Earth Reinforcement Practice, Ochiai, Hayashi & Otani (eds) © 1992 Balkema, Rotterdam. ISBN 90 5410 093 1

Soil nail pull out resistance in residual soils

G.Heymann & A.W.Rohde
University of Pretoria, South Africa

K.Schwartz
Schwartz Tromp and Associates, Johannesburg, South Africa

E.Friedlaender
Frankipile, Johannesburg, South Africa

ABSTRACT: A correlation between pull out resistance of a soil nail and soil parameters obtained from routine laboratory and field tests, would be of considerable use to the practising engineer. This paper correlates data from 40 pull out tests, done in residual soil, to different soil parameters. Evidence is presented to demonstrate the importance of soil dilatancy in the prediction of soil nail pull out resistance.

1. INTRODUCTION

Soil reinforcement using soil nails has become a popular method for lateral support of excavations and slopes in South Africa. This application has proven to be economically competitive in residual soils where fair to excellent bond between nail and soil are typically found. A research project was instituted with the objective of finding a correlation between ultimate shear stress and soil parameters determined from routine laboratory and field tests normally carried out as part of a geotechnical investigation.

The largest urban area in South Africa (the Pretoria-Witwatersrand-Vereeniging or PWV area) is located in an area where deep residual soils generally are predominant within the soil profile. To date many soil nailing projects have been completed in residual soil profiles in the PWV area. Although the first soil nailing project was completed only in 1987 (Schwartz and Friedlaender, 1989), at present approximately 90 projects have been completed, 45000 square meters of excavated face have been supported by installing 85 kilometres of nails.

2. CALCULATION OF SOIL NAIL PULL OUT RESISTANCE

For the design of a soil nail system, the pull out resistance of the individual nails are of paramount importance; Jewell (1990), Schlosser (1982), Gässler and Gudehus (1983), Gässler (1988), Juran and Elias (1988). Several authors have attempted to calculate the pull out resistance by theoretical and empirical methods, including Jewell (1990), Schlosser (1982) and Bridle and Barr (1990). All the aforementioned authors used the well known equation for the calculation of shear strength of soil as the point of departure:

$$P_{ult} = \pi D L (c + \sigma_n \tan\phi) \qquad (1)$$

where:
P_{ult} = Ultimate pull out resistance of soil nail
D = Diameter of nail or nail-grout combination
L = Nail length
c = Effective cohesion
σ_n = Normal effective stress between nail and soil at failure
ϕ = Effective angle of friction of soil

Jewell (1990) as well as Bridle (1990) maintain that the normal stress between the nail and soil at failure increases as the effective overburden pressure increases. That is $\sigma_n = K\sigma_v$ where K is an earth pressure coefficient and σ_v is the effective overburden pressure. Jewell (1990) indicates that the value of K is between 0,75 and 1,0 for lightly overconsolidated soils. Schlosser (1982) however proposed the normal stress to be constant with depth. This was confirmed by Cartier and Gigan (1983), reporting a constant ultimate pull out force of soil nails at different depths. Schlosser (1982) comments that for soil nails grouted into pre drilled holes, the pull out force would depend greatly on the dilatancy characteristics of the soil, as the normal stress would increase significantly during pull out of a nail embedded in a dilatant soil. Schlosser further points out the difficulty in measuring the increase of normal stress, but suggests a ten fold increase from the initial normal stress, to the normal stress at failure to be possible.

3. FIELD PULL OUT TESTS

Pull out tests are done as a matter of routine on all soil nailing projects in South Africa to verify the pull out resistance assumed at the design stage. Test nails typically comprise bonded lengths of 1.0 m to 1.5 m and a diameter of the order of 100 mm. Geotextile is wrapped around the unbonded length of the nail to minimize end effects. The test is done rapidly (10 to 15 minutes) and undrained conditions are assumed in saturated soils. However most tests are carried out in partially saturated soils. The ultimate shear stress, as determined by a pull out test, is defined as:

$$T_{ult} = \frac{P_{ult}}{\pi DL} \qquad (2)$$

A typical pull out test result is shown in Figure 1.

4. SOIL NAIL PULL OUT RESISTANCE IN RESIDUAL SOILS

The procedure used to predict soil nail pull out resistance (equation 1) has been used to compare calculated ultimate shear stress, with

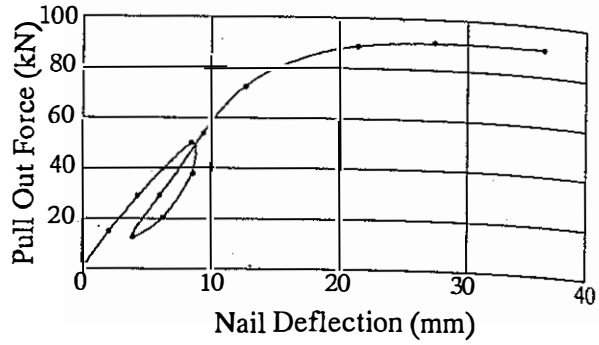

Figure 1.
Typical soil nail pull out test result.

the ultimate shear stress (T_{ult}) obtained from pull out tests (Figure 2). These comparisons indicate that an extremely conservative value for ultimate shear stress is obtained if equation 1 is used:

$$T_{ult}\,(\text{actual}) = \rho T_{ult}\,(calculated) \qquad (3)$$

Figure 2 shows: $2<\rho<4$

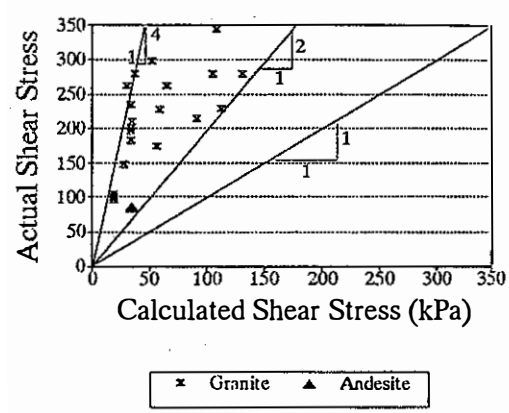

Figure 2.
Comparison of calculated shear stress (equation 1) and actual shear stress in residual soils.

The results of pull out tests carried out on a number of different soils also indicate that the ultimate shear stress is independent of depth. A typical result is shown in figure 3 for a site with a soil profile comprising residual andesite. The scatter of T_{ult} between depths 2 m and 3 m may be attributed to zones of highly weathered and

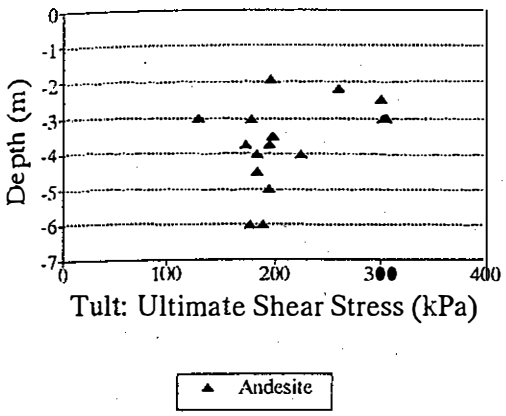

Figure 3.
Soil nail pull out results for residual andesite.

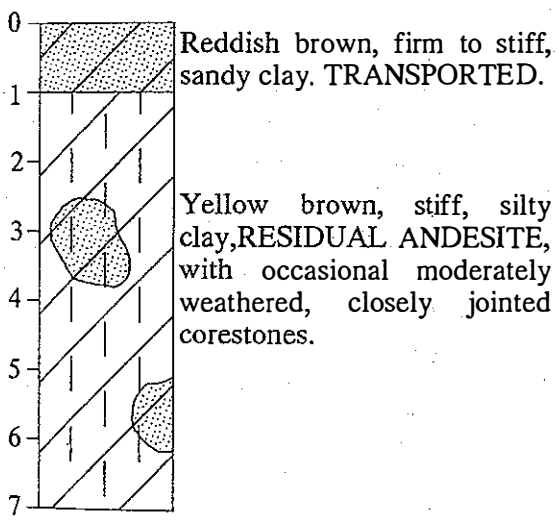

Figure 4.
Soil profile for test nails shown in Figure 3.

zones of less weathered soil, as is typically found in residual soil profiles. The soil profile is shown in figure 4.

5. PREDICTION OF SOIL NAIL PULL OUT RESISTANCE

As the construction of most large basement excavations are preceded by a geotechnical investigation, certain soil properties are known at the design stage of the lateral support system. The prediction of soil nail pull out resistance at the design stage of a project would be advantageous, as a more economical design would be possible. The foregoing paragraphs showed that procedures recommended in the literature for the calculation of soil nail pull out resistance could result in extremely conservative values if applied to residual soils.

Nine soil nailing projects, completed in Southern Africa in residual soils (granite and andesite), were chosen to attempt to correlate the data from soil nail pull out tests to basic soil parameters which are usually available from the geotechnical investigation. In certain cases the test nail results and geotechnical report on the site conditions were provided by the contractor. In other cases soil samples were taken during the construction stage, close to the position of the test nails and tested in the laboratory. The parameters considered were:

— Effective cohesion
— Effective friction angle
— Standard penetration tests (SPT)
— Percentage of the soil passing the 0.075 mm sieve
— Clay fraction of the soil (%<0.002 mm)
— Liquid limit (*LL*)
— Plasticity index (*IP*)

The results of these nine projects are summarised in Table 1.

Figure 5 illustrates the correlation between soil shear stress (T_{ult}) obtained from the test nails and the effective friction angle of the soil. The effective friction angle was in all cases determined by carrying out consolidated drained shear box tests on saturated samples. The shear box test was favoured instead of the triaxial test as the enforced failure plane of the shear box is considered to be more representative for comparison purposes with pull out tests. The consolidated drained test was selected even though it may not accurately model a pull out test in partially saturated soil, as the aim of this undertaking was to establish a relationship between ultimate shear stress (*Tult*) at failure and parameters normally obtained during a geotechnical investigation.

Even though figure 5 shows a considerable scatter of the data, an increase in ultimate shear stress is apparent as the effective friction angle of the soil increases. The majority of the results given are for residual granite soil which comprises a silty sand of low to moderate plasticity (see Table 1 for typical soil properties). The results are considered to be representative enough to put forward a lower bound design line that could be used for initial design purposes in similar soils:

Table 1 Typical soil properties

Project	Soil Origin	Nail no.	Depth (m)	ϕ (°)	c (kPa)	%<75 (μm)	%<2 (μm)	SPT	LL (%)	I_P	T_{ult} (kPa)
Sentinel	Granite	2	8.0					35	43	17	212
		4	5.0	36	0	35	17	29	43	17	263
		5	8.0	36	0	35	17		43	17	280
Jan Smuts	Granite	1	2.3	25	0						100
		2	2.3	25	0						96
		3	2.3	25	0						105
		4	2.5	39	1	23	3		30	12	280
		5	2.3	39	1	23	3		30	12	210
		6	2.0	39	1	23	3		30	12	263
		7	4.0	39	1	23	3		30	12	228
		8	3.8	39	1	23	3		30	12	175
		9	3.5	39	1	23	3		30	12	298
Keys ave.	Granite	1	1.0			32	11		31	11	88
		2	1.8			32	11		31	11	140
		3	1.0			32	11		31	11	105
		4	1.8			32	11		31	11	140
Nelspruit	Granite	1	1.0			14	4		16	4	139
		2	3.0			14	4		16	4	195
		3	2.8			17	6		16	4	167
		4	1.2			17	6		20	9	168
		5	3.2			17	6		20	9	181
		6	4.0			17	6		20	9	168
Arcadia	Andesite	7	3.0					76			129
		11	4.5					74			184
		12	3.8					74			172
		13	3.0					74			177
		14	4.0					74			183
		18	4.0					74			195
		21	2.0					65			195
Princess Park	Andesite	3	2.0	36	9	99	4	48	46	15	87
		4	2.0	36	9	99	4	48	46	15	84
Baker Square	Granite	1	8.5	34	28	39	7				279
		2	7.0	34	6	34	6				215
		3	7.0	34	28	39	7				229
		4	8.5	34	6	34	6				344
Gateway	Granite	1	2.0	43	0	37	4		34	13	235
		2	2.0	43	0	37	4		34	13	200
		3	2.0	43	0	37	4		34	13	183
		4	2.0	43	0	37	4		34	13	196
Comlib	Andesite	1	4.0					16			54

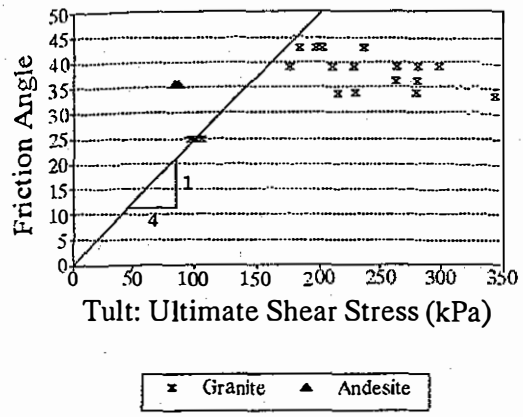

Figure 5.
Ultimate shear stress vs. effective angle of friction.

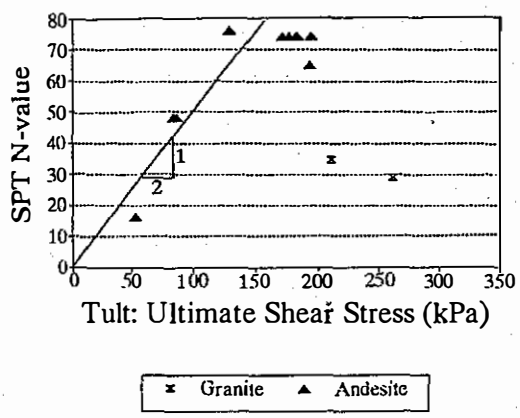

Figure 6.
Ultimate shear stress vs. SPT N-value.

$$T_{ult} = 4\,\phi \qquad (4)$$

A comparison of ultimate shear stress (T_{ult}) and SPT N-value is shown in Figure 6. From the results it is apparent that any relationship between SPT N-value and ultimate shear stress is highly dependant on soil type. A tentative design line for residual andesite (a cohesive clayey silt) is presented in Figure 6. The design line implies the following:

$$T_{ult} = 2N \qquad (5)$$

Stroud (1974) proposed a correlation between SPT N-value, plasticity index and undrained shear strength (c_u) for insensitive cohesive soils. For the residual andesite the relationship could be taken as follows:

$$c_u = 5N \qquad (6)$$

Combining equations 5 and 6:

$$T_{ult} = 0{,}4c_u \qquad (7)$$

This formula is very similar to the formulae generally used to determine the ultimate skin friction for piles in cohesive soils (Tomlinson, 1979). The value of 0,4 falls within the upper range of adhesion factors normally used for pile design in cohesive soils. The results obtained from pull out tests on cohesive residual soils therefore suggest that the procedure used in pile design to estimate the ultimate friction could be used to predict ultimate shear stress values for soil nails in cohesive soils.

The results given in Figure 6 also indicate that higher ultimate shear stress values are obtained from the residual granite soil even though the SPT N-values are much lower than those for the residual andesite. Our interpretation of this data is that the dilatancy during pull out is the predominant factor in silty sand soils such as residual granite.

The results of the other soil parameters considered (plasticity index, liquid limit, cohesion, particle size smaller than 0.075 mm and 0.002 mm) are displayed in Table 1. The ranges of these parameters were found to be insufficient to derive any definite correlations.

6. CONCLUSIONS

The ultimate soil shear stress at failure (T_{ult}) of a soil nail was found to increase as the effective angle of friction increased.

Design lines for the prediction of pull out resistance of soil nails in residual andesite and granite are suggested.

The ultimate shear stress was found to be independent of depth below ground level in residual soils.

Evidence is presented to show the importance of soil dilatancy in the prediction of soil nail pull out resistance.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the interest and cooperation of Messrs. E. Friedlaender and G. Wardle of Geofranki South Africa, for

making the data available for use in this research project.

REFERENCES

Bridle, R.J. and Barr, B.I.G. 1990. The analysis and design of soil nails. Int. Reinforced Soil Conf. Glasgow, 249-254.

Cartier, G. and Gigan, J.P. 1983. Experiments and observations on soil nailing structures. Proc. 8th ECSMFE: Helsinki.

Gässler, G. and Gudehus, G. 1983. Soil nailing -Statistical design. Proc. 8th ECSMFE: Helsinki, 491-494.

Gässler, G. 1988. Soil nailing - Theoretical basis and practical design. Int. Geotechnical Symp. on Theory and Practice of Earth Reinforcement: Fukuoka Japan, 283-288.

Jewell, R.A. 1990. Review of theoretical models for soil nailing. Int. Reinforced Soil Conf. Glasgow, 265-275.

Juran, I and Elias, V. 1988. Kinematical limit analysis approach for the design of soil retaining structures. Int. Geotechnical Symp. on Theory and Practice of Earth Reinforcement: Fukuoka Japan, 301-306.

Schlosser, F. 1982. Behaviour and design of soil nailing. Symp. on Recent Developments in Ground Improvement Techniques. Bangkok, 399-413.

Schwartz, K. and Friedlaender, E.A. 1989. Soil nailing in South Africa. Ground Profile. SAICE No. 58.

Stroud, M. A. 1974. The Standard Penetration Test in insensitive clays and soft rocks. European Symp. on Penetration Testing. Swedish Geotech. Soc. Stockholm.

Tomlinson, M. J. 1979. Pile Design and Construction Practice. Viewpoint Publications.