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ABSTRACT

The paper discusses major design aspects related to deep excavations in soft clays including; bottom heave stability; deformations and loads on the supporting structure; methods to improve stability and limit displacements; lessons learned from failures; and finally design principles and safety aspects. The various issues are illustrated by both parametric finite element studies and experiences gained from specific case histories. The results show a strong correlation between bottom heave stability, loads and displacements, and significant arching effects when the bottom heave safety factor is low. 2 and 3D FEM analyses confirm the applicability of traditional limit equilibrium bottom heave stability analyses, provided a search for critical failure surface is made and toe penetration of the supporting wall is accounted for. A concept based on using diaphragm wall with cross walls below the base is documented to be particularly effective in improving stability and limiting displacements. Ground improvement by deep mixing or jet-grouting has also been extensively used for this purpose and provides versatile design options. Some lessons learned from failures are highlighted and measures to avoid failures discussed. It is recommended to use continuum type FEM programs for design, but their use require a good understanding of soil models to be used in the analyses. It is observed that soil parameters for use in design are often based on rather poor and rudimentary soil investigations, an issue which it is of prime importance for the geotechnical profession to face up to. When using ULS safety principles in design, the use of factored strengths may lead to unreasonable design loads. FEM based design analyses should therefore be based on using slightly conservative characteristic strength and stiffness values. The resulting characteristic loads in the support structure must then be multiplied with an appropriate load factor to arrive at the design loads.

INTRODUCTION

This paper deals with design of deep excavations in urban areas, and with soft clays extending well below the base of the excavation. For such cases bottom heave stability is a crucial design issue and there is potential for large and damaging deformations to neighbouring buildings and other structures. This paper focuses mainly on five aspects of the problem:

- Reviewing main factors affecting loads and excavation induced deformations, as verified by recent parametric FEM studies and compared to empirical data.
- FEM studies to evaluate classic limit equilibrium methods for assessing bottom heave stability.
- Review of methods that can be used to improve stability and limit deformations illustrated by selected recent case records.
- Review of some failures that have occurred and lessons learned from that.
- Discussion of design approaches and safety principles.

It should be noted that this paper does not attempt to give any comprehensive literature review on the subjects dealt with, but rather focus on conveying the author's personal views and experiences.

FACTORS AFFECTING LOADS AND DEFORMATIONS

Review of some previous work

Karlsrud and Andresen (2005) reviewed past work on factors affecting loads and deformations in connection with deep excavations in soft clays. Their main findings are summarised in the following.

Measurements of strut loads in connection with braced excavations in soft clay for the Oslo subway in the early 1960's revealed that loads could become considerably higher than those predicted by the classical Rankine earth pressure acting over the height of the wall. Flaate (1966) and Flaate and Peck (1972) compared these results to other data from excavations in clay available at the time, including the Oslo and Chicago subways. On that basis, an apparent earth pressure diagram for soft to medium clays was proposed, Fig. 1. This was also included in the 2nd edition of Soil Mechanics in Engineering Practice, (Terzaghi and Peck, 1967). The apparent earth pressure in Fig. 1 is given by:

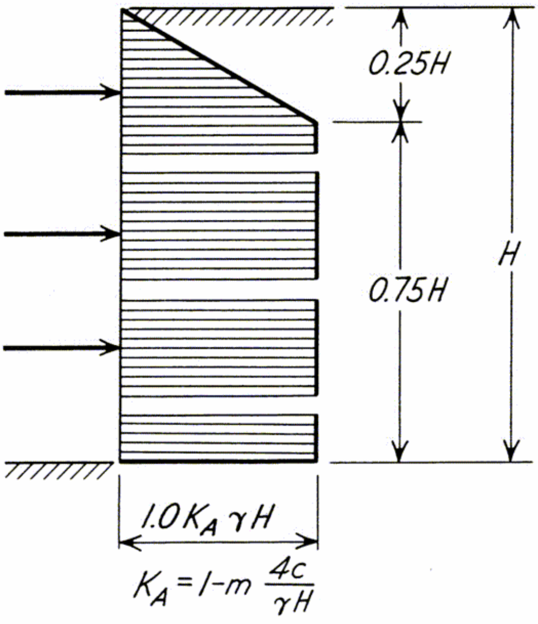

Fig. 1 Apparent earth pressure diagram of Flaate and Peek (1972)

$p_A = K_A\gamma H$, where $K_A = 1\text{-}m4s_u/\gamma H$ (1)

From Fig. 1 one will find that the total horizontal thrust, P_A, to be taken by all the struts is given by:

$P_A = 0.775Hp_A = 0.775(\gamma H^2 - m4s_u/H)$ (2)

This assumes that the distance from the lowest strut to excavation bottom corresponds to 20 % of the excavation depth, and that the load on the lowest strut is computed by multiplying p_A by half the distance to the strut above plus half the distance down to the bottom of the excavation.

Regarding the magnitude of the reduction factor, m, Flaate (1966) stated "*For clays of ordinary sensitivity and stable structure, m=1.0. For Oslo clay, provided $N=\gamma H/s_u$ exceeds about 4 and the depth of the plastic zone is not limited by a firm base, m may be taken as 0.4*". Thus Flaate (1966) implicitly related the unusually high strut loads to both poor bottom stability (high N-value and deep soft layer below base), and to possible strain softening effects (e.g. loss in undrained strength when yielding and large deformations occur). In Terzaghi and Peck (1967) much the same explanation for the m value is given.

According to a classic Rankine active pressure distribution for a smooth wall, the total active thrust over the excavation depth is, for comparison, given by:

$P_{Rankine} = 0.5\ (\gamma H^2 - 4s_uH)$ (3)

s_u in equations (1) to (3) is the average undrained shear strength value over the height, H, of the wall.

Apart from the m-value effect, the total thrust from equation (2) is a factor of 0.775/0.5=1.55 larger than the Rankine value from equation (3). The reason is that equation (2) represents the maximum strut load at any step of the excavation, whereas equation (3) would represent the sum of strut loads only at the final excavation stage and assuming that there is no unbalanced earth pressure below the base.

In the 1950's and 60's, the undrained strength was commonly determined by in-situ vane borings, or based on UC or UU tests on samples we today would probably consider to be of rather poor quality. These types of undrained strengths are probably a factor of 1.2 to 1.5 lower than the true in-situ "active" undrained shear strength, s_{uA}, as can be determined by anisotropically consolidated undrained triaxial compression tests on good quality samples.

Other researchers (e.g. Kjærnsli, 1970; Moore and Ervin, 1975) found it more convenient to compare the total apparent earth pressure to the sum of the vertical overburden pressure against the wall through the expression

$P_A = K_{total}(0.5\ \gamma H^2)$ (4)

For the extreme cases from the Oslo subway with poor bottom stability conditions the factor K_{total} was found to be as high as 1.2-1.3 (Kjærnsli, 1970).

In the 3rd edition of Soil Mechanics in Engineering Practice (Terzaghi, Peck and Mesri, 1995), the expression for p_A in equation (1) was replaced by:

$p_A = (1\text{-}4s_u/\gamma H + \Delta K)\ \gamma H$ (5)

The factor ΔK was related to bottom heave stability as proposed by Henkel (1971) and given by the expression:

$\Delta K = 2B/H(1\text{-}5.14s_{ub}/\gamma H)$ (6)

Here s_{ub} is the representative undrained shear strength for the clay in the zone below the base involved in the bottom heave failure mechanism. This strength should ideally be represented by the average of the "active" undrained triaxial compression strength, s_{uA}, the "passive" undrained triaxial extension strength, s_{uP}, and the "horizontal" direct simple shear strength, s_{uD}.

The apparent earth pressure coefficient K_A in equation (1) must not be confused with the local horizontal limiting active undrained earth pressure coefficient k_a defined as:

$\sigma_{ha} = \sigma_v - k_a s_{uA}$ (7)

where σ_v is the vertical overburden pressure and s_{ua} the "active" undrained shear strength as may be determined by anisotropically consolidated triaxial compression tests.

For the embedded part of the wall underneath the bottom of the excavation the limiting passive pressure is correspondingly given by:

$\sigma_{hp} = \sigma_v - k_p s_{uP}$ (8)

The earth pressure coefficients k_a and k_p depend on the direction and relative magnitude of clay/wall interface shear stress defined by the ratio $r= \tau_w/s_u$, and the assumed shape of the failure surface, see for instance Janbu (1972). For a positive r =0.5, k_a and k_p are approximately equal to $2(1+2/3r)^{0.5} = 2.31$.

Aas (1984) and Karlsrud and Aas (1995) proposed that the net earth pressure below excavation level should also account for non-planar failure surfaces and 2- and 3D effects in a similar fashion as the bottom heave stability number. This reasoning leads to a net earth limiting pressure below the bottom of the excavation of:

$$\sigma_{net}= \sigma_{ha} -\sigma_{hp} = \sigma_v + q - fN_c s_u \quad (9)$$

Here N_c is the bottom heave stability number according to Bjerrum and Eide (1956) discussed further in Section 3 of this paper.

Equation (9) implies that the limiting active and passive earth pressure coefficients are equal to $k_a=k_p= 0.5fN_c$. For positive r=0.5 the factor f has been estimated to be about f=0.9 (Aas, 1984).

As discussed in a fairly comprehensive review by Bjerrum et al. (1972), arching effects have, since the 1930's, been recognized to influence the earth pressure and bending moments in sheet pile walls or other flexible earth retaining structures. Bjerrum et al. (1972) partly contributed the high strut loads observed for some of the Oslo subway excavations to such arching effects. Earth pressure measurements made on the sheet pile wall for the Vaterland 1 excavation in Oslo (e.g. Norwegian Geotechnical Institute (NGI), 1962) verified the existence of such arching effects. Bjerrun et al (1972) also demonstrated arching effects by FEM analyses, and on that basis they strongly advocated that using continuum finite element methods was the only way one could realistically predict the complex state of deformations around a braced excavation and give a realistic picture of the earth pressures, strut loads and bending moments. There is every reason to wonder why it seems to have taken 25-30 years after this statement was made before use of such programs have come into common practice.

As mentioned above it was a weakness of the earliest case records that the undrained shear strengths were determined from what we today consider rudimentary strength testing (UU, UCT, fallcone) on rather poor quality samples or based on in-situ vane borings. Bjerrum et al. (1972) therefore reinterpreted results from earlier instrumented excavations in soft clay by establishing their best estimates of the true in-situ s_{uA} and s_{uP} values. They further calculated from equation (7) and (8) the net total resultant earth pressure acting down to a depth where measured lateral displacements were close to zero, assuming no interface friction (e.g. $k_a = k_p = 2$), and found that this total resultant pressure generally agreed well with the total sum of strut loads measured at a given stage of excavation.

Through the 1970's and 80's FEM programs were primarily used as a research tool, but gradually also became applied in design (e.g. Karlsrud. 1981).

Clough et al. 1979; Mana and Clough, 1981; Clough et al, 1989 confirmed the significant importance of the bottom heave safety factor and the depth of soft clay below the excavation on the deformations that occurred, and which was found to be in very good agreement with measurements. The semi empirical procedure for predicting displacements proposed by Clough et al (1989) and Clough and O'Rourke (1990) also accounts for the effect of stiffness of the support system and the soil. In relation to observed displacements Long (2001) has also made a valuable summary of 240 case records grouped into different ground conditions and wall embedment conditions.

The referenced FEM studies by Clough and co-workers also showed some examples of calculated earth pressures against the walls analyzed and the existence of arching effects, but they did not systemize their results.

Hashash and Whittle (2002) presented some FEM studies, using the non-linear anisotropic MIT-E3 effective stress model for clays to analyze an internally strutted diaphragm wall. Their results confirmed the potentially very significant effect of arching on the earth pressure distribution. Karlsrud (1997) showed, by a FEM study of the earth pressures against a wall pre-strutted prior to any excavation, an even more extreme case of arching effects, where the earth pressures against the lower supported part of the wall actually approached a passive condition. The effect of arching was also clearly demonstrated by Mortensen & Andresen (2003).

Recent parametric finite element studies

Fig. 2 shows the geometry and the main material parameters considered in the parametric study by Karlsrud and Andresen (2005). The soil profile comprises a layer of fill and dry crust down to 5 m depth, and soft clay below that. Initial pore pressures were taken as hydrostatic below a groundwater table at 2 m depth. The excavation width was 16 m and maximum depth 10 m. The sheet pile wall (SPW) is braced internally with four strut levels at depths 1 m, 3.5 m, 6 m and 8 m. The excavation was performed sequentially in five steps to depths 0.5 m below the struts with successive installation of the struts.

Table 1 summarise all cases analysed. The depth of the sheet pile wall D_W, depth to firm layer D_F, and the undrained shear strength s_u, of the soft clay were varied. In addition one case (D1) considered the effect of pre-stressing the struts. The analyses were performed with the PLAXIS 8.2 (PLAXIS, 2004) FEM program.

The fill and dry crust were modelled as a drained linear elastic - perfectly plastic material using the PLAXIS Mohr-Coulomb model with friction angle $\phi' = 30^o$, Poisson's ratio $\nu = 0.3$ and with shear modulus G = 10 MPa. Full friction was assumed between the SPW and the soil.

The soft clay from 5 m depth was for all cases modelled using the ANISOFT constitutive model (Andresen and Jostad, 2002) implemented as a user defined model in PLAXIS.

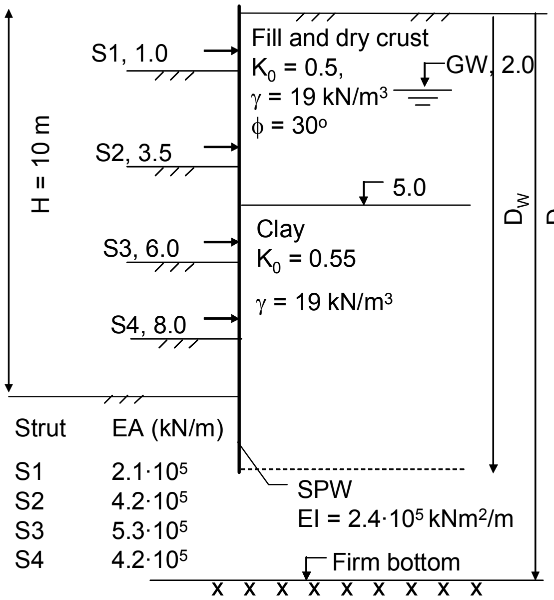

Fig. 2 Illustration of parametric FEM parametric study (from Karlsrud and Andresen, 2005)

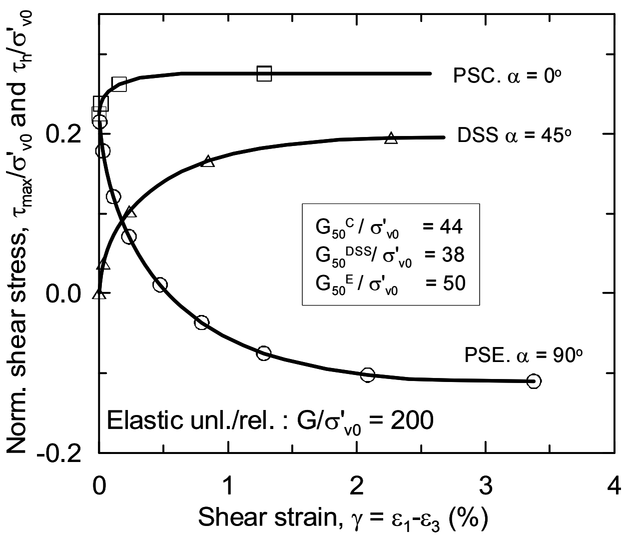

Fig. 3 ANISOFT stress – strain relationship for the base case clay (from Karlsrud and Andresen, 2005)

The ANISOFT model accounts for the non-linear and anisotropic stress-strain relationship of clays using a framework of incremental hardening elastoplasticity and an anisotropic Tresca yield criterion. Unloading/reloading is modelled as being elastic with stiffness given by G_0.

The undrained shear strength of the clay was assumed proportional to the in-situ vertical effective stress. The shear strength for a horizontal shear plane was taken as a typical simple shear strength for normally consolidated clays, and corresponding to DSS strength s_{uD}/σ'_{v0} = 0.2 for all cases except A2 and A3, where this strength ratio was taken as 0.24 and 0.28. The "active" compression and "passive" extension shear strengths were taken as a factor of respectively 1.43 and 0.57 times the simple shear strength. Fig. 3 illustrates the normalised stress strain relations used for the base case. (Note that in Fig. 3 PSC stands for plane strain compression and PSE plane strain extension). The failure strains were taken as γ_f^C = 0.75 %, γ_f^{DSS} = 3 % and γ_f^E = 3.5 % and the initial tangent and loading/unloading modulus was taken as G_0/σ'_{v0} = 200 for all cases. The assumed K_0 = 0.55.

Interface elements with a shear strength corresponding to that of the natural ground were used to model the thin shearing zone between the SPW and the soil. The struts were only allowed to carry a compressive force.

Fig. 4 presents calculated bending moments and horizontal displacements for the final stage of excavation for most cases. It is apparent that the moments and displacements for the 12 m deep SPW (Cases A, C and D) are dominated by cantilever movement, with a maximum positive moment occurring at strut level 3. For cases B with a deeper sheet pile wall, there is a net unbalanced earth pressure to some depth below the bottom, and a net supporting earth pressure at larger depth. For these cases the maximum bending moment occurs at a level close to the bottom of the excavation.

Fig. 5 presents earth pressure distributions at the final stage of excavation for most cases analyzed. They are, as a reference, compared to classical Rankine earth pressures for a smooth wall, and the in-situ total lateral horizontal stress, $\square_{h0}$. These pressure distributions show a pronounced "arching effect". Along the top 4 to 6 m the earth pressures are almost twice the Rankine pressure, and well above the in-situ vertical total stress. Below final excavation level the earth pressures are significantly lower than the classical Rankine pressure for a smooth wall (k_a= k_p=2.0). The minimum values are close to what was calculated by the Aas (1984) approach, equation (7), which for this case gives k_a= k_p=decreasing from 3.38 at the bottom of the excavation to 2.82 at 30 m depth.

Table 1 summarizes the main results of the different cases including normalised maximum strut loads, bending moments and horizontal displacements at any stage of excavation down to the final depth of 10 m.

Table 1- Overview parametric FEM study

	A1	A2	A3	B1	B2	B3	C1	C2	D1
S_{uD}/σ_{v0}'	0.20	0.24	0.28	0.20	0.20	0.20	0.20	0.20	0.20
D_W, m	12	12	12	15	20	20	12	12	12
D_F , m	20	20	20	15	20	30	15	30	20
K_{total}	1.60	0.97	0.76	1.03	1.21	1.28	1.46	1.60	1.77
M_{max}, kNm/m	793	323	133	594	631	568	729	807	681
δ_{hmax}/H (%)	1.88	0.85	0.51	0.49	0.79	0.94	1.70	1.89	1.73

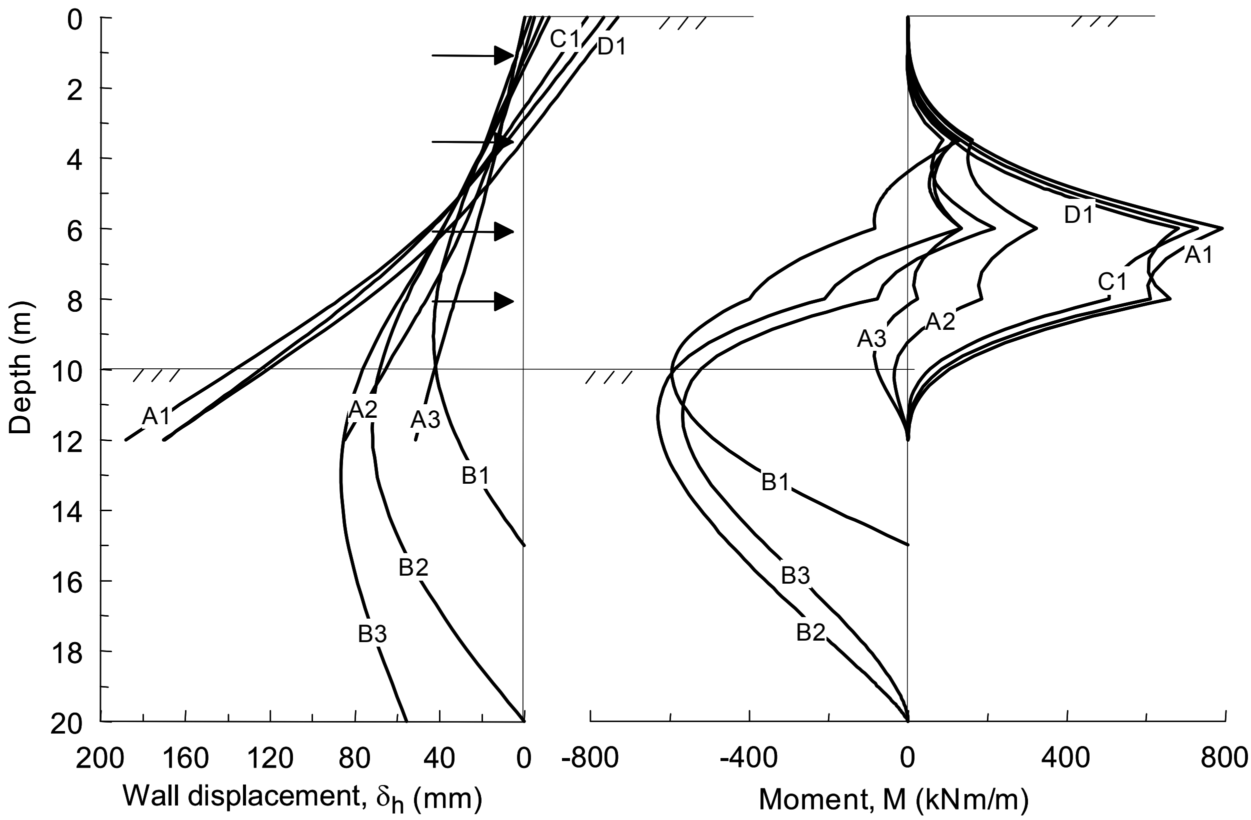

Fig. 4 Calculated wall displacements and bending moments (from Karlsrud and Andresen, 2005)

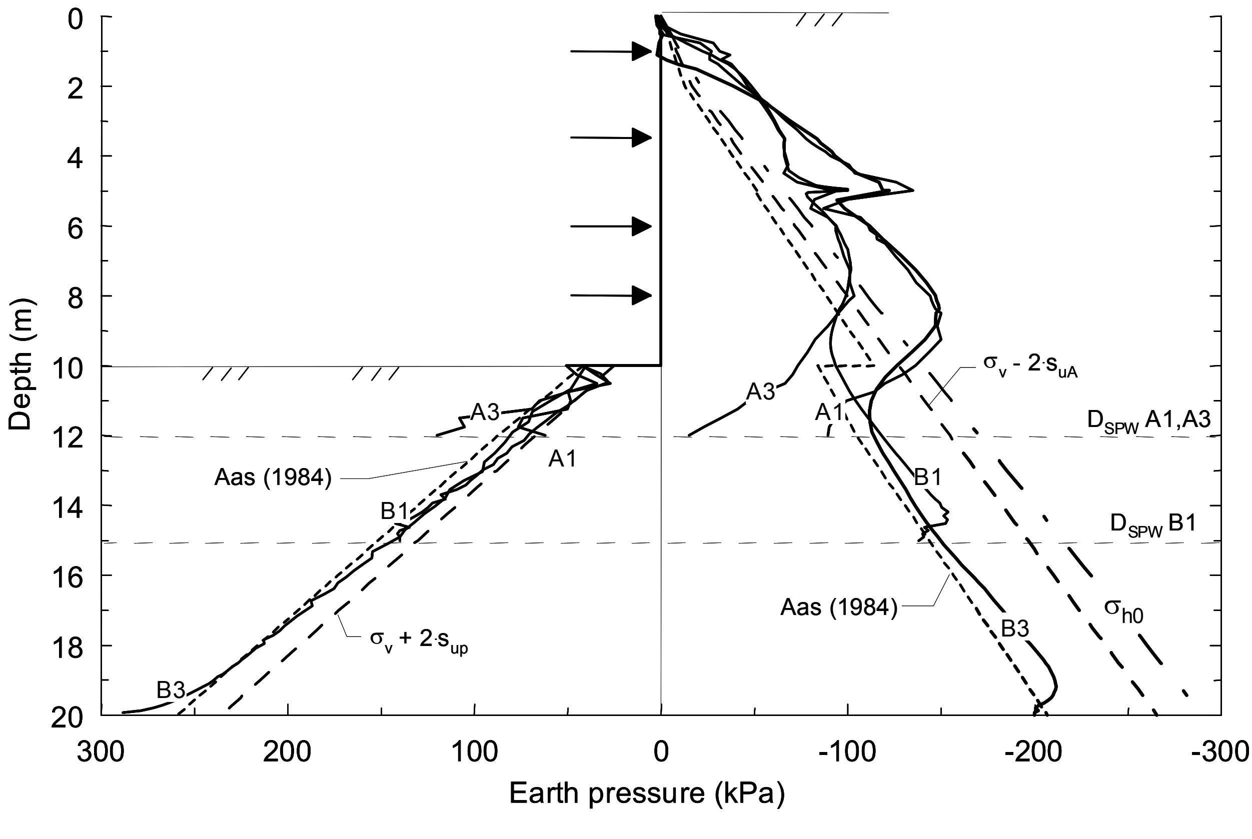

Fig. 5 Total horizontal earth pressures for selected cases (from Karlsrud and Andresen, 2005)

The results from all excavation stages, apart from the first excavation stage to the top strut level, have been normalised in a similar manner as for the final excavation stage. For all these stages the normalised loads and displacements have been correlated to an "apparent" bottom heave stability safety factor, $F_{ba,}$ at the given stages of excavation, as defined by:

$$F_{ba} = N_c s_{ub}/(\gamma H) \qquad (10)$$

Note that the effects of the embedded part of the sheet pile wall and the moment capacity of the sheet pile wall on the bottom heave stability were neglected. Therefore F_{ba} does not represent the true safety factor for the excavation, and should only be considered as a correlation parameter having a major impact on loads and deformations as shown in the following.

The bottom heave stability number Nc in equation (10) was taken from the diagram by Janbu et al (1966) which is a slightly modified version of Bjerrum and Eide's (1956) diagram. N_c was determined for the actual depth (varying for the different excavation stages) and the actual width of B=16 m of the excavation. In accordance with Janbu et al (1966) the undrained strength s_{ub} in equation (10) was taken as the average over a depth corresponding to 2B/3 below the excavation. The average of $s_{uA,}$ s_{uD} and s_{uP} was used as basis.
Figs. 6, 7 and 8 show that the apparent bottom heave safety factor, F_{ba}, has a large effect on the maximum wall displacement, bending moment and strut loads. Some key observations can be made from these figures:

- Wall displacements and bending moments increase very strongly when F_{ba} drops below 1.4-1.5.
- Wall displacements increase from about 0.25 % to about 2 % of the height H, or about 8 times, when F_{ba} decreases from 2.0 to 1.0. This is in close agreement with the results reported by Mana and Clough (1981). The B-cases, where the wall is particularly deep or is fixed at bedrock level, naturally give the smallest displacements and less effect of F_{ba}.
- The maximum bending moment in the wall also increases about 8 times when the safety factor F_{ba} decrease from 2.0 to 1.0, but the B-cases fall in this case more in line with the rest.
- The apparent total strut load coefficient K_{total} increases from about 0.7 to 1.60, or by a factor of 2.1, when F_{ba} decrease from 2.0 to 1.0.
- For all results it can be observed that loads and displacements increase with the depth to firm bottom, but when the depth to firm bottom exceeds about twice the excavation depth there is little or no further effect.

The largest calculated K_{total} of 1.88 in Fig. 8 is even larger than the largest measured value of about 1.3 derived from strut load measured in the Oslo subway referred to earlier (e.g.Kjærnsli, 1970). The smallest calculated K_{total} value of 0.49 is more in line with the lowest measured values from cases with relatively good stability conditions and/or limited depth to firm bottom.

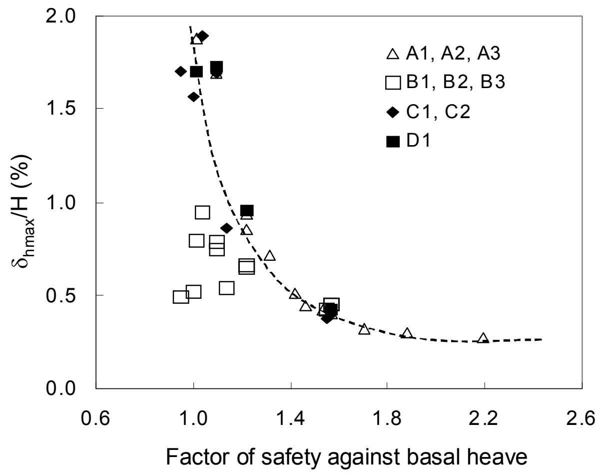

Fig. 6 Normalised maximum wall displacement, δhmax/H,

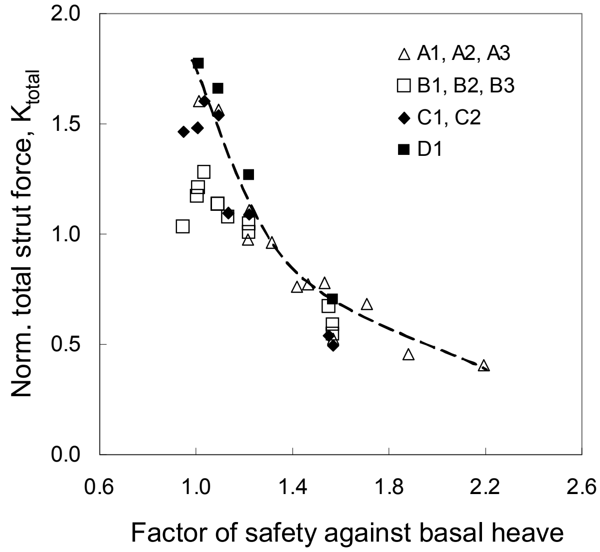

Fig. 7 Maximum bending moment, M_{max}, versus apparent bottom heave safely factor, F_{ba}

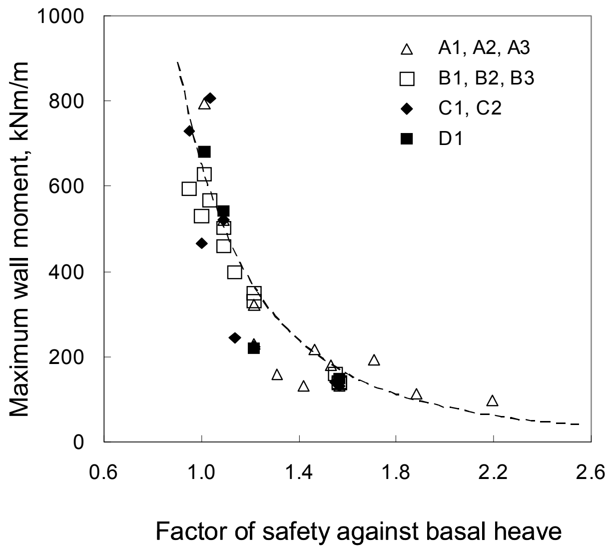

Fig. 8 Normalised maximum strut load coefficient, Ktotal, versus apparent bottom leave safely feaster, Fba

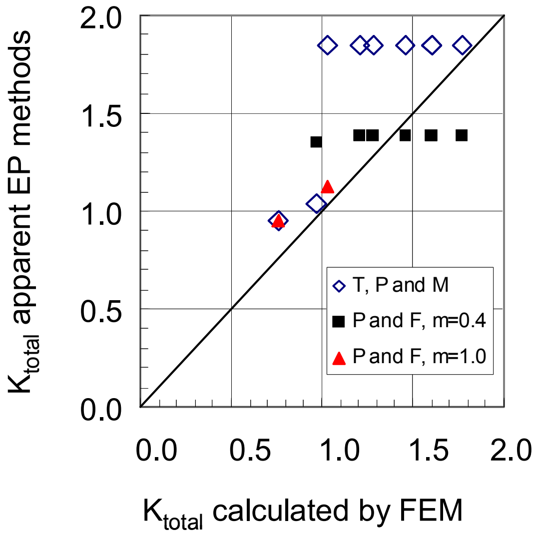

Fig. 9 Comparison between K_{total} values from FEM and empirical methods

Fig. 9 compares K_{total} calculated by the FEM analyses against the K_{total} values based on the empirical procedures of Flaate (1966), equation (2), and Terzaghi, Peck and Mesri (1995), equations (5) and (6). For the Flaate (1966) method m=1.0 was assumed relevant for Case A3 with the highest strength and for Case B1 with the wall fixed to bedrock at 15 m depth. For all other cases m=0.4 was assumed. The s_{uD} strength at 5 m depth was used as basis. It appears that the Flaate method tends to underpredict the cases with largest K_{total} from the FEM analyses (A1, C2 and D1), and overpredict the others. The Terzaghi, Peck and Mesri (1995) method seems on the conservative side for all cases. The FEM analyses by Hashash and Whittle (2002) also showed that the apparent strut loads could become significantly higher than that given by equation (2) with m=0.4.

The effect of penetration depth of the SPW can be seen by comparing cases A1, B1, B2 and B3, but these cases to some extent also reflect the effect of taking the SPW down to a firm base. Case B1, with $D_F = D_W = 15$ m, naturally gives the smallest bending moments and strut loads, corresponding to a factor of 1.33 and 1.55 lower moments and strut loads than for case A1.

Case D1 is the same as Case A1 with the exception that the struts were pre-stressed to a level corresponding to about 75% of the maximum loads. As expected, this increased the strut loads, but only by 10%. The maximum bending moment and displacement reduce only slightly. This relatively minor effect of pre-stressing is believed to be due to the critical stability condition for cases A1 and D1, and that the forces for these cases are mostly driven by the deep-seated soil movements. For walls extending to bedrock, for better bottom heave stability conditions and for stiffer walls, pre-stressing of struts or tie- back anchors can give a much larger relative increase of bending moments and strut loads, and reduce displacements more significantly.

Effect of soil model

Karlsrud and Andresen (2005) also analysed Cases A1 to A3 with the isotropic linearly elastic – perfectly plastic MC model available in the PLAXIS program. To adopt the MC model to the anisotropic and non-linear undrained shear strength parameters used with the ANISOFT model, some approximations were made: the undrained shear strength was taken equal to the s_{uD} strength, and the MC constant shear modulus was taken to be half the value of the initial tangent stiffness of the ANISOFT model, e.g. $G/\sigma'_{v0} = 100$.

The rather interesting and somewhat surprising result that came out for these cases was that bending moments and strut loads calculated with this MC-approximation were within 10% of what was calculated with the ANISOFT model. The reason for the small difference is that poor stability conditions and the progressive development of yield zones below the excavation bottom impacts the stresses more than the detailed stress-strain relation of the clay. For cases with better stability conditions the effect of soil model will in the authors' experience be more significant.

Comparison to beam-on-spring approach

Many practicing engineers use beam-on-spring type finite element models for the design of braced or anchored excavations, rather than continuum FEM models as used above.

To study possible errors involved with the use of a simpler beam-on-spring approach, Karlsrud and Andresen (2005) also analysed cases A1 and B2 using the finite element program SPUNT-A3 (Kavli et al, 1999). The program models the soil spring reactions on the front and back of the wall by a set of different springs. Each spring is non-linear according to a hyperbolic formulation up to yield level. The yield level can be specified such that it corresponds to the limiting active or passive earth pressure at any stage of excavation.

In the analyses the spring stiffnesses were selected so that 50% mobilization would correspond to the approximate formulation proposed by Karlsrud (1999). Compared to the PLAXIS-ANISOFT results, the strut loads from SPUNT-A3 were on average smaller by a factor 1.3 for Case A1 and 1.20 for Case B2. In terms of maximum bending moments, the SPUNT-A3 results were a factor of 1.54 to 2.16 lower than the PLAXIS-ANISOFT results. The maximum displacements differed even more by factors of about 3. These results show that beam-on spring models are not well suited for analysing deep excavations in soft clays with large depths to firm bottom and/or poor bottom stability conditions.

DETERMINATION OF BOTTOM HEAVE STABILITY

The two most frequently quoted methods for analysing bottom heave stability has traditionally been the limit equilibrium approach based on Terzaghi (1943), and Bjerrum and Eide (1956). The latter actually stems from Skempton (1951) and is based on the similarity between a bottom heave failure and

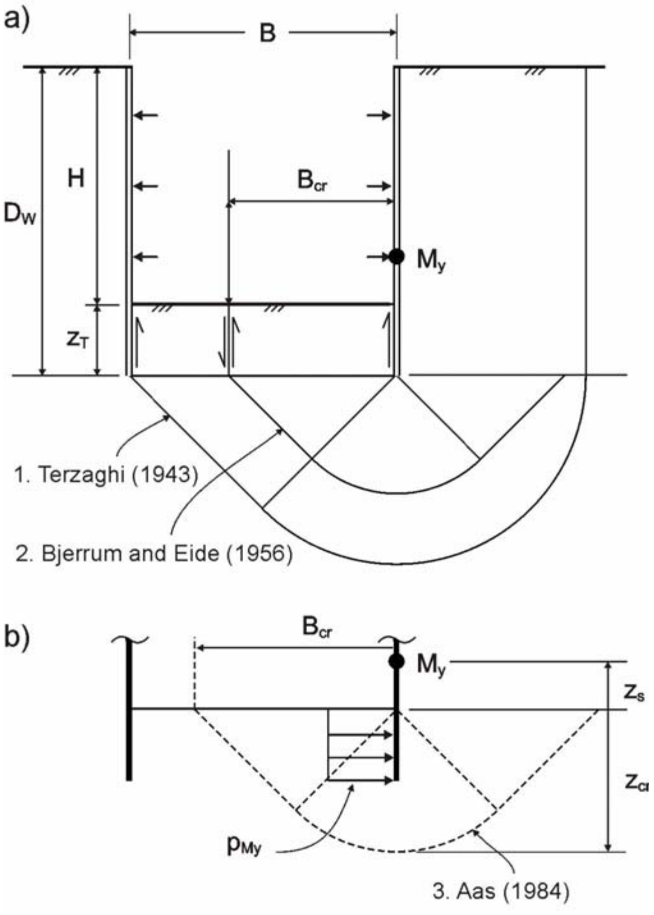

Fig. 10 Illustration of different bottom leave failure mechanism

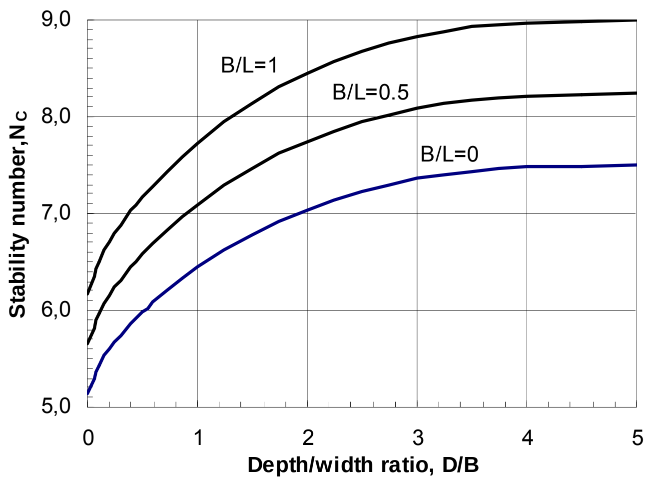

Fig. 11 Bottom heave stability chart from Janbu et al (1966) based on the Bjerrum and Eide (1956) approach

the bearing capacity of a deeply embedded footing. The Bjerrum and Eide (1956) approach was calibrated against observed bottom heave failures. Fig. 10 defines the assumed failure surfaces implied by the two approaches and the geometric dimensions used in the formulas as described in the following.

Equation (11) is based on Terzaghi (1943), but the shear strength terms are rearranged so that they are all in the numerator. Thereby the safety factor corresponds to the material safety factor as in Ultimate Limit State (ULS) design principles. The second term in the numerator accounts for the resistance along the vertical failure surface over the height. The third term accounts for the effect of toe penetration of a rigid wall to a depth z_T below the excavation bottom. In his text Terzaghi (1943) stated that this effect should be accounted for, but it was not included in the original formulae. Terzaghi (1943) also stated that if there is limited depth to firm strata, the width B of the excavation shall be replaced by the dimension $\sqrt{2}(D_F-D_W)$.

$$F = (5.7s_{ub} + \sqrt{2}s_{uDw}D_W/B + 2suz_T/B)/(\gamma H+q) \qquad (11)$$

s_{ub} = average strength in bottom heave failure zone below the tip of the wall
s_{uDw}= average strength over height D_w of the wall
s_{uT} = average strength over toe depth z_T of the assumed rigid wall, and accounting for possible strength reduction at wall/ clay interface.

The original Terzaghi (1943) equation was based on a uniform strength profile. For a non-uniform strength profile it would be reasonable to assume that it is appropriate to use the average strength representative for the different failure zones, and as defined in equation (11).

Equation (12a) present a modified version of the original Bjerrum and Eide relation, as it has been practiced at NGI over the past 20 years. Figure 11 shows the chart representing N_c in relation to depth, width, and length of an excavation as given in Janbu et al (1966). This is a slightly modified version of the original by Bjerrum and Eide (1956) chart. The form of the bottom heave failure surface shown in Fig. 10 was not shown in the original paper, but has in NGI's practice been assumed to correspond to the classical Prandtl failure surface for a smooth strip footing.

The effect of rigid wall penetration is similar to the second strength term in equation (11) and follows the concept in Eide et al (1972). The last term in the denominator in equation (12b) accounts for the yield moment capacity, M_y, of a flexible wall similar to the suggestion by Aas (1989). Aas (1984) and Karlsrud (1986, 1997) have further pointed out that the full width B of the excavation does not necessarily represent the critical width of the failure surface inside the excavation. In general it is necessary to search for a critical width, B_{cr} as will be illustrated later. The full width B is the critical one only for a case of constant undrained strength with depth,

$$F = (N_c s_{ub} + 2s_{uT}\, z_T/B_{cr})\,/(\gamma H+q) \qquad (12a)$$

$$F = 0.94N_c s_{ub}\,/(\gamma H+q-p_{My}(z_T\,/z_{cr}\,)) \qquad (12b)$$

$$p_{My} = (2M_Y - \sigma_{ha}z_S^2\,)/(z_T^{\,2} + 2z_S z_T)$$

σ_{ha} = average horizontal earth pressure on supported side from lowest strut to bottom of excavation.

Several studies have in the past addressed the merits of the two approaches. Chang (2000) introduced the effect of length of the excavation on the Terzaghi (1943) approach and carried out some comparative case studies for a uniform shear strength profile. Compared to the Bjerrum and Eide (1966) approach he found for B/L ranging from 0 to 1.0 that the Terzaghi (1943) method gave safety factors that were about 30 to 40 % larger than for the Bjerrum and Eide (1956) approach. Chang (2000) also developed some further modifications to Terzaghi's approach and also some new failure models that fit better to the Bjerrum and Eide approach.

Ukritchon and Whittle (2003) also compare the two methods and discuss various modifications over the years. For an infinite long case with uniform shear strength and a wall with z_T=0 they found that the Terzaghi approach gave 11 % higher safety factor than the Bjerrum and Eide approach for a very narrow excavation, increasing to 35 % for a very wide excavation.

FEM analyses should give a more precise answer to bottom heave stability. Some new parametric FEM studies have therefore been carried out with the PLAXIS (2004) program to determine the bottom heave safety factor. The results have been compared to the Terzaghi (1943) and Bjerrum and Eide (1956) approaches. For the selected cases it was assumed an isotropic shear strength profile corre sponding to normally consolidated clay with s_{uD}/σ'_{v0}=0.24 as for Case A2 in Table 1. The excavation depth was for all cases taken as H=10 m, with 4 strutting levels as defined in Fig. 2. As shown in Table 2 the width of the excavation, B, toe depth of the wall, D_w, and depth to firm bottom, D_F, were varied. For cases 1 to 6 the sheet pile wall has infinite moment capacity, and for cases 7 and 8 it is assumed to have plastic moment capacity, M_y, of respectively 600 and 900 kNm/m.

Table 2- Parametric study of bottom heave

Case	B (m)	D_W (m)	D_F (m)	PLAX F	Bj.-Eide F	Terz. F
1	4	10	30	1.14	1.11	1.17
2	16	10	30	1.12	1.11	1.23 (1.12)
3	4	14	30	1.92	1.81	1.99
4	16	14	30	1.63	1.63	1.63 (1.61)
5	16	10	16	1.12	1.11	1.12
6	16	14	16	2.28	2.17	2.32
7[1)]	16	14	30	1.26	1.23	-
8[2)]	16	14	30	1.32	1.30	-

1) For M_y = 600 kNm/m
2) For M_y = 900 kNm/m

The results show that equations (12a) and (12b) based on the Bjerrum and Eide approach give safety factors that in general are slightly on the low side (0 to 6 %) of the PLAXIS results. Fig. 12 shows the relationship between the assumed width of the failure surface inside the excavation and the safety factor for some selected Bjerrum and Eide cases. The critical width is small for Cases 1 and 2 with the sheet pile just extending to the bottom of the excavation (D_w= 10m), which is why these

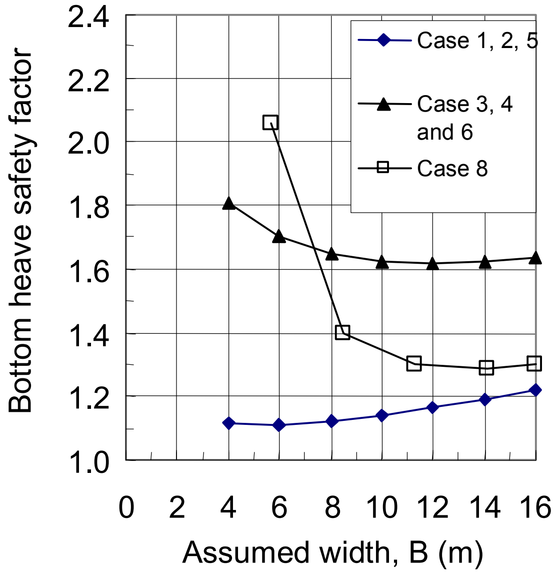

Fig. 12 Influence of assumed failure width, B, on bottom heave safely factor

cases show identical safety factors. The PLAXIS analyses nicely confirm these results. For case 4 the critical width is about 10 m, and larger than B=4 m as for Case 3. That is why Case 3 gives higher safety factor than Case 4. It is especially comforting to see the close agreement with PLAXIS for the rather extreme geometry of Case 6, with only 2 m clay below the tip of the wall.

For cases 1 to 6 the original Terzaghi approach gives safety factors from 0 to 10 % on the high side of the PLAXIS results. If for the Terzaghi cases it is also searched for a critical width, the minimum safety factors become as given in parenthesis in Table 2. This reduces the overshoot with the Terzaghi approach to 0 to 5.5 %. Shirlaw (2005) stated that FEM analyses tend to confirm that the Terzaghi (1943) approach gives reasonable results, but did not give any documentation.

The results for Cases 7 and 8 show that equation (12b) gives a reasonable representation of the effect of moment capacity of the wall. It can be mentioned that O'Rourke (1993) has proposed a different approach for accounting for the moment capacity of the wall based on the elastic strain energy of a bending wall.

Figure 13 shows an example of incremental strain and failure model from the PLAXIS analysis for Case 4 with B= 16 m, D_W=14 and D_F= 20 m. It can be observed that the failure surface below the toe level of the rigid wall closely resemble the classical Prandtl failure surface for a deep embedded footing. Furthermore, there is no distinct failure surface extending to the ground surface. That agrees with the basic assumptions of the Bjerrum and Eide approach, and contradicts the vertical shear surfaces assumed in the Terzaghi (1942) approach, Fig. 10.

The 3D-version of PLAXIS has also been used to lo analyse a few selected cases as summarised in table 3. The general assumptions are as for Case 4 in table 2, but the length of the excavation is varied.

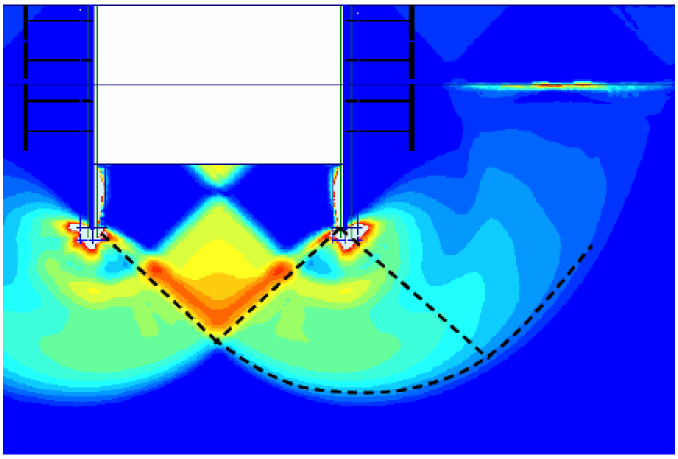

Fig.13 Example of bottom heave failure mechanism based on incremental strain at failure as calculated by PLAXIS for Case 4.

Table 3- 3D bottom heave case study

Case	B (m)	L (m)	B/L	PLX	Bj.&Eide
4	16	∞	0	1.63	1.63
4-1	16	32	0.5	1.80	1.78
4-2	16	16	1	1.99	1.95

Table 3 also shows excellent agreement between the Plaxis results and the Bjerrum & Eide approach.

MEASURES TO IMPROVE STABILITY AND LIMIT DISPLACEMENTS

Excavating in sections

Excavating to final grade and casting the bottom slab in sections, is in general the first and most obvious option considered for improving bottom heave safety, and thus, also reducing loads and displacements. The bottom slab may in addition be loaded or anchored down before the next section is excavated. If the section length is selected to correspond to the critical width of the excavation, the safety factor will according to Fig.11 be enhanced by a factor of 1.2. The potential impact on loads and displacements can be even larger, ref. Figs. 6 to 8.

Deeply embedded high capacity wall

Today it is possible to establish walls with very high moment capacity. The heaviest double HZ sheetpile section available on the market in Europe has for instance a design moment capacity of the order 3000 kNm/m, which is about the same as for a 1.4 m thick diaphragm wall. It is also possible to make

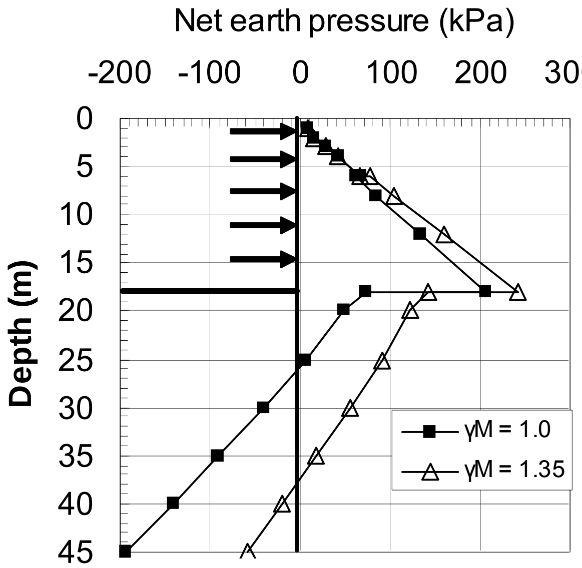

Fig. 14 Example of net apparent earth pressure distribution for H = 18 m (shear strength as in Fig. 25)

Table 4- Example of calculated required toe depth and bending moment for 18 m deep excavation in NC Oslo clay

Material factor	Required toe depth (m)	Maximum moment (kNm/m)
1.0	25	2360
1.35	47.8	4700

T-shaped diaphragm wall panels with even larger capacity, but there are some practical limitations due to problems with lifting and installing long and heavy reinforcement cages.

Even for deep excavations with soft normally consolidated clays extending to very large depth, there will normally be a depth at which there is a net resultant earth pressure. Fig. 14 shows, as an example, the net earth pressure distribution for a case of an 18 m deep excavation in normally consolidated clay with unit weight of 18.5 kN/m3 and shear strength as shown in a later Fig. 25. The earth pressures below the base were computed in accordance with equation (9), using a material factor on the undrained strength of both γ_M= 1.0 and 1.35.

Based on equilibrium analyses the required toe depth and the maximum bending moment in the wall below excavation level are for these two cases as given in Table 4.

Note that in the equilibrium analyses the lowest strut was assumed located 3 m above the base (Fig. 14) and the SPW was given a moment capacity of M_y = 3000 kNm/m. The results show very large impact of the material factor on the earth pressures, required toe depth and maximum bending moment. The significance of that aspect will be discussed in Section 6 of this paper.

If there is firm bottom at a higher level, the toe resistance must be ensured by penetration of the wall into the firm strata. If the firm stratum is bedrock, rock dowels will be needed.

For the case as considered in Fig. 14 there is probably a practical upper limit to the excavation depth around 18 m (that is, without any other measures to improve stability). The disadvantage of using a deeply embedded wall as the only measure to achieve stability is the potentially relatively large displacements. Even a high capacity wall will deform significantly if it shall carry an unbalanced pressure down to 10 - 20 m below the excavation bottom, especially when the toe is only supported in soft clay. A deep wall is therefore often combined with a sectional excavation approach to enhance stability and limit loads and displacements.

Underwater excavation and base slab construction

With this concept excavation and strutting is normally carried out in the dry to the maximum depth allowed from a bottom stability point of view. Then excavation under water is carried out to the desired depth and a base slab tremie concreted under water. The base slab must after emptying transfer the expected vertical uplift forces required to give sufficient bottom heave stability. Depending on the specific case this may require that the slab is anchored into the walls and/or that the slab is loaded down or tied down by anchors or piles into deeper firm strata before dewatering. It is in this connection also necessary to verify that there is sufficient stability against lifting of the entire structure. If there are permeable layers within close proximity of the excavation bottom, hydraulic stability as well as classical undrained bottom heave stability must be ensured. The underwater slab also provides lateral support for the wall, and due to its early construction it can contribute significantly to limit displacements.

One of the larger applications of this principle was for construction of the Marina Bay station and adjoining tunnels for the Singapore Metro as described by Denman al (1987) and Shirlaw et al (2005). The excavation was 1100 m long and 18-20 m deep. The ground conditions were recent fill over the soft normally consolidated Singapore marine clay which at the site extended up to 45 m depth. The bottom slab was in this case tied down by bored piles extending into deeper firm layers. The design was reported to function as intended.

Another more recent example was for a locally deep excavation needed to construct part of the basement structure for the new Oslo Opera, located at the waterfront in Oslo harbour. The deep part of the excavation shall house the stage machinery and lifting systems for the Opera. The excavation was designed by NGI under direction of the first author, and included some new elements as described in the following.

The excavation was designed as a 38 m diameter cylindrical shaft using AZ18 type sheet piles to create the perimeter wall. Half of the excavation stretched into the harbour, where land had to be reclaimed as part of the works. Fig. 15 shows a typical cross section from the harbour and onto land.

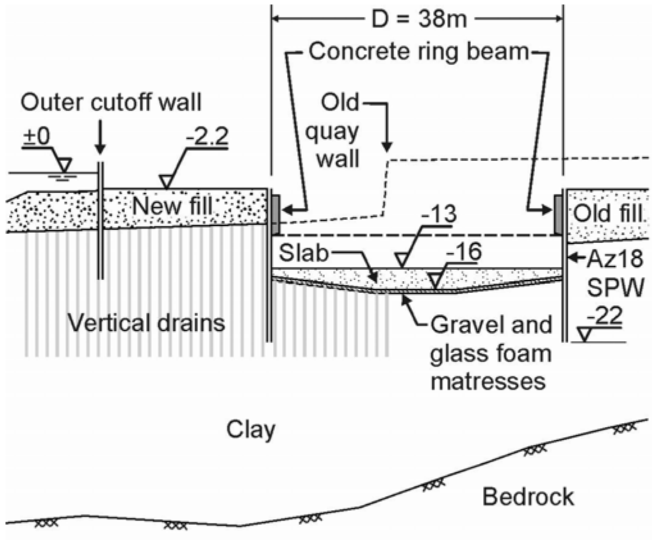

Fig. 15 Cross section through cylindrical excavation for Oslo Opera

The water depth was typically 8 m. By means of an outer perimeter cut-off sheet pile wall around the entire Opera construction site, the infilling level was limited to Elev. -2.2, and land on the inside was also excavated to this level prior to the construction of the cylindrical excavation pit. The bottom of the centre of the excavation lies at Elev. -16. The sheet pile wall was designed to extend to Elev.-22. The un-reinforced bottom slab was given a conical shape to optimise its capacity to take and transfer uplift forces to the sheet pile walls.

The silty clay in the area has water content in the range 35 to 45 % and plasticity index, Ip, of 15-20 %. Vertical drains were installed prior to filling in the sea in order to achieve sufficient stability during filling as well as during the subsequent excavation. The undrained shear strength of the sea side clay after filling and consolidation was determined from direct simple shear tests and correspond to $s_{uD}/\sigma'_{ac} = 0.24$, where σ'_{ac} is the axial consolidation stress applied in the test. Fig. 16 shows that the clay under the old fill on the land side is significantly stronger than under the new fill because it has consolidated under a larger vertical effective stress. Undrained triaxial compression and extension strengths were estimated from the empirical correlations shown in Karlsrud et al (2005) to be a factor 1.3 and 0.7 times the DSS strengths, respectively.

Excavation inside the cellular cofferdam was first carried out stepwise in the dry to Elev. -8 m, with 3 concrete ring beams cast sequentially to support the wall against the lateral earth pressures, Fig. 15. Then excavation under water to Elev.-16 was undertaken. Divers were used to clean the sheet pile walls in the contact zone between the sheet pile wall and the slab. Prior to concreting of the bottom slab, a 30 cm thick gravel drainage layer was first placed at the excavated bottom and then a 20 cm thick compressible layer of glass foam mattresses. The purpose of the glass foam mattresses was to allow some vertical deformations in the clay below the slab and thus, to limit the bottom heave uplift pressures acting on the bottom slab.

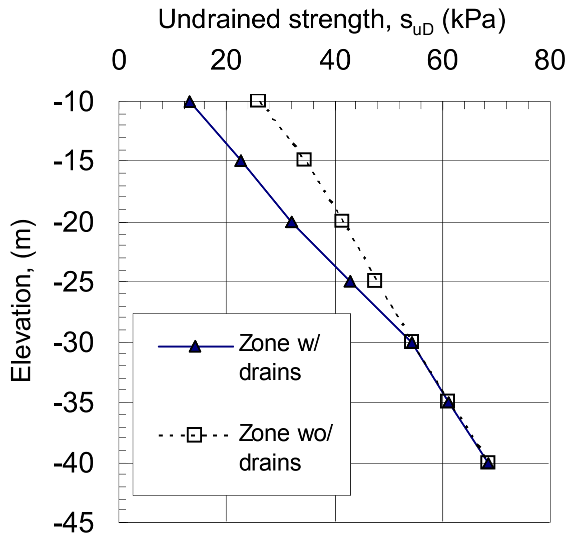

Fig. 16 Characteristic undrained shear strength profiles at Oslo Opera site

All load transfer was based on interface friction between the wall and the slab. Without such a compressible layer, the slab would initially be subjected to a pressure almost corresponding to the full water pressure of 140 kPa, which would have required a heavily reinforced slab with shear connectors to the perimeter wall and/or that the slab was anchored down into the underlying bedrock.

The design analyses were carried out with an equivalent plane strain FEM model that could capture the difference in soil properties on the sea and land sides, respectively. To account for 3D effects the undrained strengths below the base were increased by a factor of 1.2 corresponding to the difference in bottom heave stability number for an axi-symmetric and a plane strain condition. The calculated net bottom heave uplift pressure on to the slab after emptying the water was on average 22 kPa (including a load factor of 1.6), giving an interface shear force at the sheet pile wall of 322 kN/m. The calculated characteristic lateral load from the sheet pile wall to be transferred to the bottom slab was at this stage 1360 kN/m (based on the FEM analyses), giving a design interface friction coefficient of 0.26, which was considered fully acceptable.

Sampling and CPTU cone penetration tests had shown that the clay at this site was homogenous almost all the way to bedrock at around Elev. -35 to -45, and with no sand or silt layers apart from a few very thin seams (less than a centimetre or so). Therefore hydraulic stability was not an issue. The base slab was designed to be permanently drained to avoid the permanent uplift water pressures of about 170 kPa. The drainage layer under the base functions as a storage reservoir in the permanent situation. The amount of water coming into the drainage layer and which needs to be permanently pumped has been measured to about 1 l/min, which means that the storage capacity of the drainage layer will last for about 2 months in the case of pump failure.

Construction of the cofferdam was successfully completed in 2005. During the final dewatering stage there were no measurable vertical movements of the bottom slab. Inclinometer measurements showed a maximum horizontal displacement at end of construction of 12.3 cm on the harbour side and 7.8 cm on the land side. For comparison the FEM design analyses carried out in advance predicted a maximum displacement of 11.2 and 8.4 cm respectively, which is within 8-10% of what was measured.

<u>Excavation under air pressure</u>

Excavation under an excess air pressure was commonly used in soft ground tunnelling up until say the late 1970's. In relation to deep excavations in soft clays it was used on a relatively large scale in connection with the construction of 770 m length of the Oslo subway in the early 1960's (Bjerrum et al, 1966). It was combined with top-down construction, in which excavation was first carried out for a roof plate at about 5 m depth, with backfilling on the roof before excavation under an excess air pressure of typically 1 atmosphere.

The method is considered rather slow due to the need for sluicing all equipment and excavated material in and out. Health issues and strong limitations on allowable working time and needs for decompression and rest periods, have also contributed to set a practical stop for this method.

<u>Diaphragm cross-wall concept</u>

Eide et al (1972) developed a concept for improving bottom heave stability and limit displacements in deep excavations based on using diaphragm walls to act as cross-walls below the base level (hereafter referred to as DCW-concept). The following describes some experiences with the design and performance of the concept in connection with four different projects in Scandinavia.

<u>Rail and subway way tunnel Oslo 1973-75</u>

The DCW-concept was first developed for constructing a 240 m long section of a double decked subway and railway tunnel through Studenterlunden in central Oslo, Norway. The ground conditions consisted of some fill and weathered clay crust followed by soft normally consolidated Oslo clay with water content 35-45 % and plasticity index I_P from 10 to 30 %.
Fig. 17 shows a cross section of the adopted solution. The excavation depth was H=15.5 m, the width B=10.5 m, the toe penetration of the walls z_T= 4.5 m, and the wall thickness t= 1.0 m. The cross-walls had the dual benefit of improving bottom heave stability and acting as a pre-installed bottom strut, which in itself has a significant impact on minimising lateral displacements during excavation. The main construction steps for this tunnel were as follows:

1. Installation of the 1 m thick and 4.5 m wide longitudinal diaphragm wall panels to 21 m depth, 5.5 m below the base of the excavation. In this case the longitudinal diaphragm walls also formed the permanent tunnel walls.
2. Installation of the cross-wall diaphragm wall panels to 4.5 m depth below the base. These cross-walls were placed at mid point of each longitudinal wall panel, e.g. at spacing of 4.5 m. The cross-wall panels were cast with full grade concrete to form a

bottom support below the excavation and a temporary support just beneath the middle deck in the tunnel. The rest of the panels were backfilled with very low-grade concrete or crushed rock as described in more detail by Karlsrud (1975).

3. Excavation and construction of the roof deck just below the ground surface and backfilling over the roof plate, but keeping a construction opening through the roof deck at one end of the tunnel
4. Excavation for the upper tunnel and construction of the middle deck. The middle deck was tied into the longitudinal tunnel walls by bending out reinforcement bars that were part of the reinforcement cage for the longitudinal wall panels, but protected on the outside by 100 mm Styrofoam.
5. Excavation of the lower tunnel and construction of the bottom deck.

Eide et al (1972) developed the following modified bottom heave stability expression for a long excavation which account for both the effects of side friction and tip resistance against the cross walls:

$$F= (N_c s_{ub}+ N_{sf} s_{uw} + N_t s_{ut})/(\gamma H+q), \text{ with} \tag{13}$$

$N_{sf} = 2(B+l)z_T/(B(l+t)$,side friction component
$N_t = 7.5TB/B(l+t)$, tip resistance component
B = width of excavation
z_T= Depth of cross-walls below the base
t= thickness of cross-walls
l= free space between cross walls
s_{ub} = average undrained strength within depth 2/3B below the tip of the walls
s_{uw} = average wall/clay interface strength over the height, h, of the cross walls
s_{ut} = undrained strength at the tip of the cross walls

The actual tunnel was successfully constructed between 1973 and 1975 as documented by Karlsrud (1975 and 1981). Fig. 17 shows a maximum lateral displacement of only 43 mm, which occurred just below the bottom of the cross-walls. This displacement corresponds to only 0.3 % of the excavation depth, which at the time was quite remarkable for such a deep excavation in soft NC clay. The displacement of 32 mm at the level of the cross-walls is larger than the 0.3 mm or so that was expected to arise from ideal compression of the concrete in the cross-walls. The reason lies in imperfect cleaning of the interface between the longitudinal walls and the cross walls and possibly, some imperfections in the panel joints. The ground settlements were about twice the lateral displacements, which was due to pore pressure reduction at bedrock level causing some consolidation settlements in the lower part of the clay deposit. The pore pressure reduction was caused by drainage effects originating where the tunnel base reached into permeable bottom moraine overlying bedrock at each end of the tunnel.

<u>Rail tunnel Oslo 1973-75</u>

The DCW concept was similarly and successfully used to construct the Jernbanetorget part of the Oslo city rail tunnel (Karlsrud, 1981). In that case the distance between the tunnel

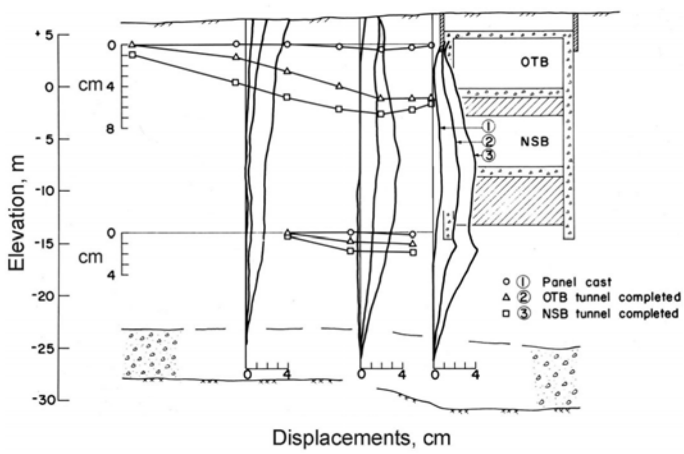

Fig. 17 DCW concept and measured displacements, Studenterlunden (after Karlsrud, 1981

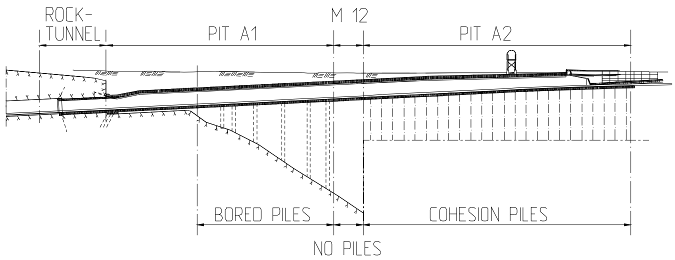

Fig. 18 Longitudial profile Lilla Bommen tunnel, Gothenburg (after Karlsrud et al, 2005)

walls was larger than in Studenterlunden, from 12 to 20 m, but the excavation depth was smaller, from 8 to 12 m. The maximum lateral displacement of the tunnel wall was here limited to 20 mm.

<u>Road tunnel Gothenburg 2001-2004</u>

To the authors knowledge the next application of the DCW concept was to construct a 230 m long part of the 6 lane Lilla Bommen road tunnel in Gothenburg during the period 2002-2005. Karlsrud et al (2005) have described the design and performance of the temporary excavation support in detail, but some main issues and observations will be repeated and discussed herein. The maximum excavation depth was 17 m below present ground surface at the western end where the tunnel leaves bedrock, and gradually came up to 8 m depth at the eastern end, where the tunnel changes to an open culvert, Fig. 18. The required permanent tunnel box structure had dimensions of 35-40 m in width and 8-9 m in height.

The ground conditions consist of old fill material to a depth of about 6 m followed by a homogenous normally consolidated soft plastic clay deposit extending practically all the way to bedrock The depth to bedrock increases progressively eastwards (Fig. 18), and is about 60 m at the mid point. The soft marine clay below the fill has a water content of 60 to 80 % and liquid limit of 45 to 67 %.

A main reason for selecting the DCW concept for this project was in addition to the poor stability conditions the closeness to existing buildings and critical infrastructure like tramlines. The owner therefore required that lateral wall displacements and settlements outside the excavation should be limited to 20 mm in the most sensitive areas.

Fig. 19 compares the isotropic undrained strength profile the owner had required that this design and build contract should be based upon, to the "best estimate" of the in-situ undrained strength originally proposed by the first author based on the various strength data made available at tender by the owner. This "best estimate" represents an average strength profile (average of triaxial compression, extension and DSS strength). The "best estimate" is 30-40 % larger than that given by the owner for use in design. This implies that there could be a corresponding additional inherent safety factor when the owner's undrained strength profile was used for design. The owner otherwise required that a minimum material or safety factor of 1.5 should be applied in the design.

The internal bottom heave stability was analysed on basis of equation (13). To satisfy a safety factor of 1.5 a cross-wall spacing of 4.5 m was selected, which required that the longitudinal walls and the cross- walls had to extend up to 7 m below the base of the excavation where the tunnel depth was 16 m and the depth to bedrock about 30m. At the shallowest end, a cross-wall depth of 4.0 m was required. The support system otherwise consisted of a concrete capping beam on top of the longitudinal walls and a single level of pipe struts near ground surface, Fig. 20. With the large width of the excavation, and a shear strength increasing with depth, it was necessary to search for the critical depth or width of the bottom heave failure surface as described in Section 3. For this case the critical width inside the excavation was typically
8-10 m.

In addition to analysing the potential for bottom heave inside the excavation, it was necessary to check the possibility of the entire support structure being pushed up. For an infinite long excavation case this case turned out to be critical. An acceptable safety factor was achieved by excavating the deepest 3 m for construction of the bottom slab in 12m long sections.

As described by Karlsrud et al (2005) design earth pressures and support loads for the longitudinal walls were primarily calculated by a beam-on-spring type FEM program. The design moments and forces were according to the owners requirements taken from the worst of two cases: 1) Without any material factors applied to the soil strength, but the calculated moments and other support forces that came out of the analyses were multiplied by a load factor of 1.2. 2) A safety factor of 1.5 was applied to the strengths, but no load factor was applied to these results. Due to the large stiffness of the support system, the analyses showed that the horizontal displacements of the walls were so small that the earth pressures were close to the assumed in-situ at-rest conditions. In such a context the use of case 2) makes little sense, as will be discussed further in Section 6 of this paper.

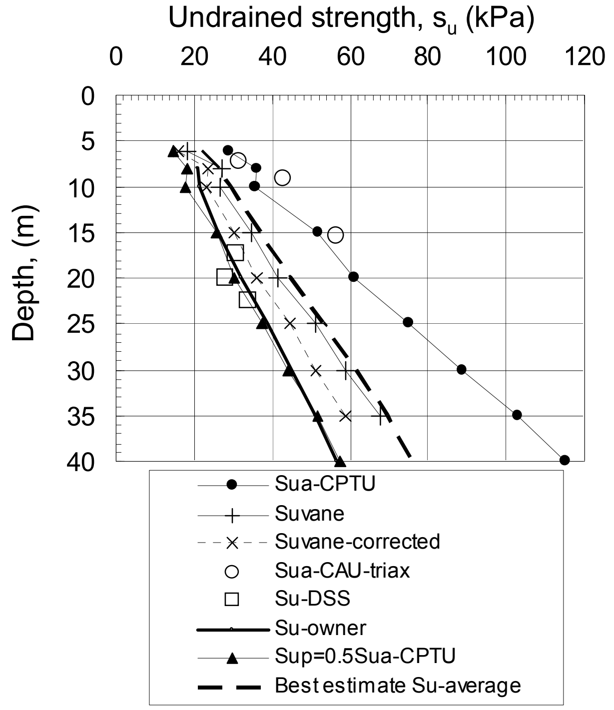

Fig. 19 Undrained shear strength profiles Lilla Bommen tunnel (after Karlsrud et al, 2005)

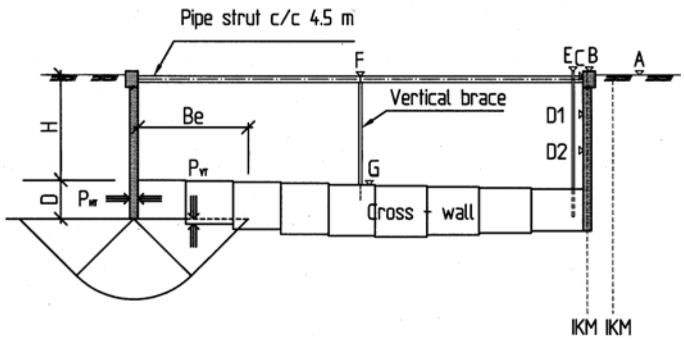

Fig. 20 Typical cross section Lilla Bommen tunnel (after Karlsrud et al, 2005)

An important design element, considering the rather slender un-reinforced cross walls, was to verify that the cross-walls could withstand the combination of axial (horizontal) loads and vertical loads they would be subjected to. Because the shear force between the cross-walls and the longitudinal walls that were to be taken purely as friction concrete to concrete, it was necessary to get a reasonable upper and lower bound to the ratio between total uplift force and horizontal load acting on the cross walls.

To get an upper bound to the design forces, different safety factors or load factors had to be considered for different parts of the analyses. For instance, assumptions that could give the lowest horizontal force in combination with the largest vertical uplift load would give the worst condition in terms of mobilised interface friction, and possibly also the worst stress conditions at the mid span of the cross-walls. In terms of contact stresses and the potential for local overstressing of the cross walls, the potentially largest horizontal force and bending moment at the connection was of prime interest.

The uplift force on the cross walls was determined by calculating what vertical pressure, p_v, had to be applied at the base of the cross-walls to provide a certain safety against bottom heave. Fig. 21 shows that the total uplift force on a cross-wall depends strongly on the safety factor or the degree to which the undrained strength is assumed mobilised, but it is limited upwards to a value corresponding to full mobilisation of tip and side shear resistance. The design value of the uplift force was selected for a value of F=1.5. This gave a force corresponding to 44% of the theoretical maximum value. The owner required that the friction at the interface between cross-walls and longitudinal walls should not exceed 0.49. That requirement was always fulfilled with the method described above to determine the uplift force.

During the design process there was seen a need to verify the fairly simple design analyses outlined above by continuum finite element analyses that could better capture the interaction between structure and soil. The computer program PLAXIS (2001) was used for that purpose. One challenge in this modelling was how to simulate the effects of the cross walls in an equivalent 2-dimensional model. Fig. 22 shows the equivalent 2-D PLAXIS model used to analyse a typical cross section. In this model the cross walls were replaced by equivalent internal longitudinal walls that have the same height, thickness and spacing as the actual cross-walls.

The equivalent cross-walls were connected to the outer walls through an imaginary steel truss with the same axial and bending stiffness across the excavation as the actual cross-walls.Thus, it could capture all potential deformations of the structural system. The model was further used in combination with the "best estimate" of the undrained strength (Fig. 19), and using the anisotropic non-linear stress-strain model ANISOFT, Andresen and Jostad (2002). Fig. 23 shows the normalised stress-strain curves selected for use. The curves were partly established from the undrained triaxial compression tests from the site.

For the longitudinal walls two restraint conditions were considered in the analyses: 1) Free vertical movement throughout all excavation stages 2) Vertical movements were locked when 3 m of excavation remained to simulate the sectional excavation effects. Table 5 compares key results for these two cases. The restraint condition had largest impact on the uplift force on the cross-walls. The uplift forces correspond to respectively 25 % and 17 % of the theoretical maximum value, and a mobilised interface friction against the longitudinal walls which is well below that calculated using the bottom heave approach. The mobilised friction was also well below the acceptance level of 0.49

The maximum bending moments calculated with the PLAXIS model were quite similar to what came out of the simpler beam-on-spring model used for design, but the horizontal load in the cross-walls was about 30 % larger. The explanation lies in the relatively high earth pressures predicted with PLAXIS against the lowest part of the wall.

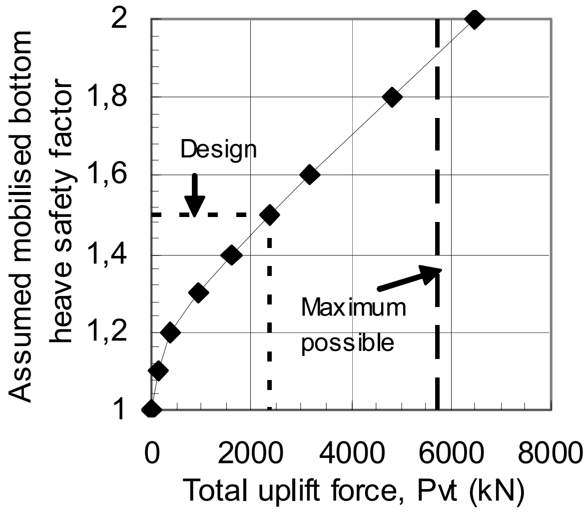

Fig. 21 Relationship between net uplift force on cross-wall and applied "safely factor", Lilla Bommen tunnel (after Karlsrud et al, 2005)

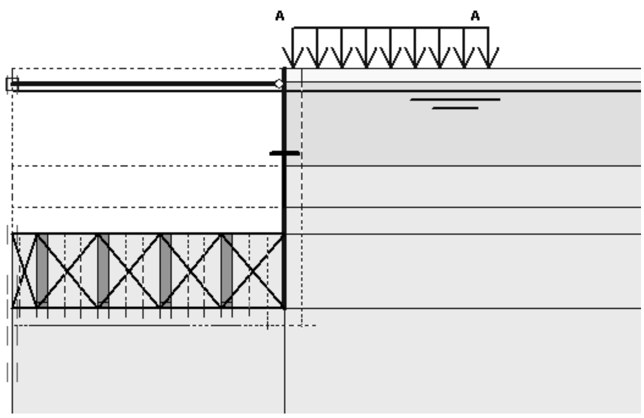

Fig. 22 PLAXIS model used, Lilla Bommen tunnel (after Karlsrud et al, 2005)

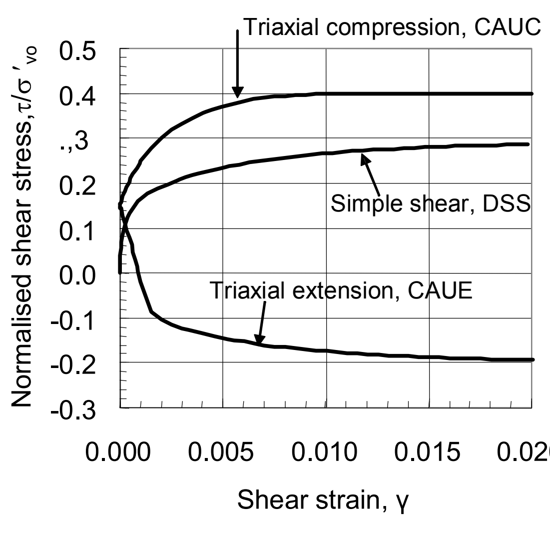

Fig. 23 Normalised stress-strain model used in PLAXIS-anesoft analysis (after Karlsrud et al, 2005)

Table 5. Summary of PLAXIS results at km 2.770 (from Karlsrud et al, 2005)

		Case 1)	Case 2)
Max.support moment	(kNm/m)	1010	1280
Max. field moment	(kNm/m)	915	795
Horizontal load cross-wall	(kN)	8470	8290
Vertical interface shear force	(kN)	2080	2980
Vertical disp. LW	(mm)	19	14
Vertical disp. mid span CW	(mm)	25	23
Max. horizontal disp. LW	(mm)	6.8	7.3

This was due to the tendency for soil to move underneath the walls and up into the excavation in combination with small lateral wall displacements at the toe. This is an arching effect which, as discussed earlier, can not be predicted with beam-on-spring type models.

As described by Karlsrud et al (2005), during excavation for the M12 cofferdam (Fig. 18) it was discovered that there remained up to 20 cm of clay in the joint between the longitudinal wall and the cross walls at some locations. Up until then the joint had been mechanically cleaned before casting of the cross wall panel in contact with the longitudinal wall. The contractor had to remedy these imperfections by core drilling and grouting. He later changed his procedure and attached a double steel plate to the reinforcement cage at the joint location. The outer steel plate was pulled off when the adjoining cross wall panel was excavated. In this way the cross-wall panel was cast against a clean steel plate. This solution gave essentially perfect contact.

Fig. 24 shows very close agreement between predicted and measured horizontal displacement of the longitudinal walls. The maximum measured horizontal displacements at the end of excavation ranged from only 15 to 30 mm, and occurred at a level a few meters below the base of the walls. It may be noted that the analyses were carried out well in advance of the excavation works. Thus in accordance with the definitions of Lambe (1973), this can be considered a “Class A” prediction. These observations otherwise suggest that the “best estimate” of the clay strength and stiffness were close to reality.

When the final excavation level was reached, the points B and G in Fig. 20 showed heave in the range of 20 to 40 mm. This was on the high side of the PLAXIS calculations (Table 4), which may be explained by some swelling of the clay due to the excavation, which was not accounted for in the PLAXIS analysis. The relative vertical movement between cross-walls and the longitudinal walls (heave of point C minus point E in Fig. 20) nowhere exceeded 5 mm, which confirms that there was good contact between the longitudinal walls and the cross walls, and that friction in the joint could be taken up as intended. The relative upward movement between the mid point F of the cross-walls and point B on the longitudinal wall was up to 15 mm. This is about twice that predicted with the PLAXIS, Table 4, which could be due to some local overstressing or slight imperfections in the joints between cross-wall panels.

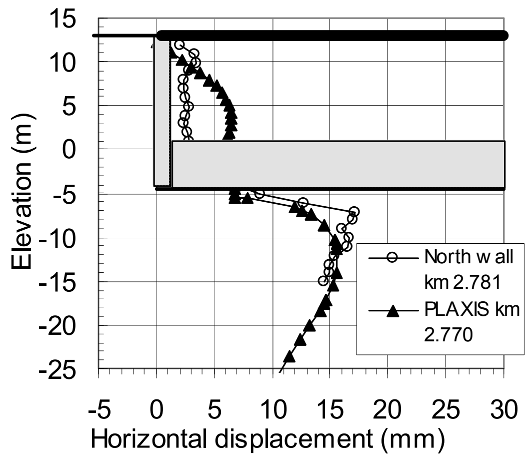

Fig. 24 Predicted and measured horizontal displacement, Lilla Bommen tunnel (after Karlsrud et al, 2005)

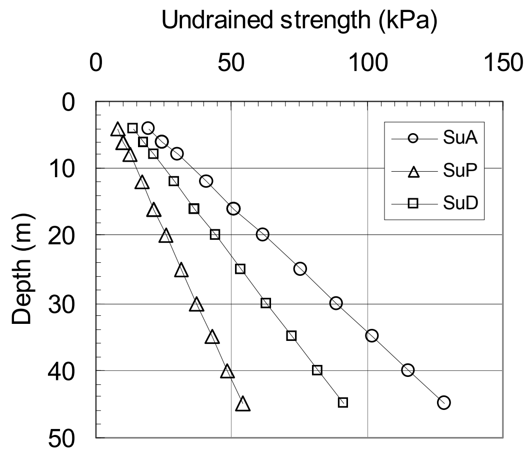

Fig. 25 Characteristic undrained shear strength profiles at Sørenga,Oslo

<u>Road tunnel Oslo, 2006-2007</u>

A new submerged 6 lane road tunnel is presently under construction in the Oslo harbour. It requires a 400 m long cut-and-cover tunnel over land at Sørenga to connect further into an existing rock tunnel. A 228 m long part of this cut-and-cover tunnel was designed with the DCW concept, and the construction was just completed in the fall of 2007. The design was developed by the Norwegian Geotechnical Institute (NGI) in cooperation with Aas Jakobsen AS as structural engineers. The client is the Oslo Road Administration. The main contractor has beenthe Norwegian AF Group. Zueblin AG, Germany was sub-contracted by them to undertake the diaphragm wall installation.

The ground conditions at the site consist typically of 2-4 m of fill and clayey, sandy silts, followed by normally consolidated clay. The soft clay extends to bedrock at a depth of 35 to 50 m below present ground. It has a water content of 35-45 % and plasticity index of 15-25 %. Fig. 25 shows the undrained shear strength profiles used. They were partly established from triaxial tests and partly from CPTU type cone penetration tests using cone factors as recommended by Karlsrud et al (2005).

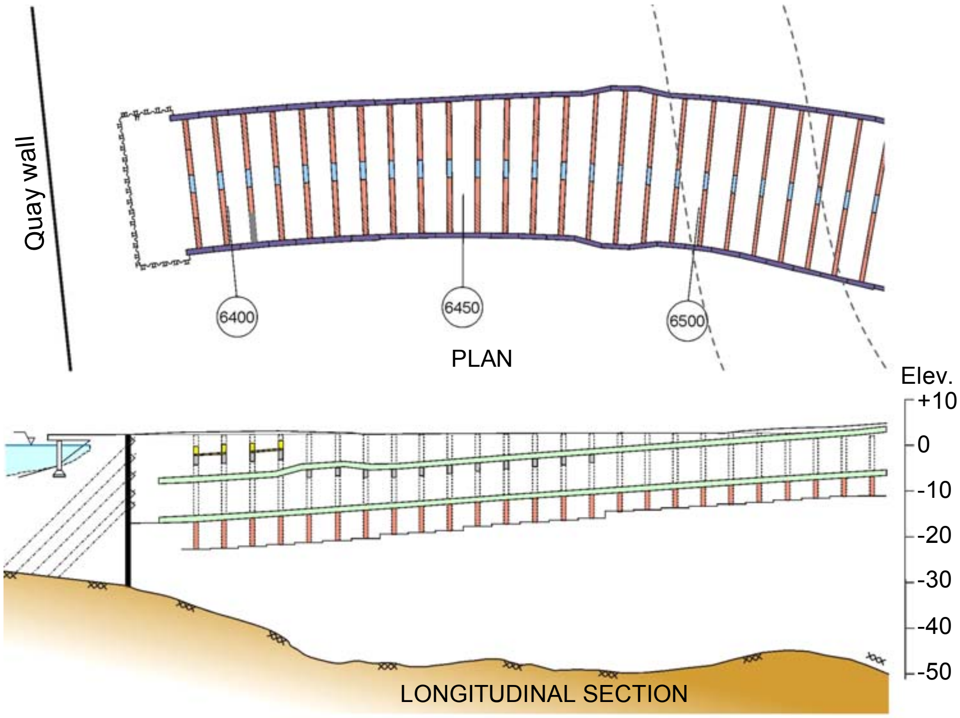

Fig. 26 Plan and longitudinal profile along deepest part of the Sørenga tunnel

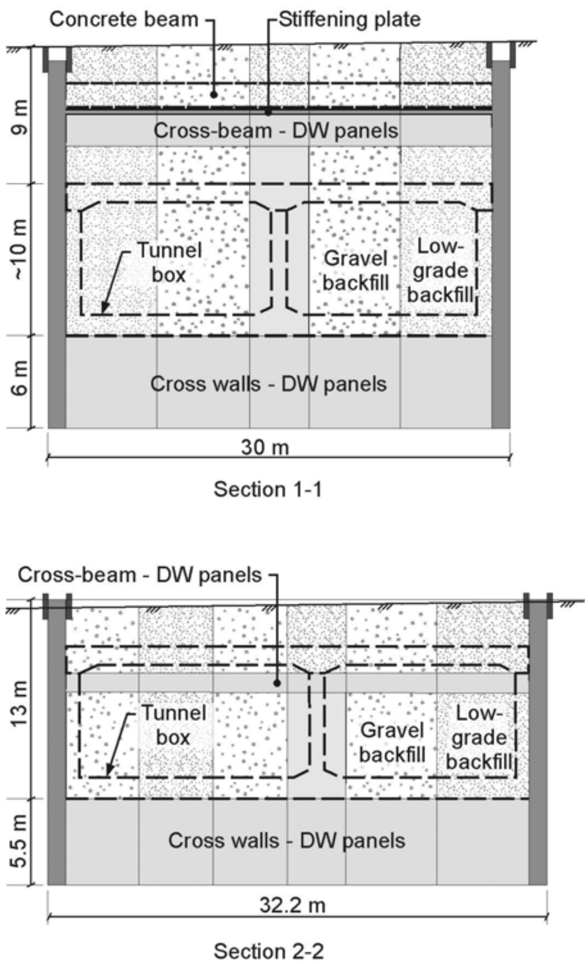

Fig. 27 Typical cross sections Sørenga tunnel

The tunnel depth ranges from 19 m to 10 m. The required width of the tunnel increases from 29 m at the deepest end to 38 m at the shallow end. Fig. 26 shows a longitudinal profile of the western end of the tunnel, and Fig. 27 shows two typical cross sections.

For this tunnel, efforts were made to optimise further the DCW concept that was used in Gothenburg by making the length of diaphragm wall panels and distance between the cross-walls as large as possible. This optimisation required assessment of three main issues:

1. Stability during excavation for the diaphragm wall panels
2. Bottom heave stability
3. Anticipated loads imposed on the cross-walls and the longitudinal walls, and the ability of these structural elements to carry the loads.

These assessments led to a length of 6 m of the longitudinal wall panels, and a corresponding distance of 6.0 m between the cross-walls.

The trenches for the diaphragm wall panels had in reality a length up of about 6.5 m to give room for the stop end plate placed at the end of the trenches during concreting. To avoid stability problems during excavation for the panels, it was necessary to use slurry with unit weight 11.5 kN/m3. This requirement was established based on experiences with test trenches deliberately brought to failure in Oslo in the early 1970's and the method of analyses developed on that basis by Aas (1976). The contractor, who in this case had to take full responsibility for the stability of the trenches, also carried out a test trench at the site and undertook some 3D-FEM analyses as reported by Cudmani and Sedlacek (2006). These analyses confirmed that for 6.5 m long trenches it was indeed necessary to use slurry with a unit weight of 11.5kN/m^3. It can be mentioned that the weight of the excavator (700kN) had a significant impact on the trench stability which was also confirmed by deformation measurements during trenching. It was therefore essential to the stability of the trenches to restrict the time the excavator and other heavy equipment like concreting trucks and cranes for equipment was allowed to stay close to a trench after excavation for a panel was completed. Larger unit weights and longer panel lengths might have been applied, but the contractor took clear reservations about the potential negative impact of heavier slurry on the quality of the concrete, which the owner accepted. In this context it can be mentioned that slurries with unit weight up to 13.0 kN/m^3 were successfully used in Oslo in the 1970's to achieve acceptable stability for some longitudinal wall panels next to a raft-founded 5 storey buildings (Karlsrud ,1975 and 1981). This did not cause problems with achieving good flow and high quality of the concrete.

The design basis agreed with the owner was to have a minimum material factor of 1.35 on the undrained strength when considering bottom heave stability. To satisfy that the cross-walls would only need to be 1.5 to 3.0 m tall. The toe depth was however governed by the concrete design of the cross walls under the combined horizontal loads and vertical uplift forces, and had to be 6 m at the deepest end and 3 m at the shallow end.

At the deepest end, the cross wall panels were also cast with concrete to form a 2 m high temporary concrete strut at a level corresponding to 2.5 to 4.5 m above the tunnel roof, Fig. 27. This temporary concrete cross-wall strut was replaced by a new cast- in- place reinforced concrete beam with a thin slab in between to prevent buckling when the excavation had reached that level. During subsequent excavation to the bottom of the roof plate, the temporary cross-wall strut was demolished. Excavation to and casting of the roof slab was undertaken in an open pit excavation, without any further support of the walls. The remaining excavation to the bottom floor slab then took place from underneath the roof plate. Section 2 in Figs. 26 and 27 shows that for the shallower part, the temporary cross-wall strut was cast just below the permanent roof slab. This means that the longitudinal walls were allowed to cantilever down to a depth of up to 8 m below ground surface without any other support. At the very shallowest end the roof plate was cast at ground surface and provided the only support in addition to the cross-walls.

Fig. 27 also shows that a 3 m wide part of the cross-wall panels were cast with full grade concrete up to the level of the roof plate to act as temporary vertical support for the roof plate. This will be replaced by a cast-in-place partition wall after the bottom slab and the permanent tunnel walls are cast.

To ensure a good joint connection, the contractor in this case chose to place a corrugated stop end plate at the joint when concreting the longitudinal wall panel. This stop end plate was lifted one meter or two early in the curing process to loosen it (like the normal stop-end plates used between panels). Core

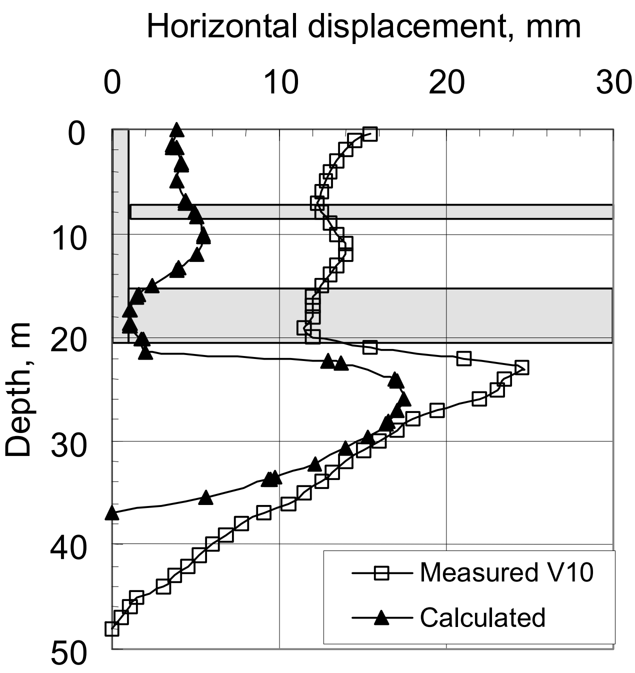

Fig. 28 Predicted and measured displacements for typical cross section at Sørenga

drilling and later inspections during excavation has confirmed that this gave a generally nice and clean joint.

The design analyses of the support system were in this case completely based on FEM analyses using the PLAXIS (2004) program, and using the non-linear and anisotropic ANISOFT soil model. The cross walls were simulated in the same way as presented for the Lillla Bommen tunnel.

Fig. 28 shows a typical example of measured and calculated horizontal displacements. The maximum measured displacement of only 25 mm occurred below the toe of the walls, and is in very close agreement with the PLAXIS analysis. As for the Lilla Bommen tunnel the measured displacement at the level of the cross-walls is larger than calculated, probably due to slight imperfections in the panel joints. Measured interface displacement between cross-wall panels and the longitudinal walls have been hardly measurable, less than 5 mm.

Ground improvement by deep mixing methods

Deep mixing methods (DMM) applicable to soft clays include the mechanical dry- and wet mixing methods. Terashi (2003 and 2005) and Holm (2005) have given a comprehensive overview of the methods and their use for improving soils in general. The dry mixing method started in Sweden and Japan in the 1960's using only slaked lime as binder, then it gradually changed to a mixture of lime and cement in Scandinavia and mostly to pure cement in Japan. Other binders like fly ash and granulated blast furnace slag have more recently been combined with cement and lime, e.g. Holm (2005) and Åhnberg (2006).
The wet mixing methods first developed in Japan started off with using pure cement slurry as binder in the early 1970's,

but also in this case other binders like gypsum and fly ash have been included to engineer strength and stiffness properties that are desirable for specific projects, e.g. Terashi (2003, 2005).

The strength and stiffness properties that can be obtained depends on a large number of factors such as: Soil type, type and amount of binders used, the type of mixing tool, input mixing energy, curing time and curing conditions. The amount of binders used with the relatively light weight Scandinavian dry mixing equipment typically range from about 80-120 kg/m^3. On ideally mixed laboratory specimens of low organic clays the unconfined compression strength using 100 kg/m^3 of binder with 50-50% lime and cement, typically lie in the range q_{uc}= 200 to 1000 kPa, e.g. Holm (2005) and Åhnberg (2006). With the heavier double shaft equipment developed in Japan more binder is commonly used, with the dry method, typically 200-500 kg/m^3. With pure cement q_{uc} values on laboratory prepared specimens are reported by Terashi (2005) to mostly lie in the range 200 to 2000 kPa. With the wet method using only cement Terashi (2005) reports unconfined compression strengths in the range q_{uc}= 1 to 6 MPa.

Terashi (2005) presents correlations between strengths obtained on samples taken in-situ of the mixed and cured columns, against strength on laboratory prepared specimens. The field taken specimens lies in general on the low side of the laboratory prepared specimens and show in general also more scatter. This agrees with NGI's experiences from Norway and data presented from Sweden (e.g. Åhnberg, 2006). The design strength is therefore commonly selected a factor or 2 or so lower than the laboratory strength, but local experiences and verifications are important elements in selecting design values for a specific site.

Lime-cement columns installed by the dry mixing method have been used as stabilising measure on about 30 soft clay excavation projects in Norway since the late 1970's. The columns have most commonly been set in overlapping patterns to form ribs or walls of stabilised clay as illustrated in Fig. 29 (after Karlsrud, 1999). The columns have always been set after sheet pile wall installation, because driving sheet pile walls into stabilised ground may be difficult. This means that it is not possible to achieve perfect contact between the stabilised clay and the wall. To compensate for that, extra columns are sometimes set in the contact area, Fig. 29. The wet mixing method has also been successfully applied on many deep excavation projects using rib type improvement in much the same way as for the dry method, see for instance Terashi (2003). Applications also include block type and cellular type improvement.

The ribs of stabilised soil provide both extra lateral support of the walls and contribute to improve bottom heave stability. Bottom heave stability can be computed by the traditional approach described in Section 3, and accounting for the internal shear resistance along the vertical shear planes inside the excavation. The effect of the stabilised ribs on the interface wall-clay strength is in NGI's practice disregarded. As long as the ribs can provide sufficient lateral support, their effect on

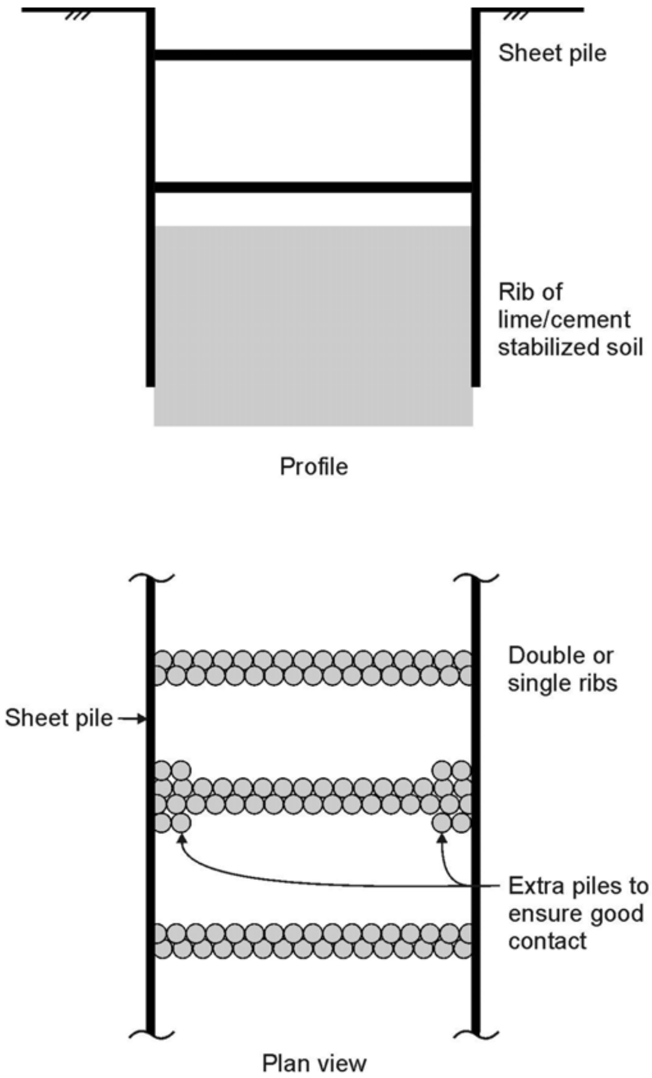

Fig. 29 Typical layout of Lime-cement stabilised ribs in excavations (after Karlsrud, 1999)

bottom heave stability for a normally consolidated clay profile comes largely from the effect of the increased depth of the bottom heave failure surface and thereby an increased strength s_{ub} in equation 12a) compared to a case with no improvement. Achieving good rib to wall contact is therefore mostly a deformation issue.

An important part of the design is to consider and account for the potential negative impact the in-situ mixing tool will have as it is pushed/rotated in and out of ground that is not planned to be treated with binder. Such soil can be severely disturbed by the mixing tool. It is therefore common practice to treat also at least a portion of the columns all the way up to the ground surface. Another aspect for design is to consider the potential negative impact the in-mixing can have on the strength of the clay underneath the treated zone and in between ribs and column. In this relation the volume expansion implied by the materials added in the ground can also set up large horizontal earth pressures and pore pressures in the ground. This may cause stability problems if the improvement is carried out close to existing slopes.

Jet-grouted slabs or ribs

Jet-grouted columns can be used to form stabilised soil bodies of nearly any shape in the ground. In that respect it has a clear advantage as it at least in principle can form a better contact against sheet pile walls, diaphragm walls or other structures than what can be achieved with dry or wet mixing methods. Jet grouting has been fairly widely used as "below-bottom support" since the early 1990's, and most notably in Singapore soft clays. Shirlaw (2003) and Shirlaw et al. (2005) summarise some of these experiences. In the examples from Singapore, jet-grouting has most commonly been carried out to create 2-3 m thick massive slabs of treated clay below the base of the excavation, with the primary intention to reduce displacements. The jet-grouted slabs are as underwater slabs often combined with piles or anchors to take up some of the uplift pressures that may act on the slab (bottom heave, swelling or hydraulic). For 17-18 m deep excavations in the Singapore Marine clay Shirlaw (2005) reports that use of jet-grouted slabs in combination with 80 cm thick diaphragm walls and internal strutting has limited maximum lateral wall displacements to 30 to 70 mm. Shirlaw et al (2005) also describe two major failures when this system has been used (see Section 6 of this paper). It may be questioned if not taller jet-grouted ribs is preferable to thinner more massive jet-grouted slabs, both from a cost and technical perspective. The use of ribs may also do away with the needs for anchoring.

In soft normally consolidated clays, the normally used double or triple jetting system has been shown to produce treated ground with unconfined compression strength of q_u = 1-3 MPa (e.g. Shibazaki, 2003), much in the same range as can be reached with the wet deep mixing method. But recent experiences in Norway shows that q_u values of 10-15 MPa can be achieved in soft clays by decreasing the water/cement ratio during jetting.

With proper workmanship jet-grouting in soft clays has the potential for creating a somewhat more homogenous material than can be achieved with the dry or wet mixing method. On the other hand, the radius of the jet grouted cylinder depends a lot on the jetting equipment and jetting procedures applied. The actual diameter of the jet grouted cylinder and overlap between jet grouted columns is also difficult and fairly costly to verify. Thus, much depends on past experiences from excavated areas under similar ground conditions and with the use of comparable jetting systems. Shibazaki (2003) describes how theoretical considerations can be used to guide selection of jetting pressure and nozzles to give an expected diameter for given ground conditions. Due to the small drill rods generally used, it may be difficult to control the position at large depths, and thus, to ensure overlap between columns which is generally required for stabilising purposes and also to limit displacements as far as possible.

Jetgrunn AS in Norway has developed the jetting technology a bit further. Rather than mixing/jetting the soil with cement slurry they just use the air and water jetting to create a cylindrical hole and replace all material jetted out by full grade concrete in a continuous operation. The method has been name "Jetcrete" method. Jetgrunn AS has instrumented the drill string with sonic sensors which can detect the diameter of the jetted cylinder during construction. Continuous monitoring of the jetted and concreted volumes is also used to verify the produced diameter. Examples of applications can be found in Hoksrud (2000) and Simonsen and Bye (1999). With this jet-concrete method one can in principal achieve concrete cross-walls or struts in the same way as with the DCW concept, and thus, limit displacements even further than with the ordinary jet grouting method.

LESSONS LEARNED FROM FAILURES

General

A number of failures or incipient failures causing unexpected and large ground movements in connection with deep excavations have been published over the years. A recent paper that contain many elements of general interest is that by Shirlaw et al (2005) reviewing failures or near failures in connection with deep excavations in Singapore. Failures are discussed in relation to their common main causes, and how such causes may be avoided in the future. However, first a quick review will be made of the large and dramatic Nicoll Highway collapse in Singapore in April 2004.

The Nicoll Hw collapse

Fig. 30 shows a typical cross section of the excavation. It was 33.7 m deep and about 14 to 21 m wide. Underneath a reclaimed fill layer there was soft marine clay extending to near the bottom of the excavation. Below the marine clay there is an Old Alluvium (OA) consisting of layered firm clayey silts and silty sand strata. The wall was braced at 10 levels and had two levels of jet-grouted slabs, the lowest one just beneath the bottom slab. The wall toed 5 to 10 m into the OA. The failure occurred after excavation below the 9^{th} strut level and the beginning of removal of the upper jet-grouted slab. The complete and dramatic failure developed rapidly and involved a 220 m length of the excavation. The failure has been broadly investigated and assessed by various expert groups. Most factual data can be found in reports by Arup (2004) and Davies (2004) made for the Singapore Authorities. It is not the intention herein to present an independent opinion on the main cause(s) of the collapse, but rather to point to some factors that most investigators seem to agree have played some role in causing this major and complete collapse.

It seems to be an established fact that the design analyses were made using the FEM program PLAXIS (2001). In the analyses the designers chose to use the undrained effective stress Mohr-Coulomb (MC) model available in PLAXIS to model the soft normally consolidated marine clay layer. The effective friction angle for the marine clay was taken as $\varphi' = 22^0$ and $c'=0$. The undrained MC effective stress model does not account for the contractive undrained failure mechanism of soft NC clays. The pore pressure change at failure just corresponds to the change in mean octahedral stress. Therefore the actual undrained strength when using this model will be isotropic and correspond to a normalised undrained strength of $s_u/\sigma_{vo}' = 0.41$. This is about 60 % larger than what would be expected to be the typical average undrained shear strength of this marine clay.

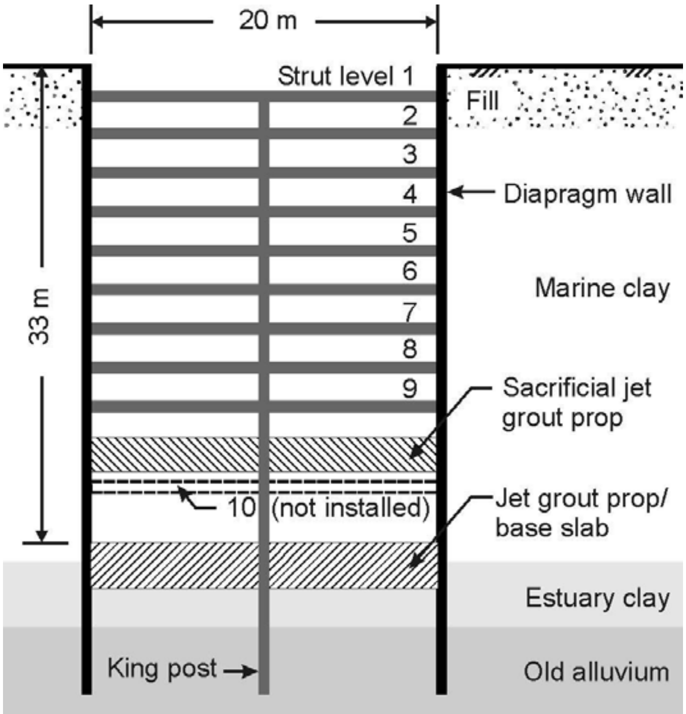

Fig. 30 Typical cross section of MRT excavation near Nicoll highway in Singapore which collapsed (based on New Civil Eng., 2004)

The author's have experienced that the same mistake, in terms of using the effective stress based undrained MC soil model in PLAXIS, has been made on other projects. These observations therefore suggest a possible general lack of understanding of fundamental soft clay behaviour in parts of the profession. It also points to a need for better tutorials/manuals which discuss in more common terms the applicability of the alternative soil models that go into commercial FEM programs now on the market.

The MRT excavation in Singapore was closely monitored during the works. Already a month before the failure, when excavation to and installation of the 8th strut level was completed, the measured wall displacements were in the range 20-25 cm at the level of the two jet-grouted slabs. This is a factor of about 2-3 larger than the maximum displacement reported by Shirlaw (2003) from previous excavations in Singapore where jet-grouted slabs were used. Jet grouted clay is a rather brittle material almost like weak concrete. A displacement of 20-25 cm gives a mean lateral strain of about 2-2.5 %, which is about 3 to 4 times larger than the typical axial strain at failure in UC tests on jet-grouted clays (e.g. Shibazaki, 2003). Thus, the jet-grouted slab may well have been at the verge of collapse after excavation to the 8th strut level. As a further observation, it may be questioned if or to what extent there was a proper understanding of possible failure mechanisms and what would be critical displacement levels when the works were undertaken. This is an aspect of general importance and one that in the authors experience is often given insufficient attention when planning and implementing monitoring programs (e.g. Karlsrud, 1986).

Questions have also been raised about possible lack of proper overlap between jet-grouted columns in the two slabs. This points to a possible lack of good procedures for documenting and controlling the results of jet-grouting, as mentioned in Section 4.4 above.

Some investigators have suggested that failure was induced by hydraulic uplift of the excavation due to high pore pressures in the more permeable OA layer. There does not seem to be sufficient pore pressure data to fully verify if this could be correct. In any case it serves as a reminder to have proper control on the pore pressures in permeable water bearing strata below an excavation in low permeable clays.

Several investigators have pointed to the possible under design of the strut-waler-wall connection, and that this was under-designed by as much as 50 %. It has also been suggested that prior to the failure there were indications of vertical upward deformations in at least some parts of the excavation, including the king-posts (possibly due to tendencies for hydraulic uplift?). Vertical movement of the king posts will cause eccentric loading and can reduce the bearing capacity of the struts significantly. A comment in this respect is that structural details should be designed by professional structural engineers, and not left to the geotechnical specialists. Whether or not this was done in the current case is not known.

The struts seem to have been rather rigidly connected to the walers and further to the diaphragm wall. If there are significant wall rotation at support levels, as it partly was in this case, this will introduce moments at the strut connection which will also reduce it capacity to carry lateral loads. The first author had a rather unpleasant experience with such forced strut bending in connection with large wall displacements in the early stages of the Lilla Bommen M2 cofferdam excavation due to the improper cleaning of the joint between cross-walls and longitudinal described earlier. That situation was however, observed and remedied before it came out of control. A way to avoid such forced bending is to make a central "pinned type" massive steel connection between struts and walers or walls.

<u>Other common failure causes</u>

General causes of failures are in the following briefly discussed and grouped into three categories:

1. Direct construction errors like poor workmanship, over excavation, and un-intended stockpiling, have caused a number of failures. Avoiding such mistakes requires a proper construction follow-up program involving all parties involved, e.g. the owner's representative, the original designer, sub-and main contractors. A start-up seminar with all parties involved to go through and identify construction aspects of most importance to the safety of the works is a good starting point. Establishment of a program for inspection, control and documentation that all parties shall adhere to is also essential. The first author has over the years seen a tendency for more focus at construction sites on costs and claims for extra work, rather than on implementing good control

and inspection routines, but there are recent signs of improvement in this respect.

2. Lack of proper site investigations to reveal, capture and correctly account for the complexity and variability of ground conditions is often a direct or indirect cause of failures. It is a challenge to the geotechnical profession to show the clients the benefit of a thorough and high quality job in these respects. Setting site investigations out on tender and accepting the lowest price is not the way to go. As indicated earlier, the geotechnical profession in general must take advantage of the possibilities that lie in modern and improved site investigation techniques.
3. Use of design methods that are inappropriate, or overlooking important detailed design aspects. The Nicoll Hw. collapse seems to include several such factors. Shirlaw (2005) showed another example in relation to internal strutting of corners, in which the need for transferring the forces from the corner struts into the waler and further longitudinally into the wall was overlooked.

DESIGN APPROACHES AND SAFETY PRINCIPLES

The analyses in Section 2 show that to obtain reasonable design loads in the support structure and limit displacements in connection with deep excavations in soft clays in urban areas, it is vital to have a **real** safety margin in relation to bottom heave instability.

During the review of some published case records over the past 5-10 years it was surprising to find that many major and important projects were still de signed on the basis of rather rudimentary site investigations and laboratory testing, and without accounting for strength anisotropy and the non-linear behaviour of soft clays. This is so even if it is more than 30 years since this understanding of soft clay behaviour was well established, e.g. Bjerrum (1973) and Ladd et al (1977).

With modern site and laboratory investigations it is in general possible to determine the true undrained shear strength characteristics of soft clays within about 10-15 % of the true in-situ values. For soft normally consolidated clays it is also important to recognise when choosing design strengths that there is a lower limit to the undrained shear strength. That lower limit depends on plasticity index and the time under which the clay deposit has been subjected to the present state of effective stress (e.g. how much volumetric creep it has undergone). For clays that have undergone little creep or ageing, the average undrained shear strength ratio (average of triaxial compression, extension and direct simple shear) typically increases from s_u/σ_{v0}'= 0.20 for low-plastic clays to about 0.30 for very high plastic clays, e.g. Bjerrum (1973).

Bauduin et al (2000) discuss use of FEM in Ultimate Limit State (ULS) design. They discuss both the factored strength approach (Case C in Eurocode 7, En 1997-1, European Standard, (2004)), and the model factor approach (Method B in Eurocode 7) in which characteristic strength and stiffness parameters are used in the FEM analysis and the resulting actions (loads) in structural members at the end are multiplied by a load or model factor (1.35 is generally recommended value). They end up with recommending a bit of both, and that one actually goes through quite an elaborate stage-wise process where characteristic and design strengths are used through all main construction steps.

In terms of the geotechnical (bottom heave) stability it is reasonable to use the factored strength approach as for other geotechnical stability problems. For a partly embedded wall it is considered appropriate to use the yield moment capacity of the wall in the analysis (e.g. in equation 12b). For deep embedment, plastic hinges may be allowed to develop in the wall in such stability analyses.

For the determination of toe depth of cantilever or single propped walls it may be reasonable to make use of a factored design strength approach in the equilibrium analyses, as suggested by Simpson and Powrie (2002), but not necessarily for calculating bending moments and reaction forces. Determination of design loads and moments for multi-propped or anchored walls are as discussed in the following even less suited for a factored strength approach.

Factoring strength to ensure structural safety and to account for possible modelling or construction errors is not considered logical for three main reasons:

1. There is a dramatic non-linear relationship between the strength reduction factor and resulting actions or loads. (e.g. Figs. 7 and 8). The potential modelling and construction uncertainties can hardly be expected to have such dramatic effects.
2. In cases where the bottom heave safety factor is large (because the wall penetrates into firm layers at small depth or special stabilising measures have been applied to improve stability or limit displacements), the use of a material factor on the strength may have little or no impact on the loads. In such cases the choice of soil stiffness, possible modelling errors or deviations in the construction process will have a larger impact on the loads than the use of a factored strength.
3. Factoring strength can imply a state of failure in the ground even before any excavation is carried out.. As an example consider a case of a medium plastic soft clay profile with "active" strength corresponding typically to $s_{uA}/\ \sigma_{v0}$'= 0.32. With a typical K_0= 0.5 the initial shear stress on a 45^0 plane is corresponds to τ/σ_{v0}'= 0.25. If the undrained compression strength is reduced with a material factor of 1.4, as typically recommended in Eurocode 7, EN 1997-1, this means that the active design strength corresponds to $s_{uAd}/\ \sigma_{v0}$'= 0.22, which is lower than the shear stress already acting under the K_0= 0.5 condition. Depending on the soil model used, that condition can lead to completely unrealistic displacements and actions.

To have a consistent approach it is therefore considered most appropriate to use characteristic strength and stiffness values in the FEM design analyses, assuming that these in a reasonable manner account for uncertainties. In other words one should use characteristic values on the slightly conservative side.

Factors related to uncertainties in how the actual construction process is undertaken, for instance in terms of over-excavation or stockpiling, can be accounted for directly in the analyses if so wished. When it comes to the structural support system, the design practice in Norway has been to consider loss of an anchor or internal strut as an accidental loading case. Loads in neighbouring struts or anchors are then typically increased by 20 %, but with no further load factor applied to the characteristic values coming out of the FEM analyses, and with a material factor of unity on the strength of structural members.

It is of concern that codes like Eurocode 7, EN 1997-1 (European Standard, 2004) tend to develop in a direction where it goes into very much detail on some specific issues without fully considering the consequences for all types of structures and soil conditions. The code also tends to become rather overwhelming in its definitions, including the large number of factors with mysterious subscripts a poor designer has to deal with. It is suggested that the codes should rather move in a direction of simplicity, allow for more flexibility in choice of material and load factors, and give benefit to use of realistic design models and soil models that best capture reality. The code should also to a larger extent encourage and open for that many uncertainties in design are best dealt with by parametric or sensitivity studies.

SUMMARY AND CONCLUSIONS

The parametric studies presented in this paper confirm general observations from actual measurements that bottom stability and arching effects have a very pronounced effect on earth pressures, strut loads and bending moments for excavations in soft clays. The presented normalised charts for strut loads and maximum bending moments may be used to make first estimates of design loads. The FEM study also supports earlier findings regarding the significant impact of bottom stability conditions and depth of clay layer on displacements.

For very low safety factors against bottom instability the strut loads can be even larger than predicted on the basis of the empirical apparent earth pressure diagram for soft clay by Flaate (1966), whereas the method in Terzaghi, Peck and Messri (1995) tend to be on the conservative side.

The choice of soil model (e.g. isotropic and bi-linear versus anisotropic and fully non-linear) has an impact on the predicted loads and displacements, but the effect depends on the specific case. As long as the average strength is reasonably well represented, the impact on loads and displacements is relatively speaking small when the bottom heave safety factor is low.

Comparative analyses between continuum FEM and with "beam-on-spring" type FEM analyses has not surprisingly revealed that the “beam-on-spring” approach can lead to severe under prediction of loads and displacements when the soft clay layer extends well beyond the base of the excavation, and/or the wall does not toe into a firm layer at shallow depth.

The new parametric FEM studies of bottom heave failure show that a slightly limit equilibrium approach based on Bjerrum and Eide (1956) give very reliable results slightly on the conservative side (0 to 5 %). In this respect it is important to search for the critical width of the failure surface, and not automatically take that as the full width of the excavation. If such a search for critical width is also done when using the Terzaghi (1943) approach, that method will also give reasonably correct answers for normally consolidated clay profiles, but slightly on the un-conservative side (0 to 5 %). Other studies have suggested that the Terzaghi (1943) approach could be more un-conservative for overconsolidated clay profiles with an undrained shear strength that is more constant with depth.

Various schemes have over the years been used to improve the bottom heave stability and limit deformations in connection with deep excavations in soft clays. Underwater excavation and tremie concreting of a bottom slab may today not be an optimal solution, but combined with a drainage and compressible layer under the slab, as used for the Oslo Opera excavation, it may still have its place. The diaphragm-cross-wall concept used on 4 major projects in Scandinavia has so far proved to be best design when it comes to limiting displacements. Dry deep mixing methods is for stability purposes very competitive, but has limitations in terms of strength and stiffness that can be reached. With wet cement deep mixing methods and jet-grouting one may reach strength and stiffness of the treated ground of the same magnitude, and about 2 to 10 times larger than with the dry method. The newer Jetcrete method in which jetted soil is fully replaced by ordinary concrete has the potential for reducing displacements to the same level as the DCW concept.

The authors question the use of a 2-3 m thick jet-grouted slab as it has been applied on several projects in SE Asia, and suggest that taller jet-grouted ribs represent a more optimal and safer concept. Jet-grouting is otherwise a very flexible approach and there are many impressive applications to be found in the literature.

Published failures in connection with some excavations suggest that many in the profession lack proper understanding of the soil models that go into FEM programs. More attention should also be given to the detailed structural design of internal bracing systems, for instance accounting for the impact on wall rotation and forced displacements of king posts on localised stresses in struts and walers.

During the review of some published case records over the past 5-10 years it was surprising to find that many major and important projects were designed on basis of rather rudimentary site investigations and laboratory testing, and without accounting for strength anisotropy and the very non-linear behaviour of soft clays. The profession should encourage clients and designers to use proper procedures and equipment for determination of such soil parameters. The potential cost saving and reduced risk can be very substantial.

In terms of safety concepts and the use of ULS design principles, the geotechnical (bottom heave) stability should be analysed with a factored strength approach whether or not it is based on FEM analyses or limit equilibrium methods. Use of factored strengths in FEM analyses can on the other hand lead to completely unrealistic loads on the support structure. For FEM based design it is therefore recommended to carry out the analyses with characteristic strengths slightly on the conservative side. The resulting loads or actions must then at the end be multiplied with a load factor to arrive at the design loads. Case B in Eurocode 7, ENV 1997-1 actually allows for such an approach.

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