Geotechnical Aspects of Underground Construction in Soft Ground – Ng, Huang & Liu (eds)

Analysis and numerical modeling of deep excavations

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ABSTRACT: The sixteen papers comprising the general theme "Analysis and Numerical Modeling of Deep Excavations" of the 6th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground are summarized herein. General characteristics of all papers are presented as are brief summaries of each paper. Most papers included presentation of results of finite element simulations and attendant comparisons with various aspects of observed field performance. Some of the pitfalls for making comparisons between numerical results and field observations of deep excavation performance are discussed briefly. In particular, the effects of modeling construction details and selection of appropriate constitutive models are presented. Recommendations are tendered regarding the essential information that should be conveyed in papers that present results of numerical calculations.

1 INTRODUCTION

Sixteen papers concerning analysis and numerical modeling of deep excavations were published in the Proceedings of the 6th International Symposium on Geotechnical Aspects of Underground Construction in Soft Ground. Of these papers, ten were presented orally at the symposium. An overview of the papers is made to provide a snapshot of the state of the practice as regards to this topic. All papers are summarized and trends in the contents are discussed. Because most papers included presentation of results of finite element simulations and attendant comparisons with various aspects of observed field performance of deep excavations, general comments regarding factors that can be important in order to make an accurate prediction. Finally, recommendations are tendered regarding the essential information that should be conveyed in papers that present results of numerical calculations.

2 OVERVIEW

The sixteen papers covered the broad topics summarized in Table 1. The classification is somewhat arbitrary and several papers could have fit into more than one topic. It is clear, however, that the majority of papers explicitly included comparisons of computed results and some type of performance data.

Of these 16 papers, results of finite element analyses were reported in ten of them. Table 2 summarizes the FE codes that were used. These results agree with the author's experience that the commercial codes FLAC and PLAXIS are most commonly used in both geotechnical practice and research. The ease of use of these codes has progressed to the point where three-dimensional analyses have become more common, as suggested by the number of such analyses presented in the papers in this session.

While the use of finite element analyses has become more common in practice, likely as a result of the

Table 1. Topics and number of papers.

Topic	No. of papers
Numerical analysis and measurements	7
Numerical analysis	3
Back-analysis	2
Measurements	1
Design	1
Stress path	1
Earth pressure	1

Table 2. Summary of finite element codes used in papers.

Analyses	Number and code used
Three-dimensional simulations	2 -Plaxis 3D Foundations 2- FLAC3 2-Research code (?)
Plane strain simulations	6-Plaxis 1-GeoTunnel 1-Research code (?)

Table 3. Constitutive model summary.

Model	No. of applications
Mohr-Coulomb	6
Hardening Soil	3
Duncan-Chang	1

improved i/o of commercial codes, the accuracy of the results depends the faithful representation of activities that induce stress changes in the ground during excavation and on the constitutive responses assumed in the analyses. Simple models are easy to use, but are limited in the types of computed responses that will agree with observations.

A key aspect to applying finite element analysis to practical problems in geotechnics remains the selection of the soil model and its individual parameters. Of the ten papers that presented results of FE analyses, six assumed soil responded as a Mohr-Coulomb material, as shown in Table 3. This assumption limits the predictive capabilities of a FE simulation of deep excavations or tunneling in that elastic response is assumed until the soil fails. Presumably, this choice was made because of lack of detailed laboratory or field characterizations of the soils considered by the authors in their papers. At the cost of simplicity, the Hardening-Soil and Duncan-Chang models are non-linear and can account for different responses in loading and unloading. But these models also are limited in their predictive capabilities in that they do not account for the incremental non-linearity and small strain stiffness responses that all soils exhibit. In any case, the factors leading to the selection of a soil model should be discussed in a paper to put the results in context. In the same spirit, the parameters and a rationale for their selection also should be included in a paper.

3 SUMMARY OF PAPERS

3.1 *Numerical analysis and measurements*

Li and Huang presented "Construction monitoring and numerical simulation of an excavation with SMW retaining structure." A SMW retaining structure was used to support two long excavations in Shanghai. Bearing and deformation mechanisms of the SMW were analyzed briefly and the structural analysis of SMW was discussed. Based on the in-situ excavation procedures, the authors simulated construction of the wall numerically using the FE code FLAC3D. They represented soil behavior with a Mohr-Coulomb model and considered two cases. Case 1 was the typical construction situation at the site wherein the supports were installed in a timely fashion. Case 2 considered the situation wherein the supports were not installed in a timely fashion in the lateral direction, thereby leaving excessively large amounts of wall without lateral support. They compared the computed deformations of the retaining structure, the horizontal displacement at the top of SMW and the axial forces of steel pipe supports with the field observation data for both instances. The authors concluded good agreement was shown between the computed and observed results. From a practical point of view, they showed that the normal construction sequencing in case 1 resulted in a stable and safe excavation; the axial forces were lower than the alarm values and the displacement due to excavation were within the permissible range. In case 2, however, the computed results showed the excavation was close to becoming unstable, and measures had to be taken to protect the retaining structure from failure.

Popa et al. presented "Numerical modeling and experimental measurements for a retaining wall of a deep excavation in Bucharest, Romania." They summarized a case history of a diaphragm wall for a deep basement of a new building in the center of Bucharest. The excavation impacted a number of historic structures, leading to the use of "top-down" techniques to support the excavation. The numerical results obtained by plane strain FE simulation were compared with measurements recorded during construction. Soil behavior was assumed to be that of a Mohr-Coulomb material. The computed lateral displacements were 15% and 75% of the observed values, depending on if the comparisons were made in an area with or without a grouted wall – not explicitly modeled in the FE simulations – adjacent to the diaphragm wall.

Schweiger et al. presented "3D finite element analysis of a deep excavation and comparison with in situ measurements." The paper describes the results of FE analyses using Plaxis of a deep excavation project in clayey silt in Salzburg. The excavation was supported by a diaphragm wall, a jet grout panel and three levels of struts. The soil responses were represented by the Hardening-Soil model. Because of insufficient information available at the time of design on the material properties of the jet grout panel, the authors varied its stiffness in a parametric study. The effect of taking into account the stiffness of a cracked diaphragm wall on the deformations also was investigated. In some of the 3D calculations, the authors simulated non-perfect contact between the diaphragm wall and a strut by means of a non-linear behaviour of the strut. The evaluation of the results and comparison with in situ measurements showed that analyses which took into account the reduced stiffness of the diaphragm wall due to cracking achieved the best agreement with the measurements. Furthermore settlements of buildings could be best reproduced by the three-dimensional model, although the predicted settlements were not in good agreement with the observations.

Zhang and Huang presented "Monitoring and modeling of riverside large deep excavation-induced ground movements in clays." They discussed a deep excavation located at the Shanghai international passenger center that was 800 m long and 100–150 m wide with the depth of 13 m. The south side of the deep excavation was within 4.6 m of a parallel flood wall of the Huangpu River. The north side of the excavation was 5 m from a historic building. Because of the differences in the conditions on the two sides of the excavations, Plaxis FE analyses were conducted which explicitly included both sides of the excavation, rather than a centerline symmetric condition. Soil responses were assumed to correspond to that of a Mohr-coulomb material. Computed differences of lateral wall movements on each side differed by as much as 50%, as was verified by field observations made during construction.

Hsi et al. presented "Three-dimensional finite element analysis of diaphragm walls for top-down construction." They discussed the Tugun Bypass Tunnel in Gold Coast, Australia. The tunnel was constructed using diaphragm walls with the top-down cut-and-cover method to allow simultaneous construction of an airport runway extension above the tunnel, and excavation of the tunnel beneath. The tunnel was built in deep deposits of saturated, alluvial and estuarine soils with the toes of the walls founded in soil deposits. There was a potential risk for differential settlements between the diaphragm wall panels, caused by the runway fill placed over the tunnel roof during excavation. Three-dimensional numerical modeling was undertaken with Plaxis 3D Foundation to predict the differential settlements of the tunnel arising from the variable subsurface profile, staged excavation and dewatering, non-uniform loading and soil-structure interaction. Soil was assumed to behave as a Hardening-Soil material. Settlements measured after construction were within the range of those computed with the finite element simulations.

Phienwej presented "Ground movements in station excavations of Bangkok first MRT." The characteristics of the lateral movements of the diaphragm walls at excavations for 18 stations of the first Bangkok underground MRT line were evaluated. Three modes of deflected shapes of the walls were observed at different excavation depths, namely a cantilever mode and braced modes with a bulge in soft clay and a bulge in stiff clay. The ratio of maximum lateral wall deflection as a function of excavation depth and the ratio of ground surface settlement to excavation depth and the normalized ground surface settlement varied with the mode of wall deflection. Undrained undrained Young's moduli for a Mohr-Coulomb constitutive response for different soil layers were back-calculated from wall movement data of three selected stations using the 2-D Plaxis FE code. The modulus values, which were higher than those commonly obtained from conventional triaxial tests, can be used as guideline for future excavations in Bangkok.

Ota et al. presented "Consideration of design method for braced excavation based on monitoring results." They compared observed and design values of wall deflections at several cut-and-cover excavations through soft and sensitive clay ground at the Osaka Subway Line No.8. A beam-spring model was employed in the braced design method which accounted for the characteristics of the Osaka soft ground. While there was good agreement between the observed data and design values in past results, the observed wall deflections in this study were larger than that expected for construction sites wherein the excavations encountered 10 to 20 m thick, soft and sensitive clay layer. The authors discuss how they evaluated the horizontal coefficient of subgrade reaction k_h on the excavation side of soft clay layer. The authors make new recommendation regarding selection of k_h, and show that the calculated wall movements with the revised values agree with the observations. These recommendations are applicable to the soft and sensitive Osaka clays.

3.2 *Numerical analysis*

Li and Yang presented the paper "Numerical evaluation of dewatering effect on deep excavation in soft clay." They described a FLAC3D analysis that modeled top-down construction of a 33.7 m deep underground transformer substation in the downtown area of Shanghai. There are both unconfined and confined aquifers on the site of this project and drainage by desiccation in the foundation pit was adopted. Assuming a Mohr-Coulomb soil response, the effective stress methods of analysis incorporated excavation and dewatering of the foundation pit as part of the simulation of construction activities. The computed wall deflections, basal heaves and surface settlements based on analyses that did not consider dewatering were compared to those that did. Results of analyses that considered leakage through the wall and leakage between the aquifers are presented as well. The analysis shows that although the computed differences in lateral wall movement and basal heave were small, due to the low permeability of the soil, dewatering increased the amount of computed surface settlements as a result of drawdown of the water outside the walls of the excavation.

Li et al. contributed the paper "Analysis of the factors influencing foundation pit deformations. They presented results of FE computations based upon 3-D Biot's consolidation theory, assuming the soil responded as a nonlinear Duncan-Chang's material. The finite element equations explicilty considered the coupling of groundwater seepage and soil skeleton deformation during excavation. They presented results that showed the individual effects of the influence of soil permeability, rigidity and levels of lateral supports, rigidity of retaining wall and excavation duration on ground surface settlement, wall horizontal displacement and basal heave of an excavation.

Siemiñska-Lewandowsk and Mitew-Czajewska presented the paper "The effect of deep excavation on surrounding ground and nearby structures." They described problems related with the construction of a 29 m deep excavation of Nowy Swiat Station (S11) of 2nd metro line in Warsaw. A critical section of the project consisted of 7 stations and 6 running tunnels – 6 km length in total. Running tunnels will be constructed using TBM while the stations are to be constructed using cut and cover techniques. Deep excavation will be made with diaphragm walls supported by several levels of slabs and struts. They presented results of 2-D Plaxis FE analyses in terms of ground surface settlements, displacements of surrounding foundations and lateral wall movements, assuming the soil behaves as a Mohr-Coulomb material. Additionally, settlements of the surface were calculated above the TBM (running tunnels). Resulting values of settlements in both cases were discussed, and formed the basis of design predictions that will be verified during construction.

3.3 *Back analysis*

Zghondi et al. presented the paper, "Multi-criteria procedure for the back-analysis of multi-supported retaining walls." They described a numerical back-analysis procedure for multi-supported deep excavations based on the optimization of several indicators, taking in account the forces in the struts and the differential pressures derived from the wall displacement. The evaluation of the procedure is based on results of 1 g small scale laboratory experiments on semi-flexible retaining walls embedded in a Schneebelli material. The proposed numerical procedure was applied to an excavation with 2 levels of struts with low stiffness. The optimized Hardening Soil Model parameters form the basis of calculations of response of 14 different tested configurations. The results are compared with the classical methods, SubGrade Reaction Method, Finite Element analysis with Mohr Coulomb model and with the back-analysis using Hardening Soil Model parameters based on biaxial tests results.

Zhang et al. contributed the paper "Study on deformation laws under the construction of semi-reverse method." Taking a 24.1-m-deep foundation pit of Shanghai Metro Line 1 which uses the semi-reverse construction process of "three open excavating-one tunneling" as an example, they determined deformation laws of a foundation pit under the construction of a semi-reverse method based on analysis of field monitoring data and forward and back analyses methods. They employed Plaxis v8 and assumed the soil acted as a Hardening-Soil material in their computations. The authors stated that results of this approach indicated that the semi-reverse method is an effective way to improve rigidity of the exterior support, control the deformation of excavation, and ensure safety of the surrounding buildings and pipelines.

3.4 *Measurements*

Zhang et al. contributed the paper "GPS height application and gross error detection in foundation pit monitoring." The authors introduced a deformation monitoring model that combined traditional survey technology and GPS measurements. They illustrated foundation pit deformation monitoring based on their experience of deep foundation pit construction of an underground tunnel in Lishui Road, Hangzhou city. When analyzing GPS height conversion to improve the reliability of the GPS datum, they employed Dixon's test in the GPS datum mark to determine potential height anomalies. The authors concluded that this approach is a convenient way to search and delete raw data that includes gross errors.

3.5 *Design*

Chang contributed the paper "Optimization design of composite soil-nailing in loess excavation." Excavations through loess have unique characteristics compared with the others due to its structural properties and collapsibility. To evaluate the mechanisms of support and to develop reasonable methods to design composite soil-nailing in loess excavation, the authors used results of finite element analysis to design a soil nail support system. Their optimization design methods are based on the results of finite element analysis apparently assuming Mohr-Coulomb soil responses. They conducted the simulations to determine the regularity of deformation and the safety factor, as functions of selected design variables. The authors justified their methods by reporting that the lateral deformations of the example excavation were limited to 16 mm.

3.6 *Stress path*

Zhou et al. contributed the paper "Comparison of theory and test on excavation causing the variation of soil mass strength." In view of the characteristic unloading caused by excavations, they deduced the strength ratio of the unloaded soil to soil subjected to compression loadings. Laboratory tests simulating excavation were carried out based on Hvorslev's strength theory. By comparing theoretical results with the laboratory data, they concluded that the soil mass is overconsolidated. As a consequence, the authors stated that the soil microstructure is damaged, and the soil mass strength is reduced in the unloading process. The authors concluded that analyses of the results are helpful to the understanding of the effect of excavation unloading on the variation of the soil mass strength.

3.7 *Earth pressure*

Lin and Lee contributed the paper "A simplified spatial methodology of earth pressure against retaining piles of pile-row retaining structure." When using a pile-row retaining structure to support excavation, the authors stressed the importance of obtaining the magnitude and distribution of the earth pressure against the retaining piles. Based on the mode of failure, a new methodology is proposed to evaluate the earth pressure against the retaining piles of such a structure. In the proposed method, both the spatial effect and intermediate principal stress effect are considered. The authors provide an example of the methodology applied to engineering practice. They demonstrated that the strength theory has more influence on earth pressure.

4 COMMENTS REGARDING COMPUTED AND OBSERVED RESULTS

Many factors affect ground movements caused by excavations, including stratigraphy, soil properties, support system details, construction activities, contractual arrangements and workmanship. In this theme, most papers described numerical simulations that analyzed ground response arising from excavation. Finite element predictions always contain uncertainties related to soil properties, support system details and construction procedures. Furthermore, while supported excavations commonly are simulated numerically by modeling cycles of excavation and support installation, it generally is necessary to simulate all aspects of the construction process that affect the stress conditions around the cut to obtain an accurate prediction of behavior. This may involve simulating previous construction activities at the site, installation of the supporting wall and any deep foundation elements, as well as the removal of cross-lot supports or detensioning of tiedback ground anchors. Issues of time effects caused by hydrodynamic effects or material responses may be important. The following sections summarize some of the factors that may impact computed responses of ground movements associated with excavations. Proper consideration must be given to such factors when making such analyses, as well as when critically evaluating published results of the same.

4.1 *Drainage conditions*

An important preliminary decision in any analysis is to match the expected field drainage conditions, which impacts the formulation required. Clough and Mana (1976) and field data have shown that for excavations through saturated clays with typical excavation periods of several months, the clays remain essentially undrained with little dissipation of excess pore pressures.

For undrained conditions, one can employ either a coupled finite element formulation where both displacements and pore water pressures are solved for explicitly (e.g. Carter et al. 1979) or a penalty formulation (e.g. Hughes 1980) wherein the bulk modulus of water – or a sufficiently large number that depends on the precision of the machine making the computation - is added to the diagonal terms in the element stiffness matrix during global matrix assembly. This additional term constrains the volumetric strain to nearly zero, i.e., undrained. In both these approaches, the constitutive response of the soil is defined in terms of effective stress parameters. A simpler, alternate approach is to define undrained constitutive response in terms of total stress parameters, with care being taken to make the diagonal terms of the element stiffness matrix large, typically by using a Poisson's ratio close to 0.5. In this case, a Young's modulus corresponds to an undrained value and failure is expressed in terms of an undrained shear strength, S_u (e.g., $\varphi = 0$ and $c = S_u$).

However, there may be cases (e.g., O'Rourke and O'Donnell 1997) where substantial delays during construction occur and excess pore pressures partially dissipate, and in these cases one must use a mixed formulation to account for the pore water effects. When using top-down techniques to excavate, it can take up to several years to reach final grade for large excavations, and hence partially drained conditions would apply therein, requiring a coupled finite element simulation.

4.2 *Initial conditions*

A reasonable prediction of the ground response to construction of a deep excavation starts with a good estimate of the initial stress conditions, in terms of both effective stresses and pore water pressures. The effective stress conditions for excavations in well-developed urban areas rarely correspond to at-rest conditions because of the myriad past uses of the land. Existence of deep foundations and/or basements from abandoned buildings and nearby tunnels changes the effective stresses from at-rest conditions prior to the start of excavation. For example, Calvello and Finno (2003) showed that an accurate computation of movements associated with an excavation could only be achieved when all the pre-excavation activities affecting the site were modeled explicitly. They used the case of the excavation for the Chicago-State subway renovation project (Finno et al. 2002), wherein construction of both a tunnel and a school impacted the ground stresses prior to the subway renovation project.

Ignoring these effects made a difference of a factor of 3 in the computed lateral movements.

One also must take care when defining the initial ground water conditions. Even in cases where the ground water level is not affected by near surface construction activities, non-hydrostatic conditions can exist for a variety of reasons. For example, Finno et al. (1989) presented pneumatic piezometer data that indicated the presence of a downward gradient within a 20 m thick sequence of saturated clays. This downward flow arose from a gradual lowering since the 1950s of the water level in the upper rock aquifer in the Chicago area. A non-hydrostatic water condition affects the magnitude of the effective stresses at the start of an excavation project.

An engineer has two choices to define such conditions – to measure the *in situ* conditions directly or to simulate all the past construction activities at a site starting from appropriate at-rest conditions. Because both approaches present challenges, it is advantageous to do both to provide some redundancy in the input. In any case, careful evaluation of the initial conditions is required when numerically simulating supported excavation projects, especially in urban areas.

4.3 *Wall installation*

Many times the effects of installing a wall are ignored in a finite element simulation and the wall is "wished-into-place" with no change in the stress conditions in the ground or any attendant ground movements. However, there are abundant data that show ground movements may develop as a wall is installed.

O'Rourke and Clough (1990) presented data that summarized observed settlements that arose during installation of five diaphragm walls. They noted settlements as large as 0.12% of the depth of the trench. These effects can be evaluated by 3-dimensional modeling of the construction process (e.g., Gourvenec and Powrie 1999), but not without several caveats. The specific gravity of the supporting fluid usually varies during excavation of a panel as a result of excavated solids becoming suspended – increasing the specific gravity above the value of the water and bentonite mixture and subsequently decreasing when the slurry is cleaned prior to the concrete being tremied into place. Consequently, it is difficult to select one value that represents an average condition. Furthermore the effects of the fluid concrete on the stresses in the surrounding soil depend upon how quickly the concrete hardens relative to its placement rate. Some guidance in selecting the fresh concrete pressure is provided by Lings et al. (1994).

It is less straightforward when modeling diaphragm wall installation effects in a plane strain analysis because the arching caused by the excavation of individual panels cannot be taken directly into account. To approximate the effects of this arching when making such an analysis, an equivalent fluid pressure, generally higher than the level of the fluid during construction, can be applied to the walls of the trench to maintain stability. Thus, some degree of empiricism is required to consider these installation effects in a plane strain analysis. One can back-calculate an equivalent fluid pressure corresponding to the observed ground response if good records of lateral movements close to the wall are recorded during construction. More data of this type are needed before any recommendations can be made regarding magnitudes of appropriate equivalent pressures.

The effects of installing a sheet pile wall are different than those of a diaphragm wall, yet the effects on observed responses also can be significant. In this case, ground movements may arise from transient vibrations developed as the sheeting is driven or vibrated into place and from the physical displacement of the ground by the sheeting. The former mechanism is of practical importance when installing the sheeting through loose to medium dense sands, and can be estimated by procedures proposed by Clough et al. (1989). However, these effects are not included in finite element simulations. The latter mechanism in clays was illustrated by Finno et al. (1988). In this case, the soil was displaced away from the sheeting as it was installed. This movement was accompanied by an increase in pore water pressure and a ground surface heave. As the excess pore water pressures dissipated, the ground settled. The maximum lateral movement and surface heave was equal to one-half the equivalent width of the sheet pile wall, defined as the cross-sectional area of the sheet pile section per unit length of wall. Sheet-pile installation can be simulated in plane strain by using procedures summarized in Finno and Tu (2006).

In addition to the movements that occur as a wall is installed, installing the walls can have a large influence on subsequent movements, especially if the walls are installed relatively close to each other, as may be the case in a cut-and-cover excavation for a tunnel. Sabatini (1991) conducted a parametric study as a function of the depth, H, to width, B, of an excavation, wherein the effects of sheet-pile wall installation in clays were compared with simulations where the walls were wished into place. The results of the study are shown in Figure 1 where the computed normalized maximum lateral movements, $\delta H_{(max)}/H$, are plotted versus H/B.

It is apparent for wide excavations ($H/B \leq 0.25$) that the decision to include installation effects in a simulation is not critical. However, these effects become pronounced for narrow excavations (H/B » 1) and should be explicitly considered. The results also show that for the "wished-in-place" case when the sheet-pile installation effects are ignored, the lateral

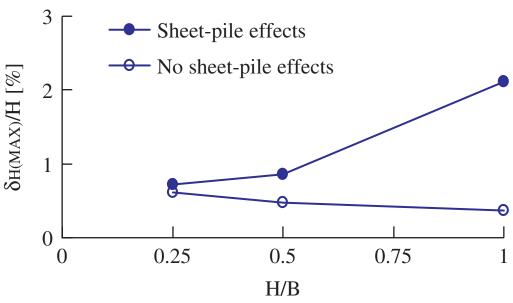

Figure 1. Effects of sheet-pile installation on computed lateral movements.

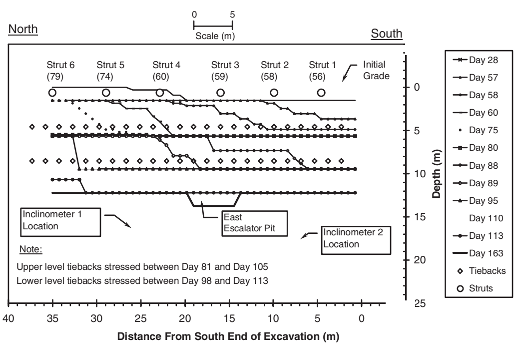

Figure 2. Construction progress at excavation in Chicago (Finno et al. 2002).

movements are larger for wider excavations, a similar trend reported by Mana and Clough (1981).

Sheet-pile installation has two main effects: the soil adjacent to the excavation is preloaded and the shear strength on the passive side is (partially) mobilized prior to the beginning of the cycles of excavation. Wall installation tends to preload the soil on the active side of the excavation as a result of the reduction in shear stress at approximately constant mean normal effective stress. This mechanism provides the soil outside the walls with more available shearing resistance when the cycles of excavation start. However, the soil between the walls has less available passive resistance as a result of the preloading and this promotes the larger movements during excavation as compared to the case of ignoring the sheet-pile effects (Finno and Nerby 1989).

4.4 *Plane strain versus 3-dimensional analyses*

Figure 2 illustrates some of the challenges of using field observations to calibrate numerical models of any kind, even when detailed records exist. This figure summarizes the construction progress at the Chicago-State excavation in terms of excavation surface and support installation on one of the walls of the excavation for selected days after construction started. Also shown are the locations of two inclinometers placed several meters behind the wall. If one is making a computation assuming plane strain conditions, then it is clear that one must judiciously choose data sets so that planar conditions would be applicable to those selected.

Even when a sufficiently extensive horizontal excavated surface is identified, 3-dimensional effects may still arise from the higher stiffness at the corners of an excavation. These boundary conditions lead to smaller ground movements near the corners and larger ground movements towards the middle of the excavation wall. Another, and less recognized, consequence of the corner stiffening effects is the maximum movement near the center of an excavation wall may not correspond to that found from a conventional plane strain simulation of the excavation, i.e., 3-D and plane strain simulations of the excavation do not yield the same movement at the center portion of the excavation, even if the movements in the center are perpendicular to the wall (Ou et al. 1996). This affect can be quantified by the plane strain ratio, PSR, defined herein as the maximum movement in the center of an excavation wall computed by 3-D analyses divided by that computed by a plane strain simulation. Finno et al. (2007) developed the following expression for PSR from the results of a finite element parametric study of excavations through clay:

$$PSR = \left(1 - e^{-k\,C(L/H_e)}\right) + 0.05\left(L/B - 1\right) \tag{1}$$

where L is the excavation length along the side where the movement occurs, B is the other areal dimension, and H_e is the excavation depth. The value of C depends on the factor of safety against basal heave, FS_{BH}, and is taken as:

$$C = 1 - \{0.5\,(1.8 - FS_{BH})\} \tag{2}$$

The value of k depends on the support system stiffness and is taken as:

$$k = 1 - 0.0001\left(\frac{EI}{\gamma h^4}\right) \tag{3}$$

where EI is the bending stiffness of the wall, γ is the total unit weight of the soil and h is the average vertical spacing between supports. When L/H_e is greater than 6, the PSR is equal to 1 and results of plane strain simulations yield the same displacements in the center of an excavation as those computed by a 3-D simulation. When L/H_e is less than 6, the displacement computed from the results of a plane strain analysis will be larger than that from a 3-D analysis. When conducting an inverse analysis of an excavation with a plane strain simulation when L/H_e is relatively small, the effects of this corner stiffening is that an optimized stiffness parameter will be larger than it really is because of the lack of the corner stiffening in the plane strain analysis. This effect becomes greater as an excavation is deepened because the L/H_e value increases as the excavated grade is lowered. This trend was observed in the optimized parameters for the deeper strata at the Chicago-State subway renovation excavation (Finno and Calvello 2005).

5 CONSTITUTIVE MODEL CONSIDERATIONS

When one undertakes a numerical simulation of a deep supported excavation, one of the key decisions made early in the process is the selection of the constitutive model. In general, this selection should be compatible with the objectives of the analysis. If the results form the basis of a prediction that will be updated based on field performance data, then the types of field data that form the basis of the comparison will impact the applicability of a particular model. Possibilities include lateral movements based on inclinometers, vertical movements at various depths and distances from an excavation wall, forces in structural support elements, pore water pressures or any combinations of these data. When used for a case where control of ground movements is a key design consideration, the constitutive model must be able to reproduce the soil response at appropriate strain levels to the imposed loadings.

5.1 *Incremental non-linearity*

It is useful to recognize that soil is an incrementally nonlinear material, i.e., its stiffness depends on loading direction and strain level. Real soils are neither linear elastic nor elastic-plastic, but exhibit complex behavior characterized by zones of high stiffness at very small strains, followed by decreasing stiffness with increasing strain. This behavior under static loading was identified through back-analysis of foundation and excavation movements in the United Kingdom (Burland, 1989). The recognition of zones of high initial stiffness under typical field conditions was followed by efforts to measure this ubiquitous behavior in the laboratory for various types of soil (e.g., Jardine et al. 1984; Clayton and Heymann 2001; Santagata et al. 2005; Calisto and Calebresi 1998, Cho 2007).

To illustrate this behavior, Figure 3 shows the results of drained, triaxial stress probes conducted on specimens cut from block samples of lightly overconsolidated glacial clays obtained at an excavation in Evanston, IL. Each specimen was reconsolidated under K_0 conditions to the in-situ vertical effective stress σ'_{v0}, subjected to a 36 hour K_0 creep cycle, followed by directional stress probing under drained

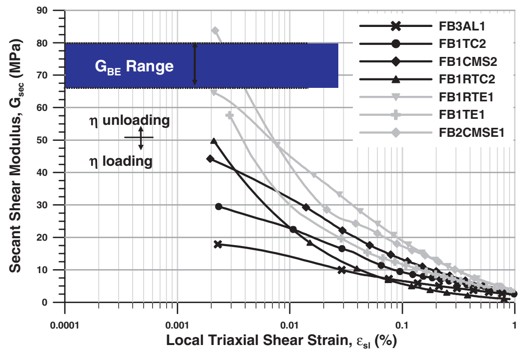

Figure 3. Secant shear modulus as a function of direction of loading.

axisymmetric conditions. Bender element (BE) tests were conducted during all phases of the tests. The secant shear moduli are plotted versus triaxial shear strain in Figure 3 for natural specimens whose stress probes involved changes in the shear stress q. The overconsolidation ratio of these specimens was 1.7, so if one assumes the response is isotropic and elastoplastic, then G should be the same for at least the initial portion of all curves. The stress probes wherein q and the stress ratio, $\eta = q/p'$, is increased ("η loading") are clearly softer than those where q and η initially decrease ("η unloading"). There are no obvious zones of constant G_{sec} at shear strains greater than 0.002%, and thus no elastic zone is observed in these data for strain levels. Complete details and results of the testing program are presented by Cho (2007).

Burland (1989) suggested that working strain levels in soil around well-designed tunnels and foundations are on the order of 0.1 %. If one uses data collected with conventional triaxial equipments to discern the soil responses, one can reliably measure strains 0.1% or higher. Thus in many practical situations, it is not possible to accurately incorporate site-specific small strain non-linearity into a constitutive model based on conventionally-derived laboratory data.

5.2 *Model selection*

There are a number of models reported in literature wherein the variation of small strain nonlinearity can be represented, e.g., a three-surface kinematic model develop for stiff London clay (Stallebrass and Taylor 1997), MIT-E3 (Whittle and Kavvadas 1994), hypoplasticity models (e.g. Viggiani and Tamagnini 1999), and a directional stiffness model (Tu 2007). To derive the necessary parameters, these models require either detailed experimental results or experience with the model in a given geology. While the models can be implemented in material libraries in some commercial finite element codes, these routines are not readily

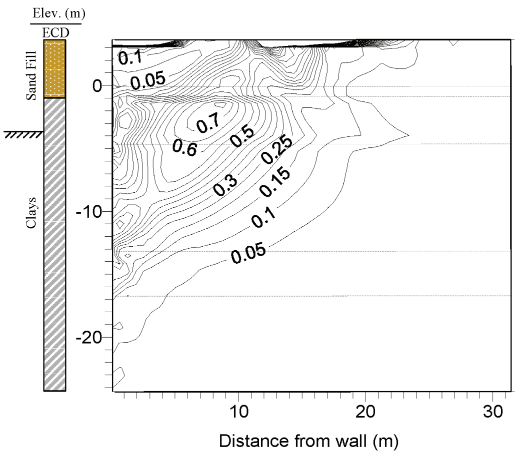

Figure 4. Shear strains behind excavation: 57 mm maximum lateral movement (contours in %).

available to most practitioners. Thus for most current practical applications, one uses simpler, elasto-plastic models contained in material libraries in commercial codes.

For these models, a key decision is to select the "elastic" parameters that are representative of the secant values that correspond to the predominant strain levels in the soil mass. Examples of the strain levels behind a wall for an excavation with a maximum lateral wall movement of 57 mm are shown in Figure 5. These strain levels were computed based on the results of displacement-controlled simulations where the lateral wall movements and surface settlements were incrementally applied to the boundaries of a finite element mesh. The patterns of movements were typical of excavations through clays, and were based on those observed at an excavation made through Chicago clays (Finno and Blackburn 2005). Because the simulations were displacement-controlled, the computed strains do not depend on the assumed constitutive behavior.

As can be seen in Figure 4, maximum shear strains as high as 0.7% occur when 57 mm of maximum wall movement develop. The maximum strains are proportional maximum lateral wall movement; for example, when 26 mm maximum lateral wall movement develops, the maximum shear strain is about 0.35%. These latter strain levels can be accurately measured in conventional triaxial testing, and thus if one can obtain specimens of sufficiently high quality, then secant moduli corresponding to these strain levels can be determined via conventional laboratory testing. Because the maximum horizontal wall displacement can be thought of as a summation of the horizontal strains behind a wall, the maximum wall movements can be accurately calculated with a selection of elastic parameters that corresponds to these expected

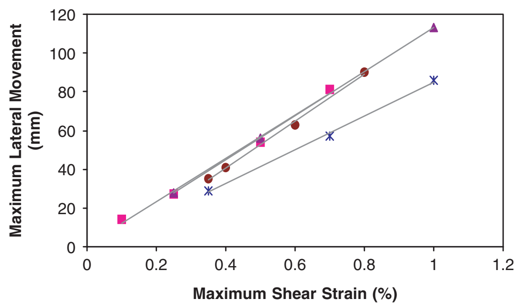

Figure 5. Relation between maximum wall movement and shear strain.

strain levels. In this case, the fact that small strain non-linearity is not explicitly considered will not have a large impact on the computed horizontal wall displacements because the maximum horizontal movement at the wall is dominated by the larger strains in the soil mass. These computed movements would be compatible with those measured by an inclinometer located close to the wall.

However, if one needs to obtain an accurate representation of the distribution of ground movements with distance from the wall, then this approach of selecting strain-level appropriate elastic parameters is not applicable. Small strain non-linearity of soil must be explicitly considered to find the extent of the settlement because the strains in the area of interest vary from the maximum value to zero. As a consequence, many cases reported in literature indicate computed wall movements agree reasonably well with observed values, but the results from the same computations do not accurately reflect the distribution of settlements. Indeed, this was the case in several papers presented as part of this theme. Good agreement at distances away from a wall can be obtained only if the small stain non-linearity of the soil is adequately represented in the constitutive model.

The relation between lateral wall displacements and shear strain levels in the soil behind the wall can be evaluated from results of displacement-controlled finite element simulations. Similar to the results shown in Figure 4, different displacement profiles were studied by imposing lateral wall displacements and settlement profiles, representing conditions with maximum lateral movements at the excavated surface, cantilever movements, deep-seated movements and combination of the latter two (Andrianis 2006). The stratigraphies used in the models were based on typical Chicago soils. The results in Figure 5 show that the relationship between maximum shear strain behind the wall and maximum displacement of the wall is almost linear for lateral wall displacements between 10 and 110 mm. Figure 5 also shows that the results form a narrow band, suggesting that the relation between strain and wall displacement is not greatly affected by the type of movement.

Figure 5 can be used to estimate shear strains for a specified maximum wall movement. With this value of shear strain, the secant shear moduli for use in conventional elasto-plastic models can be estimated based on strain-stress data from high quality laboratory experiments. The values of maximum shear strains, even in the cases with the relatively low values of displacements, are about 0.2% and increase as the specified displacement becomes larger. This is important when one determines soil stiffness in the laboratory. Conventional soil testing without internal instrumentation allows one to accurately measure strains as low as 0.1%. Thus for many cases, the secant shear moduli can be determined from conventional laboratory tests on high quality samples. However, if strain levels are 0.1% or less, then one must select these moduli from test results based on internally-measured strains in equipment not normally available in commercial laboratories.

In summary, using a simulation based on conventional elasto-plastic models limits the type and location of the data that can be used as observations in an inverse analysis. Both vertical and lateral movements measured at some distance from a wall cannot be calculated accurately in this case because the variation of stiffness with strain levels must be adequately represented in the soil model. Only the lateral movements close to a support wall can be reasonably computed with conventional models since that result is dominated by the zones of high strains behind the wall.

6 CONCLUDING REMARKS

The papers presented at the symposium included widely variable levels of information regarding the details of the finite element analyses. As such, the author tentatively proposes that the following information be included in any paper describing the results of any finite element simulation of geotechnically-related construction.

1 The finite element code used.
2 The assumed drainage conditions, e.g., drained, undrained or partially drained.
3 The dimensionality of the problem, e.g., plain strain, axisymmetric or three-dimensional.
4 The constitutive model(s) employed for both soils and structural elements.
5 The parameters for each material and a discussion of the basis of their selection.
6 A description of the mesh, including boundary conditions and type of elements used for soil, structural components and interfaces.
7 Construction records, simulation steps and details of how each construction activity was idealized in the finite element simulation.

Finally, comparisons between computed and observed results, as well as a discussion of the comparisons, should be included.

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