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Improved Geotechnical Analysis through Better Integration and Dynamic Interaction between Site Characterization and Analytical Theory

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ABSTRACT

Since the earliest days of geotechnical engineering, soil properties developed during the site characterization phase of a project have typically been viewed as fixed quantities independent of the deterministic analyses in which they are eventually used. This paper explores the premise that given all the advances that have occurred in geotechnical engineering what is needed in both routine practice and research alike is a fundamental change in philosophy to one that links or couples site characterization outcomes and analytical theories in a seamless, integrated, dynamic fashion to converge at a unique final result that contains the desired answer to the analysis. The application of this concept to the common foundation engineering problem of bearing capacity of a spread footing is presented in detail to illustrate the process. Noteworthy is the fact that no unusual or sophisticated exploration, testing, or analytical tools are required which means this new conceptual approach is immediately applicable to the simplest of projects and on a routine basis.

CURRENT STATE OF PRACTICE

Historically, site characterization outcomes on a project have been viewed in a fixed or static fashion with minimal or no linkage to the deterministic analyses in which they are eventually used on that project. For example, a Standard Penetration Test (SPT) *N*-value is usually assumed to correlate in a fixed, unchanging manner to some value of relative density, D_r, or the Mohr-Coulomb angle of internal friction, ϕ, regardless of the final analytical usage of these parameters.

An outcome of this traditional non-linked, uncoupled approach is that when calculated values from some analysis are compared to measured values in the field there can be significant disparity. The conclusion often drawn is that the deficiency lies with the analytical methodology and that a new and better methodology is required.

The argument put forth in this paper is that this is not necessarily true in all cases. The reason is that this deduction fails to consider whether the soil properties from the site characterization process were relevant and appropriate to the particular analysis performed. Thus before an analytical theory is judged to be inaccurate or incorrect due diligence requires that the other components of the overall analytical process, the site characterization methodology and link between site characterization and analysis, be vetted first.

PROPOSED CHANGE IN PRACTICE

Introduction

The argument presented in this paper is that the traditional unlinked, uncoupled approach between site characterization and geotechnical analysis is an artifact-of-necessity of an earlier era that not only can but needs to be discarded. The numerous shortcomings and constraints in various aspects of geotechnology that once justified this approach have not existed for decades. This has been noted recently by Salgado and Fox (2010):

"The enormous progress that has taken place in theoretical soil mechanics (particularly in the last 30 years) has yet to be substantially integrated into practice, and thus the role that theory plays in the design process is not significantly different from what it was decades ago."

As a result, this traditional approach can and should be abandoned in favor of a more modern integrated approach where site characterization and geotechnical analysis are always linked or coupled in a dynamic, interactive fashion to produce a unique analytical outcome that is inherently superior to the traditional approach. Only after such a modern, integrated approach is used can an accurate assessment be made of the adequacy of existing analytical methodologies and the need for new ones.

This new integrated approach does not require any new, experimental, extraordinary, or unusual effort for either the site characterization or geotechnical analysis component as it makes synergistic use of the myriad advances in several aspects of geotechnical engineering practice that have occurred in recent decades: site characterization and concomitant correlations to soil properties; fundamental understanding of soil behavior; and computational speed of digital computers. Thus the methodology presented in this paper should be attractive to practitioners on even the smallest of projects in addition to academicians and other researchers.

Concept

The fundamental concept underlying the methodology presented in this paper is that *stress state* is the single most important factor that must always be considered for both the site characterization and geotechnical analysis aspects of a problem. In this context, stress state includes:

- vertical and horizontal effective *overburden stresses*, σ'_{vo} and σ'_{ho}, at the point in the ground at which a piece of in-situ test data was obtained;
- *stress history* as relates to the yield stress (a.k.a. maximum past effective stress, etc.), σ'_{vm}, and overconsolidation ratio, *OCR*, at the point in the ground at which a piece of in-situ test data was obtained; and
- both absolute and relative (to failure) operative *stress levels* for the conditions under which the soil property is being used in some geotechnical analysis.

This last point is particularly important to the methodology presented in this paper and merits elaboration. It has long been recognized that key engineering properties of soil such as Young's modulus, E, and ϕ vary non-linearly as a function of stress state. This is illustrated qualitatively in Fig. 1 for ϕ. However, for simplicity, especially in routine practice, it was long assumed that ϕ was constant for a given soil. This simplicity must be abandoned once and for all if the state of practice is to advance.

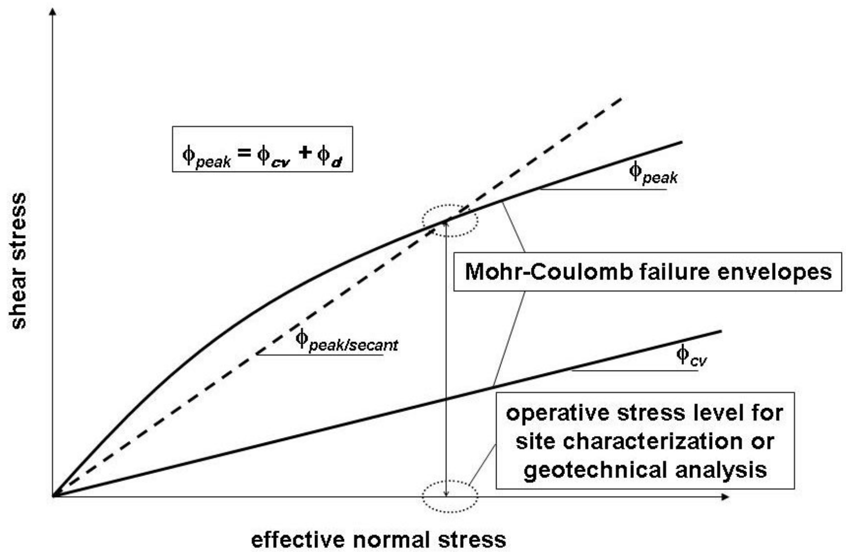

Figure 1. Mohr-Coulomb strength parameter definitions.

Implementation Overview

Stress state plays a crucial role in the proposed process twice. Initially, stress state is considered when developing correlations for both index properties such as D_r and engineering properties such as ϕ during the site characterization process that defines the preexisting, free-field conditions. Then the stress state that is presumed to exist under some condition being analyzed, e.g. bearing failure of a spread footing, is defined and used to scale the necessary soil properties to that stress state using the free-field stress state as a starting point. Note that the stress state being scaled to for analysis purposes is completely general and not necessarily that for an ultimate-failure condition. It could just as easily be some condition under service loads for the purposes of evaluating a serviceability issue.

Example Application

The classic foundation engineering problem of calculating the bearing capacity of a spread footing on coarse-grain soil will be used to illustrate the implementation of the proposed methodology in practice. This is a particularly good example to illustrate the power of the proposed methodology as it is well known that, independent of which particular bearing capacity solution is used (Hansen, Meyerhof, Terzaghi, Vesic, etc.), the calculated gross ultimate bearing capacity, q_{ult}, is extremely sensitive to the assumed value of ϕ because the magnitudes of the bearing capacity factors (N_c, N_q, N_γ) are each very sensitive to ϕ. Consequently, selection of an appropriate value of ϕ for bearing capacity analysis is always problematic in practice and thus the methodology presented in this paper that determines a unique, problem-specific design value of ϕ on a rational basis holds particular promise and attraction.

The specific data in this example are from a site in Texas that was used for a spread footing prediction symposium (Briaud and Gibbens 1994, Horvath 1994) as it allows the rare opportunity to compare calculated bearing capacity results with those measured on full-scale footings that were instrumented and loaded to bearing failure in a controlled, intentional fashion.

Site Characterization Algorithm

The first step in the process is to determine the index and engineering properties of the soil under the stress state existing at the time of the site characterization investigation. A planar, horizontal ground surface is assumed and the following soil properties are assumed to be known for the entire soil profile to the maximum depth of interest as they are used as input into the solution algorithm:

- cone penetrometer (CPT) tip resistance, q_c
- effective soil unit weight(s).

If only SPT data are available, then the following are used:

- SPT N_{60} (N_{field} corrected, if necessary, for 60% driving efficiency)
- D_{50} (particle diameter in millimetres for which 50% of the soil is finer by weight)

to convert the N_{field} to equivalent q_c data (in kPa) using the following relationship from Kulhawy and Mayne (1990):

$$q_c = N_{60} \cdot p_{atm} \cdot 5.44 \cdot \left(D_{50}\right)^{0.26} \qquad \textbf{(1)}$$

where p_{atm} is the atmospheric pressure in kPa. The N_{field}-to-N_{60} correction is either based on actual field measurements of energy delivered or empirical correlations such as in Kulhawy and Mayne (1990) that are based on the type of hammer system used.

CPT data were available for the Texas site and were used solely for the example presented in this paper. SPT data were also available for this site but not used although for the sake of completeness the D_{50} at this site is noted to be 0.20 mm. However, the writer has applied the methodology presented in this paper to other sites where both SPT and CPT data were available from explorations made in close proximity and found that Eq. 1 produced results that correlated well with the actual CPT data (Horvath 2002,2003,2004).

To illustrate the maximum utility of the methodology presented in this paper, in lieu of assuming soil unit weights beforehand the writer took things a step further by having the solution algorithm calculate these properties. This required the following addition assumptions (values used by the writer for the Texas site are indicated in brackets):

- Depth to ground-water table, z_w [4.9 metres].
- Unit weight of water, γ_w [9.8 kN/m^3].
- Natural water content, w_n, in percent within the vadose zone [14% above and 17% below foundation level for the footing analyzed].
- Maximum, γ_{dmax}, and minimum, γ_{dmin}, dry unit weights of the soil (e.g. from Kulhawy and Mayne 1990) [13.5 kN/m^3 and 15.9 kN/m^3 respectively].
- Specific gravity of solids, G_s [2.65].

Note that that no extraordinary field or laboratory testing is required to generate the required input for the site characterization algorithm, even with the added complexity of having the algorithm calculate soil unit weights. This was done intentionally by the writer when crafting the algorithm to create an overall procedure that was amenable for use in routine practice on even the smallest and simplest of projects with minimal site characterization budgets.

The following is the current version of the site characterization algorithm used by the writer. It reflects the original version (Horvath 2000a,2002) with updates and revisions as outlined in Horvath (2003,2004).

Each piece of CPT data (actual as here or from SPT data) represents a depth-point-of-interest that is evaluated on a top-down basis beginning from the ground surface. Note that because the algorithm includes equations containing multiple variables that are initially unknown the solution requires some strategy to deal with this. The writer used an iterative process built into a purpose-written computer program named *HINT*. Commercially available spreadsheet or mathematics software could also be used and the algorithm is even solvable by manual calculation although it quickly becomes onerous for any significant number of SPT or CPT data.

<u>Step 1</u>: Assume $D_r = 0$ and $OCR = 1$ for initial iteration (note that D_r varies between 0 and 1, not 0% and 100% as is more typical, for the purposes of this algorithm).

<u>Step 2</u>: Calculate the dry unit weight, γ_d:

$$\frac{1}{\gamma_d} = \left(\frac{1}{\gamma_{d\,\min}}\right) - \left[D_r \cdot \left(\frac{\gamma_{d\,\max} - \gamma_{d\,\min}}{\gamma_{d\,\max} \cdot \gamma_{d\,\min}}\right)\right]. \qquad \textbf{(2)}$$

<u>Step 3</u>: If the depth of interest, z, is within the vadose zone (i.e. $z \le z_w$), calculate the total (a.k.a. damp, wet, moist) unit weight, γ_t:

$$\gamma_t = \gamma_d \cdot [1 + (w_n / 100)] \qquad \textbf{(3a)}$$

otherwise (i.e. $z > z_w$) calculate the saturated unit weight, γ_{sat}:

$$\gamma_{sat} = \gamma_b + \gamma_w = \{[(G_s - 1)/G_s] \cdot \gamma_d\} + \gamma_w \qquad \textbf{(3b)}$$

where γ_b is the buoyant unit weight.

Step 4: Calculate the increment of vertical effective stress, $\Delta\sigma'_{vo}$, from the data point just above the depth of interest to the depth of interest using the appropriate unit weight from either Eq. 3a or 3b (the appropriate pore-pressure subtraction must also be made if below the ground-water table). σ'_{vo} at the depth of interest is the sum of $\Delta\sigma'_{vo}$ and σ'_{vo} at the data point just above the depth of interest.

Step 5: Calculate ϕ_{tc}, which is $\phi_{peak/secant}$ (see Fig. 1) in triaxial compression, at σ'_{vo} using the following empirical relationship in Kulhawy and Mayne (1990):

$$\phi_{tc} = 17.6 + \left\{ 11.0 \cdot \log_{10} \left[\frac{q_c / p_{atm}}{\left(\sigma'_{vo} / p_{atm} \right)^{0.5}} \right] \right\}. \tag{4}$$

Step 6: Calculate σ'_{ho} using the following empirical relationship per Mayne (personal communication, 2003):

$$\sigma'_{ho} = 0.30 \cdot q_c^{\ 0.22} \cdot \sigma'^{\ 0.69}_{vo} \cdot OCR^{0.27}. \tag{5}$$

Note that the original version of this algorithm (Horvath 2000a,2002) used a different empirical relationship given by Kulhawy and Mayne (1990). The sensitivity of calculated results to this change was discussed in Horvath (2003,2004).

Step 7: Calculate the coefficient of lateral earth pressure at rest, K_o:

$$K_a \le K_o \left(= \sigma'_{ho} / \sigma'_{vo} \right) \le K_p \tag{6}$$

where K_a and K_p are the Rankine active and passive coefficients of lateral earth pressure respectively calculated for plane-strain conditions using $\phi = \phi_{ps}$ where $\phi_{ps} = 1.1 \cdot \phi_{tc}$ from Eq. 4 (Kulhawy and Mayne 1990).

Step 8: Calculate K_{onc}, the value of K_o under normally consolidated ($OCR = 1$) conditions:

$$K_{onc} = 1 - \sin \phi_{cv} \tag{7}$$

where ϕ_{cv} is the constant-volume (critical-state) value of ϕ as suggested by Mesri and Hayat (1993). Earlier versions of this algorithm (Horvath 2000a,2002,2003) used ϕ_{tc} as suggested by Mayne and Kulhawy (1994), not ϕ_{cv}. The sensitivity of calculated results to this change was discussed in Horvath (2004).

Step 9: Calculate a revised, updated estimate of OCR using the following relationship given by Kulhawy and Mayne (1990) and others:

$$OCR = \left(K_o / K_{onc} \right)^{(1/\sin\phi_{cv})} \ge 1. \tag{8}$$

The same comments made in Step 8 concerning the choice for ϕ used in Eq. 8 (i.e. ϕ_{cv} versus ϕ_{tc}) apply here.

Step 10: Compare the value of *OCR* determined in Step 9 to that assumed at the beginning of the iteration cycle (which will always be 1 for the first iteration). If they are judged to be sufficiently close in magnitude the iteration can be considered converged and closed. Proceed to Step 11. If not, calculate an updated value of D_r using a simplified version of an empirical relationship in Kulhawy and Mayne (1990):

$$D_r = \sqrt{(D_r)^2} = \sqrt{\left[\frac{1}{305 \cdot OCR^{0.18}}\right] \cdot \left[\frac{q_c / p_{atm}}{(\sigma'_{vo} / p_{atm})^{0.5}}\right]} \leq 1 . \qquad \textbf{(9)}$$

Then return to Step 2 and perform another iteration but this time using the updated estimates of *OCR* (from Step 9/Eq. 8) and D_r (from this step/Eq. 9).

Step 11: Calculate:

$$\sigma'_{vm} = \sigma'_{vo} \cdot OCR . \qquad \textbf{(10)}$$

Step 12: Calculate ϕ_d, the dilatancy component of the Mohr-Coulomb friction angle, using the empirical relationship in Kulhawy and Mayne (1990):

$$\phi_d = 3 \cdot \left\{ \left\{ D_r \cdot \left[10 - \ln\left(\frac{100 \cdot \sigma'_f}{p_{atm}} \right) \right] \right\} - 1 \right\} \geq 0 \qquad \textbf{(11)}$$

where σ'_f is the *mean effective stress at failure* which in this case is defined as:

$$\sigma'_f = \{[\sigma'_{vo} + (2 \cdot \sigma'_{ho})]/3\} \qquad \textbf{(12)}$$

and the overburden stresses are those calculated previously in Steps 4 and 6.

Step 13: Using results from Steps 5 and 12, calculate:

$$\phi_{cv} = \phi_{peak/secant} - \phi_d = \phi_{tc} - \phi_d . \qquad \textbf{(13)}$$

Geotechnical Analysis Algorithm

The generic site characterization algorithm presented above is followed by a geotechnical analysis algorithm that is always application-specific, in this case for footing bearing capacity. The only additional soil property required for this particular analysis is $\phi_{peak/secant}$ at the operative stress level for bearing failure. Note that the mean effective stress at failure, σ'_f, for bearing capacity differs from that given by Eq.

12 which is for overburden stress conditions and was assumed to be that suggested by DeBeer (1987):

$$\sigma'_f = \{[q_{ult} + (3 \cdot \sigma'_{vo})]/4\} \cdot (1 - \sin\phi_{peak/secant}) \quad \textbf{(14)}$$

where q_{ult} is the gross ultimate bearing capacity from theory. Hansen's solution was used based on a comparative assessment among four different solutions as discussed in Horvath (2000b). σ'_{vo} is the value at the footing foundation level.

Because various parameters necessary to the solution are initially unknown the writer again used an iterative solution approach incorporated into *HINT*:

Step A: For the initial iteration, assume $\phi_{peak/secant} = \phi_{cv}$ and calculate q_{ult}. The value of ϕ_{cv} to use is obtained from the outcomes of the site characterization algorithm and is the average value within the theoretical failure zone that extends a depth B (= effective footing width) below foundation level.

Step B: Use the parameter values from Step A to calculate σ'_f using Eq. 14.

Step C: Use this estimate of σ'_f to calculate ϕ_d using Eq. 11.

Step D: Calculate a revised, updated estimate of $\phi_{peak/secant}$ using the value of ϕ_d from Step C in Eq. 13.

Step E: Use $\phi_{peak/secant}$ from Step D to recalculate q_{ult} then return to Step B to recalculate σ'_f using these new values for $\phi_{peak/secant}$ and q_{ult}.

Steps B through E are repeated until $\phi_{peak/secant}$ converges to sufficient precision (0.1° was used by the writer) with the final q_{ult} considered the final answer.

Results

These algorithms were applied to the "1.5-metre footing" at the Texas site. The site characterization algorithm yielded an average $D_r \approx 50\%$ and $\phi_{cv} = 32.6°$ within the theoretical bearing-failure zone that extends to a depth B beneath the footing. The bearing capacity algorithm outlined above converged at a value of $\phi_{peak/secant} = 37.0°$ which produced an estimated net ultimate column load at failure = 3306 kN that compares very favorably to the measured failure load (3400 kN) obtained using data presented in Briaud and Gibbens (1994) and the recommendation made by Vesic (1975) that shallow-foundation bearing failure occurs at a settlement = $0.15 \cdot B$. Note that the calculated net ultimate column load at failure using ϕ_{cv} is only 1596 kN which is less than half that measured even though the difference between $\phi_{peak/secant}$ and ϕ_{cv} in this case is only 4.4° (37.0° versus 32.6°). This emphasizes the well-known fact noted earlier that calculated bearing capacity is always extremely sensitive to the assumed value of ϕ.

Additional Work and Applications

Calculation of moduli (Young's, shear, drained-constrained) in both the recompression and virgin ranges can be and have been added to the site characterization algorithm as these properties are required for a variety of other geotechnical applications. Space limitations preclude inclusion of the additional steps required in this paper. Details can be found in Horvath (2002).

To date, the writer has also used the methodology presented in this paper for calculating the geotechnical ultimate axial-compressive capacity of driven piles. Details of this work can be found in Horvath (2002,2003,2004) and Horvath and Trochalides (2004).

CONCLUSIONS

The methodology of synergistically integrating site characterization and geotechnical analysis as presented in this paper holds great promise for both practitioners and researchers alike. Only after rationally obtained 'best' estimates of soil properties are used in some geotechnical analysis can a rational assessment of the accuracy of that analytical method be made. Thus using the approach presented in this paper should simultaneously allow practitioners to use existing analytical methods more rationally and accurately, and researchers to assess the need for new or improved analytical methods more objectively.

The writer considers the methodology presented in this paper to be one that is constantly changing and improving, particularly with regard to updating as well as expanding to fine-grain soil the site characterization algorithm (Horvath 2002a):

"Site characterization is clearly the key component of the proposed analytical methodology. Therefore the various correlations and algorithms used and presented herein should be updated on an ongoing basis to take advantage of the latest developments in this regard. The emerging picture of the key, underappreciated role played by site characterization in foundation design also indicates that future research and development as well as technology transfer funding should be devoted to this topic which is clearly the crucial, fundamental heart of deep foundation capacity calculation."

With regard to analytical methods, it is clear that efforts need to be made to develop and refine theoretical relationships to define the stress state either at failure or under service-load conditions as appropriate for various analytical applications.

One interesting and unexpected outcome of experience using this integrated approach in actual applications is that it clearly identifies which parameter(s) dominate the calculated outcomes of an analysis and how sensitive these parameters are to soil properties. For example, the extreme sensitivity of bearing capacity for both shallow and deep foundations to ϕ was noted. Noting which soil properties most affect the outcome of a given analysis is important not only for the analysis itself but it also suggests where future research into site characterization is best and most efficiently channeled (in this case improving empirical correlations for ϕ).

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