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Norwegian practice related to design of braced excavations

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SYNOPSIS: Most braced excavations in Norway have been carried out in soft clays. Safety against base heave is a critical design issue and has a major impact on support loads and ground movements. Earth pressure loading is also significantly influenced by interface wall friction and 3-dimensional effects. Steel sheet pile walls are most commonly used as retaining structure and have recently also been used as permanent structure. Norwegian Codes are based on the Limit State principle.

1 INTRODUCTION

The design of braced excavations has been given much attention in Norway, both with respect to development of construction methods and design principles. New development has also been closely tied to performance monitoring of real structures. The emphasis in the following is given to current practice.

2 TYPICAL PROBLEMS AND CONSTRUCTION METHODS

The majority of deep braced excavations in Norway have been carried out in soft normally consolidated marine clay deposits. The works related to the Oslo subway in the late 50's and early 60's represented an early challenge. Bottom heave stability was the main critical issue for these cut- and cover excavations extending to depths of 8-12 m. Driven sheet pile walls with internal strutting was used throughout, but to ensure bottom heave stability in the most critical areas, one made use of excavation under air pressure, excavation under water and driving of a bottom shield ahead of the excavated front (Bjerrum et al., 1966).

A new method of preventing bottom heave failure was introduced for construction of railway and subway tunnels through Oslo (Eide et al., 1972). The excavation for this double-deck tunnel was 15 m deep, 5 m deeper than the critical depth against base heave. The longitudinal diaphragm walls were first constructed to 20 m depth. Then cross-lot trenches were excavated every 4,5 m along the tunnel and cast with concrete up to the level of the base slab. The rest of the trench was backfilled with cement stabilized sand, but a temporary strut was also placed just below the middle deck. Thereby one obtained pre-strutting as well as bottom heave prevention by means of the below bottom cross-lot wall panels. This principle worked out very nicely (Karslrud, 1981).

Ground improvement by means of deep cement/lime mixing has found increasing applications for ensuring bottom heave stability, act as below bottom support and to form the retaining structure itself. Jet-grouted columns have so far had limited applications for excavations in Norway. The cost effectiveness (cost/strength ratio) is significantly less than for the current deep mixing methods. The jet-grouting method will probably still find its place where space, strength, and water cut-off are critical issues.

Driven sheet-pile walls are the most commonly used retaining wall in Norway today, even for deep excavations where diaphragm walls used to dominate. The reasons are mainly a cost issue, availability of heavy pile driving rigs, and that also sheet pile walls are now used as the permanent structure (Bruskeland, 1991; Finstad, 1991). The largest sheet-pile wall driven is a combined HZ profile with a moment capacity of 4200 kN/m, about the same as a 1.2 m thick diaphragm wall, driven to 25 m depth.

The temporary retaining walls are usually supported by internal steel strutting when the width of excavation is limited (say < 10-15 m), and by tie-back anchors drilled and grouted into bedrock when the width becomes larger and the depth to bedrock is limited (say < 30 m). Tie-back anchors into rock with capacity up to 5000 kN have been used.

Grouted soil anchors are not so well suited in soft clays, but have been applied in silts and sands. The special Soilex Expander Body anchor developed in Sweden has found increasing applications also in soft clays. It can usually be driven into the ground giving installation times many times faster than drilled and grouted anchors, and gives a well defined anchor body.

For large basement excavations, casting of the basement floor slabs directly on the ground surface can be an attractive alternative to anchoring and internal strutting. An open area is often left in the middle to allow easy access. Piles must of course be installed first to support the slabs. This principle has also been combined with parallel construction of the superstructure with excavation of the basement and casting of the slabs (Finstad, 1991).

Aas (1984) presented another case of a large excavation of about 125 x 75 m to a depth of about 10 m with soft clay extending to more than 50 m. A 12 m wide portion of the basement structure along the periphery was first excavated for and constructed within internally braced sheet-pile walls. After this outer frame was established, the remaining central area was freely excavated for in sections with successive construction of the basement structure.

3 DESIGN ANALYSES

3.1 Base failure

For all excavations in soft clays, base failure is a critical design issue. The well known inverse bearing capacity formula is used to predict the safety factor, F, from

$$F = \frac{N_c \cdot s_u}{\gamma_t D + q} \qquad (1)$$

The stability number, N_c, varies from 4.14 to 9.0, depending on the width, B, length, L, and depth, D, of the excavation (Janbu et al., 1966). It should be noted that the undrained shear strength, s_u, that is used with this formula, is the weighted average strength along the below-base level failure surface. When the undrained shear strength below the base varies with depth, one must in general try different failure geometries as illustrated in Fig 1. Note that the N_c-value is also influenced by the choice of B_f, and that B_f may be limited by the depth to a firm layer.

A common means to improve the safety against base failure in soft clay is to excavate, cast the bottom plate, and reload in shorter sections. If the length of the sections, L_f, becomes smaller than the full width, B, one must remember to use $B_f = L_f$ as an upper limit for determination of N_c.

In Norway the undrained shear strength used is very often based upon in-situ vane shear strengths, s_{uv}, but multiplied by a correction factor, μ. Today one commonly uses a relationship between μ and the normalized shear strength ratio s_{uv}/σ'_{vo} (σ'_{vo} = in-situ vertical effective stress), as given by Aas et al. (1986) rather than the correlation to plasticity index proposed by Bjerrum (1973). The alternative is to use an average value from triaxial compression, CAUC, extension, CAUE and direct simple shear, DSS, as illustrated in Fig. 1. For an excavation which is open for a relatively long time, one should in general check for the effects of pore pressure dissipation and swelling on the undrained shear strengths directly under the base.

For in geologic terms, the undrained shear strength of normally consolidated clays is proportional to the vertical effective stress, σ'_{vo}, i.e. $s_u = \beta \cdot \sigma'_{vo}$. This implies that the in-situ pore pressure condition indirectly is an important factor for excavations in clay. Furthermore, the safety factor will not be so dependent on excavation depth when the shear strength increases with depth.

For excavations in sands and silts below the water table one must ensure sufficient safety against hydraulic uplift, which is governed by the upwards seepage gradient. Acceptable gradients can be achieved by letting the cut-off wall extend to a certain depth below the base. In seepage analyses, it is of great importance to carefully consider variations in horizontal and vertical permeability with depth. Relief wells or pumping wells below the base may be an alternative way to reduce seepage gradients and required toe depth of the walls, especially in layered soils.

3.2 Limiting earth pressures

For clays, and assuming that an undrained response has been shown to be relevant, the limiting active (a) and passive earth (p) pressures can in general be expressed by

$$P_{a/p} = \sigma_v \pm N_{a/p} \cdot s_{u\,a/p} \qquad (2)$$

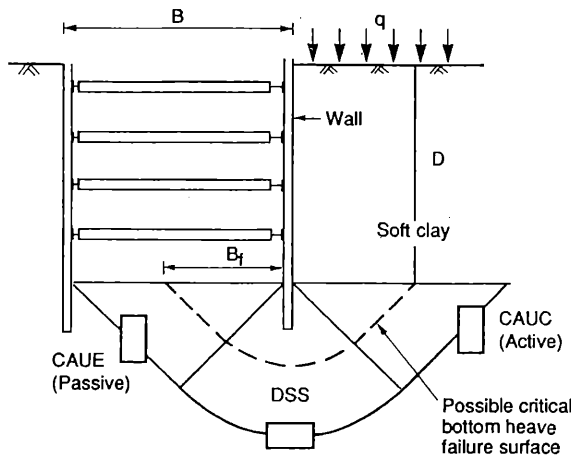

Figure 1. Illustration of base heave failure mechanism.

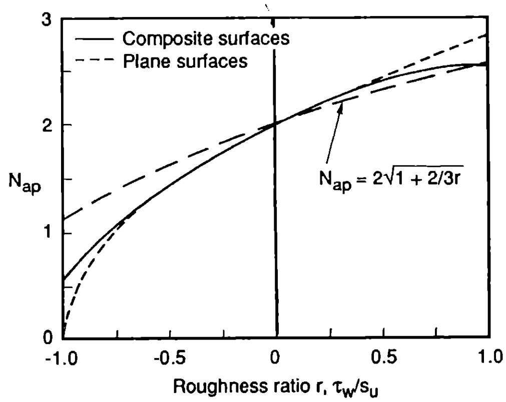

Figure 2. Total stress active and passive earth pressure coefficients (Janbu, 1972).

where σ_v = overburden stress, and $s_{u\,a/p}$ corresponds to undrained strengths from triaxial compression and extension respectively.

$N_{a/p}$ is equal to 2.0 if there is no interface friction between soil and wall. For positive or negative wall friction as expressed by the coefficient, $r = \tau_w/s_u$, the values of $N_{a/p}$ have been given by Janbu (1972) from limiting equilibrium theory with composite type surfaces, Fig. 2, $N_{ap} = \pi/2 \pm 1$ for $r = \pm 1$. The planar wedge case gives higher values when $r \geq 0.5$. The commonly applied approximate expression (Janbu et al., 1966) of $N_{ap} = 2\sqrt{1 + 2/3r}$ is only reasonable for positive values of r. The choice of r on the active and passive sides requires careful consideration of the direction of relative displacements, how much displacements are required to fully mobilize the limiting value, and that vertical equilibrium of the wall is satisfied. The physical upper limit of r may also be less than 1.0, especially for smooth steel sheetpile walls, and will take time to build up when a sheetpile wall is driven into clay.

From Equation 3 one will find that the net resultant earth pressure, p_{net}, below excavation when r = 1, is given by

$$p_{net} = \gamma D + q - N_a s_{ua} - N_p s_{up} = \gamma D + q - 5.14\,\bar{s}_u \qquad (3)$$

Here $\bar{s}u$ is the average of s_{ua} and s_{ap}. One will rapidly recognize that the factor 5.14 corresponds to N_c in Equation 1 for an infinitely long and wide excavation. To be consistent with the bottom heave stability calculation, Aas (1985) and Karslrud (1986) have proposed that the earth pressures below excavation level should also account for 3-D geometry effects. This can be done by multiplying the values of N_a and N_p in Equation 3 by a factor corresponding to $N_c/5.14$.

The implication of this is that when the safety factor against base heave is less than 1.0, p_{net} becomes a net unbalanced inward earth pressure below excavation level. For sheetpile walls driven to firm strata to prevent base failure, one will find that the net resultant pressure distribution, the bending stresses in the wall and the support loads are very sensitive to the choice of r and N_c in the earth pressure calculation.

For soils exhibiting true effective cohesion, c, or attraction a = c · cotg ϕ', (Janbu, 1985), it is most convenient to work with attraction and expressing the limiting effective earth pressure as:

$$p^1_{a/p} = (\sigma'_v + a)k'_{ap} - a \tag{4}$$

In frictional soils, the choice of interface friction, as defined in this case by $\gamma = \tan\delta/\tan\phi'$, where δ is the interface friction angle, has an even more pronounced effect on the limiting earth pressures (Janbu, 1972). It is only for r = 0 that the classical planar Rankine failure wedges are valid, and these will always give an unconservative upper bound when $|r| > 0$. For $|r| > 0$ there exist no undisputable theoretical solutions for other than weightless soil. Common practice in Norway is to use the values of k'_a and k'_p developed for weightless soil (Janbu, 1972). As illustrated for k'_p when $r > 0$ in Fig. 3, this solution is conservative when compared to other published solutions which include soil weight.

It may also be appropriate to treat clay soils as frictional materials when it can be shown that the negative pore pressures set up by the excavations, will more or less fully dissipate during the construction stage. For highly overconsolidated clays one should be very cautious with relying on an undrained total stress analysis and carefully evaluate effects of pore pressure dissipation during excavation.

It should also be possible for effective stress analyses to introduce the positive effects of limited width and length of an excavation on the limiting earth pressures, at least in an approximate manner.

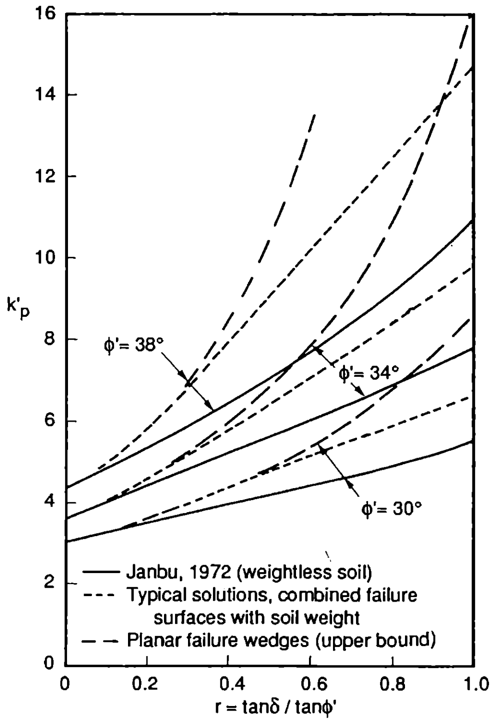

Figure 3. Effective passive earth pressure coefficients (for $|r| \geq 0$).

3.3 Design of support system

Empirical apparent earth pressure diagrams (Peck, 1969), are today primarily used for early estimates or checks against computer analyses. It may be noted that the diagram for soft clays was a result of the unexpectedly large strut loads measured in the Oslo subway works and in accordance with Flaate (1966). The large loads were clearly related to a low factor of safety against base heave, and as a consequence, a net unbalanced earth pressure extending deep below the base of the excavation, as also discussed by Aas (1985) and Karlsrud (1986). Deep seated movements below the base also contribute to an "arching effect", tending to increase earth pressures and strut loads along the upper portions (Bjerrum et al., 1972).

Currently, the most common practice in Norway is to calculate moments in the retaining wall and lateral support loads using finite element programs based on the "beam on Winkler spring" concept. Two such programs have been developed in Norway, "SLISS-SPUNT" (e.g. Bruskeland, 1991) and SPUNT-A2 (SINTEF, 1988). Common features of these programs are that they can model step-wise excavation and installation of support (including preloading), and appropriate adjustment of the limiting active and passive earth pressures (yield limit of soil springs) as excavation proceeds both on a total and effective stress basis. The soil springs are non-linear with a higher stiffness in unloading/reloading than unidirectional loading. This feature may not be so common elsewhere. They also handle external loads in an approximate manner.

Continuum type finite element models, including the soil and all relevant structural components, have been used since the early 70's for excavations in soft clays, primarily for back-calculations (Karlsrud, 1981) or verification/checking of simpler design calculations. As continuum type models are getting more user-friendly and versatile, they will probably find increasing applications in future designs.

For the "Winkler" models there is definitely a need to develop better guidelines for selection of soil spring constants from stress-strain relations. There also seems to be a general misconception that the spring constant is only a material property, whereas in reality it depends strongly on the geometry of the excavation, and to some extent stiffness of the structural system.

3.4 Ground movements

Prediction of ground movements comes directly out of the Winkler and continuum FEM analyses discussed above. The Winkler model only gives wall deflections, but surface settlements can be estimated with reasonable accuracy from that. Note, however, that settlements induced by dewatering and consolidation of the surrounding soils must be added, and can be quite important.

Ground movements can also be fairly well estimated on basis of previous experiences. For excavations in soft clays the bottom heave safety factor has a rather dramatic effect on the ground movements (Flaate, 1966; Peck, 1969; Karlsrud, 1986) as also confirmed by FEM analyses (Mana and Clough, 1981). The maximum surface settlement typically increase from 0,5% of the excavation depth for F $\geq$2.0 to 2.0% as F approaches 1.0. Wall stiffness, support, spacing and preloading are other factors with significant influence on ground movements.

4 DESIGN CODES

The Norwegian Building Codes are based on the Limit State design principles. Only a few key aspects are dealt with below.

For the Ultimate Limit State (ULS), one shall in general divide soil strengths by a material factor, γm, before they are entered into the design calculations. There is no load factor applied to unit weight of the soil or water pressures, but only on other external dead and live loads. These load factors are respectively 1.2 and 1.6.

The soil material factor used in design have in the current Guidelines to the Norwegian Code been specified to vary with consequence of failure, and to what extent one is dealing with strain hardening dilatant soils or strain softening contractive soils. Typically applied values are γm = 1.2 to 1.5.

The use of material factors on soil strengths have, however, been a much debated issue, and can lead to rather unreasonable results. This applies in particular to soft clays and use of continuum finite element models and also to some extent to limiting earth pressures used with Winkler models. In such cases it may be a better approach to use a material factor of γm = 1.0 on soil strengths and instead multiply the calculated loads in the supporting structure by a load factor.

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