Three-Dimensional Finite Element Analysis of Deep Excavations[a]

Discussion by Hoe I. Ling[4] and Andrew J. Whittle,[5] Members, ASCE

The hyperbolic model originates from best-fitting a function relating stress (σ) and strain (ε) in an isotropically consolidated shearing test: $\sigma_1 - \sigma_3 = \varepsilon/(a + b\varepsilon)$, where a and b are constants. The values of a and b vary with the confining pressure. Kondner (1963) correlated these two constants with the "apparent" soil properties, such as E_i. The parameters, obtained by best-fitting the laboratory test results, are not necessarily the "true" strength and stiffness of soil. However, if the stress-strain curves are not available, an estimation of relationships, based on soil type or c and ϕ, may be made (e.g., Duncan et al. 1980, Tables 5 and 6). It appears that sufficient experiments were conducted for this construction site (and described in the paper in the section "Finite Element Analysis"), and the experimental results could have been used to calibrate the model and then extended to finite element analysis.

The discussers do not quite agree that back-calculated soil parameters should be used for best-fitting field performance without questioning validity of soil and finite element modeling. The concept of finite element analysis should be based on accurate modeling of the physical problem, including a realistic soil model, so that the framework of analysis can be established and extended to other general cases. If the analysis is not based on accurate modeling, the validity of methodology will be limited to a case-by-case basis. For excavation problem, it has been shown that accurate finite element modeling can be achieved using soil properties obtained directly from laboratory testings and using a realistic soil model (e.g., Whittle et al. 1993) without resorting to use of back-analyzed soil parameters.

Contrary to statements in the section "Modeling of Structure and Soil," the soil model that the authors have employed does not simulate the elasto-plastic behavior of soils. The hyperbolic model is based on the theory of elasticity. This, however, appears to have been correctly addressed in a separate publication (Ou and Lai 1994). The success of hyperbolic model in analyzing loading problems, such as embankments and dams, is well recognized. In an excavation analysis, soil elements around the vicinity of excavated surface may be subjected to predominantly unloading stress paths. Under such circumstances, the analysis is based on (2), in which the tangent Young's modulus is linear elastic, but its magnitude depends on confining pressure. The discussers believe that in an excavation analysis such a model may not work realistically because the unloading path of the soil is nonlinear and exhibiting hysteresis. Linear elastic unloading behavior is not limited to Duncan-Chang model, however; it is described by other conventional elasto-plastic models, such as the modified Cam-Clay model, if the stress states considered are indeed within the yield surface. An excavation analysis can be improved significantly by considering the nonlinear unloading relationships, using the bounding surface model (e.g., Whittle 1993; Kaliakin 1990).

The authors stated that initial condition has a major effect on the results of analysis. However, the values of $K_0 = \sigma_3/\sigma_1$ used for each soil layer in the analysis were not stated. If the authors have used the well-known relationship $K_0 = \nu/(1 - \nu)$ or $K_0 = 1 - \sin \phi$ (where ν is Poisson's ratio), the value of K_0 should be about 1.0 for most clay layers under undrained conditions (since ν approaches 0.5). Some of the common finite element codes employing a hyperbolic soil model (such as EXCAV, SSCOMP, FEADAM) allow users to specify varying values of K_0 to represent the initial stress conditions. The discussers conducted a two-dimensional analysis to investigate this effect. CRISP (Britto and Gunn 1987) was modified for this discussion by incorporating a modified Duncan-Chang model (Seed and Duncan 1984). This model detects the primary loading and unloading/reloading elastic moduli more efficiently than its earlier versions. The values given in Table 2, including a constant Poisson's ratio, were used instead of a pressure-dependent bulk modulus. The stiffness value of the strut (bar elements) and diaphragm wall was based on that reported by the authors. The finite element mesh identical to that presented in Fig. 15 was used. Note that eight-node elements, with a 3×3 integration, were used to represent the soil layers and diaphragm wall. A total of seven excavation steps (see Fig. 21) were used, and the load was distributed over 10 increments within each step.

Fig. 21 shows that the resulting wall deflections indeed differ by several times when $K_0 = \sigma_3/\sigma_1$ varies from 0.3 to 1.0. A larger value of K_0 results in a larger initial confining pressure and therefore a stiffer soil response and smaller wall deflections. The calculated deflections are, however, larger than those presented by the authors (see Figs. 16, 18, and 19). Since two different finite element programs of varying features, including possible different versions of hyperbolic model, were involved, it is not possible for the discussers to draw any conclusions about the value of K_0 the authors have adopted. However, estimation of K_0 at any excavation site is a complicated task. More importantly, finite element modeling by applying an anisotropic stress ratio (K_0) to an isotropic hyperbolic model produces results contrary to the field performance. In reality,

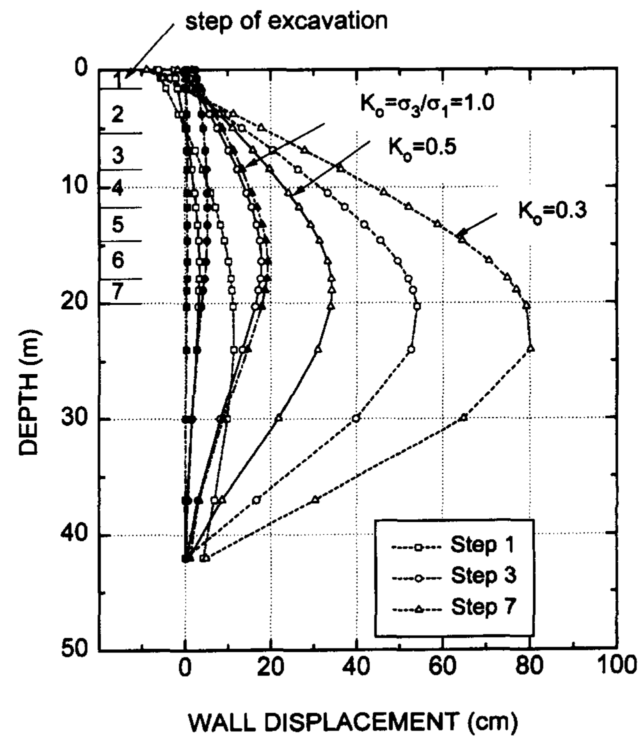

FIG. 21. Effects of Initial Stress Ratio on Wall Deflection in Isotropic Hyperbolic Model

[a]May 1996, Vol. 122, No. 5, by Chang-Yu Ou, Dar-Chang Chiou, and Tzong-Shiann Wu (Paper 7498).

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if anisotropically consolidated tests were conducted, a stiffer soil response would be anticipated when the value of K_0 decreases (e.g., Ling and Tatsuoka 1996).

The validity of the analysis may be established from other aspects of performance, such as base heaving and surface settlements. It would be very helpful if the authors made those comparisons.

APPENDIX. REFERENCES

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Closure by Chang-Yu Ou,[6] Associate Member, ASCE, Dar-Chang Chiou,[7] and Tzong-Shiann Wu[8]

Thanks to Professor Ling and Professor Whittle for their interesting discussion. First, it must be emphasized that in the paper undrained analysis is used for cohesive soil. In the paper K_0 represents the ratio of effective horizontal stress to effective vertical stress and is determined from Jaky's equation $K_0 = 1 - \sin\phi'$, in which σ' is the drained friction angle. For most normally consolidated clays, K_0 is approximately equal to 0.5. Therefore, the initial horizontal total stress $\sigma_h = K_0\sigma'_v + u_w$, in which u_w is the initial porewater pressure.

Many engineers dream of obtaining analytic results close to field observations using soil parameters obtained directly from laboratory tests. Such analysis may be based on either a realistic soil model or simplified model. However, this analysis is still in the research stage and many problems remain to be resolved. A realistic soil model usually involves complicated finite element formulation and a difficult soil parameter determination procedure. It is not easy to obtain a good understanding of the relationship between input soil parameters and soil stiffness during the computation routine of a computer program because it is not straightforward. In the literature, excavation problems are often analyzed using the finite element method based on a realistic soil model. However, not all the analysis results are consistent with field observations. As shown in Fig. 22, a large amount of movement at the bottom of the diaphragm wall is commonly predicted in model-based analysis. However, field observations have shown that the bottom of the supported wall normally has little movement, even in soft clay, if the wall is stable. Field observations (Clough and O'Rourke 1990) indicate that ground surface settlement normally attenuates to zero at a distance from the wall equal to twice the excavation depth (2*H*). Some researchers have indeed predicted small bottom movement using a realistic soil model (e.g., Whittle et al. 1993) because the diaphragm wall penetrates the hard soil stratum, to which high soil stiffness is directly assigned. However, many analyses, especially those using the complicated soil model, do not yield results consistent with field observations.

Analysis based on a hyperbolic model performs better than analysis based on a more complicated model. Although the hyperbolic model has no solid theoretical basis, the relationship between input soil parameters and soil stiffness is straightforward. As the soil element approaches to the failure state, the soil stiffness becomes very small. The success of the hyperbolic model is not only in the problems of embankments and dams but also in excavation, as validated by Clough and his group (e.g., Osaimi and Clough 1979) and many other researchers (e.g., Wong and Broms 1989), although the model may not exactly simulate the actual stress-strain behavior of soil, especially for the soil around the excavation surface.

The original hyperbolic model was based on the monotonic loading condition, in which the confining pressure is assumed to be unchanged. The original model for differentiating between primary loading and unloading/reloading behavior does

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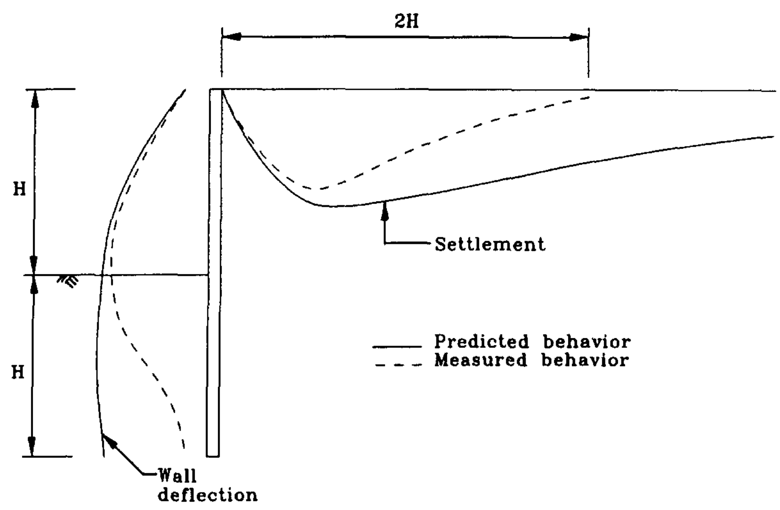

FIG. 22. Behavior Predicted in the Literature versus Measured Behavior

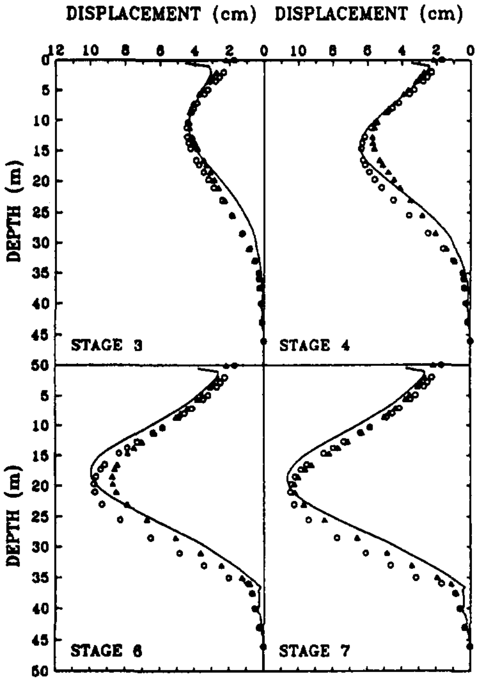

FIG. 23. Observed Wall Deflections versus Wall Deflections Computed Using Original and Hyperbolic Models

not appear to be a rational model for actual stress-strain behavior of soils in excavation problems. Seed and Duncan (1984) used a simple function to differentiate the primary and unloading/reloading states. However, the function is somewhat empirical.

The authors of the present study further modify the original hyperbolic model by incorporating the concept of plasticity theory. The yield function is used to detect the primary loading state and unloading/reloading elastic state. The modified form of the model can actually describe the stress-strain behavior of the soil (Hsieh and Ou 1997). Fig. 23 compares the wall deflections computed based on modified and original versions of the model with field measurements for some key construction stages. Exactly the same soil parameters are used in both types of model analysis. As this figure indicates, the wall deflections from both types of model analysis are consistent with the field measurements.

Back-analysis or an observational approach has been used widely in geotechnical analysis. A realistic soil model is certainly the first choice in back-analysis. Based on the authors' experience, other simplified models also work very well. After studying Duncan-Chang model and elasto-plastic model (by von Mises) in the excavation analysis, Clough and Mana (1976) concluded that if appropriate soil parameters are chosen, the type of soil model does not significantly affect predicted behavior.

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LOAD-DISPLACEMENT PREDICTION FOR HORIZONTALLY LOADED VERTICAL PLATES[a]

Discussion by Abdul-Hamid Soubra,[2] Pierre Regenass,[3] and Sergio Carboni[4]

The problem of the passive resistance of plate anchors interests the discussers. In his paper, the author collected laboratory and field results reported in the literature and derived generalized expressions for predicting an easy estimation of the ultimate horizontal pullout resistance of vertical anchor plates and the displacement associated with the level of the applied load. The dimensionless factors adopted by the author are the pullout capacity factor $(P_u/\gamma' AH)\tan\phi$ and the geometry factor H^2/A. The author stated that these factors incorporate all the parameters that significantly affect the behavior of horizontally loaded vertical anchor plates. The purpose of this discussion is to comment on the dimensionless parameters adopted by the author.

Regenass and Soubra (1995, 1997) have investigated the problem of the limit load of strip anchors and have established design charts of the ultimate failure load using the upper-bound method of limit analysis theory. In this case, one force coefficient $(M_\gamma = 2P_u/\gamma' Hh)$ and one geometry factor (H/h) are necessary to accurately estimate the anchor limit force P_u. On the other hand, the discussers (1997) have investigated the problem of the limit load of rectangular anchors and have established a kinematically admissible three-dimensional failure mechanism composed of four rigid blocks (Fig. 4). The results obtained from this failure mechanism for $\phi = 40°$ and $\delta/\phi = 2/3$ are presented, in Fig. 5, in the form of one pullout capacity factor $(M_\gamma = 2P_u/\gamma' Hhb)$ and two geometry factors $(H/h$ and $b/h)$. Contrary to the author's definition of geometrical dimensionless parameter H^2/A, the discussers suggest that the analysis of the horizontally loaded vertical plates should include two factors, one representing the width and the other the embedment depth of the anchor plate (i.e., b/h and H/h). Table 2 presents the discussers' results for different H, h, and b values obtained using the theoretical model of the upper-bound method. According to the author's approach, for a given value of the parameter H^2/A—say, $H^2/A = 16$—one must obtain a unique value of the parameter $(P_u/\gamma' AH)\tan\phi$. Table 2 shows

[a]January 1997, Vol. 123, No. 1, by Ashraf M. Ghaly (Technical note 10413).

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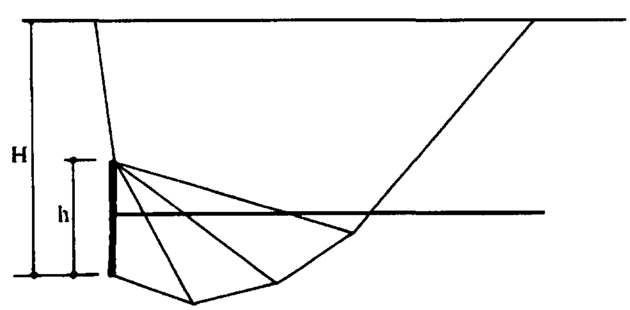

FIG. 4. Kinematically Admissible Three-Dimensional Failure Mechanism Composed of Four Rigid Blocks