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CONSTITUTIVE MODELS PREDICTING THE RESPONSE OF CLAYS ALONG SLIP SURFACES

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ABSTRACT

The paper proposes and validates a constitutive model simulating the change of resistance along clay slip surfaces under both undrained and drained conditions. The proposed model is based on (a) the critical state theory and (b) the assumption that the critical state changes once failure is reached, in terms of the further shear displacement. Under undrained conditions, the proposed model simulates the excess pore pressure generation and subsequently the continuous change of clay resistance along the slip surface from its initial value to the peak strength and then at large displacement the residual value measured in constant-volume ring shear tests. Under drained conditions, the model simulates the normal displacement change and subsequently the change of clay resistance along the slip surface in clays as measured in drained ring shear tests.

INTRODUCTION

Earthquake-induced slides usually occur due to generation of large excess pore pressures along slip surfaces (Sassa et al., 1996, Stark and Contreras, 1998). The sliding-block model (Newmark, 1965) is frequently used to predict the displacement of these slides (Kramer, 1996, Modaressi et al., 1995, Stark and Contreras, 1998). The sliding-block model is generally successful in estimating small ground deformations without earthquake-induced loss of strength (Whitman, 1993). However, when the ground deformations are large, this model is not accurate, primarily because of earthquake-induced loss of strength in saturated soils along a slip surface. Constitutive equations coupled with the sliding-block model are needed in order to simulate the triggering of the slides and predict accurately the seismic displacement.

The best laboratory device simulating soil response along slip surfaces is the ring shear test. In this test, similarly to field conditions, relative displacement at the slip surface can be very large, larger than a few centimeters, or even meters. Under cyclic loading, ring-shear tests show that the virgin shear stress-displacement soil response is almost identical to the response under monotonic loading (Trandafir and Sassa, 2005). In saturated soils, along slip surfaces, large excess pore pressures develop after failure is reached. Failure is defined as any state on the failure line in the effective normal stress - shear stress space. Large excess pore pressures after failure in saturated sands are associated with grain crushing (Sassa et al., 1996), while in saturated clays they are associated with a collapse of the soil structure and forced orientation of some clay particles parallel to the direction of shear (Idriss 1985). These excess pore pressures cause a post-failure decrease in shear strength (Stark and Contreras, 1996, Sassa et al, 1996). The residual shear strength is reached after many centineters or meters of shear displacement (Trandafir and Sassa, 2005, Igwe et al, 2007, Stark and Contreras, 1998). Under drained conditions, along slip surfaces, for either sands or clays, the residual shear strength is reached just after a few milimeters, while the normal contractive displacement continues to accumulate with shear displacement for centimeters or even meters (Stark and Eid, 1994, Igwe et al, 2007).

Constitutive models of various complexities simulating the response of localized discontinuities in saturated soils have been proposed and are used in both finite element codes and sliding-block models. In the context of numerical approaches using finite elements, if the interface is to represent strain at localized or failure zones in a soil medium, the constitutive model of the interface should be compatible and derived from the constitutive model of the surrounding materials. Based on this concept, Aubry et al. (1990) relate stresses to displacement along and normal to the slip surface. The model may simulate adequately the soil response towards failure, but may not simulate the post-failure shear strength loss because of large excess pore pressures, described above.

In association with the sliding-block model, Sarma (1977) proposed a simplified model predicting pore pressures build-up based on Skempton's A and B parameters. A more elaborate model predicting the continuous change of resistance along a slip surface has been developed by Modaressi H. et al. (1995). This model is based on the elasto-plastic model by Aubry et al. (1993) and relates shear stress to shear displacement, by introducing only five parameters. Recently, Gerolymos and Gazetas (2007) proposed a constitutive model for grain-crushing-induced pore-water pressures of sands. The model extends Hardin's theory for crushing of soil particles and has 12 parameters. A model predicting the response of clays along slip surfaces under both undrained and drained conditions, calibrated with soil response measured in ring shear tests, was not found in the literature.

The purpose of the paper is to propose and validate a constitutive model simulating the change of resistance along a clay slip surface with shear displacement both under undrained and drained conditions. The proposed model will be validated by the prediction of ring shear test results on clays found in the literature. The proposed model will simulate only soil response under static loading, consistently to the observation indicated above by Trandafir and Sassa (2005) that under cyclic ring shear tests, the virgin stress-strain response is almost identical to the response under monotonic loading. The proposed model will simulate clay behavior in terms of the Overconsolidation ratio (OCR).

SOIL RESPONSE

Laboratory devices

Soil response under shearing is measured in a variety of laboratory devices: triaxial, direct-shear, simple shear and tortional shear. Unlike the triaxial cell, in the devices of direct-shear, simple shear and tortional shear, similarly to the slides in the field, the direction of loading is parallel to the slip surface. From these devices, only in the ring shear device the displacement can be large, larger than a few centimeters or meters, as happens in situ. It is inferred that the most representative device to measure the response at a slip surface is the ring shear device. Ring shear devices have been constructed by different researchers (Stark and Contreras, 1996, Wang et al. 2007). For the study of the undrained response of saturated soil, these devices retain a constant volume in the soil sample (Stark and Contreras, 1996, Wang et al. 2007). Fig 1 compares the shear stress - shear displacement relationship and the pore pressure - horizontal displacement relationship on Drammen clay with confining effective stress σ'_o=255kPa measured in the direct simple device with that measured in the ring shear device (Stark and Contreras, 1996). It can be observed that the undrained peak shear stress is similar, but there is a difference in the shear displacement at which the peak stress is mobilized. This difference in shear displacement is attributed to the nonuniform shear displacement across the ring shear specimen (Stark and Contreras, 1996).

Response of clays under undrained conditions

Undrained clay response towards peak strength is well-known and has been measured in simple-shear tests (Ladd and Foot, 1974, Andersen et al, 1980). Fig. 2 gives typical shear stress-shear strain response measured in simple-shear tests on Boston Blue clay and Drammen clay. Test results show that the response can be normalized in terms of the OCR and the initial vertical effective stress. For normally consolidated clays, as deformation increases, shear stress increases and pore pressure starts to build-up. Peak strength occurs when the failure line is reached, at shear strain 5 to 15%. For over-consolidated clays, negative pore pressures are generated. Fig. 2b gives the ratio of the normalized peak undrained soil strength in terms of OCR for different clays.

Fig. 3 gives the shear stress and pore pressure versus displacement results of a constant-volume direct shear test on Boston Blue Clay (Taylor, 1952). Table 1 gives the characteristics of the soil and the loading condition. In table 1, u is shear displacement, σ'_o is the initial vertical effective stress, σ'_p is the maximum past effective vertical stress, τ_m is the maximum shear stress and τ_r is the residual shear stress. It can be observed that the peak strength occurs and the failure line is reached at shear displacement 1mm. Once peak strength is reached, pore pressure continues to increase at a decreasing rate with displacement, while shear stress also decreases at a decreasing rate. An undrained residual condition is not reached in the test because of the limited shear displacement allowed by the direct shear apparatus.

Constant-volume ring shear tests of six clays under monotonic loading are reported in the bibliography (Stark and Contreras, 1996, 1998). Figs 4 to 6 plot the results. Table 1 gives the main classification characteristics of the soils and the results. Unfortunately, different data is available for each case, as described in table 1. Only in one test of Drammen clay shear stress and pore pressure from very small to very large displacements are given. Yet, all test results illustrate that after the peak shear resistance is reached, at shear displacement of about 1mm, as shear displacement increases further, generation of additional excess pore-water pressures occurs, presumably as a result of a collapse of the soil structure. This generation of excess pore-water pressure results in a decrease in the effective stress towards a residual shear strength condition along the failure line. The residual shear strength is reached at shear displacement 10 to 130mm.

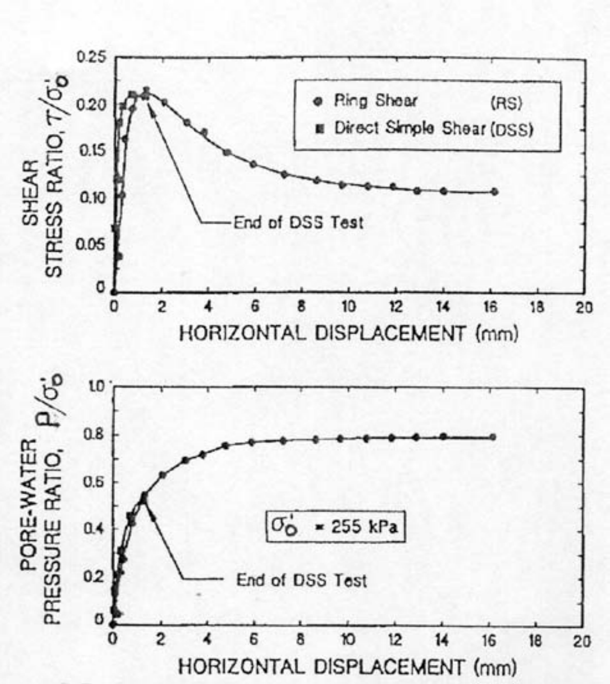

Fig. 1. Comparison between response measured in constant-volume ring shear and simple shear tests in Drammen clay (Stark and Contreras, 1996).

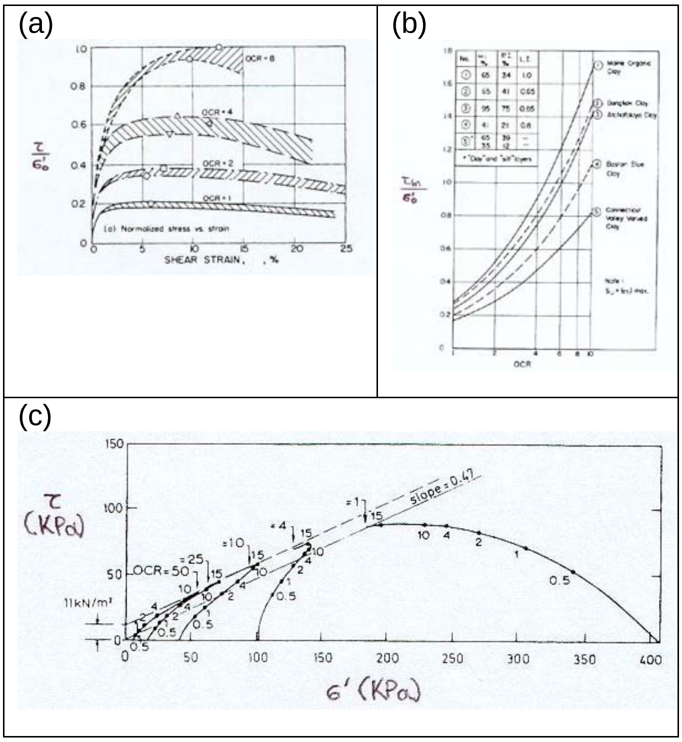

Fig. 2. Typical shear stress-shear strain-pore pressure response in undrained simple shear tests. (a) The case of Boston Blue clay (Ladd and Foot, 1974). (b) The ratio of the normalized peak undrained soil strength in terms of the OCR value for different soils (Ladd and Foot, 1974). (c) The case of Drammen clay (Andersen et al, 1980).

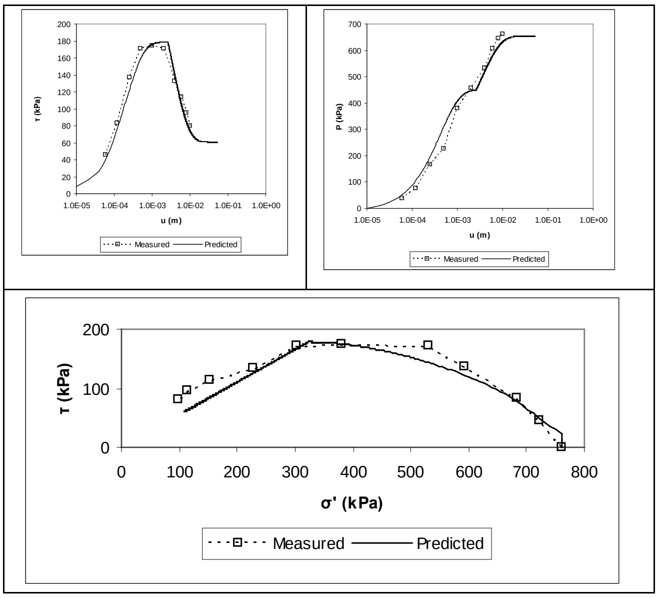

Fig. 3. Response measured at constant-volume direct-shear test on Boston Blue clay (Taylor, 1952). The predicted response is also given.

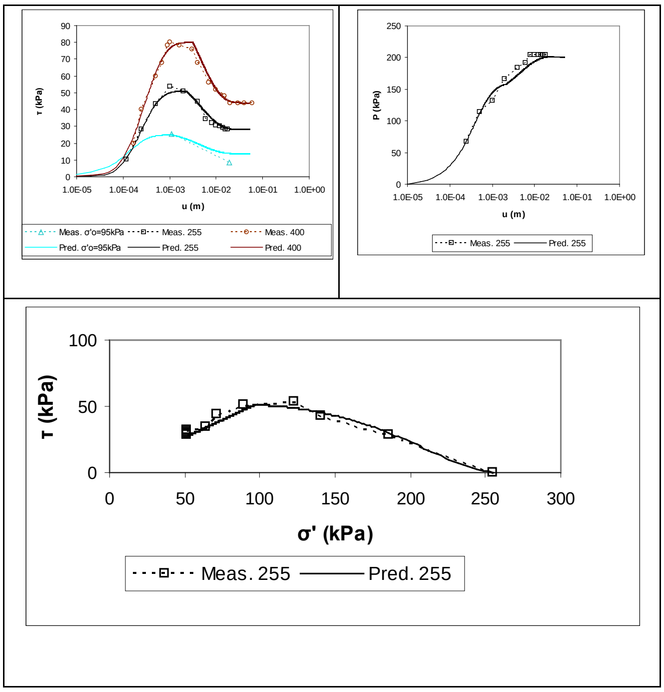

Fig. 4. Response measured in constant-volume ring shear tests on Drammen clay (Stark and Contreras, 1996). The predicted response is also given.

Table 1. Summary of (a) constant volume direct-shear test (Taylor, 1952), (b) constant volume ring shear tests (Stark and Contreras, 1996, 1998) and (c) drained ring shear tests (Stark and Stark and Eid, 1994) on clays considered in this study. Description of soil and partial results. Available data is also described.

No	Soil deposit	Liquid Limit (%)	Plastic Limit (%)	Clay size (%)	σ'_o (kPa)	σ'_p (kPa)	τ_m/σ'_o	u at τ_m (mm)	τ_r/σ'_o	u at τ_r (mm)	Available data
(a). Constant-volume direct-shear											
1	Boston Blue clay	46	24	-	760	<760	0.23	1.0	-	-	Shear stress and pore presure versus displacement but only till 7mm
(b). Constant-volume ring shear											
2a	Drammen clay, Norway	47	23	70	95	140	0.27	1.1	0.09	19	Only the values of peak and residual shear stress with the corresponding displacements
2b		48	24	72	255	140	0.22	1.3	0.11	16	Shear stress and pore presure versus displacement
2c		47	25	65	400	140	0.20	1.1	0.11	60	Shear stress versus displacement response
3a	Bootlegger Cove clay,4th Avenue – Alaska	40	20	59	100	300	0.28	1.2	0.07	55	The shear stress and pore pressure at displacement at and above the peak shear stress
3b		34	19	57	230	300	0.28	1.1	0.07	75	
3c		36	21	56	300	300	0.24	1.3	0.06	75	
3d		38	21	55	400	300	0.23	1.8	0.06	120	
3e		39	20	62	500	300	0.23	1.8	0.06	130	
4a	Bootlegger Cove clay, outside Fourth Ave. landslide	42	23	47	150	405	0.31	1.5	0.11	95	Only the values of peak and residual shear stress with the corresponding displacements
4b		40	21	42	225	405	0.32	1.6	0.10	110	
4c		42	23	49	400	405	0.31	1.7	0.11	125	
4d		41	22	45	500	405	0.30	1.7	0.11	140	
5a	Cohesive alluvium, Enid Dam, Enid, Mississippi	30	22	19	95.8	122	0.19	2.2	0.10	52	
5b		28	22	20	147	139	0.27	1.1	0.05	77	
5c		23	19	17	191	81	0.24	1.1	0.07	70	
5d		25	22	20	287	143	0.23	1.2	0.07	72	
5e		30	22	20	383	134	0.23	1.2	0.06	75	
6a	Cohesive alluvium, Jackson, Alabama	59	31	51	51.8	76	0.21	0.50	0.13	36	
6b		59	31	51	79.4	76	0.23	0.35	0.16	50	
6c		59	31	51	100	76	0.23	0.37	0.14	38	
7a	Upper Bonneville clay, Salt Lake City, Utah	46	23	33	47.9	48	0.32	0.30	0.11	39	
7b		46	23	33	95.8	96	0.36	0.60	0.15	25	
7c		46	23	33	191.5	191	0.31	1.2	0.12	29	
7d		46	23	33	383	383	0.34	2.0	0.14	36	
(c). Drained ring shear											
8a	Altamira Bentonic Tuff	98	37	68	60	700	0.33	0.20	0.27	1.00	The shear stress and normal displacement at displacement between 0.1 and 2.5cm
8b					120		0.22	2.00	0.22	2.00	
8c					240		0.18	4.00	0.18	4.00	
8d					480		0.13	1.00	0.13	1.00	
8e					850		0.10	1.00	0.10	1.00	

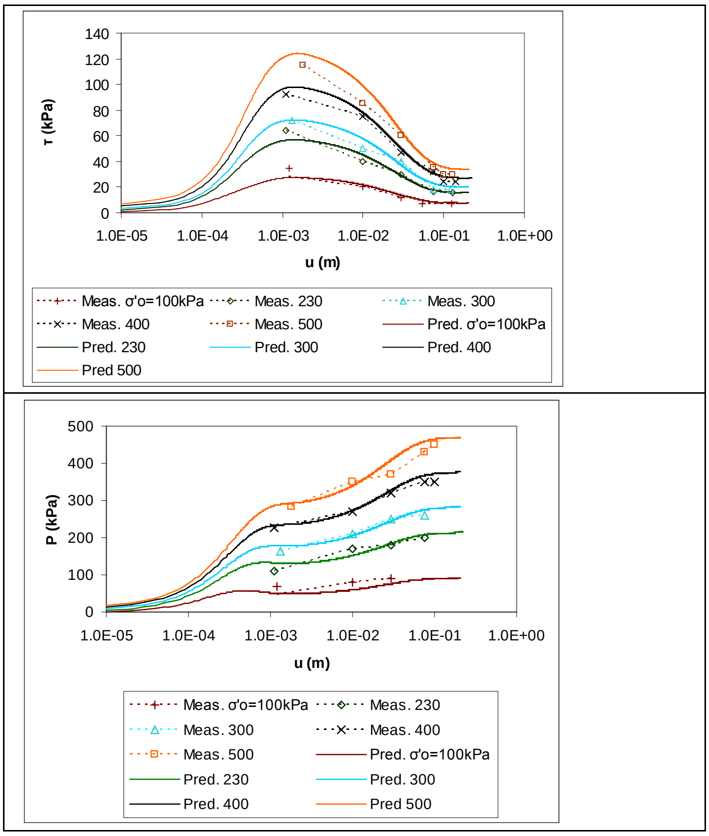

Fig. 5. Response measured at large displacement in constant-volume ring shear tests on the Bootlegger Cove clay from the 4th Avenue – Alaska slide (Stark and Contreras, 1998). The predicted response is also given.

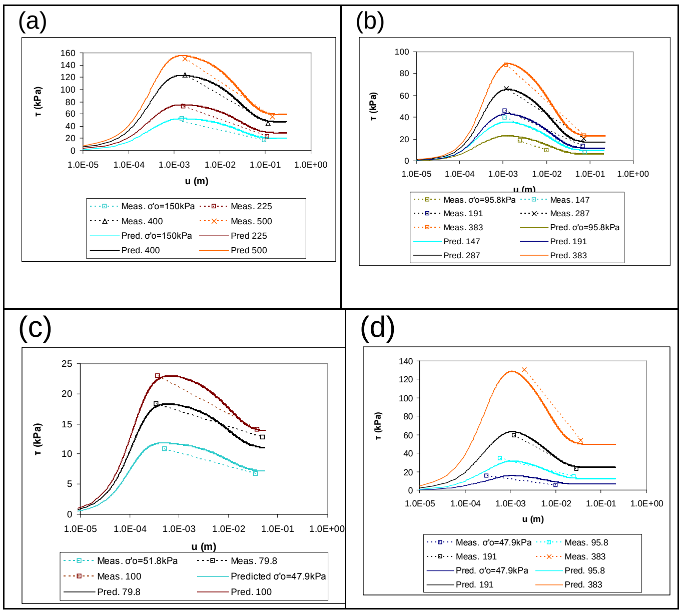

Fig. 6. Measured shear displacement and corresponding shear stress at the peak and residual values of shear stress in constant-volume tests in the clays (a) No. 4 (b) No. 5, (c) No. 6 and (d) No 7 of table 1 (Stark and Contreras, 1998). The predicted response is also given.

Response of clays under drained conditions

The most complete study of the response of clays under drained conditions at large displacement is reported by Stark and Eid (1994). Results of ring shear tests at different clays illustrate that at large shear displacement, the residual effective friction angle (φ'_r) reduces from about 30^o for clays with Liquid Limit (LL) of 30% to 6^o in clays with LL of 200% (Fig. 7a). At LL of 100% and Clay Fraction (CF) larger than 50%, the effective residual friction angle depends considerably on the effective stress (Fig. 7b).

Shear stress – displacement data is given for only one clay, the Altamira Bentonitic Tuff. A consolidation stress of 700kPa was chosen to represent the maximum effective stress. A shear surface was created in the overconsolidated soil by slowly rotating the top platen in the direction of shear. As illustrated in Fig. 8, for OCR smaller, or equal to 6, shear stress increases with shear displacement, and in 1 to 4 mm of displacement, the residual shear stress is reached in all cases. For the test with OCR=12, the peak shear strength occurs at shear displacement 0.3mm, while the residual shear stress develops at displacement 1mm. At larger displacements, the shear stress more-or-less does not change with further shear displacement, but considerable contractive normal displacement accumulates in all cases. Accumulation of contractive normal displacement decreases as the OCR value increases.

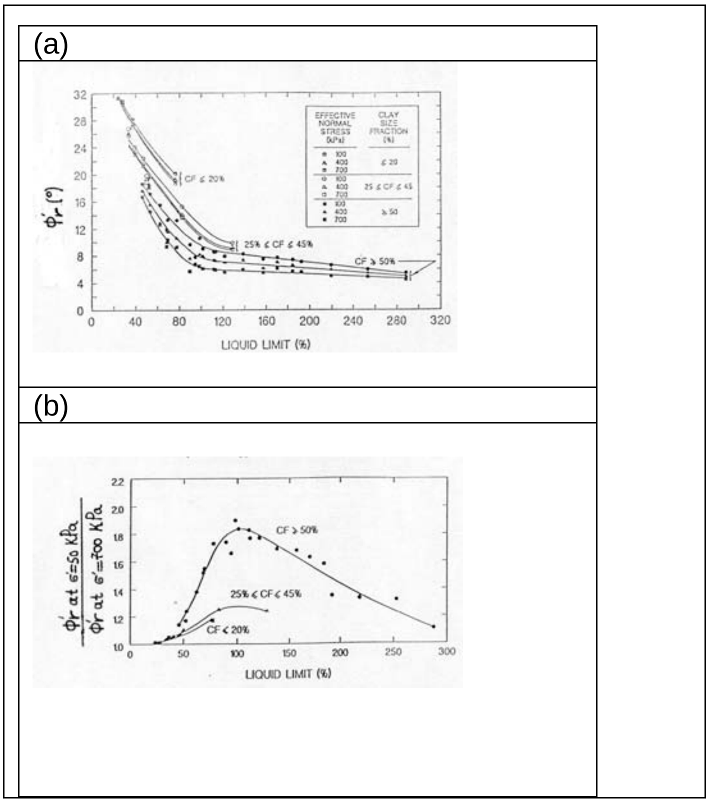

Fig. 7. Effective residual friction angle (φ'_r) measured in drained ring shear tests, in terms of the Liquid Limit, the effective stress and the clay size fraction (CF) for different clays (Stark and Eid, 1994).

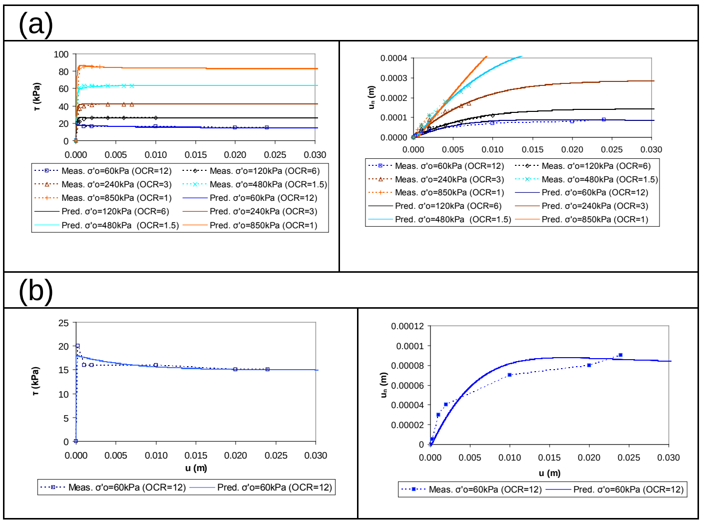

Fig. 8. Response measured in drained ring shear tests in Altamira Bentonitic Tuff (Stark and Eid, 1994). The predicted response is also given. (a) All tests, (b) detail giving only test with σ'_o=60kPa (OCR=12).

CONSTITUTIVE MODEL OF CLAY RESPONSE ALONG SLIP SURFACES

Proposed model

As earthquake-induced slides usually occur as a result of generation of large excess pore pressures, of primary importance is the response under undrained conditions. The following equations are used to simulate clay response under undrained conditions:

$$\tau = \tan\varphi'_{cs}\, \sigma'\, r\, f \qquad (1)$$

$$dr = du\, (1 - r^n) / a \qquad (2a)$$

$$f = 1 - \ln [\sigma'/(\sigma'_{cs})] \qquad (3)$$

$$\sigma'_{cs} = \sigma'_{cs\text{-}o} + (\sigma'_{cs\text{-}o} - \sigma'_{cs\text{-}f}) (1 - \exp(-\lambda\, u_{post})) \qquad (4)$$

$$dP = -d\sigma' = du\; k\; (\tan\varphi'_{dil} - \tau/\sigma') \qquad (5a)$$

where

$$k = k_1(\sigma'/Pa)^{k_2} \qquad (5b)$$

In addition,

$$u_{post} = \begin{cases} 0 & \text{if } u<u_o \\ u-u_o & \text{if } u>u_o \end{cases} \qquad (6)$$

Also,

$$\sigma' = \sigma'_o - P \qquad (7)$$

Application of equation (2a) requires knowledge of the initial (prior to shearing) value of the parameter r, r_o. The parameter r_o can be estimated from equations (1), (3) and (4) as:

$$r_o = \frac{\tau_o}{\sigma'_o \tan\phi'_{cs} [1 - \ln(\sigma'_o / \sigma'_{cs-o})]} \qquad (2b)$$

In the above equations τ is the shear stress, σ' is the effective normal stress, compressive positive, u is the shear displacement along the slip surface, r is a hysteretic dimensionless factor controlling the nonlinear effective shear stress ratio (τ/σ') versus u response of the soil, d implies differential, σ'_{cs} is the effective stress at the critical state (at the current density and shear displacement), f is a dimensionless factor that gives the effect of the distance from the critical state on soil response, k is the normal elastic stiffness, Pa is the atmospheric pressure, P is the excess pore pressure, u_o is the shear displacement when the failure state (defined as f≈1 and r≈1) is first reached, u_{post} is the post-failure shear displacement and τ_o and σ'_o are the initial vertical effective and shear stresses respectively. In addition, φ'_{cs}, φ'_{dil}, a, n, λ, k_1, k_2 are model parameters, described in detail below. Displacement is in m and stresses and pressures in kPa. The factor $\sigma'_{cs\text{-}o}$ is the effective normal stress at the critical state at the initial void ratio of the clay. It is estimated in terms of the OCR value as

$$\sigma'_{cs\text{-}o} = S1\, OCR^{S2}\, \sigma'_o / \tan\varphi'_{cs} \qquad (8a)$$

where S1 and S2 are model parameters.

The factor $\sigma'_{cs\text{-}f}$ is the effective normal stress at the final (residual) state. It is estimated as

$$\sigma'_{cs\text{-}f} = S3\, \sigma'_{cs\text{-}o} \qquad (9a)$$

where S3 is a model parameter.

Under drained conditions, equations (1), (2), (3), (6) do not change. Equations (4) and (5) are replaced by

$$\sigma'_{cs} = \sigma'_{cs\text{-}o} \exp(-\beta\, u_n) + (\sigma'_{cs\text{-}o} - \sigma'_{cs\text{-}f})(1 - \exp(-\lambda\, (u_{post}))) \qquad (10)$$

$$du_n = du\, (\tan\varphi'_{dil} - \tau/\sigma') \qquad (11)$$

In the above equations, u_n is the displacement normal to the slip surface, contractive positive, and β is a model parameter applicable for the drained case.

Discussion of the proposed model

Consistently with the description of clay behavior and the definition of failure above (section 1), the model divides soil response in two parts: (a) the part until failure is reached, at small shear displacement, less than a few millimeters, and (b) the post-failure part at large displacement. This division facilitates modeling, as part (a), unlike part (b), is measured in ordinary laboratory devices, such as the direct-shear and the simple-shear, and existing models can be utilized.

The proposed model for part (a) are equations (1) to (11) with equations (4) and (10) without their second term (Note that as in part (a) u_{post}=0, the term {1 - exp (- λ (u_{post}) } equals zero). The model is based on the critical state theory that predicts that (i) as shear strain increases soil gradually reaches a steady state and (ii) the response is affected by the distance from the critical state. The factor f gives the effect of the distance from the critical state. At the critical state the factors f and r equal unity. Furthermore, the model is based on (a) the fact that plastic shear strain depends on the τ / σ' ratio, (b) a Roscoe type dilation equation and (c) extension from the drained to undrained conditions using (i) the elastic stiffness of the soil to estimate pore pressures from volumetric changes under drained conditions and (ii) effective stresses. Last but not least, the model is based on the proposition made by Aubry et al. (1990) that the constitutive model of displacement and stress of an interface should be of the same form and derived from the constitutive model of strain and stress of continuum soils.

The model structure is similar with the previous model by Modaressi H. et al. (1995). It differs from this previous model in the form of the factors σ'_{cs-o}, f, k and r. The critical state σ'_{cs-o}, for clays is expressed in terms of the model parameters S1 and S2 because according to clay behavior described above, the peak strength can be normalized in terms of the initial vertical effective stress and the OCR value. For the factor f, the expression proposed by Modaressi A. et al. (2001) is used because it is more general and simulates the changes in clay behavior in terms of soil density (expressed in terms of σ'_{cs-o}). Unlike the previous model, it is assumed that k depends on the confining stress. Taking k as $[k_1 (\sigma'/Pa)^{k2}]$ is consistent with the measured dependence of the elastic moduli of soils on consolidation stress (Hardin and Black, 1968). Regarding the factor r, a function used by Gerolymos and Gazetas (2007) is used because it was found that it predicts better the test results.

For part (b) of soil response, the second term of equations (4) and (10) is introduced to simulate the post-failure change in the critical state in terms of the applied post-failure shear displacement. The critical state normal effective stress is reduced from the value corresponding to initial failure, defined in chapter 1 above, σ'_{cs-o}, to the value corresponding to very large shear displacement (σ'_{cs-f}). The critical state σ'_{cs-f} is expressed in terms of the dimesionless parameter S3 and the factor σ'_{cs-o} because it was found that this simulates accurately the measured residual strength of clays in undrained conditions in terms of the initial effective stress and the OCR. Finally, in equations (4) and (10) the exponential function in terms of displacement is used because it was found that it simulates accurately the measured rate that the critical state decreases from the peak to the residual value, under both undrained and drained conditions.

The model parameters

To simulate the undrained response of clays, the constitutive model has 10 parameters: φ'_{cs}, φ'_{dil}, S1, S2, S3, a, n, k_1, k_2, λ. For the drained case, the constitutive model has one more parameter than the model for the undrained case, the parameter β, and does not use the parameters k_1, k_2, of the undrained case. The parameters φ'_{cs}, $\varphi'_{dil,}$ S1, S2, β, a, k_1, k_2 can be estimated accurately from simple-shear, direct-shear or ring shear tests. The parameters S3, λ correspond to large displacement and can be estimated only with ring-shear tests.

In particular, the parameter φ'_{cs} corresponds and can be estimated from the ratio of the shear stress and the effective vertical stress at failure, as $\arctan(\tau_f/\sigma_f')$ where τ_f is the failure shear stress and σ_f' is the corresponding normal effective stress. In the proposed model framework, the parameter φ'_{dil} equals φ'_{cs} (Modaressi and Lopez-Caballero, 2001).

The parameters S1, S2, a, n, k_1, k_2, β correspond to the pre-failure and failure soil response. The parameter S1 can be estimated from the peak shear stress of normally-consolidated clays under undrained conditions ($\tau_{m(OCR=1)}$), as

$$S1 = \log(\tau_{m(OCR=1)}) / \sigma'_o \qquad (8b)$$

The parameter S2 describes the effect of OCR on the peak soil strength under undrained conditions and can be obtained when for a given soil, tests with different OCR values exist as

$$S2 = \log(\tau_{m(OCR)} / \tau_{m(OCR=1)}) / \log(OCR) \qquad (8c)$$

where $\tau_{m(OCR)}$ is the maximum shear stress measured at the current OCR. Under drained conditions, the parameters S1, S2 cannot be estimated directly. The parameters a and n affect mainly the pre-failure shape of the shear stress-displacement curve. The parameter k_1 affects primarily the pre-failure rate of excess pore pressure generation with displacement under undrained conditions. The parameter k_2 describes the effect of the confining stress on the rate of excess pore pressure generation under undrained conditions. It can be obtained when for a given soil, shear tests with different confining stresses exist. The parameter β affects the normal displacement that accumulates under drained conditions.

The parameters S3 and λ control the post-failure soil response. The parameter S3 can be estimated from the residual shear stress (τ_r) and maximum shear stress (τ_m) under undrained conditions as

$$S3 = \tau_r/\tau_m \qquad (9b)$$

Under drained conditions, the parameter S3 cannot be estimated directly. Finally, the parameter λ controls the rate in terms of displacement, that the critical state decreases from the peak to the residual value, under both undrained and drained conditions.

According to Fig. 7a, the parameter φ'_{cs} decreases with the Liquid Limit of the clay and varies between 31^o and 5^o. According to Fig. 2c, the parameter S1 takes values between 0.15 and 0.30 and the factor S2 equals about 0.7. In addition, theory (Wood, 1990) predicts that the factor S1 is proportional to φ'_{cs}. Thus, S1 for clays with LL>80% may take values less than 0.15.

The typical value of k_1 given for clays in the previous model by Modaressi H et al. (1995) is $6(10^5)$kPa*m. According to Hardin and Black (1968) the elastic soil modulus is proportional to the square root of the confining stress. It is inferred that k_2 equals 0.5. For the parameters a and n, Gerolymos and Gazetas (2007) use the values of 10^{-4}m and 1. Assuming a logarithmic relationship between the volumetric strain (ε^p) and the vertical stress (σ') measured in the oedometer test, one can write

$$\sigma'_{cs} = \sigma'_{cs\text{-}o} \exp(-\beta' \varepsilon^p) \qquad (12)$$

where β' is the plasticity constrained modulus. Typical values of β' for low-plasticity and high-plasticity clays are 26 and 10 respectively (Modaressi and Lopez-Caballero, 2001). The parameter β equals (β'/dw) where dw is the width of the soil layer affected by the shearing. A typical thickness value of shear bands in granular materials is 4mm (Muhlhaus and Vardoulakis, 1987). Assuming similar shear bands for sands and clays, it is inferred that a typical value of β for clays is 4000 m^{-1}. For the parameters λ and S3 no previous propositions exist.

Comparison between measurements and predictions

Excel worksheets were programmed to simulate (a) the undrained and (b) the drained response of clays, as described by equations (1) - (11). As of primary importance is the simulation of loss of strength under undrained conditions, application is first made in this case. The model was calibrated to predict the tests of table 1. Model parameters were varied for each soil type, and not for each individual test. As a first attempt, according to the above discussion, k_2 was taken 0.5, φ'_{dil} equal to φ'_{cs}, n equal to 1. In the cases where pore-pressure data does not exist, k_1 was assumed $6(10^5)$kPa*m. It was not found necessary to change these initial guesses of parameters. The remaining model parameters were estimated using the procedures described above.

Then, the model was applied to predict the results of the drained tests of Altamira Bentonitic Tuff. As this clay is a special soil type where the residual effective friction angle varies considerably with confining stress, φ'_{cs} was varied in terms of confining stress. As a first attempt, according to the above discussion, φ'_{dil} was taken equal to φ'_{cs} and n equal to 1. It was not found necessary to change these initial guesses of parameters. The remaining model parameters were estimated using the procedure described above. Model parameters were not varied in terms of the OCR value, except from a small variation of the parameter S3 in order to predict more accurately the post-failure normal displacement versus shear displacement curves.

Table 2 gives the model parameters that were obtained for all tests of table 1. Figs. 3, 4, 5, 6 and 8 compare the measured response with the predictions for all tests. In all cases predictions are in good agreement with measured response.

Table 2. Model Parameters that fit the test results of table 1
(a) undrained tests

No	**1**	**2**	**3**	**4**	**5**	**6**	**7**
φ'_{cs}	29	28	28	28	28	28	28
φ'_{dil}	29	28	28	28	28	28	28
S1	0.23	0.20	0.24	0.31	0.24	0.23	0.37
S2	-	0.7	0.14	0.16	0.3	0.10	-
S3	0.34	0.55	0.23	0.35	0.25	0.60	0.35
a (m)	$5*10^{-5}$	$1*10^{-4}$	$3*10^{-4}$	$3*10^{-4}$	$3*10^{-4}$	$8*10^{-5}$	$3*10^{-4}$
n	1.0	1.0	1.0	1.0	1.0	1.0	1.0
k_1	$7*10^5$	$4*10^5$	$6*10^5$	$6*10^5$	$6*10^5$	$8*10^5$	$6*10^5$
k_2	0.5	0.5	0.5	0.5	0.5	0.5	0.5
λ	300	220	40	35	80	100	150

- : Cannot be measured as tests in different OCR values are not given.

(b) Drained tests

No.	**8 (σ'_o=60 kPa)**	**8 (σ' =120 kPa)**	**8 (σ'_o=240 kPa)**	**8 (σ'_o=480 kPa)**	**8 (σ'_o=850 kPa)**	**8. Ave.**
φ'_{cs}	15	12	10	7.8	5.7	**10.1**
φ'_{dil}	15	12	10	7.8	5.7	**10.1**
S1	0.1	0.1	0.1	0.1	0.1	**0.1**
S2	0.45	0.45	0.45	0.45	0.45	**0.45**
S3	0.66	0.65	0.5	0.4	0.4	**0.52**
a (m)	10^{-4}	10^{-4}	10^{-4}	10^{-4}	10^{-4}	**10^{-4}**
n	1	1	1	1	1	**1.00**
β (m^{-1})	3000	3000	3000	3000	3000	**3000**
λ	200	200	200	200	200	**200**

DISCUSSION

A model predicting the response of clays along slip surfaces under either undrained or drained conditions, calibrated with soil response measured in ring shear tests, was not found in the literature. The paper proposes such a model. The proposed model is based on (a) the critical state theory and (b) the assumption that the critical state changes once failure is reached, in terms of the applied further shear displacement. Qualitatively, this model structure predicts (a) under undrained conditions the post-failure increase in pore pressures and decrease in effective shear stress and (b) under drained conditions the post-failure increase in normal displacement without change in shear stress.

The proposed model has 11 parameters. Model parameters were varied for each clay of table 1. An exception is the Altamira Bentonitic Tuff where as the residual effective friction angle exhibits an unusually nonlinear dependency on the confining stress, φ'_{cs} and S3 were varied in terms of confining stress. For the wide range of clays considered, $\varphi'_{dil}=\varphi'_{cs}$, k_2=0.5, and n=1. For the other 8 parameters, table 3 gives the range of variation of the obtained model parameters. Table 3 also validates the range obtained of the model parameters by comparing it with variations proposed by previous researchers.

It should be noted that the data set considered to validate the model is far from complete. Under undrained conditions (a) different aspects of shear stress and pore pressure displacement curves are given in each soil, as described in table 1, and (b) only results on clays with a Liquid Limit at the range 34 to 60% exist. Under drained conditions (a) the response at small displacement (less than 0.5mm) cannot be retrieved from the data set and (b) shear stress – displacement data is given only for one clay, which is different from those studied under undrained conditions, as is, additionally, of a particular case, having a large value of the Liquid Limit (98%) and (unusually) strong effect of the effective stress on the effective residual friction angle. Checking the model thru a more complete data base is desirable.

Table 3. Variation of model parameters in predictions

Parameter	Variation	Discussion
φ'_{cs} (^{o})	5.7-29	Variation consistent with Fig. 7. Trend of decrease with Liquid Limit and confining stress, consistent with Fig. 7
S1	0.10-0.37	Consistent with Fig. 2 and the fact that S1 is proportional with the friction angle. The small values correspond to small friction angle.
S2	0.1-0.7	Consistent with Fig. 2. The small values correspond to cases where OCR values are small, and thus a reliable estimate of this parameter cannot be made.
k_1 (kPa*m)	$4*10^5$-$7*10^6$	Consistent with the value $6(10^5)$kPa*m suggested by the previous similar model by Modaressi H et al. (1995)
a (m)	$5*10^{-5}$-$3*10^{-4}$	Consistent with the value of 10^{-4} used by Gerolymos and Gazetas (2007)
n	1	Consistent with the value used by Gerolymos and Gazetas (2007)
k_2	0.5	Consistent to Hardin and Black (1968)
φ'_{dil}	$=\varphi'_{cs}$	Consistent to Modaressi and Lopez-Caballero (2001)
β (m^{-1})	3000	Consistent with discussion of section 3.
S3	0.23-0.66	No previous proposition is available
λ	35-300	No previous proposition is available

CONCLUSIONS

The paper proposes and validates a constitutive model simulating the change of resistance along a slip surface with shear displacement of clays both for the undrained and drained cases. The proposed model is based on (a) the critical state theory and (b) the assumption that the critical state changes once failure is reached, in terms of the applied further shear displacement. Under undrained conditions, the proposed model simulates the excess pore pressure generation and subsequently the continuous change of resistance along the slip surface in clays from its initial value to the peak strength and then at the large displacement residual value as measured in constant-volume ring shear tests. Under drained conditions, the model simulates the normal displacement change and subsequently the change of resistance along the slip surface in clays as measured in drained ring shear tests. The proposed model has 11 parameters. Typical values of these parameters are given.

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APPENDIX. NOTATION

S1, S2, S3, a, n, k_1, k_2, λ, β	=	Fitting parameters of the proposed constitutive model
CF	=	Clay Fraction
f, r ,k	=	Factors of the constitutive model
LL	=	Liquid Limit
Meas.	=	Measured response in laboratory tests
OCR	=	Overconsolidation ratio
P	=	Excess pore pressure
Pa	=	Atmospheric pressure
Pred	=	Predicted response using the proposed model
u	=	The displacement along the slip surface
u_o	=	The shear displacement when the failure state (defined as f≈1 and r≈1) is first reached
u_n	=	The displacement normal to the slip surface
u_{post}	=	The displacement after the failure state is first reached
σ'_c	=	Steady-state effective stress normal to the slip surface
σ'_{c-o} , σ'_{c-f}	=	Initial and final steady-state effective stress
σ'	=	Effective stress normal to the slip surface
σ'_o	=	Effective stress normal to the slip surface prior to the initiation of shearing
τ	=	Shear stress
τ_r	=	Shear stress at very large displacement (=residual shear strength)
τ_m	=	Maximum shear stress
φ'_{cs}	=	The effective critical state friction angle of the constitutive model
φ'_{dil}	=	The dilation effective friction angle of the constitutive model
φ'_r	=	The effective residual friction angle