International Conference on Case Histories in Geotechnical Engineering

(1988) - Second International Conference on Case Histories in Geotechnical Engineering

03 Jun 1988, 10:00 am - 5:30 pm

Design and Construction of Anchored and Strutted Sheet Pile Walls in Soft Clay

Bengt B. Broms
Nanyang Technological Institute, Singapore

Recommended Citation

Design and Construction of Anchored and Strutted Sheet Pile Walls in Soft Clay

Bengt B. Broms
Professor, Nanyang Technological Institute, Singapore

SYNOPSIS: The design and construction of anchored and strutted sheet pile walls in soft clay are reviewed in the paper based on experience gained mainly in Singapore during the last 10 years where mainly strutted sheet pile walls and contiguous bored piles are used. It is important to consider in the design also the high lateral earth pressures on the sheet piles below the bottom of the excavation when the depth of the excavation is large compared with the shear strength of the clay. The strut loads and the maximum bending moment in the sheet piles can be considerable higher than indicated by a conventional analysis. Different methods to increase the stability have also been investigated. With jet grouting, embankment piles and excavation under water it is possible to reduce significantly the maximum bending moment, the strut loads, the settlements outside the excavated area and the heave within the excavation.

INTRODUCTION

The design of anchored and strutted sheet pile walls in soft clay had to satisfy the following criteria.

o that the sheet pile wall should be stable and the factor of safety be adequate with respect to complete collapse both during and after the construction of the wall (ultimate limit state)

o that the displacements and deformations of the sheet pile wall and of the support system at working loads should be small so that the sheet pile wall will function as intended in the design (serviceability limit state).

o that the settlements or lateral displacements caused by the installation of the sheet piles or of the support system (e.g. the driving of the sheet piles or the installation of the anchors) should be small so that adjacent buildings or other nearby structures are not damaged. The settlements from an unintentional lowering of the ground water level in soft clay due to e.g. pumping can be large.

The main factors affecting the behaviour of anchored or braced excavations in soft clay can be classified as follows :

o Geometry of the excavation
(depth, width, shape and excavation sequence)

o Soil and ground water conditions
(strength and deformation properties of the soil and the ground water level)

o Properties of the sheet piles
(stiffness and depth of sheet piles and the chosen construction method)

o Properties of the support system
(type, spacing and preloading of ground anchors or of struts)

o Loading conditions
(surcharge and traffic loads)

o Workmanship

Thus a large number of factors can affect the behaviour of both anchored and strutted sheet pile walls. In this paper experience with strutted and anchored sheet pile walls primarily in Singapore has been reviewed. Limitations of different wall and support systems are analyzed. Methods that can be used to calculate lateral earth pressure and the stability of deep excavations with respect to bottom heave and excessive settlements have been evaluated as well as methods to increase the stability. The following review is mainly based on experience gained in Singapore during the last 10 years where numerous deep excavations in soft clay have been required for high rise building, subway stations and tunnels.

SOILS CONDITIONS IN SINGAPORE

There are extensive deposits of very soft marine clay and organic soil with a thickness of up to 35 m or more along the coast and in the buried river valleys in Singapore. It is mainly these soils that have caused difficulties during the construction of both anchored and braced sheet pile walls, e.g. large lateral displacements and settlements. The organic content of the marine clay is normally 3% to 5%. The water content varies usually between 65% and 100%. The undrained shear strength (c_u) which is usually low close to the ground surface increases approximately linearly with depth. Tan (1983) has reported a c/p - ratio (c_u/σ'_{vo}) of 0.315 based on the results from field vane tests. Tan (1970) and Ahmad and Peaker (1977) have indicated somewhat lower values, 0.27 and 0.25, respectively. The effective friction angle ϕ' as determined by consolidated undrained or drained triaxial tests (CD or CU-tests) has been very consistant, 21 to 22 degrees. Settlement observations and oedometer tests indicate that the clay is slightly overconsolidated. The overconsolidation ratio (OCR) is 1.1 to 1.5. The coefficient of consolidation when the clay is normally consolidated is typically 1 to 2 m^2/year.
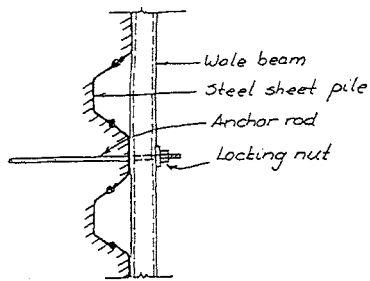
WALL SYSTEMS

Different wall systems can be used as illustrated in Fig 1 depending on the soil conditions. In Fig 1a is shown a conventional anchored sheet pile wall. The lateral earth pressure on the wall is transferred to the ground anchors through wale beams, normally U-, H- or I- beams.

Soldier pile and lagging construction is shown in Fig 1b. This support method, also called Berliner wall construction, is commonly used in the United States and in Europe mainly in sand, silt or gravel above the ground water level. The method is not suitable in soft clay. The soldier piles or beams, usually H-piles or channels, are driven or placed in predrilled holes and grouted. The spacing of the piles is normally 1.0 to 2.0 m. Lagging (wooden boards) is placed during the excavation between the flanges of the soldier piles.

It is important that the lagging is carefully wedged against the soil behind the boards in order to reduce the settlements around the excavation. Also precast or cast-in-place concrete panels can be used as shown in Fig 1c. The deep excavations required for some of the subway stations in Hong Kong have been stabilized by this method.

In silt or in fine sand there is a risk of erosion of the soil below the ground water level and the resulting settlements can be large. Soldier piles and lagging construction should therefore be avoided in these soils when the ground water level is high. The ground water level can, however, be lowered temporarily with well points or filter wells to prevent erosion and failure of the excavation by bottom heave. In stiff to hard clays it may be advantageous to use pairs of channels as soldier piles instead of steel H-piles (Fig 1c). The wale beams can then be eliminated in order to
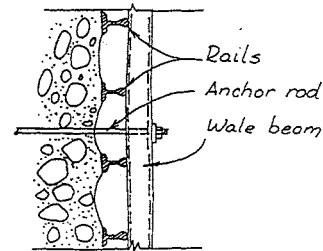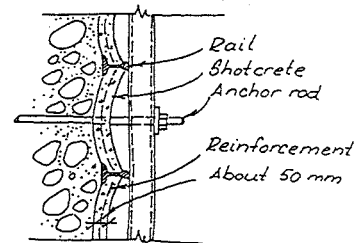

a. Steel sheet piles.
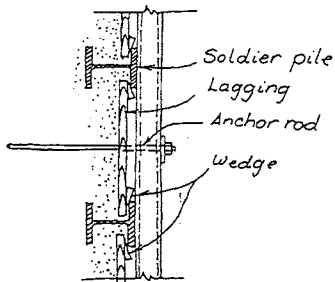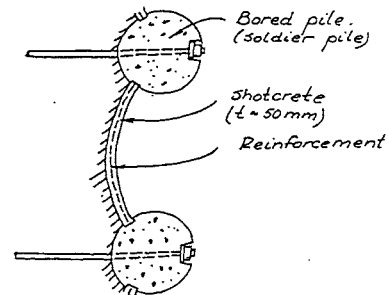

b. Soldier piles and lagging.
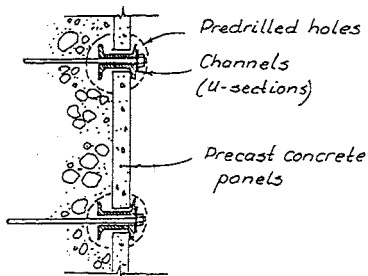
c. Soldier piles and precast concrete panels.

d. Rails

e. Rails and shotcrete

f. Bored piles and shotcrete.

Fig 1a   Wall systems

1516

reduce the costs since the anchors can be placed between the two channels.

Rails are used as lateral support in Fig 1d. The spacing is usually 0.2 to 0.3 m. This support method is mainly used in stony or blocky soils above the ground water level. The rails are often placed in predrilled holes when the content of stones or boulders is high since the rails cannot be driven. The rails are often brittle due to the low ductility of the steel (high strength steel). They are difficult to splice by welding. Therefore, bolted joints are often used. In dry sand above the ground water level plywood boards are sometimes placed between the rails to contain the sand. In stiff medium to stiff clays or in silty soils, the soil is normally protected by shotcrete as illustrated in Fig 1e. The reinforced shotcrete arches transfer the lateral pressure from the soil to the rails. The thickness of the shotcrete is normally about 50 mm.

Also bored piles can be used as lateral support in deep excavations as illustrated in Fig 1f. In soft clay the piles should overlap while in medium to stiff clay overlapping is not required. The distance between the piles can be relatively large. The unprotected area between the piles is often covered by shotcrete. Overlapping bored piles, so-called contiguous bored piles, are common in Singapore also in soft clay as foundation for high rise buildings and as lateral support.

ANCHORS AND STRUTS

Different support systems can be used for a deep excavation in soft clay or silt as illustrated in Fig 2 depending on the soil and ground water conditions and on the size (width, length and depth) of the excavation.

The choice of support system depends mainly on the costs, on restrictions at the worksite, on available equipment in the area and on the experience of the consultant or of the contractor. For example, adjacent buildings may be damaged by excessive settlements if a cantilever sheet pile wall is used to support a relatively deep excavation. Also water mains, sewer lines and heating ducts can be damaged by the resulting large settlements and lateral displacements. Excessive settlements can also be caused by the installation of the anchors as well as by the driving of piles inside the excavation. Struts may, therefore, be chosen instead of ground anchors to reduce the risks. The settlements can be reduced further by preloading the struts or the ground anchors. If the anchors are left permanently in the ground they may interfere with future construction such as the driving of sheet piles. However, different anchor systems have been developed during the last few years which can be removed after use and where the settlements caused by the installation of the ground anchors will be small.

The lateral earth pressure behind a cantilever sheet pile wall (Fig 2a) is resisted by the passive earth pressure below the bottom of the excavation while for an anchored or strutted sheet pile wall the lateral earth pressure is resisted by ground anchors or by struts as shown in Fig 2b and 2c, respectively. Ground anchors or struts are normally required in soft clay when the depth of the excavation exceeds 2 to 3 m.
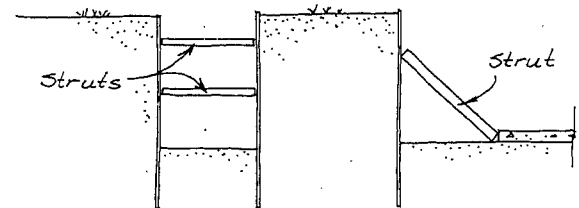
In a large and wide excavation the length of the struts will be large if the struts are horizontal. They had to be braced to prevent buckling as can be seen in Fig 3. The struts will, however, interfere with the
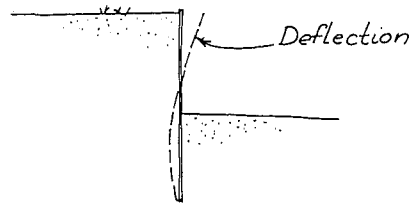
(a) Cantilever sheet pile wall
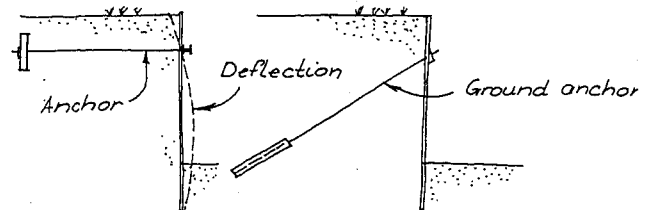
(b) Anchored sheet pile wall

(c) Strutted sheet pile wall.

Fig 2   Support systems

work in the excavation and reduce the efficiency. Horizontal bracing is common in Singapore.

The anchors or the struts can either be horizontal or inclined. In narrow deep cuts horizontal struts are used while in large and wide excavations the struts are often inclined. The inclined struts are generally supported at the bottom of the excavation by a concrete slab or by separate individual concrete footings. It should be observed that the inclined struts or anchors will cause an axial force in the sheet piles which affects the stability of the wall.

A number of different ground anchor systems using bars, wires or strands have been developed during the last 20 years as described by e.g. Hanna (1982). A relatively high pressure is often used in sand or silt for the grouting of the tendons in order to enlarge the hole so that a bulb is formed around the tendons within the grouted section, the fixed anchor length. The tube-à-manchette method can be used especially in sand, gravel and rock to control the grouting. The bore hole can be enlarged mechanically in stiff clay, using a special cutting device in order to increase the tensile resistance of the ground anchors. Also, H-beams have been used as ground anchors in Sweden in very soft clay. The pull-out resistance is high due to the large surface area.
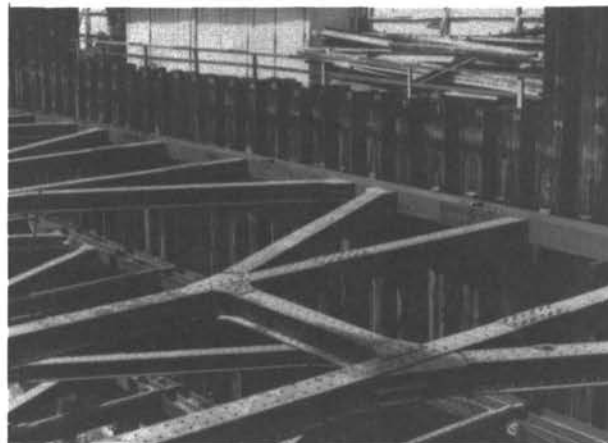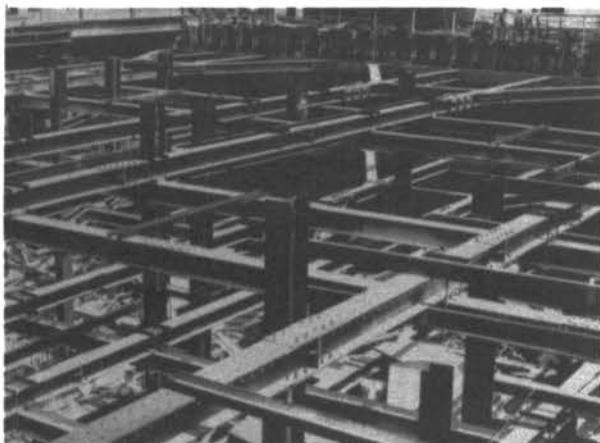
1517

Fig 3   Braced sheet pile wall

Rods (bars) are normally used when the load in the anchors is relatively low, less than about 400 kN, while cables (wires or strands) are utilised as tendons when the load exceeds about 400 kN. The anchor rod or wires are often prestressed in order to reduce the horizontal displacements and the deformations of the wall and thus the settlements during the excavation. Ground anchors are mainly used for temporary structures because of the risk of corrosion of the tendons or of the anchor rods. The corrosion can be reduced for permanent anchors by enclosing the tendons and by introducing a fluid between the covering and the tendons. Also cathodic protection can be used.

A recent development is expander bodies. This new type of anchor consists in principle of a folded thin steel sheet, which can inflated in-situ through the injection of cement grout as shown in Fig 4 (Broms, 1987). The expander bodies can either be driven into the soil or placed in predrilled cased holes depending on the soil

Fig 4   Expander bodies

conditions. The volume of the grout required for the expansion and the pressure should be measured in order to check the ultimate resistance. The maximum grout pressure in granular soil is 3 to 4 MPa. The main advantage with this new type of ground anchor is that the size and the shape of the anchors are controlled.

In Sweden, the Lindö and the JB methods where the casing is provided with a sacrificial drilling bit are used for the drilling of the boreholes. Also different eccentric drilling methods have been developed e.g. Odex, Exler and Alvik to facilitate the installation of the casing and to reduce the costs. An additional method is the In-Situ Anchoring Method where the anchor rods are used as drill rods during the drilling of the boreholes. Casing is not required. However, the allowable load is relatively low for this type of anchor and the method is therefore relatively expensive.

The chosen method of installation of the struts and of the anchors affects both the total lateral earth pressure as well as the earth pressure distribution. When relatively stiff struts are used, the lateral earth pressure can be considerably higher than the active Rankine earth pressure particularly close to the ground surface while at the toe the lateral earth pressure can be lower than the active Rankine earth pressure. The reason for this difference is the relatively small lateral deflection of the sheet pile wall close to the ground surface during the construction since the struts are normally wedged and preloaded.

A certain small lateral deflection is required to mobilize the shear strength of the soil behind the wall and to reduce the lateral earth pressure. In dense sand a lateral displacements of 0.05% of the depth of the excavation is normally sufficient to reduce the lateral earth pressure to the active Rankine earth pressure. When the sand is loose the required lateral deflection is approximately 0.2% of the depth. A much larger deformation is required in soft clay.
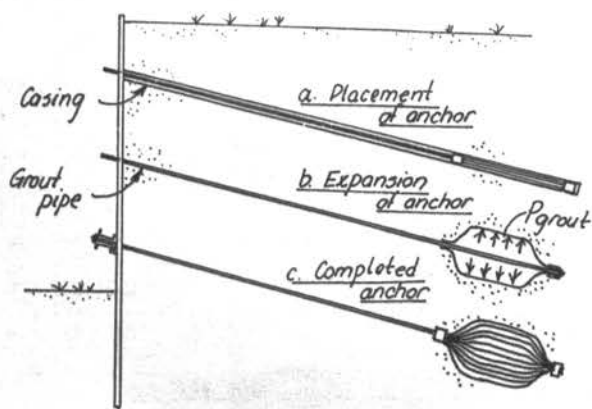
1518

DESIGN PRINCIPLES

The following four steps are normally followed in the design of a sheet pile wall :

o   Evaluation of the magnitude and the distribution of the lateral earth pressure behind the sheet pile wall

o   Calculation of the required penetration depth

o   Determination of the moment distribution in the sheet piles

o   Estimation of the axial force in the ground anchors or in the struts

Extensive investigations are normally required in the field and in the laboratory to determine the depth and the thickness of the different soil strata and of the underlying rock as well as their strength and deformation properties as indicated, for example, in the British Code of Practice (CP2001). Penetration tests are mainly used in cohesionless soils (sand and gravel) in order to estimate the relative density, the angle of internal friction and unit weight. Cone penetration tests (CPT) and weight soundings (WST) are preferred before the standard penetration test (SPT) because of the uncertainties connected with this testing method. However, representative samples are obtained with SPT so that the soil can be classified. The size and the shape of the soil particle are important as well as the gradation since these parameters affect the friction angle of the soil.

The driving of the sheet piles are affected by stones and boulders in the soil. The stone and boulder content of the different strata and the difficulties that may be encountered during the driving of the sheet piles can normally be evalauted from weight (WST) or ram soundings (DP) or from cone penetration tests (CPT). Driving tests with full size sheet piles may be required for large jobs. Stress wave measurements can be helpful to determine the driving resistance and the efficiency of the driving. It is also important to determine the location and possible variations of the ground water level.

For anchored or strutted walls the depth of any soft clay or silt layers below the bottom of the excavation and the variation of the thickness of these layers are particularly important since the stability of the wall depends to a large extent on the passive earth pressure that can develop at the toe of the sheet pile wall. The depth to a firm layer below the bottom of the excavation can usually be determined by penetration tests. Also seismic methods can be used.

Penetration tests especially cone penetration tests (CPT) and weight soundings (WST) are useful in cohesive soils in order to determine the sequence and the thickness of the different layers. The undrained shear strength of the clay is normally evaluated by field vane tests. Undisturbed samples obtained preferably by a thin-walled piston sampler are usually required when the shear strength of the soil is evaluated in the laboratory by, for example, unconfined compression, fall-cone or laboratory vane tests. Undrained triaxial tests are often used to determine the undrained shear strength of stiff fissured clay. The water content, the liquid and plastic limits of the clay should also be measured. Drained triaxial or direct shear tests are required for heavily overconsolidated clays in order to evaluate ϕ_d or ϕ'. The difference between the

two angles is usually only a few degrees. An estimate of the long term ground water level and the changes that may occur with time is also necessary. Percussion drilling and coring are normally required in rock. The quality of the rock can often be estimated from the drilling rate. The compressive and tensile strengths can be determined by unconfined compression and or point load tests.

The conditions of the adjacent structures should also be investigated (dilapidation survey). The type of foundation (spread footings, raft or piles) is important since it can affect the choice of support system.

Cause of failure	Failure mechanism
a. Failure of upper strut or anchor	
b. Failure of middle strut or anchor	
c. Failure of lower strut or anchor	
d. Moment capacity is insufficient at the top.	
e. Moment capacity is insufficient at the centre	
f. Penetration depth and moment capacity are insufficient	

● = plastic hinge
× = failure of strut or anchor

Fig 5   Failure mechanisms

In the design of anchored or braced sheet pile walls it is preferable to use characteristic strengths and characteristic loads which takes into account the uncertainties connected with the determination of the shear strength of the soil or of the rock and the loading conditions. A design strength $f_d = f_k/\gamma_m$ is used in the calculation of the lateral earth pressures where f_k is the characteristic strength of the soil or the rock and γ_m is a partial factor of safety larger than 1.0. External loads are treated in a similar way. A design load $F_d = F_k\gamma_f$ where γ_f is a partial coefficient and F_k is the characteristic load, is then used in the calculations of the lateral earth pressures. The probability that the characteristic load will be exceeded in the field should not be greater than 5%. The failure mode or failure mechanism and the deformation required to mobilize the peak resistance of the soil should also be considered when the required partial factor of safety is evaluated as well as cracks and fissures. A statistical analysis of the test results may in some cases be helpful.

A global factor of safety F_s is often used in the design of both anchored and strutted sheet pile walls. A value of 1.5 on F_s is often chosen for clays with respect to the required penetration depth in order to prevent failure by rotation of the sheet pile wall about the anchor level. For cohesionless soils a global factor of safety of 2.0 is normally required.

LATERAL EARTH PRESSURE

Possible failure mechanisms of anchored or strutted sheet pile walls supported at several levels are shown in Fig 5. Failure may occur when the anchors or struts rupture or buckle (Figs 5a, 5b or 5c) or when the moment capacity of the wall has been exceeded (5d, 5e or 5f). The deformations of the sheet piles during the excavation affect both the magnitude and the distribution of the lateral earth pressure behind the wall. The lateral earth pressure can be considerably lower than the active Rankine earth pressure between the support levels due to arching when the lateral deflections of the wall are large. At the strut or anchor levels the lateral earth pressure can be considerably higher that the active Rankine earth pressure as pointed out by e.g. Rowe (1957).

The earth pressure distribution for temporary structures in clay is shown in Fig 6. This distribution is in principle the same as that proposed by Terzaghi and Peck (1967). A trapezoidal earth pressure distribution can be used in the calculation of the force in the anchors and in the struts as well as of the required penetration depth. The lateral earth pressure is assumed to be $[\rho H - 4c_u]$ above the bottom of the excavation when the depth of the excavation exceeds $4c_u/\rho$ and $0.35\rho H$ when the depth is less than $4c_u/\rho$.

Below the bottom of the excavation the net pressure, the difference in the lateral earth pressure on both sides of the wall is $(\rho H - N_{cb}c_u)$ where N_{cb} is the bearing capacity factor of the soil with respect to bottom heave. This factor depends on the dimensions of the excavation (depth, width and length). The net pressure will be negative and contribute to the stability when $\rho H < N_{cb}c_u$ and positive when $\rho H > N_{cb}c_u$.
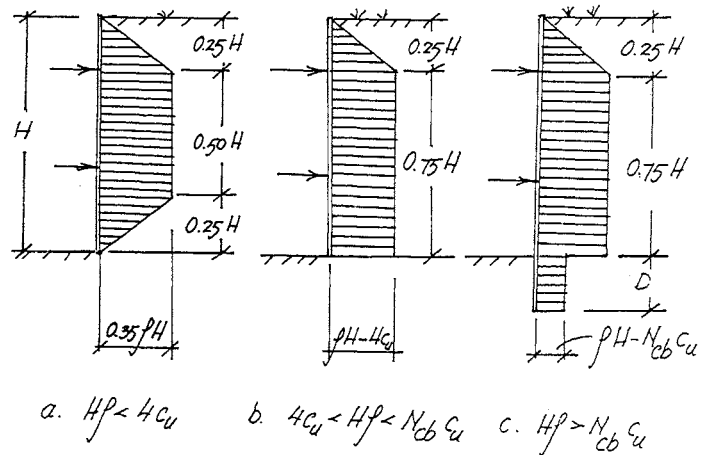
Fig 6   Design of anchored and braced sheet pile walls in soft clay

It is proposed to use the net pressure below the bottom of the excavation at the design instead of the coefficient m as proposed by Terzaghi and Peck (1967) to take into account the increase of the strut or anchor loads when the shear strength of the clay is low below the bottom of the excavation compared with the total overburden pressure. A similar calculation method has been proposed by Aas (1984) and by Karlsrud (1986).

It has been assumed in the calculation of the net earth pressure that the adhesion (c_a) along the sheet piles corresponds to the undrained shear strength of the clay (c_u). The bearing capacity factor N_{cb} will be reduced when $c_a < c_u$. For an infinitely long excavation $N_{cb} = 4c_u$ when $c_a = 0$, a reduction by about 30%.

A relatively large lateral deflection is required to develop the passive lateral earth pressure in front of the wall and thus the net pressure when the shear strength of the clay is low. Adjacent buildings can be damaged by the resulting large settlements. It may therefore be advisable for soft clay to use a lower lateral earth pressure than the net pressure in the calculation of the required penetration depth.

The total lateral earth pressure when the depth of the excavation is less than the critical depth $4c_u/\rho$ corresponds approximately to the lateral earth pressure at rest ($K_o \approx 0.7$ to 0.8). This earth pressure may be used in the design of permanent structures in soft clay. The preload in the anchors and in the struts should preferably be adjusted periodically especially in soft clay to compensate for creep and consolidation of the soil behind the wall.

In a heavily overconsolidated clay it is important that the lateral earth pressure is sufficiently high close to the ground surface to eliminate any tensile stresses in the soil and to prevent cracking of the clay. Vertical tensile cracks may reduce the shear strength of the clay and increase the lateral pressure when the cracks are filled with water after a heavy rainstorm.

1520

BOTTOM HEAVE

In the design of a strutted or anchored sheet pile walls in soft clay, failure by bottom heave had to be considered as illustrated in Fig 7. The part of the sheet piles that extends below the bottom of the excavation in Fig 7a must resist a lateral earth pressure that depends on the depth of the excavation and on the undrained shear strength of the clay.

It is proposed to use the net earth pressure as shown in Fig 6 for the part of the sheet pile wall that extends below the lowest strut level. This part of the wall functions as a cantilever which carries the load caused by the lateral earth pressure behind the sheet piles. This load is partly resisted by the passive earth pressure between the two sheet pile walls.

The passive earth pressure is affected by the distance (B) between the two walls. If this distance is less than approximately the penetration depth (D) then the passive earth pressure at the bottom of the sheet piles can be evaluated from the relationship

$$\sigma_p = 2 \ c_u + D\rho + 2 \ c_u D/B \qquad (1)$$

When the distance B between the sheet piles exceeds the penetration depth D (B>D) it is proposed to evaluate the passive earth pressure from the following relationship (Janbu, 1972)

$$\sigma_p = 2c_u\sqrt{1 + \beta} + D\rho \qquad (2)$$

where $\beta = c_a/c_u$. It should be noticed that the passive undrained shear strength as determined from triaxial extension tests should be used in the calculations. This shear strength may be lower than that determined by e.g. field vane tests.

A load factor equal to 1.0 has been used with respect to the unit weight of the soil and the water. In the soft clay below the bottom of the excavation the net lateral pressure is $[\tau_f q + \rho H_1 - \rho_w H_w - N_{cb} c_u/\tau_m]$ where N_{cb} is the stability factor with respect to bottom heave (Fig 8). In the intermediate sand layer the net pressure will be positive and contribute to the stability of the wall. The lateral earth pressure will to a large extent depend on the pore water pressure in this layer.

A 2.0 m thick unreinforced concrete slab will be cast below water at the bottom after excavation down to the required depth to prevent heaving when the water level in the excavation is lowered.

If the adhesion (c_a) along the sheet piles corresponds to the undrained shear strength (c_u) of the clay, then

$$\sigma_p = 2.83 \ c_u + D\rho \qquad (3)$$

When the penetration depth is large compared with the width B, the passive pressure between the two rows will normally be larger than the outside earth pressure and the sheet piles will be supported at least partly by the passive earth pressure between the two walls.

The uplift pressure at the bottom of the sheet pile wall depends on the depth of the excavation H, the penetration depth D, the undrained shear strength of the clay as well as on the shape of the excavation

$c_u \leq 50 \ kPa$ $c_u \leq 50 \ kPa$

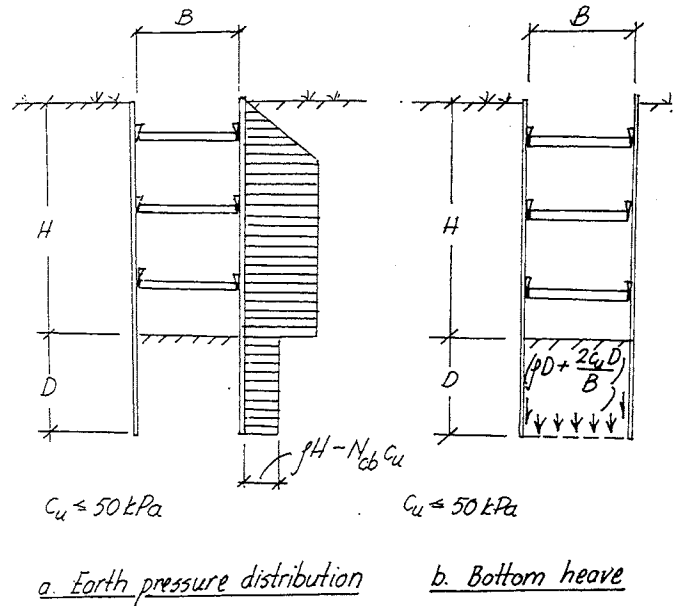a. Earth pressure distribution   b. Bottom heave

Fig 7   Design of braced sheet pile walls in soft clay

(B/L). The uplift pressure at the bottom of the sheet piles (Fig 7b) can be evaluated from the equation

$$\sigma_u = (H + D) \ \rho - N_{cb} \ c_u \qquad (4)$$

where N_{cb} is a stability factor (Fig 8) which can be determined from the following relationships (Bjerrum and Eide, 1956).

$$N_{cb} = 5 \ (1 + 0.2 \ H/B) \ (1 + 0.2 \ B/L) \qquad (5)$$

when H/B $\leq$ 2.5 and from

$$N_{cb} = 7.5 \ (1 + 0.2 \ B/L) \qquad (6)$$

when H/B > 2.5.

This uplift pressure had to be resisted by the weight of the soil below the bottom of the excavation and by the adhesion c_a of the clay along the sheet piles.

$$\sigma_u = D\rho + 2c_a D/B \qquad (7)$$

In the calculation of the required penetration depth is is advantageous to use load factors (τ_f) and partial safety factors (τ_m) as mentioned previously.

The proposed design method is illustrated in Fig 9a for a braced sheet pile wall. The sheet piles have been driven through soft marine clay (Upper Marine Clay, M) into an underlying intermediate layer with sand (F1). Below this intermediate layer is a second layer with soft marine clay (Lower Marine Clay, M). The shear strength of the clay is low.

It is anticipated that the excavation of the fill and the soft clay will be carried out below water in order to prevent failure of the excavation by bottom heave

1521
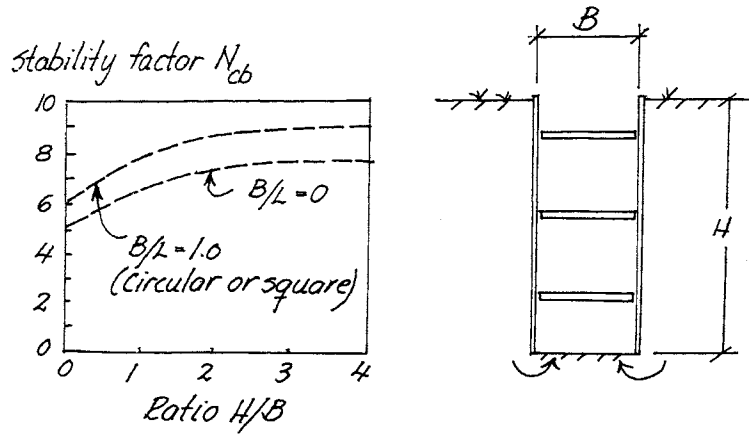
Failure by bottom heave
(after Bjerrum & Eide, 1956)

Fig 8   Stability factor N_{cb}

due to the very low shear strength of the clay. The water level in the excavations will be kept at or above the ground level in order to increase the stability of the excavation. Bored piles are used to support the bottom slab. The piles will be installed before the start of the excavation and provided with a permanent casing to prevent necking of the concrete during the casting because of the low shear strength of the clay.

The earth pressure distribution when the excavation has reached the maximum depth is shown in Fig 9a. The lateral earth pressure above the bottom of the excavation corresponds to $[\gamma_f q + \rho H_1 - 4c_u/\gamma_m]$ where γ_f

is a load factor and γ_m is a partial factor of safety. The uplift pressure on the concrete slab will vary. A higher uplift pressure (q_1) is expected on the slab next to the two sheet pile walls compared with that (q_3) at the center of the slab as shown in Fig 9b and Fig 9c, respectively.

The uplift pressure q_1 in Fig 9b depends on the total overburden pressure ($\gamma_f q + \rho H_1$) outside the sheet pile wall at the level of the concrete slab, on the lateral resistance of the sheet piles q_2, on the shear strength
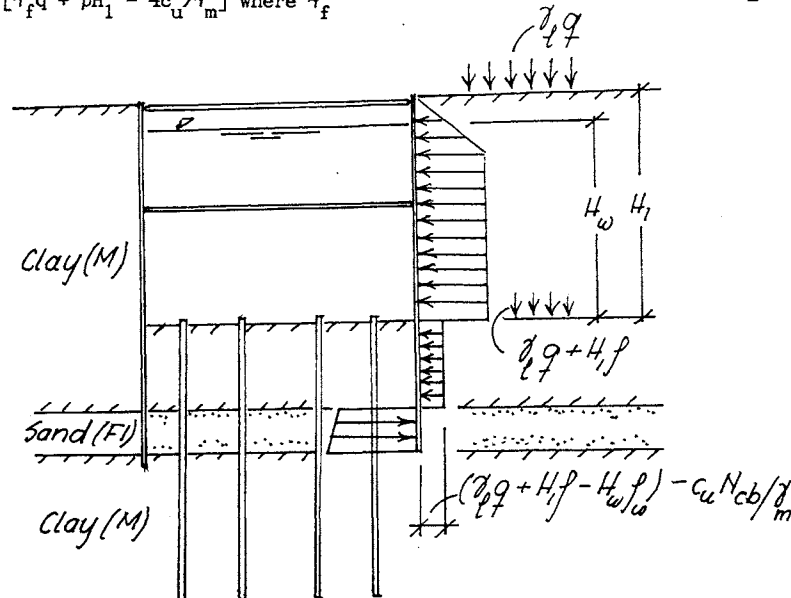

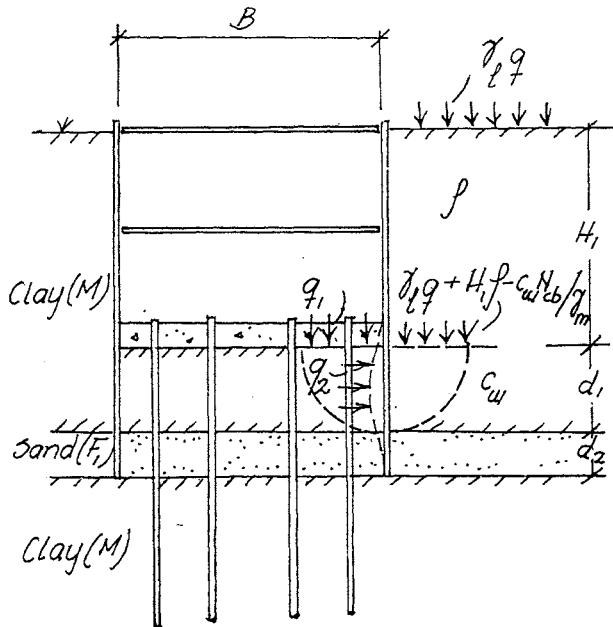Fig 9a   Proposed design method for a strutted sheet pile wall in soft clay
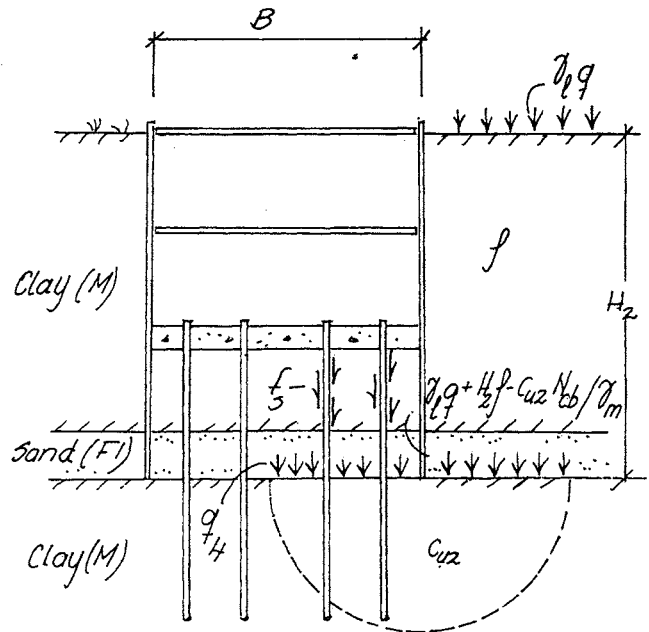
1522

Fig 9b   Bottom heave (upper clay layer)

Fig 9c   Bottom heave (lower clay layer)

of the clay c_{u1} and on the stability factor N_{cb}. This uplift pressure will act on a strip with a width that corresponds to the depth of the clay layer below the concrete slab.

The stability number for the excavation ($D/L = 0.58$) is 5.9 when the excavation is long compared with the width ($B/L = 0$) as can be seen from Fig 7. However, a relatively large deformation will be required to mobilize the average shear strength of the clay. A partial factor of safety of about 1.4 is required to limit the maximum wall movement to 1% of the excavation depth (Mana and Clough, 1981).

The uplift pressure within the center part of the excavation can be estimated as shown in Fig 9c. This uplift pressure q_3 will be lower than that next to the two sheet pile walls (q_1) because of the relatively high shear strength or the lower marine clay (c_{u2}). The overburden pressure at the bottom of the fluvial material F1 depends on the average unit weight of the soil above this layer.

The confining pressure q_4 below the bottom of the intermediate layer (F1) at the centre of the excavation can be estimated from the equation.

$$q_4 = q_3 + d_1\gamma_1 + d_2\gamma_2 + \Sigma Q_s/B \qquad (8)$$

where ΣQ_s is the total skin friction resistance per unit length along the sheet piles and the piles in the marine clay and in the F1 material (f_{s1} and f_{s2},

respectively) and B is the total width of the excavation. The adhesion (c_a) along the sheet piles and the piles in the soft clay is estimated to $0.8c_u$, where c_u is the undrained shear strength as determined by e.g. field vane tests. It is suggested that the unit skin friction resistance in the sand (F1) can be taken as 1% of q_c, where q_c is the cone resistance as determined by cone penetration tests (CPT). It has thus been assumed that the total skin friction resistance along the piles and the sheet piles can be distributed uniformly over the total width of the excavation.

SHEET PILE WALLS SUPPORTED BY INCLINED ANCHORS

An anchored sheet pile walls may fail when the vertical bearing capacity of the sheet piles is exceeded as illustrated in Fig 10 in the case the anchors are inclined. The inclined anchors produce a vertical force in the sheet piles which may cause the sheet piles to settle if the embedment depth is not sufficient. A settlement (δ_v) will also cause the wall to move outwards (δ_h) a distance $\delta_v \tan \alpha$ where α is the inclination of the anchor rods or of the cables at the level of the anchor (Fig 10). The inclination of soil anchors in soil is often 20 degrees while for rock anchors the inclination is normally 45 degrees. The inclination can be increased in order to reduce the length of the anchor rods or of the cables and thus the cost. The vertical component of the anchor force along the sheet piles is, therefore, often higher when the sheet piles have been driven into rock compared with

1523

a. Failure mechanism

$V = S \sin \alpha$
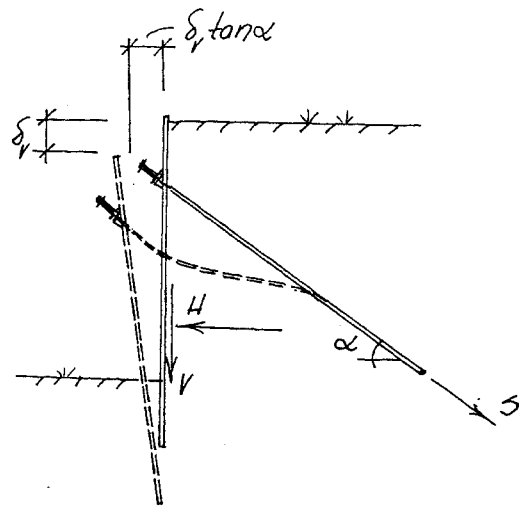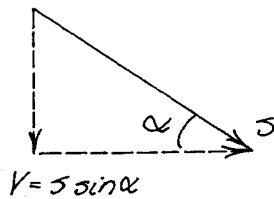
b. Force polygon.

Fig 10   Vertical stability of sheet pile wall with inclined anchors

soil anchors because of the difference in inclination of the tendons. The sheet piles can generally be driven to a higher resistance when competent rock is located close to the bottom of the excavation and rock anchors are used. It is then relatively easy to resist the high vertical force in the sheet piles.

When the depth to rock or to a layer with a high bearing capacity is relatively large and soil anchors had to be used then it is difficult to resist the vertical component of the anchor force by adhesion or by friction along the sheet piles. It may then be more economical to reduce the inclination of the anchors and to increase the length of the anchor rods or of the cables. Then the length of the sheet piles can be reduced because of the reduced axial force.

Figs 11a and 11b illustrate the forces acting on a braced and anchored sheet pile walls in clay, respectively. The normal force N and the shear force T (T is proportional to the active undrained shear strength of the soil c_u) act along the assumed failure plane. The weight (W) of the sliding soil wedge is approximately the same for the two cases. The force (C_a) along the sheet piles depends on the adhesion (c_a) between the sheet piles and the clay below the bottom of the excavation. The inclination and the magnitude of the force (R) in the anchors or in the struts will, however, be different.

It can be seen from the two force diagrams in Fig 11 that both the normal force N on the failure plane and the passive earth pressure force P_p which are required for equilibrium will be larger for an anchored sheet pile wall when the anchors are inclined than for a braced or a strutted wall when the struts are horizontal. Thus a larger penetration depth and a higher passive earth pressure will be required for an anchored wall where the tendons are inclined compared with a braced wall.

The stability of an anchored sheet pile wall can be expressed by the stability factor N_{cb} defined by the
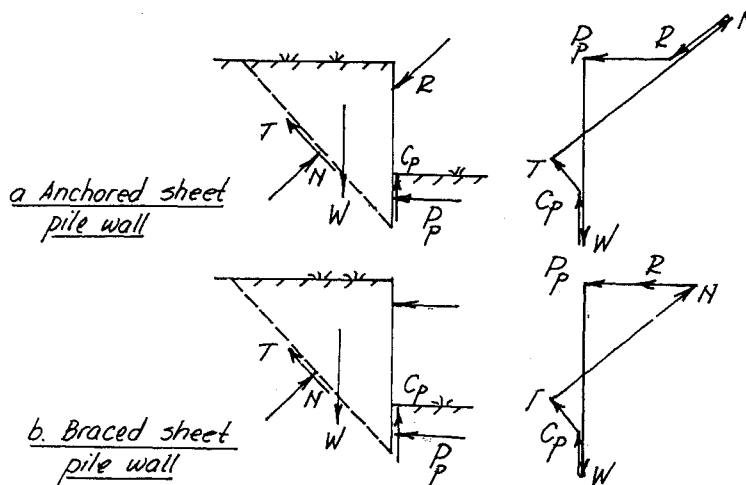

a Anchored sheet pile wall

b. Braced sheet pile wall

Fig 11   Stability of anchored and braced sheet pile walls

1524

equation $(\rho H_{cr} + \gamma_f q) = N_{cb}\, c_u/\gamma_m$ where $(\rho H_{cr} + \gamma_f q)$ is the total overburden pressure at the bottom of the excavation H_{cr} is the critical depth and c_u is the undrained characteristic shear strength of the clay. The total overburden pressure depends on the critical depth of the excavation H_{cr} (the maximum depth when the excavation is still stable), the unit weight of the soil ρ and on the surcharge load q.

The stability factor N_{cb} as shown in Fig 12 is a function of the inclination of the anchors (α), the penetration depth (D) of the sheet piles below the bottom of the excavation and the adhesion (c_a) between the sheet piles and the clay. At $\beta_p = 1.0$ the adhesion corresponds to the undrained shear strength of the soil c_u. At $\beta_p = 0$ the adhesion is equal to zero. It can be seen from Fig 12 that the stability factor N_{cb}
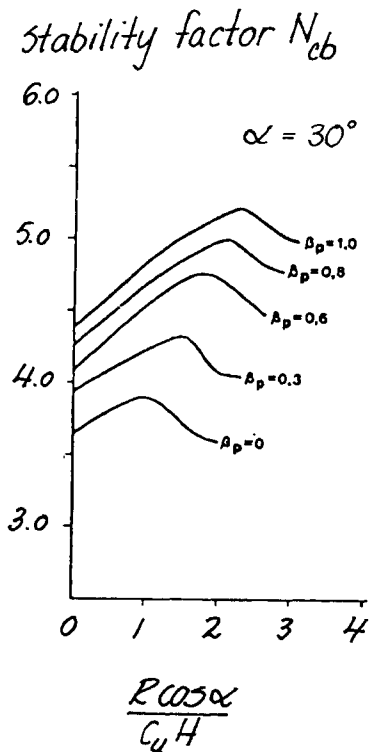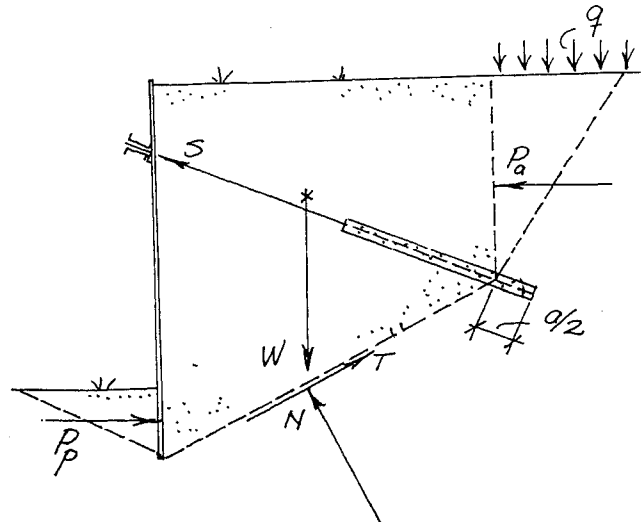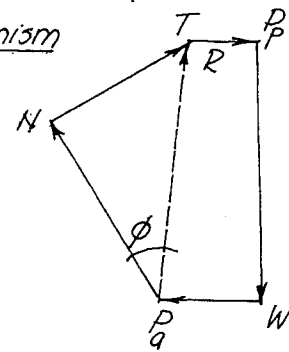

Fig 13   Total stability of an anchored sheet pile wall

Stability factor N_{cb}

Fig 12   Stability Factor N_{cb}

increases with increasing value on β_p and with increasing force R in the anchors until a critical value has been reached. If this critical value is exceeded then N_{cb} will decrease.

In order to simplify the calculations Sahlstrom and Stille (1979) have proposed for soft normally consolidated clay that the stability factor N_{cb} should be taken as 5.1 when the sheet piles are driven to a hard stratum so that the end bearing capacity of the sheet piles will be sufficient to resist the axial force caused by the inclined anchors. In the case the

sheet piles have not been driven to refusal in a hard layer and the vertical stability of the wall is low then a value on N_{cb} of 4.1 should be used in the calculations.

The stability may be reduced especially in silty clays when piles have been driven close to an existing sheet pile wall due to the remoulding of the soil and the resulting increase of the pore water pressures that take place during the driving. In this case a value equal to 3.6 on N_{cb} can be used.

In most cases failure takes place in the undisturbed soil between the flanges $(\beta_p = 1.0)$ of the sheet piles since the perimeter area is large. Usually a layer of clay will cling to the surface and come up together with the sheet piles when they are pulled.

The length of the anchors should be sufficient so that the stability of the sheet pile wall will be adequate with respect to a deep-seated failure. In Fig 13 is shown the forces acting on an anchored sheet pile wall in a cohesionless soil and the corresponding force diagram. The rear face of the indicated sliding wedge had to resist the lateral earth pressure P_a. The required passive earth pressure $P_{p'req}$ at equilibrium can be determined as shown in Fig 13 (Broms, 1968) which is a modification of the Kranz method which is

1525

widely used in Germany and Austria. It has been assumed in the analysis that the critical failure surface is located (a/2) from the end of the anchors, where a is the spacing of the anchors. It has thus been assumed that the inclination of the failure surface behind the anchors is (45° + 1/2 ϕ'). The main advantage with the proposed calculation method is its simplicity.

It is also necesary to check the stability of the wedge located above the fixed anchor length as illustrated in Fig 14. The failure surface has been assumed to extend a distance (a/2) from the end of the anchor block as shown. The passive resistance of the soil in front of the sliding soil wedge should be sufficient to resist the lateral displacement of the wedge. It is proposed to use partial safety factors and load factors in the calculations.
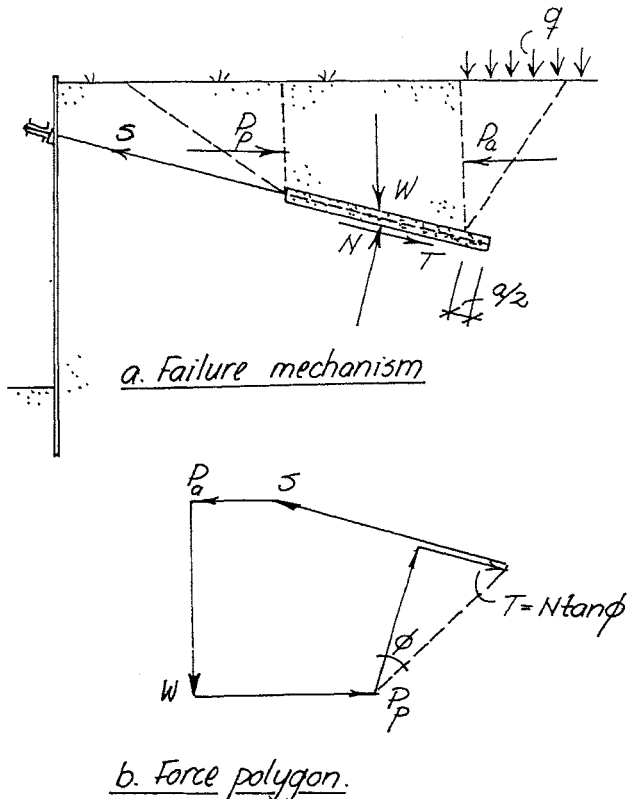

a. Failure mechanism

b. Force polygon.

Fig 14   Stability of anchor block

STRENGTH OF ANCHORS

The design of ground anchors has been reviewed by Littlejohn (1970). The method that can be used to calculate the tensile resistance of soil anchors is illustrated in Fig 15. The ultimate tensile resistance Q_{ult} depends on the friction resistance Q_{skin} along the grouted part of the anchor and on the end resistance Q_{end} as expressed by the relationship
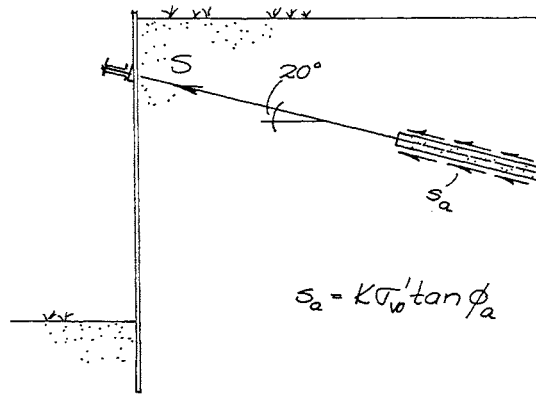
$$Q_{ult} = Q_{skin} + Q_{end} \qquad (9)$$

Fig 15   Tensile resistance of ground anchors

The displacement required to develop the maximum skin friction is small, a few mm, compared with the relative large displacement which is required to mobilize the end resistance.

In cohesionless soils (sand and gravel) the pull-out resistance (s_a) depends on the effective overburden pressure σ'_{vo} and on the friction angle ϕ'_a between grouted part of the anchors and the soil as expressed by the equation

$$s_a = K\sigma'_{vo} \tan \phi'_a \qquad (10)$$

The friction angle ϕ'_a is normally assumed to correspond to the angle of internal friction of the soil ϕ' or ϕ_d. The coefficient K depends mainly on the relative density of the soil. This coefficient can for dense, coarse and wellgraded sand or gravel be as high as 2 to 3 due to the dilatancy of the soil. In loose fine sand and silt the coefficient K can be as low as 0.5. The assumed value on K should be verified by load tests.

The tensile resistance can also be estimated from the grout pressure used during the installation of the anchors, from the grout pressure required for the expansion of the expander bodies or from the penetration resistance as determined by e.g. cone penetration tests (CPT), standard penetration tests (SPT) or weight soundings (WST).

It is proposed to use the equations suggested by Baquelin et al (1978) for bored piles to estimate the pull-out resistance from the maximum grout pressure (p_{grout}). The tensile resistance of the anchors increases generally with increasing grout pressure especially in hard rock and in dense sand and gravel. The capacity of the anchors will also increase with increasing length of the grouted zone, the fixed anchor length. In sand and gravel there is, however, a maximum effective length. If this effective length is exceeded then there is no further increase of the anchor force. The critical length is about 6 m for sand and gravel. Cyclic loading will, however, reduce this length. The fixed anchor length is usually 3 to 6 m.

1526

According to Baguelin et al (1978) the net base resistance of a bored pile q_{end} can be evaluated from the limit pressure p_ℓ determined from pressuremeter tests

$$q_{end} = k (p_\ell - p_o) \tag{11}$$

where p_o is the initial total horizontal pressure in the ground at the base of the pile and k is a coefficient that depends on the embedment length and on the magnitude of the limit pressure.

It is expected, however, that the limit pressure will correspond to the maximum grout pressure.

$$p_\ell = p_{grout} + z\, \rho_{grout} \tag{12}$$

where p_{grout} is the grout pressure at the ground surface, ρ_{grout} is the unit weight of the grout and z is the depth.

For the case the tensile resistance corresponds to 70% of the ultimate bearing capacity of a bored pile then the end resistance of the anchors can be calculated from the equation

$$Q_{end} = 0.7\, k\, p_{grout}\, A_{end} \tag{13}$$

where k is a coefficient that depends on the embedment length and on the magnitude of the limit pressure and A_{end} is the cross-sectional area.

The unit skin friction resistance f_s of a pile in sand or gravel will normally be 0.5% to 2% of the point resistance (Meyerhof, 1956). The skin friction will generally increase with decreasing particle size and increasing cone resistance. It is suggested for sand and gravel that the skin friction resistance should be taken as 1% of the unit end resistance. For silt 2% is proposed.

The total skin friction resistance Q_{skin} of the expander bodies will be 12% of the total end resistance for sand and gravel and 24% for silt. Then for sand and gravel

$$Q_{ult} = 0.78\, k\, p_{grout}\, A_{end} \tag{14}$$

where k is a bearing capaicty factor which depends on the embedment depth. For silt

$$Q_{ult} = 1.24\, Q_{end} = 0.86\, k\, p_{grout}\, A_{end} \tag{15}$$

The ultimate pull-out resistance of the expander bodies as determined by Equs (14) and (15) has been plotted in Fig 16 as a function of the maximum grout pressure. It can be seen that the tensile resistance increases rapidly with increasing grout pressure. It should be observed that the depth of the expander bodies should be at least eight times the diameter. Otherwise the resistance will be reduced.

The tensile resistance can also be calculated from the penetration resistance of different penetration tests such as cone penetration tests (CPT) standard penetration tests (SPT) and weight soundings (WST). A comparison between the different penetration tests is shown in Table I for cohesionless soils (silt, sand and gravel). For example, a standard penetration resistance (N_{30}) of 30 blows/0.30 m in a medium sand
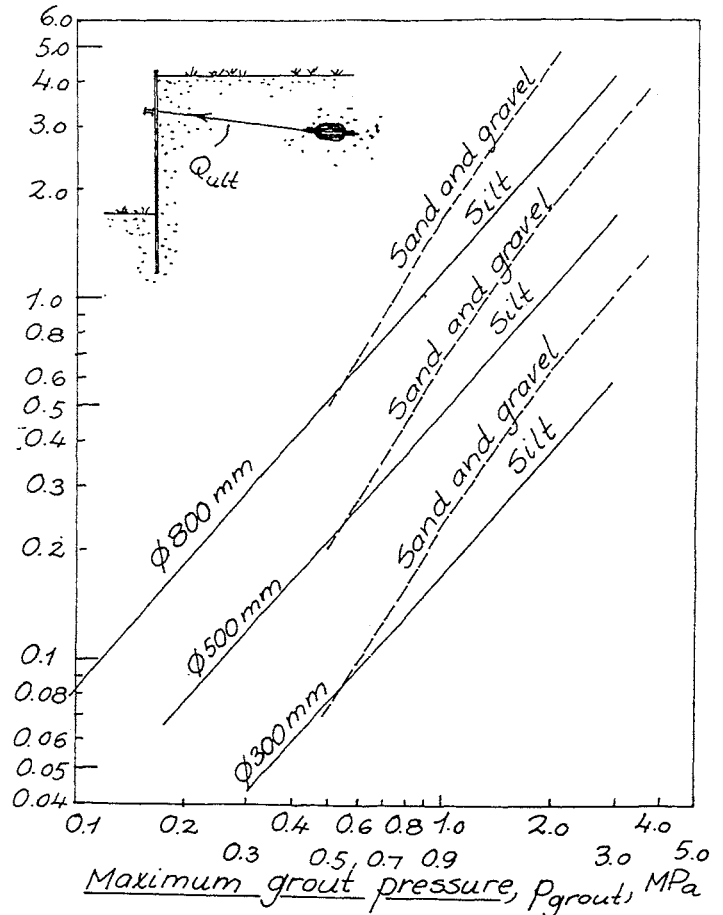

Fig 16   Pull-out resistance of Expander Bodies

corresponds a cone penetration resistance of about 10 MPa. It should be noted that the results are affected, for example, by the particle size, the depth below the ground surface and the location of the ground water level. For silt, sand and gravel the cone penetration resistance in MPa is approximately $0.2\, N_{30}$, $0.4\, N_{30}$ and $0.6\, N_{30}$, respectively.

However, the result from the weight soundings are at large depths (> 10 m) influenced by the friction along the sounding rod since a casing is not used, while at SPT the results are affected by the method used to lift and to release the hammer. The energy delivered by a free falling hammer is considerably higher than that when the rope and pulley method is used.

Load tests indicate that the end bearing capacity corresponds closely to the cone penetration resistance (CPT) within a zone that extends one pile diameter below and 3.75 pile diameters above the pile point (van der Veen and Boersma, 1952). In cohesionless soils the tensile resistance will be lower than the end bearing capacity because of the reduction of the over-burden pressure as mentioned above. It is, therefore,

1527

TABLE I

COMPARISON BETWEEN DIFFERENT PENETRATION TESTS

(after Broms and Bergdahl, 1982)

Relative Density	Cone Penetration Tests (CPT), Point Resistance q_s, MPa	Standard Penetration Tests (SPT), Penetration Resistance N_{20}, blows/30 cm	Weigth Sounding Tests, Penetration Resistance N_w, ht/0.2 m
Very loose	< 2.5	< 4	< 4
Loose	2.5 – 5	4 – 10	10 – 30
Medium	5 – 10	10 – 30	30 – 60
Dense	10 – 20	30 – 50	60 – 100
Very dense	> 20	> 50	> 100

suggested that the tensile resistance of soil anchors should be taken as 70% of the bearing capacity of an equivalent pile.

Test data indicate also that the tensile resistance of the expander bodies will decrease with increasing diameter. It is, therefore, suggested that the unit tensile resistance of 0.5 m and 0.8 m diameter expander bodies should be taken as 80% and 50%, respectively of the resistançe of expander bodies with 0.3 m diameter.

The net end resistance in clay can be estimated from

$$q^*_{end} = 9\ c_u \qquad (16)$$

when the anchor is located at least four diameters below the ground surface.

Also the skin resistance (c_a) will depend on the undrained shear strength c_u of the clay

$$s_a = \alpha\ c_u \qquad (17)$$

where α is a reduction coefficient which decreases with increasing shear strength. It is suggested that α should be taken as 0.8 for soft clays ($c_u \le 50$ kPa) and as 0.5 for medium to stiff clays when $c_u > 50$ kPa.

It should be noted that the tensile resistance will gradually increase with time after the installation due to the reconsolidation of the clay. Particulary the skin friction resistance is affected. About 1 to 3 months will be required in soft clay to reach the maximum resistance while in medium to stiff clay the calculated tensile resistance usually will be obtained within a few weeks. In weathered rock and residual soils a value 0.45 c_u is commonly used. The tensile resistance can be increased further by enlarging the boreholes by underreaming.

The pull-out resistance of ground anchors in rock has been correlated with the unconfined compressive strength. The allowable shear resistance is often taken as 0.1 q_u where q_u is the unconfined compressive

strength of small diameter rock cores. The maximum shear resistance is normally limited to 4 MPa. However, the spacing and the orientation of the joint in the rock can have a large influence on the pull-out resistance. The reduction of the shear resistance has been related to the RQD-value of the rock. Failure of rock anchors located close to the ground surface (D < 1.5 m) often occurs when a cone of rock is pulled out together with the anchor rod or the cable. The tensile resistance will in that case correspond to the weight of the rock cone and thus to the unit weight of the rock mass.

SETTLEMENTS AND LATERAL DISPLACEMENTS

Deep excavations in soft clay can cause settlements around the excavation. As a result surrounding buildings can be damaged. The damage can be related to either the angular distortion, the relative deflection (sagging and hogging) or the lateral deformation of the building. Buildings are in general more affected by large relative deflections or by large lateral deformations than by an angular distortion. Structures are also more sensitive to hogging than to sagging. Buildings located close to an excavation are often loaded in compression while buildings located further away are subjected to lateral tension (elongation) and may therefore crack. The location of the building within the settlement trough around an open excavation is thus important.

The lateral displacement of the soil around deep excavations and its effect on nearby buildings has attracted so far relatively little attention. The resulting lateral movement can damage buildings close to the excavation and other structures. A tensile strain of only 0.1% to 0.2% is often sufficient to cause extensive cracking of masonary structures. E.g. O'Rourke (1981) has observed large lateral strains behind an 18 m deep excavation. The resulting lateral displacements were high enough to cause extensive cracking of masonary structures located up to 9 m behind the excavation.

Some settlements will always occur even when the best available construction technique is been used and the

1528

soil conditions are favourable. The installation of the top strut is particularly important. When the costs of different methods to reduce settlements are estimated, it is important to consider also the indirect costs e.g. loss of time and business caused by congestion around the site due to the construction. Grouting and freezing require, for example, space for drilling rigs, mixing and refrigeration units, pipes and pumps as well as for storage of various chemicals and aggregates. For a particular job it is important that the total costs including indirect costs should be as low as possible.

The finite element method (FEM) provides an alternative approach to analyze deep excavations with respect to settlements and lateral displacements. This method can handle complicated soil and boundary conditions. The nonlinear behaviour of the soil and of the support system can be considered as well as the construction sequence. Many case records have been reported in the literature where FEM has been used to analyze the results (D'Appolonia, 1971; Clough and Davison, 1977; Burland et al, 1979; Karlsrud et al, 1980; Mana and Clough, 1981 and O'Rourke, 1981). Both braced and anchored excavations have been investigated (Egger, 1972; Clough and Tsui, 1974; Stroh and Breth, 1976, Clough and Mana, 1977 and Clough and Hansen, 1981). These studies show that the settlements and the lateral displacements of sheet pile or diaphragm walls in soft clay are to a large extent affected by the factor of safety with respect to base heave, by the stiffness of the wall, by the support system (ground anchors and struts), by the geometry of the excavation and by the chosen construction method.

Settlements should be measured frequently during the excavation by level surveying and the results should be plotted and evaluated so that remedial measures, if necessary, can be taken in time. Inclinometers can be e.g. used to determine the lateral displacements of sheet pile or of diaphragm walls. There are inclinometers available with a high resolution (1:10,000) so that lateral displacements as small as 1 to 2 mm can be detected. FEM can be helpful to locate the source of the settlements or of the lateral displacements. Lee et al (1986) have recently described the monitoring of a deep excavation in soft clay in Singapore.

The lateral displacements of braced and anchored sheet pile or of diaphragm walls depend to a large extent on the stiffness of the walls. The displacement is often expressed in terms of a stiffness factor $E_s \ell^4 / E_w I_w$ where ℓ is the vertical spacing of the struts or the anchors, E_s and E_w are the moduli of elasticity of the soil and of the wall material respectively, and I_w is the second area of moment of the sheet piles.

Field observations as well as FEA indicate that it is important to place the struts as soon as possible after the excavation has reached the strut level. Frequently the struts are not installed until the excavation had advanced an additional two to three meters. In that case the settlements and the lateral displacements can easily increase 50% to 100%. It is also important that the wale beams are tightly wedged against the sheet piles in order to reduce the settlements. Gaps should be filled with concrete or be shimmed.

The lateral displacements can be reduced by increasing the stiffness of the wall or by decreasing the vertical spacing of the struts or of the anchors E.g. a diaphragm wall can be used instead of sheet piles. Anchors are very effective since they can be placed close to the bottom of the excavation and be preloaded. Raked struts and temporary berms can then be avoided.

One difficulty with FEM is the choice of parameters since they should reflect both the in-situ behaviour of the soil or of the rock as well as the effect of e.g. workmanship and time. It is important to check the calculations at an early stage with field measurements. The design should be reanalyzed using back soil properties if the discrepancy is large.

INSTALLATION OF SHEET PILES

It is often difficult to drive the sheet piles sufficiently deep into the underlying rock in order to provide sufficient lateral resistance so that the high lateral earth pressure behind the wall can be resisted especially when the depth of the excavation is large. This is frequently the case in Sweden where the soft clay often is underlain by unweathered hard granite with a compressive strength of 150 to 200 MPa or more. Steel dowels are often used which are driven into the rock or placed in predrilled holes and grouted in order to increase the lateral resistance of the sheet piles as illustrated in Fig 17a. The drilling is normally done through steel pipes which have been attached to the sheet piles before the driving. The lateral resistance of the steel dowels depends on the strength of the rock and on the dimensions of the dowels. It is also possible to install additional ground anchors close to the bottom of the excavation as shown in Fig 17b to increase the lateral resistance of the sheet piles in order.

Another common case is illustrated in Fig 18a where it has not been possible to drive the sheet piles sufficiently deep because of stones or boulders in the soil which interfere with the driving. Additional anchors may be required at the toe of the sheet piles in order to increase the lateral resistance. However, an additional row of anchors will increase the vertical force in the sheet pile which has to be considered.
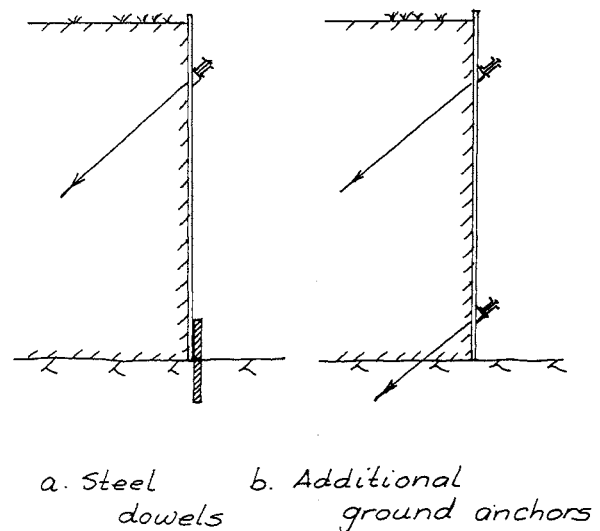

a. Steel dowels   b. Additional ground anchors

Fig 17   Prevention of toe failure for anchored sheet pile walls

1529

Fig 18   Vertical stability of anchored sheet pile walls

transferred through the walls. In this case, the columns will function as an additional level of struts below the bottom of the excavation. The required spacing of the lime or cement columns depends on the increase of the shear strength that can be obtained with lime (quick lime) or with cement. This can be investigated in the laboratory by mixing the clay with different amounts of lime and cement. The optimum lime content is usually 6% to 10% with respect to the dry unit weight. About 15% to 25% cement is usually required in order to reach the required shear strength of the stabilized soil. Gypsum in combination with quicklime can be beneficial in organic soils.

The columns will increase the average undrained shear strength of the soil. In soft clay the average shear strength can usually be doubled if the 0.5 m diameter lime or cement columns are spaced 1.4 to 1.5 m apart. Lime or cement columns can also be placed behind the sheet piles in order to reduce the lateral earth pressure acting on the wall.

The soil at the ground surface has been excavated in Fig 20 in order to reduce the total overburden pressure at the bottom of the excavation. The reduction of the lateral earth pressure on the wall will be large below the excavation especially when the total overburden pressure at the bottom of the excavation is approximately equal to $N_c c_u$. The excavated soil can be replaced by light weight fill e.g. expanded shale, slag or flyash. In the Scandinavian countries and in Finland sawdust, bark and peat are often used. With slag or flyash, pollution of the ground water might become a problem.

Also jet grouting and quick lime columns can be used to increase the stability as shown in Fig 20 as has been the case in Singapore. At the quicklime column method
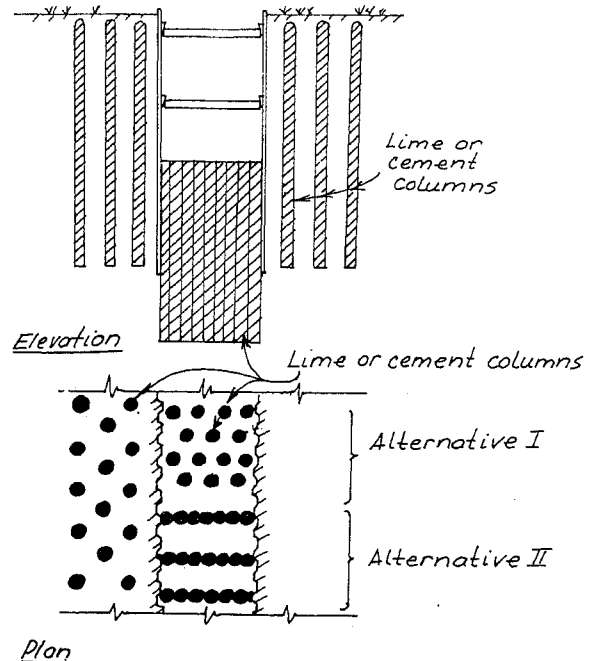
Erosion may even occur below the boulders or the stones if the surface of the cut is not protected by, for example, shotcrete. Drain holes will be required to reduce the high water pressure that otherwise may develop behind the shotcrete layer.

Fig 18b illustrates the case when the vertical stability of the sheet pile wall is not sufficient and the vertical force in the sheet piles from the inclined anchors will cause the sheet piles to settle. The vertical stability of the wall can be increased by driving steel H-piles in front of the wall as shown. The H-piles should be welded to the sheet piles so that the vertical force from the anchors can be transferred to the piles. The bearing capacity of the H-piles should be sufficiently high so that they will be able to carry the vertical force.

IMPROVEMENT OF THE STABILITY IN SOFT CLAY

Different methods can be used to increase the stability of braced or anchored sheet pile wall in soft clay as illustrated in Figs 19 through 22. Lime or cement columns have been installed in Fig 19 in front of or between the two rows of sheet piles in order to increase the average shear strenght of the clay and thus the passive resistance of the soil.

The lime or cement columns can also be installed in such a way that they form a series of continuous walls between the two sheet pile walls to keep them apart. The lateral earth pressure acting on the sheet piles below the bottom of the excavation will then be

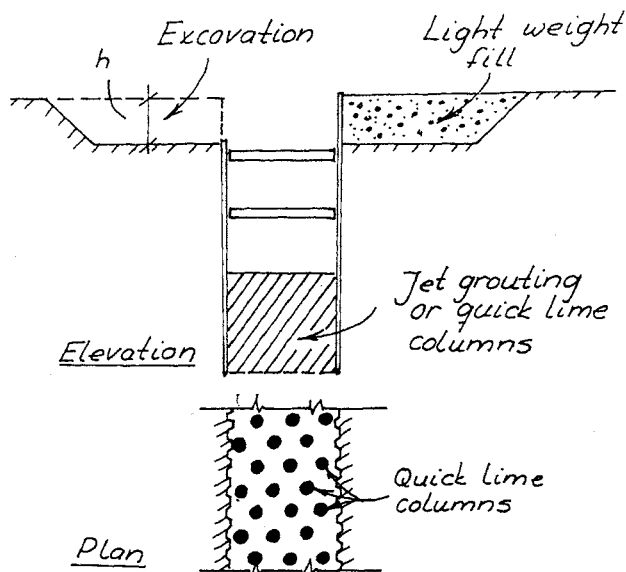Fig 19   Stabilization with lime or cement columns

1530

Fig 20   Stabilization with light-weight fill, jet grouting or quicklime columns
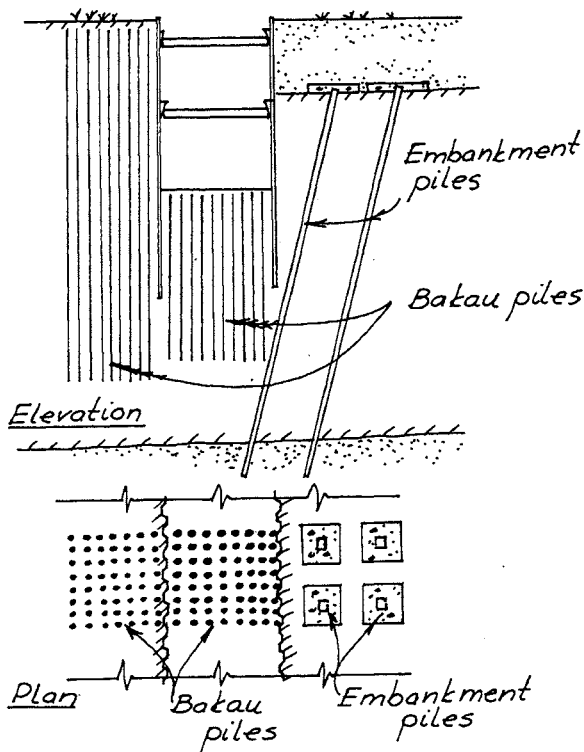

Fig 21   Stabilization with Bakau piles and embankment piles

large diameter holes which are filled with quicklime are used. At this method, the expansion that takes place when the unslaked lime reacts with water is utilized. The method is mainly effective in silty soils with a low plasticity index where a small change of the water content will have a large effect on the shear strength. The effectiveness of the method is, however, reduced when the soil is stratified. Then the expansion of the quicklime columns will occur faster than the consolidation of the soft soil around the columns. As a result, the soil will be displaced and heave rather than consolidate.

Embankment or Bakau piles are used in Fig 21 in order to reduce the lateral earth pressure acting of the sheet pile wall. The piles will carry part of the weight of the clay due to the friction or adhesion along the piles. The efficiency of the embankment piles can be increased if the piles are provided with concrete caps which will transfer the weight of the soil above the caps to the piles. Pile caps are required especially when concrete or steel piles with high bearing capacity are used because of the large length required to transfer the load from the soil to the piles though adhesion or friction along the piles. The transfer length will be large because of the relatively high pile loads which are required in order to make the method economical. Embankment piles are common in Sweden, Finland and Norway particularly in soft clay. Bakau piles are extensively used as embankment piles in Southeast Asia. They have the advantage that the surface area is large, that the transfer length is small and that they are cheap. The diameter is usually 80 to 100 mm. The maximum length is about 6 m. If longer piles are required they had to be spliced.

The stabilizing effect of embankment piles is equivalent to that caused by an increase of the unit weight of the soil below the excavation bottom as illustrated in Fig 22. The equivalent unit weight γ_{eff} of the soil when the embankment piles are used to stabilize an embankment or slope can be estimated from the equation

$$\gamma_{eff} = \gamma + \frac{\pi\, d\, c_a}{a^2}$$

where d = diameter of the piles
c_a = adhesion of the clay along the piles
a = spacing of the piles
γ = unit weight of the soil between the piles

An example where an 7.6 m deep excavation in soft marine clay was successfully stabilized with 6 m long Bakau piles has been described by Broms and Wong (1985).

Other methods that have been used to increase the stability with respect to bottom heave are shown in Fig 27. The stability can be improved by driving a few sheet piles to a soil layer with high bearing capacity so that part of the weight of the soil can be carried by the skin friction along the sheet piles. It is also possible to use inclined anchors in order to increase the vertical stability of the sheet pile wall as shown. This method can be economical if there is a concrete slab next to the excavation. The stability can be increased as well by placing the bottom level of struts in trenches below the bottom of the excavation. Thereby the effective length of the sheet piles below the lower strut level will be reduced.
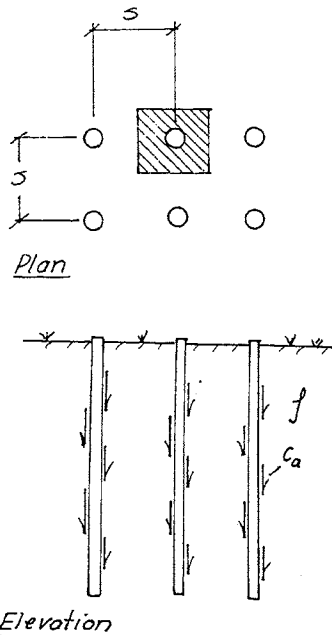
1531

Plan

Elevation

Soft clay

Anchor

Struts in trenches

Sheet piles driven into a firm layer

Sand

Fig 22   Increase of the equivalent unit weight using embankment piles
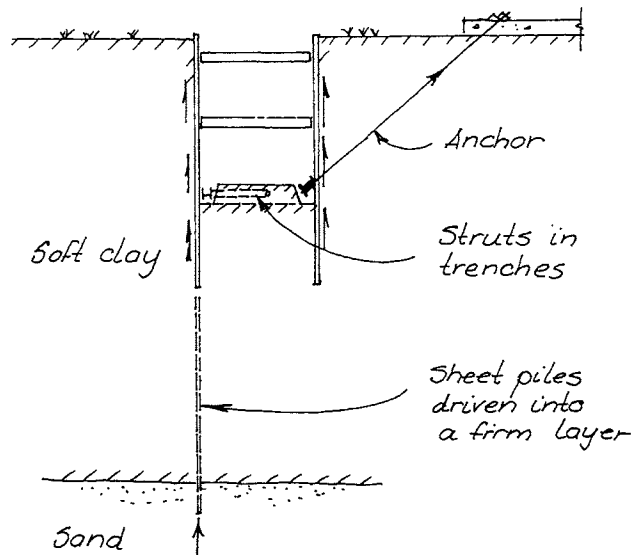
Fig 23   Inclined anchors and lowering of the strut level

FAILURE OF A SINGLE ANCHOR

The redistribution of the load that takes place when one or several of the anchors or struts fail has been investigated by Stille (1976) and by Stille and Broms (1976). In Fig 24 is shown the load redistribution that was observed for an anchored sheet pile wall at Mölntorp, Sweden in a very soft clay with an average shear strength of 18 kPa when one or two of the anchors were unloaded. For this sheet pile wall which was anchored at two levels it was observed that the maximum increase of the load in the adjacent anchors was 9% when one anchor was unloaded and that the load increased by an additional 8% when the load in a second anchor was released. It is interesting to note that the total increase of the load in all anchors was only 36% of the initial load in the unloaded anchor. Thus the total lateral earth pressure on the sheet pile wall decreased by 64% of the initial load in the unloaded anchor. When the second anchor was unloaded then the total increase of the load in the adjacent anchors was only 16% of the initial load in that anchor. Thus the total lateral earth pressure on the wall decreased by 84% with respect to the initial anchor load.

The corresponding load redistribution for a sheet pile wall at Bergshamra, Sweden with three anchor levels is shown in Fig 25. In this case (Panel B1) the maximum increase of load in the adjacent anchors was to 35% of the initial anchor force before the first anchor was unloaded. The total lateral earth pressure on the wall increased by 32% with respect to the initial anchor load. In a second panel (Panel C1) the maximum increase of the anchor force in the adjacent anchors was 14% with respect to the initial load when the load in one of the anchors was released. In this case the total lateral earth pressure on the wall increased by 4% with respect to the load in the unloaded anchor compared with a decrease of 64% at Mölntorp. The behaviour of this sheet pile was thus different. This difference in behaviour can be explained by the difference in mobilized shear strength of the clay behind the wall.
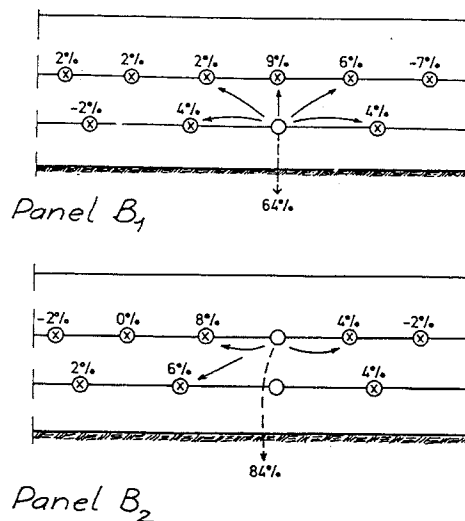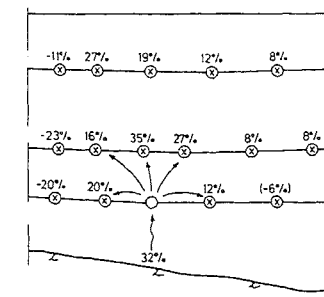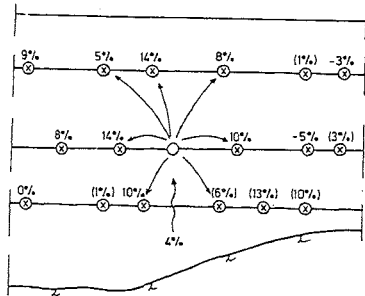

Panel B₁

Panel B₂

Fig 24   Load redistribution at Mölntorp, Sweden at failure of one or two ground anchors (after Stille, 1976)

1532

Panel B₁

Panel C₁

Fig 25   Load redistribution at Bergshamra, Sweden at
          failure of one or two ground anchors (after
          Stille, 1976)

Mobilization of
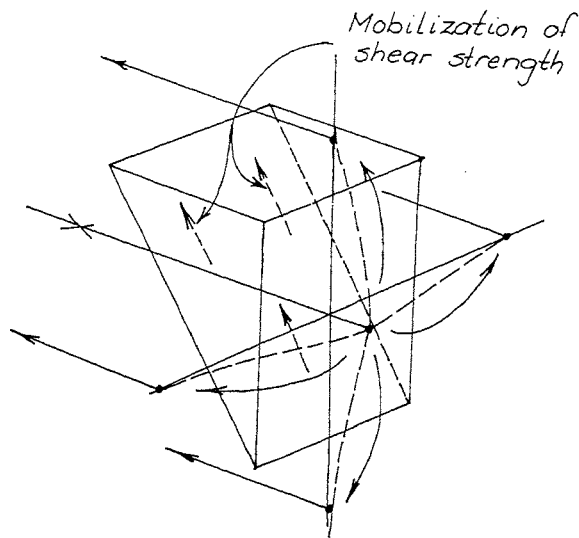shear strength

Fig 26   Load redistribution due to mobilization of
          shear strength

The lateral earth pressure acting on a braced or an anchored sheet pile wall depends on the lateral displacement required to mobilize the shear strength of the soil behind the wall and on the factor of safety used in the design. The wall will deflect laterally when the load in one of the anchors is released or the anchor fails. The increase of the lateral deflection of the wall is generally .sufficient to mobilize the shear strength of the clay along a potential failure surfaces behind the wall as illustrated in Fig 26. A relative small deflection is normally required to develop the maximum shear strength of even soft clay compared with the displacement required to develop the ultimate resistance of the anchors or of the struts. In the case the factor of safety initially is relatively high then only a small part of the available shear strength will initially be mobilized. A reduction of the force in one of the anchors will then mainly increase the average shear stress along potential failure surfaces in the clay. In this case, the increase of the load in the adjacent anchors will be small and the total lateral earth pressure on the wall will decrease when one of the anchors is unloaded or fails as was the case at Molntorp.

If on the other hand the factor of safety is low and the shear strength of the clay has been fully mobilized before the release of the force in one of the anchors then the failure of one of the anchor will result in a large increase of the load in the adjacent anchors. The total load on the sheet pile wall may even increase when the peak strength of the clay has been exceeded and the residual shear strength is lower than the peak strength. This was the case at Bergshamra where the total force acting on the sheet pile wall increased when the load in one of the anchors was released.

The consequences when one of the anchors fail will thus depend to a large part on the chosen factor of safety. If a relatively high factor of safety has been used in the design ($\geq$ 1.5) and only part of the shear strength of the soil will be mobilized at working loads then the increase of the load in the adjacent anchors will be small when one of the anchors fails. If on the other hand the factor of safety is close to 1.0 then the failure of one of the anchors will cause a large increase of the load in the adjacent anchors which also may fail. The total lateral earth pressure on the sheet pile wall may even increase and cause a progressive failure of the whole wall (zipper effect).

STABILITY OF THE BASE OF A SHEET PILE WALL

Several failure of anchored walls have been occurred in Sweden in soft clay. In Fig 27 is shown an anchored wall constructed of large diameter bored piles (Broms and Bjerke, 1973). The exposed clay between the piles was shotcreted during the excavation. Clay started to flow into the excavation below the shotcreted part of the wall almost like tooth paste squeezed out of a tube when the depth of the excavation was 5.5 m. Within a few minutes the excavation was filled with soft remoulded clay due to the high sensitivity of the clay. Failure took place when the total overburden pressure at the bottom of the excavation was about 6 c_u where c_u is the undrained shear strength of the clay as determined by field vane tests. The factor 6.0 corresponds to the stability factor N_{cb}. This type of construction using bored piles and shotcrete is therefore not suitable for soft clay when the depth of the excavation is large and the total overburden pressure at the bottom of the excavation exceeds $N_{cb}c_u$
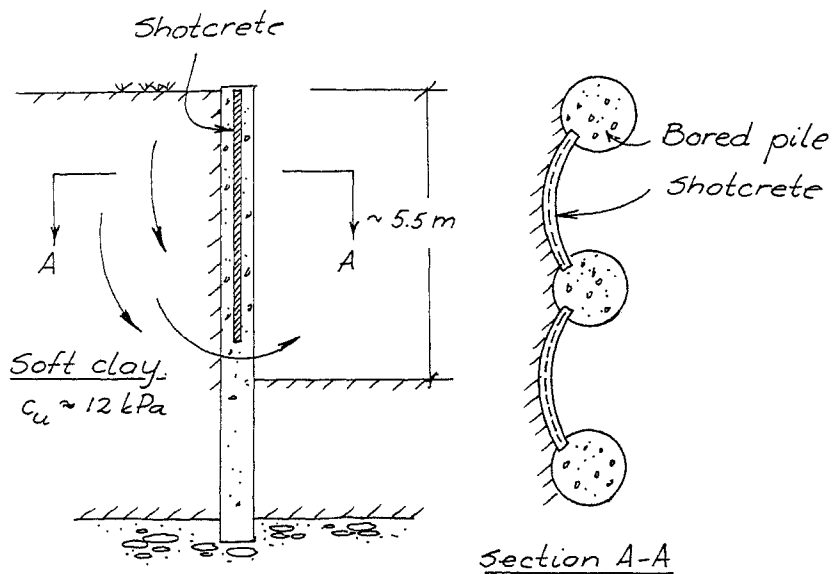
1533

Fig 27   Failure of a vertical cut in soft.clay (after
          Broms & Bjerke, 1973)

(Broms and Bennermark, 1967).   Steel sheet piles or contiguous bored piles should have been used instead.

Several failures have also occurred in Sweden when the sheet piles have been driven to rock through a deep layer of soft clay.   Because of the high compressive strength of the granite it is not possible to drive the sheet piles into the rock.   Soft clay was squeezed into the excavation through the triangular openings which were formed between the bottom of the sheet piles and the rock as shown in Fig 28 since the surface of the rock was inclined.   Large settlements were observed outside the wall.   The diameter of the depressions corresponded approximately to the depth of the excavation.

STABILITY OF DEEP EXCAVATIONS IN SOFT CLAY IN SINGAPORE

Three deep excavations in soft marine clay in Singapore have been analyzed using a modified version of the computer program EXCAV.   In the original program which was developed at the University of California at Berkeley by Chang and Duncan (1977) a non-linear hyperbolic soil model (Duncan et al, 1980) is utilized to describe the soil behaviour.   The program can model the excavation layer by layer, the installation and the preloading of the struts and the application of a surcharge load.

The first project involves a braced sheet pile wall, where the sheet piles have been driven into a deep stratum of soft marine clay.   In the second project the excessive plastic yielding of a braced sheet pile wall has been investigated.   The third project is concerned with the prediction prior to the construction of wall movements for a deep excavation in soft clay.

The short term conditions have been investigated with a total stress analysis using the undrained shear strength of the soft clay.   The soft marine clay has been assumed to be saturated and incompressible.   A Poisson's ratio of 0.495 has been used in the analysis. The elastic modulus (E_u) that corresponds to undrained

conditions has been assumed to 100 c_u to 200 c_u.   This equivalent modulus corresponds to the initial tangent modulus, E_i of the soft clay.   The tangent modulus, E_t, is a function of E_i and of the stress level.
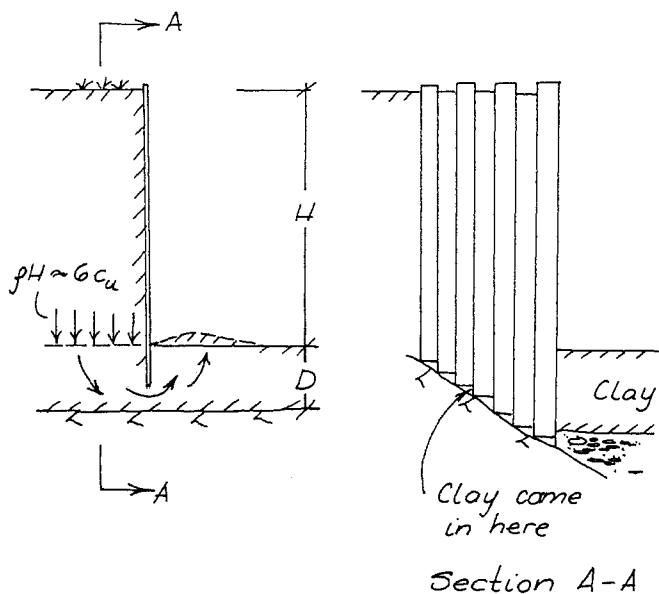

Fig 28   Failure by bottom heave

Project A. This project is located in the Central Business District (CBD) of Singapore (Fig 29). The size of the 11.1 m deep excavation is 42.6 m x 27.0 m. The walls of the excavation were supported by 30 m long sheet piles (FSP IIIA) which were driven 19 m below the bottom of excavation. Six levels of struts supported the wall. The vertical spacing of the strut varied between 1.5 m to 2.5 m. The horizontal spacing was about 6 m.

The excavation proceeded in stages. The struts supporting the sheet piles were installed during each excavation stage 0.5 m above the bottom of the excavation and they were preloaded to 15 percent of the design load. The site was divided into three sections during the excavation. In the present study the behaviour of the sheet pile wall in the middle section of the excavation has been analyzed.

Six slope indicator pipes were installed behind the sheet pile wall as shown in Fig 29. Surface monuments were established to determine the settlements behind the sheet piles. Strain gages were attached to selected struts in order to evaluate the strut loads.

A typical soil profile is shown in Fig 30. A sandy fill about 1 to 2 m thick is located at the ground surface . The fill was followed by a deep layer with soft marine clay which belonged to the Kallang Formation. The clay consists of two distinct members, an upper layer which is approximately 25 m thick and an approximately 7 m thick lower layer. The two layers are separated by a layer of loose to medium dense silty sand. A layer of stiff sandy silt, basically decomposed granite was found below the marine clay.
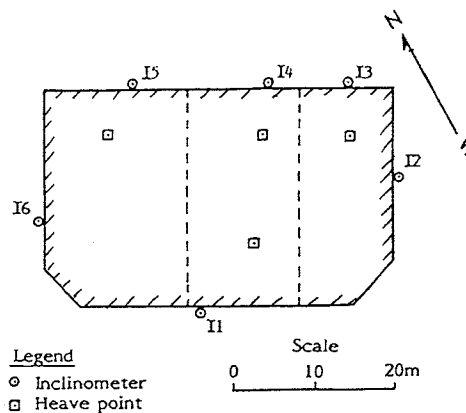

Fig 29   Instrumentation – Project A

The water contents of the upper and lower members of the soft marine clay were 70% and 50%, respectively. The liquid and plastic limits of the upper marine clay varied between 80% and 105% and between 60% and 70%, respectively. The liquid and the plastic limits of the lower marine clay were 70% and 50%, respectively.

Oedometer tests indicated that the marine clay was slightly overconsolidated. The undrained shear strength for the upper and lower members of the marine clay increased almost linearly with depth.
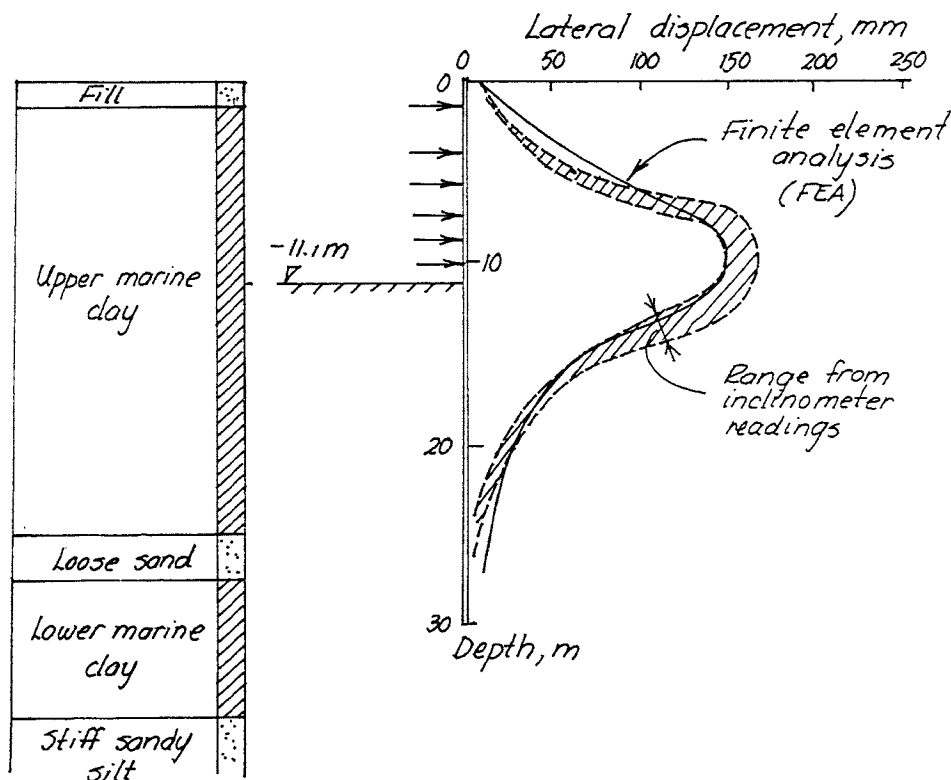

Fig 30   Measured and calculated wall deflections – Project A

1535

Field measurements indicated that the wall gradually moved inwards with increasing depth of the excavation. The maximum deflection of the middle section of the excavation was 150 to 170 mm when the excavation had reached its final depth of 11 m. This is about 1.5% of the excavated depth A comparison between the measured and the computed deflections is shown in Fig 30.

The observed surface settlements when the depth of excavation was 5.75 m and 11.1 m are shown in Fig 31. The lateral displacements of the wall thus caused large settlements that spread far behind the wall. The maximum settlement was about 1% of the final excavation depth. It occurred at a distance from the excavation equal to about half the excavation depth.

The measured settlements are plotted in Fig 32 as proposed by Peck (1969). It can be seen that the settlements even at a distance of 3.5 times the excavation depth were large. This behaviour can be explained the restraint of the lateral deformations and of the settlements of the sheet pile wall by the sand layer at the toe of the wall as illustrated in Fig 33. This was confirmed by FEM.

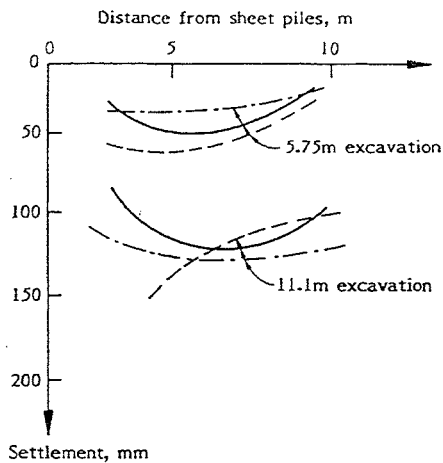Fig 32   Normalized settlements (Peck, 1969)

Fig 31   Measured ground settlements - Project A

The maximum bending moment in the wall has been back calculated from the curvature of the sheet piles which was determined from the inclinometer measurements. These measurements indicated that local yielding of the sheet pile occurred during the final stage of the excavation as indicated in Fig 34. The yield moment of FSP IIIA sheet piles is about 380 kN/m/m. The computed maximum bending moment by FEM was 372 kN/m/m. The finite element analysis also indicated that the wall was highly stressed down to about 6 m below the bottom of excavation.

Both field measurements and FEA indicate that the strut load increased rapidly with increasing depth of the excavation. The strut loads reached a maximum just before the installation of the next level of struts. Thereafter, the strut loads decreased slightly with increasing excavation depth.

A comparison between measured and computed strut loads is shown in Fig 35 for the top three levels. The measured strut loads agreed closely with those calculated by FEM. The pressure distribution determined by the tributary area method is shown in Fig
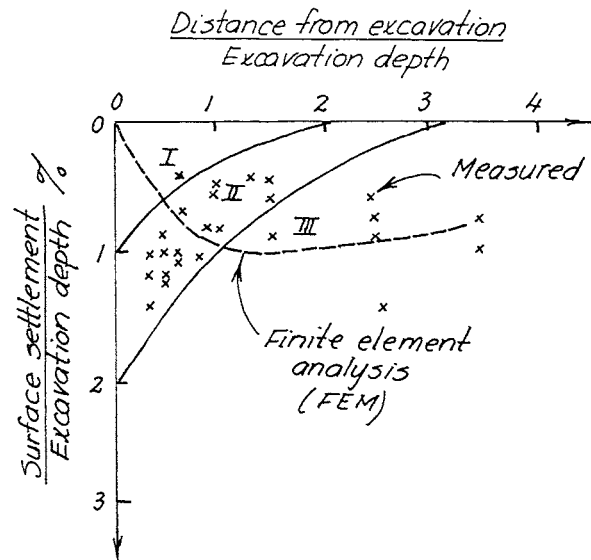
36. It can be seen that the measured and the computed values are in close agreement. It appears that the apparent pressure diagram proposed by Terzaghi and Peck (1967) at m = 0.4 is conservative. A better match is obtained with m = 0.7.

The penetration depth of the sheet piles (19 m) below the bottom of the excavation was 1.73 times the depth (11.1 m). An analyses using FEM indicated that the penetration depth could have been reduced by 13.5 m without any significant increase of the strut loads. This conclusion concurs with the observation by Peck (1969) that very little is gained in soft to medium stiff clay by driving the sheet piles far below the bottom of the excavation provided the stability of the excavation with respect to bottom heave is sufficient.
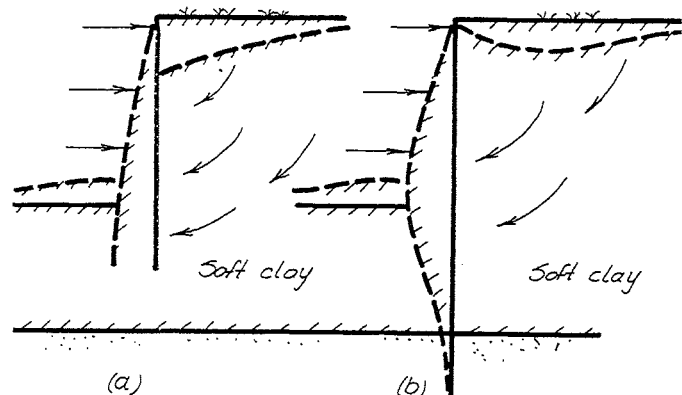

Fig 33   Relationship between settlements and lateral displacements - Project A
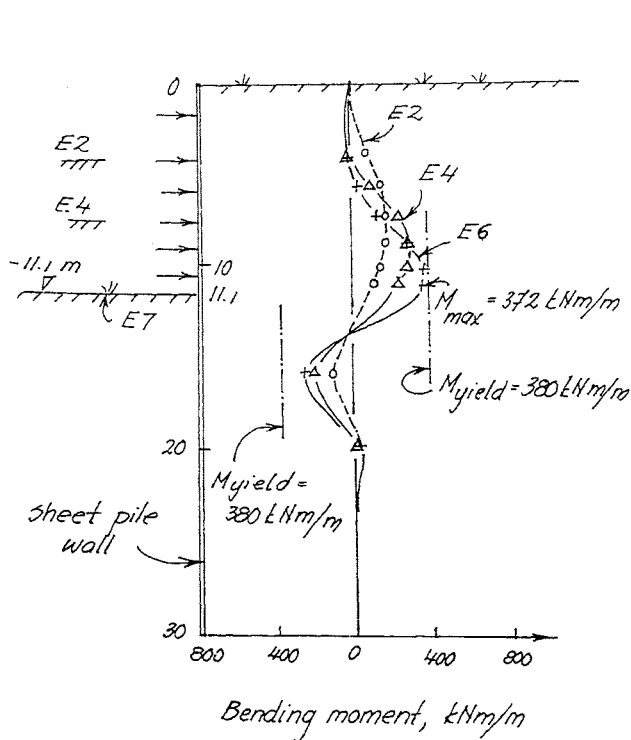
1536

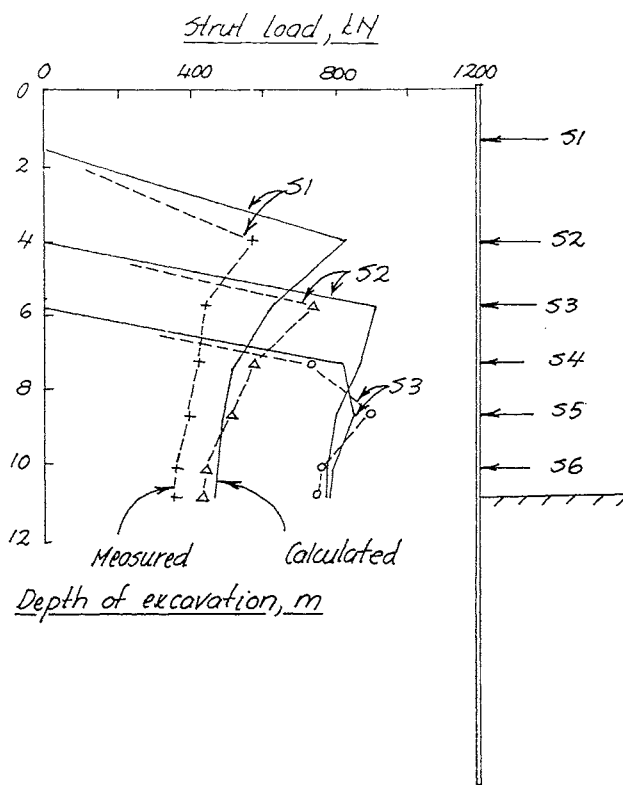Fig 34   Moment distribution at different stages of construction – Project A

Fig 35   Strut loads – Project A

Legend.
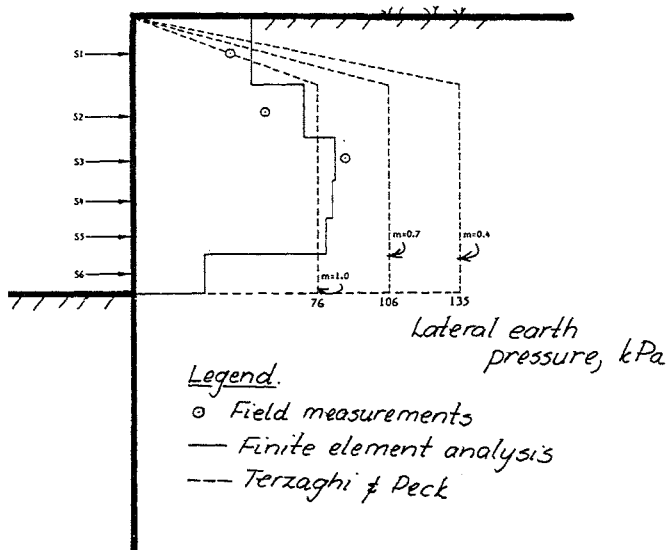⊙ Field measurements
—— Finite element analysis
--- Terzaghi & Peck

Fig 36   Comparison between measured and calculated strut loads

Project B. This project, which is located just outside the Central Business District in Singapore illustrates the influence of the construction sequence on the performance of braced excavations. The size of the 14.7 m deep excavation was 200 m x 35 m. A cross-section of the excavation is shown in Fig 37. FSP IV sheet piles with a total length of 18.5 m were driven 3.8 m below the bottom of the excavation (D = 0.26H). The sheet piles were supported at six levels. The vertical spacing of the struts varied between 2 m to 2.5 m. The horizontal spacing was about 5.5 m.

The soil condition at this site was highly variable. A soil profile along section A-A is shown in Fig 38. On the west side, the sheet piles were driven into a stiff sandy silt or clay (decomposed granite). On the east side, the soft marine clay extended the full depth of the excavation. The ground water level was located about 1.0 m below the ground surface.

The soil profile on the east side of the excavation is similar to that at Project A. The upper and lower members of the Kallang Formation with soft marine clay are separated by a layer of loose to medium dense sand. Below the marine clay is a deep stratum of decomposed granite, a stiff sandy silt or clay. The upper marine clay is organic (peaty) with an average undrained shear strength of about 10 kPa. The average undrained shear strength of the lower marine clay is 15 kPa. The decomposed granite has an estimated undrained shear strength of about 70 kPa. This material was very difficult to sample and to test.

1537

The soft clay on the east side was adopted in the FEM analysis since it is more critical than the stiff soil on the west side. The excavation was carried out in seven stages. It should be noted that the struts S1 were placed after level E1 had been reached (Fig 37). The excavation proceeded down to level E2 prior to the installation of the struts at this level. This sequence was continued down to level E7. The struts were preloaded to 70% of the design load.

The observed wall movements are shown in Fig 38a. The maximum deflection was 270 mm which is 1.8% of the final excavation depth. This deflection is relatively large since the sheet piles were driven into a stiff soil. The computed maximum deflection was only about 200 mm regardless of the strength and stiffness of the soils when the wall was assumed to be linearly elastic, i.e. non-yielding.

The computed deflections at the different stages of the excavation are shown in Fig 38b for the case when the wall yields. It can be seen that the computed lateral deflections are in good agreement with the measured values.

The maximum settlement, 100 mm, occurred at a distance from the excavation which corresponded to about one-half the excavation depth. The computed settlements fall within Zone I of the normalized settlement chart proposed by Peck (1969).

The measured strut loads were low. A comparison between the measured and computed apparent lateral earth pressures is shown in Fig 40. The measured loads were considerably smaller than those computed by FEM except for the two strut levels at the bottom of the excavation. One possible explanation of this behaviour
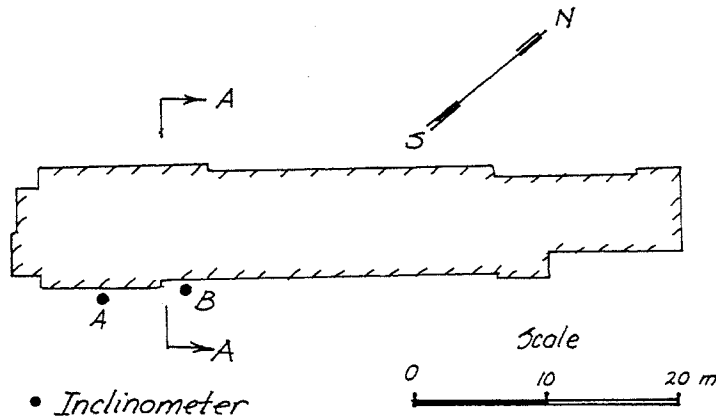
Site plan
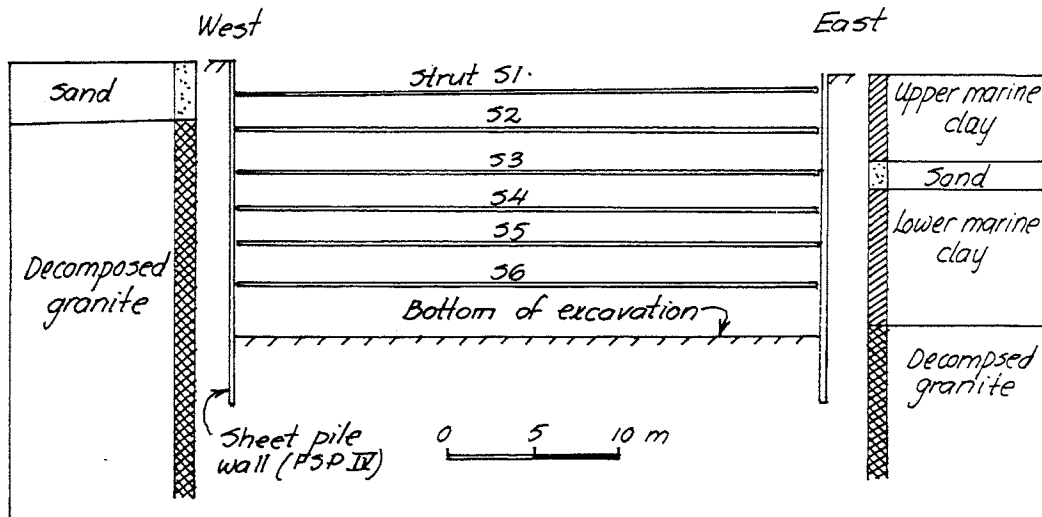
Cross-section A-A

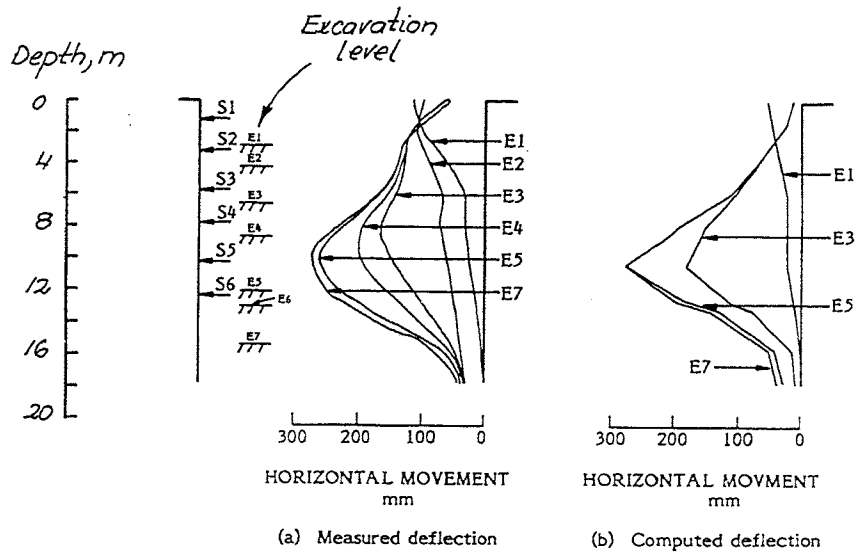Fig 37   Plan and cross-section for Project B

1538

(a) Measured deflection   (b) Computed deflection

Fig 38   Measured and computed wall deflections –
           Project B

is the stiff soil at the west side of excavation. It
has been assumed in the analysis that the soft clay
extended over the entire excavation.

The measured maximum wall deflection, 275 mm,
corresponds to about 1.9% of the depth of the
excavation which is rather high for a sheet pile wall
driven into stiff soil. This large deflection could
have caused by yielding of the sheet piles at an early
stage of the excavation.

The analysis indicates that yielding occurred when the
excavation reached Level E3, only 7 m below the ground
surface due to overexcavation prior to the installation
of the struts. Especially the first level of struts is
affected.

The installation of the struts lagged behind the
excavation of the soft clay by as much as 2.0 m which
undoubtly increased the bending moments in the sheet
pile wall. It is thus very important to limit the
difference between the strut level and the excavation
level as much as possible when the struts are
installed. This difference should not exceed 0.5 m.

The effect of the construction sequence was also
investigated assuming that the depth of the excavation
and the strut levels are the same when the struts are
installed. In this case, the computed maximum
deflection is only 120 mm as shown in Fig 40 which is
less than half the measured values. The computed
surface settlements and the strut loads were also much
smaller. In fact the maximum bending moment in the
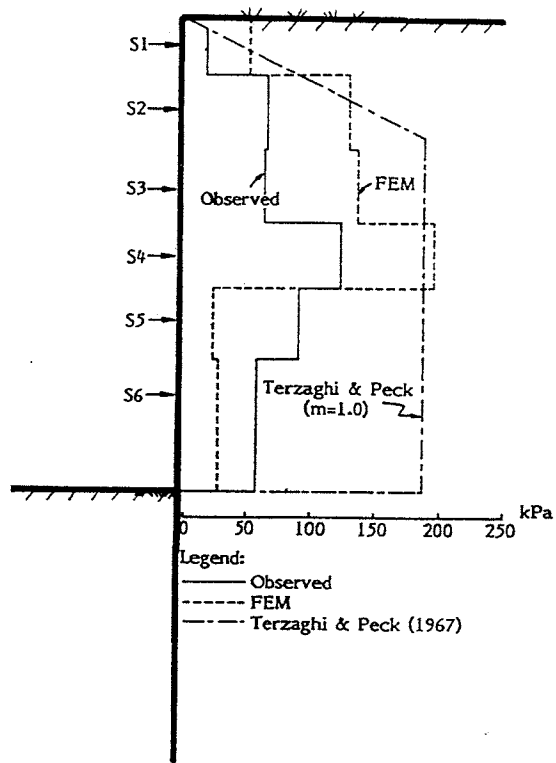wall never reached the yield moment of the sheet piles,
590 kN/m/m.

Legend:
——— Observed
------ FEM
—·— Terzaghi & Peck (1967)

Fig 39   Computed and measured lateral earth pressures
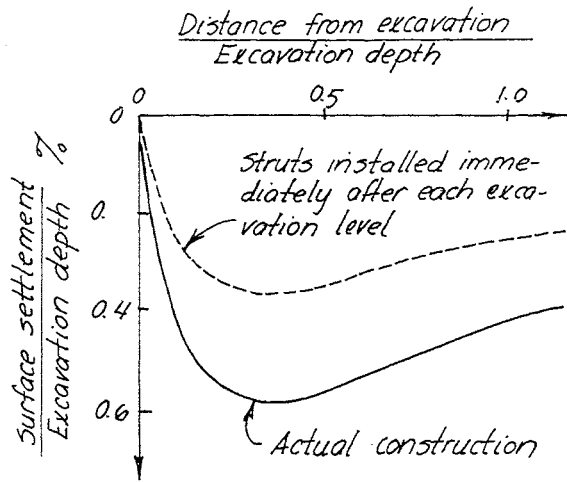   – Project B

1539

Fig 40   Computed surface settlements – Project B

Project C With the knowledge gained from Projects A and B, an attempt was made to predict the wall movement at Site C prior to construction. This site is located about one kilometer away from Project B. The length of the excavation is about 66 m. The width varies from 6.0 m to about 12.0 m as shown in Fig 42. The total depth is 15.0 m. The field instrumentation included one inclinometer pipe, a number of strain gages attached to selected struts and several survey markers.

Steel sheet piles (FSP VIL) supported at five levels were used as shown in Fig 42. The 26 m long sheet piles were driven 11 m (D = 0.73 H) below the bottom of the excavation. The vertical spacing of the struts varied between 2.0 and 3.5 m. The horizontal spacing was 3.7 m.
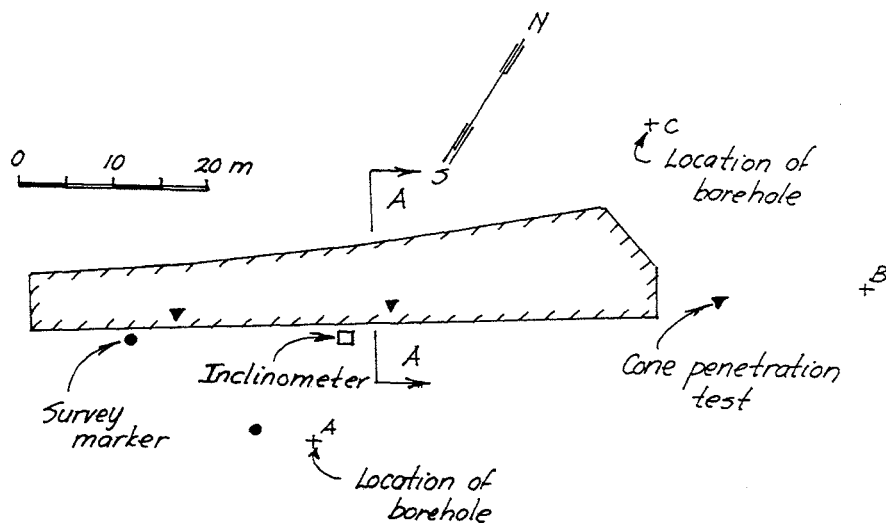
Two series of analysis were performed. The first was done prior to construction while the second series was carried out after the excavation had been completed. Soil data from only three boreholes were available prior to excavation. The soil conditions varied considerably between the three holes which were located relatively far from the site (Fig 42). Both the upper and the lower members of the soft marine clay were present in Borehole A whereas only the upper member could be found in Boreholes B and C. The depth to the bottom of the soft clay layer was 16.7 m at Borehole A, 11.5 m at Borehole B and 9.4 m at Borehole C.

The soil conditions at Borehole A was used in the analysis (Case I) since it was the closest of the three boreholes to the investigated section. The average undrained strength of the upper and lower layers of the soft marine clay was 10 and 15 kPa, respectively. This is about the same shear strength as that observed in Project B. Because of the close proximity and the similarity of soil conditions between Projects B and C, the soil parameters in Project B were used in the analysis. An E_u/c_u ratio of 150 was used for the upper layer since the upper marine clay was less peaty than at Project B. A value of 200 was used on the E_u/c_u-ratio for the lower marine clay.

A comparison of the measured and computed wall deflections after the excavation had reached the final depth is shown in Fig 43. The calculated maximum deflection, 75 mm, was only about half of the observed maximum deflection (150 mm).

A parametric study was done prior to the excavation because of the variable soil conditions, in order to assess the effect of the thickness of the soft clay. In one case, the soft marine clay was assumed to extend down to 20 m depth. This assumption was later verified by a cone penetration test (CPT) next to Section A-A. For this case, the computed maximum wall deflection was 142 mm (Fig 44) which agreed closely with the observed maximum deflection of 150 mm. The surface fill as well as the intermediate sand layer were assumed to be absent.

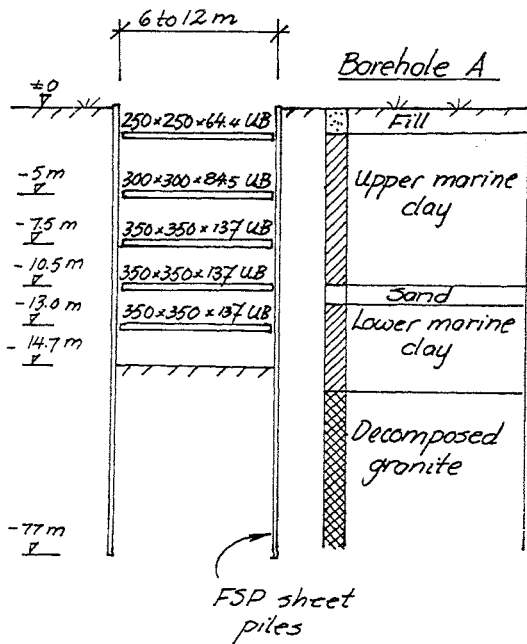Fig 41   Site plan – Project C

1540

Fig 42   Section A-A - Project C

Shortly after the excavation had been completed, a second series of analyses were carried out using an updated soil profile based on the cone penetration test next to Section A-A. There were no other changes. A comparison between of the computed and measured wall deflections at different stages of the excavation is shown in Fig 45. It can be seen that the computed

values were about 20 percent smaller than the measured values. However, the computed shape of the deflected sheet piles compares well with that which was measured.

There are a number of factors that can account for the relatively small computed wall deflections. It has been assumed in the analyses that the excavation depths and the strut levels were the same when the struts were installed. However, the excavation levels during the construction were at least 0.5 m lower than the strut levels. In fact, the first level of struts was not installed until the excavation was 2.0 m to 2.5 m below the ground surface. This accounts for the large lateral deflections observed at the first excavation level as shown in Fig 45. Furthermore, the lowest level of struts (S5) was not installed until the final depth of the excavation had been reached. This accounted for the large observed deflection during the final stage of the excavation. Also the measured ground settlements were much larger than those computed.

The strut loads were not measured at this project. The computed strut loads are shown in Fig 45. The high strut load at level S4 was caused by the intermediate sand layer. A similar phenomenon was observed at Project B.

The analysis indicates that an E_u/c_u-ratio of 100 to 200 gives reasonable results for the soft marine clay in Singapore with respect to settlements, lateral displacements and strut loads and that lateral deflections can be reduced significantly by installing the strut as early as possible and by preloading or prestressing the struts.

For a floating sheet pile wall in a deep stratum of soft clay, the depth of penetration has little effect on the overall behaviour. A penetration depth equal to one-half the excavation depth appears to be adequate provided that the critical depth will not be exceeded.
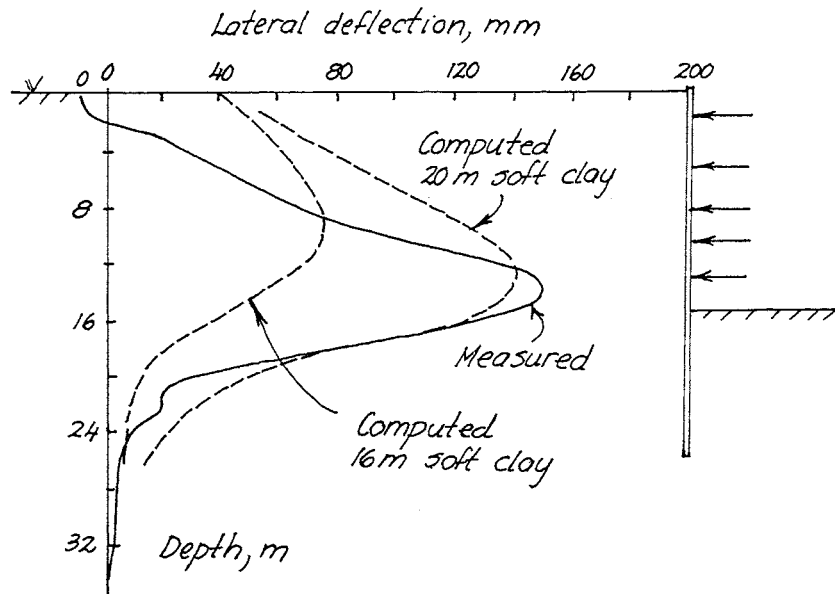

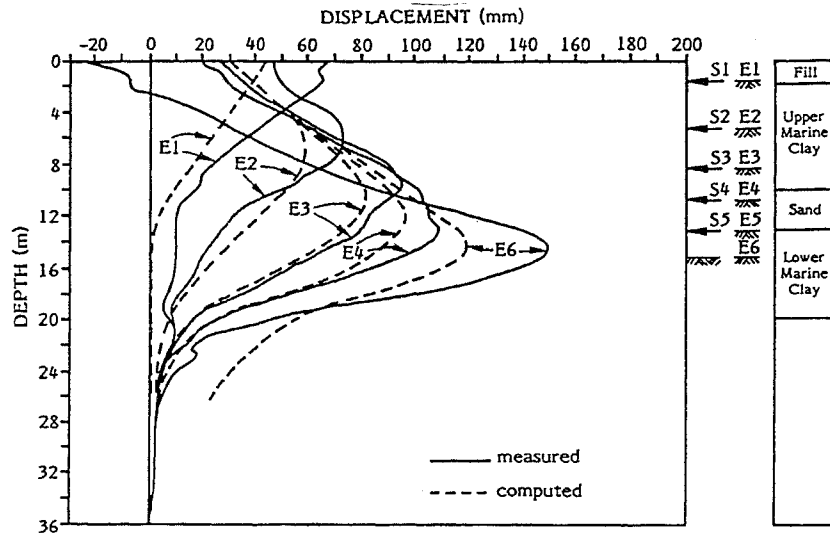Fig 43   Measured and calculated wall deflections - Project C

1541

DISPLACEMENT (mm)

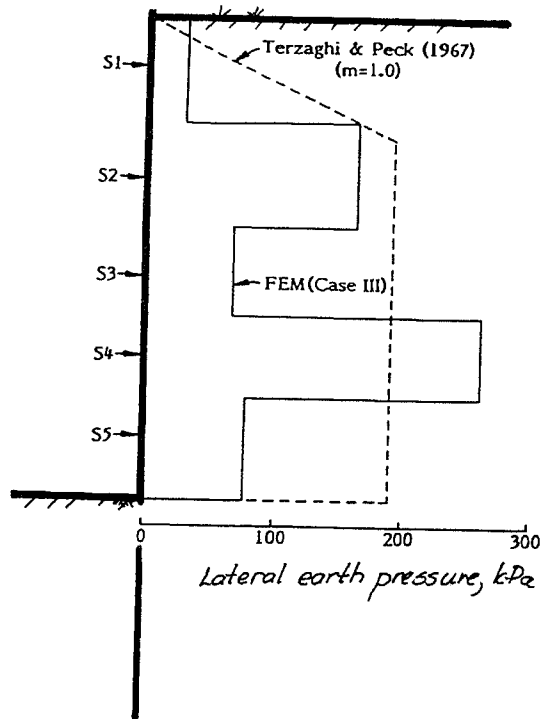Fig 44   Measured and computed wall deflections –
          Project C

Fig 45   Calculated earth pressures – Project C

STABILIZATION OF DEEP EXCAVATIONS IN SOFT CLAY

Bottom heave is frequently a problem for deep excavations in soft marine clay in Singapore. Failure of bottom heave can occur when the excavation depth exceeds about 5 m to 6 m due to the very low shear strength of the clay. Different methods can be used to increase the stability. The effect of jet grouting, excavation under water and embankment piles (soil nailing) has been investigated for a 11 m deep and 33 m wide excavation in soft clay using a modified version of the computer program EXCAV (Chang and Duncan, 1977). It has been assumed in the analysis that the sides of the excavation are stabilized by 33 m long sheet piles FSP IIIA which have been driven 22 m below the excavation bottom. The sheet piles are supported by struts at four levels. The vertical spacing of the struts is 2.5 m. The top level is located 1 m below the ground surface.

The 50 m deep layer with soft marine clay has been assumed to be slightly overconsolidated down to 11 m depth. Below it is normally consolidated. The undrained shear strength (c_u) is constant, 16 kPa, from the ground surface down to a depth of 11 m. Below, c_u = 16 + 1.25Z kPa where Z is the depth in metres below El. -11 m. The increase of c_u corresponds to a c/p ratio of 0.25 $(c_u/\sigma_v' = 0.25)$.

The short term conditions have been evaluated using a total stress analysis. The soft marine clay has been assumed to be saturated and incompressible. A Poisson's ratio of 0.495 has been used in the analysis. The E_u/c_u ratio has been assumed to be 200. A value of 0.9 has been used to estimate the lateral earth pressure at rest (K_o) with respect to the total stress.
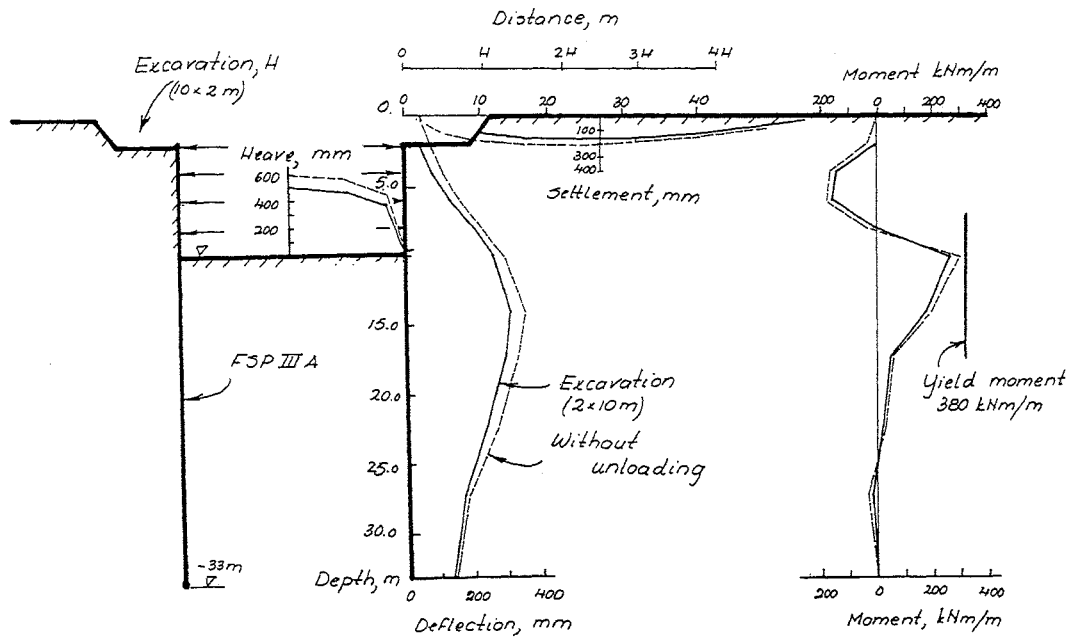
1542

Fig 46   Effect of unloading

These values have been found to be appropriate for deep braced excavations in soft marine clay in Singapore (Broms et al, 1986) and the predicted performance has agreed well with that which has been observed.
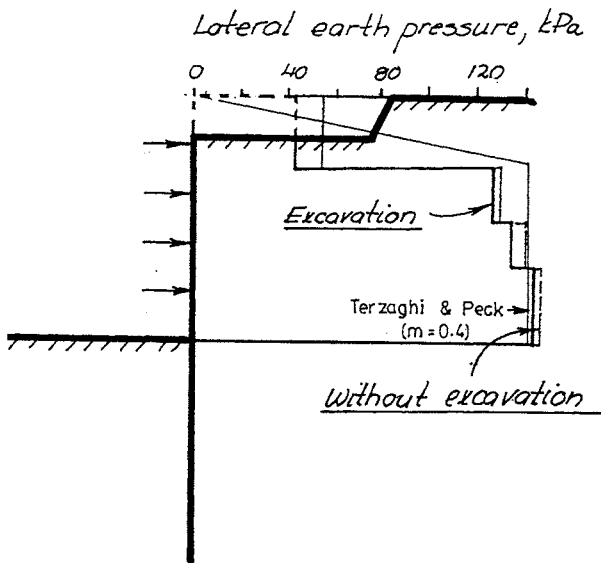

Fig 47   Effect of unloading on strut loads

The lateral deflections of the sheet pile wall are shown in Fig 46 when the depth of the excavation is 11 m. The maximum lateral deflection of the wall is about 400 mm. The ground settlements outside the excavtion and the base heave within the excavation are large as shown in the figure. The calculated maximum settlement and the maximum base heave are about 200 mm and 600 mm respectively. The analysis indicates that the maximum bending moment in the sheet pile wall increases rapidly with increasing depth of the excavation. The maximum bending moment approached the yield strength of the FSP IIIA sheet piles, 80 kNm/m.

The lateral deflection of the sheet piles, the settlements around the excavation and the bottom heave are also shown in Fig 46 when a 10 m wide strip of the soil has been removed along the excavation. It can be seen that the unloading had only a marginal effect on the settlements, the lateral deflections of the sheet pile wall and on the base heave. Also the effect on the strut loads is small as can be seen from Fig 47.

Jet grouting has also been used in Singapore to improve the soft marine clay (Miki, 1985). At this method contiguous or overlapping cylindrical cement columns are formed in-situ in the clay. The diameter of the columns can be up to 2.0 m. The method has, for example, been used to stabilize a 15 m deep excavation for the Newton Circus Station of the Mass Rapid Transit System (MRT) in Singapore and to stabilize tunnels excavated in the soft marine clay and in loose sands.

The construction sequence followed at the jet grouting has been modelled in the FEM-analysis. First the stability of the sheet piles during the installation has been analyzed. Thereafter, the effect of the jet-grouting of a 3 m thick zone of soft clay between the two sheet pile walls below the bottom of the
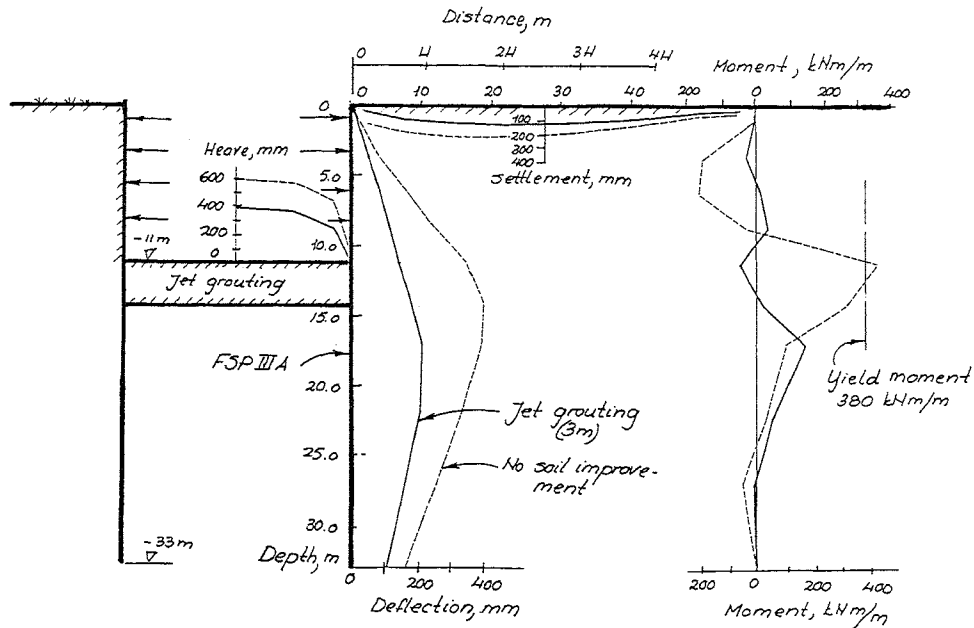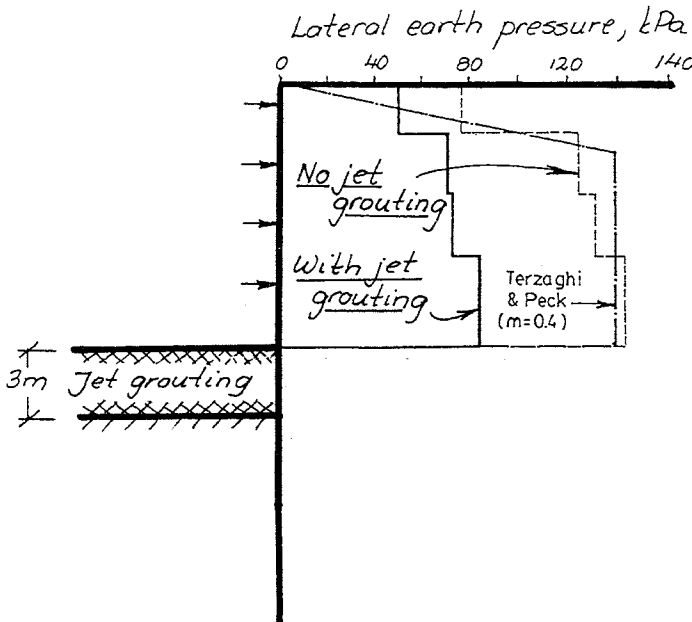
1543

Fig 48   Effect of jet grouting (3 m)

Fig 49   Effect of jet grouting on strut loads

excavation has been investigated. An undrained shear strength of 150 kPa has been assumed for the stabilized 3 m thick layer. Cores of the grouted soil from actual projects indicate that the shear strength of the jet-grouted material can be much higher than 150 kPa

A comparison with the case where no soil improvement has been used shows as indicated in Fig 48 that the performance of the excavation is improved considerably by the jet grouting and that the maximum lateral deflection of the sheet piles is reduced by about 50 percent. Also the settlements and the strut loads are reduced significantly as shown in Fig 49 as well as the maximum bending moment in the sheet piles. Jet grouting has been found to be a very effective method to improve the overall stability of excavations in soft clay.

A further improvement can be obtained by increasing the thickness of the jet grouted zone to 6 m as can be seen in Fig 51. Mainly the deflections of the wall and the bottom heave are reduced. The strut loads are also reduced significantly at all levels (Fig 52). The largest reduction was observed for the bottom level of struts as shown in Table II as could be expected.

FEM has been used to evaluate the stabilizing effect of embankment piles. It was assumed in the analysis that the spacing of the 6 m long Bakau piles with 100 mm diameter is 0.5 m. The piles are driven below the bottom of the excavation using a follower. The tip level is located 17 m below the ground surface. The results of the analysis are presented in Fig 52 and in Table III and compared with the case without soil improvement. The analysis indicate that for a 11 m deep and 33 m wide excavation, four to eight rows of
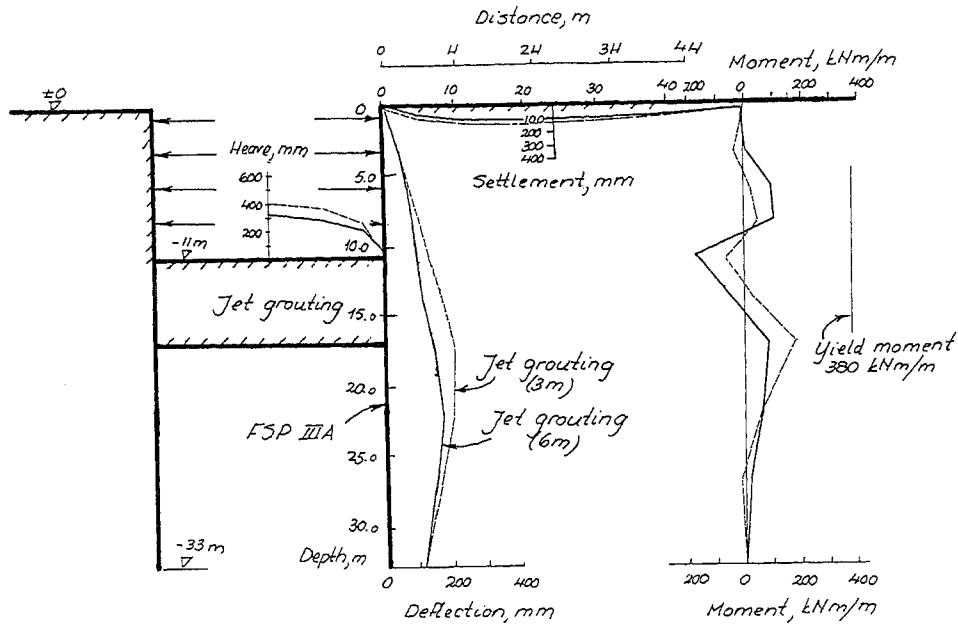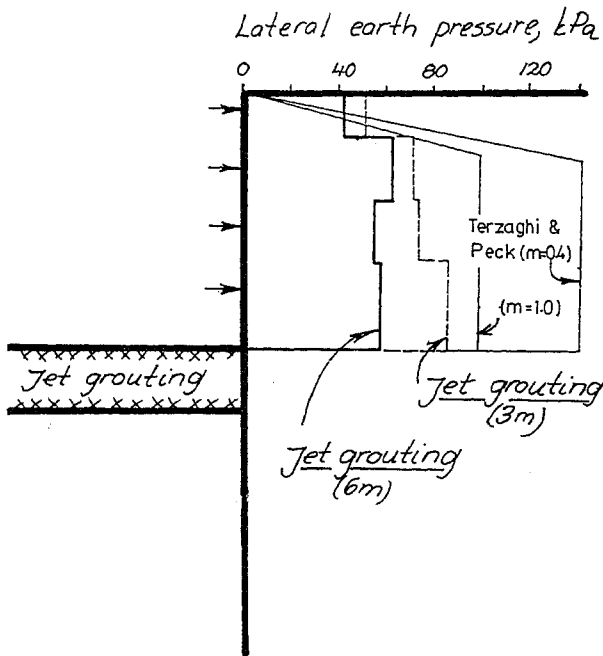
1544

Fig 50   Effect of jet grouting (6 m)

Fig 51   Effect of jet grouting (3 m and 6 m) on strut loads

Table II Effect of jet grouting on the performance of a 33 m wide and 11 m deep braced excavation

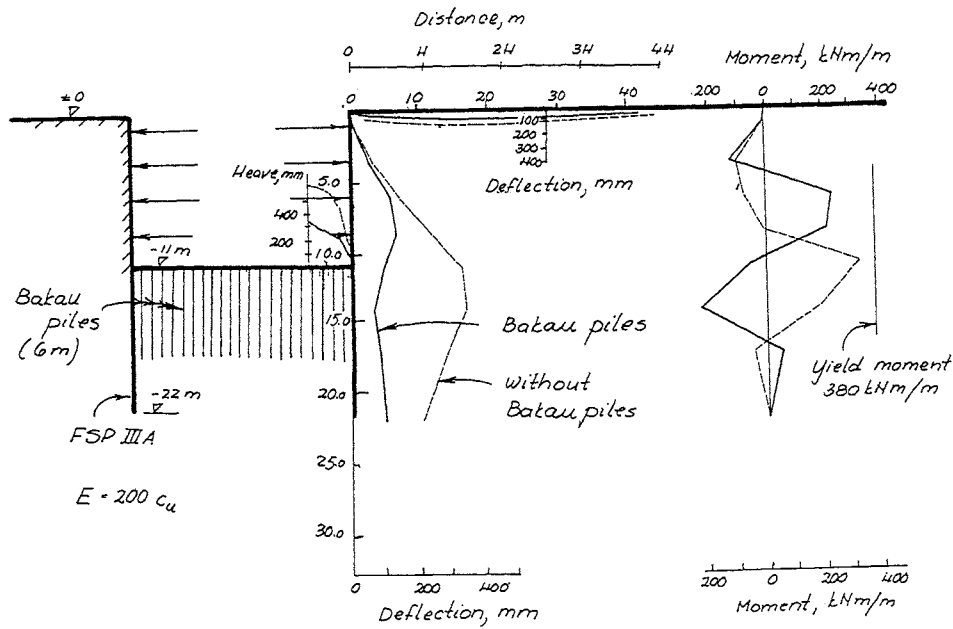|	Without jet grouting	Jet grouting (3m)	Jet grouting 6m
Maximum wall deflection, mm	400 mm	208 mm	167 mm (41%)
Maximum moment, kNm/m	430	180	165 (38%)
Maximum surface settlement, mm	209 mm	120 mm	100 mm (48%)
Maximum base heave, mm	590 mm	396 mm	320 mm (54%)
Maximum strut loads, kN/m			
Level 1	172	126	94 (54%)
Level 2	315	177	153 (48%)
Level 3	329	182	135 (41%)
Level 4	358	209	145 (40%)

1545

Fig 52   Effect of Bakau piles

Bakau piles in front of each wall could reduce the maximum wall deflection by up to 29% and the maximum bending moment in the sheet piles by 35%. The results also indicate a substantial reduction of the strut loads at the two bottom levels (Fig 53) and an increase of the passive pressure in front of the wall. The effectiveness of the Bakau piles was found to increase with decreasing width of the excavation.

	Without Bakau piles	Bakau piles (4 rows)	Bakau piles (8 rows)
Maximum wall deflection, mm	431 mm	332 mm	306 mm (71%)
Maximum moment, kNm/m	450	306	295 (65%)
Maximum surface settlement, mm	261 mm	220 mm	262 mm
Maximum base heave, mm	594 mm	560 mm	539 mm
Maximum strut loads, kN/m			
Level 1	159	142	128 (80%)
Level 2	324	268	250 (77%)
Level 3	333	264	246 (74%)
Level 4	356	286	272 (76%)
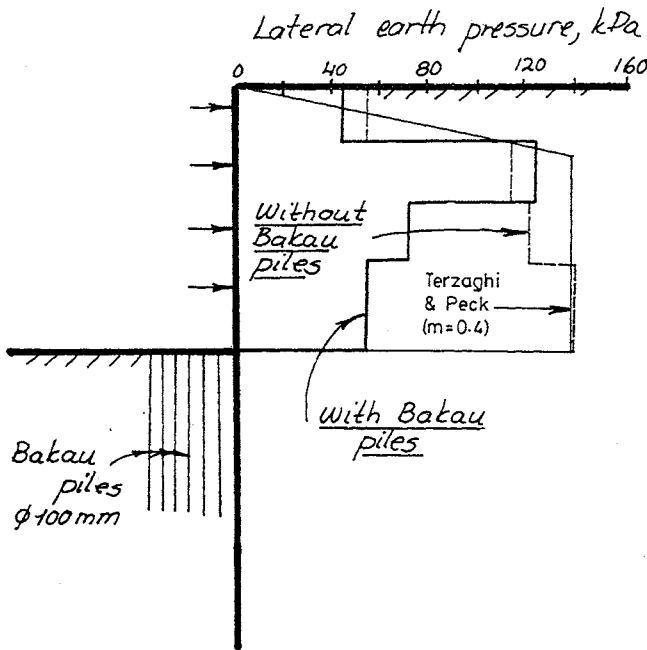
Fig 53   Effect of Bakau piles on strut loads

Table III Effect of embankment piles (Bakau piles) on the performance of a 33 m wide and 11 m deep braced excavation

1546

1. Concreting of base slab under water

2. Dewatering and installation of strut level 2

3. Dewatering and installation of strut level 3

4. Final stage of excavation

Fig 54   Construction sequence

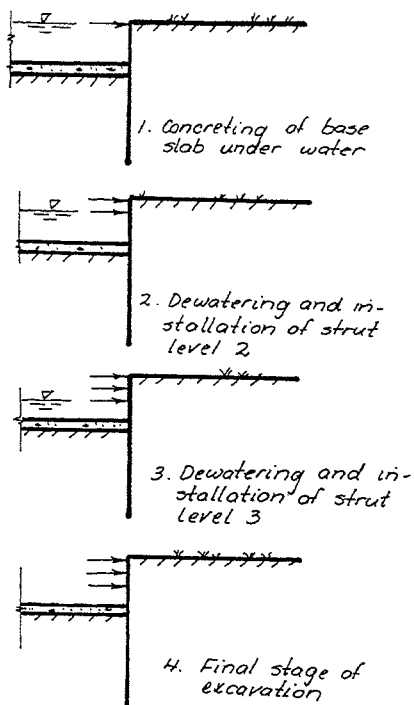The stability of deep excavations in soft clay can also be increased by excavating the soft clay under water. The initial excavation can be done dry until the first one or two rows of struts have been installed. Next, the excavation is flooded so that the soft clay can be excavated down to the final depth. After the base slab has been cast under water the excavation is dewatered and the intermediate struts are installed.

A 15 m deep and 33 m wide excavation has been analyzed using FEM. The sides of the excavation are supported by sheet piles Z-45 with a section modulus (SM) equal to 4500 cm^3/m. Two sets of analysis were conducted. In the first set a conventional excavation method with five levels of struts was investigated. The second set was concerned with the excavation of the soft clay under water. Three levels of struts are used to support the sheet piles. The 2.0 m thick base slab will be cast under water as shown in Fig 54.

The results show a significant improvement of the overall performance (Fig 55). The maximum wall deflection was reduced by 53%. A 44% reduction of the base heave and a 50% reduction of the ground settlement were also obtained. The loads in the second and third level struts are reduced significantly as well (Fig 56 and Table IV). Because the base slab will be installed before the second and third level struts, the axial load in the base slab will be high compared with that in the two levels of struts (Table IV).

The FEM analysis indicates that excavation under water down to 15 m depth is feasible. The calculated maximum wall deflection, 130 mm, and a maximum ground settlement of 150 mm are much less than those observed for actual excavations using conventional methods even when the maximum depth is less than 11 m.
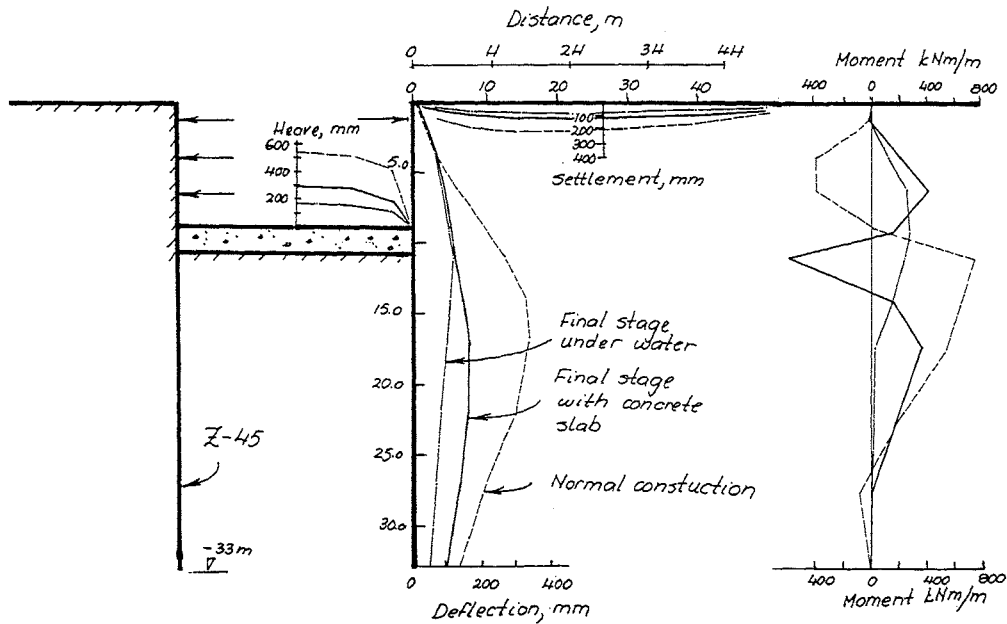

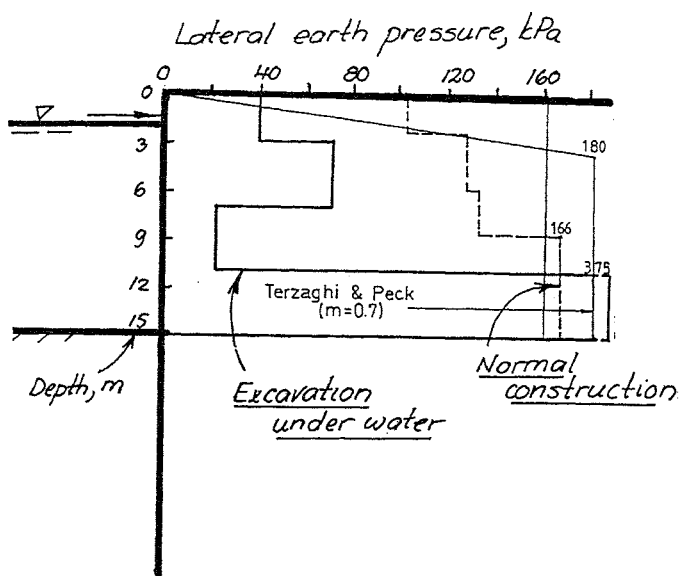Fig 55   Effect of excavation under water

1547

Fig 56   Effect on strut loads of excavation under water

Excavation under water has the main advantage that the stability with respect to base heave is governed by the submerged unit weight of the soft marine clay (about 6 kN/m^3) rather than the total unit weight (about 16

	Normal construction	After excavation to 11 m, before concreting	Final stage of construction
Maximum wall deflection, mm	347 mm	124 mm	165 mm
Maximum moment, kNm/m	200	80	110
Maximum base heave, mm	540 mm	170 mm	300 mm
Maximum strut loads, kN/m			
Level 1	187	187	158
Level 2	358	—	130
Level 3	363	—	70
Level 4	551	—	953 (slab)

Table IV Effect of excavation under water on the performance of a 33 m wide excavation

kN/m^3). The wall movements during the dewatering and the installation of the struts after the installation of the base slab under water will mainly occur below the slab. The lateral deflections of the sheet piles above the slab will be small.

SUMMARY

The design and construction of anchored and strutted sheet pile walls in soft clay have been reviewed. Most failures have been caused by insufficient penetration depth of the sheet piles when the walls rotate around the level of the anchors or of the struts. Failure can also be caused by rupturing of the anchor rods or by buckling of the struts. The strut or anchor loads can for deep cuts especially at the bottom of the excavation be considerably higher than those calculated by a conventional method. Failure by bottom heave is also a possibility which must be considered in the design.

When inclined anchors are used it is also important to take into account the vertical force caused by the inclined anchors or by the struts. This vertical force can reduce considerably the stability of particularly anchored sheet pile walls. Several failures have occurred which have been caused by insufficient vertical stability of the sheet piles and where the vertical force caused by the inclined anchors was not considered in the design.

Failure of one of the anchors or struts may lead to progressive failure and complete collapse (zipper effect) of the wall. If a sufficient high factor of safety is used in the design then the increase of the load in the adjacent anchors or struts will be small at failure of one of the anchors.

REFERENCES

Aas, G. (1984). "Stability Problem in a Deep Excavation in Clay," Proc. Int. Conf. on Case Histories in Geotechnical Engg., St Louis, Vol. 1, pp 315 - 323.

Ahmad, A. S. and Peaker, K. R. (1977). "Geotechnical Properties of Soft Marine Singapore Clay," Proc. Int. Symposium on soft Clays, Bangkok, Thailand, pp 3-14.

Baguelin, F., Jezequel, J. F. and Shields, D. H. (1978). "The Pressuremeter and Foundation Engineering," Trans. Tech. Publications, Clausthal, Germany, 617 pp.

Bjerrum, L. and Eide, O. (1956). "Stability of Strutted Excavations in Clay," Geotechnique, Vol. 65, No. 1, pp. 32 - 47.

Broms, B. B. (1968). "Swedish Tie-Back System for Sheet Pile Walls," Proc. 3rd Budapest Conf. on Soil Mech. a. Found. Engg., Budapest, pp. 391 - 403.

Broms, B. B. (1984). "Design of Sheet Pile Walls," Proceedings of Seminar on Precautions and Solutions to Environmental Problems Caused by Tunnelling and Excavations organized by ABV-LCM Engineering Pte Ltd, April 1984, Singapore, 27 pp.

· 1548

Broms, B. B. and Bennermark, H. (1967). "Stability of Clay at Vertical Openings," Journ. Soil Mech. a. Found. Div., ASCE, Vol. 93, No. SM1, pp 71 - 94.

Broms, B. B. and Bergdahl U. (1982). "The Weight Sounding Test (WST)," State-of-the-Art Report, ESOPT II, Amsterdam, 12 pp.

Broms, B. B. and Bjerke, H. (1973). "Extrusion of Soft Clay Through a Retaining Wall," Canadian Geotechnical Journal, Vol. 10, No. 1, pp 103 - 108.

Broms, B. B. and Wong, I. H. (1984). "Foundation of Tall Buildings," Proceedings of Int. Conf. on Tall Buildings, Singapore, pp 547 - 590.

Broms, B. B., Wong, I. H. and Wong, K. S. (1986). "Experience with Finite Element Analysis of Braced Excavations in Singapore." Proc., 2nd Int. Symp. on Numerical Models in Geomechanics, NUMOG II, Ghent, Belgium, pp 309 - 324.

Burland, J. B., Simpson, B. and St. John, H. D. (1979). "Movements Around Excavations in London Clay," Proceedings of Seventh European Conf. on Soil Mechanics and Foundation Engg., Vol. 1, Brighton, pp. 13 - 29.

Chang, C. S. and Duncan (1977). "EXCAV : A Computer Program for Analysis of Stresses and Movements in Excavations," Geotechnical Report No TE 77-4, University of California, Berkeley. 35 pp.

Clough, G. W. and Davidson, R. R. (1977). "Effects of Construction on Geotechnical Performance," Proceedings, Specialty Session III, Ninth Int. Conf. on Soil Mech. a. Found. Engg., Tokyo, 1977, pp. 15 - 53.

Clough, G. W. and Hansen, L. A. (1981). "Clay Anisotropy and Braced Wall Behaviour," Journ. of the Geotechnical Div., ASCE, Vol. 107, No. GT 7, July, pp. 893 - 914.

Clough, G. W. and Mana, a. I. (1977). "Lessons Learnt in Finite Element Analysis of Temporary Excavations in Soft Clay," Proc. 2nd Int. Conf. on Numerical Methods in Geomechanics, Blacksburg, Virginia, ASCE, Vol. 7, pp. 496 - 510.

Clough, G. W. and Tsui, Y. (1974). "Performance of Tied-Back Walls in Clay," Journ. of the Geotechncial Div., ASCE, Vol. 100, No. GT12, pp. 1259 - 1274.

D'Appolonia, D. J. (1971). "Effects of Foundation Construction on Nearby Structures," Proc. 4th Pan American Conf. on Soil Mech. a. Found. Engg., Puerto Rico, Vol. 1, pp. 189 - 236.

Duncan, J. M., Byrne, P., Wong, K. S. and Mabry, P. (1980). "Strength, Stress-Strain, and Bulk Modulus Parameters for Finite Element Analysis of Stresses and Movements in Soil Mass". Dept of Civil Engg., University of California, Berkeley, 70 pp.

Egger, P. (1972). "Influence of Wall Stiffness and Anchor Prestressing on Earth Pressure Distribution," Fifth European Conf. on Soil Mech. a. Found. Engg., Vol. 1, Madrid, pp. 485 - 498.

Hanna, T. H. (1982). "Foundations in Tension - Ground Anchors," Trans Tech Publications, McGraw Hill Book Co., Series on Rock and Soil Mech., Vol. 6, Clausthal, Germany, 573 pp.

Janbu, N. (1972). "Earth Pressure Computations in Theory and Practice," Proc. 5th European Conf. on Soil Mech. a. Found. Engg., Vol. 1, Madrid, Spain, pp. 45 - 54.

Karlsrud, K. (1986). "Performance Monitoring of Deep Supported Excavations in Soft Clay," Proc. 4th Int. Geotechnical Seminar, Field Instrumentation and In-Situ Measurements, Nanyang Technological Institute, Singapore, pp. 187 - 202.

Karlsrud, K., DiBiagio, E., and Aas, G. (1980). "Experience with Slurry Walls in Soft Clay." Proc., Symp. on Slurry Walls for Underground Transportation Facilities, U.S. Dept. of Transportation, Report No. FHWA-TS-80-221, March, pp. 383 - 408.

Lee, S. L., Yong, K. Y., Karunaratne, G. P. and Chua, L. H. (1986). "Field Insturmentation for a Strutted Deep Excavation in Soft Clay," Proc. 4th Int. Geotechn. Seminar, Field Instrumentation and In-Situ Measurements, Nanyang Technological Institute, Singapore, pp. 183 - 186.

Littlejohn, G. S. (1970). "Soil Anchors," Proc. Conf. on Ground Engg. The Institution of Civil Engineers, London, England, pp. 33 - 44.

Mana, A. I. and Clough, G. W. (1981). "Clay Anisotropy and Braced Wall Behavior," Journ. of the Geotechnical Div., ASCE, Vol. 107, No. GT 6, pp. 759 - 778.

Meyerhof, G. G. (1956). "Penetration Tests and Bearing Capacity of Cohesionless Soils," Journ. Soil Mech. a. Found. Div., ASCE, Vol. 82, SM1, pp. 1 - 19.

Miki, G. (1985). "Soil Improvement by Jet Grouting," Proc. 3rd Int. Seminar, Soil Improvement Methods, Singapore, pp. 229 - 236.

O'Rourke, T. D. (1981). "Ground Movements Caused by Braced Excavations," Journ. of the Geotechnical Div., ASCE, Vol. 107, No. GT 9, pp. 1159 - 1178.

Peck, R. B. (1969). "Deep Excavations and Tunnelling in Soft Ground," Proc of Seventh Int. Conf on Soil Mech. a. Found. Engg., State-of-the-Art-Volume, pp. 225 - 290.

Rowe, P. W. (1957). "Sheet Pile Walls in Clay," Proc. of Institution of Civil Enggs., Vol. 7, Jul 1957, pp. 629 - 654, London, England.

Sahlström, P. O. and Stille, H. (1979). "Förankrade Sponter (Anchored Sheet Pile Walls)," Swedish Council for Building Research, Report No T30 : 1979, Stockholm, Sweden, 128 pp.

Stille, H. (1976). "Behaviour of Anchored Sheet Pile Walls," Thesis, Royal Inst. of Tech., Dept. of Soil and Rock Mech., 192 pp.

Stille, H. and Broms, B. B. (1976). "Load Redistribution Caused by Anchor Failures in Sheet Pile Walls," Proc. 6th European Conf. on Soil Mech. a. Found. Engg., Vienna, Austria, Vol. 1.2, pp. 197 - 200.

Stroh, D. and Breth, H. (1976). "Deformation of Deep Excavations," Proc., Conf. on Numerical Methods in Geomechanics, Vol. II, Blacksburg, June, pp. 686 - 700.

Tan, S. B. (1970). "Stability Investigations of the Whampoa Slip," Journ. South East Asia Geotechnical Society, Vol. 1, No. 2, pp. 95 - 101.

Tan, S. L. (1983). "Geotechnical Properties and Laboratory Testing of Soft Soils in Singapore," Int. Seminar on Construction Problems in Soft Soils, Nanyang Technological Institute, Singapore, 1 - 3 December 1983, 47 pp.

Terzaghi, K. and Peck, R. B. (1967). "Soil Mechanics in Engg Practice", 2nd Edition, John Wiley and Sons, New York, 729 pp.

van der Veen, C. and Boersma, L. (1957). "The Bearing Capacity Predetermined by Cone Penetration Test," Proc. 4th Int. Conf. Soil Mech. a. Found. Engg., Vol. 2, pp. 72 - 75.