayey soils and weakly bonded soils and rocks).

The peak shear strength for design purposes is expressed by the Mohr-Coulomb failure criteria (equation 2):

$$\tau_f = \sigma_f \tan \phi + c \qquad [2]$$

In [2], σ_f is the normal stress applied on the failure plane, ϕ represents the angle of resistance (inclination of a lineal failure envelope) and c is the cohesion intercept. The last parameter has been largely discussed from the focus of its physical meaning, however, in most cases, its presence in a given material is considered as a contribution to the resistance for practical purposes.

The residual shear strength, in turn, is defined as one that persists in the soil skeleton after large deformations/displacements on the rupture plane have occurred, so the intercept cohesion is reduced to a null value and resistance has the minimum possible magnitude (Bishop et.al, 1972). In the residual condition there are only frictional mechanisms governing the available shear stress, independently of the material type.

Specifically in clayey soils, Gomez (2000) mentioned that the principal mechanism on shear behaviour of interfaces is the adhesion produced between the soil and the engineering material in the peak condition. However, there is little information available for the residual state.

2.2 Interface Shearing Resistance and its Impact on Geotechnical Analyses

The shear strength/friction on interfaces has been recognized as a fundamental variable in the analysis of many classical soil structure-interaction problems (i.e earth retaining structures stability, lateral earth pressures, quantification of arch effects, pipeline loads, friction loads on piles, etc). More recently, the friction between geotechnical materials and men made materials (like concrete, steel or geosynthetics) has been incorporated in advanced numerical analysis where their value controls the magnitude of predicted stresses and strains near the interfaces in boundary problems (Hashash & Finno, 2008).

With regard to the residual shearing resistance of interfaces, the study of this condition is important because many post-failure events in geotechnical engineering are governed by that magnitude. Among the most common cases of residual shearing conditions are the available strength in failure surfaces of landslides (after failure events) and the shaft resistance of driven piles.

For the above mentioned reasons, the determination of friction/shear strength parameters on interfaces is a critical activity to achieve a good convergence between geotechnical designs and the direct costs of infrastructure elements designed in contact with ground. This benefit-cost relationship is especially relevant in the lacustrine environment of Bogotá City, which is characterized by the presence of soft to medium, compressible clayey soils where the interaction of concrete structural elements with the ground is relatively unknown. Support this idea many locally reported cases of failure/deflections of structural members, inadmissible settlements during construction of infrastructure and damage caused during these activities to neighbouring structures and other components of physical environment (Pineda-Jaimes, 2010).

3 MATERIALS AND METHODS

Peak and residual shear strength of soft clay-concrete interfaces were evaluated using the direct shear test in consolidated-drained conditions. In these tests, a concrete surface of variable roughness was installed in one half of the shear box perpendicular to the shear stress direction (Pineda & Montejo, 2005). On the other half, an intact specimen of Bogotá soft clay was put carefully at natural water content. Below some specific aspects of the materials and methods used in this research are presented.

3.1 Soft Clay Properties

The clayey material was extracted from a site located at northern Bogotá area (Villa del Prado neighbourhood) in the soft soil zone (DPAE, 2010). Figure 1 shows the location of this site. The material was extracted using Shelby tubes of 3 inches diameter in a manual borehole of 6m depth, following the ASTM D-1587 standard. Table 1 presents the principal index properties and table 2 shows some physical properties of the soil studied.

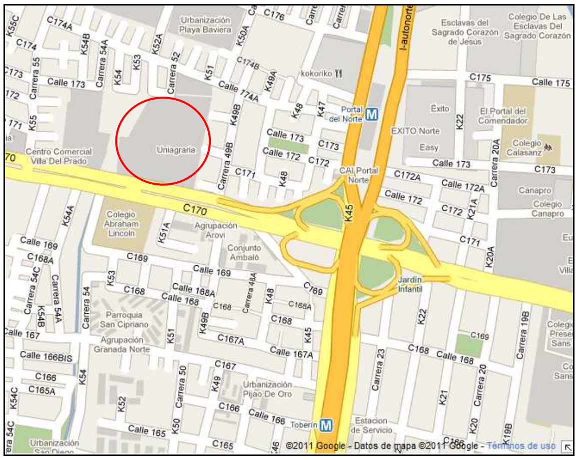

Figure 1. Investigation site. Red circle shows the location.

Sample	Depth (m)	WL (%)	WP (%)	WN (%)	IP (%)
1	1.75	NP	NP	160	NP
2	2.25	82	39	80	43
3	3.25	131	60	130	71
4	4.25	130	53	129	77
5	5.50	135	60	125	75

Table 1. Clayey material index properties

Sample	Depth (m)	IL (%)	γ_t (KN/m^3)	e_o	Cu (kPa) – VST-
1	1.75	-	-	-	-
2	2.25	0.95	15.5	2.1	15
3	3.25	0.99	14.2	3.2	12
4	4.25	0.99	14.1	3.4	13
5	5.50	0.87	14.0	3.3	20

Table 2. Clayey material physical properties

In general, the materials classify within the MH/CH groups according to USCS system. The in-situ void ratio (e_o) range from 2.1 to 3.3, consistently with liquidity index variations (positive values close to 1.0). The undrained peak strength deduced from field vane shear tests (VST following ASTM D-2473) varies between 12 to 20kPa. Values of index and physical properties suggest that the clayey material has soft to very soft consistency (Mitchell and Soga, 2005), high to very high deformability and high compressibility. These magnitudes are consistent with previous locally reported values (Ingeominas, 1998; DPAE, 2010) and the materials can be considered as normally consolidated.

3.2 Concrete Side Characteristics

The concrete side of the interfaces had a variable roughness, covered before the solidification with a mortar mixture during the curing process in the shear box (square of 50mm side and 1.25 mm thickness). The geometry of the irregularities was imposed via manual methods using scalpels and needles with known dimensions. Despite the roughness coefficient established initially by Pineda & Montejo (2005), which was useful as a first approximation, in this paper the roughness was recalculated using the criteria proposed by Degarmo et al., (2007), following equations 3 and 4. These expressions are framed in the ISO 25178 standard of surface roughness measurement in manufacturing processes.

$$R=Rn/L$$
[3]

$$Rn=(b_{prom}/H_{prom})$$
[4]

Where Rn is the ratio between the average base (b_{prom}) and average height (h_{prom}) of the irregularities and L is the separation between adjacent canals. Figure 3 shows schematically these characteristics. In this work; $b_i \leq L$ in all cases. Implicitly, the [R] coefficient assumes that the irregularities canals are rectangular; this can leads in a limitation of the proposed method as explained later.

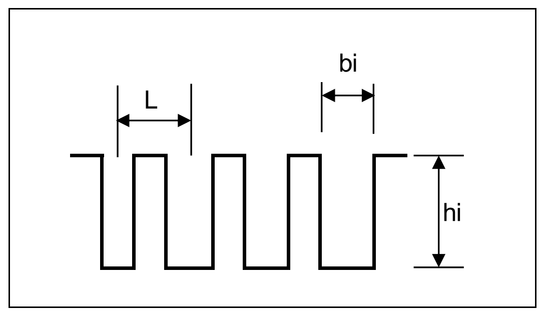

Figure 3. Geometric parameters of surface roughness

In table 3 the roughness coefficients of the three interfaces are presented.

Roughness	bi (mm)	Hi (mm)	L(mm)	R
1	0.500	0.500	0.500	2.00
2	1.000	0.500	1.000	1.67
3	1.500	1.500	4.000	0.25

Table 3. Surface roughness characteristics

3.3 Direct Shear Tests

Direct shear tests were executed following ASTM D-3080 standard, under drained conditions. The displacement velocity of each test was 0.01mm/min for the shear peak parameters and 0.1mm for the residual strength parameters. For the last condition, two reversal post-rupture cycles were implemented. In all cases, the shear stresses were applied at the end of primary consolidation after placement normal stresses of 50kPa, 100kPa and 150kPa.

4 EXPERIMENTAL RESULTS

Figures 4, 5, 6 and 7 show the experimental results of shearing resistance, peak and residual conditions, of the soft clay studied and the three interfaces implemented. The clay used was that from 3.25m depth (see Table 1).

The strength envelope of figure 4 presents a peak resistance angle near 17°, with a negligible cohesion intercept.

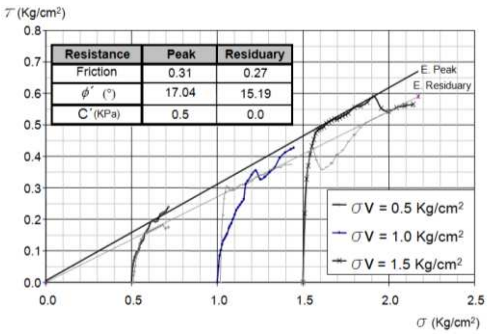

Figure 4. Peak and residual strength parameters of Bogotá soft clay.

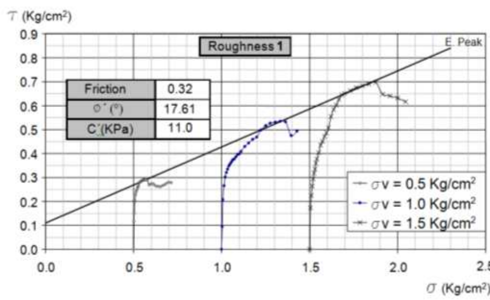

Figure 5. Peak and residual strength parameters of concrete-soft clay interphase (Roughness 1)

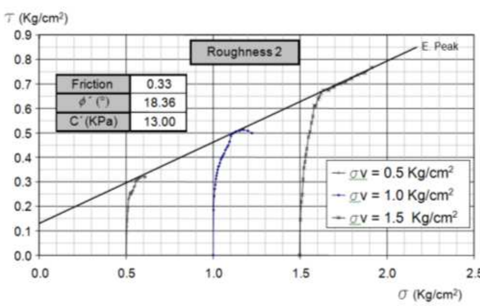

Figure 6. Peak and residual strength parameters of concrete-soft clay interphase (Roughness 2)

The results are consistent with those previously published by Moya & Rodriguez (1987), DPAE (2010), for normally consolidated Bogotá clays. According to Mitchell & Soga (2005), the stress history of normally consolidated materials does not result in the evolution of digenetic links between particles that can lead to true cohesion intercepts in Mohr-Coulomb representations. This is the case for the materials studied.

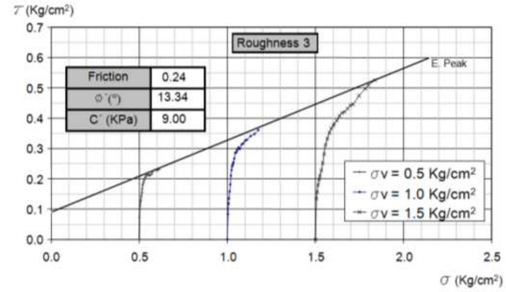

Figure 7. Peak and residual strength parameters of concrete-soft clay interphase (roughness 3)

Results shown in figures 5 and 6 for the two highest roughnesses suggest that the peak resistance angle is very similar in magnitude to the intact soft clay. Nonetheless the cohesion intercept is greater than zero and it varies from 11kPa to 13kPa.

For the interface of the lowest rugosity (figure 7), the peak resistance angle is at least 4° smaller than that of the intact soil and at the same time is slightly smaller than those established for the other two roughnesses. For this third case, the cohesion is slightly smaller also (9kpa). This fact can be interpreted in terms of the increase of (b) parameter (according to figure 3) in the concrete face, this feature leads to a greater contact area with a smooth surface with the clayey soil. For this reason, it is possible that in relatively smooth portions of the concrete face the frictional mechanism is reduced, explaining in part the declining tendency of the peak resistance angle.

In table 4 the results of shearing resistance at peak states are presented and table 5 shows the results of residual states.

Roughness	Peak angle of shearing resistance	Intercept cohesion (kPa)
1	17.6	11
2	18.3	13
3	13.3	9

Table 4. Peak strength parameters for clay-concrete interphases

Roughness	Residual angle of shearing resistance	Intercept cohesion (kPa)
1	13.1	0
2	13.5	0
3	12.7	0

Table 5. Residual strength parameters for clay-concrete interphases

As expected, in residual states the cohesion is null independently of the initial roughness of the concrete side.

According to Mitchell & Soga (2005), the well-crystallized particles of kaolinite/smectite argillaceous minerals have an average length variable from about 0.1μm to 4μm. In this case, Bogotá clay is predominantly kaolinite/illite-type mineral (Ingeominas, 1998; Pineda-Jaimes, 2003). Note that [b_i] dimension of equation [3] is significantly greater than this particle size, therefore it can be assumed that most of the canals in the irregularities on the three concrete faces are full of clay particles in both peak and residual states. This fact could explain, in part, why the angles of shear strength are fairly constant after large displacements. Figure 9 shows this tendency in terms of Tan (d) – tangent of shear resistance angle at interphase- as a function of (R) parameter (dotted line shows the residual state tendency).

In figure 9, it is evident that the shearing angle is more dependent on surface roughness at peak states confirming the observations mentioned above from figures 5, 6 and 7. However, the variation of the value of this angle is not significantly large, so it is possible that the frictional mechanism originally considered for the interpretation of results is not entirely applicable in clayey soils.

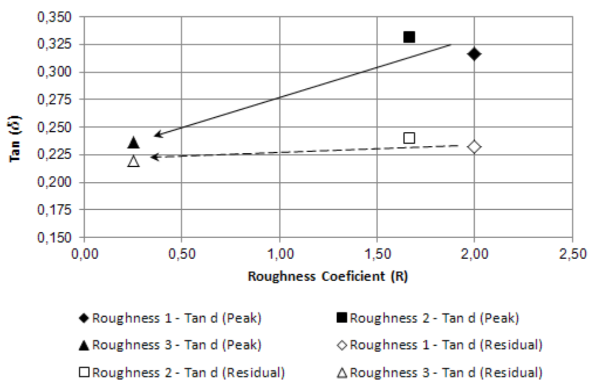

Figure 9. Relationships between Tan (d) and (R) factor in peak and residual conditions.

Conducting a review of the experimental results by using equation [1], it is observed that the ratio 2/3 does not hold for the studied interfaced materials. In contrast, this ratio varies between 0.85 and 1.00. The results then could be interpreted considering that the main mechanism acting at the interfaces is the adhesion. This means that the cohesion intercept on shearing peak interface states, could be interpreted principally as an adhesion between the concrete side and the clayey soil, which is destroyed once the residual state is reached.

Adhesion is the property of matter by which two surfaces are joined when they come into contact and held together by intermolecular forces (Santamarina, 2001). Despite the physical meaning of adhesion, in geotechnical engineering this concept was previously investigated by many authors to study the relationship between interfaces of piles and the surrounding soil, assuming a total stress approach for clayey soils (Tomlinson, 1970; Poulos and Davis, 1980). The adhesion in those problems was defined as the ratio between the pile material – soil resistance (C_a) and the undrained shear strength of the clayey material following a Tresca approach (Cu for Ø=0). Equation [5] shows this parameter, called in literature (a).

a= C_a/Cu
[5]

In driven piles, (a) varies between 0.80 and 1.10 and in bored piles this factor can range from 0.88 and 1.05 for Cu values less than 20kPa (Poulos, 2005). Note that the results of undrained shear strength calculated for the material used in the direct shear tests from VST tests vary from 11.5kPa to 16kPa. It is interesting that the cohesion values at peak states obtained in figures 5, 6 and 7 are very close to the presented ranges of the adhesion coefficient. Probably, this tendency could confirm that the adhesion mechanism is the most important factor in concrete sides/soft clay interfaces, even under drained conditions.

Considering the results presented above, the following relationships were identified in concrete-soft clay interfaces:

$0.8 < (\delta_P/\emptyset_P) < 0.95$
[6]

$\emptyset_R \sim \delta_P \sim \delta_R$
[7]

$(\delta_R/\emptyset_R) = 1$
[8]

Where

δ_P is the angle of interface peak resistance
δ_R is the angle of interface residual resistance
$\emptyset_P$ is the angle of soil peak resistance
$\emptyset_R$ is the angle of soil residual resistance

5 SUMMARY AND CONCLUSSIONS

The reinterpretation of the experimental work and results described above, leads to the following main conclusions:

1. The ratio 2/3 of the peak angle of shearing resistance does not apply in soft Bogotá clay/concrete interfaces.
2. The cohesion intercept of the interfaces is completely destroyed at large deformations, independently of the initial concrete side roughness.
3. Peak angles of shearing resistance of interfaces (δ_P) vary from 0.8 and 0.95 times the peak angle of the soil ($\emptyset_P$).
4. Residual angles of shearing resistance (δ_R) of interfaces and residual angle of intact soft clay ($\emptyset_R$) are almost very similar.
5. Residual angles of shearing resistance are approximately 0.8 times the peak angle of interface, independently of the initial roughness.
6. It is postulated that the main mechanism of interaction between interfaces involving soft cohesive soils, is adhesion instead of friction even in drained cases.

The results presented are novel in the local context and need to be validated by more rigorous tests and from the analysis of in-situ test results. There are some limitations in the presented results:

a. The tendencies observed are deducted from a limited number of tests. It is possible that other tendencies could be established if more results are incorporated to the analysis.
b. The influence of over-consolidated states of the clayey material is not taking account on the shear response.
c. In the same way, the influence of pore water pressure increments is not analyzed during shearing.
d. The roughness calculated does not take into account the microscopical scale, so these values could be apparent.

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