Available online at www.sciencedirect.com

SciVerse ScienceDirect

Procedia Engineering 57 (2013) 222 – 231

Procedia Engineering

www.elsevier.com/locate/procedia

11th International Conference on Modern Building Materials, Structures and Techniques, MBMST 2013

Influence of Pressure Grouting on the Anchors Carrying Capacity in Fine Grained Soil

Juraj Chalmovský[a], Lumír Mičа[b,]*

[a,b]*Department of Geotechnics, Faculty of Civil Engineering, Brno University of Technology, Veveri 95, 602 00 Brno, Czech Republic*

Abstract

The main purpose of this paper is to analyse the influence of pressure grouting on ground anchors pull out capacity in fine grained soils, using the finite element method (FEM). The grouting process is simulated by volumetric strain of grout body, which is determined from excavated test anchors. Three constitutive models (Mohr-Coulomb, Hardening soil model and Hypoplasticity model for clay) are applied. Obtained pull-out capacities for the cases with respectively without grouting simulation are compared with the results of appropriate analytical and empirical methods. Cylindrical cavity expansion theories are also applied for the purpose of results evaluation. The influence of consolidation process after pressure grouting simulation and the value of volumetric strain are later analysed. It can be concluded that pressure grouting simulation has a substantial effect on the anchors pull-out capacity. Consolidation analysis following grouting simulation turned to be an important factor. Results from study, in which volumetric strains were varied, confirmed that the influence of grouting diminishes with increasing volumetric strain/grouting pressure.

Selection and peer-review under responsibility of the Vilnius Gediminas Technical University

Keywords: ground anchor; pull out capacity, pressure grouting; volumetric strain; fine grained soil; finite element method; constitutive models.

1. Introduction

After the first application during the Cheurfas dam (Algeria) reconstruction in 1934, ground anchors have become widely used stabilising elements in geotechnical engineering. They are primarily used to ensure stability of retaining structures, anchoring foundation, anchoring dams and floor slabs against the effects of water buoyancy and many others. One way to significantly increase the carrying capacity of ground anchors is to use pressure grouting or post – grouting.

The purpose of the presented paper is to analyse the influence of grouting on the anchor capacity using the finite element method (FEM).

The current state of knowledge is briefly described in the second part. The third and fourth parts are devoted to the description of created mathematical models, applied constitutive models and input parameters. The results are presented in the fifth part and finally in the last part conclusions are drawn. Analyses are performed for the ground anchors situated in overconsolidated clays. The proposed article is the initial numerical study of the research project, whose aim is to refine ground anchors pull-out capacity determination. The research project is being carried out at the Brno University of Technology, Faculty of Civil Engineering and Department of Geotechnics.

* Corresponding author.
E-mail address: mica.l@fce.vutbr.cz

Selection and peer-review under responsibility of the Vilnius Gediminas Technical University
doi:10.1016/j.proeng.2013.04.031

2. Current state of knowledge about the influence of grouting on anchors pull-out capacity

Xanthakos [1] and Ostermeyer, Barley [2] stated, that the skin friction can be significantly increased by post – grouting. Ostermeyer [3] provided an example that the skin friction can be increased from about 120kN/m^2 to about 300kN/m^2 using the post – grouting technique of stiff clays with the medium to high plasticity. He also determined valuable dependence between post – grouting pressure and skin friction, see Fig. 1.

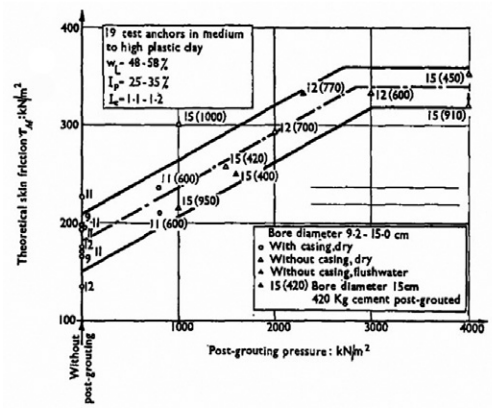
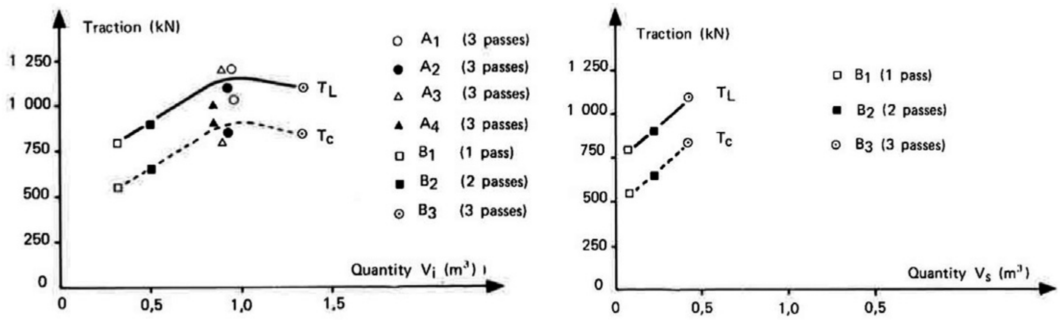


Fig. 1. Dependence between post – grouting pressure and skin friction [3]

Bustamante et al. [4] carried out anchor load tests in overconsolidated plastic clays and he derived dependence between the volume of injected grout V_i (Fig. 2a) and the volume of fixed anchor length V_S (Fig. 2b) on the number of passes during additional post – grouting and ultimate load T_{ult} .



(a) (b)

Fig. 2. Dependences between the volume of injected grout V_i (a), volume of fixed anchor length V_s (b), number of passes during additional post – grouting and ultimate load T_{ult} [4]

Jones et al. [5] proposed a modification of the standard formula Eq. (1) used for the ultimate load determination according to Littlejohn [6] where D and L are diameter of fixed anchor length and its length respectively, s_u is undrained shear strength and α is the reduction factor. The α parameter values are in the range of 0,28÷0,66. However this formula is primarily used for anchors without pressure grouting (gravity grouting).

$$T_{ult} = \pi \times D \times L \times \alpha \times s_u \,, \tag{1}$$

Jones et al. [5] also made series of loading tests on GEWI micropiles, which were equipped the post – grouting system. The modified α_{MOD} factor values were determined from back analysis of these loading tests. The increase in carrying capacity (α_{MOD} factor) due to post – grouting was approximately threefold, see Table 1.

Table 1. α_{MOD} values derived from back analysis of post – grouted GEWI micropiles [7]

Load transfer length (m)	Depth below ground level (m)	Average undrained cohesion of post – grouted soil (kN/m^2)	Derived α_{MOD} value
4	5–9	115	1,6–2,1
7	5–12	125	1,2–2,1
14	5–19	156	0,6 – 0,8

Bustamante and Doix [8] carried out an interesting study in which they derived correlation between the limit pressure p_{1m} obtained from the Menard prebored pressumeter tests and skin friction in different ground conditions. The correlation function for IGU (Global and Unique Injection) and IRS (Selective and Repeatable Injection) type of anchor in clay and silt are shown in Fig. 3. They also determine increase in anchor fixed length diameter for different soil types.

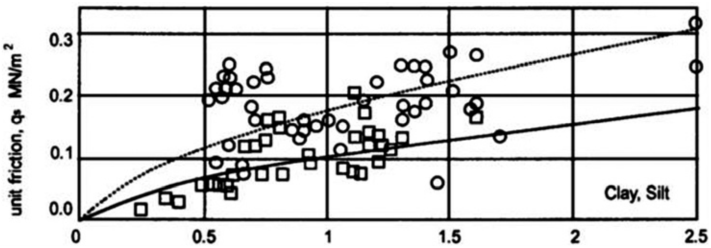


Fig. 3. Correlation between limit pressure p_{1m} obtained from the Menard prebored pressumeter tests and unit skin friction in different ground conditions (clay and silt) for IGU type of anchor (full line) and IRS type of anchor (dashed line) [8].

In the following text, only the general term pressure grouting is used. There is no distinction between different technologies of grouting.

3. Description of the created numerical model

Ground anchor geometry was based on the results of Mišove [9], who carried out extensive anchor load tests in overconsolidated Neogene clays on the testing site near the city of Brno. Ground anchors were excavated after loading tests which allows determining their fixed lengths diameters. Anchors were equipped with tube – a – manchette pipes (TAM), which allows the additional post – grouting procedure. The cross – section through the excavated fixed anchor length is shown in Fig. 4.

Axisymetric mathematical models were done in Plaxis 2D ver. 2011 software [10]. Both tendon and grout body were modelled as volumetric elements, which places high demands on mesh quality. The element connectivity plot around the bottom half of the fixed length is shown in the Fig. 5.

Fig. 4. The cross section of ground body after excavation [9]

Basic geometrical data about the modelled anchor are listed in the Table 2.

Table 2. Geometrical properties of the modelled anchor

Parameter	Value
Free length	4 m
Fixed length	8 m
Borehole diameter	156 mm
Fixed - length diameter	190 mm

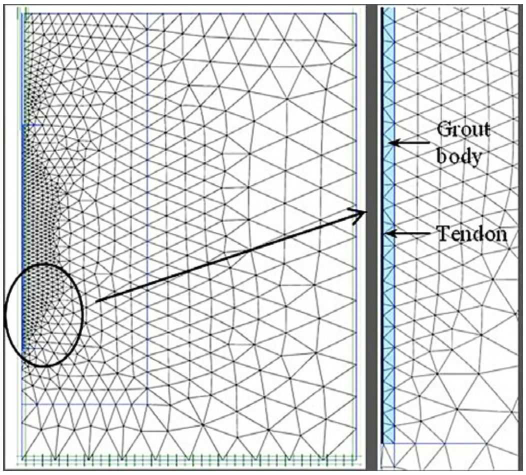


Fig. 5. Element connectivity plot for the whole FEM model an near of the bottom half of the fixed anchor length

Grout volumetric strain was chosen for the purpose of grouting simulation. The value of the volumetric strain was calculated from the borehole diameter and the fixed length diameter determined after ground anchor excavation. The delay between grouting and tensioning (10 days) was considered as consolidation analysis. Tendon was considered as linear elastic material. Mohr – Coulomb constitutive model was used for the grout material in order to limit the tensile strength of the grout according to reference [11]. Loading was simulated with the prescribed displacement in the anchor head. The initial unit prescribed displacement was increased by using incremental multipliers. Two types of models were created: one without grouting simulation (var. A) and another with grouting simulation and subsequent consolidation phase (var. B). Sequence of performed computational steps is summarized in Table 3. Details about each step are given in part 5.

Table 3. Performed computational steps

ID	Description	Objectives
1	Var. A	Determination of the influence of constitutive model
2	Var. B	Determination of the influence of pressure grouting simulation
3	Var. B	Determination of the influence of consolidation after pressure grouting simulation
4	Var. B	Determination of the influence of different volumetric strains of the grout

4. Applied constitutive models and input parameters

Three constitutive models were used: Mohr – Coulomb Model (MC), Hardening Soil Model (HS) and Hypoplastic Model for Clays (HC).

The MC model is a basic linear elastic – perfectly plastic constitutive model, which is commonly used for the purpose of numerical modelling in Czech practise. Five parameters are needed as inputs: E_{ref} – Young´s modulus, ν – Poisson´s ration, ψ – angle of dilatancy, c´– cohesion, φ´– angle of internal friction. Alternatively, it is possible to input shear modulus G and the oedometric modulus E_{oed} in the Plaxis software. Despite its prevalence, this model has several disadvantages. The soil stiffness is constant in pressure (depth). This disadvantage can be partially overcome by using linear increase of stiffness and strength parameters with depth. The MC model is not capable of modelling volumetric and shear hardening. There are no plastic strains before reaching the yield surface. The stiffness during unloading – reloading is the same as the stiffness during primary loading. The stress path followed during undrained loading do not account for the pore pressure change due to contractant / dilatant soil behaviour.

The HS model [12], [13] is elastoplastic model with double shear and volumetric hardening. Nine input parameters are required: E_{oed}^{ref} –tangent stiffness for primary oedometer loading, E_{50}^{ref} –secant stiffness in standard drained triaxial test, E_{ur}^{ref} –unloading – reloading stiffness, m – power for stress level dependency of stiffness, ψ –angle of dilatancy, c´ – cohesion, φ´– angle of internal friction, ν_{ur} – Poisson´s ratio for unloading – reloading, p^{ref} – reference stress for stiffness, K_0^{nc} – coefficient of earth pressure at rest for normal consolidation. In contrast to the MC model, the HS model involves many features of real soil behaviour: hyperbolic deviatoric stress – axial strain relationship, soil stiffness stress dependency according to power rule, stiffness during unloading – reloading is different from primary loading stiffness, plastic strains occurs during isotropic loading, memory of preconsolidation stress. The elastic zone is bounded by shear and cap yield surface. The presence of elastic zone can be considered as a HS model disadvantage. This problem is becoming an issue when dealing with highly overconsolidated soils.

Hypoplasticity presents different direction in the constitutive models development in comparison with elastoplastic models. Basic hypoplastic equations relate to the stress rate (objective Jauman stress rate) and strain rate (stretching tensor). Hypoplastic models were originally developed for cohesionless soils [14]. Subsequently, models were modified for cohesive soils [7]. The hypolastic model for clays also includes concept of critical state soil mechanics, the influence of stress state (barotrophy factor) and loading history (pyknotrophy factor). The model requires five parameters: N – void ratio at reference pressure (1kPa), λ^* – NCL line slope, κ^* – URL line slope, φ_c – critical angle of internal friction, e – initial void ratio and r – ratio between shear and bulk stiffness in isotropic virgin states.

Input parameters of Brno Neogene clay for MC (Table 4) and HS model (Table 5) were calibrated from the triaxial CIUP and oedometric tests. The parameters c´ and φ´ are the same for the HS model as for the MC model. The input parameters for the HC model (Table 6) were taken from [15].

Table 4. Input parameters for MC model

E^{ref}	c´	φ´	ν	K_0
[MPa]	[kPa]	[°]	[-]	[-]
9,35	6	24	0,35	1,21

Table 5. Input parameters for HS model

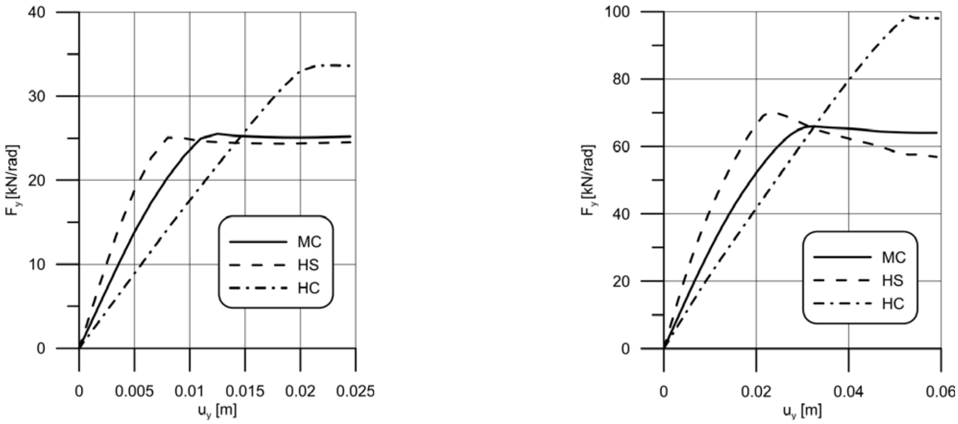
E_{50}^{ref}	E_{oed}^{ref}	E_{ur}^{ref}	m	ν_{ur}	R_f	p_{ref}	K_0	POP
[MPa]	[MPa]	[MPa]	[-]	[-]	[-]	[kPa]	[-]	[kPa]
11,86	11,86	36,17	0,55	0,2	1,0	100	1,21	1800

Table 6. Input parameters for HC model

φ_c	λ^*	κ^*	N	r
[°]	[-]	[-]	[-]	[-]
19,9	0,128	0,01	1,506	0,45

5. Results of numerical analysis

Anchor loading simulations without considering pressure grouting were performed in the first computational step for all three constitutive models. Anchor head displacement – force diagrams are shown in Fig. 6a. The greatest resistance is attained using HC model, which is partly due to different stress paths. Effective stress paths (ESP) near the anchor surface, in the middle of its fixed length, are shown in Fig. 7. Comparison between ultimate pull out forces obtained using FEM and different analytical and empirical methods is given in Fig. 8a.



(a) (b)

Fig. 6. Anchor head displacement – force diagrams – gravity grouting (a) and pressure grouting (b)

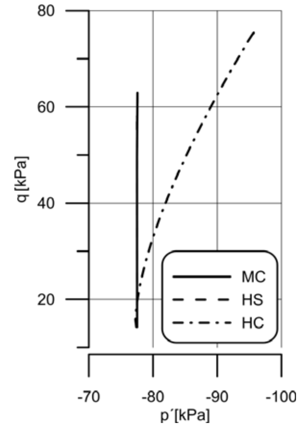
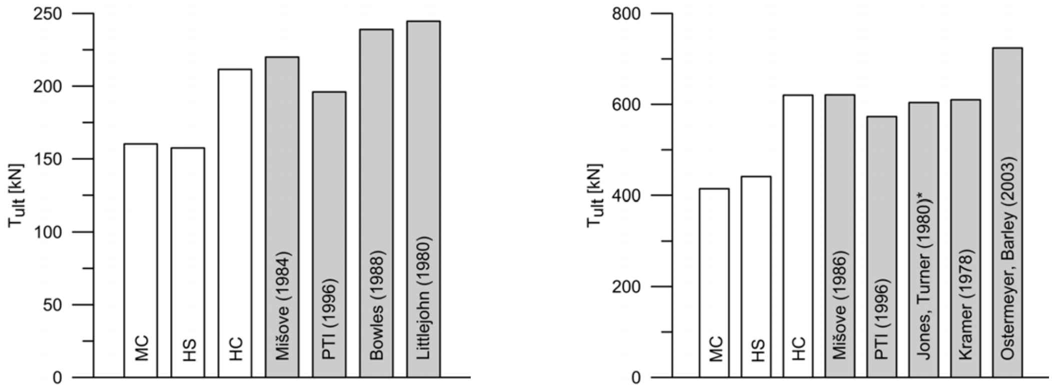


Fig. 7. Effective stress paths

Due to dilatancy of the highly overconsolidated Brno clay together with undrained shearing, additional negative pore pressure is generated in the fixed anchor length vicinity, the ESP curves right, which results in effective mean stress p´ increase and consequently also shear strength increase. This effect is, however, strongly dependent on the initial void ratio e. The effective stress paths for both HS and MC model have the same shape (perpendicular to the effective mean stress p´ axis). This is due to using the POP parameter in HS model - the initial position of volumetric plasticity surface is shifted (as POP or OCR functions) and elastic domain increases. The best match between FEM results and empirical, analytical methods is obtained when applying the HC model.

The simulation of pressure grouting using volumetric strain concept was added to the numerical models in the second computational phase. The resulting anchor head displacement – the force diagrams for each constitutive model and comparisons with analytical and empirical methods are shown in Fig. 6b and Fig. 8b.

Pressure grouting simulation results in average two and half times increase in anchor pull out capacity. This is in good match with the findings of Veloso [16] and Ostermeyer [3], who recorded three times increase in average contact skin friction due to post – grouting. Such a difference is caused partially by the fixed length diameter increase but mainly by the radial effective stress increase and consequently the shear strength increase. Clarke [17] compared grouting process to Menard's prebored pressumeter test, which is essentially the expansion of the cylindrical cavity. Mecsi [18] also used the analogy between the pressure grouting and the cylindrical cavity the expansion theory. It is therefore possible to compare results obtained using FEM with cylindrical cavity expansion theories. Three of them were applied - Vesic [19]; Randolph et al. [20] and Cunze [21]. The excess pore pressures after grouting simulation are compared at the horizontal cut in the middle of the fixed length see Fig. 9. The resulting excess pore pressures for the HS were higher than for the MC model. It is probably due to the additional pore pressures generated during the undrained cavity expansion which is basically a shearing process. The theory of Vesic agrees better with the results for the MC model. It is understandable, the Vesic theory was based on linear elastic – perfectly plastic material model. On the contrary, theories of Randolph et al. [20] and Cunze [21], both incorporating more general elasto – plastic soil behaviour, are in better match with results for HS model. Generated pore pressures for the HC model were negative, which is in contrast with results when using HS, MC model and previously mentioned cavity expansion theories. D´Appolonia and Lambe [22] applied the undrained triaxial extension test for cavity expansion simulation. It is known that for high OCR values negative, pore pressure may arise during the undrained triaxial extension test. From this point of view, existence of negative pore pressure after cavity expansion is understandable. Their magnitude is, however, strongly dependent on the OCR.



(a) (b)

Fig. 8. Ultimate pull out forces comparisons - gravity grouting (a) and pressure grouting (b) (*effectivity coefficient according to [23])

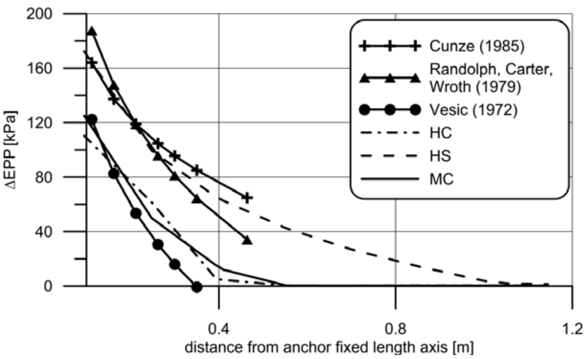


Fig. 9. Dependence between distance from anchor fixed length axis and excess pore pressures after grouting simulation

In the third step, consolidation following the grouting simulation was analysed for HS and HC model. The Consolidation analysis, up to various degrees of consolidation (25%, 50%, 75% and 90%) was therefore added to calculation phases. The anchor loading followed each consolidation analysis. The radial excess pore pressure dissipation occurred during consolidation. The decrease in excess pore pressures for different degrees of consolidation is shown in Fig. 10a for HS model and in Fig. 10b for HC model. Excess pore pressure dissipation turns to be an important factor in anchor pull out resistance. Fig. 11 show the anchor head displacement – force diagrams for both constitutive models after consolidation analysis. Fig. 12a shows ultimate pull out forces as a function of the degree of consolidation.

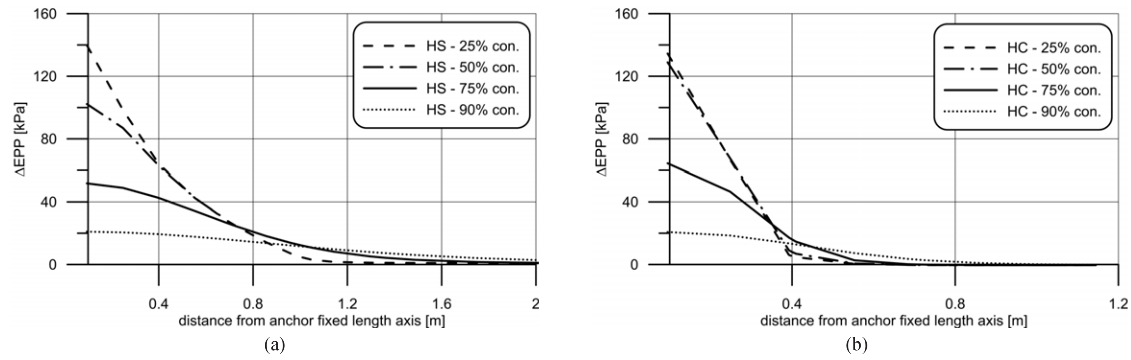


Fig. 10. Dependence between distance from anchor fixed length axis and excess pore pressures for different degrees of consolidation – HS (a) and HC (b) models

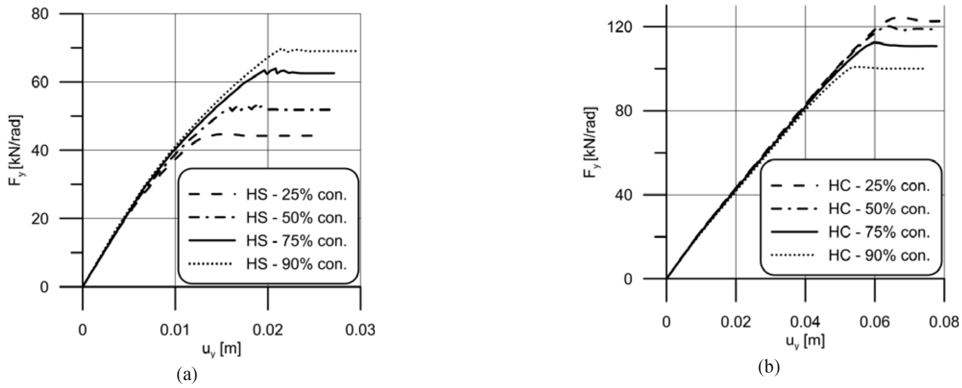


Fig. 11. Anchor head displacement – force diagrams for HS (a) and HC (b) model and different degrees of consolidation

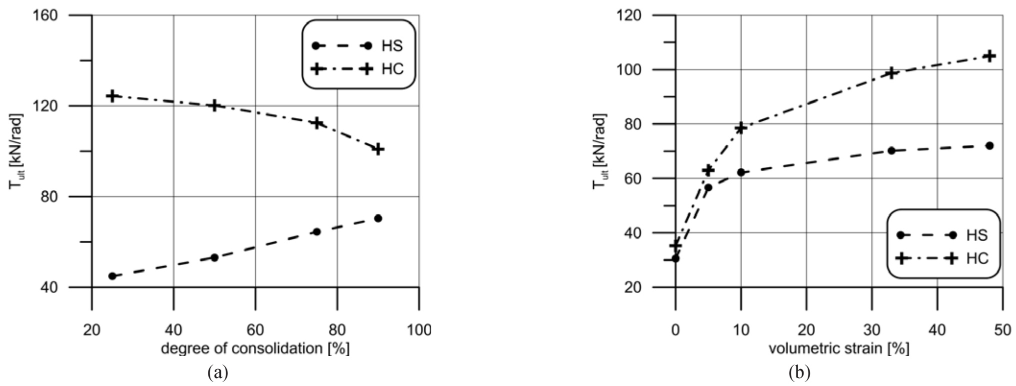
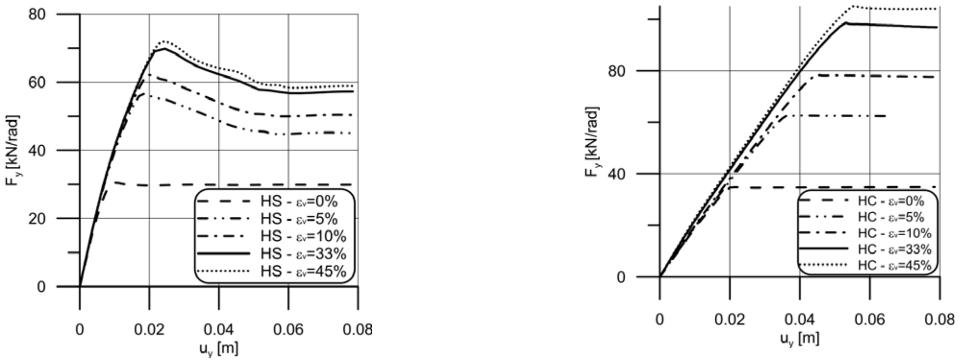


Fig. 12. Degree of consolidation –ultimate pull out force dependence (a) and volumetric strain – ultimate pull out force dependence (b)

The contradictory results are caused by opposite pore pressures generated due to pressure grouting. In the case of the HC model, negative pore pressures are dissipated, which causes the effective mean stress and consequently shear strength decrease. The pull-out force therefore decreases with increasing degree of consolidation. When using the HS model, the situation is opposite – the decrease in positive pore pressures causes the effective stress and shear strength increase. The ultimate pull-out force is therefore proportional to the degree of consolidation. Based on the literature review, there is no clear answer to the time dependence of ground anchors pull out capacity. However, there are resources dealing with time dependent capacity of driven piles usually in soft soils. Pile driving is essentially a process of creating cavity, therefore some of the conclusions are presented here. Chen et al. [24] and Titi, Wathugala [25] both concluded that in soft marine clays, the pull-out capacity and skin friction increase with time after pile driving. It is caused by dissipation of high pore pressures in remoulded zone near pile surface which leads to higher undrained shear strength [24]. Smoltczyk [26] states that for soft soils, the pile capacity increases with time but in dense soils it decreases with time due to consolidation and swelling in the vicinity of pile. This conclusion would, justify the results obtained for the HC model but there are not enough tests for conformation.

Ostermeyer [3] found out that the increase in average contact skin friction (Fig. 1) is negligible from a certain value of grouting pressure. Therefore, in the last computational step, grout volumetric strain was varied. Originally, the increase in the anchor fixed length diameter from 156mm to 190mm corresponds to 33% volumetric strain of grout. Three other volumetric strains were used - 5%, 10%, and 48%. Fig. 13 show the anchor head displacement – force diagrams for different volumetric strains for HS a and HC b models. The dependence between the volumetric strain and ultimate pull out force is shown in Fig. 12b. Results for both HS and HC model confirm that the increase in ultimate pullout force is from certain volumetric strain rather small. This phenomenon is more obvious for the HS model.



(a) (b)

Fig. 13. Anchor head displacement – force diagrams for HS (a) and MC (b) model and different volumetric strain

6. Conclusions

Partial conclusions for each phase are given in the previous chapter. In general, it might be stated that it is possible to partly model the effect of pressure grouting using the volumetric strain tool. A significant increase in anchor pull out capacity was confirmed. The consolidation analysis following grouting simulation turned to be an important factor. However, two opposite results by applying the HS and HC models were found. This is due to the generation of opposite pore pressures during the grouting simulation. The authors did not find clear answer for this problem in any available literature. Therefore, the time dependence anchor pull out capacity issue is subject to further investigation.

The results from the study in which volumetric strains were varied, confirmed that the influence of grouting diminishes with increasing volumetric strain/grouting pressure. The disadvantage of the purposed procedure is in using volumetric strains, not in the grouting pressure. The volumetric strain (increase in diameter) cannot be estimated from the amount of injection grouting mixture due to losses during grouting process. It is, however, possible to use empirical data such those in PTI 1996 [27], Recommendations T. A. 95 [28] or Mišove [9], in which diameters of fixed anchor length for different soil types are given. The proposed procedure also does not include creation of fractures due to grouting. The above presented results were obtained from initial numerical studies. The numerical models will be still improved and elaborated in next stage of the research project.

Acknowledgements

The authors greatly appreciate the financial support primarily by the research project FR-TI4/329 MPO – TIP and also project OP VAVpI CZ.1.05/2.1.00/03.0097 AdMaS.

References

[1] Xanthakos, P., 1991. Ground anchors and anchored structures. John Wiley and Sons, Ltd, p. 704.
[2] Ostermayer, H., Barley, A. D., 2003. Fixed Anchor Design Guidelines. Geotechnical Engineering Handbook, Vol. 2. Pub. Ernst & Sohn, p. 17.
[3] Ostermayer, H., 1975. "Construction, carrying behaviour and creep characteristics of ground anchors", Proceedings Conf. on Diaphragm Walls and Anchorages, 18-20 September. London, pp. 141-151.
[4] Bustamante, M., Delmas, F., Lacour, J., 1978. Behaviour of prestressed anchors in plastic clay, Revue francaise de géotechnique 3, pp. 79-87.
[5] Jones, D. A., Spencer, I. M., 1984. Clay anchors – A Caribbean case history, Ground Engineering 17(1), pp. 36-42.
[6] Littlejohn, G. S., 1980. Design Estimation of the Ultimate Load-Holding Capacity of Ground Anchors, Ground Engineering 13(8), pp. 25-39.
[7] Mašín, D., 2005. A hypoplastic constitutive model for clays, International Journal for Numerical and Analytical Methods in Geomechanics 29(4), pp. 311-336.
[8] Bustamante, M., Diox, B. 1985. Une methéde pour le calcul des traints et des micropieux injectés, Bull. De Liaison de LCPC, paris, 140, pp. 75-92.
[9] Mišove, P., 1984. Konštrukcia predpätých horninových kotiev a ich únosnosť, Kandidátska dizertačná práca. VÚIS, p. 184. Slovakia (in Slovak).
[10] Brinkgreve, R. B. J. et al., 2011. Plaxis Manual. Plaxis bv., The Netherlands.
[11] Mayer, A., 2008. FE – Berechnungen zum Tragverhalten von Ankern in Boden. Diploma thesis, TU Graz, Austria.
[12] Schanz, T., 1998. Zur Modellierung des Mechanischen Verhaltens von Reibungsmaterialien, Habilitationsschrift, Mitteilung 45 des Instituts für Geotechnik der Universität Stuttgart, p. 152.
[13] Schanz, T., Vermeer, P. A., and Bonnier, P. G., 1999. "The hardening soil model: Formulation and verification", Proceedings "Beyond 2000 in Computational Geotechnics – 10 years of Plaxi". Balkema, Rotterdam, pp. 1-16.
[14] Von Wolffersdorff, P.-A., 1996. A hypoplastic relation for granular materials with a predefined limit state surface, Mechanics of Cohesive-Frictional Materials 1, pp. 251-271.
[15] Svoboda, T., Mašín, D., 2008. "Impact of a constitutive model on inverse analysis of a NATM tunel in stiff clays", Proceedings ITA-AITES World Tunnel Congress, Vol. 1, Kanjlia, V. K., et al. (Eds.), Agra, India, pp. 627-636.
[16] Veloso, G. A., 2007. How to Enhance Ground Anchors Capacity in Soft Soils, (http://www.leeds.ac.uk/ygg/Veloso%20Presentation.pdf)
[17] Clarke, B.G., 1995. Pressuremeters in geotechnical design, in Blackie academic and professional, Glasgow, UK, p. 364.
[18] Mecsi, J., 1997. "Some practical and theoretical aspects of grouted soil anchors", Proceedings ICE Conference on Ground Anchors and Anchored Structures, March, London, pp. 119-130.
[19] Vesic, A. S., 1972. Expansion of cavities in infinite soil mass, J. Soil Mech. Found. Div. 98(3), pp. 265–290.
[20] Randolph, M. F., Carter, J. P., Wroth, C. P., 1979. Driven piles in clay – the effects of installation and subsequent consolidation, Géotechnique 29(4), pp. 361-393.
[21] Cunze, G., 1985. Ein Beitrag zur Abschätzung des Porenwasserüberdruckes beim Rammen von Verdrängungspfählen in bindigen Böden: Dissertation. Uni Hannover.
[22] D´Appolonia, D. J., Lambe, T. W., 1971. Performance of four foundations on end bearing piles, Journal of Geotechnical Engineering 8, pp. 434-455.
[23] Barley, A. D., 1995. "Theory and practice of the single bore multiple anchor system", International symposium on anchors in theory and practice. Salzburg, Austria, pp. 287-293.
[24] Chen, C.S., Liew, S.S. & Tan, Y.C., 1999. "Time effects on the bearing capacity of driven piles", 11th Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, Balkema, Rotterdam, Hong *et al.* (Eds.), pp. 175-178.
[25] Titi, H. H., Wathugala, G.W., 1999. Numerical Procedure for Predicting Pile Capacity- Setup/ Freeze, Journal of the Transportation Research Board, TRR No. 1663, National Research Council, Washington, D.C., pp. 25-32.
[26] Smoltczyk, U., 2003. Elements and Structures, Geotechnical Engineering Handbook, Volume 3, John Wiley & Sons, p. 666.
[27] Post Tensioning Institute PTI Recommendations for prestressed rock and soil anchors. PTI Phoenix, 1996, p. 108.
[28] Recommendations T.A. 95 Recommendations concernant la conception, le calcul, l'exécution et le controle des tirants d'ancrage. Bureau Sécuritas, Editions Eyrolles, Paris, p. 149.