Analysis of geotechnical failures, theoretical development, 3-dimensional behaviour

The re-examination of significant and well documented case histories of failure or foundation performance using modern methods of analyses, adds a measure of realism to the validation of the traditional design methods. Moreover, the incorporation of relevant aspects of the state-of-the-art knowledge and modern research into geotechnical problems, contributes to the decrease of gap existing between theory and practice. In some cases, the revisiting procedure can trigger off further theoretical development of interesting geotechnical issues, where the literature is quite limited. Following the above-mentioned, the subjects of the present doctoral thesis are: (a) Parametric analyses of the behaviour of retaining structures by prestressed anchorages under 3D and 2D conditions. (b) Re-evaluation of geotechnical failures, performing 3D and 2D F.E. analyses, with special emphasis on in-situ conditions or inherent factors, which had been ignored or underestimated from the original design. (c) Theoretical analyses of several issues arose from the investigated case histories. 1. 3D Behaviour of retaining structures with prestressed anchorages The behaviour of retaining structures of vertical cuts supported with prestressed anchorages on piles, especially the global safety factor (SF) and the displacements, horizontal (ux) or vertical-settlements (uy), were investigated by 3D and 2D F.E. analyses. The influence of several important factors was examined, as: i) The relative embedded depth of piles D/H (H the height of slope). ii) The shear strength, which is mobilized due to the prestress forces, for two basic soil types, S1 (weathered weak rock with high internal friction) and S2 (very stiff O.C. clay with high cohesion), using the linear elastic-perfectly plastic model according to Mohr-Coulomb. Additionally, the corresponding soil types HS1 and HS2 for soil simulation, according to Hardening Soil Model were introduced in the F.E. analyses. iii) The normalized total design resistance of the anchorages R = ΣRa,d/s·γ·H2 (Ra,d: design resistance of each anchorage, s: horizontal distance between supporting piles), iv) The normalized length of the excavation L/H and the aspect ratio L/B of the plan view, as well. v) The coefficient of in situ effective horizontal stresses at rest, Ko. Referring to the global safety factor, from the more important 3D analyses, it was verified that the ratio L/H has a dominant effect on SF. For the narrower investigated case L/H = 1 (and aspect ratio L/B = 1), the resulted SF is much higher than this from 2D conditions. Nevertheless, the favourable effect of triaxial conditions (owed to the contribution of shear forces at the edges) significantly decreases for L/H ≥ 2. Moreover, the favourable effect of increasing R on the SF becomes insignificant for low L/H values (i.e. for L/H = 1 the safety factor remains practically constant for increasing R, mainly in the case of soil type S2). On the other hand, for plain strain conditions (from 2D F.E. analyses), one of the basic factors, which influences the global safety factor is the normalized resistance of the anchorages, R. The rate of SF increase is higher in the soil with the higher angle of internal friction, φ΄. Another important factor which influence directly the mode of failure and indirectly the factor SF is the normalized embedded depth of piles, D/H, while the coefficient of horizontal stresses at rest, Ko has not any effect. Although most of the analyses were carried out for three rows of anchors (n = 3) and H = 12 m, additional calculations for n = 2-6 (and variable height H) verified that the effect of R on SF is almost independent from the number of anchors. The most important factor, which influences the horizontal displacements ux at the crest of retaining structure, besides the soil type, is the earth pressure coefficient at rest, Ko. The maximum horizontal displacements in 3D conditions, ux and uz, are sometimes lower and sometimes higher than those resulting from 2D F.E. analyses. Despite the fact that generally, these horizontal displacements were decreasing with the increase of the anchor forces, in 3D conditions the effect of normalized design resistance, R, becomes less significant for low values L/H. Especially for L/H = 1, the displacements ux, uz seem not to be influenced from the total resistance, R. The non-uniform distribution of horizontal displacements along the crest was investigated though the ratio of minux at the corners and maxux at the middle, ranging in the analyzed cases from 0.10 to 0.15. As it is already known from the literature, the distribution of surface settlements, based on measurements, has usually the shape of hogging or sagging. The probability of damages on surface structures depends not only on the maximum settlements, but on the shape of the deformed surface in the primary zone of influence, as well. From 3D F.E. analyses the settlements distribution along the cross-section at the axis of symmetry was presented in diagrams, indicating in most cases sagging, since the maximum settlement develops at a distance from the crest. On the contrary, 2D analyses resulted in lower displacements and heave at the crest. The basic differences between 2D and 3D analyses are pinpointed at the unavoidable non accurate simulation of the embedded part of piles below the excavation level, as continuous plates (2D) and not distinct elements, as in the 3D case. A more accurate simulation in 2D was achieved through appropriately lower shear strength reduction factor, Rint, below the level of excavation. The application of soil model HSM (in order to differentiate the modulus of elasticity for loading and unloading) results in more reliable surface settlement distribution, since the effects of soil heave due to the excavation, become less important. Usually, in the literature, the surface settlements were normalized in the form uy/H and relevant diagrams have been repeatedly presented, mainly for braced excavations. However, the settlements must be proportional to the term γ·H2 (and not to the height H) and also depend on a basic modulus of elasticity (E for the model EL-PL and E50 for the HSM). Consequently, the presented diagrams in the doctoral thesis were illustrated the relationship ux,y ·(E/γ·H2) − xo/H, where xo the distance from the face of excavation. 2. Back analyses of geotechnical failures The selection of case histories was based on the sufficient documentation, the interest of the case, as well as on the previous investigations. The new analyses were performed by the F.E. method, using 3D and 2D programs. In total, six slope cases of failure and four cases of foundation failure were analyzed. Among the slope failure cases, the following were included: i) A major slide occurred in the Kimola Canal (Helsinki, Finland) area, which is covered by N.C. or slightly overconsolidated clay. ii) Slope failure at the portal excavation of the tunnel S2 (Egnatia Highway, Epirus, Greece), in weathered flysch formation. iii) A great slope failure at an open pit mine (Megalopolis, Greece), in a short-term slope. iv) Landslide along a presheared surface in marly clay (Crete, Greece). v) Failure of a slope temporary stabilized by bored piles (Highway, Attica, Greece). Following the last case (restraint of slope by piles), the 3D behaviour of slopes stabilized by piles was investigated by coupled analyses, which consider simultaneously the piles response, the displacements and the safety factor. The effects of some important factors were investigated either by main 3D parametric analyses or 2D simplified ones: i) The influence of normalized distance of piles (arching effects), ii) The interplay of normalized pile length, iii) The position of the row of piles on the slope. Two representative cases of the piles arrangement and three soil types were considered in the parametric analyses. Among the foundation failure cases, the collapse of Transcona Grain Elevator (Canada) has a special interest. The new analyses were performed after the accurate simulation of structure (in 3D), the available geotechnical data and the loading steps. During the 3D F.E. analyses, the consolidation process was taken into consideration, due to its great importance. The basic conclusions from the new analyses are the following: (a) A main reason of the catastrophic failure was the rapid loading by the grain in conjunction with the low permeability of soft clay layers consisting the subsoil in the area. (b) The elevator was underlain by a two-layered soil system, where a weaker clay layer was located below the typical stiff clay crust of the area. These soil conditions had not been detected during the design of foundation, since the fined-grained sediments were seemingly uniform. (c) From the current 3D F.E. analyses, the global safety factor just after the loading step at the time of failure was estimated slightly higher than unity. However, it was verified that several factors, as the exact level of bedrock the scattering of undrained shear strength values or even temporary horizontal forces could influence the SF value. The potential effects of an initial tilt ωo of structure on bearing capacity and leaning stability were estimated by 3D F.E. analyses after the calculation of secondary rotation Δω. It was confirmed that the contribution of low tilt of the order of ωo = 1° on the foundation failure should be quite low. The results from simplified calculations using the simulation after Winkler were compared with those from more rigorous 3D F.E. analyses and the convergence or deviation were commented. 3. Leaning instability of high structures−The Tower of PisaFor an initial tilt ωo of a high structure, the secondary rotation Δω, due to the diversion of gravity center, the concentration of contact pressures at the foundation, e.t.c. can be calculated. The final tilt at equilibrium (if this is the case), ω = ωo + Δω is of peculiar interest. The progressive increase of initial angle ωo, could potentially resulted in leaning instability of a high structure under several preconditions. This case was examined through the external equilibrium, where the problem is pinpointed on the ability of foundation−soil system to balance the disturbing moment and the eccentricity of vertical forces. The stability condition is MR(ω) ≥ MD(ω), where MD(ω) the disturbing moment for given tilt, ω, and MR(ω) the maximum stabilizing moment from the foundation-soil system. In a first step, preliminary calculations were performed by the simple Winkler’s model, using different spring constants, KSU for unloading and KSL for loading. From the ratio fk = KSU/KSL (fk > 1) for rectangular and cyclic footings, the normalized transposition of the "neutral" axis (rotation axis), xR/B and xR/R, was calculated correspondingly, as well as the safety factor for leaning instability, SF = MR(ω)/MD(ω). The case fk = 1 is the most unfavourable one, thus the ratio SF/SF1 (for fk >1 and fk = 1) indicates how the differentiation of spring constants affects the stability of structure. The convergence of high adjacent structures (i.e. the so-called "kissing silos") was examined with 3D F.E. analyses, where the linear elastic-perfectly plastic, according to Mohr-Coulomb soil model were used. The methodology and the conclusions, which were drawn are the following: i) Besides the self-evident interaction between the footings and the resulting "initial" tilt, ωo, secondary effects were expected (Δω), due to the diversion from the vertical, the disturbing moments on each foundation, e.t.c. It must be noted that according to literature, the convergence should be attributed to the overlap of pressure bulbs beneath the foundations, only. The contribution of each factor on the final rotation ω = ωo + Δω was illustrated through the ratio ω/ωo ≈ sinω/sinωo. ii) The initial tilt ωo was firstly calculated according to the linear elastic soil model, as function of the normalized uniform loading q* = (q·(1-v^2))/E and the interaction factor, ΔI, depending on the dimensionless free distance of the footings (So/B). A best fitting algebraic expression was presented for the function ΔI = f(So/B), based on the 3D F.E. results. The final rotation Δω was calculated, depending also on the relative height of the center of gravity (hs/B). iii) The effects of relatively low safety factors against bearing capacity were examined for the linear elastic-perfectly plastic soil model and especially for clays under undrained conditions. It seems that the effects of low safety factors on the "initial" tilt ωo, owed to the interaction of footings are insignificant. On the contrary, in the case of insufficient undrained shear strength, these effects on the additional tilt (Δω) and the final one (ω) are of great importance. This trend was illustrated through the ratio sinωp/sinωe, where the final tilts ωp and ωe correspond to elastoplastic or linear elastic soil model. The leaning instability was examined in terms of the safety factor, normalized uniform loading q*, distance (So/B) and relative height of the structure (hs/B). The leaning Tower of Pisa: Several important issues regarding the stability of the Pisa Tower were investigated by 3D F.E. analyses, as the time dependent bearing capacity, the potential leaning instability and the interpretation of the time-tilt observations. According to the detailed geotechnical investigations performed at the site, the subsoil has been divided into three major horizons, the most important of which are the upper two: A (10 m thick) consisting from inhomogeneous cohesionless sublayers (SM, ML) and B (total thickness 30 m), where the upper sublayer B1 (Pancone clay and the lower one B4) are of great importance, while the stress history of both A and B is of peculiar interest. The 3D F.E. analyses were carried out by the Program Plaxis 3D Foundation. The Tower was modeled by adequate detail, taking into account the three main construction phases, the accurate dimensions of both the superstructure and ring foundation and the applied loadings (total mean pressure 507 kPa, A´ phase 65%, B´ phase 95%). Three different soil models were used in the thesis, namely the elastic-perfectly plastic according to Mohr-Coulomb (EL-PL), the Soft Soil Model (SSM) and the Soft Soil Creep model (SSCM). Evidently, the latter models better reflect the effects of unloading due to the progressive tilting. The main conclusions from the analyses are the following: (a) The available safety factor against bearing capacity instability was investigated in both undrained and drained conditions, depending on the corresponding construction period or later of the last C´ phase. After the A´ phase (65% of loadings), the available safety factor was estimated SF = 1.8 − 2.6. After the B´ phase (95% of loadings), this factor decreased due to the progressive inclinations, keeping not lower than SF = 1.70 and finally (year 1990) SF ≈ 1.50. Consequently, the view (according to several references), that general failure mechanism had influenced the stability of the Tower, was not verified by the 3D analyses. (b) The main compressible layers A, B1 and B4 had been strongly influenced by the final loads. On the contrary, the total effective stresses were almost equal or marginally higher than the preconsolidation ones after phase A´. These differences might explain the well-known history of tilting. (c) The leaning instability of Tower was investigated following the simple conceptual model of inverted pendulum (Burland et al, 2003). By the comparison of diagrams sinω-e and sinωs-e, where ω, ωs the active tilt and the calculated one on the basis of the disturbing moment and the rotational rigidity, for varying eccentricities e, the following were ascertained: i) For eccentricities e > 2 m (corresponding to ω > 4.7°) or even lower, it seems that leaning instability had been arisen, since ωs > ω. ii) From the comparison of 3D F.E. results performed either with SSCM or SSM soil models the significance of creep settlements was verified. Moreover, using the model SSCM the back calculated mean settlement, approaches very well the measured one on 1990 (sm = 2.95 m). (d) In order to examine the reasons for the development of the initial tilt after the A´ phase of construction (ω ≈ 1.1°), back analyses of the differential settlements were performed, assuming various hypotheses of inhomogeneity of the upper layer A´. Although the geotechnical model used was approximative, it seems that the certain inhomogeneity between North and South side of the Tower is the main reason of the initial tilt. The effects of systematic pumpings were also investigated by 3D F.E. analyses for various hypotheses for the hydraulic heads. Despite the fact that the pumpings result in additional settlements of the area, the increase of tilt is not certain in any case. (e) The rotational rigidity of foundation, which decreases when the eccentricity increases, was also investigated by 3D F.E. analyses, either for homogeneous or inhomogeneous soil. The function Kω = f(ω) was illustrated with diagrams for both cases and best fitting algebraic relations were also presented. 4. Bearing capacity of footings on two-layered clayThe undrained bearing capacity on a two-layered clay of rigid strip, rectangular and ring footings was studied parametrically with 2D, 3D and axisymmetric finite element analyses. The shear strength ratio of two layers (SR = su,2/su,1) and the relative thickness of the uppermost layer (H1/B), with respect to pertinent foundation dimension were the key parameters investigated. In addition, for rectangles and rings, the aspect ratio (length to width) and the ring width to external radius ratio (b/R1) were reporte d. The results were portrayed in diagrams of bearing capacity and shape modification factors, in the familiar soil mechanics form, along with best fitted algebraic expressions to facilitate their numerical use. The visual understanding of the effects of second layer (either weaker or stronger) was facilitated by the modification coefficients, λN = NC1/NC and λ*N = (N*C1)/(N*C), for strip or other footing shape. The failure mechanisms were presented for a number of characteristic cases, giving additional insight into the mechanics of the problem. Two cases, SR < 1 and SR > 1 were separately examined due to different failure mechanisms in each of them, either for central loads or for combined V, M loading.(a) Central loads, SR < 1: In the most interesting case of upper stiff crust and strip footings, three types of failure (I,II, III) were ascertained. Bound values (combinations H1/B and SR) were presented in each case. For a wide range of parameters, the B.C. factor NC,1 (referring to the undrained shear stre ngth su,1) is linearly related with H1/B. For rectangular or ring footings, the failure mechanisms are modified accordingly. Modification coefficients λN and λ*N and shape factor sc as well, were presented and discussed. In case of SR >1 (stiffer lower clay) and strip footings, two additional failure mechanisms were noted (IV and V). The modification coefficient (λN > 1) rapidly increases with increasing SR, reaching the value maxλN. The bound values H1/B, for which the favourable effect of stronger second layer becomes insignificant, independently from SR, were estimated (i.e. H1/B ≥ 0.5 for strip and even lower for ring and square footing). Closed form analytical expressions, best fitting the F.E. results were presented in several cases of strip and square footings, either for SR < 1 and SR > 1. (b) Combined (V,M) loading: The cases of strip and square footings were investigated for SR < 1 and SR > 1.The effects of normalized eccentricity were examined for a wide range (0 ≤ e/B ≤ 0.48), with 2D or 3D F.E. analyses through the B.C. factors NC1,e and N*C1,e. The familiar concept of effective width (B΄= B-2e) with reference to the estimation of ultimate load of strip or square footings, was verified by the F.E. analyses, for SR = 1. The same form of equations for the ultimate eccentric vertical load was also adopted for two-layered clays, so the B.C. factors NC1,e and N*C1,e were calculated, considering the effective width instead of B. The case of stronger upper layer (SR < 1) is of peculiar interest, since for centric loads and low SR values, the failure mechanism could be extended deep enough into the lower weaker clay, even for high normalized thickness, H1/B. Following the normalized eccentricity increase, the failure surface moves up, so the B.C. factors NC1,e and N*C1,e also increase. This trend could be considerable for low SR and relatively high H1/B values. From the diagrams Vu,e/Vu for both strip and square footings, it was concluded that the unfavourable effect of lower clay on the ultimate load, decreases with increasing the eccentricity, e/B. Consequently, for high e/B values, the predominant factor affecting the B.C. is the undrained shear strength of the crust. In the case of weaker upper layer (SR > 1), the B.C. factors increase with increasing SR, reaching the limit values max NC1,e and maxN*C1,e for strip and square. The eccentricity results in shrinking up of failure mechanism, thus the favourable effect of second layer decreases, as the ratio e/B increases. For square footings, the values maxλ*N were quite low if e/B > 0.15, even in cases of low H1/B, therefore the two-layered system could be considered as uniform clay with undrained shear strength, su,1. The shape and maximum values of the interactive diagrams of normalized ultimate vertical loads (Vu,e/Vu) versus ultimate moments (Mu/(Vu,o·B)), where Vu,o corresponds to e/B = 0, depend on the ratios NC1,e/NC1 or N*C1,e/N*C1 for strip and square footings. It is deduced that the parabola for SR = 1 comes in-between the failure loci curves for SR < 1 and SR > 1. Generally, the divergence between the curves for SR = 1 and SR > 1 were small or almost negligible. (c) Depth effects: This factor, for embedded footings usually is ignored. Two type of analyses were followed in this case: Analyses I, by exact geometrical simulation of footing, e.t.c. and Analyses II by the simplified assumption of footing on the soil surface, which is loaded by extended uniform pressure equal to the effective overburden at the active foundation level. From the analyses, diagrams for depth factor were derived. (d) Additional factors were examined, such as the rigidity of footings (two cases of practical rigid and infinitely flexible) and the initial stress field at the contact area between the two clay layers. It was concluded that following the usual in the literature simulation of clay layers, as weightless, the bearing capacity might be underestimated.