Design of anchored retaining structures by numerical modelling

Calcul des ouvrages de soutènement ancrés par modélisation numérique

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ABSTRACT

The use of advanced and widely available commercial finite element computer programs in design of anchored retaining structures confront designers with several challenges that may, if not properly accounted for, lead to poor and even unsafe design. The paper addresses this issue and proposes a strategy for numerical modelling of retaining structures in medium stiff to stiff soils. The proposed strategy is verified on three case histories and a good agreement is achieved between calculated and measured displacements of the retaining structures. Measures the designers could take to ensure the structural safety, particularly with regard to its brittleness, are also proposed.

RÉSUMÉ

L'emploi des méthodes numériques de calcul par éléments finis pour les ouvrages de soutènement ancrés confronte les concepteurs avec plusieurs défis qui pourraient conduire aux calculs faibles et même aux résultats qui ne sont pas de côté de la sécurité. L'article adresse ce problème et propose une stratégie pour la modélisation numérique des ouvrages de soutènement dans les sols de moyenne rigidité et les sols rigides. La stratégie proposée est vérifiée par trois cas concrets. Les mouvements des ouvrages de soutènement calculés et mesurés sont bien en accord. Les mesures que les concepteurs pourraient prendre pour assurer la sécurité de la construction, spécialement en ce qui concerne sa fragilité sont aussi proposées.

Keywords : retaining structures, anchors, numerical modelling, parameter selection, brittleness, structural safety, observations

1 INTRODUCTION

The design of anchored retaining structures providing lateral support for deep temporary excavations in urban areas was traditionally based on simple numerical models. The selection of model parameters, only obscurely related to measurable soil properties and underlying soil-structure interaction mechanisms, required considerable engineering judgment, thus introducing distracting uncertainties into design decisions. On the other hand, recent developments and sophistications of commercially available finite element computer programs enable detailed and physically more plausible analyses of very intricate aspects of soil-structure interaction during various construction stages. This remarkable advancement in modelling capabilities, however, put before designers new challenges that may, if not properly accounted for, lead to poor, unsafe or even disastrous design (Simpson 1992, Schweiger 2002, Potts & al. 2002, Gaba & al. 2003). These challenges include the selection of appropriate soil constitutive models together with the proper selection of stiffness, strength and initial stress soil parameters, modelling undrained, transitional and drained soil behaviour, and taking into account possible brittleness of the anchoring system. The also include the interpretation of observations during construction in relation to computed values in the analysis when assessing the structural safety.

Gaba et al. (2003) stated, among others, the following reasons for these problems: inadequate constitutive models, unreliable data on soil strength, stiffness and initial stresses, inadequate modelling of undrained conditions in fine grained soils and insufficient user experience with the particular programme. They state that "Case-history-based empirical methods of prediction are to be preferred to the use of complex analyses, unless such analyses are first calibrated against reliable measurements of well-monitored comparable excavations and wall systems".

The aim of this paper is an attempt to calibrate nonlinear finite element analyses against measurements of displacements of three different anchored retaining walls embedded into gravelly and stiff clay layers. These retaining walls were constructed as temporary structures securing two excavation pits for underground car parks of two commercial buildings in Zagreb, Croatia. A strategy for modelling soil behaviour and selecting relevant soil parameters was first established and calibrated against Wall No. 1. Then, the same strategy was used in modelling Wall No. 2 and Wall No. 3 followed by a comparison of calculated and measured wall displacements.

2 CASE HISTORIES

2.1 *Modelling strategy*

In a recent paper, Szavits Nossan (Szavits Nossan 2008) proposed a provisional strategy for selecting soil stiffness, strength and initial stress parrameters for the hardening soil constitutive model with small strain stiffness for soils (HSs model) available in the commercial computer program Plaxis 2D – Version 8 (Brinkgrave & al. 2006). The model is of elastic-plastic istropic hardening type with two yield surfaces, for compression and shear, and incorporates some well established stress-strain relationships for soils (Schanz & al. 1999) including provisions for incrased stiffness and hysteretic nonlinearities at small strains (Benz 2007). The proposed strategy for soil parameter selection is restricted to gravels and stiff clays since it was calibrated against measured horizontal displacements of an anchored diaphragm wall for a 13.5 m deep temporary excavation into ground containing these soil types.

The main elements of the proposed strategy are as follows:

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(a) Effective stress Young's modulus at very small strains is to be determined from shear wave velocity measurements determined by geophysical testing at the site assuming an appropriate Poisson's ratio (say 0.2); The same Young's modulus and Poisson's ratio should be selected for the unloading-reloading conditions;
(b) Effective stress Mohr-Coulomb peak strength parameters (cohesion and friction angle) for stiff clay is to be determined from conventional consolidated undrained triaxial tests with pore water pressure measurements on undisturbed soil samples at points of maximum principal effective stress ratios (rather than at points of maximum stress difference);
(c) Effective stress peak friction angle for gravelly soil may be determined from the correlation with normalized SPT blow count, $(N_1)_{60}$, as proposed e.g. by Hatanaka & Uchida (Hatanaka & Uchida 1996), see also Mayne et. al. 2001;
(d) Dilatancy at peak strength for gravelly soil is to be determined on the basis of Rowe's stress-dilatancy relationship using either measured or assumed constant volume friction angle (φ_{cv}), as proposed in the Plaxis manual (Brinkgrave & al. 2006);
(e) A zero value of dilatancy at failure should be assigned for the stiff clay deposit despite a positive value measured in triaxial tests. This inconsistency is proposed as a convenient design compromise on the safe side necessary to alleviate the deficiency of the HSs model to properly account for undrained shear strength of stiff clays from effective stress soil parameters. This compromise will give calculated undrained shear strength considerably smaller than the one measured in laboratory on stiff clay samples;
(f) The reference shear strain, $\gamma_{0.7}$, and other stiffness parameters in the HSs model for the stiff clay, as defined in the Plaxis manual, should be adjusted by trial-and-error to values which force the normalized secant modulus reduction curve of the HSs model close to the function proposed by Fahey & Carter (1993):

$$\frac{E}{E_0} = 1 - \left(\frac{q}{q_f}\right)^g \tag{1}$$

where E is the undrained secant Young modulus, E_0 its value at very small strains, q is the principal stress difference, and q_f its value at failure. According to Mayne et al. (2001) and substantiated by laboratory experiments on silty sands by Lee et al. (2004), the exponent g in equation (1) should be selected from the range between 0.2 and 0.4. In addition, a further adjustment of stiffness parameters should be performed so as to match the calculated principal strain at failure, obtained by a simulated triaxial consolidated undrained compression test, with the same strain measured in a corresponding triaxial consolidated undrained triaxial test on an undisturbed sample. A similar procedure should be used to select the $\gamma_{0.7}$ value and other stiffness parameters for the gravelly deposit, excluding the part related to matching the calculated and the measured principal strain at failure from a triaxial test, which is usually not available. It is to be noted that the $\gamma_{0.7}$ value thus obtained is considerably smaller than the value recommended in the Plaxis manual;
(g) The influence of the over consolidation ratio (*OCR*) of the stiff clay on the coefficient of earth pressure at rest, K_0, should be taken into account for the stiff clay deposit, e.g. as proposed by Mayne and Kulhawy (1982);
(h) Other HSs model parameters may be selected as proposed by Plaxis 2D manual.

Using this strategy and the finite element program Plaxis 2D, the soil-structure analysis was performed for Wall No. 1, taking into account all significant stages of loading, soil excavation, anchor prestressing, and full consolidation after completion of the excavation. An acceptable match of calculated and measured displacements of the diaphragm wall was achieved, judged by standards tolerable in an average practical project. As a by product of the analysis, a provisional correlation with $(N_1)_{60}$ for the selection of the secant Young's modulus at 50 % mobilized shear strength for a reference effective stress of 0.1 MPa, referred to as E_{50}^{ref} in the Plaxis manual (Brinkgrave at al. 2006), was determined as

$$E_{50}^{ref}\ (\text{MPa}) \approx 5\ (N_1)_{60} \tag{2}$$

It was also shown that the disregarding of small strain soil stiffness does not alter calculated horizontal displacements of the diaphragm wall for a practically significant amount.

This paper presents two additional excavation case histories with corresponding analyses. These case histories deal with different anchored retaining structures embedded in ground with similar soil conditions as in the first case history. Measured horizontal displacements of the retaining structures were compared with those obtained by Plaxis, applying the same strategy for selecting HSs model parameters.

2.2 *Soil profiles*

The basic parameters of the foundation soil deposits encountered at two close sites are shown in Figure 1. At the site A two types of anchored retaining walls were used (Wall No. 1 and Wall No. 2), while at site B a third wall type was used (Wall No. 3). All sites are characterized by a thin surface layer of man made fill and medium stiff clay, underlain by a layer of medium dense poorly graded gravel with rounded grains, which rests on a very thick layer of stiff, overconsolidated clay of medium to high plasticity. Although a noticeable drop in the normalized SPT blow count $(N_1)_{60}$ below the ground water level in the gravelly layer is apparent, a common peak friction angle was used in the analysis. Although the two sites are stratigrafically similar and not very far from each other, they yet slightly differ with relation to SPT blow counts at respective depths. Site A was more extensively tested (SPT and down-hole geophysical measurements of shear wave velocities up to 30 m depth, triaxial consolidated drained tests on undisturbed clay samples), whereas at site B no triaxial tests and no geophysical in situ tests were performed. Based on similar profiles of SPT blow counts in the stiff clay layers at both sites, it was assumed that effective strength parameters for these layers were the same at both sites, while differences in blow counts rendered different peak friction angles for gravelly layers. The detailed analysis and selected model parameters for site A may be found in Szavits Nossan (2008). The same soil parameters were used for site B except for the peak friction angle for the gravelly layer.

2.3 *Anchored retaining walls*

Three different anchored retaining walls were constructed at sites A and B. Their cross sections are shown in Figures 2, 3 and 4 respectively.

The retaining wall No. 1 is a reinforced diaphragm of cast in place concrete, anchored by three rows of BBR type prestressed high grade steel ground anchors. The excavation depth, measured from the top of the wall, was 13.5 m. The retaining wall No. 2 is a Larsen type sheet pile wall driven into the ground and anchored by two rows of BBR type high grade steel prestressed ground anchors. The excavation depth, measured from the top of the wall, was 9 m. The retaining wall No. 3 is also a Larsen type sheet pile wall driven into the ground with two rows of Ischebeck Titan type ground anchors. The excavation depth, measured from the top of the wall, was 7 m. The characteristics of structural elements of the three anchored retaining walls are listed in Table 1.

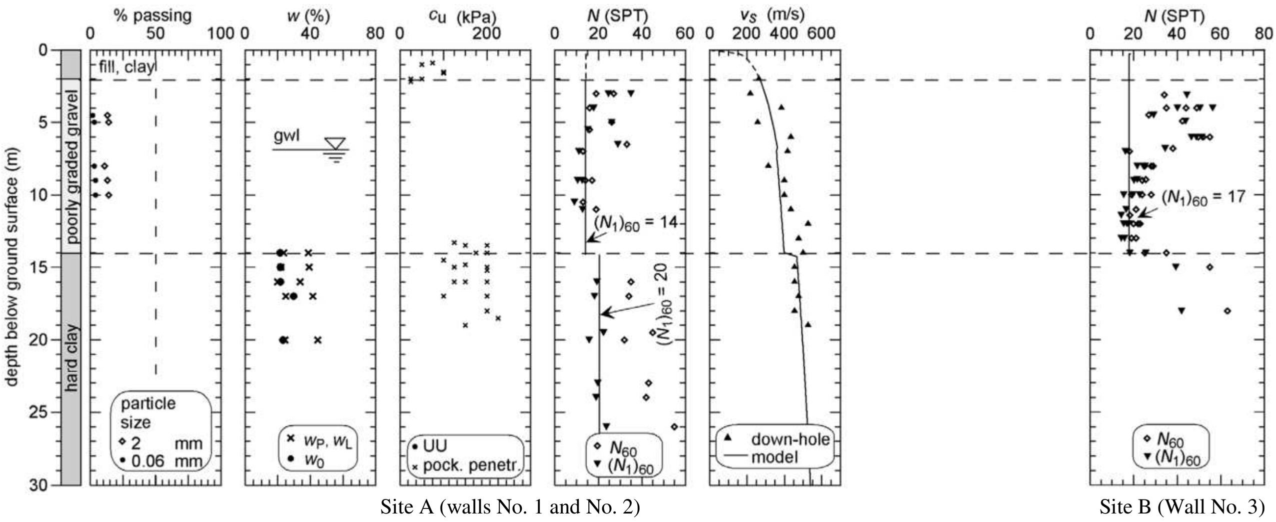

Figure 1 The soil profile at site A for walls No. 1 and No. 2 (left) and at site B for Wall No. 3 (right)

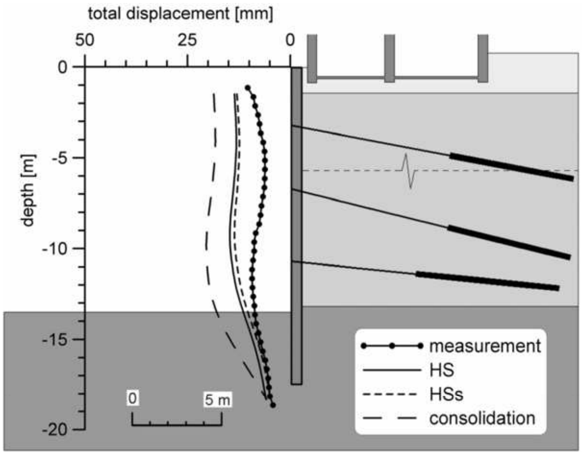

Figure 2 Anchored retaining wall No. 1 at site A: layout and horizontal displacements

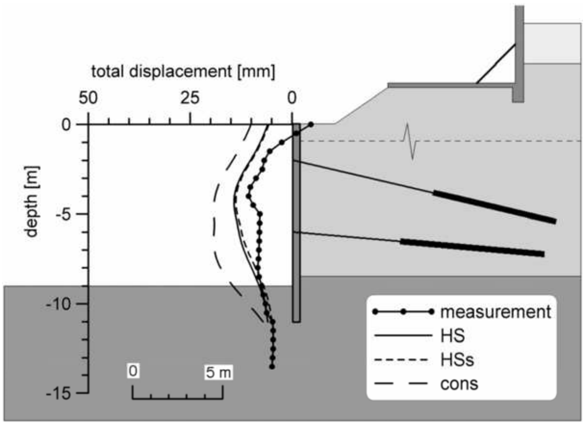

Figure 3 Anchored retaining wall No. 2 at site A: layout and horizontal displacements

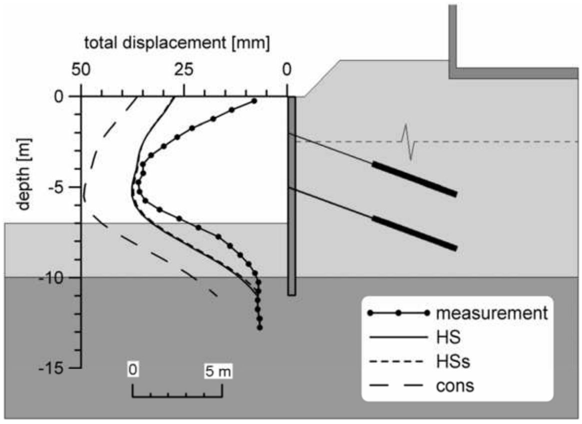

Figure 4 Anchored retaining wall No. 3 at site B: layout and horizontal displacements

2.4 *Analysis and comparison with measured displacements*

The soil-structure interaction analyses for all three anchored retaining walls were performed by the program Plaxis 2D Version 9 using the stage construction option. This allowed detailed modelling of various construction sequences: excavation and ground water lowering, anchor installation, anchor prestressing, etc. Figures 2, 3 and 4 show calculated total horizontal displacements in the bottom stiff clay stratum after reaching the final excavation phase for undrained conditions

Table 1 Design characteristics of structural elements of anchored retaining walls

Characteristics	Wall No. 1	Wall No. 2	Wall No. 3
Wall	Diaphragm	Sheet pile	Sheet pile
Type	Cast-in-place (t = 0.6 m)	Larsen 43	Larsen AU14
Stiffness (EI), MNm^2/m	450	50	21
Plastic bending moment, MNm/m	0.4	0.298	0.212
Anchors	Prestressed	Prestressed	Prestressed
Type	BBR 1860/1660	BBR 1860/1660	Ischeb. 40/20
Number of rows	3	2	2
Spacing, m	2.5	2.0	1.5
Num. of strands per row	4 (first row) 5 (lower rows)	4 (first row) 5 (lower rows)	-
Stiffness (EA), MN	117/146	117/146	150
Tendon strength, MN	0.844/1.06	0.844/1.06	0.33
Prestressing force, MN	0.5/0.6	0.3	0.165

(full lines) and after full dissipation of induced excess pore water pressures (long dashed lines). Both, the undrained and drained analyses were performed by disregarding the strong nonlinear soil behaviour at small strains (soil model HS). The undrained analyses were also performed by taking into account the small strain behaviour (soil model HSs). These results are shown in Figures 2, 3 and 4 by short dashed lines. Despite opposite expectations, they differ very little from the results with the HS soil model for the undrained analyses. The probable reason for the small difference is the high mobilization of the soil shear strength where small strain behaviour has little influence.

The calculated horizontal displacements of the anchored walls were compared with measurements taken during excavation stages by inclinometers, with tubes which were either embedded into the reinforced concrete diaphragm wall or installed into the ground on the back side of the wall. Since inclinometers measure only the relative horizontal displacements (displacements only up to the rigid body translation), the comparison with calculated displacements was obtained by adding a constant value to measured results so as to match measurements with the calculated horizontal displacement at the bottom of the retaining wall. Such "corrected" measurements are shown in Figures 2, 3 and 4 by dotted lines. According to the present general prediction quality in geotechnical engineering, particularly regarding anchored retaining structures, a remarkable agreement of calculated and measured wall displacements was achieved. This is a promising result for the proposed modelling strategy for anchored retaining structures in medium stiff to stiff soils.

3 YIELDING AND BRITTLENESS OF STURCTURAL ELEMENTS AND GLOBAL STRUCUTRAL SAFETY

In order to assess the behaviour of structural elements and the global structural safety, another set of analyses was performed with a retaining wall similar to the one of three case histories, by using the proposed strategy.

After modelling the last construction stage, several phi-c reduction procedures were used. In this procedure the soil shear strength is gradually decreased until the soil collapse occurs. This was done using various assumptions on the strength of structural elements. Four cases were considered: (1) all structural elements have elastic behaviour; (2) the wall may develop a plastic hinge, but retains its strength with large deformations (ductile elastic-plastic behaviour); (3) the anchor tendons may also yield but retain as well their strength for large deformations (ductile elastic-plastic behaviour); (4) anchor tendons may break and loose their strength completely for deformations beyond the yield point (brittle elastic-plastic behaviour).

The results of these analyses are shown in Figure 5. It is obvious that the brittle anchor behaviour greatly reduces the global structural safety. Although not often considered in the usual design practice, the brittleness of the retaining structure should be taken into account, and possibly avoided by sufficient tendon strength.

4 CONCLUSIONS

A modelling strategy for use in advanced nonlinear finite element analysis in design of anchored retaining structures in medium stiff to stiff soils was proposed. It was calibrated against one and checked against another two case histories. The strategy employs a combination of known aspects of soil behaviour with routine laboratory and field test results properly interpreted. An advanced commercially available finite element computer code was used for numerical modelling. The comparison of calculated and measured retaining wall displacements is very encouraging.

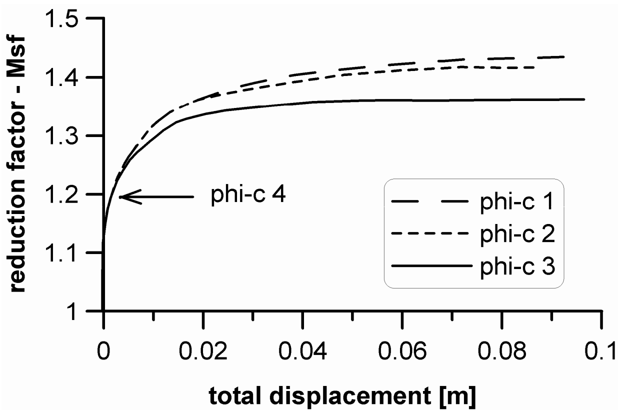

Figure 5 Development of the phi-c reduction factor with wall displacements after the last construction stage using different assumptions on the strength of structural elements: infinite strength (1), ductile strength of the wall only (2), ductile strength of the wall and anchor tendons (3), ductile strength of the wall and brittle strength of anchor tendons (4).

It was also demonstrated by numerical modelling that the structural brittleness should be taken into account in order to properly assess the global safety of the structure. The proposed strategy allows for this analysis.

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