Perspectives in Forensic Geotechnical Engineering

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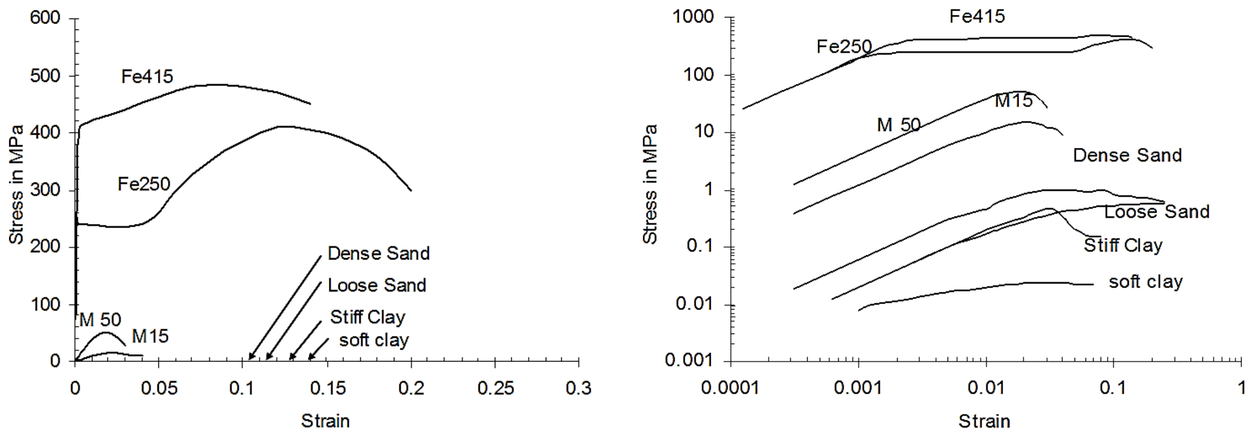
ABSTRACT

The paper presents some perspectives in forensic geotechnical engineering with respect to (i) structural vs. geotechnical engineering, (ii) bearing capacity vs. leaning instability, (iii) linear vs. non-linear and thin vs. thick layer responses. Traditional approaches for the estimation of ultimate bearing capacity of shallow foundations use limit equilibrium methods that are based only on soil strength apart from the geometry of the foundation element. However, the effect of the height of the structure on its stability is ignored. Tall structures, particularly towers, resting on or in soft ground, could lean or fail by a mechanism called leaning instability due to the compressibility of the ground rather than its strength. Terzaghi's one-dimensional consolidation theory is based on a linear void ratio–effective stress relationship, and is applicable for thin layers only. The differences between the linear theory and a theory of non-linear consolidation of a thick clay deposit considering linear void ratio–log effective stress relationship are highlighted. Some parallels are drawn between geotechnical engineering and the practice of medicine.

INTRODUCTION

Structural vs. Geotechnical Engineering. Structural engineers work with engineered materials such as concrete and steel that possess unique and well-defined properties of density, elastic modulus, compressive/tensile strengths, and flexural stiffness among others. On the other hand, geotechnical engineers deal with soil, a material made by Nature/God with highly variable spatial and temporal properties. Structural designs are code-based since theoretical closed-form solutions are derived based on the given geometry, material properties, and loading. However, geotechnical designs are judgement-based due to highly variable geometry, complex loading, and material properties that are not precisely determinable. According to Gray (1992), "Men are from Mars and Women are from Venus"; extending the analogy, indeed "Structural engineers are from Mars and Geotechnical Engineers are from Venus".

Stress–Strain Relationships. Figure 1(a) and (b) show the stress–strain curves of various civil engineering materials in linear and log-log scales, respectively. Soils are so weak and highly deformable that their stress–strain curves are not visible on the linear scale in comparison with those of steel (Fe250 and Fe415) and concrete (M50 and M15) but become discernable if plotted on a log-log scale. The ultimate strength and stiffness of steel are 415 MPa and 210 GPa, respectively, while those of soils range from about 10 kPa (soft clay) to 400 kPa (dense sand) and 0.5 to 10 MPa, respectively. Thus, these plots contrast the significantly large differences in the strength and stiffness of soils in comparison with those of steel and concrete. Soils are generally subjected to much higher strains at failure (of the order of about 15 to 20% for loose sands and soft clays, and 5 to 10% for dense sands and stiff clays) when compared to concrete (0.3%) or steel (about 0.2 to 0.4%)



(a) (b)

Figure 1. Stress-strain relations for various materials: (a) natural scale and (b) log-log scale

BEARING CAPACITY vs. LEANING INSTABILITY

Bearing Capacity of Shallow Foundations. Prandtl's theory is the starting point for the estimation of bearing capacity of shallow foundations. Terzaghi (1943) modified the same and proposed his theory for a strip footing resting on a cohesive–frictional (c–ϕ) soil (Figure 2). The slip mechanism consists of an active rigid/elastic wedge defined by the angle of shearing resistance ϕ, a fan region of continuous plastic deformation (distortion+rotation), and the passive wedges defined by the angle ($\phi/4$-$\pi/2$) with respect to the horizontal. Prandtl's solution modified for c–ϕ soils (c being the cohesion component) with the active wedge defined by ($\phi/4$+$\pi/2$) instead is adopted as appropriate for geotechnical applications. Soils do not fail in 'general shear' and a new failure mode termed 'local shear failure' was identified as a possible alternative. The ultimate bearing capacity of shallow foundations for local shear failure is estimated using the general shear failure equation but with both the strength parameters, c and tan ϕ, reduced to two-thirds of their corresponding values (Terzaghi and Peck 1967). Vesic (1973) extended this concept and identified a third failure mode, 'punching shear failure', occurring in loose soils at shallow depths and at depth in case of dense soils. Figure 3 classifies the three failure modes as dependent on both the relative density of granular soils and the relative depth D/B of the footing. Vesic (1973) proposed a general expression for the ultimate bearing capacity of shallow foundations based on cavity expansion analysis accounting for the compressibility of ground through a rigidity index $I_r = G/s_u$, the ratio of the shear stiffness to the undrained shear strength of ground. Unlike in structural engineering where ultimate capacity is based on the ultimate strength of the material, geotechnical designs are governed by both the strength as well as the compressibility of the ground.

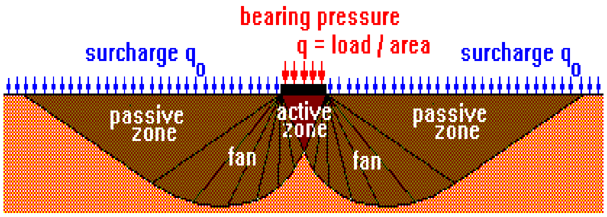


Figure 2. General shear failure for c–ϕ soil (Terzaghi 1943)

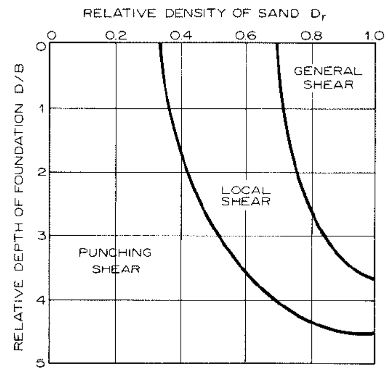


Figure 3. Modes of failure of shallow foundations in sand (after Vesic 1973)

Structure-Foundation-Ground Interaction (Leaning Instability). In traditional design, the influence of the height of the structure on its stability is ignored, while only foundation–ground interactions are considered. However, the height of a tall structure plays an important role in the overall behaviour of the system and leads to a different failure mechanism, termed 'Leaning Instability', examples of which are the famous Leaning Tower of Pisa and the more recent case of a 58-storey condominium in San Francisco. Leaning instability occurs at a critical height to width ratio when the overturning moment caused by a small inclination cannot be compensated by the corresponding resisting moment mobilized by the foundation (Hambly 1985, 1990). Leaning instability is due to the high compressibility of the ground and is not dependent on the strength. Figure 4 depicts (i) a structure whose height H is ignored (H = 0), (ii) a medium to low rise structure with H/B<1 and (iii) a high rise structure with H/B>1, where B is the width or diameter of the footing. It can be shown experimentally and analytically that the ultimate capacity of the footing decreases with increased height of the structure.

Studies by Hambly (1985, 1990), Cheney et al. (1991), Lancellotta (1993) and Potts (2003) quantify the effect of the height of the structure on its stability as somewhat akin to that of buckling of long columns. Incidentally, the buckling of long slender columns is controlled by the flexural stiffness of the structure and not by the strength of the material. Figure 5 illustrates a leaning instability model with the ground represented by a series of Winkler springs. In advanced mechanical modelling, the time-dependent response of the ground is represented by a Kelvin–Voigt model as shown in Figure 6.

Potts (2003) models a simple tower of 60 m height and 20 m diameter with an initial tilt of 0.5^0, resting on a uniform deposit of clay with s_u of 80 kPa and G of 10, 100 and 1000 times s_u. The clay was modelled as a linear-elastic Tresca material. Conventional theory predicts identical bearing capacities of the tower for all the three cases. However, if the rotation of the tower is plotted against its weight, the effect of G/s_u becomes significant (Figure 7). The weights of the tower at failure are 60, 110 and 130 MN for G/s_u of 10, 100 and 1000, respectively. Failure is abrupt for very stiff soils when compared to that of relatively softer soils.

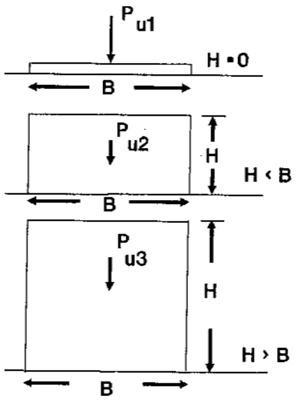


Figure 4. Structures with different heights *H* relative to their width/diameter *B*

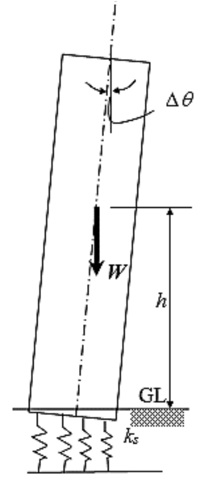


Figure 5. Model for leaning instability

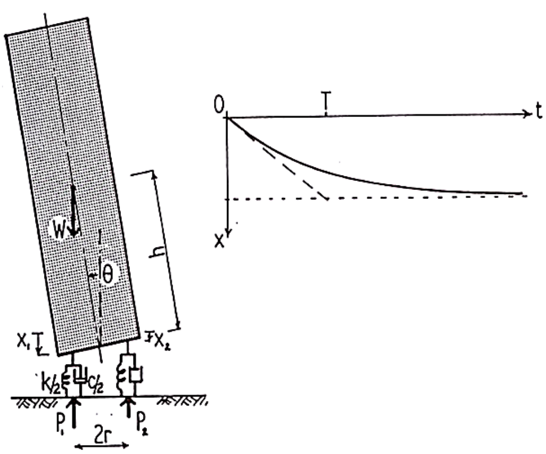


Figure 6. Kelvin–Voigt model for time-dependent behaviour of ground

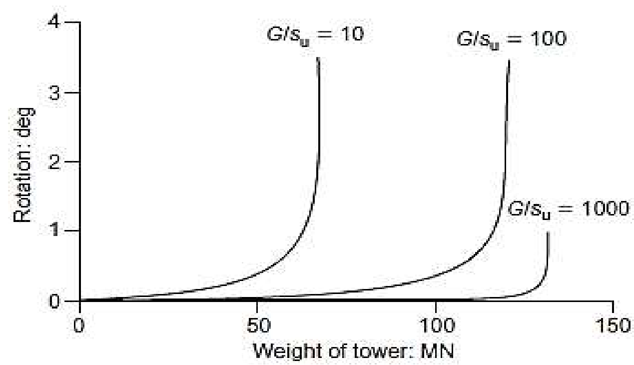


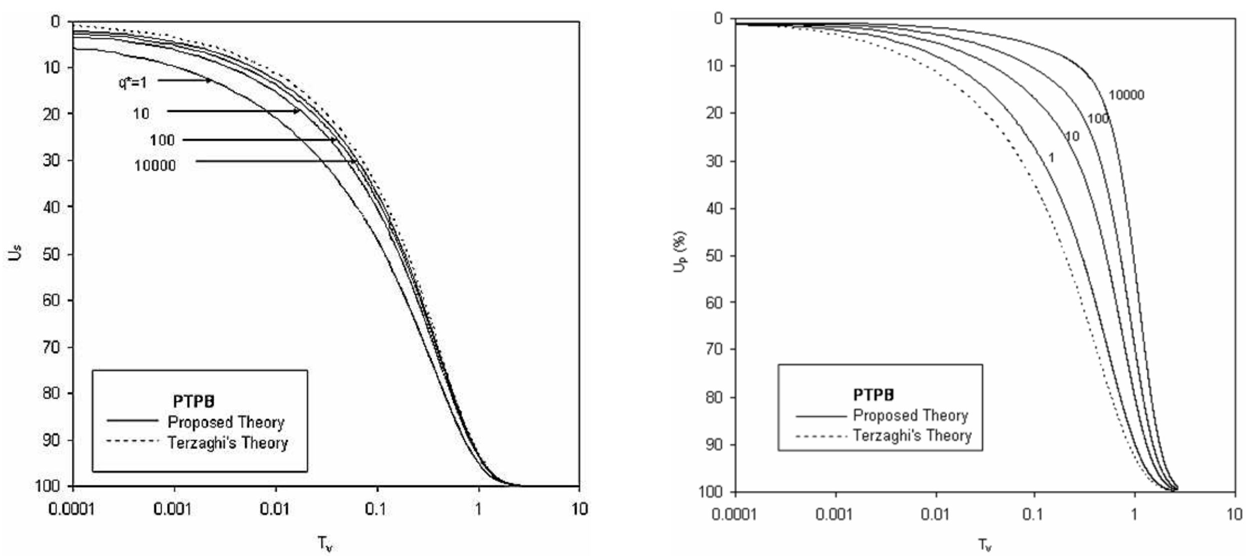
Figure 7. Rotation of tower with increase in its weight (Potts 2003)

THIN vs. THICK LAYER RESPONSES

Vertical Consolidation. Terzaghi's one-dimensional consolidation theory neglects the effect of self-weight of soil, assumes infinitesimal strain, linear relationship of void ratio and effective stress, and is valid for thin layers only. A theory of non-linear consolidation was proposed by Khan et al. (2010a) for a thick clay layer considering void ratio–log effective stress relationship by assuming (i) constant coefficient of consolidation (Madhav and Miura 2004) (the coefficient of hydraulic conductivity and the coefficient of volume change are inversely proportional to the vertical effective stress), (ii) constant thickness of clay layer, and (iii) constant initial void ratio with depth, but accounting for the variation of initial vertical effective stress with depth as an extension of the thin layer non-linear theory of consolidation developed by Davis and Raymond (1965). In the linear theory, the void ratio–effective stress relationship is considered to be bilinear with respective coefficients of volume change representing the slopes of the recompression and virgin compression lines. However, in the non-linear theory, the void ratio–log effective stress relationship is considered to be bilinear with the recompression index C_r and the compression index C_c representing the slopes of the recompression and virgin compression lines, respectively.

Figure 8(a) shows the average degree of settlement for the entire thickness of the clay layer U_s versus the time factor for vertical flow T_v (= $c_v.t/H_{dr}^2$) where c_v is the coefficient of consolidation for vertical flow, t is the time needed to complete the required degree of consolidation, and H_{dr} is the length of the drainage path (= H/2 for the case of double drainage or pervious top and pervious bottom PTPB, where H is the thickness of the clay layer). The results from the thick layer theory tend to those from the conventional thin layer theory for normalized load intensity q^* (= $q/(\gamma'H)$) ≥ 10,000 (i.e., for H tending to zero, thin layer). For T_v of 0.197, the degree of consolidation increases from 51% to 60% for q^* decreasing from 10,000 to 1. Thus, the theory of consolidation for thin layers underestimates the degree of settlement.

Figure 8(b) presents the average degree of dissipation of excess pore pressures for the entire thickness of the clay layer U_p versus T_v. The degree of dissipation of excess pore pressure from the non-linear theory is slower than the degree of settlement. While the degree of settlement is 51% (Figure 8(a)), the corresponding degree of dissipation of excess pore pressure is only 8% for q^* of 10,000 in thick layers, against a value of 50% from the thin layer theory, for T_v of 0.197. Thus, the conventional thin layer theory overestimates the degree of dissipation of excess pore pressures.



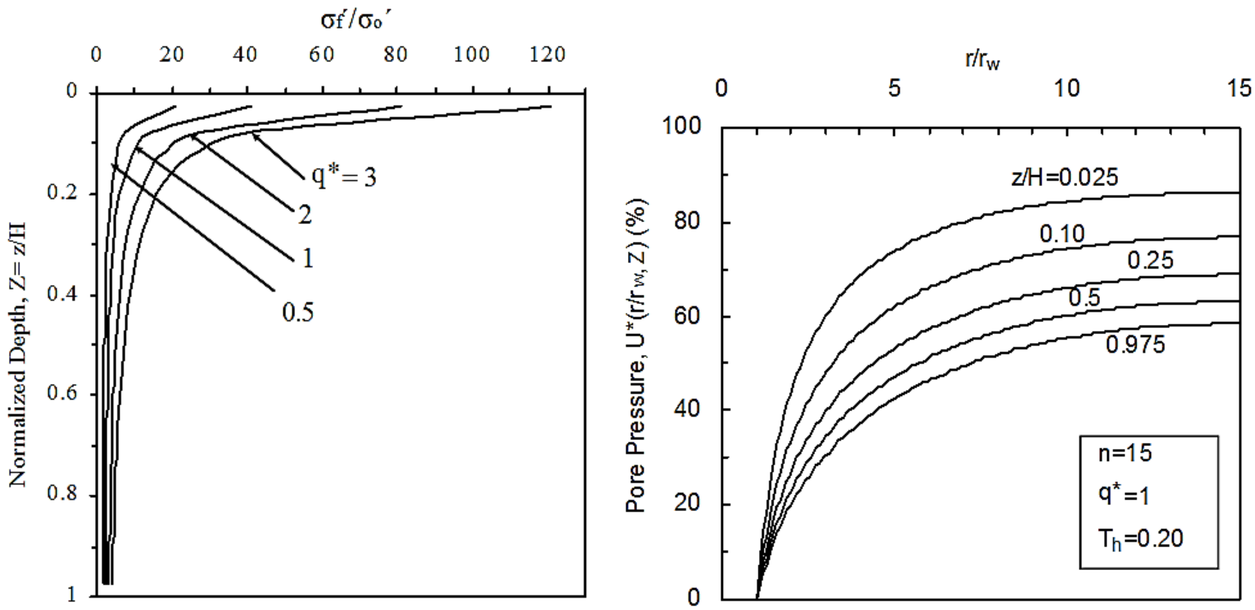
(a) (b)

Figure 8. PTPB results: (a) degree of consolidation vs. time factor and (b) average degree of dissipation of excess pore pressures vs. time factor (Khan et al. 2010a)

Radial Consolidation. Preloading with prefabricated vertical drains (PVDs) is one of the most effective methods of soft ground improvement. The classical theory of Barron (1948) is based on the assumptions of small strains, a linear void ratio–effective stress relationship and constant coefficients of volume compressibility m_v and hydraulic conductivity in the horizontal direction k_h. However, for a relatively large applied stress range, void ratio is not proportional to vertical effective stress and the coefficients of compressibility and hydraulic conductivity decrease during consolidation. Khan et al. (2010b) developed a theory of non-linear consolidation for radial flow around a PVD in a thick deposit of clay based on the non-linear theory of consolidation for vertical flow presented by Davis and Raymond (1965). Both initial and final (initial+applied load) vertical effective stresses were considered to vary linearly with depth.

Figure 9(a) depicts the variation of σ'_f/σ'_0, the ratio of the final to the initial vertical effective stress, with normalized depth Z (= z/H) for different values of q^*. The vertical effective stress ratio σ'_f/σ'_0 decreases sharply from values as high as 21–121 near the ground surface ($Z = 0.025$) to 6.3–33 at $Z = 0.1$ for values of q^* increasing from 0.5 to 3.0. The sharp decrease of σ'_f/σ'_0 is due to the low initial vertical effective stress near the ground surface.

The decrease of σ'_f/σ'_0 with depth is significant for Z = 0.1–0.4, but is negligible for $Z > 0.4$. Hence, the effect of non-linear consolidation in a thick clay layer by PVD is pronounced at shallow depths compared to that at deeper depths. Figure 9(b) illustrates the excess pore pressure variation with radial distance at different depths for $q^* = 1$, $n = 15$ and $T_h = 0.2$, where $n = d_e/d_w$ (d_e is the equivalent diameter of the zone of influence of the PVD and d_w is the equivalent diameter of the PVD) and T_h is the time factor for horizontal flow.



(a) (b)

Figure 9. Results for non-linear thick layer consolidation by PVD: (a) variation of σ'_f/σ'_0 with depth (Khan et al. 2009) and (b) variation of excess pore pressures with radial distance – effect of depth (Khan et al. 2010b)

$r/r_w = 1$ corresponds to the edge of the PVD while $r/r_w = 15$ corresponds to the edge of the unit cell. The normalized excess pore pressure U^* (= u/u_0, where u_0 is the initial excess pore pressure) is relatively large at shallow depths where σ'_f/σ'_0 is extremely large, but decreases with depth since σ'_f/σ'_0 also decreases. The excess pore pressure variation with radial distance r is relatively more significant in the upper half of the deposit compared to that in the lower half. At the edge of the unit cell ($r/r_w = 15$) and for T_h of 0.2, U^* decreases from about 87% near the ground surface to 63% at $Z = 0.5$ and to about 58% near the bottom of the deposit ($Z = 0.975$).

GEOTECHNICAL vs. MEDICAL PRACTICES

Several similarities can be observed between the practices of medicine that deals with the human body and geotechnical engineering that deals with the ground. Firstly, both are not manufactured to specifications, though off late, cloning is becoming possible. Secondly, both the human body and the ground have evolved over long periods of time, by natural evolution in the case of the former, and by geological processes in the case of the latter. A human being has the usual set of organs, limbs, bones, and muscles. While these features appear to be the same for most human beings, however, each human is very different from another because of genetics, pedigree, upbringing, parental care, and environment. Thus, we have extroverts or introverts, traits such as sad/happy, helpful (friendly), neutral or unfriendly, and positive or negative attitudes. Likewise, soil can be characterized as porous, saturated/unsaturated, non-homogeneous, anisotropic, inelastic (elasto-viscoplastic), dilatant, sensitive, with failure state varying from brittle to ductile, and a material with memory (preconsolidation stress, overconsolidation ratio).

While there are several parallels between the practices of medicine and geotechnical engineering (Madhav and Abhishek 2016), there are, however, some major differences:

1. In medicine, the patient goes to a doctor, whereas, a geotechnical engineer has to go to the site to diagnose the problem.
2. The patient talks to the doctor, whereas, a geotechnical engineer listens to the ground.
3. The failures of doctors are often buried or cremated in the ground, whereas, the successes of geotechnical engineers get buried and failures show up glaringly.
4. Doctors are paid much more handsomely than geotechnical engineers.

CONCLUDING REMARKS

Some perspectives in forensic geotechnical engineering that need to be considered are presented. Consideration of the height of a tall structure rather than just the bearing capacity of the foundation leads to an interesting failure mechanism termed leaning instability that is governed by the compressibility of the ground rather than its strength. The non-linear, thick layer theory of consolidation indicates that the conventional linear, thin layer theory underestimates the degree of consolidation and overestimates the degree of dissipation of excess pore pressures. The non-linear theory for radial flow into PVDs has similar effects on the degree of settlement, degree of dissipation of excess pore pressures and variation of excess pore pressures with respect to time, radial distance and depth. An interesting outcome is the induced vertical flow in a radial flow problem. In sum, if important mechanisms or responses are not taken into account during a forensic geotechnical investigation, then the predicted cause of failure would be completely different to what the real cause actually might have been.

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