Residual shear strength behavior of swelling soils

Comportement de résistance résiduelle au cisaillement des sols gonflants

Markou I.N.
Democritus University of Thrace, Xanthi, Greece

ABSTRACT: Three clay soils from North-Eastern Greece, presenting "high" or "very high" swelling potential, were used in this investigation. The residual strength behavior of these soils was evaluated by performing ring shear tests on remolded specimens. The specimens were prepared either with the optimum water contents resulted from Standard compaction test (26% - 31%) or with higher water contents (46% - 47%), derived from the specimen moistures at the end of the preceding tests with optimum water contents. Ring shear tests were performed using a Bromhead apparatus, according to the procedures described in BS 1377 – Part 7. The results obtained from the consolidation stage of the ring shear tests conducted in swelling clays, do not present the typical form attained in ordinary clays. Although the failure envelopes in residual state present curvature in some cases, they can be considered as linear for effective normal stresses up to 250 kPa. The resulting values of residual friction angle do not exceed 14^o and, in some cases, are affected by the water content of the specimens used in the tests.

RÉSUMÉ: Trois sols argileux provenant du Nord-Est de la Grèce et présentant un potentiel de gonflement « haut » ou bien « très haut », ont été utilisés dans le cadre de ce travail. Le comportement de la force résiduelle de cisaillement a été évalué à l'aide des essais de cisaillement annulaire sur des échantillons remués. Les échantillons étaient préparés soit avec le pourcentage de contenu de l'eau optimal, résultant par l'essai de compactage standard (26 %-31 %), soit avec des pourcentages de contenu de l'eau plus élevés (46 %-47 %) qui dérivent par l'humidité des échantillons à la fin des essais précédents avec des contenus de l'eau optimaux. Les essais de cisaillement annulaire ont été réalisés à l'aide de l'appareil de Bromhead, selon la procédure décrite dans BS 1377-Partie 7. Les résultats obtenus par le stage de consolidation du cisaillement annulaire, conduits sur des argiles gonflantes, n'atteignent pas la forme classique des argiles ordinaires. Bien que l'enveloppe de rupture présente une courbe dans quelques cas, elles peuvent être considérées comme linéaires pour des contraintes normales effectives allant jusqu'à 250 kPa. Les valeurs résultantes de l'angle de friction résiduelle, n'excèdent pas 14^o et, dans quelques cas, elles sont affectées par le contenu de l'eau des échantillons utilisés lors des essais.

KEYWORDS: soil behaviour, swelling soils, expansive clays, residual shear strength, laboratory investigation, ring shear tests

1 INTRODUCTION

The residual shear strength of cohesive soils plays a part in the stability of old landslips, in the assessment of the engineering properties of soil deposits which contain pre-existing shear surfaces, and in the assessment of the risk of progressive failure in stability problems in general. Extensive investigations on the residual strength have been carried out and directed towards: (a) developing suitable laboratory techniques for its measurement, (b) identifying the influence of geomechanical features on the residual strength, (c) understanding the basic mechanisms involved in the mobilization of shear strength during the residual stage of deformation and (d) establishing correlations with soil index properties. Although the residual strength has been studied in the laboratory using experimental techniques such as the triaxial test procedure proposed by Chandler (1966), the ring shear test and the multi-reversal direct shear test are widely used to measure the residual strength of soils. Published results (Lupini et al. 1981, Skempton 1985) show that residual shear behavior changes significantly as the clay content of cohesive soil increases, and that a change in shearing mechanism also occurs. Apart from the clay fraction, the mineralogy of the clay also has an effect on residual strength, especially when the clay fraction is large (Kalteziotis 1993). A number of correlations between residual strength and clay fraction, plasticity index and liquid limit have been proposed (e.g. Skempton 1964, Lupini et al. 1981, Hawkins and Privett 1985, Skempton 1985, Mesri and Cepeda-Diaz 1986), but Lupini et al. (1981) suggested that all these correlations can not be general.

Soils containing expansive clay minerals, called swelling or expansive soils, have created problems of uplift and instability on many structures, because there is an opportunity for water to become available and thus facilitate the expansion (swelling) of the clay minerals. Swelling clays are often subject to extreme changes in shear strength because of extreme moisture changes. In addition to the strength factors related to the minerals involved, the interrelation of moisture, density and load plays an important part in the strength (Gibbs et al. 1960). Swelling soils also exist in Greece and have created a number of problems and/or failures in projects (Christodoulias and Gasios 1987, Stamatopoulos et al. 1989). Therefore, the properties and the swelling characteristics of swelling soils from Greece were investigated (Xeidakis 1993, Tsiambaos and Tsaligopoulos 1995, Kollaros and Athanasopoulou 1997), preventive and corrective measures against swelling were applied (Christodoulias and Gasios 1987, Stamatopoulos et al. 1989) and the treatment of swelling soils using various methods for reducing swell potential and increasing strength was examined (Stamatopoulos et al. 1992). It is, therefore, of merit to investigate the residual shear strength behavior of swelling soils. Toward this end, a laboratory investigation was conducted in order to evaluate the residual shear strength parameters of selected swelling soils having different moisture contents and the results obtained and observations made, are reported herein.

2 SOIL PROPERTIES

Three soils from the region of Thrace (North-Eastern Greece) were used in this investigation because of their swelling characteristics. According to the properties presented in Table 1, all three soils can be considered as clay soils since the clay fraction (grain sizes <0.002 mm) ranges from 70% to 80%. The P1-S2 and P2-S2 soils are classified as CH, while the P2-S1 soil is classified as MH in accordance with the Unified Soil Classification System. The values of maximum dry unit weight, γ_{dmax}, and optimum moisture content, w_{opt}, were obtained by conducting compaction tests with standard compaction effort.

All three soils present "high" or "very high" swelling potential, according to the known correlations of soil index properties with swelling characteristics (Papakyriakopoulos and Koudoumakis 2001). One-dimensional swell tests (ASTM D4546, Method A) were conducted using laboratory-compacted specimens (ASTM D698) of these soils (Koudoumakis 2000). The results obtained from specimens with initial moisture contents similar to the optimum moisture contents of the soils (Table 1), are presented in Table 2. It can be observed that the swell pressure ranges from 170 kPa to 820 kPa and the free swell ranges from 11% to 19%. These values are indicative of the swelling potential of the soils used in this investigation.

3 EXPERIMENTAL PROCEDURES

The residual shear strength behavior of the soils was evaluated by performing ring shear tests on remolded specimens. The use of remolded specimens allowed the adequate control of the specimen moisture content. The specimens were prepared with the moisture contents shown in Table 3. At first, the optimum moisture content, resulted from the Standard compaction test (Table 1), was used for each soil. The following specimens were prepared with larger moisture content, obtained for each soil as the average value of the specimen moisture contents at the end of the preceding tests started with optimum water content. The specimens were placed in the cell of the ring shear apparatus by kneading the soil with the desired moisture content evenly to fill the annular cavity between the confining rings of the cell, using a small spatula (BS 1377 – Part 7).

Table 1. Properties of soils.

Soil designation		*P1-S2*	*P2-S1*	*P2-S2*
Sampling depth (m)		1.5-3.0	0.0-1.6	1.6-3.0
Specific gravity G_s		2.82	2.68	2.73
Grain size analysis	Sand (%)	11.5	5.6	3.8
	Silt (%)	9.3	23.9	25.8
	Clay (%)	79.2	70.5	70.4
Atterberg limits	Liquid limit w_L	86	82	87
	Plasticity index I_p	63	42	55
Compaction characteristics	Maximum dry unit weight γ_{dmax} (kN/m^3)	13.80	13.95	14.27
	Optimum moisture content w_{opt} (%)	31.2	27.0	25.8

Table 2. Typical results of one-dimensional swell tests.

Soil designation	*P1-S2*	*P2-S1*		*P2-S2*
Initial moisture content w_0 (%)	30.63	31.13	24.02	27.48
Initial void ratio e_0	0.990	0.946	1.054	1.045
Initial degree of saturation S_{r0} (%)	87.25	88.19	61.08	71.79
Dry unit weight γ_d (kN/m^3)	13.87	13.52	12.81	13.09
Swell pressure P_s (kPa)	820	335	170	410
Specimen height increase Δh (mm)	3.85	2.20	2.52	2.83
Free swell $\Delta h/h_0$ (%)	19.23	11.60	13.28	14.17

Table 3. Testing program.

Soil	Moisture content w (%)	Effective normal stress σ'_n (kPa)			
P1-S2	31.2	25	75	250	800
	45.8	25	50	100	200
P2-S1	27.0	25	75	200	600
	47.1	25	50	100	200
P2-S2	25.8	25	75	200	600
	46.4	25	50	100	200

Ring shear testing was based on the procedure described in BS 1377 – Part 7. The tests were conducted using a Bromhead ring shear apparatus (Bromhead 1979) and annular specimens of 5 mm thickness with internal and external diameters of 70 mm and 100 mm, respectively. The specimens were consolidated for a period of 24 hours under the effective normal stresses, σ'_n, presented in Table 3 and, subsequently, were sheared at a constant rate of angular displacement equal to 0.048 degrees/min. The selection of this rate of angular displacement was dictated by the unconventional results of the consolidation stage of the tests, described in the next section, and was based on the fact that this rate has been found satisfactory for a large range of soils (BS 1377 – Part 7). One of the objectives of the present study was to investigate the residual strength behavior of the soils for a wide range of effective normal stresses reaching or even exceeding the values of swell pressure shown in Table 2. Although this goal was accomplished for the specimens prepared with the optimum moisture contents (Table 3), the use of effective normal stresses larger than 200 kPa was not feasible in the tests performed with soil moisture contents ranging from 46% to 47% because of excessive specimen loss during testing.

4 RESULTS AND DISCUSSION

Typical "specimen length change" – "log time" curves obtained from the consolidation stage of ring shear tests conducted with optimum moisture content and with moisture content ranging from 46% to 47%, are shown in Figures 1a and 1b, respectively. It can be observed (Figure 1a) that, in several cases, the curves do not present the usual form attained in ordinary clays, either due to the expansion of specimens or because the consolidation was not completed within the predetermined period of 24 hours. It can also be observed (Figure 1b) that, in general, the classic type of curves appears in the test performed under the highest effective normal stress of each test series. The overall behavior of the three soils in the consolidation stage of the ring shear tests is summarized in Table 4. More specifically, the tests are divided in those exhibited specimen expansion and those demonstrated specimen compression during consolidation. It is evident that specimen expansion occurred almost in all series of tests and that the interchange of expansion and compression takes place at higher values of effective normal stress in the tests conducted with optimum water contents. This can be attributed to the denser condition of the specimens in these tests, since their values of initial dry unit weight, γ_{d0}, range from 1.24 gr/cm^3 to 1.59 gr/cm^3 and are larger than those (γ_{d0}= 1.10 gr/cm^3 – 1.27 gr/cm^3) in the tests conducted with moisture contents ranging from 46% to 47%.

The residual shear stress, τ_r, is the minimum constant value of shear stress, determined at the end of the shearing stage of each test and used in the drawing of the residual failure envelopes of soils. As typically shown in Figure 2a, the residual failure envelopes resulted from the ring shear tests conducted on specimens with optimum water contents are curved, in agreement with the observation of Stark and Eid (1994) that the non-linearity of failure envelopes is significant for soils with a

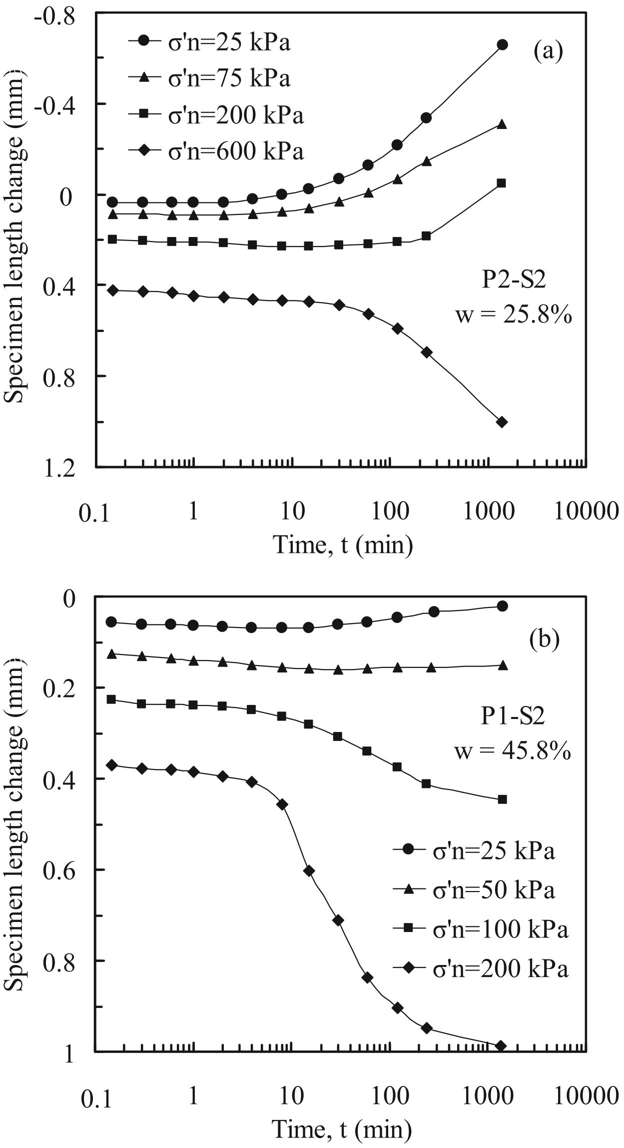

Figure 1. Typical results from the consolidation stage of ring shear tests.

Table 4. Soil behavior during the consolidation stage of ring shear tests.

Soil	Moisture content *w* (%)	Expansion	Compression
P1-S2	31.2	25, 75 [a]	250, 800 [a]
	45.8	25, 50	100, 200
P2-S1	27.0	25, 75, 200	600
	47.1	---------	25, 50, 100, 200
P2-S2	25.8	25, 75, 200	600
	46.4	25	50, 100, 200

[a] Values of effective normal stress σ'_n (kPa) used in the tests

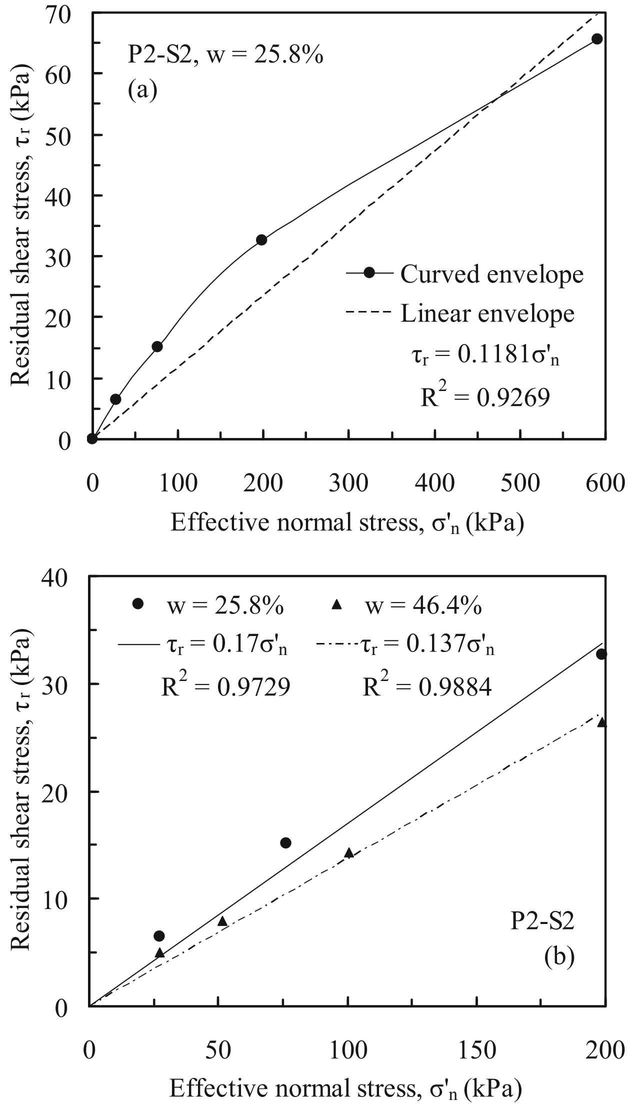

Figure 2. Typical residual failure envelopes from ring shear testing.

Table 5. Values of residual friction angle φ'_R (degrees).

Soil	w_{opt}	w_{opt}	*w= 46% – 47%*	Difference (%)	
P1-S2	7.3 [a]	8.8 [b]	8.8	17.0 [c]	0.0 [d]
P2-S1	8.4 [a]	9.1 [b]	8.7	7.7 [c]	4.4 [d]
P2-S2	6.7 [a]	9.6 [b]	7.8	30.2 [c]	18.8 [d]

[a] For all values of effective normal stress used in the tests
[b] For values of effective normal stress up to 250 kPa
[c] Between the two angle values determined for w_{opt}
[d] Between angles determined for w_{opt} ($\sigma'_n \leq 250$ kPa) and w= 46%-47%

clay fraction > 50% and a liquid limit between 60 and 220. The fitting of the same experimental data with a linear failure envelope is applied in Figure 2b, without considering the measurement corresponding to the maximum effective normal stress. As a result, a very high correlation coefficient, R^2, is obtained indicating that the failure envelopes of the soils with optimum water contents can be considered as linear for effective normal stresses up to 250 kPa. Typical residual failure envelope obtained from specimens prepared with moisture contents ranging from 46% to 47%, is also presented in Figure 2b. It can be stated with confidence that these failure envelopes are also linear because the resulting correlation coefficients range from 0.98 to 0.99. Linear residual failure envelopes were also obtained from ring shear tests conducted on Greek clayey soils with effective normal stresses ranging from 50 to 400 kPa (Kalteziotis 1993).

The values of the residual friction angle, φ'_R, obtained in this investigation, are summarized in Table 5. These values are similar to those reported by other researchers (Bishop et al. 1971) and were determined after it was ascertained that the values of residual cohesion, c'_R, are negligible and can be set equal to zero. If the curvature of the failure envelopes for optimum water contents is not taken into consideration and all experimental data are fitted with a linear failure envelope (Figure 2a), the correlation coefficients are satisfactory ($R^2 > 0.92$) but the resulting φ'_R values are even by 30% lower than the values obtained for effective normal stresses up to 250 kPa (Table 5). The relatively low difference (7.7%) in P2-S1 soil is attributed to the not so pronounced curvature of its failure envelope. Xeidakis (1993) has reported that the residual friction angle decreases as the moisture content of swelling soils increases. This effect of moisture content on the residual friction angle was verified in the present research only for P2-S2 soil (Table 5), probably because the variation of moisture content used, ranges from 14% to 21% and is low, compared to the variation of 35% used by Xeidakis (1993).

The results of residual shear strength tests are often presented by plotting the values of residual friction coefficient, τ_r/σ'_n, against the corresponding values of effective normal stress, σ'_n, (Lupini et al. 1981, Hawkins and Privett 1985). Thus, the "complete residual failure envelopes" (Hawkins and Privett 1985) can be obtained and the effect of effective normal stress on residual shear strength can be evaluated. The residual friction angle can be expressed as (Hawkins and Privett 1985):

$$\phi'_R = \tan^{-1}\frac{\tau_r}{\sigma'_n} \tag{1}$$

The results of ring shear tests conducted on specimens with optimum water contents were analyzed using Equation 1 and the resultant values of residual friction angle are presented in Figure 3. It is observed that the residual friction angle decreases with increasing effective normal stress and that P2-S2 soil presents the most pronounced curvature of failure envelope and, as a result, the maximum variation of residual friction angle. Finally, it appears that the "lowest constant residual strength" (Hawkins and Privett 1985) was not reached by the tested soils for the range of effective normal stresses used in this study.

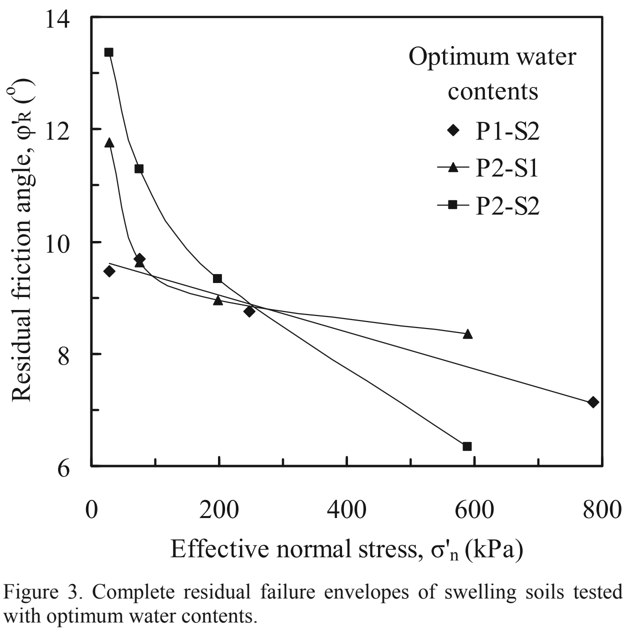

Figure 3. Complete residual failure envelopes of swelling soils tested with optimum water contents.

5 CONCLUSIONS

Based on the results of this investigation and within the limitations posed by the soils used and the number of tests conducted, the following conclusions may be advanced:

- The behavior of swelling soils in the consolidation stage of ring shear tests depends on the specimen moisture content and the effective normal stress used.
- The residual failure envelopes obtained for swelling soils, tested with the optimum moisture contents resulted from the Standard compaction test, are curved. Consequently, the residual friction angle decreases with increasing effective normal stress and does not attain a minimum constant value for the range of effective normal stresses used in this study.
- All residual failure envelopes obtained in this investigation can be considered as linear for effective normal stresses up to 250 kPa, regardless of the moisture content of soils.
- The residual friction angle does not always decrease as the moisture content of soil increases.

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7 REFERENCES

ASTM D698 1998. Standard test method for laboratory compaction characteristics of soil using Standard effort (12,400 ft-lbf/ft^3 (600 kN-m/m^3)). *Annual Book of ASTM Standards* 04.08 (I), 77-84.

ASTM D4546 1998. Standard test methods for one-dimensional swell or settlement potential of cohesive soils. *Annual Book of ASTM Standards* 04.08 (I), 663-669.

BS 1377 1990. *Soils for Civil Engineering Purposes – Part 7: Shear Strength Tests (Total Stress)*. British Standards Institution, London.

Bishop A.W., Green G.E., Garga V.K., Andresen A. and Brown J.D. 1971. A new ring shear apparatus and its application to the measurement of residual strength. *Géotechnique* 21 (4), 273-328.

Bromhead E.N. 1979. A simple ring shear apparatus. *Ground Engineering* 12 (5), 40-44.

Chandler R.J. 1966. The measurement of residual strength in triaxial compression. *Géotechnique* 16 (3), 181-186.

Christodoulias J. and Gasios E. 1987. Investigation on the motorway damage due to expansive soil in Greece. *Proc. 6th Int. Conf. on Expansive Soils*, New Delhi, 241-245.

Gibbs H.J., Hilf J.W., Holtz W.G. and Walker F.C. 1960. Shear strength of cohesive soils. *Proc. Research Conf. on Shear Strength of Cohesive Soils*, Boulder, 33-162.

Hawkins A.B. and Privett K.D. 1985. Measurement and use of residual shear strength of cohesive soils. *Ground Engineering* 18 (8), 22-29.

Kalteziotis N. 1993. The residual shear strength of some Hellenic clayey soils. *Geotechnical and Geological Engineering* 11, 125-145.

Kollaros G. and Athanasopoulou A. 1997. The character and identification of swelling soils in road construction projects. *Proc. Engineering Geology and the Environment*, Marinos et al. (eds.), Balkema, 187-192.

Koudoumakis P. 2000. *Personal communication*. Democritus University of Thrace, Xanthi, Greece.

Lupini J.F., Skinner A.E. and Vaughan P.R. 1981. The drained residual strength of cohesive soils. *Géotechnique* 31 (2), 181-213.

Mesri G. and Cepeda – Diaz A.F. 1986. Residual shear strength of clays and shales. *Géotechnique* 36 (2), 269-274.

Papakyriakopoulos P. and Koudoumakis P. 2001. Swelling soils in the area of Thrace. *Proc. 4th Hellenic Conf. on Geotechnical and Geoenvironmental Engineering* 1, Athens, 163-170. (in Greek)

Skempton A.W. 1964. Long-term stability of clay slopes. *Géotechnique* 14 (2), 75-101.

Skempton A.W. 1985. Residual strength of clays in landslides, folded strata and the laboratory. *Géotechnique* 35 (1), 3-18.

Stamatopoulos A., Gassios E., Christodoulias J. and Giannaros H. 1989. Recent experiences with swelling soils. *Proc. 12th Int. Conf. on Soil Mechanics and Foundation Engineering* 2, Rio de Janeiro, 655-658.

Stamatopoulos A., Christodoulias J. and Giannaros H. 1992. Treatment of expansive soils for reducing swell potential and increasing strength. *Quarterly Journal of Engineering Geology* 25, 301-312.

Stark T.D. and Eid H.T. 1994. Drained residual strength of cohesive soils. *Journal of Geotechnical Engineering* 120 (5), 856-871.

Tsiambaos G. and Tsaligopoulos Ch. 1995. A proposed method of estimating the swelling characteristics of soils: some examples from Greece. *Bulletin of the International Association of Engineering Geology* 52, 109-115.

Xeidakis G. 1993. Swelling soils in Thrace, Northern Greece: origins and properties. *Proc. Geotechnical Engineering of Hard Soils – Soft Rocks* 1, Anagnostopoulos et al. (eds.), Balkema, 863-870.