ject of a historical keynote paper by Terzaghi[34]. This method of construction, which is widespread on waterfronts where it is used to achieve a quay line with reclamation behind it at minimum cost, does not strictly lie within the subject of deep excavations, but no technical discussion on braced sheeting would be complete without reference to the paper and the controversial discussion that followed it. The principal purpose of the paper was to identify and rectify errors in accepted bulkhead design at that time on the basis of tests and observations by Terzaghi and model tests by Rowe[20,35,36]. The paper reached the following principal conclusions.

(*a*) The identification of the type of soils and fills and their in situ properties of uniformity, relative density and strength are vital matters which are frequently overlooked in anchored bulkhead design.
(*b*) The distribution of earth pressure on the bulkhead is unlikely to conform to the Coulomb distribution, because of the extent of deformation of the soil structure. This deformation depends on soil and wall stiffness.
(*c*) If maximum bending moments are calculated on walls in sand assumed to extend to sufficient depth to achieve full fixity irrespective of wall flexibility and sand relative density, errors are likely and these are on the unsafe side.
(*d*) For sheet piles driven into clay, the assumption of full fixity at depth will probably not apply as time elapses and no reduction in the calculated maximum moment in the wall should be allowed due to wall section flexibility to compensate for this loss.
(*e*) Anchor tension depends on several factors other than the properties of the backfill material and the flexibility of the wall or sheeting. Therefore, the anchor pull should be computed on the assumption of free earth support. Anchor pull may be greater than that calculated using Coulomb's theory, and may increase due to repetition of loading and unloading from heavy surcharge. An unequal yield of adjacent anchorage produces variations in tie rod pull. Given these risks, more conservative stresses should be used in anchor design than are applied in sheet pile bulkhead design.

These conclusions broadly still apply, although alternative methods of analysis have been developed in which soil, wall and anchor stiffness can be modelled and deformation and induced stress in all three can be calculated. Terzaghi concluded: 'Because of the great variety of subsoil conditions which may be encountered, the subject (anchored bulkheads) does, and always will, leave a wide margin for judgment – and also for misjudgment.'

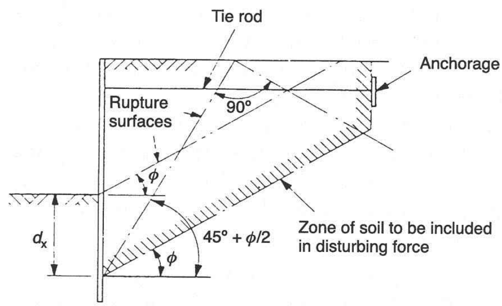
Anchorage location

Anchorages, deadmen or injected tendons must be located behind potential failure surfaces at the rear of the wall. Figure 5.25 shows the recommended geometry for analysis of deadmen locations (from BS 6349[27]).

Foundation failure

The risk of base failure to an excavation by upward heave applies particularly in very soft and soft clays and silty clays, typically, quick estuarine deposits. The failure is analogous to a bearing capacity failure of foundation, only in reverse; the failure is a shear failure in the soil below formation level, but caused by relief of load (the relief of overburden) and not by the application of load as occurs in a conventional foundation bearing failure.

The methods of Terzaghi[37] and Bjerrum and Eide[38] can be applied to calculate the factor of safety against base failure; these are shown in Fig. 5.26. Terzaghi's method is primarily applicable to shallow or wide excavations,



For free earth support d_x is the depth of embedment of sheet piles.
For fixed earth support d_x is ¾ depth of embedment.

Fig. 5.25. Location of deadman anchorage in granular retained fill[27]

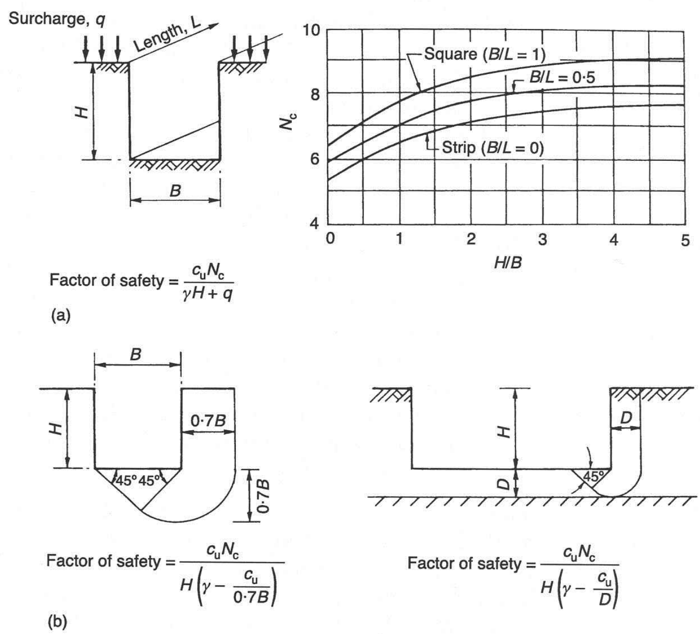
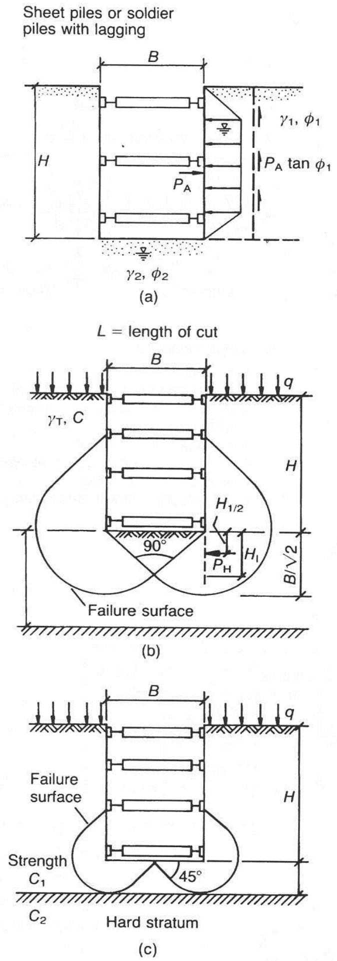


Fig. 5.26. Calculation of factors of safety against basal heave in cohesive soils: (a) deep excavations with $H/B > 1$; (b) for shallow or wide excavations with $H/B < 1$[32]

while the method of Bjerrum and Eide is suitable for deep and narrow excavations with no nearby underlying stiff clay to inhibit failure. Both methods neglect the effect of wall penetration below formation level and therefore results may prove to be conservative, especially where stiffer clays exist with depth. A third method[22] for predicting the basal safety factor where stiff clays exist at depth, is shown in Fig. 5.27.

The factor of safety against basal failure is generally required to be not less than 1·5. If uncorrected values of in situ vane tests are used, the actual factor of safety may be close to 1·0 according to Aas[39]. (A vane correction described by Bjerrum[40] is necessary to obtain more reliable values of safety factor.)

With a factor of safety, based on corrected vane results, which is less than 1·5, substantial soil deformation is likely. If such soil movement is not acceptable, a factor of safety not less than 2·0 is recommended. Increase in movement occurs as the basal factor of safety decreases, and increases rapidly as a factor of safety of 1·0 is approached. Although basal heave is rare within



(a) Stability is independent of H and B, but varies with γ, ϕ and seepage condition.

Safety factor $F_s = 2N\gamma_2 \left(\frac{\gamma_2}{\gamma_1}\right) k_a \tan \phi_1$

where $N\gamma_2$ is the bearing capacity factor (fig. 5.28).
If groundwater is at a depth of B or more below base of cut γ_1 and γ_2 are taken as moist unit weights.
If groundwater is static at base of cut γ_1 is the moist weight and γ_2 the submerged weight.
If seepage is moving upward to base of cut γ_2 = (saturated unit weight) − (uplift pressure).

(b) If sheeting terminates at base of cut the

safety factor, $F_s = \frac{N_C\, C}{\gamma_T H + q}$

N_C = bearing capacity factor, (fig 5.28.) which depends on dimensions of the excavation: B, L and H (use $H = Z$).
C = undrained shear strength of clay in failure zone beneath and surrounding base of cut.
q = surface surcharge.
If safety factor is less than 1·5, sheeting must be carried below base of cut to insure stability.
Force on buried length:

If $H_l > \frac{2}{3}\frac{B}{\sqrt{2}}$, $P_H = 0{\cdot}7\,(\gamma_T HB - 1{\cdot}4\,CH - \pi CB)$

If $H_l < \frac{2}{3}\frac{B}{\sqrt{2}}$, $P_H = 1{\cdot}5\,H_l\left(\gamma_T H - \frac{1{\cdot}4\,CH}{B} - \pi C\right)$

(c) Continuous excavation, $F_S = N_{CD}\,\frac{C_l}{\gamma_T H + q}$

Rectangular excavation, $P_S = N_{CR}\,\frac{C_l}{\gamma_T H + q}$

N_{CD} and N_{CR} = bearing capacity factors (fig. 5.28) which depend on the dimensions of the excavation: B, L and H, (use $H = Z$)

Fig. 5.27. Calculation of factors of safety against basal heave in: (a) cohesionless soil; (b) cuts in clay of considerable depth; (c) cuts in clay limited by hard stratum[22]

excavations in cohesionless soils, a basal heave analysis is included in Fig. 5.27(a) for completeness, together with basal heave analysis in clay as described in NAVFAC[22] in Fig. 5.27(b) and (c). Figure 5.28 shows the values of bearing capacity factors for use in these analyses. Field and finite element analysis predictions of the correlation between movement and basal failure factor of safety are shown in Fig. 5.29.

Cantilever and single-prop walls, particularly on sloping sites in soft clays and loose granular soils, should always be checked against risk of deep-seated circular slip failure.

Hydraulic failure

The risk of piping failure to the base of an excavation in cohesionless soils was described in Chapter 2. Design charts for penetration of cut-off walls to prevent hydraulic failure in sand and stratified soil are reproduced in Figs 5.30 and 5.31.

Wall flexibility

Rowe's work[20,35,36] in the 1950s and 1960s was initially instrumental in showing the importance of wall stiffness in design. Following a series of

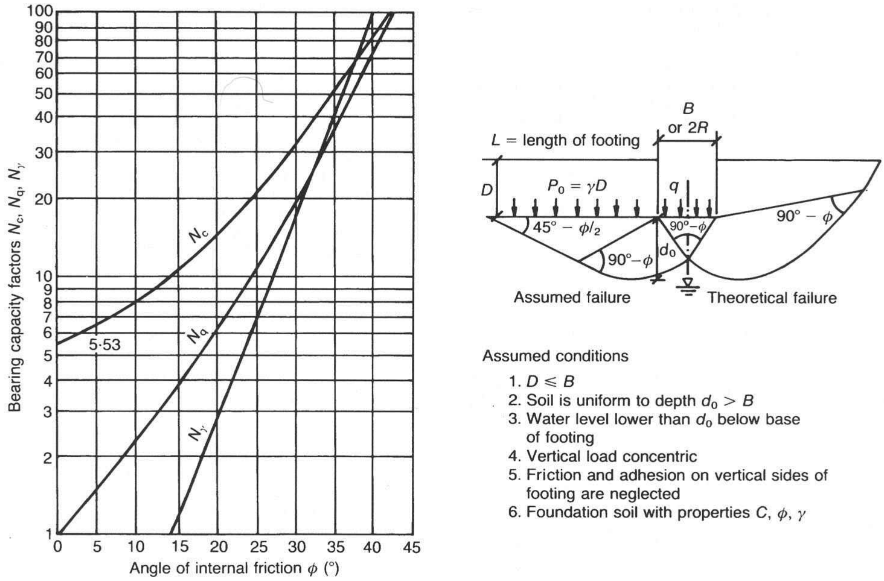


Fig. 5.28. Plot of bearing capacity factors against angle of shearing resistance[22]

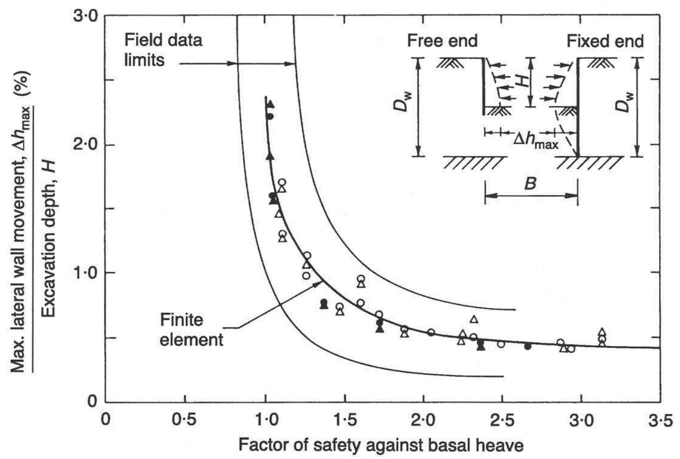


Fig. 5.29. Analytical relationship between maximum lateral wall movement and factor of safety against basal heave from field data, free end and fixed end walls, various sites (note is for finite element analysis data)

Legend:

▲ ●	End of construction	△ ○	Intermediate stages
▲ △	Free end	● ○	Fixed end

Note: $E = 300C_u$, $D_w = 30$ m, Strut stiffness = 2000 t/m per m; $B = 12$ m, PZ – 38 sheet pile, Strut spacing = 3·5 m

model tests on sands of varying relative density in Manchester, UK, Rowe was able to show that interaction between soil and wall was different for steel sheet piles and reinforced concrete sheet piles because of the greater flexibility of the steel sheet pile. This greater flexibility causes a redistribution of earth pressure which differs considerably from the Coulomb distribution, as shown in Fig. 5.32. The flexure of the wall causes reduction in pressure at mid-height and causes the resultant passive force to rise with an increase in fixity for the flexible pile. These changes reduce the design bending moment for a flexible pile, although too often such reductions are not applied in practice to ensure the pile does not crumple during driving.

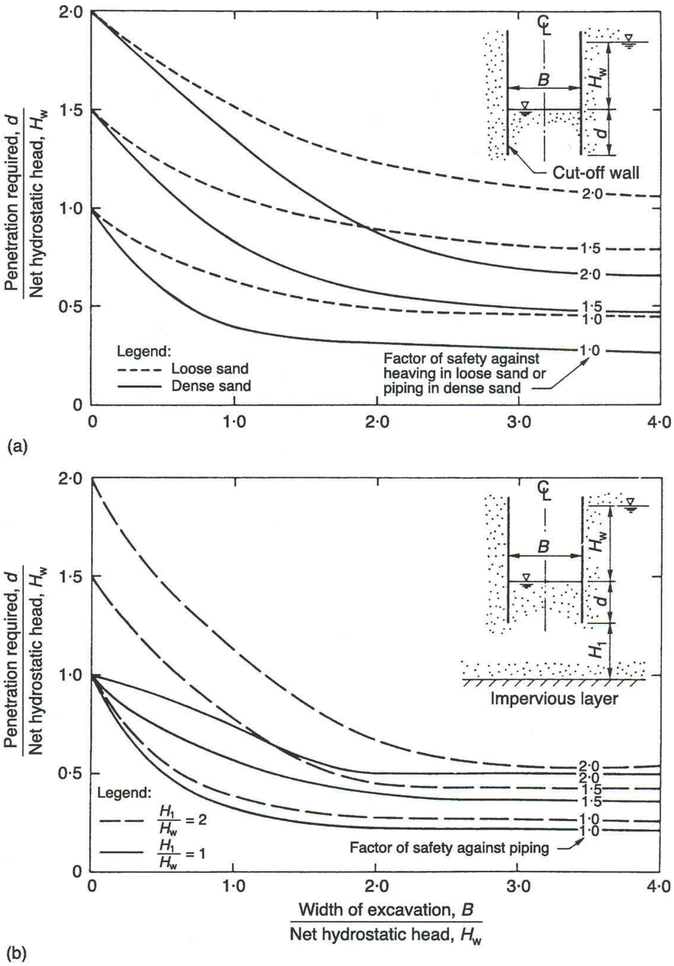


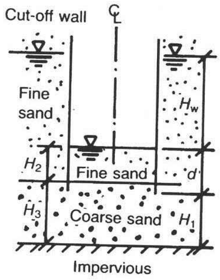
Fig. 5.30. Penetration of cut-off wall to prevent hydraulic failure in homogeneous sand: (a) in sands of infinite depth; (b) in dense sand of limited depth[22]

Materials and stresses

For cantilever and single-propped walls the component parts are designed either from limiting equilibrium hand calculations or computer program outputs using either limiting equilibrium or soil–structure interaction. The input soil parameters are those based on moderately conservative parameters with safety factors at ULS as shown in Table 5.6 (BS 8002 mobilization factors are similar) or at serviceability limit state with a partial safety factor of 1·4 (a check that moderately conservative parameters are more severe, that is lower, than worst credible parameters at ULS being made at the beginning of the design). Crack widths are calculated for serviceability limit state in reinforced concrete walls. Characteristic strengths of steel used for cantilever and single-prop sheet pile walls are given in Table 5.10.

Multi-prop walls

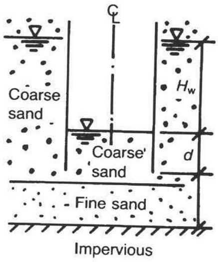
The above description of design methods for cantilever and single-propped (or anchored) walls referred to computations using limit pressures and the application of factors of safety. The methods due to BS 8002[19] have introduced design using earth pressures at the serviceability limit state. Using these methods the bending moment in the walling can be estimated relatively quickly by hand calculation (adopting Blum's methods[33] for cantilever and



(a) Coarse sand underlying fine sand
Presence of coarse layer makes flow in the fine material more nearly vertical and generally increases seepage gradients in the fine material compared to the homogeneous cross-sections of Fig 5.27.

If top of coarse layer is below toe of cut-off wall at a depth greater than width of excavation, safety factors of Fig. 5.27 (a) for infinite depth apply.

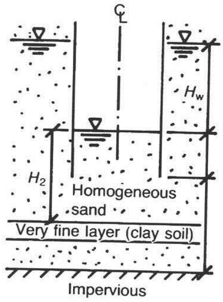
If top of coarse layer is below toe of cut-off wall at a depth less than width of excavation, then uplift pressures are greater than for the homogeneous cross-sections. If permeability of coarse layer is more than ten times that of fine layer, failure head H_w = thickness of fine layer (H_2).



(b) Fine sand underlying coarse sand
Presence of fine layer constricts flow beneath cut off wall and generally decreases seepage gradients in the coarse layer.

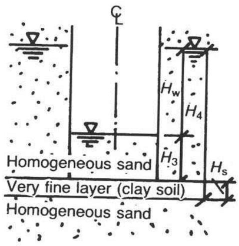
If top of fine layer lies below toe of cut-off wall, safety factors are intermediate between those derived from Fig. 5.27 for the case of an impermeable boundary at (i) the top of fine layer, and (ii) the bottom of the fine layer assuming coarse sand above the impermeable boundary throughout.

If top of fine layer lies above toe of cut-off wall, safety factors of Fig. 5.27 are somewhat conservative for penetration required.



(c) Very fine layer in homogeneous sand
If top of very fine layer is below toe of cut-off wall at a depth greater than width of excavation, safety factors of Fig. 5.27 assuming impermeable boundary at top of fine layer apply.

If top of very fine layer is below toe of cut-off wall at a depth less than width of excavation, pressure relief is required so that unbalanced head below fine layer does not exceed height of soil above base of layer.



To avoid bottom heave when toe of cut-off wall is in or through the very fine layer ($\gamma_s H_3 + \gamma_c H_s$) should be greater than $\gamma_w H_4$.

γ_s = saturated unit weight of the sand
γ_c = saturated unit weight of the clay
γ_w = unit weight of water

If fine layer lies above subgrade of excavation, final condition is safer than homogeneous case, but dangerous condition may arise during excavation above fine layer and pressure relief is required as in the preceding case.

Fig. 5.31. Penetration of cut-off wall to prevent hydraulic failure in stratified soil: (a) coarse sand underlying fine sand; (b) fine sand underlying coarse sand; (c) very fine layer in homogeneous sand[22]

fixed earth support walls) or even more conveniently using finite element, finite difference or Winkler spring analytical methods. Soil deformation behind the wall may be predicted, if needed by the finite element or finite difference programs. Design requirements and analysis methods for multi-prop walls are a different matter, however. The method of construction for these walls is usually sequential, installing the sheeting or walling and excavation in stages followed by installation of the prop or anchor at each installation stage. The sheeting or walls will, in all likelihood, penetrate the ground below the final excavation level. The extent of wall deformation in this sequence of operations is restricted, although the passive resistance of soil below excavation level at each stage is mobilized to support the wall prior to installation of the bracing or the anchor at that level. Despite the frequent

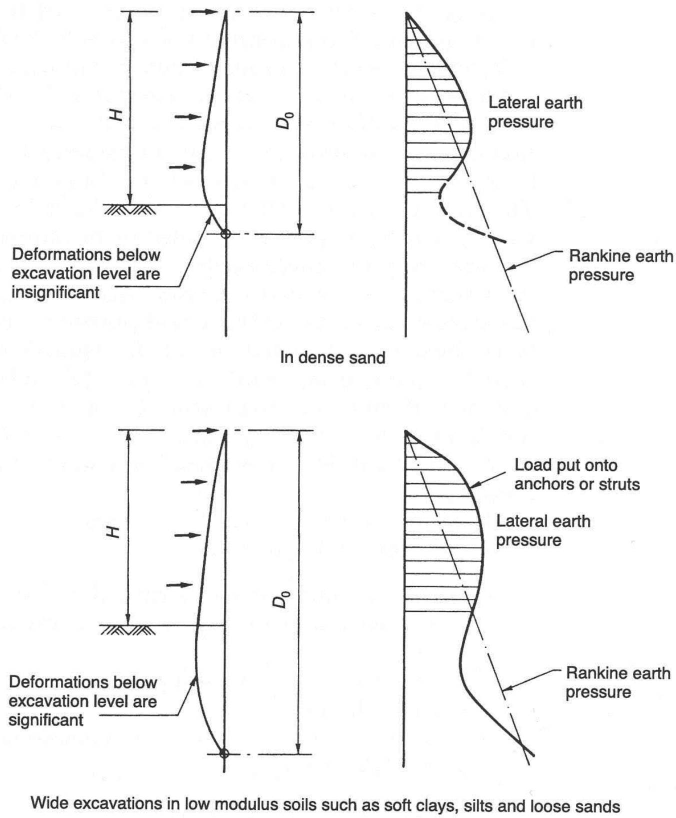


Fig. 5.32. Deflection of a sheet pile and redistribution of active earth pressure[50]

Table 5.10 Characteristic strengths for steel sheet piling

Designation EN 10027	Minimum yield strength (N/mm^2)	Minimum tensile strength (N/mm^2)	Minimum elongation on a gauge length of $L_0 = 5{\cdot}65S_0$ (A%)
EN 10248 S270 GP	270	410	24
EN 10248 S355 GP	355	480	22

support for the wall, therefore, horizontal deformation of the wall occurs at each passive soil zone prior to installation of the prop or anchor at that level. The wall distorts inwards to mobilize this passive resistance, the wall movement occurring below each stage of excavation. Pore pressure dissipation may occur in cohesive soils during the period needed for strut or anchor installation at successive levels.

The extent of wall movement also depends on the stiffness of the prop or anchor once installed at each level. Where the soil is relatively stiff, say dense sands or gravels, the extent of forward movement of the sheeting at each excavation stage to mobilize soil passive pressure will be relatively small, and active earth pressures on the wall will considerably exceed Coulomb active values above dredge level; redistribution of earth pressure will occur between the lowest strut and formation level. In wide excavations in soils such as soft clays or loose sands where stiffness is low, the successive deformations below excavation level at each propping formation level are considerable. Load is redistributed between the struts, and the sum of the maximum strut loads considerably exceeds the Coulomb values.

Where pre-stressed ground anchors are used at each excavation stage the earth pressure on the wall is determined by the pre-stress levels and subsequent relaxation, and by relative wall and soil stiffnesses. Design methods that have been used for many years have been based on calculations of anchored walls using a Coulomb distribution assuming no pre-stress applied. This non-pre-stress value of anchorage at each excavation stage is subsequently used as the actual value of pre-stress applied to the tendon. This empirical method successfully restricts soil movement but in turn inhibits the Coulomb active earth pressure distribution, on which the calculation is based, from developing, the actual pressures on the retained side of the wall being higher (and nearer K_0 or K_p values) than those calculated. More recent methods using Winkler spring and finite element programs allow an assumed anchor pre-stress load to be introduced to the analysis, from which the actual earth pressures are calculated on the basis of the soil movement permitted by wall and soil stiffness and the extent of the anchor prestress.

The ultimate limit states for multi-prop walls are similar to those for cantilever and single-prop walls:

(*a*) *overall stability* – risk of strut failure, bending stress failure in sheeting or passive failure of soil below stage excavation level or final formation level
(*b*) *foundation heave* – in soft clays, risk of failure by unloading; bearing capacity failure
(*c*) *hydraulic failure* – piping in cohesionless soils with high external groundwater table.

The serviceability limit states are as follows.

(*a*) *Deformation of sheeting* – the acceptable limits of sheeting deformation will depend on the purpose of the excavation and whether the works are temporary or permanent, or a combination of both. Where walls or sheeting are temporary the deformation must not exceed that which would occupy space required for the permanent works nor cause difficulties with sheeting removal if this is intended. For permanent works, deformation of the wall or sheeting must neither impair the durability of the substructure nor cause visual offence.
(*b*) *Soil movement behind wall or sheeting; vertical settlements* – the extent of settlement behind the support for the excavation must not exceed the permitted settlement of existing structures, highways or services, unless the consequences of this can be estimated accurately and on re-assessment are acceptable.
(*c*) *Cracking in reinforced concrete walls* – at the serviceability state, cracking will occur on the tension face of reinforced concrete walls due to application of load, in particular earth and water pressures and surcharge loading, and also on each face of the wall due to early thermal cracking of the concrete. For building substructures the provisions of BS 8110[2] will apply to crack control in walls or, where more rigorous waterproofing is needed, BS 8007[3] may be specified. For highway structures in the UK, design flexural and tension cracks complying with the BS 5400[4] are specified. Design crack widths of 0·25 mm, complying with 'severe' conditions are usual although the pressure of saline or sea water may reduce this value to 0·15 mm. Additional longitudinal steel may therefore prove necessary in diaphragm walls to control crack widths

caused by loading, although the application of rules to minimize vertical crack widths due to thermal shrinkage of concrete in panels of limited individual length may prove over-strenuous. Such rules for reinforced concrete works are referred to in the UK Department of Transport's Standard BD 28/87[41]. Eurocodes EC2 and EC7 refer to similar crack control requirements in reinforced concrete walls.

The available methods of design for multi-propped walls are:

(*a*) empirical methods; based originally on strut load envelopes proposed by Peck for three categories of soil: sands, soft to medium clays and stiff clays. Twine and Roscoe[42] more recently proposed new strut load envelopes for soft to firm clays, stiff to very stiff clays and dry clay or submerged granular soils. The use of empirical methods is recommended as a check on computed strut loads

(*b*) limit equilibrium programs are a simple solution without addressing all the matters of influence. Programs based on Winkler Spring theory are perhaps nowadays the most widely used methods

(*c*) full soil–structure interaction analysis by finite element, boundary element or finite difference methods: used where prediction of soil deformation and soil settlement requires calculated estimates

(*d*) pseudo finite element programs.

Empirical method based on strut load envelopes

The original empirical method, due to Terzaghi and Peck[18], was applied to both temporary works (including piled and diaphragm walls permanently anchored or braced by floor construction, as in top-downwards construction) and permanent works. The strut load envelopes due to Terzaghi and Peck are shown in Fig. 5.33. Note that these diagrams are not intended to represent actual earth pressure or its distribution with depth but load envelopes from which strut loads can be evaluated. Clay is assumed to be undrained and only total stresses are considered. Sands are assumed to be drained (through the sheeting) with zero pore pressure. Where drainage is precluded behind a non-permeable wall, hydrostatically distributed water pressure is added to strut loads. Sheeting or walling was then designed using the Coulomb earth pressure distribution with hydrostatic water pressure added except where drainage occurred through sheeting to relieve water pressure.

In 1969, reviewing his empirical method, Peck pointed out that his recommended method for strut design was less satisfactory in soft to medium clays

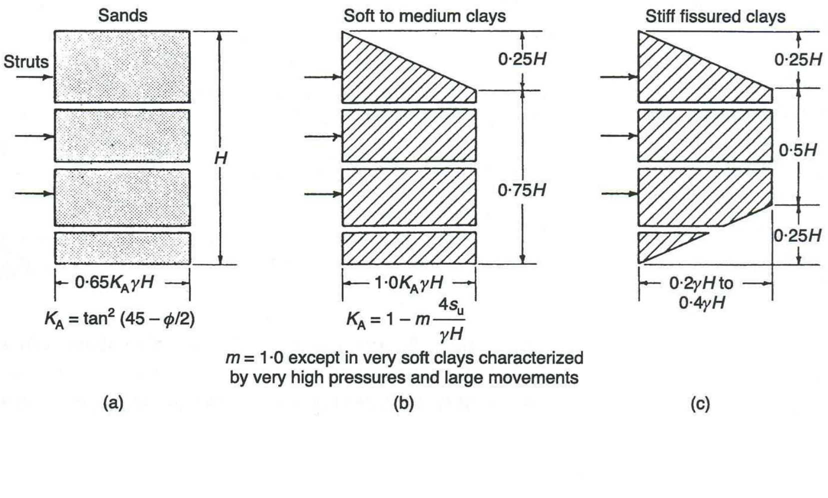


Fig. 5.33. Apparent pressure diagrams for computing strut loads in braced cuts[18]