1. **INTRODUCTION**

When vertical cuts are made in soils, it is often necessary to brace the sides of the cut so that collapse of the sides does not occur. Several techniques can be used for bracing the sides of the cut.

For shallow cuts less than about 4m in depth, vertical wooden planks can be used. The planks are braced by struts that run from one side of the cut to the other, and bear onto wales that run along the face of the planks. For deeper cuts, sheet piles or 'H' piles with lagging may be used. Such support systems are shown in Figure 1.

As the excavation proceeds, bracing can be applied to the face of the excavation. In the case of sheet piles, the piles are driven a little deeper than the excavation as a precaution against bottom heave. If 'H' piles are used, the lagging is introduced as the hole is deepened as are the bracing struts.

Other systems of bracing that are used include prefabricated steel wall panels (in conjunction with bracing), augered or 'soldier' pile walls and slurry walls. These systems are shown in Figures 2

![Diagram](image)

*Figure 1: Bracing systems used for excavations (a) sheet piles (b) 'H' piles and lagging*
Figure 2: Wall and bracing systems (a) augered pile (c) Slurry trench

Figure 3: Prefabricated wall and bracing system
Slurry walls may be constructed by excavating vertical trenches with a grab or clamshell bucket. The sides of the trench are kept from collapsing by keeping the hole full of a slurry (usually bentonite).

Excavation of the trenches can be carried out under the slurry by the grab. When excavation is complete, a reinforcing cage is placed into the trench and concrete is tremied in from the bottom, displacing the bentonite slurry. The hardened concrete forms the two walls which can be braced as soil is excavated from between them.

Whatever method of braced retaining system is used, there remain two questions to be answered if a safe retaining system is to be designed:

(i) What are the earth pressures that will act on the braced retaining walls?

(ii) Will the base of the excavation heave up into the excavation?

As well as the above two design considerations, it may be necessary to control movements of the soil surface caused by the excavation. This can be due to the stress relief caused when soil is removed from the excavation, or can be due to the water table dropping in permeable soils. A water table drop will cause an increase in the effective stress in the soil and this will in turn cause settlement of the soil surrounding the excavation.

Figure 4: Failure modes for long shallow cuts
2. BASE HEAVE FOR CUTS IN CLAY

One way in which an excavation can fail is through heave of the base. This is most likely to occur in soft clays, but can also occur in sands.

Analysis of the base heave problem can be carried out for simple two dimensional cases by using the method proposed by Terzaghi (1943).

2.1 Shallow Foundations (H/B < 1)

In this case we can assume that the failure surface reaches the ground surface. The assumed failure mechanism depends on whether the excavation has a firmer stratum beneath the required level of excavation. The two cases are shown in Figure 4, case (A) is for the case where the excavation is quite narrow and there is no layer of stiff material close to the base of the excavation. Case (B) is where the excavation is wide and has a stiffer stratum close to the base.

The failure problem can be treated as a bearing capacity failure problem with the block of soil to the side of the excavation causing the failure through its self weight minus the shearing resistance force at the side of the soil block.

Case (A)

In this case the load per metre run due to the soil block (on a-b) is:

$$0.7BH\gamma - Hc$$  \hspace{1cm} (1)

The bearing capacity per metre run will be

$$cN_c 0.7B$$  \hspace{1cm} (2)

Hence the factor of safety $F$ will be given by:

$$F = \frac{0.7cN_c}{0.7BH\gamma - Hc}$$  \hspace{1cm} (3)

In the case where the excavation becomes very wide ($B$ is large), then the factor of safety reduces to

$$F = \frac{cN_c}{\gamma H}$$  \hspace{1cm} (4)
Case (B)

In this case, the load per metre run is given by

$$ DHf - Hc $$ \quad (5)$$

and the bearing capacity by

$$ cN_e D $$ \quad (6)$$

so that the factor of safety \( F \) is given by

$$ F = \frac{cN_e}{H(\gamma - cd)} $$ \quad (7)$$

2.2 Deep Foundations (H/B > 1)

In this case, the failure surfaces will not reach the ground surface and there will be an enclosed failure surface as shown in Figure 5. For a long excavation we must use the formula provided by

![Figure 5: Failure mechanism for deep cut.](image-url)
Figure 6: Bearing capacity factors for rectangular cuts.

Bjerrum and Elde (1956)

\[ F = \frac{cN_c}{\gamma H} \]  

(8)

where the bearing capacity factor would be that for a strip i.e. 7.6.

2.3 Cuts of any Rectangular Shape in Plan

For cuts that are of width \( B \) and length \( L \) in plan, Bjerrum and Elde’s formula may still be used although the bearing capacity factors need to be those for deep foundations of general rectangular shape.

The bearing capacity factors for such a case are shown in Figure 6. An interpolation formula can be used to work out the bearing capacity factor for any rectangular shape, ranging from a square \((B = L)\) to a long excavation \((L = \infty)\). The factor of safety is then given by

\[ F = \frac{cN_{clay} (0.84 + 0.16B/L)}{\gamma H} \]  

(9)

2.4 Stronger Clay Layer Near the Base of the Cut

The cases given above apply to problems involving clays for which the undrained shear strength of the clay is treated as being constant. For the case where there is a stronger layer of clay beneath the base of the excavation Reddy and Srinivasan (1967) have provided the following formula for the factor of safety \( F \)
\[ F = \frac{c_1[N'_c]_d F_d}{\gamma H} \]  

(10)

where  
- \( c_1 \) is the strength of the upper layer of clay,  
- \( F_d \) is a depth factor which is a function of \( H/B \),  
- \( N'_c \) is the bearing capacity factor for an infinitely long cat,  
- \( F_s \) is a shape factor given by equation (11)

\[ F_s = 1 + 0.2 \frac{B}{L} \]  

(11)

The bearing capacity factor \( N'_c \) may be found from the chart provided in Figure 7 as can the depth factor \( F_d \).

(Note: \( c_1 > c_i \))

Figure 7: Values of correction factor \( F_d \) and bearing capacity factor \( N'_c \)
3. BASE FAILURE IN SANDS

With braced cuts made in sands, the danger of base failure usually occurs when the water level inside the cut is lowered, so that an upward flow of water into the excavation can take place. Piping of the sand can occur if the hydraulic gradient approaches a value of 1.

The hydraulic gradient can be determined from a flow net at the point where the flow enters the excavation. This is shown in Figure 8 where the exit hydraulic gradient $i_{exit}$ is given by

$$i_{exit} = \frac{\Delta H}{a}$$

(12)

where $\Delta H$ is the drop in total head between two equipotentials and $a$ is the shortest distance between two equipotentials across the base. In Figure 8 the value of $\Delta H$ can be calculated from

$$\Delta H = \frac{h}{N_d}$$

(13)

where $h$ is the total head drop, and $N_d$ is the total number of head drops (i.e. 8 in the example in the figure).

The factor of safety against a piping failure may be defined as shown in equation 14.

$$F_{piping} = \frac{i_{cr}}{i_{exit}}$$

(14)
A factor of safety of about 1.5 would be generally required to guard against a piping failure.

4. GROUND SETTLEMENT CAUSED BY EXCAVATION

Lateral movement of the walls of an excavation will result in movement of the ground at the surface that is generally called ground loss. There is always some movement of the walls before bracing can be applied and therefore some vertical surface movement will occur.

Peck (1969) has provided information on expected ground movements and these are shown in Figure 9. The amount of movement expected depends on the type of soil encountered, but is largest in very soft to soft clays as may be expected. The magnitude of the vertical deformation depends on the distance from the edge of the cut as can be seen from Figure 9.

![Figure 9: Variation of surface settlement with distance (Peck 1969).](image)

Tomlinson (1995) has presented data collected for many different excavations in soils of different types. He comments that the amount of horizontal movement that occurs is not dependent on the type of wall and bracing system, and that there is little difference in the movement of diaphragm walls and sheet pile walls. Plots of measured maximum horizontal movements are shown in Figure 10 for (a) soft to firm normally consolidated clays (b) stiff to hard overconsolidated clays and (c) sands and gravels.

Plots of vertical surface movements versus distance from the edge of the excavation have also been presented by Tomlinson and are shown in Figure 11. The numbers on the curves (that are presented for the same soil types as mentioned above) refer to the case studies of Figure 10. Tomlinson concludes from this data that the "maximum surface settlement was about 0.5 - 1.0 times the maximum inward yielding".
Mana and Clough (1981) have also examined the relationship between the maximum horizontal movement of the walls of an excavation and the maximum surface settlements. They found that the vertical movement is about 0.5 to 1 times the horizontal movement i.e.

\[ \delta_{v_{\text{max}}} = 0.5 \text{ to } 1 \delta_{H_{\text{max}}} \]  

(15)

The data presented by Mana and Clough (1981) is presented in Figure 11.

![Graph showing observed maximum inward movement of braced excavations](image)

Figure 10: Observed maximum inward movement of braced excavations.

3. FORCES ON BRACED EXCAVATIONS

Unlike ordinary retaining walls that can move away from the backfill and develop the active pressures in the soil, braced walls cannot move as easily, and therefore different pressures are developed on braced walls.

Peck et al. (1974) have presented empirically designed pressure envelopes that are based on measurements taken in struts. The apparent pressure envelope can then be used to compute the forces in a strut at any given level. This pressure envelope is not the true pressure distribution, but a device to allow computation of the forces in the bracing.

The forces in a strut can be computed by finding the area under the pressure envelope for each strut. This is done by assuming that each strut carries the pressure applied to the wall over the
Figure 11: Maximum vertical movement of surface versus distance from edge of excavation.

Figure 12: Relationship of maximum lateral and maximum vertical ground movement.
Figure 13: Method of calculating strut loads from apparent pressure diagram.

region going half way to the next strut as shown in Figure 13.

If no strut is placed at the bottom of the excavation, it is assumed that part of the load is taken by the soil at the base of the excavation.

For cuts made in sand, the apparent pressure envelope may be considered to be of constant magnitude with depth and to have a value of 0.65γHK where \( K_a \) is the active earth pressure coefficient \( \left[ = \tan^2(45^\circ - \phi) \right] \). It should be noted that this design procedure only applies to dry or moist sands.

For cuts made in clay, the pressure envelope depends on the parameter \( \gamma H/c \) where \( H \) is the depth of the cut. If this value is less than 4, the envelope shown in Figure 14(c) should be used. In this case the average magnitude of the apparent pressure envelope is about 0.3γH. If the ratio exceeds 4, the pressure envelope of Figure 14(d) should be used provided that the envelope is greater than that in Figure 14(c), otherwise the value of pressure from (c) is used. The cohesion \( c \) is taken as the average value of the cohesion beside the cut.

5. REFERENCES


- Figure 14: Apparent pressure diagrams (after Peck 1974).