Proceedings of the Sri Lankan Geotechnical Society

Seminar On

New British Code For Earth Retaining Structures BS 8002

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Proceedings of the SLGS Seminar on British Code for Design of Earth Retaining Structures BS 8002

Contents

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1. Developments in the field of Earth Retaining Structure Behaviour

1.1 Early Studies on Earth Pressure

As early as 1773, Coulomb identified two types of failure, an "active" and a "passive" due to the movement of the retaining structure away from or towards the retained soil respectively. Coulomb recognized that the orientation of the critical shear surface was a function only of the angle of friction of the soil. He also identified that the critical shear surface may not be planer, but can be sufficiently accurately approximated by a planer surface. He also noted that the water collecting behind retaining walls can increase the pressure on the wall and recognized the consequences of blocking weep holes. The possibility of heaving of the soil and the resulting pressure increases were also identified.

In 1808 Mayniel extended Columbus derivation to obtain expressions for earth pressures behind rough walls. The Mayniel solution was further extended in 1906 by Muller - Breslau to get the general expression for earth pressure due to a cohesionless soils allowing for sloping backfill, sloping back of the wall and wall roughness.

In 1857 through his paper 'on the stability of loose earth' Rankine derived a solution for a complete soil mass in a state of failure. With the Mohr's paper on the representation of stresses and strains in an element by a circle, Rankine conditions were conveniently illustrated by the "Mohr Circles". Mohr circles may be used with great effect to derive solutions for the Rankine analysis. In 1915 Bell extended Rankine's solution to allow for the effects of soil cohesion.

1.2 Effective Stress Concept.

Following the work of Terzaghi, concept of effective stress was proposed to have controlling influence on shear strength and compression. Subsequently earth pressures were expressed in terms of effective stress shear strength parameters c' and ϕ'

1.3 Experimental Studies

In the early 1920's and 1930's Terzaghi investigated active and passive states in a sand box by applying outward and inward rotations of the wall about the base. Triangular lateral stress distributions were achieved. Terzaghi predicted different stress distributions for other modes of wall deformation. Terzaghi (1936) identified that during horizontal translation of the wall, pressure distribution became hydrostatic only after passing through a parabolic distribution.

Terzaghi (1943) found though his models studies that a small movement of the order of 0.001 times the wall height was sufficient to reduce the lateral stresses to active values. Much larger movements of the order of 0.02 times the wall height were required for the mobilization of the passive state.

Taylor (1941) performed deformation modes of rotation about the top and lateral translation. A parabolic distribution in excess of Coulomb's theory was obtained for these tests. Ohde (1938) showed analytically that for a rotation about top arching action occur near the top of the wall.

All the above mentioned studies were for rigid translations or rotations. More flexible retaining structures may bend under the applied loads and lateral pressure distribution behind them can show significant deviations from the standard hydrostatic distribution.

In the case of walls where there is minimal or no lateral movements, lateral and vertical pressures are related by k_0 -the coefficient of earth pressure at rest. Jaky (1944) suggested that $k_0 = 1\text{-Sin}\ \phi$. This would be true for loosely filled sands and normally consolidated clays. The coefficient k_0 for a dense sand or a overconsolidated clay would be much greater.

1.4 Finite Element Studies

Finite element method overcomes most of the shortcomings associated with the conventional methods of retaining structure analysis. It can incorporate the real soil behaviour (appropriate soil stress - strain - model) and account for the soil - structure interaction effect. Initial ground condition and the construction sequence can be numerically simulated. Finite element method provides a complete solution to a static problem by satisfying the requirements of equilibrium, compatibility, material behaviour and boundary conditions simultaneously. This method has been deployed for the solution of many Geotechnical Engineering problems over the last two to three decades.

Most of the pioneering finite element work in retaining structure was confined to earth pressure studies. These studies assessed the variation of earth pressures due to different modes of imposed movements on a rigid wall. A wide range of deformation modes that a retaining structure could experience in its design life were considered by different researchers.

The variation of the total force on the wall with wall movement is illustrated in Figure 1. Sufficient outward movement of the wall would mobilize the limiting active conditions where as a sufficient inward movement of the wall would mobilize the limiting passive condition.

Potts and Fourie (1986) showed that initial earth pressure coefficient k_0 did not influence the limiting earth pressure. But a soil with a lower k_0 reached active conditions at a smaller movement. Thus the lesser movements needed for achieve stress mobilization is partly due to the closeness of “at rest” and “active” earth pressures. Potts and Fourie also showed that wall friction causes reduction in the limiting active pressure and an increase in the limiting passive pressure. Parabolic type modes of deformation used by Nakai (1985) produced “Smaller than active” and “Larger than passive” pressures at mid heights of the wall. (Ref. Figure 2).

Finite element method was also used successfully to numerically simulate the construction sequence of a retaining wall. Existing foundation conditions; construction of the structure and the backfilling with compaction in the case of gravity walls; installation of the wall and the excavation of soil in the case of embedded walls; Development of interface friction in the case of reinforced earth structures etc. can be accurately simulated numerically through the finite element method.

The method can predict the earth pressure distributions behind the retaining walls and very importantly the deformations associated with them. In urban situations it is necessary to have excavations very close to existing structures and retaining wall are expected to provide the

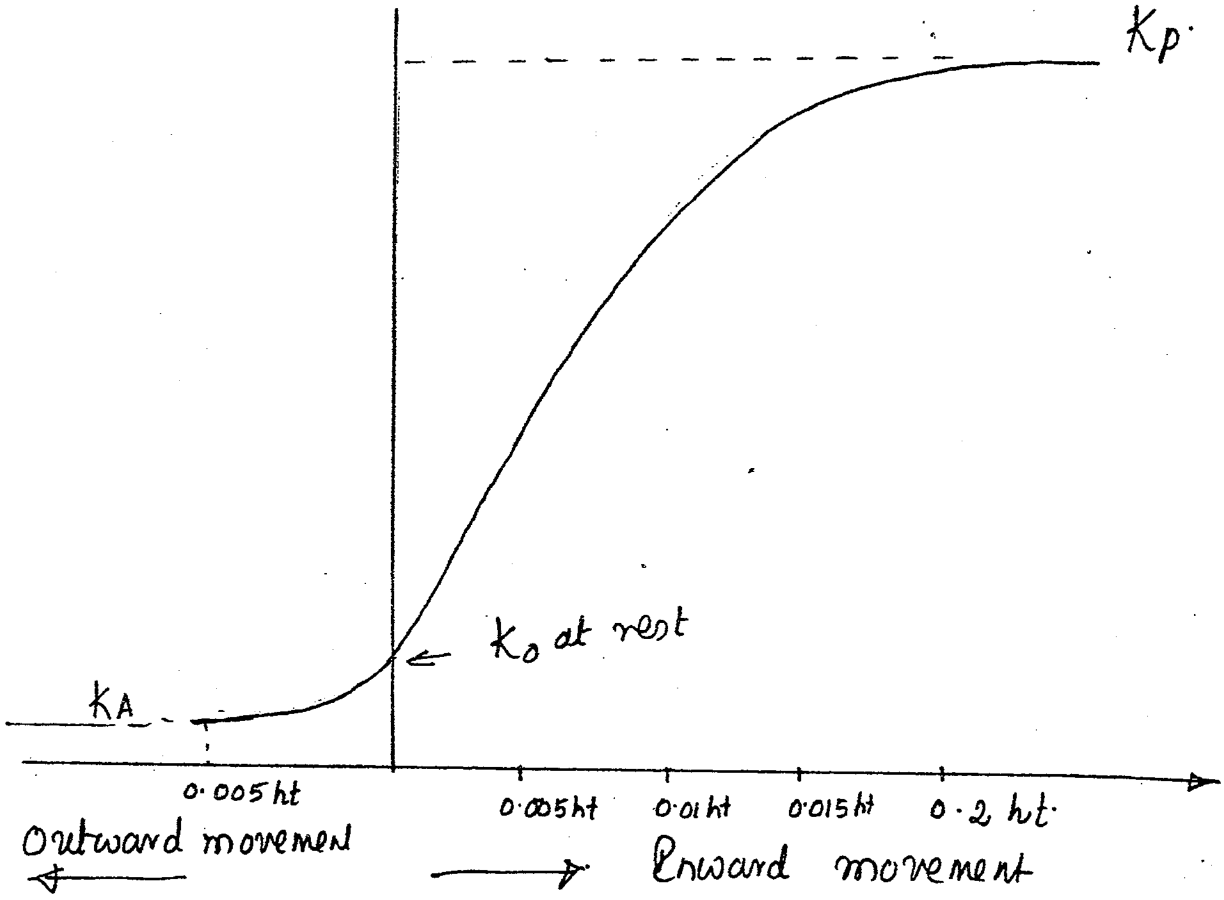
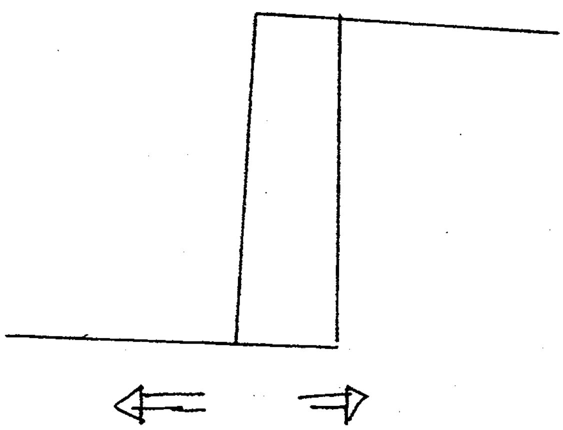

Kp
Ko at rest
KA
0·005 ht
0·005ht
0·01 ht
0·015 ht
0·2 ht
Outward movement
Inward movement

Figure- 1



necessary deformation control. Conventional design methods does not yield any information about movements in the wall or retained soil and finite element method is a very useful tool in this context.

Kulhawy (1974) simulated the incremental construction of a 31.8m high gravely retaining wall using the finite element method. The soil was modeled to have a non linear hyperbolic stress-strain behaviour. Two analyses were done with two types of backfill, namely soft and stiff soils. Earth pressures at the back of the wall were found to be only slightly reduced from the "at rest" conditions, due to insufficient wall movements (Figure 3).

Goh (1984), carried out a finite elements study on the behaviour of gravity retaining walls. Two types of backfill, a clayey sand - (cohensionless soil) and a sandy clay- (cohesive soil) were used in his study. In both cases soil was modeled as an elastic ideally plastic material and earth pressure distributions and the wall deformations were obtained. Analyses were done for foundation soils of different stiffness. This was represented by the ratio of $E_{fdnsoil}/E_{backfill}$. The ratio was changed from 1 to 100.

It was revealed that when the foundation soil is very stiff (ie $E_{fdn}/E_{backfill} = 100$), outward movement of the wall was inhibited by the rigidity of the foundation. As the wall movement is small earth pressures were only slightly reduced from the "at rest" values. (Figure 4). When the foundation is of lesser rigidly ($E_{fdn}/E_{backfill} = 1$), wall has moved out sufficiently and earth pressures were reduced to active values.

These results and many similar results obtained in experimental and numerical research confirmed the dependence of earth pressures on wall movements.

1.5 Effects of Compaction of Fill Behind Retaining Walls.

The area acquired by the filling behind a retaining wall is often used for some purpose and some compaction of the fill is carried out to minimise the in service settlements. Usually fill is placed in layers and compacted.

This incremental filling and compaction introduces additional lateral stresses on the wall. The classical theories of Rankine and Coulomb do not account for these compaction effects. But Terzaghi observed that the lateral earth pressures exerted by a compacted fill are much greater than that due a non compacted fill.

Subsequently considerable research had been carried out to understand the mechanism of compaction and the development of lateral pressures due to compaction. These studies were initiated following several retaining wall failures due to compaction behind basement walls. There were experimental studies done in laboratories in real size retaining walls and monitoring of field retaining structures.

Extensive research done in a real size retaining wall set-up in a laboratory at TRRL (Transport and Road Research Laboratory) in UK with different types of backfill material, has lead to a great understanding about the subject.

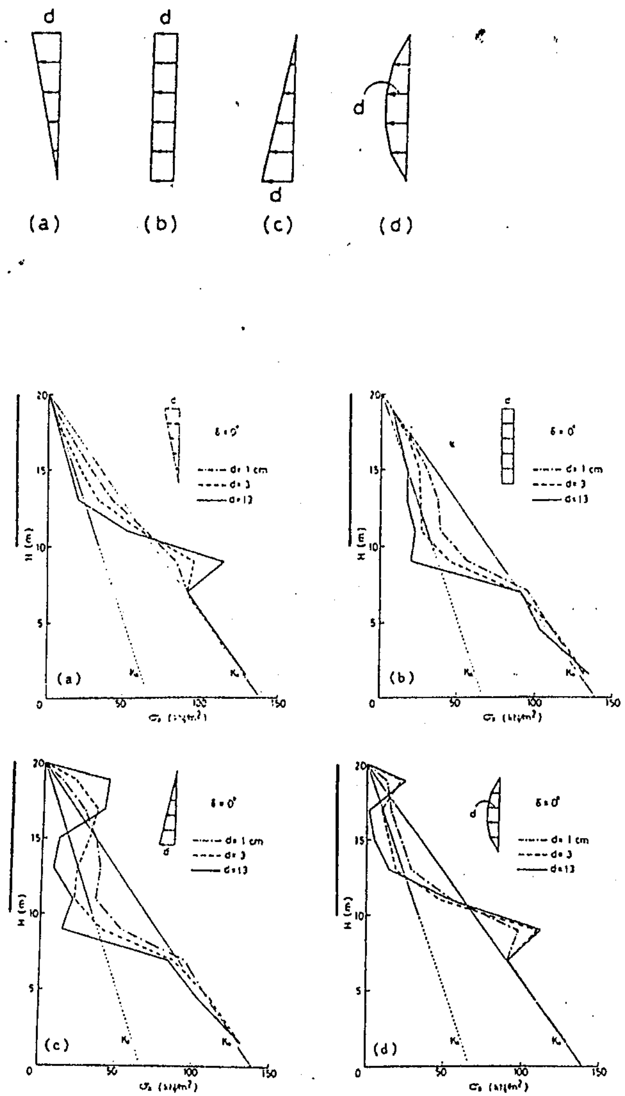


Figure 2 Active earth pressure distributions (after Nakai 1985)

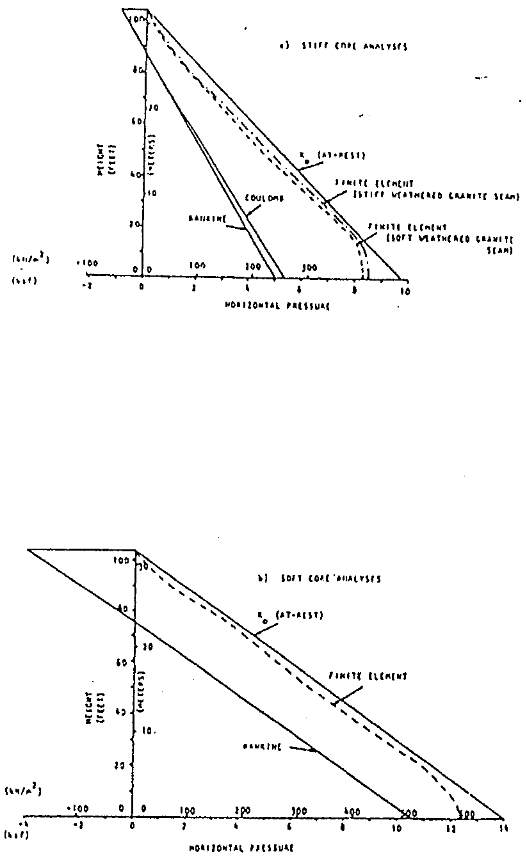
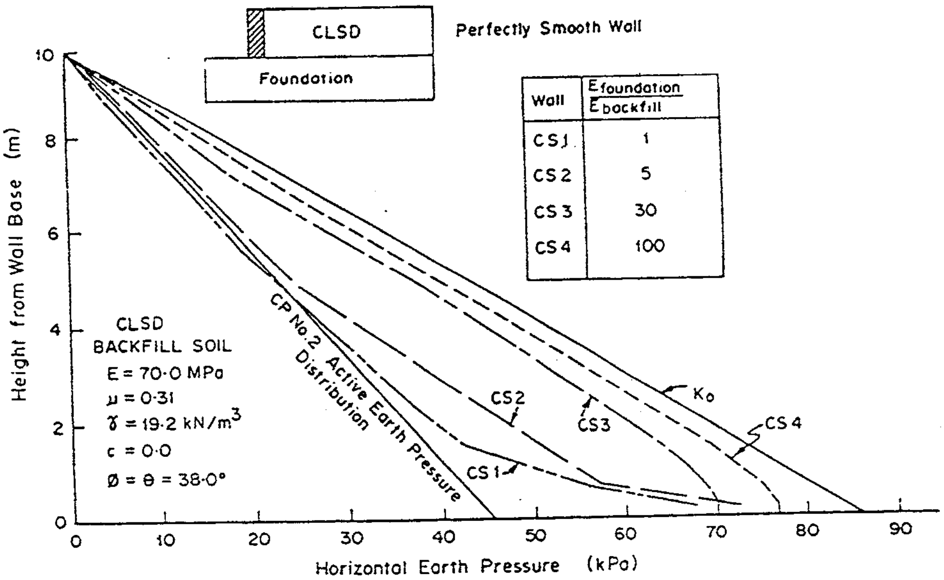


Figure 3 Earth pressure behind a gravity wall (after Kulhawy,1974)



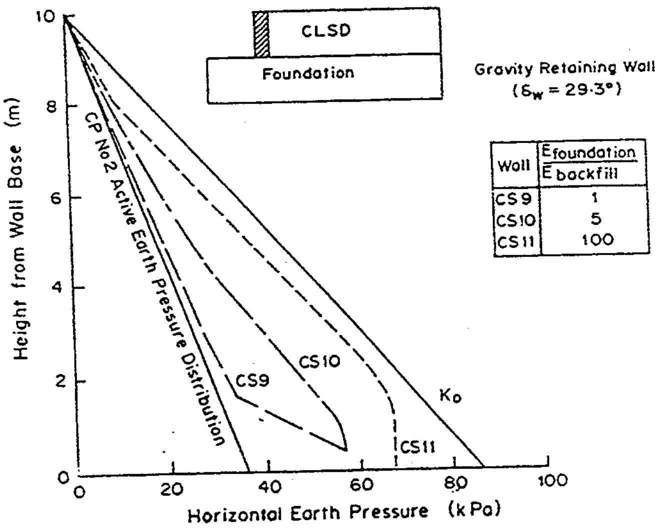


Figure 4 : Earth pressure behind a gravity wall CLSD (after Goh,1984)

Broms (1971) made the first theoretical formulation to predict compaction induced lateral stresses. Ingold (1979) made some modifications to the Broms formulation. Duncan (1983) used a different approach to simulate compaction and his formulation was incorporated in the finite element simulations done by Duncan & Seed (1983) and Kulathilaka (1990).

Theoretical Formulation by Broms (1971)

Based on laboratory and field observations Broms (1971) presented a method to determine the lateral earth pressure on rigid unyielding structures. It was presented in the form of a loading/unloading relationship. Simplified form of Broms formulation is presented in Figure 5. The initial earth pressure state of the soil is represented by point A'. There the soil has a lateral pressure of $\bar{\sigma}_{hi}$ corresponding to a effective overburden pressure σ_{vi}. When a compactor is working on the surface both the vetical and horizontal earth pressure will increase to a state given by point B'. On the removal of the compacted the stress changes will follow a path B' C' D'. Vetical stresses have reduced back to the initial values while the horizontal stresses have experienced an increase. This increase is referred to as "locked in stresses" or "residual stress increase due to compaction".
The variation of residual stress increases with the depth is as given by Figure 6 (a). Placement and compaction of soil will result in lateral stress profile given by Figure 6 (b). Based on that the design earth pressure distribution given by Figure 6 (c) is derived. Despite some limitations, Broms's proposed theory is the first theoretical explanation to agree even to some extent with many of the observations.

Extension of Broms's Method by Ingold (1979)

Ingold (1979) proposed a method as an extension to the Broms et al (1971) model. His intention was to account for the deflections of the retaining structure. This was done by assuming that the wall would yield sufficiently to create an active pressure condition in the backfill, while the compaction plant was still working on he surface. Ingold presented his idealised stress paths, during the compaction loading - unloading in the forms shown in Figure 7(a). In this the K_o line in the Broms's method has been replaced by the active pressure (K_a) line and the K'_o line has been replaced by the passive pressure (K_P) line.

Using this model, if a case where only the final fill layer was compacted was analysed, a lateral pressure distribution as given by Figure 7(b) could be obtained. If the backfill was placed in layers and each layer was compacted before the placement of the next one, a lateral pressure distribution as given by Figure 7(c) could be obtained. Ingold back analysed several experimental studies with his model and found a good general qualitative agreement.

Ingold further simplified the computation of compaction induced earth pressure in the following manners. The vertical stress increase setup by a roller is obtained by the expression derived by Holl (1941) for an infinitely long live line load on an elastic half space.

$$\Delta\sigma_v = \frac{2p}{\pi Z} \qquad \text{\_\_\_\_\_\_\_\_\_\_ (a)}$$

where p = load per unit length
Z = depth below the surface
$\Delta\sigma_v$ = vertical stress increase

By field measurements this equation was verified to be a good approximation for static rollers and the total force is to doubled in the case of vibratory rollers.

Thus at shallow depth, assuming γ_Z to be negligible the horizontal stress is given by

$$\sigma'_{hm} = \frac{2p\ ka}{\pi Z} \qquad \text{\_\_\_\_\_\_\_\_\_\_ (b)}$$

at the critical depth $\sigma'_{hm} = k_p\ \gamma Z_c$ ________ (c)

from (b) & (c)

$$\therefore k_p\ \gamma Z_c = \frac{2p}{\pi Z_c}\ k_a$$

* Note $k_p = 1/k_a$

$$Z_c = k_a\sqrt{\frac{2p}{\pi\gamma}} \qquad \text{\_\_\_\_\_\_\_\_\_\_ (d)}$$

and the maximum residual horizontal earth pressure, after removal of the roller, is given by

$$\sigma'_{hrm} = k_p\ \gamma Z_c \qquad \text{\_\_\_\_\_\_\_\_\_\_ (e)}$$

$$= k_p\ \gamma k_a\ \sqrt{\frac{2p}{\pi\gamma}} = \sqrt{\frac{2p\gamma}{\pi}} \qquad \text{\_\_\_\_\_\_ (f)}$$

for a point at a depth where compaction pressures are insignificant, maximum compaction pressure is equal to the active pressure and

$$\sqrt{\frac{2p\gamma}{\pi}} = k_a\ \gamma h_c$$

$$h_c = \frac{1}{k_a}\ \sqrt{\frac{2p}{\pi\gamma}} \qquad \text{\_\_\_\_\_\_\_\_\_\_ (g)}$$

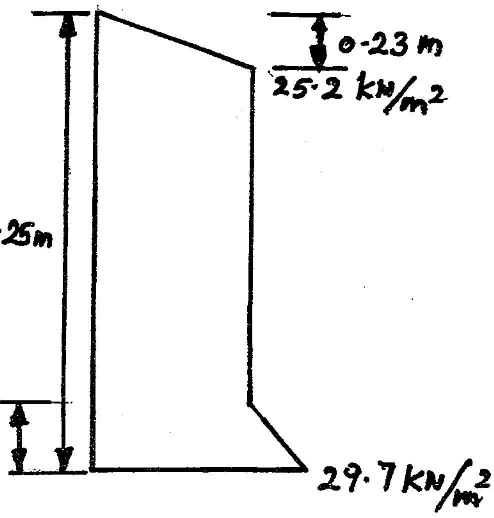
Ingold's method provides a very simple pressure distribution. (Figure 7). However the amount of lateral yielding (movement) of the wall required to mobilize active pressures is not known. Both methods produced similar expressions. k_o and k_o' are used in Broms theoty and k_A and k_p are used in Ingolds theory. BS 8002 recommends to use compaction induced lateral earth pressures through either Broms theory or Ingolds theory wherever necessary.

Hong Kong guide to retaining wall design suggest to use the Ingold theory for wall that can prove forward sufficiently to mobilize active conditions. For unyield walls they recommend the use of Broms theory.

Example

* Calculate the earth pressure behind a 8.25m high retaining wall, due to incremental backfilling and compaction.

p = 50 kN/m run $\qquad$ k_a = 0.18 $\qquad$ γ = 20 kN/m^3



$$\sigma'_{hrm} = \sqrt{\frac{2p\gamma}{\pi}} = \sqrt{\frac{2\times50\times20}{\pi}} = 25.2 \ kN/m^2$$

$$Z_c = k_a \sqrt{\frac{2p}{\gamma\pi}} = 0.18 \sqrt{\frac{2\times50}{20\times\pi}} = 0.23m$$

$$h_c = \frac{1}{k_a}\sqrt{\frac{2p}{\gamma\pi}} = \frac{1}{0.18}\sqrt{\frac{2\times50}{\pi\times20}} = 7.01m$$

σ_h at 8.25 m = $k_A\gamma H$
= 0.18 x 20 x 8.25
= 29.7 kN/m^2

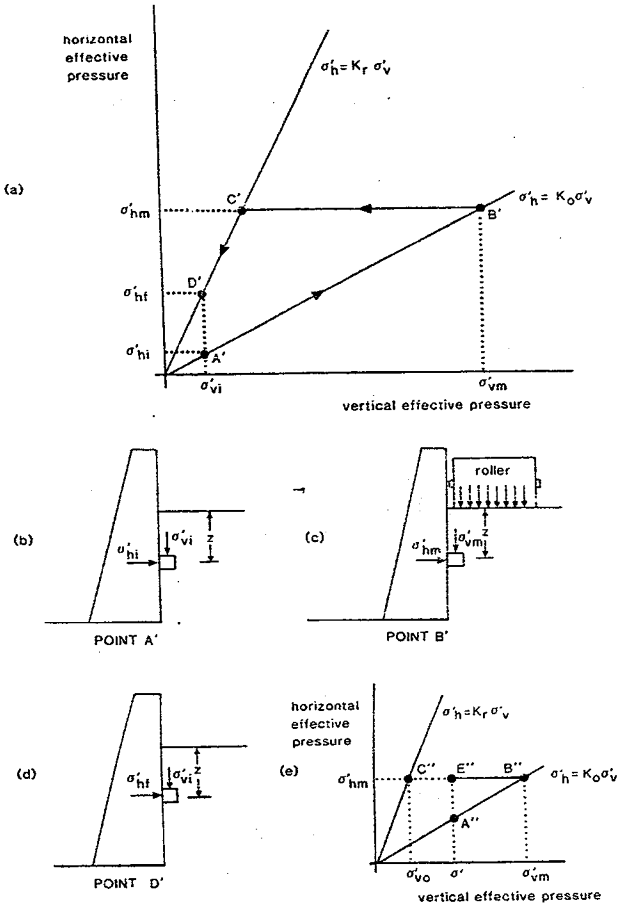
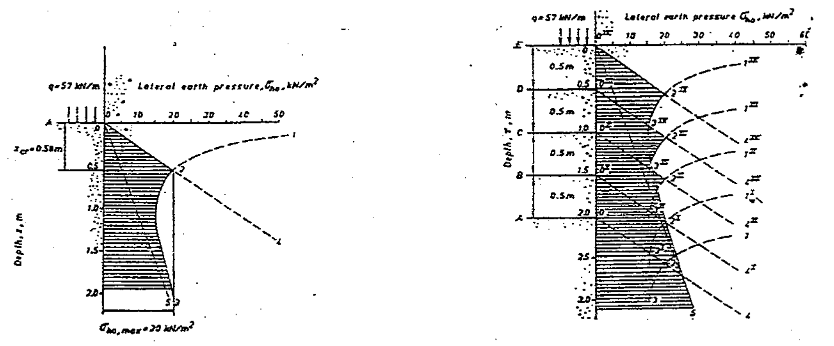


Figure 5 - Brom's Simplified Compaction Pressure Theory

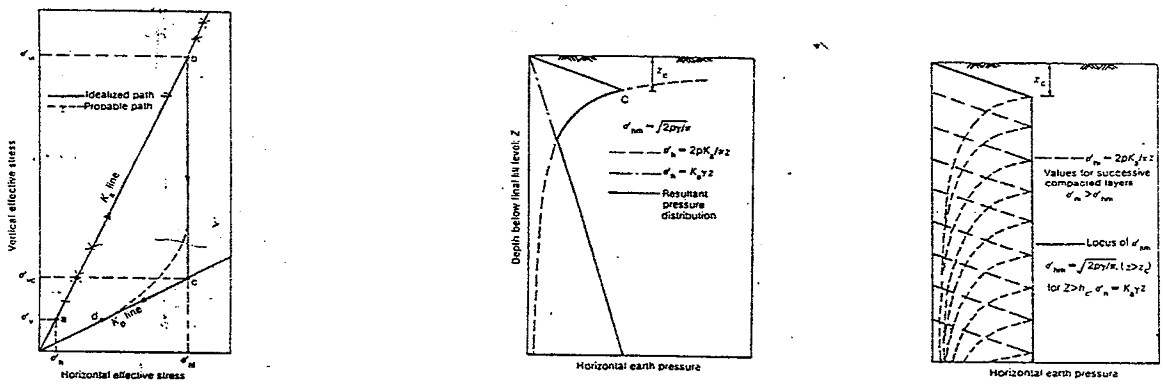


(a) Earth pressure distribution for a 10.2 ton smooth wheel roller -one layer-.

(b) Earth pressure distribution for a 10.2 ton smooth wheel roller -layered backfill-.

(c) Proposed design earth pressure distribution

Figure 6 - Earth Pressure during Compaction - after Broms 1971



(a) Idealised stress path

(b) Earth pressure distribution -one layer-

(c) Design earth pressure

Figure 7 - Extension of Broms Method for Yielding Walls - after Ingold 1979

2. <u>Type of Retaining Structures Covered by the Code</u>

BS 8002 deals with the earth retaining structures that can be classified as externally stabilized system. Internally stabilized systems such as reinforced earth, anchored earth etc., are covered by the Code BS 8006 (1995).

The main feature of the externally stabilized systems is the presence of a structural wall to take up the pressure exerted by the soil. The supporting action of the wall could have been derived from

(a). their weight
(b). their flexural rigidity or
(c). by combination of both the above

Gravity retaining walls comes under category (a) and embeded retaining walls with or without lateral supports falls into category (b). Reinforced concrete L or inverted T type walls rely on their flexural rigidity for internal stability but derive overall stability due to the weight of soil contained in side falls under category (c).

Different types of walls covered by BS 8002 are

(a). Gravity walls

(i). Mass concrete walls
(ii). Unreinforced Masonry walls
(iii). Gabion walls
(iv). Crib wall systems

(b). Embedded walls

(i). Steel sheet piling
(ii). Timber sheet piling
(iii). Reinforced and prestressed concrete sheet piles
(iv). Insitu diaphragm walls
(v). Bored pile retaining walls
(vi). Soldier pile walls
(vii). Strutted excavations
(viii).Cofferdams

(c). Reinforced concrete and reinforced masonry walls

2.1 Gravity retaining structures

Gravity retaining structures rely on their weight to ensure stability of the retained soil mass. (Over turning moments and sliding forces created by retained soil are resisted by the self weight). Gravity retaining structures were traditionally made of mass concrete, rubble or masonry. Gabion walls and crib wall systems are more recent developments falling into this category.

A small amount of back batter (up to about 10°) at the back face of the retaining wall can significantly reduce the earth pressure acting on the wall and even out the bearing pressures transferred to the ground.

Mass concrete retaining walls

Mass concrete walls are suitable for retained heights up to about 3 m. They can be designed satisfactorily for greater heights, but as the height increases other types become more economical.

Some forms of mass concrete retaining walls are depicted in Figure 8. When a battered wall is impracticable back or front of the wall can be made stepped or inclined.

Masonry or Rubble Retaining Walls

Unreinforced masonry or rubble retaining walls are suitable for small retaining heights, especially where the finished appearance is important. They do not demand any special construction plant. A simple uniform stem wall is suitable for a height upto about 1.5 m. For greater retained heights stepped or buttressed retaining walls would be required. (Figure - 9)

Gabion Walls

The main unit in a gabion wall is the gabion. Gabions are large cages or baskets usually made of woven steel wire or square welded mesh. These cages are filled with rock fragments and are tied together. They are highly flexible and can tolerate significant settlements. Hence gabion walls are particularly suitable for sites with compressible foundation material and sites which are likely to become saturated. They are widely used in water front structures. (i.e with canals, rivers and harbours). Gabion walls are relatively simple to construct and can be constructed quickly without the help of any special machinery. Front or the rear faces of the gabion may be straight or stepped and a back batter of around 1 : 10 is used with walls taller than 3m to improve stability and even out bearing pressure on the ground.

Cages are of sizes such as 1m x 1m x 1m, 1m x 1m x 2m, etc and are usually made of galvanized iron wires. In some situations these wires are covered with a PVC coating.

In recent times gabion walls were also made of wickerwork, bambooslats, nylon or poly propylene. But there are reported instances of fire damages to gabion walls constructed from flammable materials.

Figure 10 presents the basic gabion unit and some gabion retaining walls.

Gabion walls are extensively used in Colombo in the canal rehabilitation work. Gabions are highly permeable structures and no special from of drains are required. Usually filters made of geotextiles are used at the back of the structure to prevent clogging by fine particles eroding into the wall.

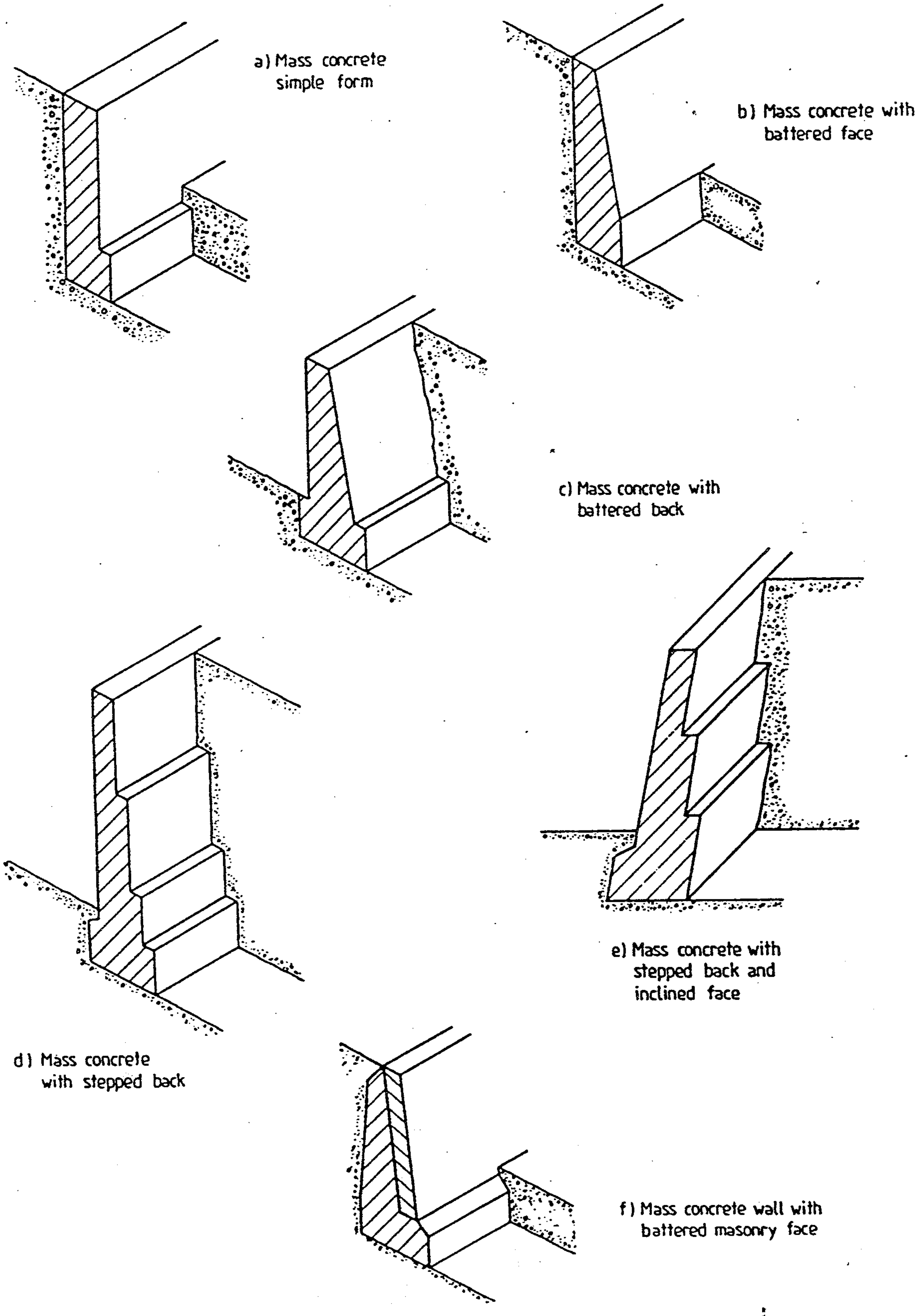


Figure 8- Basic Forms of Mass Concrete Walls

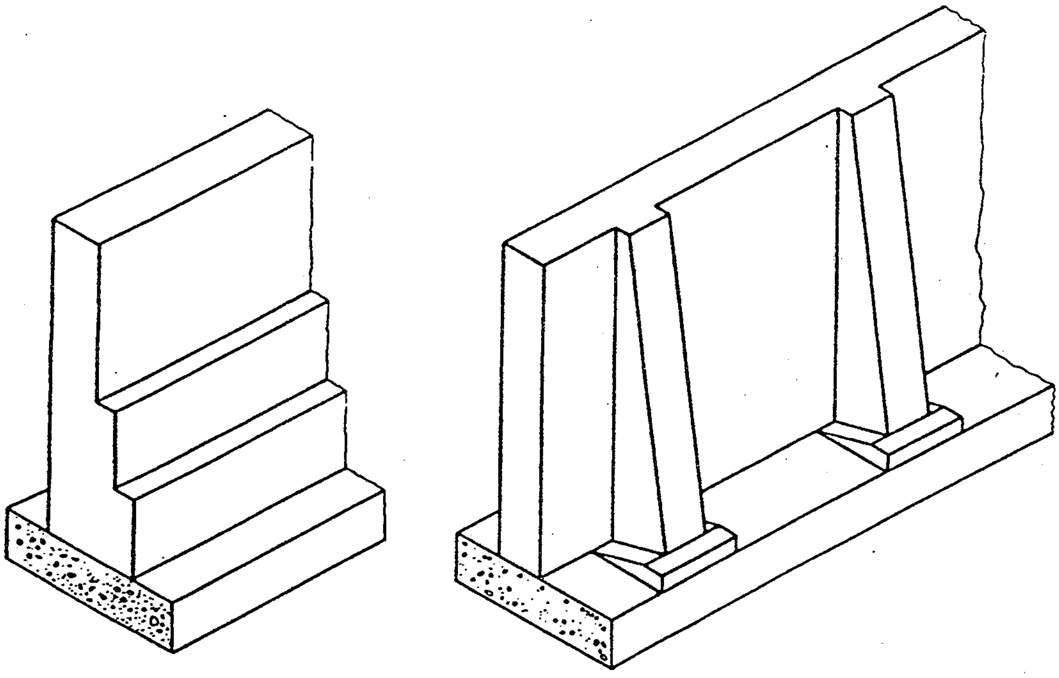


Figure 9 - Stepped and Buttressed Retaining Walls in Reinforced Masonry

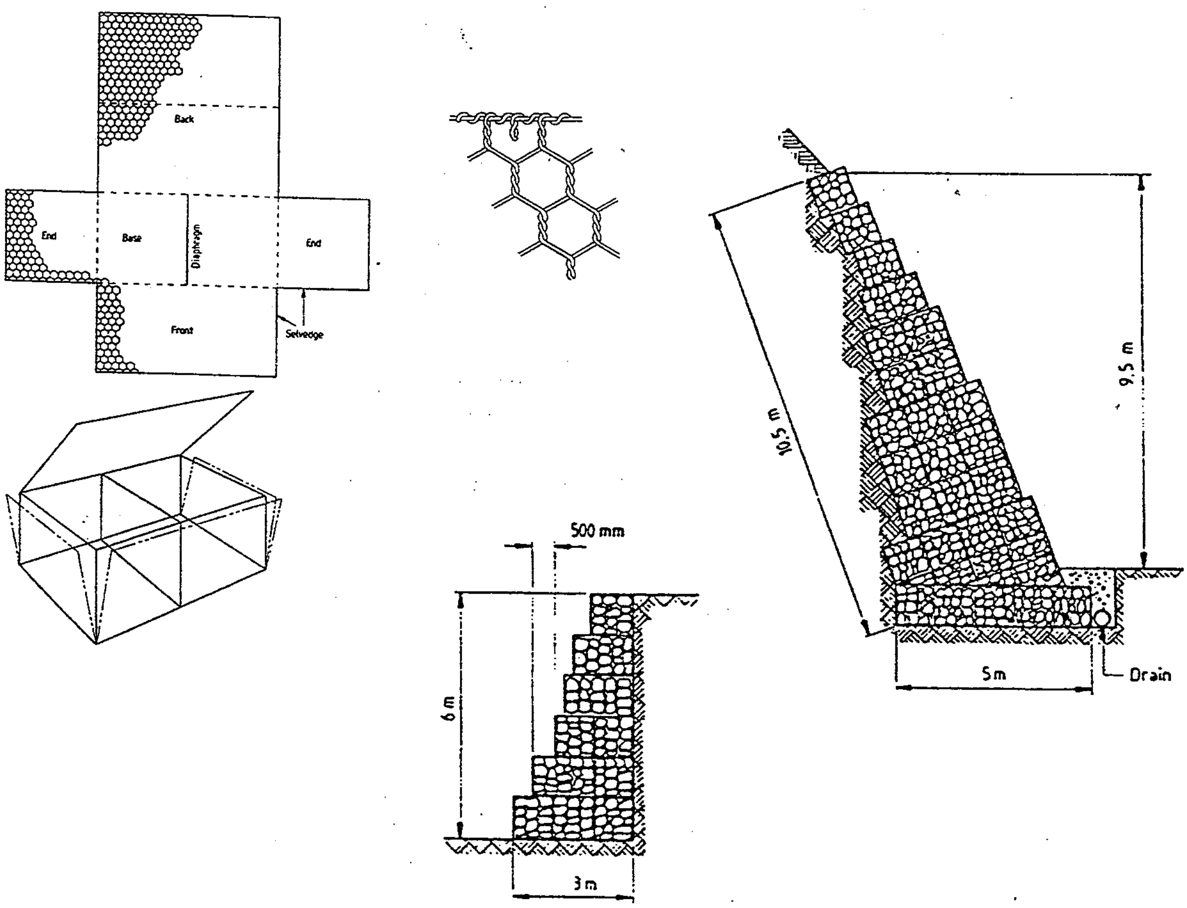


Figure 10 - Gabion Walls

Crib Walls

Crib walls are built up of individual prefabricated units assembled to create a series of box like structures containing suitable granular free draining fill.

Crib units are mainly made up of either timber or reinforced concrete (precast). Both sawn timber and timber logs are used in the construction. Timber used should be treated for durability. Crib walls are occasionally made with Aluminium also.

The front face of a typical crib wall consists of a grid of units spaced at close intervals so that the infill does not spill through the units. Horizontal members of the grid are known as 'stretchers'. These are connected by transverse members known as 'headers' to a similar grid of stretchers paralleled to the face, forming the back face of the crib wall. Where required, spaces may be used between stretchers at the front or back grids to provide additional support. They can form either a open faced or a closed face structure.

The width of a crib wall is dictated by the length of the standard precast concrete header units available. The minimum width of a crib wall should be around 1.2m. Walls of smaller heights may be vertical but walls higher than 2m are usually built with a batter.

Crib walls built of precast concrete units are very sensitive to differential settlements and there may be difficulties in building walls higher than 7m. Crib walls are normally built in straight length. But with special units curved walls can also be constructed.

Different forms of Crib walls are illustrated in Figure -11.

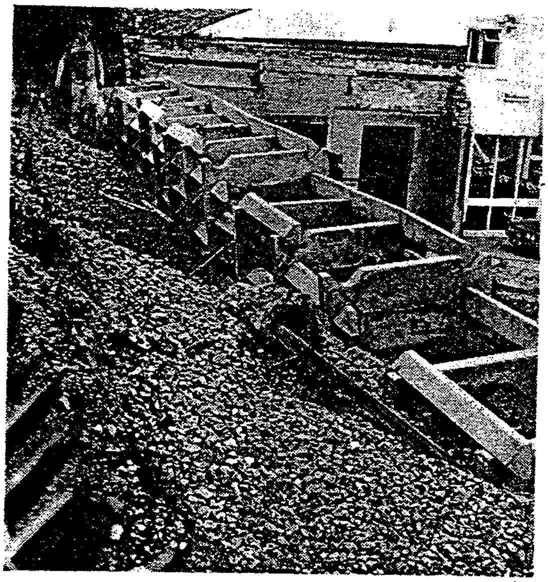
2.2 Embedded Retaining Structures

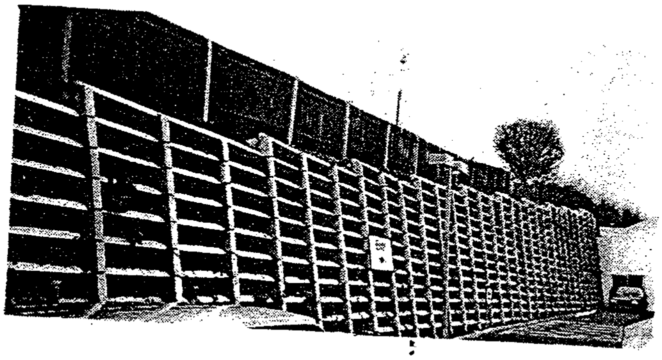
Embeded retaining structures with or without any form of lateral support rely on their flexural rigidity to support or retain the soil. They are generally thin sheet like structures, tradinally made up of steel or timber and referred to as sheet piles. Advances made in the sturry trench excavation technique led to the construction of insitu concrete diaphragm in walls and bored piles walls within the last 2 to 3 decades.

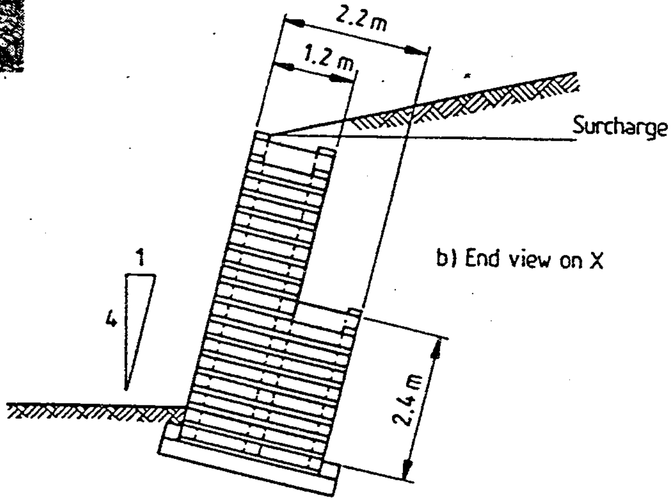
Embeded retaining structures that are not supported laterally, are referred to as cantilever walls and derive their equilibrium from the passive reistance of the soil below the excavation level. When the structure is laterally supported it is refered as a proped or anchored wall and the equilibrium is derived partly from the passive resistance of the soil and partly from the anchorage or prop system

Sheet Pile Walls

Sheet piles made of steel, timber or precast concrete are driven to the desired depth from the ground surface. Thereafter the excavation can be carried out to the required level. The method of installation does not demand any foundation and construction can be carried out in unfavourable (soft) ground conditions and also in the presence of water.

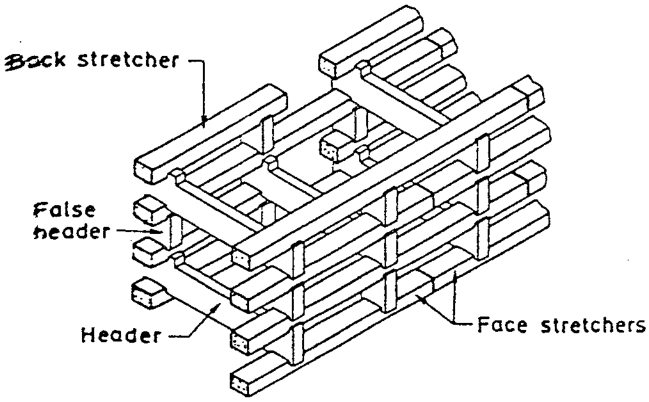




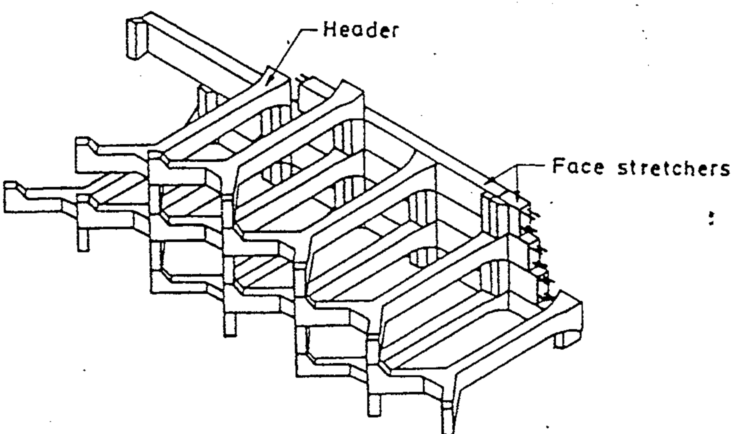


Type I - Open Faced System

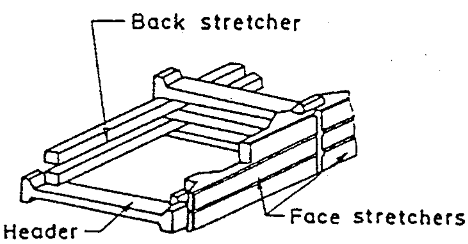
Type II & Type III - Closed Face System



(a) Type I



(c) Type III



(b) Type II

Figure 11 - Crib Walls

Wood and steel sheet piles are generally used only for temporary work. Reinforced concrete elements are used for more permanent structures. Figure 12 presents some of the standard sheet pile sections used.

Precast concrete sheet piles with tongue and grove type arrangement can be made watertight by grouting. Use of prestressed concrete elements would prevent possible cracking by the elemination of tension zone. However the high weight of the concrete elements and displacement of the soil during driving (due to thicker sections) are their disadvantages over the steel sheet pile walls.

Steel is the most commonly used material in waterfront construction, due to a number of advantages such as; variety of cross section with a wide range of strength, economy, lack of buckling under heavy driving, availability of different combinations to increase wall section modulues, reusability for temporary works, relatively light weight, and the possibility of increasing the pile length by welding or bolting.

Cantilevered sheet piles are economical only for small to moderate retained heights upto 3 - 4m. The required depth of embedment, the maximum bending moment in the wall and the lateral deflection increases rapidly with the height.

For higher retained heights sheet pile walls need to be provided with lateral supports in the form of anchors or props. Anchors may be applied at a single level or at several levels. The anchor acts to reduce the lateral deflection, the bending moment and the required depth of embedment of the wall.

Many of the problems reported with anchor sheet piling are related to wrong design or poorly constructed anchorages. Basic types of anchors are presented in Figure 13.

Insitu Concrete Diaphragm and bored-pile walls

Diaphragm Walls

Diaphragm walls constructed using the sturry trench excavation techniques and walls constructed by a continuation of bored piles are widely used at present. They are designed to be part of the final structure and are used in association with deep basements of buildings, underground transport facilities underground parking spaces, cut and cover tunnels etc.

Three factors have contributed to the expansion of the use of this type of construction :

(a). The commercial availability of bentonite
(b). Experiences of construction in urban areas and under difficult or troublesome soil conditions.
(c). Resolution of certain practical problems such as improvement in excavation techniques, and the development of on-site plants for processing slurries.

Diaphragm walling help to avoid the need for underpinning and ground water control and would allow maximum use of a small plot in a crowded city.

The construction sequence for a continuous slurry trench diaphragm wall using modern methods and equipment is illustrated in Figure 14 & Figure 15.

As illustrated in Figure - 14, the conventional diaphragm wall is constructed in sections. A section is also refereed to as a panel. End of each panel is defind after excavation and before concreting either by a steel tube stop-end or the end of an already completed panel. Excavation is made under bentonite slurry using a purpose built grab. The bentonite provides wall support removing the need of any mechanical support systems. At the surface concrete 'guide walls' are constructed prior to the commencement of the excavation to retain the slurry.

As excavation proceeds the trench must be kept full of bentonite slurry. After the placing of stop ends and the reinforcing cage, the bentonite slurry must be pumped either to waste or storage, as the concrete is placed from the bottom using tremie pipes. Once the concrete hardens to some extent the stop ends can be removed. The shape of the stop ends ensure that the adjacent panels interlock. A typical arrangement of a prestressed concrete diaphragm wall (post tensioned) is illustrated in Figure 17.

Prefabricated panel walls are also in use. However they have a limitation in the maximum managable height and width.

Bored Pile walls

Bored pile walls have the advantage that they can be constructed in almost any ground conditions. Bored pile walls may be;

(a).	Intermittent	-	Space exceeds diameter spacing > diameter	
(b).	Contiguous	-	piles in contact s = d	or
(c).	secant	-	piles interlocking,	

depending on the support requirement of the soil and the need to control groundwater. The top of the piles will often be capped by a reinforced concrete beam to distribute loads. An intermittent bored pile walls is shown in Figure 18 and a secant pile wall is illustrated in Figure 19.

2.3 Soldier Pile Walls

These consist of vertical members built at suitable centres with a system of ground support spanning between them. The piles are first installed along the perimeter of the proposed excavation. Sheeting, supporting the ground, is placed in position as excavation proceeds. The sheeting span either horizontally between the soldier piles or vertically between horizontal walling. (Figure 20 and Figure 21). Sheet piles inter locking with H - Section piles are also commonly used.

Solider piles may be used to support deep narrow, shallow or wide excavation in either sandy or clayey soil. Special care must be taken in water bearing ground. If soil is washed out from behind the sheeting unacceptable settlements would occur in supported soil.

2.4 Strutted Excavation and Cofferdams.

Different types are listed as

(a). Single skin cofferdams and strutted excavations
Embedded walls supported by a frame work within the cofferdam or by external anchorage.

(b). Cofferdams for river crossings
Used when a pipe line is to be laid across a river and it is impracticable to close the waterway.

(c). Earth-filled double wall and celluar cofferdams
They are self supporting gravity structures, either paralled sided double wall cofferdams or cellular cofferdams. The stability is dependent on the properties of filling and the soil at foundation level, as well as on the arrangement and type of the steel sheet piling.
Typical uses are, as dams to seal off temporary dock entrances so that work below water level can be carried out dry and in construction of permanent walls for land reclamation.

2.5 Walls Depend on flexural rigidity and gravity.

Reinforced concrete cantilevered walls in the form "L" or inverted "T" provide their supportive action through a combination of weight and flexural rigidity.

Stem of the wall should be of sufficient flexural rigidity to withstand the bending moments due to earth pressure from the backfill. Higher earth pressures that may exist due to backfill compaction should be considered in their design.

Weight of the soil resting on the heel portion of the wall acts together with the wall to provide resistance against overturning and sliding. The vertical surface going through the heel of the wall is referred to as the vertual back of the wall.

The very simple form of L or inverted T walls are suitable for retaining heights less than 6 m. For greater heights, width of the stem required to withstand bending moments becomes very great and the practice is to provide an increased stiffness by counterforts or buttresses.. In effect they reduce the bending moments and shear forces transferred to the stem. For simple "L" or inverted "T" shaped retaining walls wall stem is designed as a cantilever. With counterfort or buttressed retaining walls stem is designed as a continuous slab. Counterforts are designed as cantilevers.

In counterforts walls the bracing is provided at the back of the wall connecting the stem and base.. Hence it is subjected to tension. In buttressed walls bracing is in front of the wall and is subjected to compression. Buttressed walls are seledom used in practice.

Overall stability analysis often shows that the sliding resistance is a problem. Sometimes it may be necessary to increase the sliding resistance by providing a shear key. (a downward projection of the base slab). Common forms of reinforced concrete walls are illustrated in Figure 21.

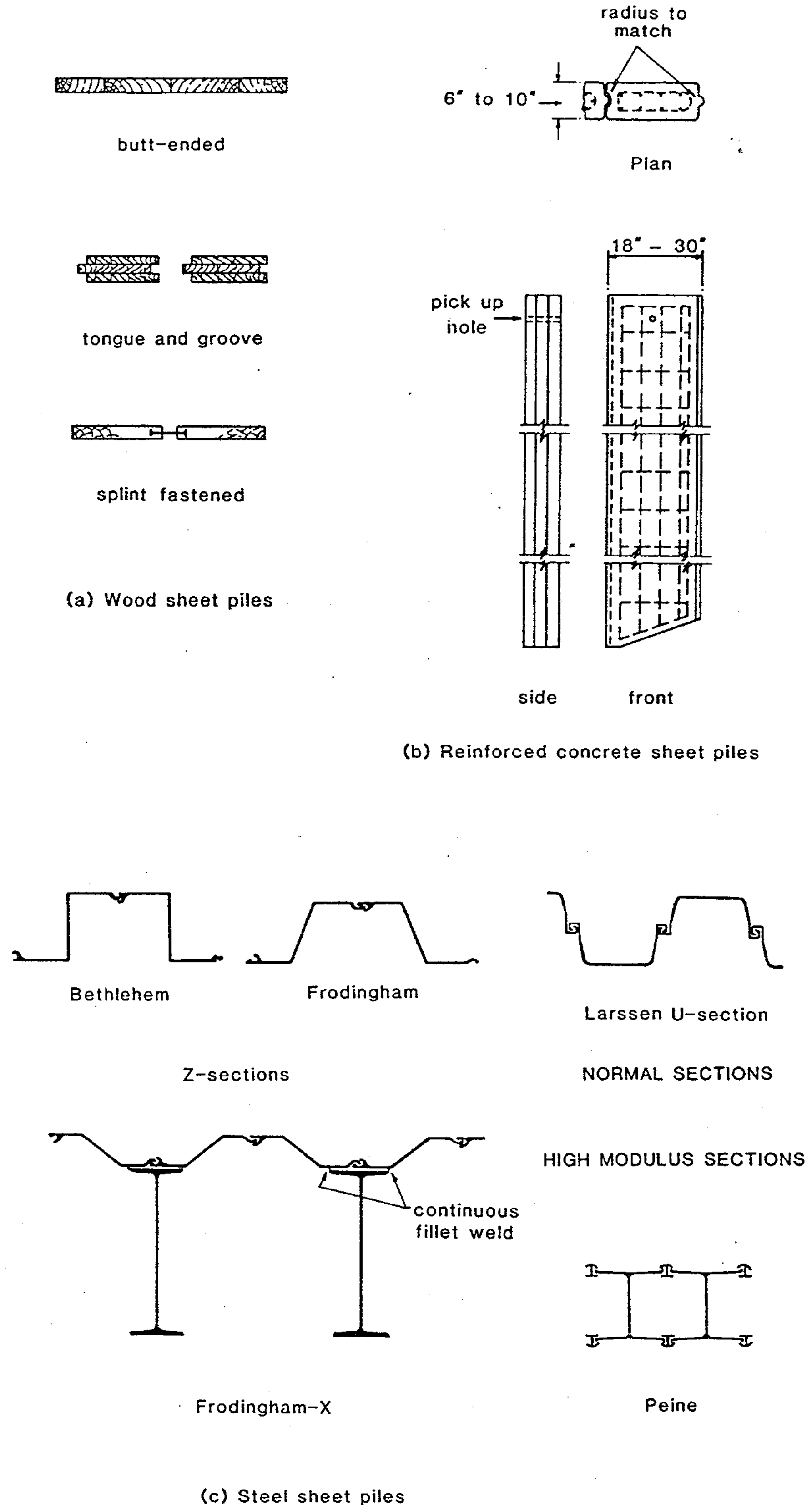
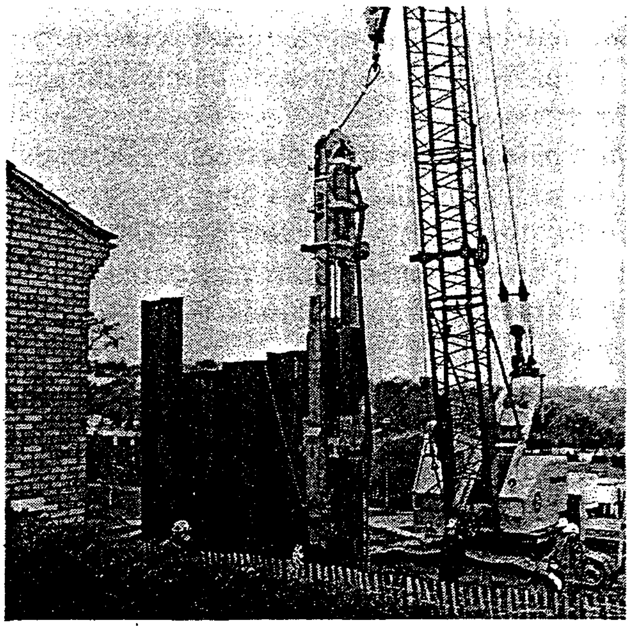
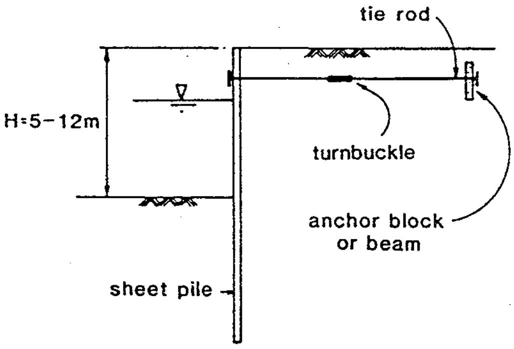


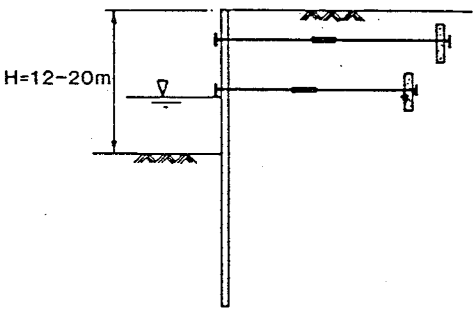
Figure 12 - Standard Sheet Pile Sections



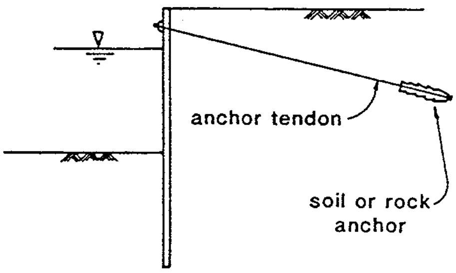
Driving Sheet Piles



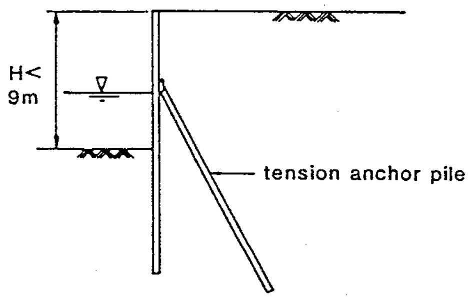
Single anchor level



Multiple anchors



Soil or rock anchors



raking pile anchor

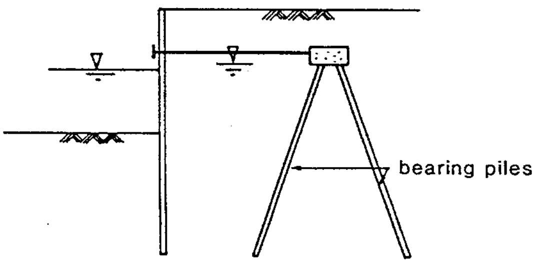


Figure 13 - Anchored Sheet Pile Wall Sections

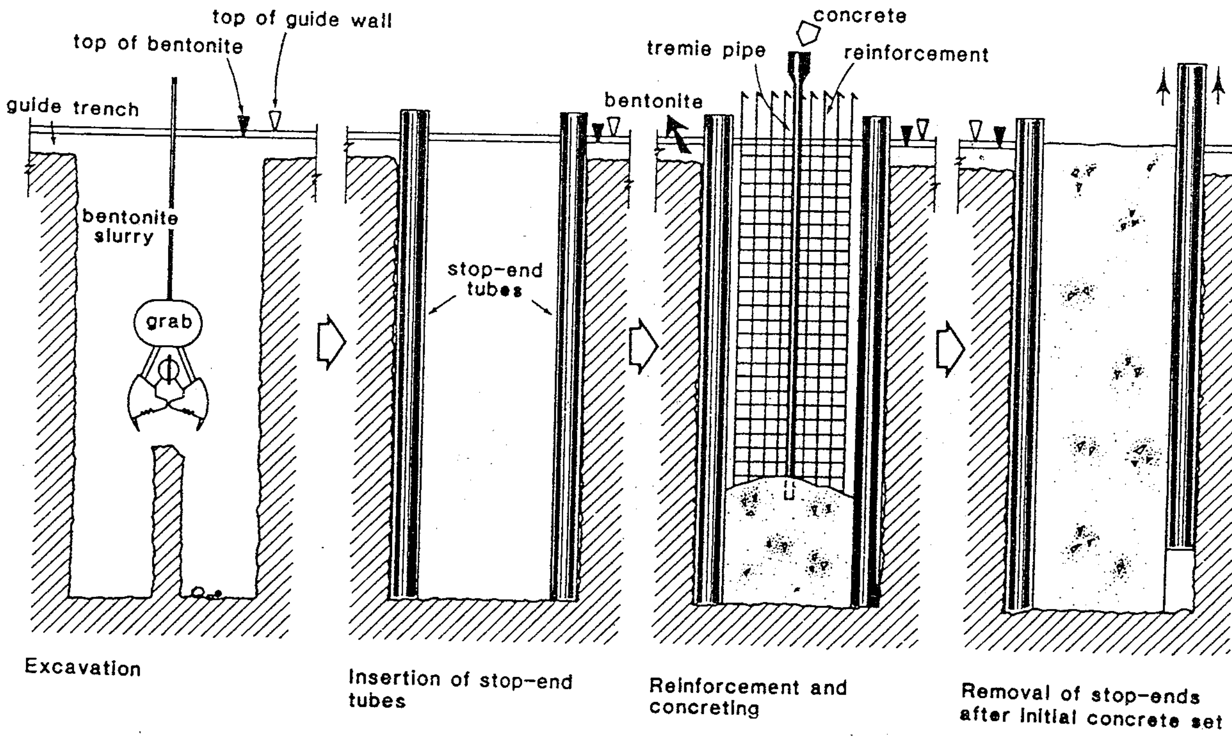
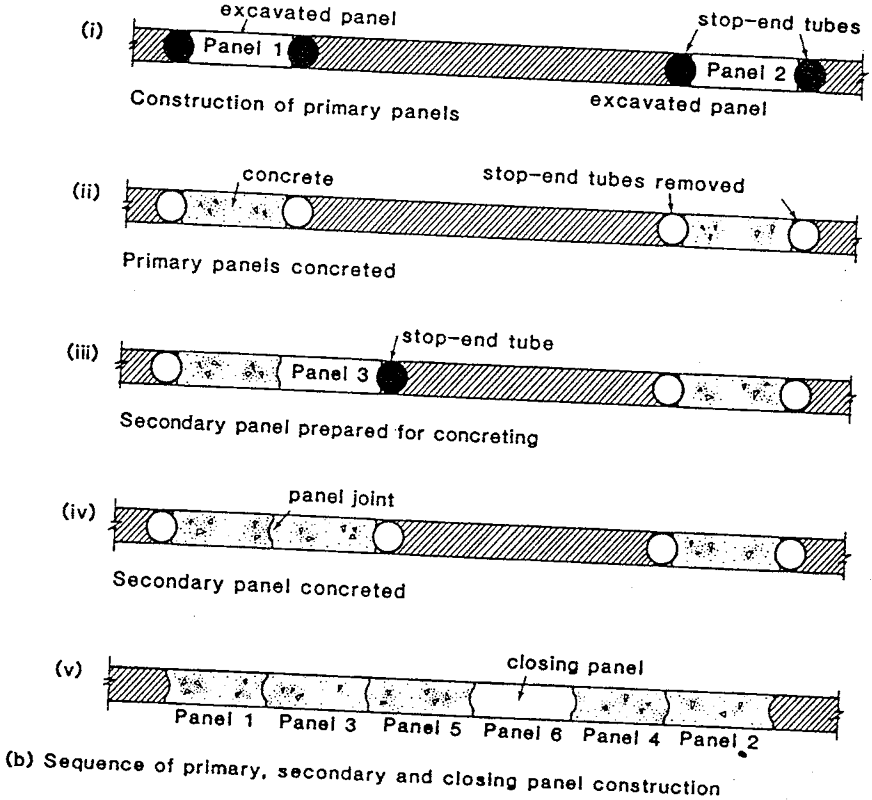


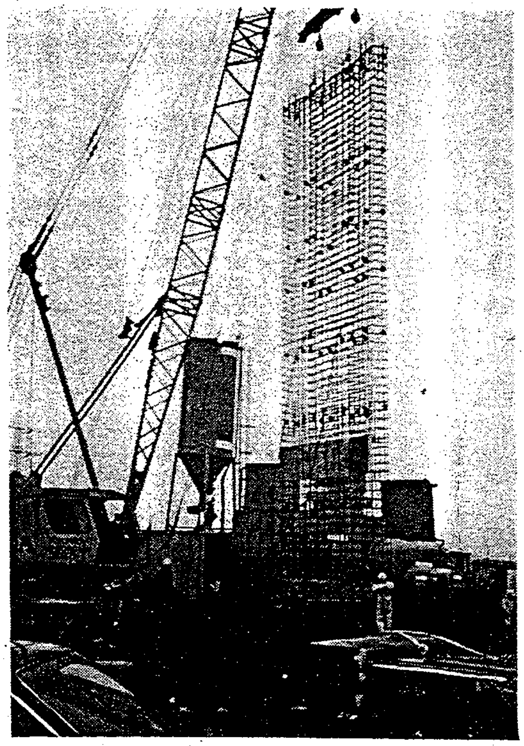
Figure 14 - Construction Sequence for a Primary Panel



Diaphragm wall construction (after Leiper, 1984).

Figure 15 - Diaphragm Wall Construction

Clam Shell Excavation Slurry trench panel



Reinforcement cage being lowered into the slurry trench

Figure 16 - Diaphragm Wall Construction

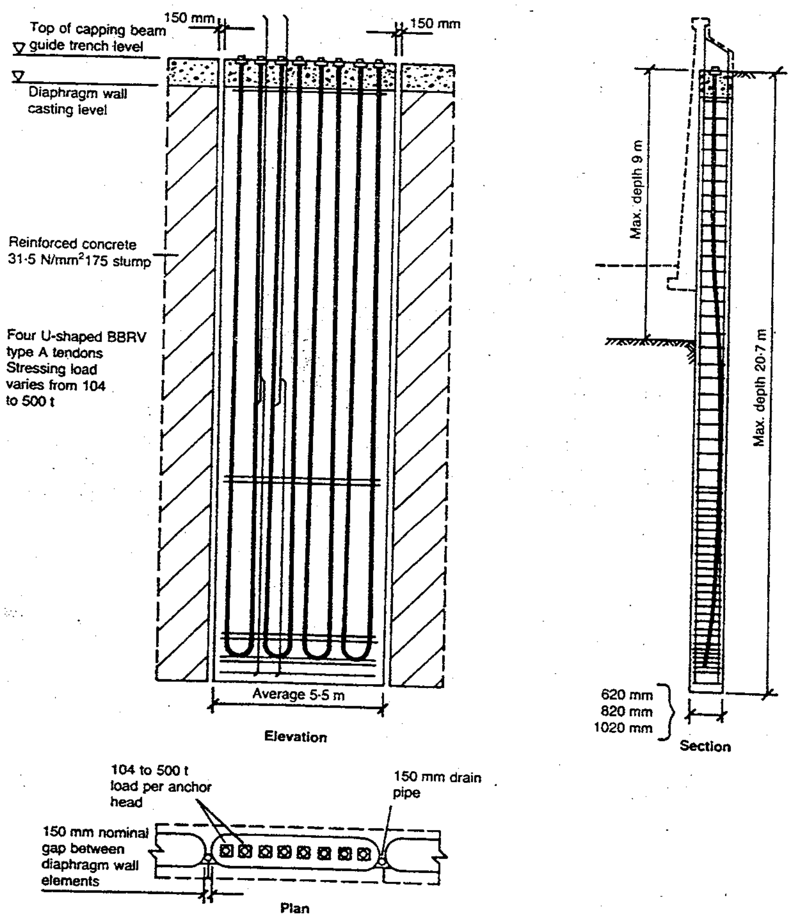


Fig. 4.32. Typical diaphragm wall reinforcement cage with curved tendons, Irlam O' Heights near Manchester, UK (Fuchsberger and Gysi[21])

Figure 17 - Prestressed Concrete Diaphragm Wall - Typical Reinforcement Cage

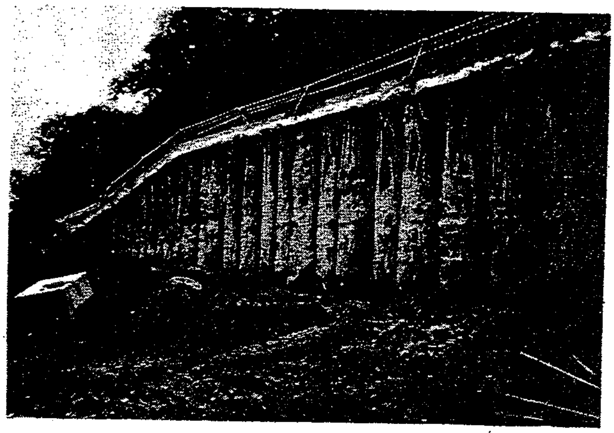


Figure 18 - Intermittent Bored Pile Wall

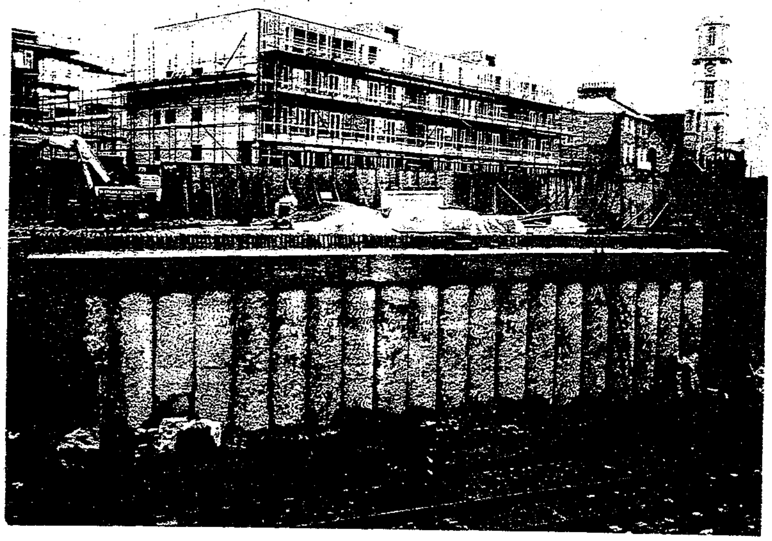


Fig. 4.15. Secant piles, Piccadilly Line underground railway, Heathrow Airport (courtesy of Lilley)

Figure 19 - Secant Bored Pile Wall

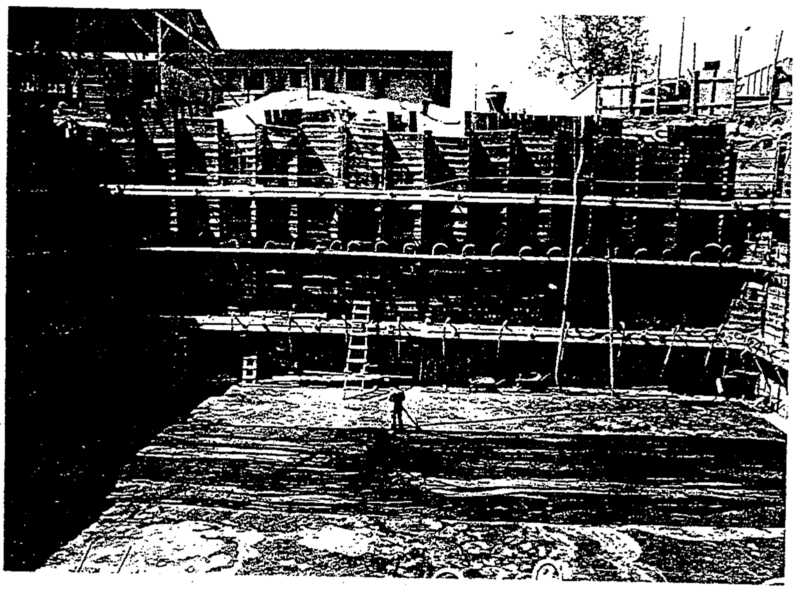
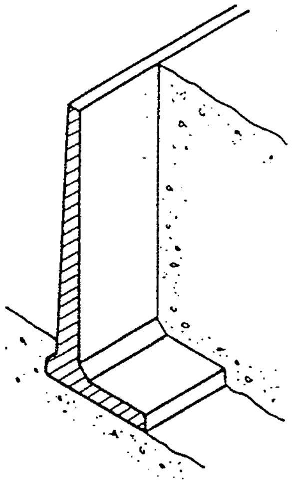
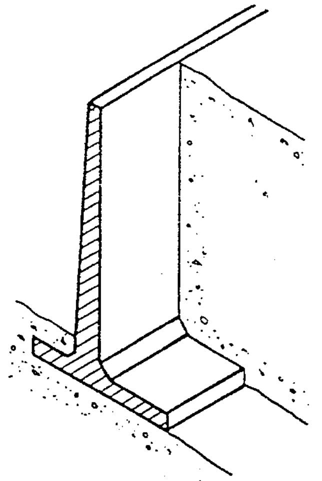


Fig. 4.8. Berlin wall method using steel soldier piles and horizontal timber laggings with wellpoint dewatering for a sewage pumphouse at Al-Khobar, Saudi Arabia (courtesy of Bauer)

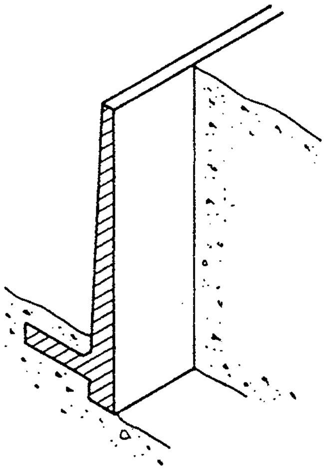
Figure 20 - Soldier Pile Wall - Berlin Method with Timber Lagging



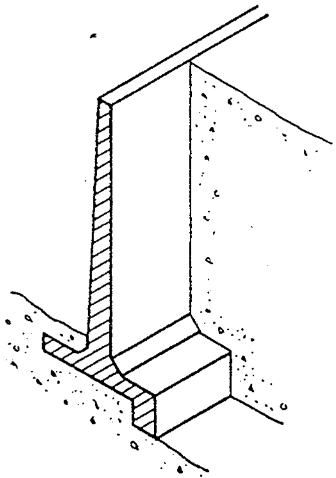
(a) L-shaped Cantilever Retaining Wall



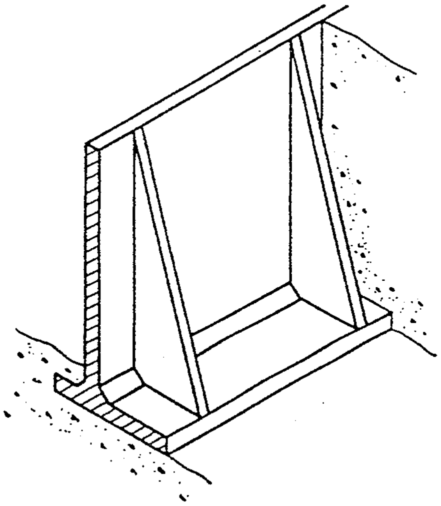
(b) Inverted T-shaped Cantilever Retaining Wall



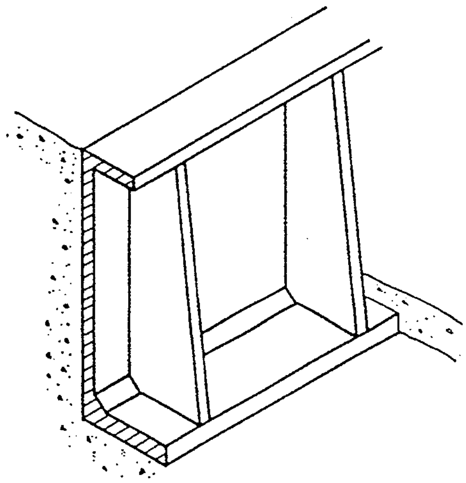
(c) Reversed L-shaped Cantilever Retaining Wall with Key



(d) Inverted T-shaped Cantilever Retaining Wall with Key



(e) Retaining Wall with Counterforts



(f) Retaining Wall with Buttresses

Figure 21 - Types of Reinforced Concrete Walls

3. Major Changes in BS 8002

3.1 Introduction

The publication of BS 8002 introduced the concept of limit state design, with greater emphasis on wall conditions at the serviceability limit state than collapse conditions at the ultimate limit state.

The basis of this method is the reduction of soil peak shear strength values on both the active and passive sides of the wall, to values which would be representative of mobilized shear strengths with the permitted wall movements. BS 8002 specified that where wall displacements are required to be less than 0.5% of the wall height, the representative undrained shear strength should be divided by a mobilization factor M not less than 1.5. For designs using effective stress values, the value of M was specified to be 1.2 to reduce horizontal wall movements to 0.5% of wall height.

3.2 Major changes from former code CP 2

(i). Effective Stress analysis is taken as the main basis of analysis of Earth pressures.
Total stress analysis may be used for some cases to analyse behaviour just after construction.

(ii). Incorporates the limit state design philosophy but it does not involve partial factors.

Code defines two limit states as;

(a). Serviceability limit state
(b). Ultimate limit state

Code is not based (cannot be based) upon ultimate state design. Design earth pressures are those that occur when the retaining wall is satisfying serviceability conditions.

In order to achieve a satisfactory service condition movement of the wall has to be relatively small. Consequently the strains in the soil mass are limited and only a fraction of the peak strength of the soil is mobilised.

(iii). Recognises the dependence of Earth pressures on wall movements.

It takes into account the difference in the strength mobilised in the backfill due to restricted movement and the peak strength as measured in a triaxial test. It also recognises that the soil shear strength may reduce to residual values when large strains occur in the soil.

(iv). Based on the above recognition it recommends the use of a "mobilization factor" to convert peak shear strengths to shear strengths mobbed in working conditions.

Deformations that can be tolerated are limited to 0.5% of the wall height in order to satisfy the serviceability conditions. The shear strength parameters mobilized at this level of permissible deformation are obtained by the use of a mobilization factor as, design $C = \dfrac{C}{M}$ design $\tan\phi = \dfrac{\tan\phi}{M}$

The mobilization factor applied as above increases the active pressure on the wall and reduces the passive resistance.

As the earth pressures determined are the most servere that can occur in service no further partial factors are needed in computing design bending moments and shear forced.

The design of the structural elements of the wall is referenced to relevant structural codes.

(v). Introduces critical state approach for soil strength.

(vi). BS 8002 mentions that its recommendations are applicable to walls with a height less than 8m. However it was stated that many of its recommendations are generally applicable.

* **<u>Note</u>**

The mobilisation factor is not a partial factor. It is applied to take into account of the limited shear strength available for use during service. It has no connection with any unfavourable deviation from the representative values. It is the function of the Engineer to arrive at representative values of shear strength parameters based on the available data.

Nevertheless it is admitted that even the ultimate limit state is checked using the earth pressures that develop under serviceability conditions. This is a conservative approach.

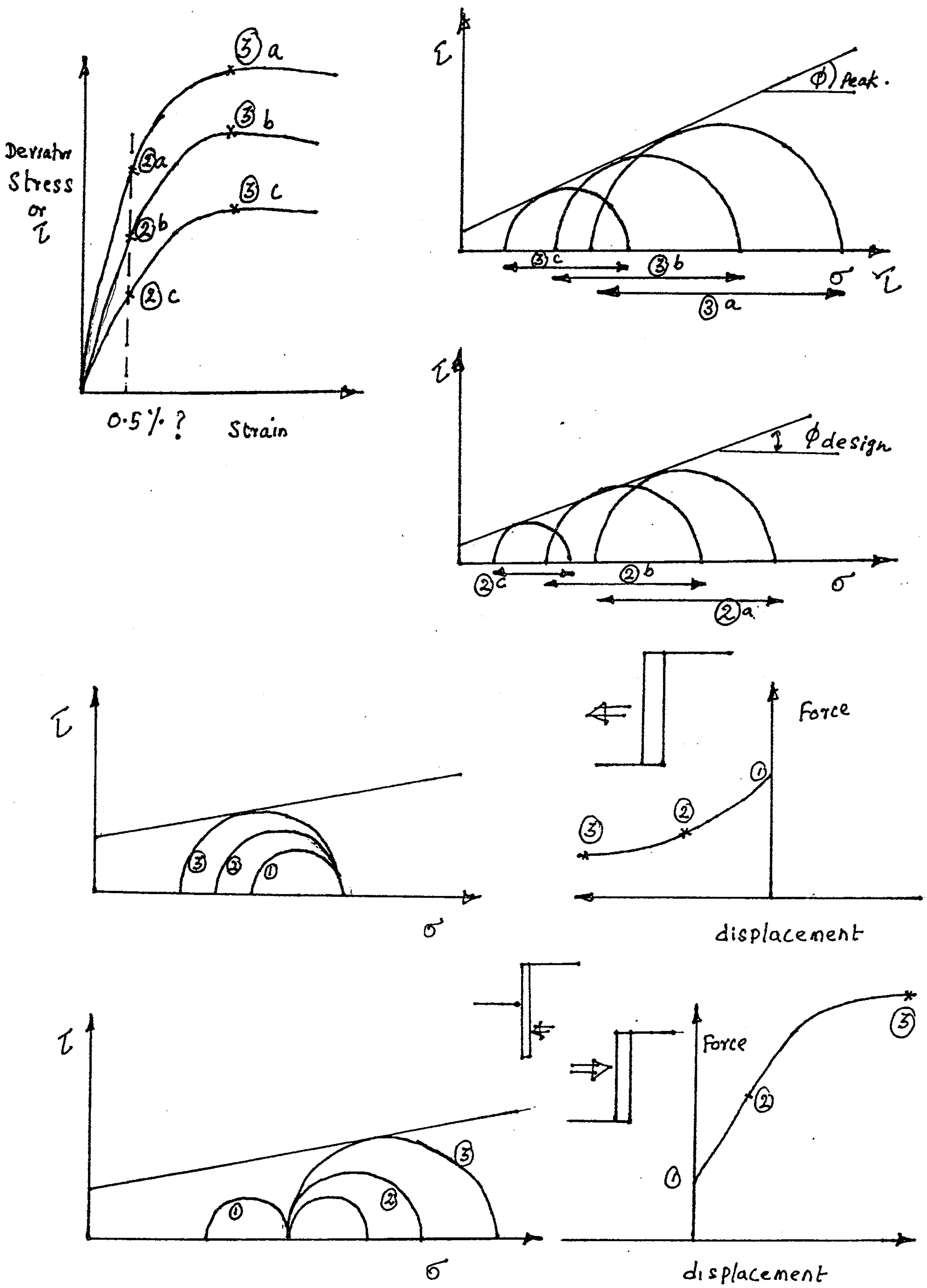


Figure 22 - Concept of Mobilization Factor

4. Comparison with other Codes for Design of Earth Retaining Structures

4.1 Guide to Retaining Wall Design - Geoguide 1 Geotechnical Engineering Office, Civil Engineering Dept. HongKong.

This is to be used in design of new permanent earth retaining walls and adopts a limit state approach. The partial safety factor approach is used on the basis that it allows "different safety factors for loading and material properties, commensurate with different reliabilities". The Hong Kong Geoguide 1 applies a partial factor of 1.0 to the unit weight of soil. This can be justified as the unit weight of a soil is one of the least variable soil parameters and the mean value can be taken as a representative value.

Geoguide 1 applies a partial factor of 1.2 to the drained shear strength of soil and a partial factor of 2.0 to the undrained shear strength, "for use in design against ultimate limit states". Appropriate partial safety factors on loading and structural material parameters are given in Table 6 and Table 7 of Geoguide 1.

- Guidelines for evaluation of geotechnical parameters are given in chapters 5, 6, 7 & 8 of Geoguide 1

- Types of walls covered by the Geoguide 1 are;
  - (a). Gravity walls - Mass concrete, Crib walls, Gabion walls
  - (b). Reinforced Concrete - RC, L shaped or T shaped Counterfort & buttressed
  - (c). Cantilevered - Bored pile walls

- Geoguide also covers the design & construction aspects of different types of walls and monitoring and maintenance.

4.2 Eurocode 7

Eurocode 7 - Final draft version of Geotechnical Design Eurocode 7, published in August 1994 also adopts a approach similar to that in Geoguide 1. The proposed design methods for retaining walls in Eurocode 7, depend on limit state methods and partial factors are applied to both characteristic loads and soil and water pressures based on characteristic soil strengths. A further partial factor is applied to thrusts, moments and shear forces for the design of structural elements.

Eurocode 7 specifies the partial factors are developed after large deformations of the retaining structure have occurred. It does not recognise that the maximum load on a retaining wall occurs during service and not at ultimate limit state.

Table 6 - Minimum Partial Load Factors for Use in Design Against Ultimate Limit States

Loading	Partial Load Factor, γ_f
Dead load due to weight of the retaining wall	1.0
Dead load due to weight of soil, rock and water	1.0
Surcharge	1.5[(1)]
Seismic load	1.0
Water pressure	1.0

Note : (1) γ_f should be set to zero for those surcharges which produce a favourable effects

Table 7 - Minimum Partial Material Factors for Use in Design Against Ultimate Limit States

Material Parameters		Partial Material Factor, γ_m
Soil :	unit weight γ	1.0
	base friction angle tan δ_b[(1)]	1.2
	drained shear strength	1.2[(2)]
	undrained shear strength S_u	2.0
	permeability k	1.0
Rock :	unit weight γ	1.0
	shear strength of joint	1.2
	compressive strength, e.g. q_c, q_u	2.0
Water :	unit weight γ_w	1.0
Granular filter and drainage materials :	permeability k	10.0
Structural materials :	unit weight γ	1.0
	strength parameters f_m	As per relevant structural code

Notes : (1) The selected value of δ_b should be based on the unfactored shear strength of the relevant soil or the soil/wall interface (see Section 5.12).

(2) For a $c' - \phi'$ (Mohr-Coulomb) strength model, the γ_m values should be applied to the selected values of shear strength parameters c' - tan ϕ'.

Table 8 - Typical Ranges of Values of Geotechnical Parameters for Selected Hong Kong Soils

Soil Type	Bulk[3] Unit Weight, γ (kN/m^3)	Dry Unit Weight, γ_d (kN/m^3)	Shear Strength Parameters		Mass Permeability k (m/s)
			ϕ'	c'(kPa)	
Compacted Fill[3]					
Completely decomposed granites	19-21	15-19	38°-42°	0 - 5	10^{-6} - 10^{-7}
Completely decomposed volcanics (tuffs & rhyolites)	18-21	15-19	35°-38°	0 - 5	10^{-6} - 10^{-8}
Crushed rock fill	18-21	18-21	45°->50°	0	>10^{-2} - 10^{-4}
Insitu Soil					
Completely decomposed granites[5] [6]	16-21	14-19	35°-44°	5 - 15	10^{-5} - 10^{-7}
Completely decomposed volcanics (tuffs & rhyolites)[5] [6]	16-21	14-19	32°-38°	5 - 10	10^{-5} - 10^{-7}
Colluvium (matrix material)[7]	15-21	13-19	26°-40°	0 - 10	10^{-4} - 10^{-7}

Notes :
(1) The ranges of values given in this Table, which are appropriate for the stress levels normally encountered in the design of retaining walls, are for general guidance only.
(2) Soils derived from the meta-sedimenatry and argillanceous rocks (see Figure 1) may have markedly different typical values of geotechnical parameters.
(3) Saturated bulk unit weights have higher values.
(4) The typical values of parameters for compacted fill are for materials which fall broadly within the grading and plasticity limits given in Table 4 and compacted to at least 95% of their maximum dry density.
(5) The typical values of parameters for completely decomposed granites and volcanics (tuffs & shyolites) corresond to tests on Class 1 samples as defined in Geoguide 2 (GCO. 1987)
(6) The lower bound value of ϕ_{cv}' for insitu completely decomposed granites and completely decomposed volcanics (tuff & rhyolites) is about 34° and 30°
(7) The relatively wide ranges of the parameters reflect the variable composition of colluvial matrix material in Hong Kong.
(8). It is not meaningful to give typical values for existing fill, which can be extremely variable.

5. Structure of the Code.

The document is presented in 4 section. ie
(1). Introduction
(2). Data for Design
(3). Design philosophy, design method and earth pressures
(4). Design of specific earth retaining structures.

Annexes
Annex - A - Graphs of Earth Pressure Coefficients
Annex - B - (Informative)
Traditional design methods for embedded walls.

6. Section - 1 - Introduction

Explains the terms used in the document and summarizes the factors influencing the choice of a retaining wall. Some important features to be noted are;

6.1 Defenition of active earth pressure

The Earth Pressure exerted on the retaining wall by the retained soil is defined as the active earth pressure. The minimum value of the active earth pressure, which occurs after sufficient movement or deflection of the retaining wall (Rankine Limit State) was defined as the "fully active earth pressure"

6.2 Defenition of Passive Earth Resistance

The earth pressure generated by the soil when it resists movement of a retaining wall is defined as the passive earth resistance. The maximum value (limiting) of earth pressure resistance which occurs after sufficient movement or deflection is defined as the "fully passive earth resistance"

It recognizes that the movement necessary for mobilisation of fully active earth pressure is usually within the serviceability limit states, but the movements necessary to develop fully passive earth resistance is generally outside the serviceability limit state.

6.3 Design Soil Strength

The soil strength which are assumed to be mobilized at the occurrence of the limit state under consideration is referred to as the design soil strength. Earth pressure computations shall be done on the computed " Design Soil Strengths" parameters. The design value of the soil strength is to be taken as the lower of either the (Peak Soil Strength/ mobilization factor) or the critical state strength.

6.4 Mobilization Factor

It is recommended to use a mobilization factor M not less than 1.5 on undrained shear strength and a mobilization factor M not less than 1.2 on effective stress shear strength parameters. This is under the assumption that the design soil strength will mobilise with a wall displacement of less than 0.5%. More detailed displacement analysis is recommended where tigher displacement criteria are to be applied and with soft or loose soils. In stiff clays subjected to cycles of strain, such as through seasonal variation of pore water pressure, the long term peak strength may deteriorate to the critical state strength.

6.5 Design value of wall friction (δ & C_ω)

The small of either the actual wall friction or adhesion measured by test or 75% of design soil strengths.

6.6 Limit State

Any state of stability beyond which the design performance requirement is not satisfied is refered to as a limit state.

Two types of limit states are defined in the code namely ;
(1). Ultimate limit state
(2). Serviceability limit state

Ultimate limit state

this refers to a state of collapse, instability or forms of failure that may endanger properly or people or cause major economic loss.

Serviceability Limit State

This refers to a state of deformation of the retaining wall such that

- Its durability as impaired
- Its maintenance requirements are substantially increased
- Non structural damage has caused
- movements have affected adjacent structures or services causing non structural damage.

6.4 Design Surcharge loads

It recommends the use of a obligatory design surcharge of 10 kN/m^2 on the retained surface. The surcharge of 10 kN/m^2 appear to be too large for low walls. There is a proposal to reduce it to 4 kN/m^2 for walls upto 2m high, 6 kN/m^2 for 3 m walls and 8 kN/m^2 for walls 4m high.

It also requires to have an allowance for extra depth of excavation of 10% of the total height retained or 0.5 m for cantilevered walls whichever in greater. For laterally supported walls it would be 10% from the level of last support or 0.5m whichever is greater.

6.5 Selection and Types of Structure

It lists the factors to be considered in the selection of a appropriate form of earth retaining structure.

The selection of a particular form of earth retaining structures will depend on;

(a). the location of the wall, its position relative to other structures and amount of space available

(b). the proposed height of the wall and the topography of the ground.

(c). the ground condition

(d). the ground water and tidal conditions

(e). the extent of ground movements acceptable during construction and in service. Effect of movement of the earth structure on existing or supported structures.

(f). external live loading

(g). the availability of materials

(h). appearence

(i). required life and main lenience

7. Section 2 - Data for Design

It recognises that the design of an earth retaining structure needs information about conditions in the vicinity of the structure. Information are required regarding nature of loading, adjacent foundations and services, ground water conditions, existence of special conditions such as geological faults, movement joints and former landslips. Representative soil parameters are to be obtained to be used in the design.

It provides guidance about the extent of site investigation required. The extent of the investigation depends upon the scale and size of the project. Major earth retaining structures require an extensive site investigation while only a preliminary investigation was required for a minor retaining structure.

A backfilled retaining wall is required to have an investigation to a depth of 2 x retained ht below the foundation level. For a embedded type wall investigation to a depth of 3 x retained height below the excavation level was recommended. If the type of the retaining structure is not known investigation to a depth of 3 x retained height is recommended.

The climatic variations such as temperature changes - ground freezing, rainfall, man made effects etc. are to be investigated. Planting of trees too close to the structure is prohibited.

7.1 Soil Properties

General recommendation is to do the design based on effective stress analysis. However under some circumstances total stress analysis is appropriate. Hence it may be necessary to determine the strength and stiffness parameters under both drained and undrained conditions.

This section provides four Tables containing Soil parameters that can be used for preliminarly design purposes. By varying the soil properties in a preliminary design the engineer can get a general idea about the properties that need to be determined accurately from his site investigations. Final design should be based on properly determined representative soil parameters.

Table 1 - provide reasonable values for unit weights of soils in the absence of reliable test results. Table 2, 3 and 4 provide guidance on the empirical relationship between classification and index tests and representative values of the angle of shearing resistance and the density of various materials.

7.2 Selection and Evaluation of Soil parameter values

It suggests that the soil test results obtained in investigation require two stages of analysis and interpretation in order to derive satisfactory design parameters forms raw geotechnical data.

Stage 1
Values of representative soil parameters are chosen from the raw insitu soil properties. A conservative approach is to be used here.

Table 1. Unit weights of soils (and similar materials)				
Material	γ_m: moist bulk weight (kN/m^3)		γ_s: saturated bulk weight (kN/m^3)	
	Loose	Dense	Loose	Dense
A - Granular				
Gravel	16.0	18.0	20.0	21.0
Well graded sand and gravel	19.0	21.0	21.5	23.0
Coarse or medium sand	16.5	18.5	20.0	21.5
Well graded sand	18.0	21.0	20.5	22.5
Fine or silty sand	17.0	19.0	20.0	21.5
Rock fill	15.0	17.5	19.5	21.0
Brick hardcore	13.0	17.5	16.5	19.0
Slag fill	12.0	15.0	18.0	20.0
Ash fill	6.5	10.0	13.0	15.0
B - Cohesive				
Peat (very variable)	12.0		12.0	
Organic clay	15.0		15.0	
Soft clay	17.0		17.0	
Firm clay	18.0		18.0	
Stiff clay	19.0		19.0	
Hard clay	20.0		20.0	
Stiff or hard glacial clay	21.0		21.0	

Table 3. φ' for siliceous sands and gravels	
A - **Angularity**[1]	*A* (degrees)
Rounded	0
Sub-angular	2
Angular	4
B - **Grading of soil**[2]	*B* (degrees)
Uniform	0
Moderate grading	2
Well graded	4
C - *N'*[3] (blows 300 mm)	*C* (degrees)
< 10	0
20	2
40	6
60	9

[1] Angularity is estimated from visual description of soil.

[2] Grading can be determined from grading curve by use of: Uniformity coefficient = D_{60}/D_{10}

where D_{10} and D_{60} are particle sizes such that in the sample, 10 % of the material is finer than D_{10} and 60 % is finer than D_{60}.

Grading	Uniformity coefficient
Uniform	< 2
Moderate grading	2 to 6
Well graded	> 6

A step-graded soil should be treated as uniform or moderately graded soil according to the grading of the finer fraction.

N' from results of standard penetration test modified where necessary by figure 2.

Intermediate vaules of *A*, *B* and *C* by interpolation.

Table 2. φ'_{crit} for clay soils

Plasticity index %	φ'_{crit} (degrees)
15	30
30	25
50	20
80	15

Table 4. φ' for rock

Stratum	φ' (degrees)
Chalk	35
Clayey marl	28
Sandy marl	33
Weak sandstone	42
Weak silstone	35
Weak mudstone	28

NOTE 1. The presence of a preferred orientation of joints, bedding or cleavage in a direction near that of a possible failure plane may require a reduction in the above values, especially if the discontinuities are filled with weaker materials.

NOTE 2. Chalk is defined here as unweathered medium to hard, rubbly to blocky chalk, grade III (see Clayton, 1990).

Stage 2
Critical evaluation of raw data with the assistance of established correlations.

It suggests that isolated low or high values should be scrutinized to determine their accuracy. Where such values are attributable to errors they should be rejected. Where values can be determined with confidence with little variation the representative value should be the mean value. Where greater variations occur or values cannot be determined with confidence then the representative value should be a cautious assessment of lower limit of the data. It recommends that the conservative bounds of the generally available parameters are to be used in the absence of detailed test information.

Matters to be considered in the selection of representative soil parameters are listed as
(a). Geological and other background information
(b). Differences between the in situ conditions and the properties measured by field or laboratory tests.
(c). The effect of construction activities on the properties of the ground.
(d). Changes which may occur in the field due to variations in the environment or weather.
(e). Relevant data from previous projects and the performance of existing facilities.

It also noted that the assessment of a parameter should depend on the mechanism or mode of deformation. For the serviceably limit state it suggest to look at the stress-strain behaviour for the determination of appropriate strength and stiffness values.

Guidance is provided on selection of strength parameters for different types of soils.

(a). Selection of parameters for clay soils

Discusses the
- Need to have C_u (undrained strength) and its dependence on the method of determination.
- Appropriate test type to be selected simulating the field condition. (Ex : triaxial extension test for passive conditions).
- Sand or silt intrusions in stiff clays would affect undrained shear strength.
- Code discusses the problems associated with tests of clays and says that the values of representative peak strength should be decided giving due allowance to the influence of sampling and testing and likely softening on excavation.
- To determine the effective stress parameter trixial tests shall be carried out under drained conditions or under undrained conditions with pore water pressure measurements. Samples are to be saturated. Test should be sufficiently slow to ensure equilibration of pressures.
- Cohesive soils with high clay content can experience significant reduction from peak to residual strength and ϕ_r to be used in association with areas of former slides.

Two approaches suggested for the linearization of the peak soil envelope (failure envelope) over the desired range of stress. (Figure 1).

- Secant value of ϕ' - (select the lowest value)
  ($c' = 0$).
- Tangent parameters c', ϕ' - (select the lowerbound value)

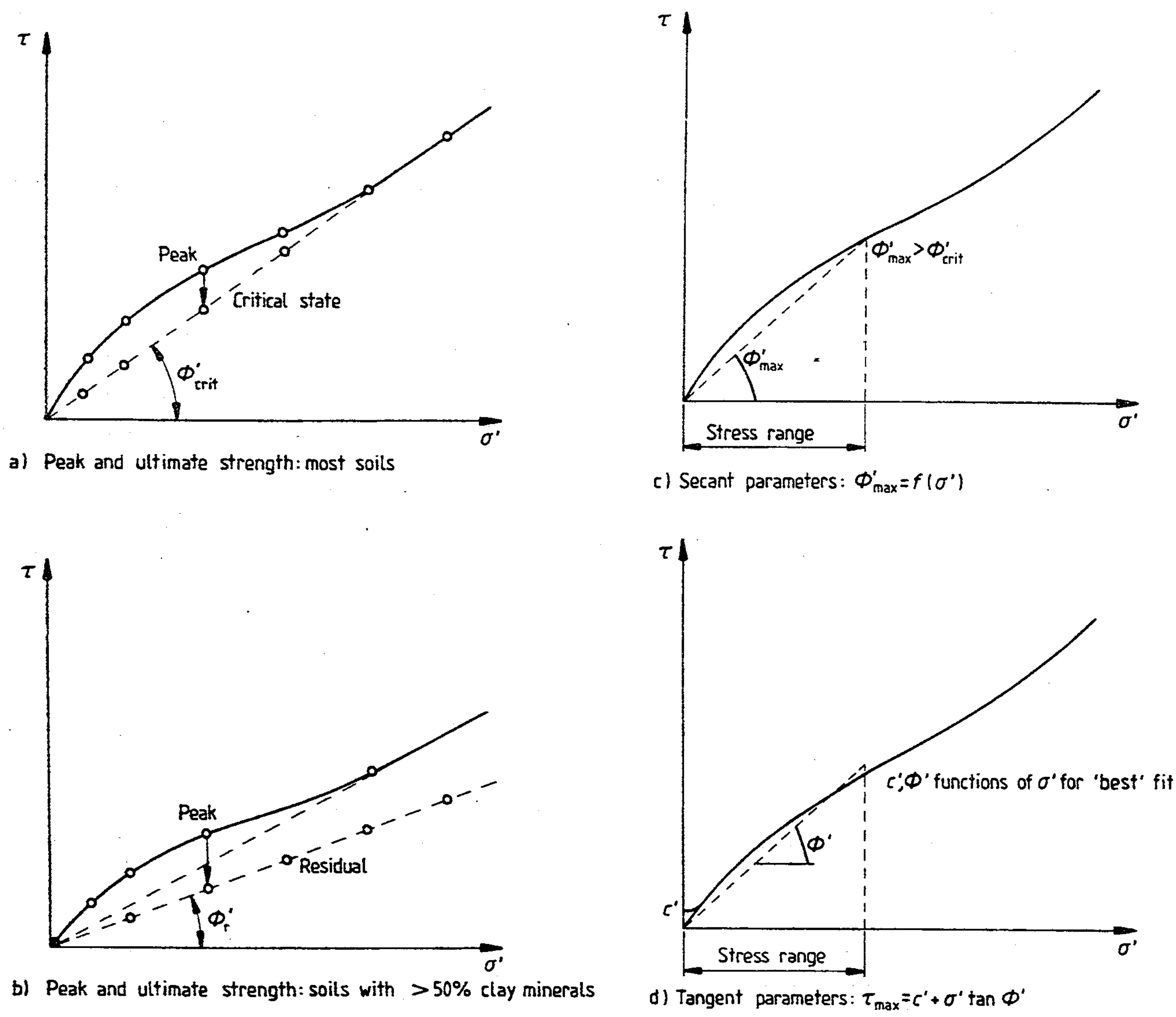


Figure 1. Strength envelopes for a given pre-consolidation

Figure 23 - Strength Envelopes for a given Preconsolidation
Figure 1 of BS 8002

(a). Selection of properties for cohesionless soils

- Strength and stiffness of cohesionless soils are determined indirectly by insitu dynamic or static penetration tests.

- Estimated Peak effective angle of shearing resistance given by $\phi' = 30 + A + B + C$

- Estimated mass critical state angle of shearing resistance given by $\phi'_{crit} = 30 + A + B$

A = Angularity of the particles
B = Grading of the sand / gravel
C = Results of standard penetration tests

Values of A, B, and C are given in Table 3.
Standard Penetration Test values are to be corrected for the effect of overburden pressure. Reference was made to (Bolton -The strength & Dilatory of sands -Geotechnique 1986 PP 65-78) paper to find ϕ'_{crit} through empirical relationships.

(b). Selection of properties of silts.

- Difficulty in obtaining samples of silts and fine sands is discussed. Liquefaction, softening & dilation of silts discussed.
- If the silt is without organic or clay mineral content it was recommended to use Table 3 in the absence of other data.

(c). Selection of Parameters for Rock

- Need to evaluate the orientation and state of the discontinuities is highlighted.
- Corrections established in BS 1377 part 9 between SPT values and strength and stiffness properties of weak rock masses are recommended. Use of compression wave and shear wave velocity measurements in obtaining rock mass properties is suggested.
- Table 4 suggests values of effective ϕ' to be used for several, closely and randomly jointed or fractured rock masses with RQD close to zero.
  (Rock is treated as to be composed of granular fragments).

(d). Fill behind Retaining walls

- Backfill behind a wall should preferably be a cohesionless granular material.
- Restrictions are placed on use of cohesive material behind the retaining walls. Clays should be with PI < 25% and LL < 45% in order to minimise problems associated with swelling and/or consolidation. Placement m.c. also to be kept close to final equilibrium values to minimise the same effects. (reference was made to some publications. LR 406, LR 522, etc for guidance on selection of fill).

(e). Wall friction, base friction, and undrained wall adhesion

- Interface strength parameters are to be obtained using a shear box test where the wall material with appropriate roughness is placed in the bottom box. Use of larger shear box is recommended for material with large scale friction.

- In the absence of experimental results, δ used in terms of effective stress shall not exceed.

  $\delta = \phi'_{crit}$ for rough surfaces.

  $\delta = 20°$ for smooth surfaces.

- No effective adhesion C′ to be taken for walls or bases in contact with the soil. If a drainage layer is placed behind the retaining wall appropriate friction parameters should be taken.

- If undrained adhesion is used in any instant it should not exceed the remoulded undrained strength of the soil.

7.3 Establishing Strength Parameters for Various Soil Types

Examples

7.3.1 Soil Type - A

Well graded granular fill, compacted in layers to at lease 92% of proctor optimum density.

With relative density variation 0 - 1.0

γ_d will vary from 1478 - 1773 kg/m^3

γ_{sat} will vary from 1922 - 2100 kg/m^3

assume saturated enough and take $\gamma = 20$ kN/m^3

Refer Table 3

Take the fill to be subangular $A = 2°$

well graded $B = 4°$

SPT of 40 $\Rightarrow$ $C = 6°$

$$\Rightarrow \phi_{crit} = 30 + A + B = 36°$$

$$\phi_{\max} = \phi_{crit} + C = 42°$$

$$\text{for design } \phi = \frac{\tan^{-1}(\tan\phi_{\max})}{1.2} = 36.9$$

but $\phi_{crit} = 36°$ Hence design for $\phi = 36$

Pressure distribution at the back of the retaining wall can be computed by

If backfill is horizontal and compaction effects are not considered.

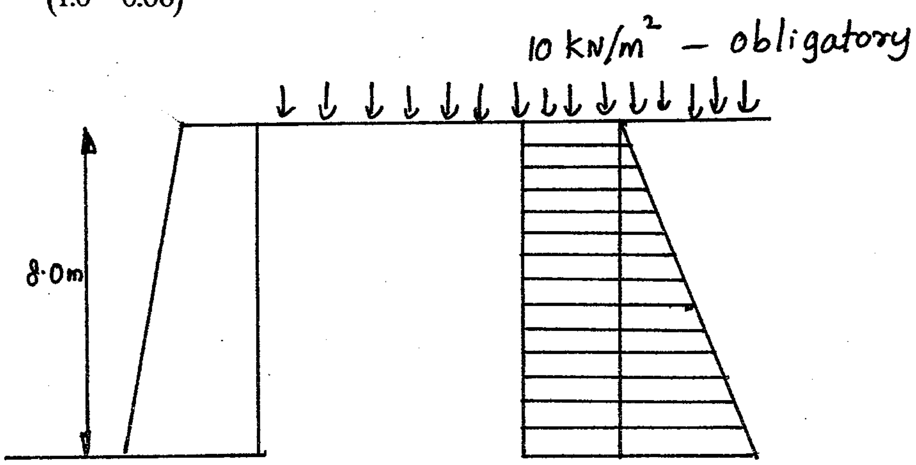
Use Figure A - 1

k_a for $\delta/\phi = 0.66 = 0.217$

$\delta/\phi = 1.0 = 0.20$

for $\delta/\phi = 0.75$

$$k_a = 0.20 + \frac{(0.217-0.20)}{(1.0-0.66)}(1.0-0.75) = 0.212$$



$$P_{AN} = \sigma_V K_A$$
$$= (\gamma z + q)k_A$$
$$= (20Z + 10)\ 0.212$$
$$= 4.24Z + 2.12$$

When Z = 0 $\quad P_{AN} = 2.12$ kN/m^2

Z = 8 $\quad P_{AN} = 36.04$ kN/m^2

$\therefore$ Design force = 1/2 (2.12 + 36.4) x 8]

= 152.64 kN/m^2

Now it is necessary to arrive at wall dimensions to withstand this force.

7.3.2 Soil Type - B

A natural fine, dune sand, with uniform rounded grains generally of medium density, but with loose medium pockets.

N corrected $\approx$ 20

take γ = 19.5 kN/m^2

A = 0 $\quad$ B = 0 $\quad$ C = 2 $\quad \Rightarrow \quad \phi_{max} = 32°$

$$\phi_{design} \leq \left(\frac{\tan\ 32}{1.2}\right) \quad \Rightarrow \quad \phi_{crit} = 30$$

= 27.5

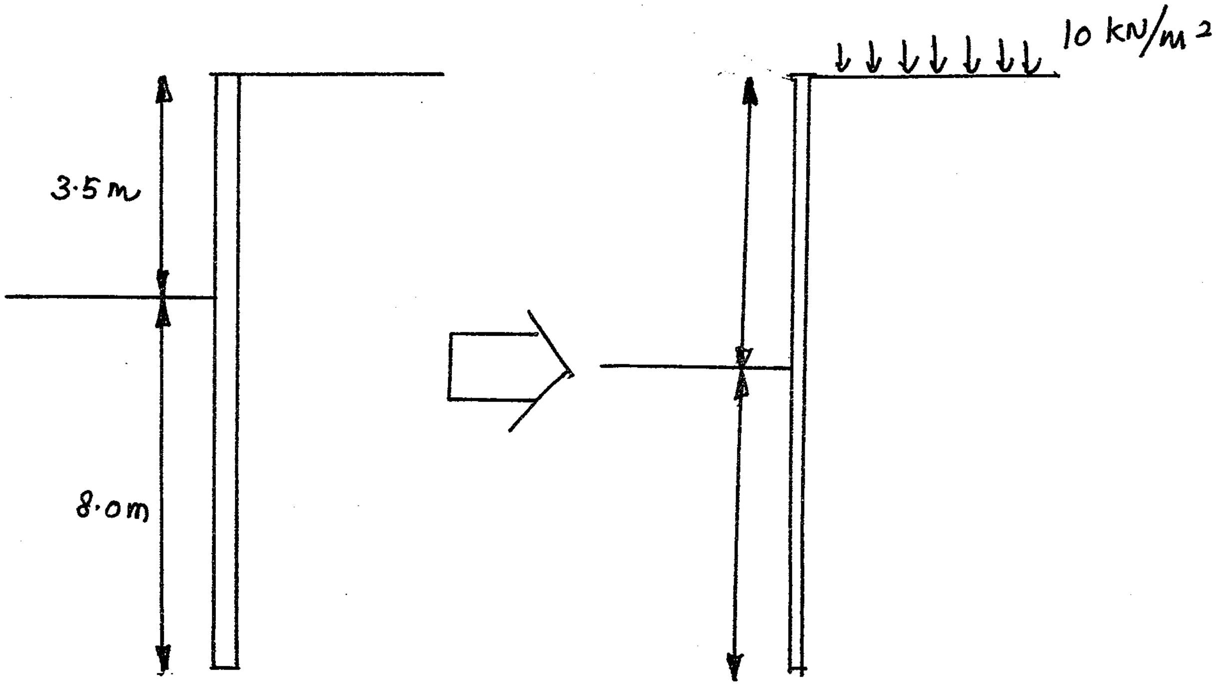
This sand is deformation - limiting with

ϕ_{design} = 27.5

The mobilization factor of 1.2 is good only for sands of medium density and cannot be guranted to control deformations in loose sands.

Assume that on cantilevered embeded wall is used to retain this soil. This is considered as an anlysis problem with a given depth of embedment of 8.0m.

- Consider the mandatory surcharge of 10 kN/m^2
- Consider the obligatory unplanned excavation of 10% of 3.5 = 0.35 or 0.5 m



Now compute the earth pressures in the active side and passive side separately.

Design ϕ 27.5 deg $\quad C \approx 0$

Assume $\delta / \phi' = 0.66$ use Figure A.1 for active pressure coefficent

$k_A = 0.315$

Use Figure A -2 for passive pressure coefficients $k_p = 4.2$

active pressure $\sigma_{an} = k_A(\gamma z + q)$

$\sigma_{an} = 0.315\ (19.5\,Z + 10)$

$= 6.14\,Z + 3.15$

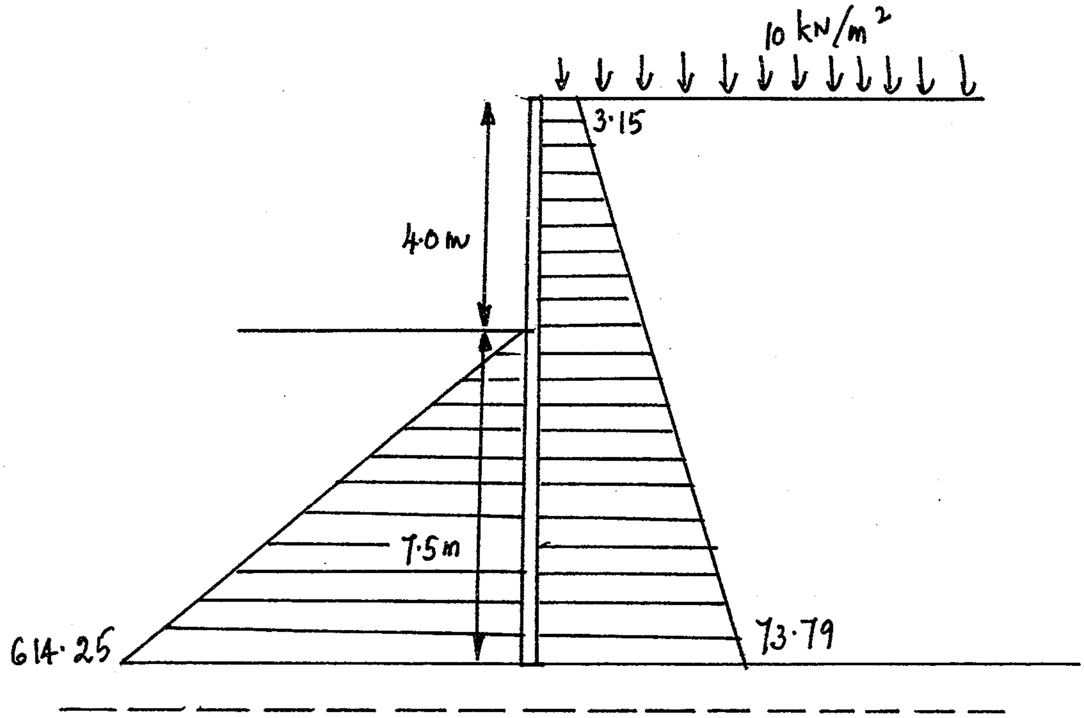
When $Z = 0$ $\qquad \sigma_{an} = 3.15$ kN/m^2

$Z = 4 + 7.5 = 11.5$ m $\qquad \sigma_{an} = 73.79$ kN/m^2

Passive pressure $\sigma_{pn} = k_p(\gamma z + q)$

$\sigma_{pn} = 4.2\ (19.5\,Z) = 81.9\,Z$

$Z = 7.5$ $\qquad \sigma_{pn} = 614.25$ kN/m^2



<u>If the water table is present at the excavation level</u>
<u>Active Side.</u>

upto the water table level

$$\sigma_{an} = 6.14\,Z + 3.15$$

when Z = 4.0 $\qquad \sigma_{an} = 27.71$ kN/m^2

below water table

$$\sigma_{an} = k_A[\gamma z - \gamma_w z + q] + \gamma_w z = k_A[(\gamma - \gamma_w)z + q] + \gamma_w z$$
$$= 0.315\,[(19.5 - 9.81)\,Z + 4 \times 19.5 + 10] + 9.81\,Z$$
$$= 12.86\,Z + 27.72 \text{ kN/m}^2$$

when Z = 0 $\qquad \sigma_{an} = 27.72$ kN/m^2

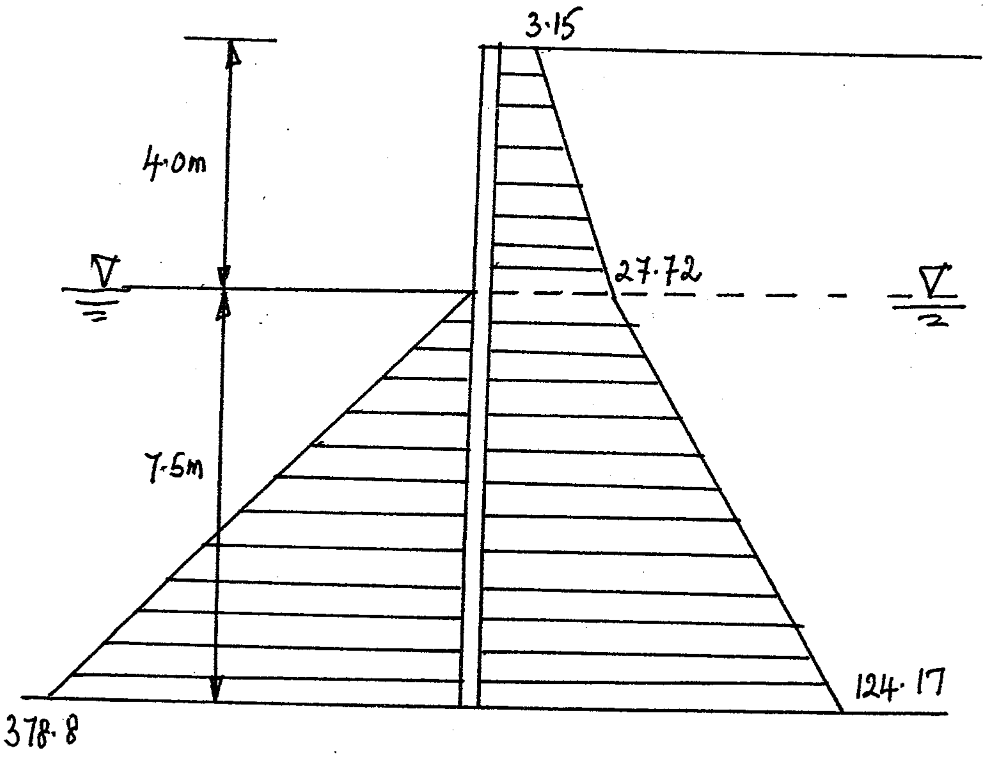
Z = 7.5 $\qquad \sigma_{an} = 124.17$ kN/m^2

<u>Passive Side</u>

$$\sigma_{pan} = (\sigma_v - u)\;k_p + u$$
$$= [\gamma z - \gamma_w z + q]k_p + u$$
$$= [(19.5 - 9.81)Z + 0\,]\,4.2 + 9.81\,Z$$
$$= 50.51\,Z$$

at Z = 0 $\qquad \sigma_{pn} = 0$

Z = 7.5 $\qquad \sigma_{pn} = 378.8$ kN/m^2



7.3.3 Soil Type - C

A natural firm to stiff, glacial clay with frequent silty and sandy laminations.

Natural water content = 15% (close to plastic limit)
Liquid Limit = 30% $\therefore$ PI = 15%
Table 2 $\rightarrow$ ϕ_{crit} 30°

A very slow direct shear test of a submerged sample is a convenient wat to obtain ϕ_{crit} in the laboratory.

More common approach would be to conduct consolidated, undrained triaxial tests in the laboratory.

This would give $\phi_{\varepsilon=0.5\%} \geq 30^\circ$ by drawing Mohr Circles of effective stress at the servicibility limiting strain.

So $\phi = 30^\circ$ could be used in the design

If $\phi = 30^\circ$ from tests ϕ design $= \tan^{-1}\left(\dfrac{\tan\ 30}{1.2}\right)$

$= 25.7$

- Undrained computations are unsafe fro retaining walls since drainage can occur.

8. Section 3 - Design Philasophy, design method and Earth Pressures

8.1 Design Philasophy

Design of the Earth Retaining Structure requires two sets of computations

- Set of Equilibrium Calculations to establish the overall proportions of the structure under relevant earth pressures.
- Set of computation to determine the size and proportions so as to resist the bending moments and shear forces developed.

Limit states of

(a). Ultimate limit state and

(b). Servicibility limit state are defined.

Ultimate limit states are illustrated in Figure 3 of BS 8002 (Figure 24).

They include

- Instability of structure, or foundations. Considered as a free body
- Failure by rapture or failure of any part.

8.2 Ultimate Limit States

Following specific ultimate limit states are to be considered.

(1).

(a). Overturning or rotational failure
Disturbing moment > Restoring moment

(b). Translational failure.
Disturbing forces > Restoring forces

(c). Bearing failure.

(d). Instability of the soil mass involving a slip failure.

(2). Failure of a structural member.

(3). Excessive deformation causing ultimate failure of adjacent structures.

Methods of Analysis.

(c) → Guided by BS 8004

(d) → Slope Stability Analysis BS 6031, BS 8081

(a) & (b) → Use the mobilized soil strengths

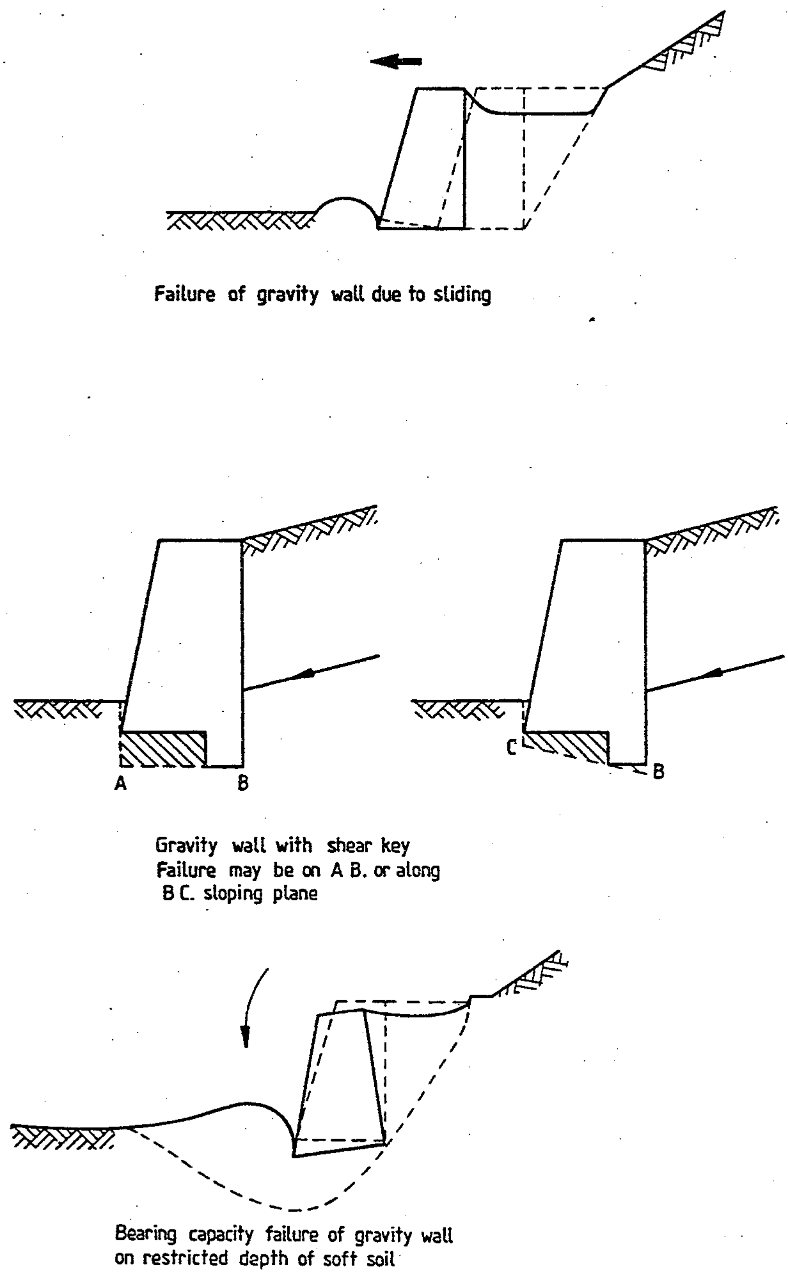


Figure 3. Limit states for earth retaining structures (*continued*)

Figure 24 - Ultimate Limit States *(Figure 3 of BS 8002)*

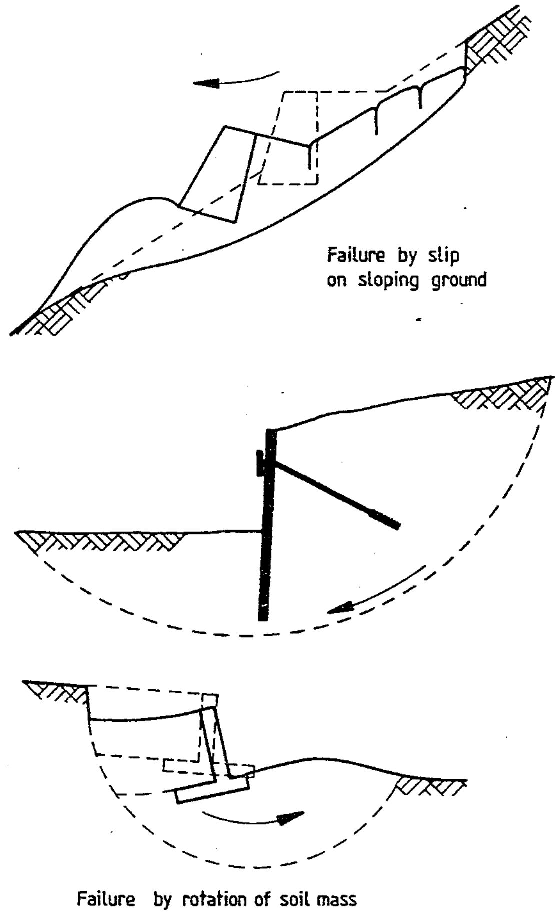


Figure 3. Limit states for earth retaining structures

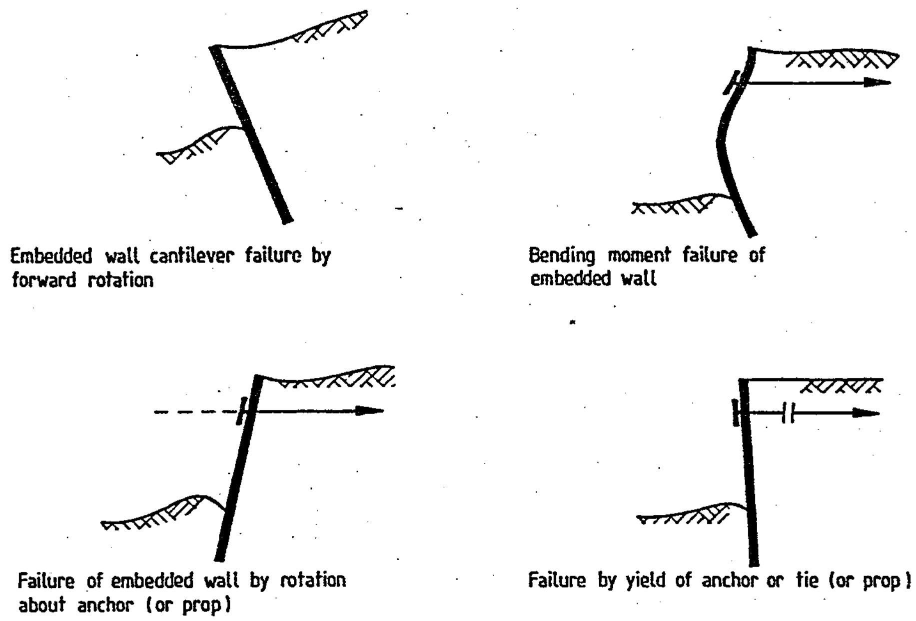


Figure 3. Limit states for earth retaining structures *(concluded)*

Figure 24 - Ultimate Limit States *(Figure 3 of BS 8002)*

8.3 Serviceability Limit States

Limit states to be considered are ;

- Substantial deformation of the structure
- Substantial movement of the ground

It is necessary to limit deformations to satisfy service requirements. As it is difficult to compute the displacements this is ensured by limiting mobilized strength.

Under the service conditions pressure on the active side will be greater than " fully active pressures" and the pressure on the passive side will be smaller than " fully passive" pressures. Design earth pressures will correspond to this service condition and it would be the most severe condition.

8.4 Applied Loads

Code requires the correct estimate of the external surcharge loads and computation of the earth pressure that will not be exceeded in the respective limit state. Code does not demand the application of a load factor.

8.5 Design Earth Pressure

When movements of the wall are small mobilized shear strength parameters are low and active pressures would be greater than fully active. When significant movements have taken place and if strength parameters have reduced to critical state values then the earth pressures would have again increased.

Therefore by adopting a design strength parameters as the lower value of $\frac{\text{Representative Value.}}{M}$ or critical state value the most unfavourable design earth pressure can be obtained.

Compaction induced earth pressures, earth pressures that can develop due to swelling, earth pressures that can develop due to various deformation modes (for Ex : Consolidation of foundation material due to weight of fill) should be considered.

8.6 Design Method

Equilibrium Calculations.

Consider different design situations applicable to the structure and draw free body diagrams.

Carry out separate calculations for different design situations. Design situations should cover all the uncertainties and risks involved. Calculations are for Global Equilibrium of the structure - (ie Force and moment equilibrium)

The minimum depth of unplanned excavation of 0.5m (or 10% height whichever is greater) in embedded walls and the minimum obligatory surcharge of 10 kN/m^2 in the retained side should be incorporated in these design stages.

Computations are to be done in terms of effective stress parameters, but when the undrained (short term) behaviour is critical an undrained analysis would also be required.

Design using Total Strength Parameters

The design C_U value should be $\frac{\text{Representative Value}}{M}$, where the mobilization factor M should be not less than 1.5.

This is with the assumption that wall movements in service would be less than 0.5% wall. For soils that require larger movements for shear strength mobilization M would be larger.

Design using effective stress parameters

The lesser of

(a). $\text{Design } \tan \phi' = \dfrac{\text{Representative } \tan \phi'_{max}}{M}$

$\text{Design } C' = \dfrac{\text{Representative } C'}{M}$ and / or

(b). Representative critical state strength of the soil.

This is also with the assumption that displacements in service will have to be limited to 0.5% of wall height. A more detailed analysis of displacements should be performed when tighter criteria are to be applied for soft or loose soils.

Design values of wall friction, base friction and undrained wall adhesion

The design values of δ, and Cω should be the lesser of

(a). the representative values determined by appropriate tests reduced by the mobilization factor.

or

(b). 75% of the shear strength mobilized in the soil

$$\therefore \text{ design } \tan \delta = 0.75 \times \text{design } \tan \phi'$$
$$\text{design } C_w = 0.75 \times \text{design } C_U$$

Since for the soil mass ;

$$\text{design } \tan \phi' = \frac{\text{representative } \tan \phi'}{1.2}$$

It will yield to give

$$\frac{\text{design } \delta}{\text{representative } \phi'} = {}^2/_3$$

Similarly in the total stress analysis

$$\frac{\text{design } C_w}{\text{representative } C_U} = 0.5 \text{ after taking } M = 1.5$$

When soil subsides in the active side or heaves up in the passive side most likely the wall will also move with one or other soil zone. (ie the relative movement between wall and soil is reduced)

The 25% reduction in the design shear strength in makes an allowance for this.

Design to structural codes

Earth pressures computed under service conditions will give the most severe condition on the wall. Hence it is not necessary to use any load factors on the Bending Moments or shear forces computed from them. These design BM & SF values can be directly used for the "Ultimate structural design" of the wall.

8.7 Computation of Disturbing force

8.7.1 Cohesionless soils

* If Rankine conditions are valid use the design ϕ' in the Rankine equation.
* for cases of uniform cohesionless soil, with uniform surcharge and no seepage active pressure at depth z is given by

$$\sigma_{an} = k_a (\gamma z + q)$$

k_a - based on design ϕ' (kerisel and Absi charts can be used) - (provided in the code)
- Wall back should be vertical

the total active force is given by

$$P_{an} = k_a \gamma \frac{Z^2}{2} + k_a qz$$

If there is static water table at a depth Z_w from surface.

$$\sigma_{an} = k_a \sigma_v' + u$$

$$u = \gamma_w (Z - Z_w)$$

$$P_{an} = \int_0^z (k_a \sigma_v' + u)\, dz$$

* When ground surface is irregular and seepage occurs, graphical methods such as wedge method can be used. It suggest to take a minimum of three trial wedges (less than five).

* Point of application of the load is taken as the point of intersection with the back of the wall of a line drawn through the centre of the gravity of the wedge parallel to the slip plane of the wedge.

8.7.2 Cohesive Soils

* If tangent c' ϕ' parameters are used in a clay it may be that $C' \neq 0$. Rankine Eq'n can be used if applicable.

* For uniform conditions, uniform surchag, no seepage forces and vertical wall back.

Compute

$$P_{an} = \int_{0}^{z} (\sigma'_{an} + u)\, dz$$

where $\sigma'_{an} = k_a \sigma'_v - 2C' \sqrt{k_A}$ with k_A from Kerisel & Absi charts. (Appendix - A)

* If short term behaviour is found to be critical and a total stress analysis is performed.

$$P_{an} = \int_{0}^{z} (\sigma_v - k_{ac}\, C_u)\, dz$$

Where

$$k_{ac} = 2 \sqrt{1 + \frac{C_\omega}{C_u}}$$

$(\sigma_v - k_{ac}\, C_u)$ Should not be used as negative.

* When the ground surface is irregular and wall back is not vertical graphical methods are recommended.

Code suggest to treat the cohesive soil as either normally or lightly overconsolidated or overconsolidated clay

Normally or lightly overconsolidated clay.

- Except for excavation in front of wall in embedded walls, short term condition is critical. Use of C_U is recommended.
- They are sensitive to disturbance. If there is a possibility of disturbance used C_U appropriate for remoulded clay at natural moisture content.
- When there is a possible of tension crack getting filled with water check the design with effective stress parameters and $C' = 0$.

Overconsolidated clay

- Clay will dilate and develop negative pore water pressure during shearing. This will cause drawing of water and clay will swell and soften.
- For short term when the consequences of failure is not severe use C_U. Influence of water filled tension crack should be considered.
- When the overconsolidated clay is fissured or weathered permeability would be high and negative pore water pressure will come to equilibrium soon. Hence an effective stress analysis is recommended.

8.7.3 Compaction Induced Earth Pressures.

Guidance to be taken from Broms (1971), Ingold (1979), Symons and Marray (1988), Clayton & Symons (1992) and TRRL Publications LR 766, LR 946, and RR 192.

8.7.4 Weak Rocks

- Discontinuity patterns to be identified. Analysis to be done in terms of effective stress because of the relatively high permeability.
- For minor structures treat weak rock as being composed of interlocking granular fragments. With an effective angle of friction. The angle of friction depends upon the inter-fragment friction and upon the particle size of grains and mineralogy. Table 4 provides guidance.

8.7.5 Surcharge Loads

A minimum obtigatory surcharge of 10 kN/m^2 is to be used. Additional loading may have to be used to account for loading due to construction plant, stacking of material etc. Elastic stress distributions to be used in dealing with line loads and point loads looked in horizontal stresses that remains even after removal of vertical loads to be considered.

8.7.6 Dynamic Loads

Dynamic loads due to earthquakes, traffic and machinary vibrations are to be considered. Assess the vulnerability of the structure to dynamic loading. Frequencies of force and structure to be compared. Undesirable to have closely matching frequencies for linking parts of the retaining structure system.

8.7.7 Loading from climatic changes

Following aspects should be considered

- Thermal expansion or contraction
- Ground freezing
- Swelling of clays
- External effects such as boiler houses or cold stores

8.7.8 Water Pressure

- Natural variations in water table to be considered.
- Provision of effective drainage highlighted and types of drains discussed.
- Influence of seepage pattern to be considered
- (flow around the retaining structure and the pore water pressures developed).
- Assuming a linear dissipation of pore water pressure around a sheet pile wall pore water pressure distribution was expressed in a simple manner. (Figure 26 & 27).

8.7.9 Water Pressure in tension crack

- Tension crack may form behind the retaining wall (may even extend over the full retained height)
- The crack can become filled with water so that the soil may have to be designed to withstand full water pressure from the surface to the base of the crack. The pressure on the wall to be taken hydrostatic down to the level where total soil pressure exceeds the possible water pressure.

When tension crack occurs design should be checked for these conditions

- For end of construction stage, use C_U with tension crack fully or partially filled with water to a level higher than the equilibrium water level.
- In hard clays or weak rock final equilibrium conditions C', ϕ', with tension crack fully or partically filled with water.

8.8 Resistance to movement

Resistance is provided by

- Mobilized passive soil pressure in an embedded wall.
- Combination of base resistance and passive soil pressure on a gravity structure.
- Struts, walings and ground anchorage.

(a). Passive Earth Resistance

- Design soil strength to be used
- Obligatory "unplanned" excavation to be used.

8.8.1 Cohesionless Soils

* Uniform cohesionless soils, vertical wall uniformly distributed surcharge with no seepage can be handled by simple equation.

$$\sigma_{pn} = k_p \sigma_v = k_p (\gamma z + q)$$

* Total passive force given by

$$P_{pn} = k_p\gamma \frac{z^2}{2} + k_p qz$$

If there is static ground water beneath a water table at depth Z_W, then for $Z < Z_W$.

$$\sigma_{pn} = k_p (\sigma_v - u) + u = k_p \sigma'_v + u$$

where $u = \gamma_w (z - z_w)$

$$P_{pn} = \int_0^z (k_p\sigma'_v + u)\, dz$$

* Simple conditions of $\delta = 0$, $\beta = 0$ can be handled by Rankine Equation.

8.8.2 Clay Soils

* For long term if secant ϕ values are used $C' = 0$ can be treated as a cohesionless soil.

* If non zero C', ϕ' exists

$$P_{pn} = \int_{o}^{z} (\sigma^*_{pn} + u)\, dz$$

where $$\sigma^*_{pn} = k_p \sigma'_v + 2C'\sqrt{k_P}$$

values of k_p given in Annex - A

* Under undrained conditions $\phi_u = 0$

$$P_{an} = \int_{o}^{z} (\sigma_v + k_{pc} C_u)\, dz$$

where

$$k_{pc} = 2\sqrt{1 + \frac{C_\omega}{C_u}}$$

* **N.C and lightly overconsolidated soils**

- Immediately after construction is most critical.
- Use a total stress analysis with C_U
- If strength of the clay reduces with time use C', ϕ' parameters.
- If excessive deformations are likely remoulded strength to be used.

Over consolidated clay

Existence of non uniform strain will result varying soil strength values.

Make allowance for this by two approximate methods of analysis.

Case - 1
Where there is no change in ground level for a long period. Equilibrium state of stress is not significantly disturbed. An effective stress analysis will take account of loss of strength created by dissipation of negative pore water pressure. In selecting C' loss of strength should be kept in mind. For short term temporary work undrained parameters may be used. For upper layers assume that C_U reduces to zero at surface.

Case - 2
Where excavation lower the ground level in front, loss of strength occurs due to the release of overburden pressure. Effective stress analysis is recommended. In selecting C' any loss of strength should be remembered.

8.8.3 Weak Rock

Pattern of discontinuities, eg. joints, fissures, present in the rock mass are important. The comments and recommendations retaining to active pressures apply.

8.8.4 Layered Soil

To be handled in the same manner as in active case.

8.8.5 Water Pressure and Seepage Forces

As in active case most adverse groundwater conditions that can be reasonably anticipated should be used. The influence of upward seepage can reduce the effective overburden pressure to zero under extreme conditions.

9. Section 4 - Design of Specific Earth Retaining Structures.

The equilibrium of the wall should be determined for the various possible failure modes.

The design strength and servicibility limits of structural materials should be as recommended in the appropriate structural code of practice.

Structural bending moments, shear forces and prop forces should be derived from the equilibrium calculations using design earth pressures & water pressures.

9.1 Gravity Walls

Gravity walls should be designed to perform adequately at both serviceability and ultimate limit states in equilibrium with design loading and design shear strength.

Design of Foundations

(a). Bearing capacity design

Foundation should be designed for bearing capacity at ultimate limit state. Use design C and ϕ. Allow for inclination and eccentricity of forces. Use N_c, N_q, N_r bearing capacity factor (Terzaghi and Peak 1967).

(b). Servicibility limit state

Deformations - ie lateral displacement sand rotations are to limited to ensure servicibility.
For medium dense or firm soil this can be ensured by satisfying bearing capacity criterion.
For C_u a mobilisation factor greater than 1.5 (around 2.0 or 3) will be required. The pressure under the toe should be checked for allowable bearing capacity.

(c). Base resistance to sliding

Base resistance to sliding can be expressed in terms of total stress or effective stress.

Total stress	-	$\tau = C_b$	——— ①
Effective Stress	-	$\sigma' \tan \delta_b$	——— ②

- All soils should be evaluated using Equation ②
- Any uplift pressures due to seepage be considered in evaluation σ'
- If the foundation soil is clayey and consolidation is slow an undrained analysis from ① is required.
- If former land slides are suspected residual strength should be used
- In weak rock orientation of discontinuties and laminations are to be considered.
- If the base resistance to sliding is inadequate then this should be increased by either
  - widening the base
  - inclining the foundation
  - or - providing a shear key

- If shear keys are provided then Equation of the combined retaining wall/soil mass above plane BCDE should be checked. (Figure 25) (Figure 11 of BS 8002).

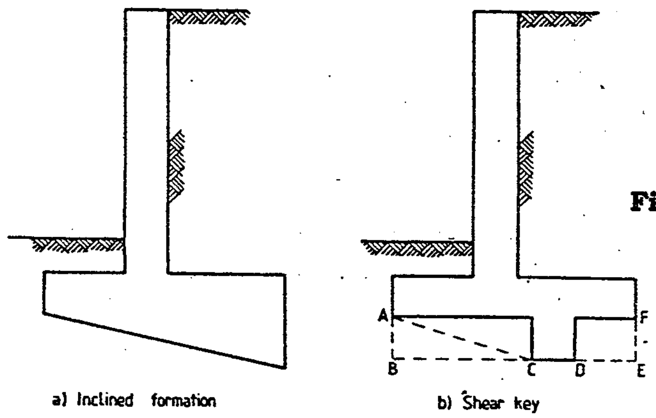


Figure 25 - Foundation s of Gravity Walls

Different types of gravity retaining walls
Different types of gravity retaining walls discussed are
- Mass concrete retaining walls
- Unreinforced masonry retaining walls
- Reinforced soil
- Gabions
- Crib walls

Their special features to be considered, need to comply with different material codes, different types or methods of construction special advantages and limitations are discussed.

9.1.1 Mass Concrete Retaining Walls

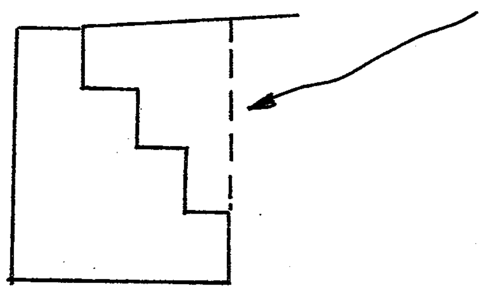
General
- Suitable for retained heights upto 3m, at greater heights other types become economical. Stepped construction in back or front and inclined or battered walls will be economical.
- Batter of about 1 : 50 in front could be useful to avoid the illusion of tilting forward.

Battered back against the backfill wil give a more uniform compresive stress over any section. But need to make sure that the wall back is stable during the construction.

Materials
- Concrete to be in accordance with BS 5328 and BS 8110 and aggregates to conform with BS 882, BS 1047.

Design recommendations

- Wall with a stepped back to be designed with a "virtual back".



- Mass concrete walls are normally designed on a no - tension basis under the design earth pressures.

However if concrete of grade 15 or more is used and construction joints are prepared according to BS 8110, BS 5400. Permissible tensile stress of 0.28 N/mm^2 and a shear stresses of 0.55N/mm^2 may be used.

- Movement joints (for expansion & construction) are to be kept with 10 - 20 mm thickness and sealed with a sealing compound
- Surface finish, masonry cladding to face, etc are discussed
- Recommended to provide joints where founding soil changes.
- Construction joints to concede with expansion/contraction joints. A longitudinal groove to be formed to generate shearing resistance.

Concrete

- Concrete of low strength such as grade C7.5 & C 10 of BS 5328 are satisfactory. Use low water/cement ratios and compact to necessary density. In aggressive environments (sulphate attaches etc.) provide additional protection.
- If masonry facing is to be incorporated need to comply with BS 5628 or BS 5390.

9.1.2 Unreinforced Masonry walls

- Suitable for small heights. For heights upto about 1.5 m can be of uniform width. Stepped or buttressed walls are suitable for greater heights.

- **Material**
- masonry units to confirm to the appropriate British standards.
- **Design**
- Structural design of the walls to be done according to BS 5628. Movement joints are to incorporated as guided by BS 5628.
- General construction to be in accordance with BS 8628 part 3 & bs 5390.
- Water proofing membrane to be provided to the retained face of the wall.

9.1.3 Reinforced Soil

For externally stability criterion reinforced soil is considered as a gravity retaining structure. It is recommended to be guided by BS 8006- code for Reinforced Earth Design.

9.1.4 Gabions

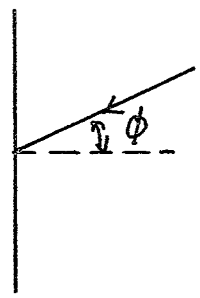
- General introduction to gabion walls is provided. Various forms of gabion walls made of steel wire, bamboo slats, nylon or polypropylene mentioned.
- advantages due to permeability of the structure
- need to use filters behind the wall
- economy in using a cellular firm when wall is wide.

- **Material**
- Unprotected gabions are used only for temporary work. But if the wire diameter is 5mm or more it can have a reasonable life time.

- Hexagonal woven mesh gabions are to be made from wire galvanized to BS 443.
- In PVC coated wire, coating should confirm to BS 4102. Thickness of the coating should be a minimum of 0.25 mm. The PVC should be sufficiently bonded to the galvanized wire core to prevent a capillary flow of water between the wire and the PVC coating.
- possibly of PVC coating getting damaged due to abrasion discussed. Special core required in coastal structures due to abrasive materials hitting the structure due to wave action.
- Edges of the end panels and diaphragm panels are to be fixed by 2.2 mm binding wire galvanized or PVC coated.
- Box sizes available are outlined.

lengths	2.0, 2.5, 3.0, ... 6.0 m
width	1 to 2 m
depth	0.3, 0.5, & 1m

- Stones used for filling should confirm to BS 5390 for hardness crushing strength and resistance to weathering. Lower limit of size is governed by mesh size. Max'm recommended size is 200mm.
- Density of the stone fill should be at least 60% of solid material. (i.e Porosity less than 0.4)



- The angle of the resultant force on the back of the wall can be ϕ' from the normal due to the high wall roughness.

  When the wall is with a stepped backface virtual wall back to be assumed.
- In the computation of sliding resistance ϕ of the founding soil should be used.

Note : (not the δ_b value $\phi > \delta_b$)

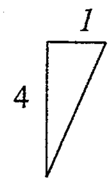
- Checks should be made at selected levels above the base to ascertain that there is sufficient sliding resistance ignoring the effect of the wire mesh

- **Construction**
- Empty cages may be placed singly or joined together in groups.
- Underwater gabions are prefilled and placed by crane.
- Horizontal internal bracing wires should be fitted between the outer and inner faces at 330 mm centres in gabions deeper than 500 mm.
- The vertical joints between individual units should be staggered in adjacent courses.
- Curves and angles in the face of the structure may be formed by cutting and folding the wire mesh to form specially shaped units.
- Stones in the basket should be tightly packed to minimise subsequent movements due to wave action in coastal applications.

9.1.5 Cribwork

- General description of crib walls are made.
  Two types of cribwalls namely timber crib walls and precast concrete cribwalls.
- Economy in cribwalls achieved by open-faced walling

  They are usually made with a batter not steeper than 4 vertical to 1 horizontal.
  Low walls may be made vertical



- Timber Cribs may be built using log or sawn timbers. Durability .and cost are to be considered. Timber used should be treated chemically.
- Concrete Cribs. Precast concrete crib units. The infill is built as each course is assembled.

- Face stretches should be positively anchored by interlocking headers for the full thickness of the structure.

- **Materials**
- Should comply with relevant codes.
  Timber - BS 5268 Reinforce concrete - BS 8110, BS 5400
- Filling - to be durable, inert and free draining.
- Preferable to keep the roadway a distance of either 4.5m or the wall height.

Detailing the Crib
- Headers to be designed as beams carrying a fill load
- Stretches to be designed for bending due to horizontal component of earth pressure.
- The connection between the headers and stretchers should be designed to resist the reactions from interlocking.
- weepholes are to be provided if fill is not free draining.

Construction
- If ground bearing pressure is adequate crib wall may be erected without a concrete foundation.
- Crib should be filled to the top of the erected wall with fill being compacted to prevent development of voids.

9.2 Design of Embedded Retaining Walls

Restriction in space, necessity to work with minimum disturbance in urban areas, difficult foundation conditions etc. impose restrictions on the use of gravity type retaining walls. Sheet pile walls (made of steel, timber or reinforced concrete) concrete diaphragm walls and concrete bored pile walls are the options available in these circumstances.

9.2.1 Special Consideration in Embedded Walls

Pore Water Pressure Computations

Embedded retaining walls are often used as waterfront structures and are subjected to an out-of-balance water pressure due to the water level difference on two sides of the wall.

Correct analysis involves sketching of a flow net or using a finite element or finite difference type analysis to obtain the pore water pressure distribution on either side of the wall. If a effective stress analysis is carried out pore water pressures on either side of the wall are required. Seepage around the wall would lead to lower than hydrostatic pressures in the active side and higher than hydrostatic pressure is the passive side. If the water condition is static the net distribution would be as shown in Figure (1) (a). Static condition is achieved by extending the retaining wall to a impermeable stratum.

CIRIA Report 104 (1984) and BS 8002 (1994) suggests that rather than drawing the flow net for each case, the pore water pressure at any point can be simply calculated by assuming that the head difference (h + i - j) is dissipated uniformly along the flow path of length (2d+h-i-j) which runs down the back of the wall and up the front - Figure 2 (a). It is illustrated by the Figure 2 (b) that the pore water pressure calculated on the basis of this simplification is less than hydrostatic behind the wall and greater in front.

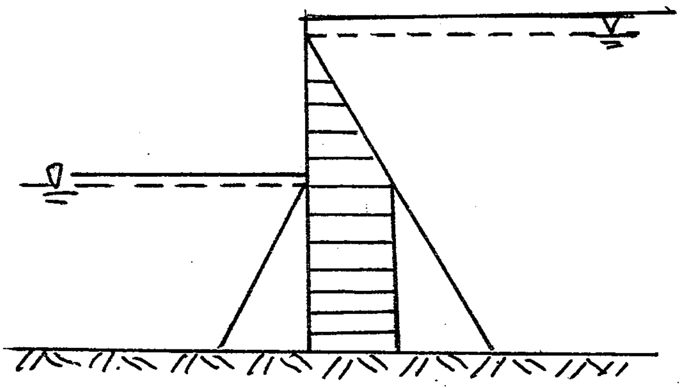


Figure 26 - Embedded Wall taken to an Impermeable Stratum

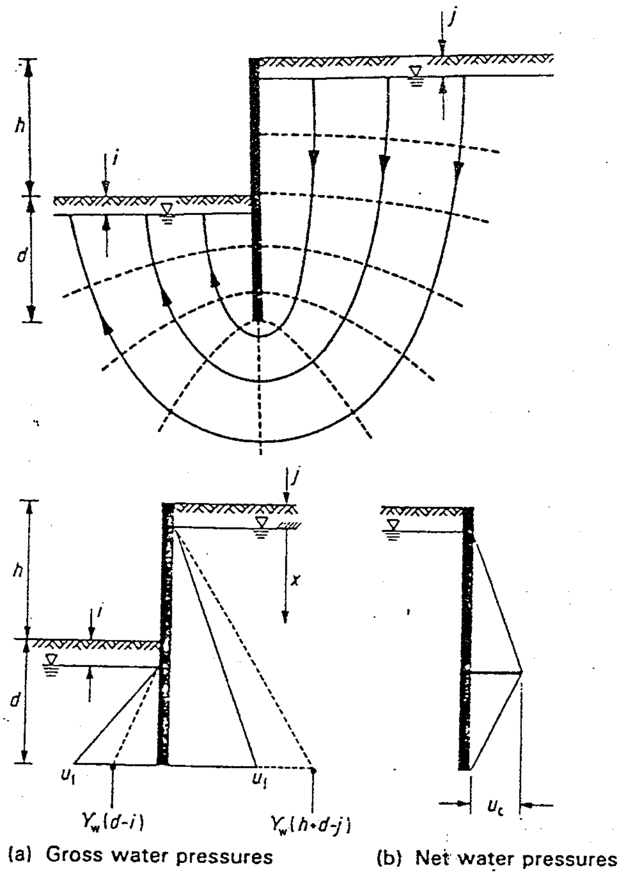


Figure 27 - Seepage around an Embedded Wall

Hx = hydraulic head at any point a distance x below the watertable behind the wall $= \dfrac{x}{2d + h - i - j}(h + i - j)$ —— ①

Pore water pressure = (hydraulic head - elevation head) γ_w —— ②

u_f = pore water pressure at foot of wall

$$= \frac{2(d + h - j)(d - i)}{2d + h - i - j}\gamma_w \quad \text{—— ③}$$

A feature of the pore water pressure distribution derived from the steady-state seepage flow-net is the pressures, u_f, are equal on either side of the wall at the toe (assuming that the wall is thin in comparision to its height).

In calculation of wall stability, it is necessary to calculate the vertical effective stress at various points. Use of Equations (1) to (3) may be cumbersome and therefore lead to errors. A shorthand may be adopted as follows. Figure 2 shows the distribution of water pressure up the front and back of the wall. The rate of increase of water pressure with depth below the watertable is

$$\mu_a = \frac{u_f}{h + d - j} \quad \text{behind the wall} \quad \text{—— ④}$$

and

$$\mu_p = \frac{u_f}{(d - i)} \quad \text{in front of the wall} \quad \text{—— ⑤}$$

So that at any depth, x, below the watertable, the pore water pressure is

$$u_x = \mu_a x \text{ or } \mu_p x \quad \text{—— ⑥}$$

It may be seen that μ_a and μ_p have units of units weight and act as if they were modified values of the unit weight of water. They may be used in place of γ_w in the equation for effective lateral pressure at any depth, Z, where seepage occurs. The equation for total lateral pressure are therefore as follows :

For **active pressure** behind the wall (z measured from excavation surface)

$$(\sigma_h)_z = K_a \sigma_v' + u_z \quad \text{—— ⑦}$$

$$= K_a\left(\gamma z - \frac{(z - j)}{h + d - j}u_f\right) + \frac{(z - j)}{h + d - j}u_f \quad \text{—— ⑧}$$

$$= K_a[\gamma z - (z - j)\mu_a] + (z - j)\mu_a \quad \text{—— ⑨}$$

For passive pressure in front of the wall (z measured from excavation surface

$$(\sigma_h)_z = K_p \sigma_v' + u_z \quad \text{—— ⑩}$$

$$= K_p\left(\gamma z - \frac{z - i}{d - i}u_f\right) + \frac{z - i}{d - i}u_f \quad \text{—— ⑪}$$

$$= K_p[\gamma z - (z - i)\mu_p] + (z - i)\mu_p \quad \text{—— ⑫}$$

where γ is the bulk density of the saturated clay.

Net water pressure

The net water pressure acting on the wall under steady-state seepage conditions is shown in Figure 2(b). The largest net water pressure occurs at the level of the watertable within the excavation :

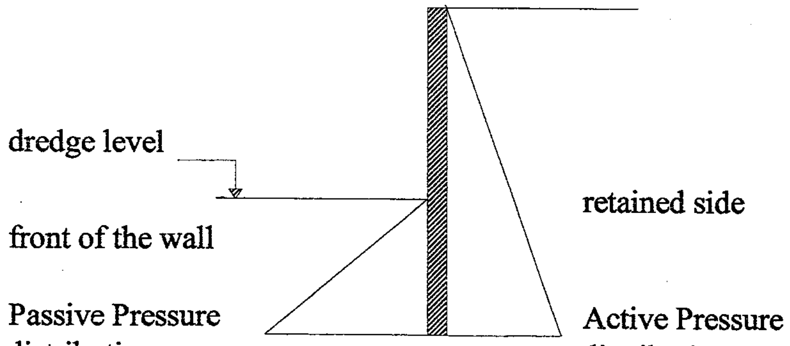
$$u_c = (h + i - j)\,\frac{2\,(d - i)}{2d + h - i - j}\,\gamma_w = \text{maximum net pressure}$$

Seepage Stresses

The method given above of calculating soil and water pressures acting on the wall under steady-state seepage conditions fully accounts for seepage stresses acting on the soil skeleton. A separate calculation of seepage stresses is therefore not required.

9.2.2 Equilibrium Consideration

After the computation of design earth pressures pore water pressures the depth of embedment required is estimated by equilibrium consideration. Resistance from any available lateral support system is also considered.



Bending moments due to the earth pressures and pore water pressure on either side of the wall and forces from lateral supports are also computed. A wall section capable of withstanding these bending moments is provided.

Use of either of (1). Fixed Earth Support Method.
(2). Free Earth Support Method is recommended by BS 8002.

9.2.3 Methods of Analysis and Design

(a). Fixed Earth Support Method

This method was developed by Blum (1931). It can be applied to analyse both cantilevered walls and walls with lateral support. The wall is considered to be flexible but driven to a sufficient depth so that it can be considered fixed at the toe. Thus the wall is assumed to rotate about a point slightly above the toe. The stresses (or forces) on the wall slightly above the toe and below the point of rotation are replaced by a single force F_C.

This method becomes very tedious with laterally supported walls. It is simplified by a procedure known as "Blums equivalent Beam Method".

(b). Free Earth Support Method

This method can be applied only in the design of laterally supported walls. Walls is assumed to be rigid rotating about the point of support (anchor or prop). The depth of wall embedment is estimated on the basis of achieving moment equilibrium at the support level. The lateral support force is then calculated on the basis of horizontal equilibrium. The point of maximum bending moment is determined from zero force in the shear force diagram.
Following the work of Rowe (1952), the design B M used in determining the Sheet pile section is obtained by reducing the maximum bending moment by a factor which depends on the relative flexibility of the wall with respect to the Soil.

This infact is the oldest and most conservative design method. It is found to be giving economical designs with smaller depths of embedment. But the B M' s computed are larger than those determined from the fixed earth support method.

9.2.4 Design of Multi Propped Walls

When the embedded wall is supported by a multi stage support system, the code suggest to use a trapezoidal distribution as shown in figure for the computation of strut loads or anchor loads. (Figure 28 - Figure 37 of Code) - taken from Terzaghi and Peak (1967). Loads due to water pressure and surcharges should be added to the values in the trapezoidal diagram.

For granular material k_A can be determined from Kerisel and Absi charts given. In soft to medium clay soils k_A can be computed from undrained shear strength using $k_A = 1 - \frac{4C_u}{\gamma H}$. The diagram for stiff fissured clays is said to be tentative and it was said that the lower pressures are relevant only when movement can be kept to a minimum and construction time is short.

Above trapezoidal pressure distribution should not be used to compute bending moments in the wall. They should be computed using Figure 29. For walls with two level of anchorage, a design method based on assuming an equivalent anchor (B. J. Jack 1971) is recommended.

9.2.5 General Recommendations of BS 8002 on Embedded Retaining Walls

Some general recommendations listed in BS 8002 on embeded retaining structures are discussed under this section. It is recommended to use Rowe's moment reductions for laterally supported flexible walls. They are not be used for cantiliver walls. Moment reduction is to be applied when the wall is free to deflect. No moment reduction factors are to be applied for rigid walls.

For Steel Sheet Piling

- Recommends to use free earth support method. It gives heavier sections and lesser depths of embedment. Heavier section is useful in driving.

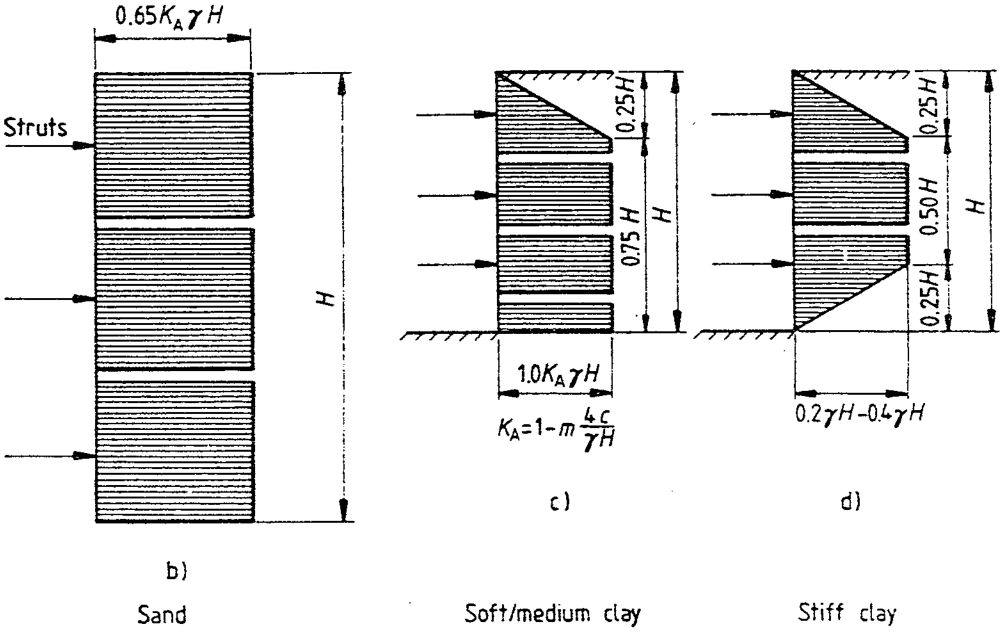


Figure 37. Active pressure diagrams relating to maximum strut loads in braced earth retaining structures

NOTE. *Source: Soil Mechanics in Engineering Practice* (second edition). Terzaghi. © 1967 Reprinted by permission of John Wiley & Sons Inc.

Figure 28 - Strut Load Diagram

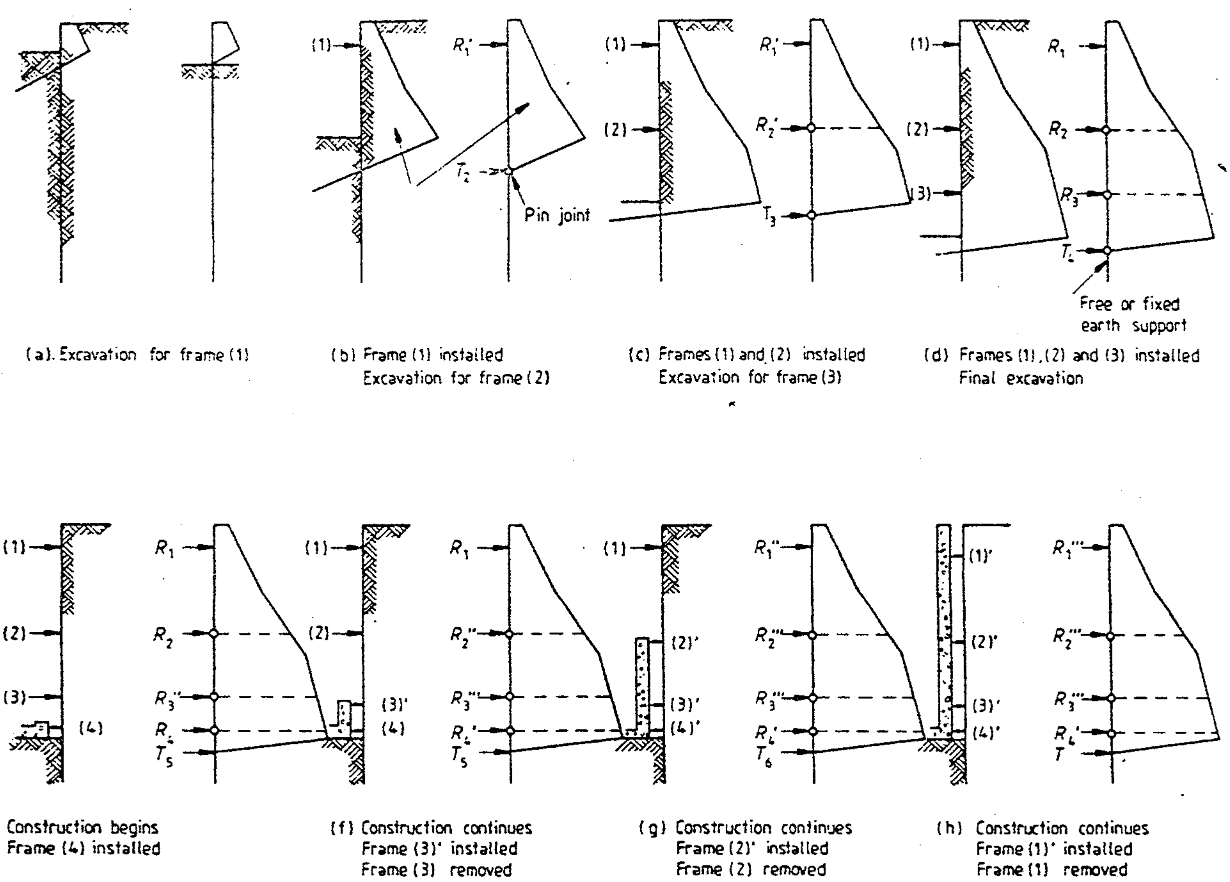


Figure 38. Illustration of method of calculation of bending moments and frame loads by successive stage analysis in cofferdams

Figure 29 - Calculation of Bending Moments in Multi Propped Walls

- Section selected may have to be increased to allow for corrosion specially in aggressive environments
- Selected section should be able to resist the stresses imposed during the installation ie. impact, vibratory or hydraulic driving

- Table 5 of the Code (pp 69) provides some guidance on this in cohesionless soils.
- Table 6 provides guidance for cohesive soils.

- Methods of increasing the life by use of coatings, concrete encasement, cathodic protection etc. are suggested.

Timber Sheet Piles

- Useful for moderate heights
- Certain softwoods & hardwoods are suitable as permanent piling
- Working stresses should not exceed values recommended by BS 5268
- Steel ban to be used at top during driving

Reinforced & Prestressed Concrete Sheet Piles.

- Jetting or preboring can be used to assist driving
- Good durability & appearance can be obtained.
- Design, manufacture & handling to be done according to relevant codes BS 8110 and BS 5400
- Thickness range from 120 - 400 mm.
- Wedge shaped toes are to be used
- tongued & grooved type joints to be used
- water tightness can be achieved by grouting
- prestressed concrete piles have ;

  high strength, can withstand hard driving
  crack free as no tensile stresses are developed
  are of greater durability

In situ bored pile walls

- Best application in cohesive soils
- Only one pile can be done at a time and this minimise effects on adjacent foundations
- Can penetrate moderately hard bedrock
- when anchors or props are used to provide lateral support, wales to be provided along the face of the wall to unify behaviour. Gap between the beam & wale to be filled properly
- A continuous structural beam to be cast along head of the piles to unify the behaviour.
- tolerances 1in 75 to 1in 100 for verticality 1 : 200 for secant piles
- vertical drains between piles to collect and dispose water
- different types of surface treatment

  structural concrete
  gunite
  precast concrete
  brick facing

Concrete diaphragm walls

- Cast in place using benotonite or polymer suspension
- Walls are formed in panels

Table 5. Selection of pile size to suit driving conditions in granular soils using impact hammers			
Dominant SPT N Value	Minimum wall modulus cm^3/m		Recommended maximum driving length m
	Grade 5275P mild steel to BS EN 10025 : 1990	Grade 5355P high yield steel to BS EN 10025 : 1990	
0 to 10	450		7
11 to 20		450	9
21 to 25	850		11
26 to 30		850	14
31 to 35	1300		16
36 to 40		1300	18
41 to 45	2300		20
46 to 50		2300	22
51 to 60	3000		24
61 to 70		3000	26
71 to 80	4200		30
81 to 140		4200	30+

NOTE 1. N is the standard penetration test (SPT) blow count. Dominant means the high average for the soils. Where piles are to be driven only to a toe hold in rock, the SPT value should be divided by a factor of 4 for that stratum only.

NOTE 2. For SPT values exceeding 50, pile damage, declutching and/or refusal may occur. Additional consideration should be given to the presence of cobbles or boulders, which may give rise to obstructed driving, damage and or declutching.

Table 6. Selection of pile size to suit driving conditions in cohesive soils			
Clay description	Minimum wall modulus, cm^3/m		
	Grade 5275P mild steel to BS EN 10025 : 1990	Grade 5355 P high yield steel to BS EN 10025 : 1990	Maximum length m
Soft to firm	450	400	6
Firm	600 to 700	450 to 600	9
Firm to stiff	700 to 1600	600 to 1300	14
Stiff	2000 to 2600	1300 to 2000	16
Very stiff	2600 to 3000	2000 to 2500	18
Hard (c_u > 200 kN/mm^2)	Not recommended	4200 to 5000	20

NOTE. The ability of piles to penetrate any type of ground depends upon attention being given to good pile driving practice. Tables 5 and 6 assume such good practice.

- Concrete placed by tremie
- Guide walls, to be build to define line and thickness of the wall
- Bentonite-two types, Sodium & Calcium Sodium bentonite is generally used
- Construction joints - stop end tubes are to be used
- If they are designed to carry vertical loads in end bearing it is advised to provide a clean base

Prestressed Cast in situ walls
- Cast in place diaphragm wall can be prestresses using post tensioning technique
- Cables should be looped so that both ends are accessible at the top of the wall.

Precast Concrete walls
- Formed by precast units placed in a bentonite filled trench.
- Different jointing system. Several prefabricated systems exists.

9.3 Recommendations about Soldier Piles

- **Material**
- Piles may be of reinforced concrete or steel sections
- Steel concrete or timber may be used for sheeting

- **Design**
- Earth pressures will depend on stiffness of the system. The design should cater for all stages of the excavation.
- Piles and walling should be able to redistribute 40% of the load to adjacent anchors in case of a failure.
- Adequate penetration of piles below formation level should be provided.
- Where timber sheeting is used 50% reduction of soil pressure may be assumed due to the arching of the soil. With the use of stiffer sheeting material the relieving effect of arching is much reduced.

- **Construction**
- Excavations of upto 1m can be carried out in clay soils prior to the placing of sheeting.
- Gaps left between sheeting to allow release of water. But loss of ground should be prevented.

9.4 Recommendation on Strutted Excavations and Cofferdams

- **Design**
- The design of piling should be checked for every construction stage and stages during removal. Presence of heavy plant should also be accounted. Depending on the type of support and type of soil free or fixed earth support methods, and Figure 38 of BS 8002 (Figure 29) can be used in the design.
- Strut loads may be approximated using trapezoidal distribution in Figure 37 of BS 8002 (Figure 28)
- Innerline of piling in the double wall cofferdams should be designed as an anchored retaining wall, while the outer line of piling acts as the anchorage.
- The width should not be less than 0.8 of the retained height.

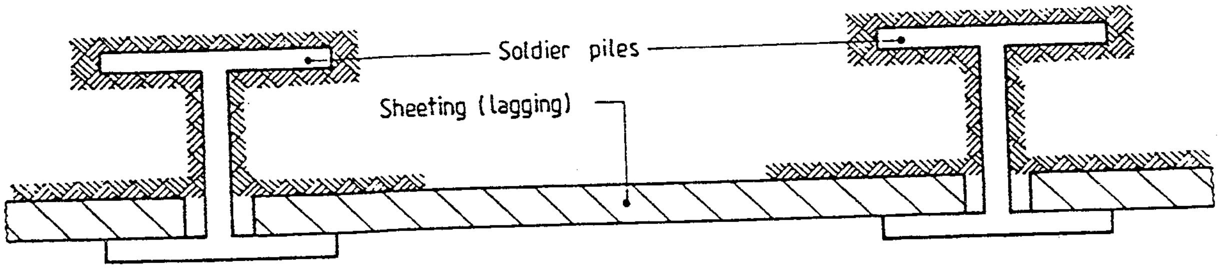


Figure 41. Horizontal sheeting (lagging)

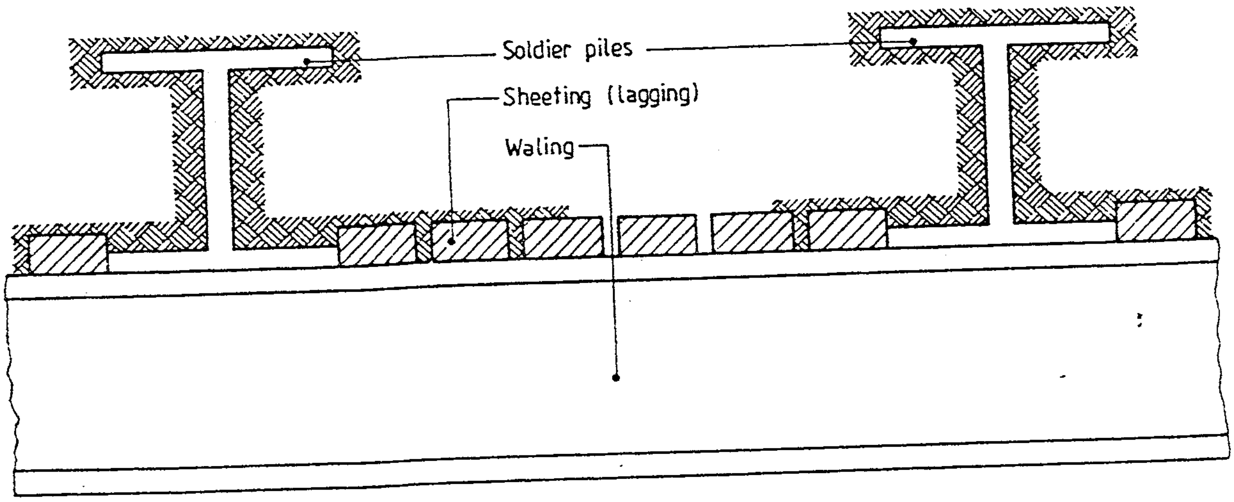


Figure 42. Vertical sheeting (lagging)

Figure 30 - Soldier Pile Walls

The inner wall is usally provided with weep holes near the bottom to reduce water pressure and to prevent a decrease in the total shear strength of fill material. As it is impossible to provide complete drainage it is recommended to have a hydrostatic head of 0.3 x retained height.

- **Construction**
- Minimum depth

  In cohesionless soils where there is an appreciable difference in water levels piles should be driven to a sufficient depth to prevent instability by piping. Alternatively use of less permeable blanket should be considered.
- Circular cofferdams

  The piles should be driven in stages as the hammer works its way several times around the circumference. A diaphragm wall can also be used to form a circular cofferdam. Earth pressures should be calculated as for straight-sided cofferdams and piles should be supported by circular ring beam, instead of wallings and struts.

Due to the deviations in practice from a true circle the ring beams are subjected to eccentric loading and a check should be made for backling in the ring with the radial working load w (in kN/m) determined from

$$w = \frac{1.5 \quad EI}{R^3 \times 10^5}$$

where E = Young's modulus of walling material in N/mm^2
I = Moment of inertia about xx axis in cm^4
R = Radius of cofferdam in metres

- Earth filled cofferdams

  Clays or silts should not be used as filling material and any soft soils of these types which may enclosed should be removed prior to filling.

Weepholes, with graded filters if necessary should be provided near the bottom of the exposed portion of the piles on the inner side.

9.5 Comments from BS 8002 on Reinforced Concrete and Reinforced Masonry Walls on Spread Footings

Code lists the types of walls as
(a). Cantilever or stem wall
(b). Counterfort wall
(c). Buttressed wall
(d). Reversed cantilever wall
(e). Precast retaining wall

Code states that for heights upto about 8m a cantilever wall is generally economical. For greater heights the greater bending moment on the stem demands a greater stem thickness and a counterfort wall is more appropriate. Buttressed retaining walls are seldom used. Material used should comply with BS 8110, part I, BS 5328 part I and part 2 or BS 5400 part 4.

Cantilevered Walls

- The base slab and the stem of the wall shall be designed to withstand the design soil pressure.

Counterfort Walls

In counterfort and buttressed walls, counterforts should be designed as cantilevers and wall stem should be designed as a continuous slab. The upper portion of the wall spans horizontally between the counterforts and the calculations should be made for unit strips carrying a uniformly distributed pressure appropriate to the depth.

The lower portion of the wall slab should be designed as cantilevering from the base and simultaneously spanning between the counterforts.(Figure 23 - BS 8002)

Also in BS 8002 ;
Need to provide proper movement joints is highlighted

Durability aspects of the wall are discussed

- Structure should be designed for durability in accordance with BS 8110
- adequate cover to be kept in concrete
- when ground water is present it is suggested to paint the back of the retaining wall with a suitable bituminous material or cover it with a self adhesive plastic sheeting cover with a suitable water proofing membrane.
- Chemical analysis of groundwater is recommended to assess its sulphate content

- **Basement walls, Excavations, Support and retention systems**

Construction processes (and aspects) in basement walls after providing initial support for the excavation through temporary support systems was discussed.

- **Reinforced and Prestressed Masonry Retaining Walls**

Reinforced masonry is said to be suitable for retaining walls over 1.5m high, while prestressed masonry is usually economical for walls over 4m high. Both methods are said to provide walls with high appearance qualities and good weathering capability.

Reinforcements provide flexural tensile capacity to the wall section and this improves the lateral load carrying capacity with respect to unreinforced masonry walls. Presstressed masonry is a technique where precompression is induced in the masonry cross section. Prestressing is usually provided by post-tensioning

The structural design and construction and workmanship of reinforced and masonry walls should comply with BS 5628.

10. Examples Design Computations

Example 1

A Gravity retaining wall has to be designed to retain a fill of 8m. Backfill material has effective shear strength parameters of $C' = 10$ kpa and $\phi' = 25$ deg. The wall adhesion can be taken as C_w =10 kpa and the angle of friction is found to be $\delta = 17°$. Bulk density of the fill material is 17 kN/m^3.

Founding soil is with effective strength parameters $C' = 15$ Kpa and $\phi' = 30°$. Base adhesion was found to be 15 kpa and the interface friction angle at the base was found to be $\delta_b = 20°$

Allowable bearing pressure of the founding soil is given as 275 kpa.

Design the retaining wall.

(a). According to CP2

(b). According to BS 8002.

Design According to CP 2

Use the code coefficients to compute the earth pressures. Peak soil strength parameters given are to used in the given formulae.

$$P_{AN} = \sigma_V\, k_A - C\, k_{AC}$$

$$\phi' = 25° \qquad \delta = 0 \rightarrow k_A = 0.40$$

$$\delta = 25 \rightarrow k_A = 0.32$$

$$\therefore\ \delta = 17 \rightarrow k_A = 0.40 - \frac{17}{25} \times 0.08 = 0.346$$

$\underline{k_{AC}}$

$$\delta = 0 \qquad \frac{C_w}{C} = 1 \rightarrow k_{AC} = 1.76$$

$$\delta = 25 \qquad \frac{C_w}{C} = 1 \rightarrow k_{AC} = 1.41$$

$$\therefore\ \delta = 17 \qquad \frac{C_w}{C} = 1 \Rightarrow k_{AC} = 1.76 - 0.35 \times \frac{17}{25}$$

$$= 1.522$$

$$P_{AN} = \gamma z\, k_A - C\, k_{AC} = 17Z \times 0.346 - 10 \times 1.522$$

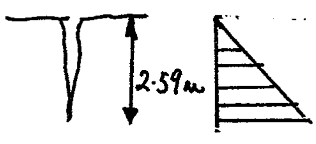
$$= 5.882\, Z - 15.22$$

when $Z = 0$ $\qquad P_{AN} = \text{-ve}$ $\qquad$ tension crack

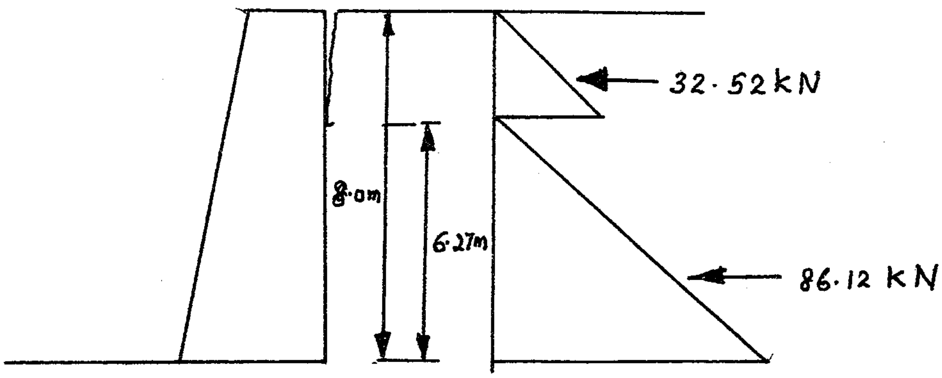
$$P_{AN} = 0 \quad \text{when} \quad Z = Z0 = \frac{15.22}{5.882} = 2.59 \text{ m}$$

when $Z = 8$ $\qquad P_{AN} = 5.882 \times 8 - 15.22 = 31.836$ kN/m^2.

water in the tension crack

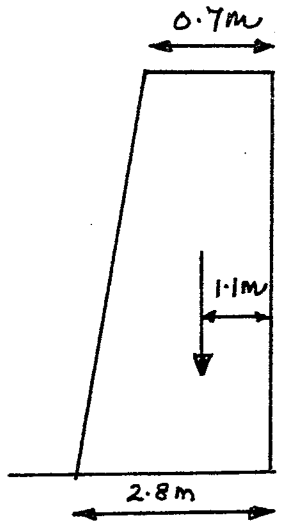


p_w = 2.59 x 9.81 = 25.41 kN/m^2
P_w = 1/2 x 2.56 x 25.41 = 32.52 kN/m
l_w = 1/3 x 2.59 + (8 - 2.59)
= 6.27m



P_{AN} = 1/2 x (8 - 2.59) x 31.836
= 86.12 kN/m
l_n = 1/3 (8 - 2.59) = 180
P_{AV} = P_{AN} tan δ + C_w x (8 - 2.59)
= 86.12 tan 17 + 10 x (8- 2.59)
= 26.33 + 54.1 = 80.23 kN

Now lets design the retaining wall. Lets have trial dimensions of



γ_{conc} = 22 kN/m^3
W = 1/2 (0.7 + 2.8) x 22
= 310 kN/m

Centre of gravity is at 1.1m from heel

FOS on overturning

Using the conventional method adopted by the code

$$\text{FOS} = \frac{P_{AV} \times 2.8 + W \times (2.8 - 1.1)}{P_w\, l_w + P_{AN}\, l_N}$$

$$= \frac{80.23 \times 2.8 + 310 \times 1.7}{32.52 \times (6.27) + 86.12 \times 1.80}$$

$$= \frac{224.8 + 527.0}{203.9 + 155.0} \quad \frac{751.8}{358.9} = \underline{2.09}$$

Hence OK

FOS on Sliding

Force causing sliding $= P_{AN} = 86.12\text{ kN} + 32.52 = 118.6\text{ kN}$

Sliding Resistance $= (W + P_{AV})\ \tan\delta + C_w B$

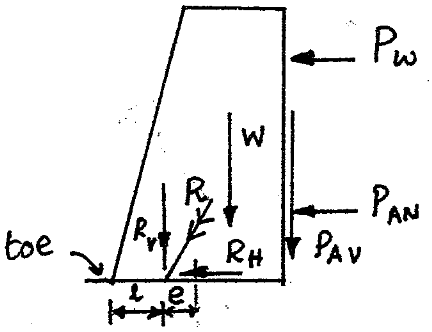
$= (310 + 80.23) \tan 20 + 15 \times 2.8$

$= 142.03 + 42 = 184.03\text{ kN}$

FOS on sliding $= \dfrac{184.03}{118.6} = 1.55$ **Hence OK**

Bearing Capacity

Find the location of the resultant force



Taking moments about toe

$R_V \times l = W \times 1.7 + P_{AV} \times 2.8 - P_w l_w - P_{AN}\ l_N$

$$R_V = 310 + P_{AV} = 310 + 80.23 = 390.23\text{ kN}$$

$$390.23 \times l = 310 \times 1.7 + 80.23 \times 2.8 - 32.52 \times 6.27 - 86.12 \times 1.8$$

$$= 527 + 22464 - 203.90 - 155.01$$

$$= 392.73$$

$$l \approx 1.0\text{ m}$$

$$\therefore\ e \approx 0.4\text{ m}$$

$$\sigma_{V,\max} = \frac{R_V}{B}\left[1 + \frac{6e}{B}\right]$$

$$= \frac{390.23}{2.8}\left[1 + \frac{6 \times 0.4}{2.8}\right] = \underline{258.86\text{ kN/m}^2}$$

σ allowable = 275 kN/m^2 Hence OK.

* Check

$$\sigma_{V,\,min} = \frac{390.23}{2.8}\left[1 - \frac{6 \times 0.4}{2.8}\right] = 19.90\ \text{kN/m}^2$$

+ ve. $\therefore$ no tensile stresses at foundation level
(Note e < B/6)

Design According to BS 8002

Given parameters are peak strength parameters. Hence design parameters are obtained by using the mobilisation factor.

for backfill

$C' = 10$ kpa $\quad C_{design} = \frac{10}{1.2} = 8.33\ \text{kN/m}^2$

$\phi' = 25$ deg $\quad \phi_{design} = \tan^{-1}\left(\frac{\tan 25}{1.2}\right)$

$= 21.2$

$\delta = 0.75 \times \phi_{design} = 15.9$

$C_w = 0.75 \times 8.33 = 6.24\ \text{kN/m}^2$

For founding soil

$C' = 15$ kpa $\quad C_{design} = \frac{15}{1.2} = 12.5\ \text{kN/m}^2$

$\phi' = 30$ deg $\quad \phi_{design} = \tan^{-1}\left(\frac{\tan 30}{1.2}\right)$

$= 25.7°$

$\tan\delta_b = \tan\ 20/1.2 = 0.303 \Rightarrow \delta_b = 16.87$

$C_w = 0.75 \times 15.0 = 11.25\ \text{kN/m}^2$

Use code expressions to compute active force consider the obligatory surcharge of 10 kN/m^2.

$\sigma_{an}' = k_a \sigma'_v - 2C' \sqrt{k'_a}$

$\sigma'_v = \sigma_v = (\gamma z + q) = 17\,Z + 10$

Refer to the charts in the code
Chart A.1
backfill $\quad \phi_{design} = 21.2 \quad C_{design} = 8.33\ \text{kN/m}^2$

for $\quad \delta/\phi' = 0.66 \qquad k_A = 0.41$

$\quad \delta/\phi' = 1.0 \qquad k_A = 0.39$

Hence of $\delta/\phi' \approx 0.75 \qquad k_A \approx 0.40$

$\sigma_{an}' = (17Z + 10)\,0.4 - 2 \times 8.33 \times \sqrt{0.40}$

$= 6.8Z + 4.0 - 10.53 = 6.8Z - 6.53$

depth of the tension crack $Z_0 = \dfrac{6.53}{6.8} = 0.96$ m

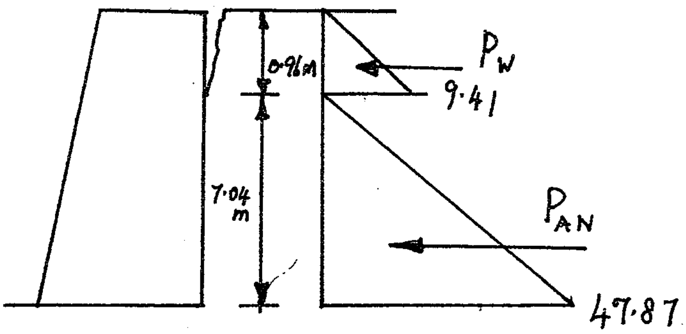
at $\quad Z = 8\text{m} \qquad \sigma'_{an} = 6.8 \times 8 - 6.53$

$= 47.87\ \text{kN/m}^2$

at the bottom of tension crack

$p_w = 0.96 \times 9.81 = 9.41\ \text{kN/m}^2$

$P_w = 1/2 \times 9.41 \times 0.96 = 4.5$



$P_w = 1/2 \times 9.41 \times 0.96 = 4.51$ kN

$P_{AN} = 1/2\ (47.87) \times 7.04 = 168.5$ kN

$P_{AV} = P_{AN} \tan\delta + C_w \times l$

$= 168.5 \tan 15.9 + 6.24 \times (8 - 0.96)$

$= 47.99$ kN $+ 43.93$

$= \underline{\underline{91.92\ \text{kN}}}$

Lets consider the resistance to sliding

Force causing sliding $= P_w + P_{AN} = 4.51 + 168.5 = 173.0$ kN

Sliding resistance without considering adhesion,

$\tan\ \delta_b = \tan\ 16.87 = 0.303$

$\tau = \sigma' \tan\delta_b$

$S = R_V \tan\delta_b$

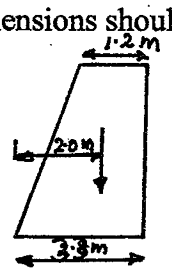
$= 401.92 \tan 16.87 = 121.91\ \text{kN} < 173.0\ \text{kN}$

$\therefore$ Forces are not in equilibrium. This serviceability condition is not satisfied. If adhesion is considered

$S = R_V \tan\delta_b + C_w B = 121.91 + 11.25 \times 2.8$

$= 153.41$ Still less than 173.0 kN

Hence the wall dimensions should be increased. Increase the base width by 0.5 m.



$\therefore$ W increase = 0.5 x 8.0 x 22
= 88.0 kN
W = 310 + 88 = 398.0 kN
Centre gravity is 2.0 m from toe

Now to find the line of action of the resultant force at the foundation level.

$R_V = 398 + 91.92 = 489.92$ kN
$\tau_f = R_v \tan 16.87 + 11.25 \times 3.3 = 185.69 \;\rangle\; 173.0 \;\; kN$

Hence OK

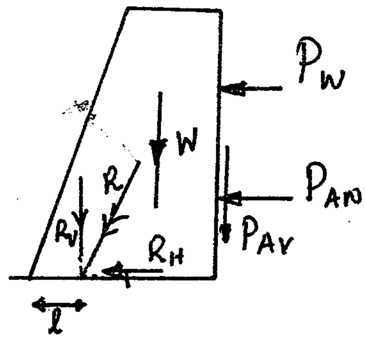
Bearing Stresses

$$R_V l = 398 \times 2.0 + 91.91 \times 3.3 - 4.51\left(7.04 + \frac{0.96}{3}\right) - 168.5 \times \frac{7.04}{3}$$

$$489.92\; l = 796 + 303.3 - 33.1 - 395.41$$
$$= 670.79$$
$$l = 1.36 \text{ m}$$
$$\therefore e = 1.65 - 1.36 = 0.29 \text{ m}$$

$$\frac{6e}{B} = \frac{6 \times 0.29}{3.3} = 0.527 < 0.55 = B/6$$

$\therefore$ no tension at the base



W = 298 kN
P_W = 4·51 kN
P_{AN} = 168·5 kN
P_{AY} = 91·91 kN

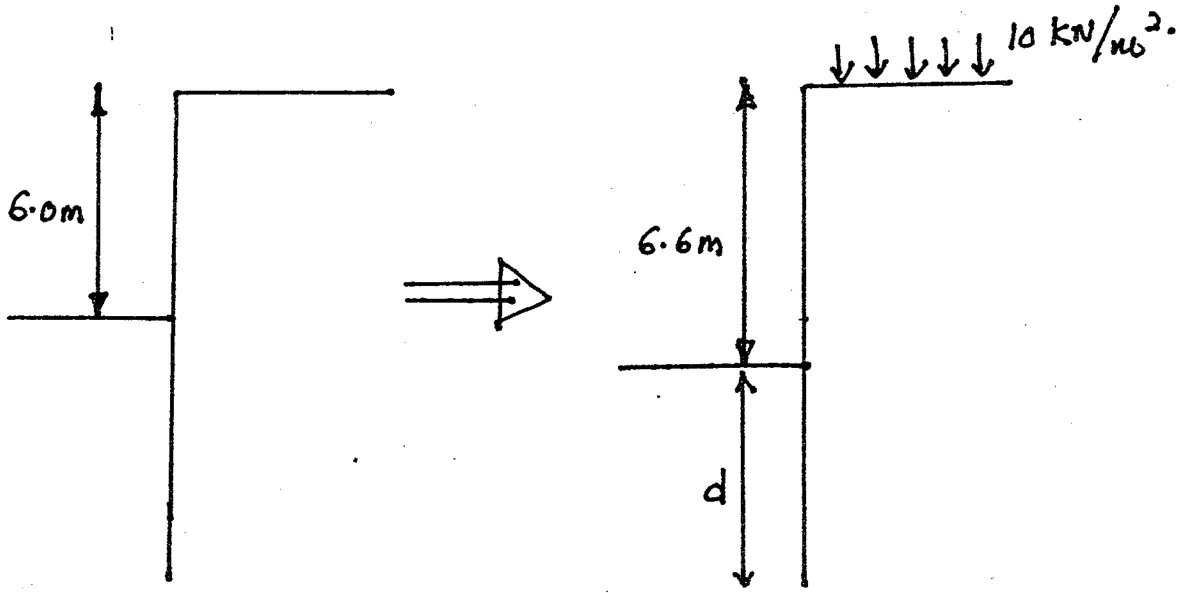
$$\sigma_{V,max} = \frac{R_V}{B}\left(1 + \frac{6e}{B}\right) = \frac{489.92}{3.3}(1 + 0.527)$$
$$= 226.73 \text{ kN/m}^2$$

$$\sigma_{all} = 275 \text{ kN/m}^2 \rightarrow \textbf{Hence OK}$$

Hence selected dimensions are satisfactory.

Example 2

Design a cantilevered sheet pile wall to retains 6 m of dry sand of $\phi = 30^{\circ}$. Bulk density of sand is 18 kN/m^3. Assume the sheet pile to be smooth and Rankine K_A values can be used.



$$\tan \phi_{design} = \frac{\tan 30}{1.2} = 0.481 \Rightarrow \phi_{design} = 25.69^{o}$$

$$\text{Rankine } K_A = \frac{1 - Sin\phi}{1 + Sin\phi} = 0.395 \quad \therefore\ K_p = 1/K_A = 2.532$$

$$P_A = \sigma_v K_A = (\gamma z + q) K_A = (18\,Z + 10)\,0.395$$
$$= 7.11\,Z + 3.95$$

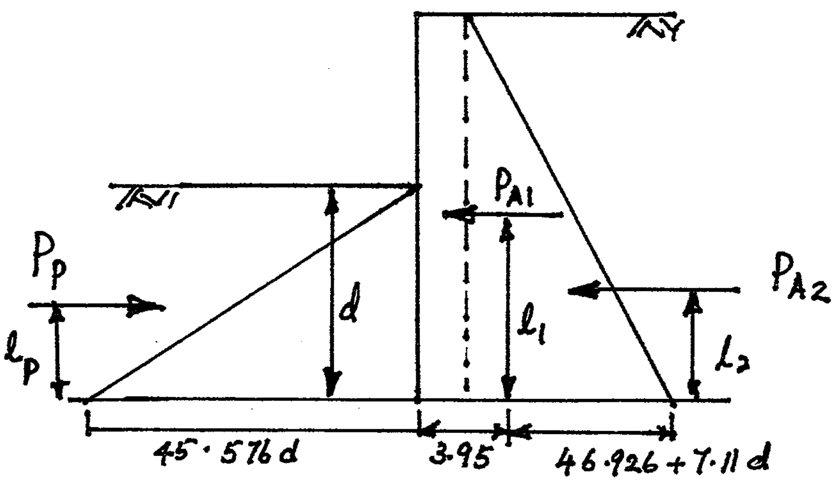
at $Z = 0$ $\quad P_A = 3.95$ kN/m^2

$Z = (6.6 + d)$ $\quad P_A = 7.11\,(6.6 + d) + 3.95 = 7.11\,d + 50.876$ kN/m^2

$$P_P = \sigma_v K_p = \gamma Z K_P = 18 \times 2.532\,Z = 45.576\,Z$$

when $Z = 0$ $\quad P_P = 0$

$Z = d$ $\quad P_P = 45.576\,d$



Taking moments about C

$$P_p \times l_p = P_{A1} \times l_1 + P_{A2} \times l_2$$

$$1/2(d)(45.576d)(d/3) = 3.95(6.6+d) \times \left(\frac{6.6+d}{2}\right) + 1/2(d+6.6)(46.92 + 7.11d) \times \frac{(d+6.6)}{3}$$

$$7.596\, d^3 = 1.975\,(d + 6.6)^2 + (d + 6.6)^3 \times 1.185$$

$$f(d) = 7.596\, d^3 - 1.185\,(d + 6.6)^3 - 1.975\,(d + 6.6)^2 = 0$$

by $d = 8.0$ m

$$f(d) = 7.596\,(8)^3 - 1.185\,(14.6)^3 - 1.975\,(14.6)^2 = -219.72$$

by $d = 8.5$ m

$$f(d) = 7.596\,(8.5)^3 - 1.185\,(15.1)^3 - 1.975\,(15.1)^2 = 116.68$$

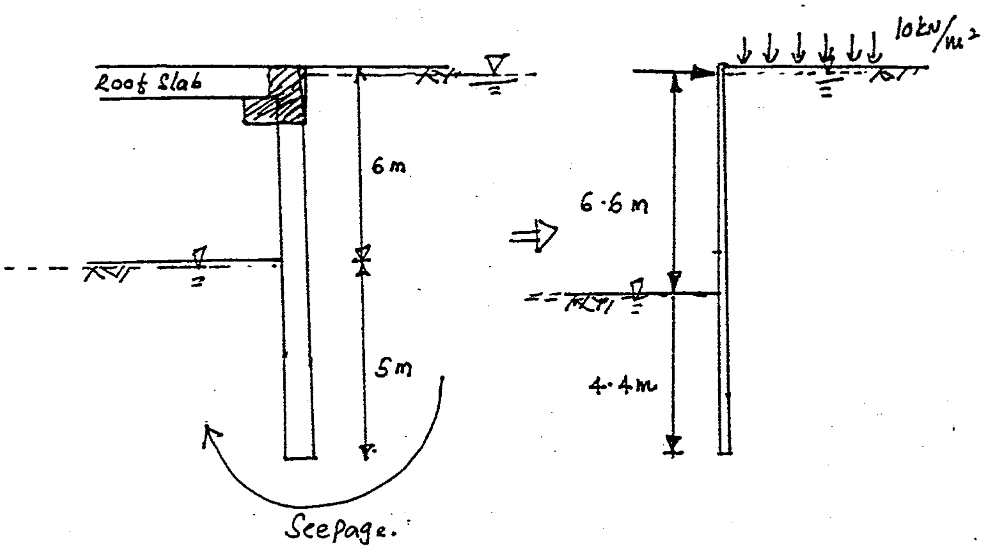
Hence use $d = 8.5$ m Increase by 20% = 10.2 m

∴ Total height of the sheet pile (Length) = 6.6 + 10.2 = <u>16.8 m</u>

Example 3

It is proposed to construct and Underground car park using the cut and cover techniques. The 6 m deep excavation is to be supported by a secant pile wall constructed around the perimeter. The roof slab is supported on the Plinth beam cast at the top level of the piles and is acting as a prop support. Ground water level is close to the surface and car park to be kept dry by continuous pumping of water. A schematic arrangement of the proposed set up is shown in the figure.

Evaluate the stability of the proposed wall.



$C^1 = 0 \qquad \phi^1 = 30^\circ$
$\gamma_{sat} = 18 \text{ kN/m}^3$

$\phi_{design} = \tan^{-1}(\tan\phi / 1.2) = 25.7$

wall is rough $\qquad \delta = 0.75\,\phi_{design} = 19.3$

(a) Evaluation of long term stability

Consider the overdig 10% of 6.0 m = 0.6 m and the minimum obligatory surcharge of 10 kN/m²

Long term flow condition around the retaining wall is similar to that in Figure 27 (page 61)

This flow condition should be considered in evaluating pore water pressures.

However, as suggested by BS 8002 pore water pressure computation process is simplified by assuming a uniform head loss around the wall.

The total head loss = 6.6 m
(Critical water level is taken as the Excavation level with overdig)

This head loss occurs over the length = 6.6 + 4.4 + 4.4
= 15.4 m

∴ Head loss up to the toe level in the active side = 6.6/15.4 x (6.6 + 4.4)
= 4.71 m

∴ Pore water pressure at toe level = $(11 - 4.71)\,\gamma_w$
= 61.66 kN/m²

Alternatively if formula (3) in page 62 is used

$$U_f = \frac{2(d+h-j)(d-i)}{2d+h-i-j} \times \gamma_w = \frac{2(4.4+6.6-0)(4.4-0)}{2\times 4.4+6.6+0-0} \times 9.8$$

$$= 61.66 \text{ kN/m}^2$$

$\sigma_{an} = K_A\,\sigma_v^1 + u$

$\phi_{design} = 25.7 \approx 26.0 \qquad \delta = 0.75\,\phi$

Refer chart A1

<u>K_A</u>

$\delta = 0.66\,\phi \qquad K_A = 0.34$
$\delta = \phi \qquad K_A = 0.33$

$\therefore \delta = 0.75\,\phi \Rightarrow K_A = 0.34 - 0.01 \times 9/34 = 0.337$

$\underline{K_P}$

$\delta = 0.66\,\phi \qquad\qquad K_P = 3.7$

$\delta = \phi \qquad\qquad\quad K_P = 4.1$

$\delta = 0.75\,\phi \quad \Rightarrow \quad K_P = 3.7 + 0.4 \times 9/34 = 3.806$

Both the active and Passive Pressure distributions would be triangular. Hence determine the values at top and bottom only.

Active side

$$\sigma_{an} = K_A \sigma_v^1 + u$$

$$= 0.337\,(18\,Z + 10 - U) + U$$

at the top level $\qquad Z = 0 \qquad U = 0$

$$\sigma_{an} = 0.337\,(10) = 3.37\ \text{kN/m}^2$$

at the toe level $\qquad Z = 11\text{m} \qquad U = 61.66\ \text{kN/m}^2$

$$\sigma_{an} = 0.337\,(18 \times 11 - 61.66) + 61.66 = 110.97\ \text{kN/m}^2$$

Passive side

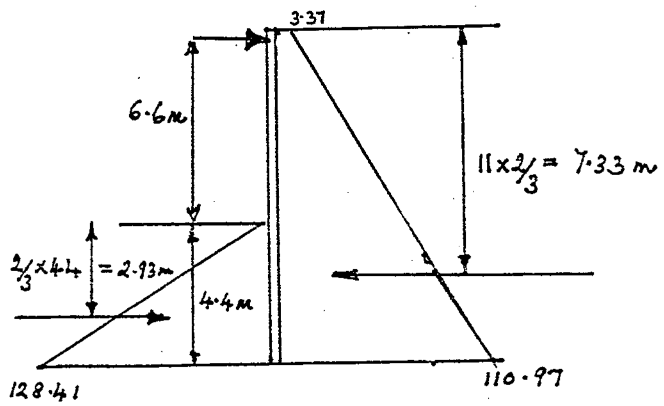
$$\sigma_{pn} = K_P \sigma_v^1 + u = 3.806\,(\gamma Z - u) + u$$
$$= 3.806\,(18\,Z - u) + u$$

at the bottom of the excavation level

$Z = 0 \qquad U = 0 \qquad \therefore\ \sigma_{pn} = 0$

at the toe level

$$\sigma_{pn} = 3.806\,(18 \times 4.4 - 61.66) + 61.66$$
$$= 128.41\ \text{kN/m}^2$$



Use the Free Earth Support method and check the stability - taking moments at the support level.

Disturbing moment = 3.37 x 11 x 11/2 + 1/2 x 11 x 107.6 x 2/3 x 11 = 4543.76 kNm

Resisting moment = 1/2 x 128.41 x 4.4 x (6.6 + 4.4 x 2/3)
= 2693.19 kNm

Resisting moment < Disturbing moment

Depth of Embedment is not adequate.

Increase the depth of embeddment by 3m. Hence after allowing for overdig, the depth of embeddment 4.4 becomes 7.4.
Now the pore water pressure at the toe level should be computed again

Total head loss of 6.6m occurs over 6.6 + 7.4 + 7.4 = 21.4

$\therefore$ Head loss upto toe level $= \dfrac{6.6}{21.4} \times (14.0)$
$= 4.32$ m

Pore water pressure at toe level = (14 - 4.32) 9.81
= 94.96 kN/m^2

Active Side

$$\sigma_{an} = k_A \sigma'_v + u$$

at depth Z $\sigma_{an} = 0.337(18Z + 10 - 94.96) + 94.96$

when Z = 14.0 m

$$\sigma_{an} = 0.337(18 \times 14 + 10 - 94.96) + 94.96$$
$$= 150.85 \text{ kN/m}^2$$

Passive Side

$$\sigma_{pn} = k_P \sigma'_v + u$$
$$= 3.806\,(18Z - u) + u$$

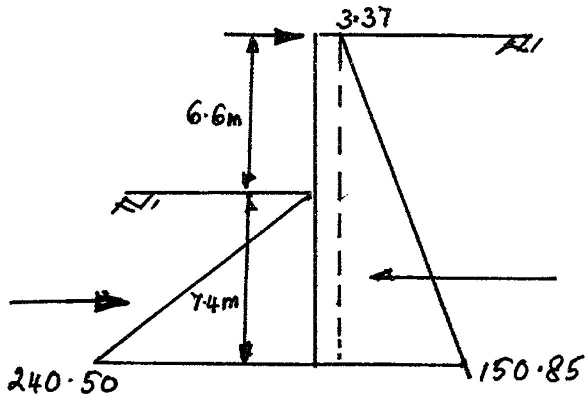
at the bottom of the excavation level

Z = 0 $\Rightarrow$ u = 0, $\sigma_{pn} = 0$

at the toe level

Z = 7.4 u = 94.96 kN/m^2

$$\sigma_{pn} = 3.806\,(18 \times 7.4 - 94.96) + 94.96$$
$$= 240.50 \text{ kN/m}^2$$



Use Free earth support method. Taking moments at the prop level

$$\text{Disturbing moment} = 3.37\times 14\times\frac{14}{2}+\frac{1}{2}\times 14\times 147.48\times\frac{2}{3}\times 14$$
$$= 330.26 + 9635.36 = 9965.62 \text{ kNm}$$
$$\text{Resisting moment} = \frac{1}{2}\times 240.50\times 7.4\times\left(6.6+\frac{2}{3}\times 7.4\right)$$
$$= 10262.94 \text{ kNm}$$

Resisting Moment > Disturbing Moment

Hence the selected depth of embeddment is satisfactory.

APPENDIX – A

Annexes

Annex A (normative)
Graphs for K_a and K_p

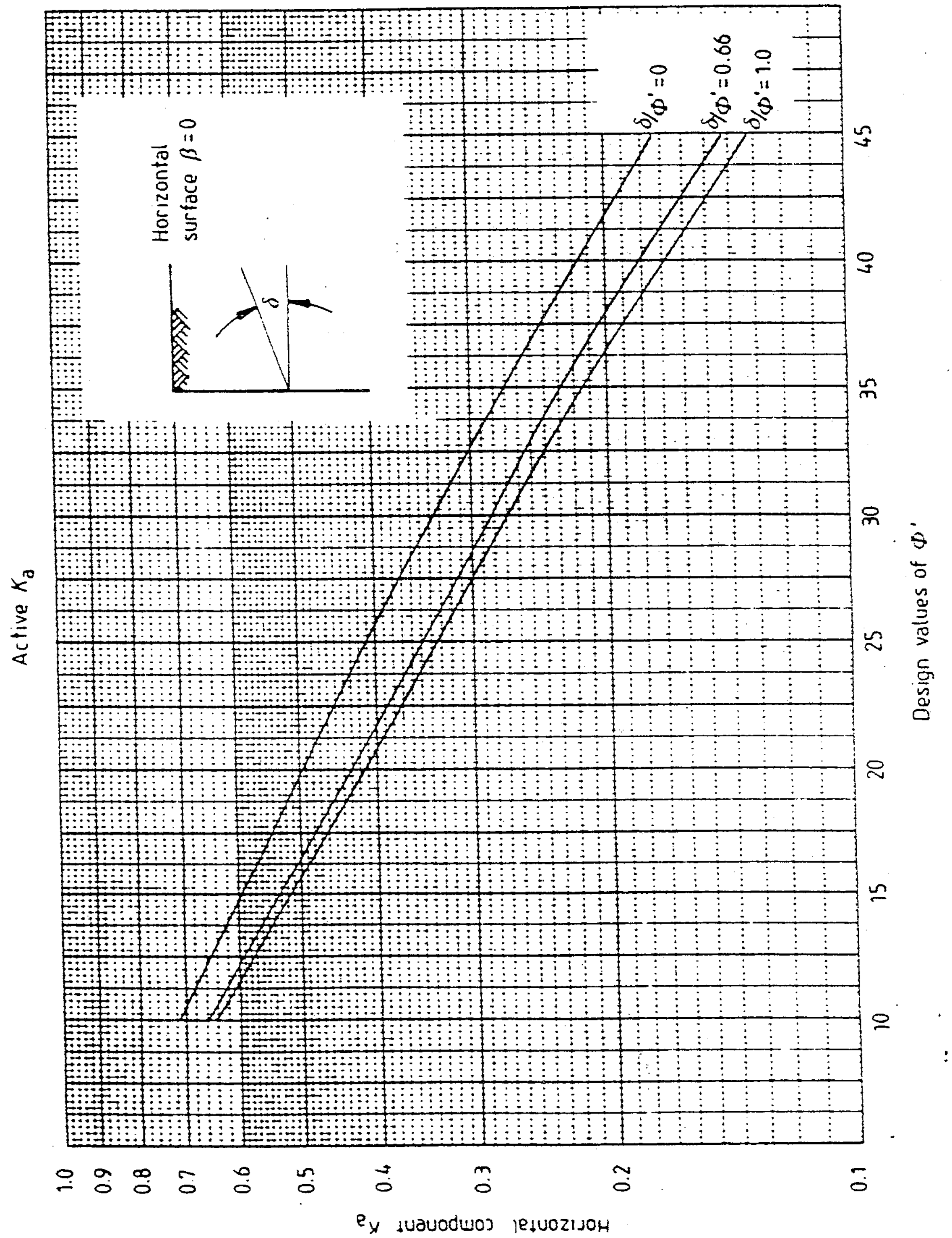


Figure A.1 Active pressure — Horizontal ground surface behind wall: Values of K_a (horizontal component)

A 1

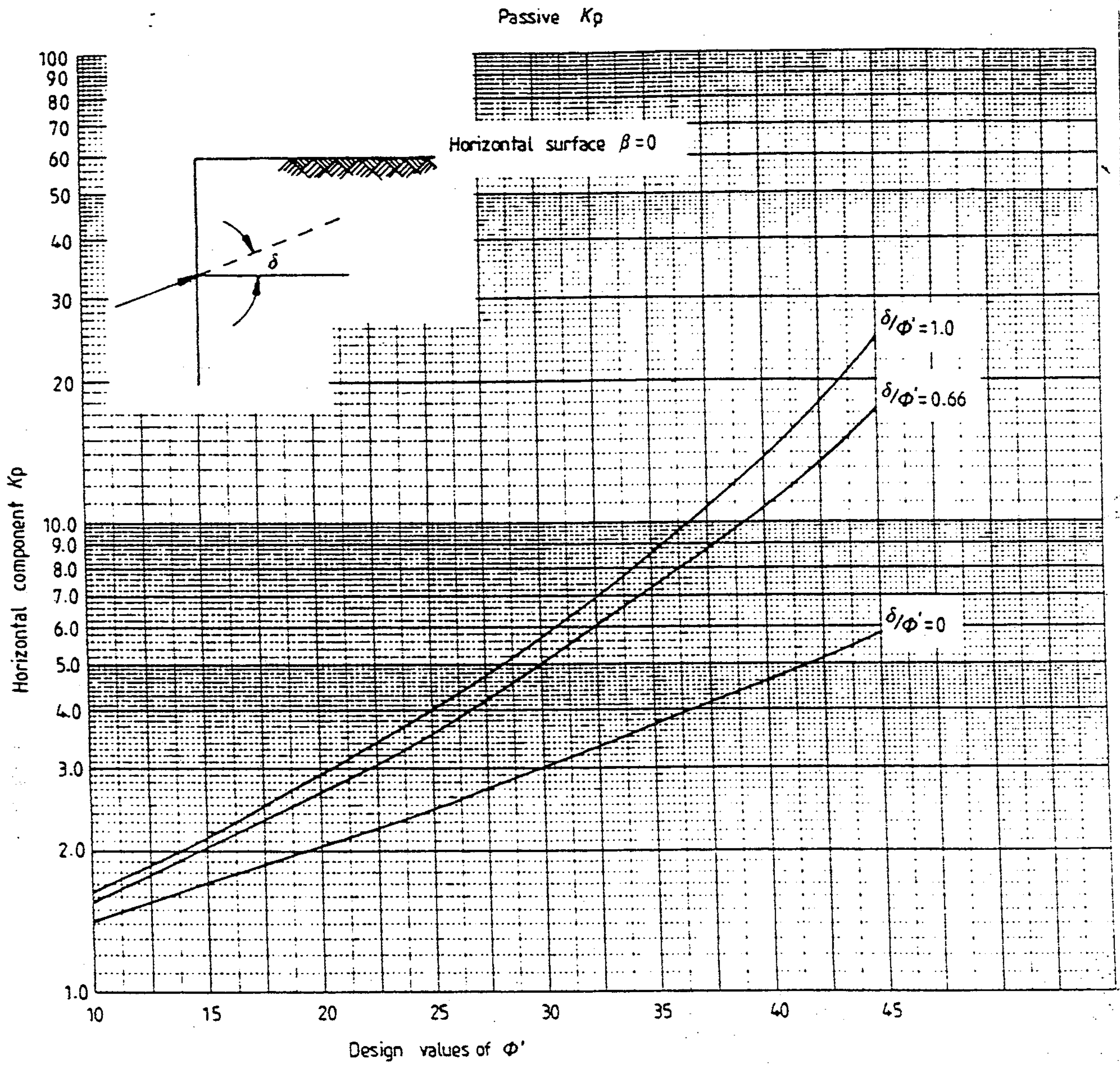


Figure A.2 Passive resistance — Horizontal ground surface behind wall: Values of K_p (horizontal component)

A 2

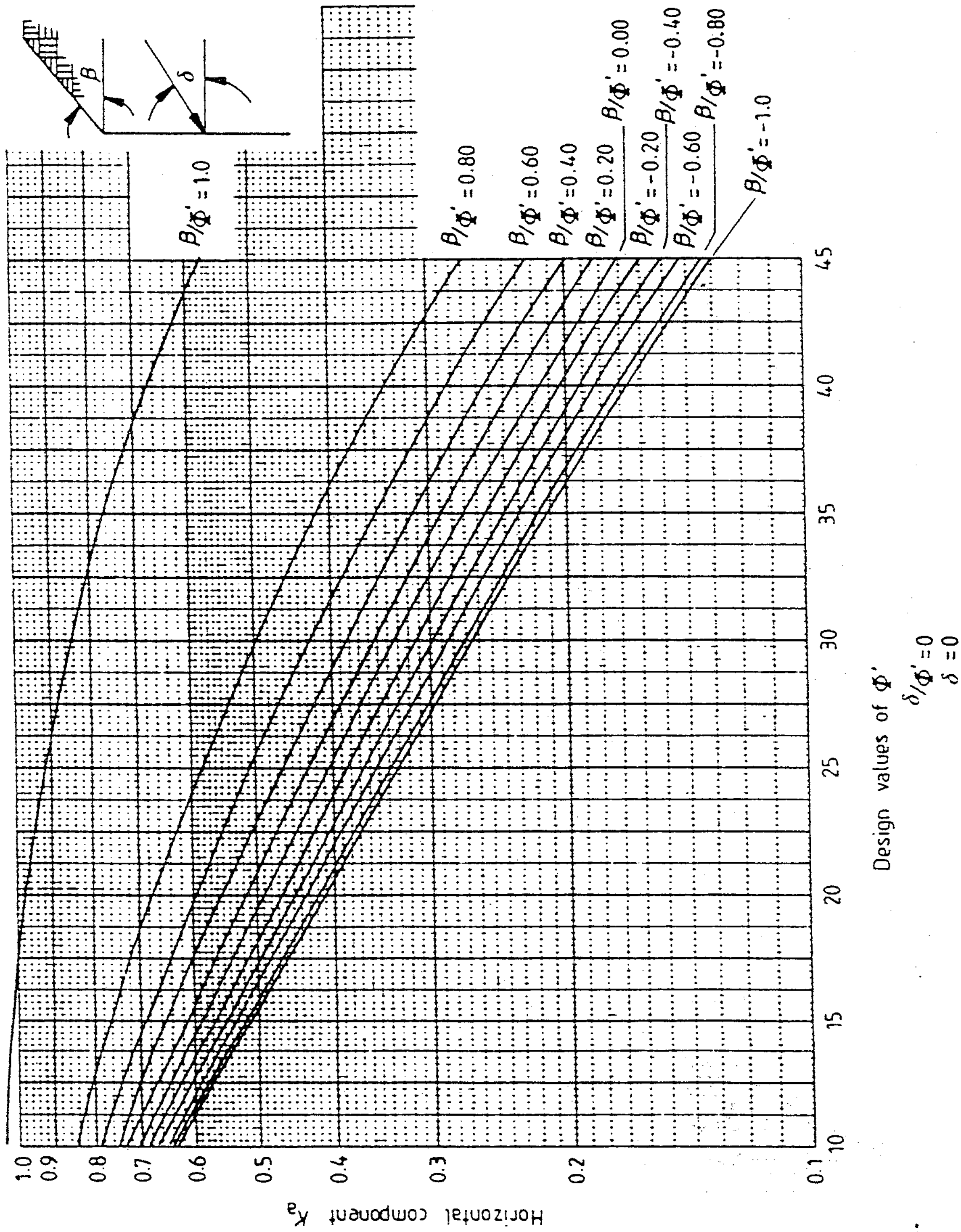


Figure A.3 Active pressure — Sloping ground surface behind wall: Values of K_a (horizontal component) (based on Kerisel and Absi, 1990)

A3

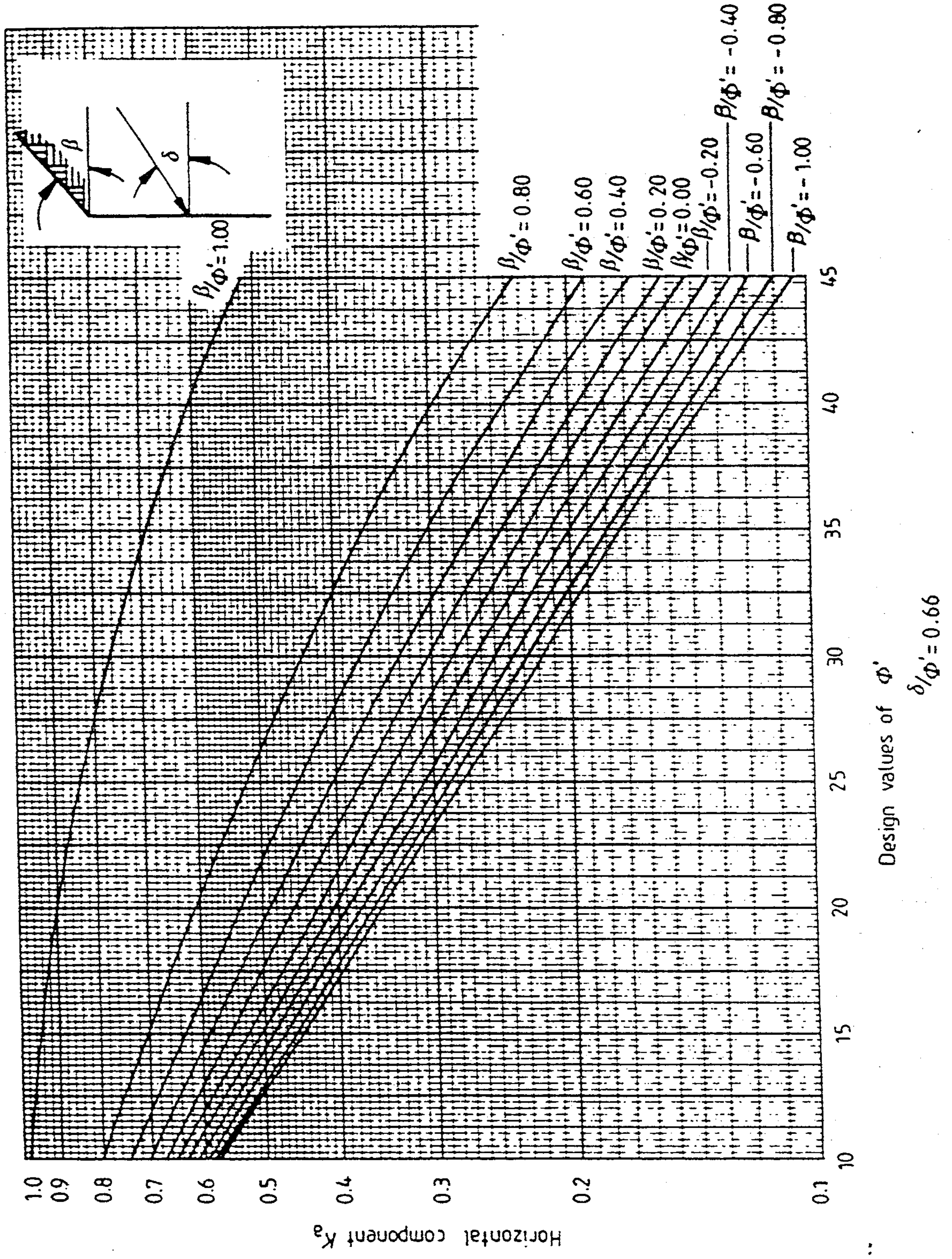


Figure A.4 Active pressure — Sloping ground surface behind wall: Values of K_a (horizontal component) (based on Kerisel and Absi, 1990)

A 4.

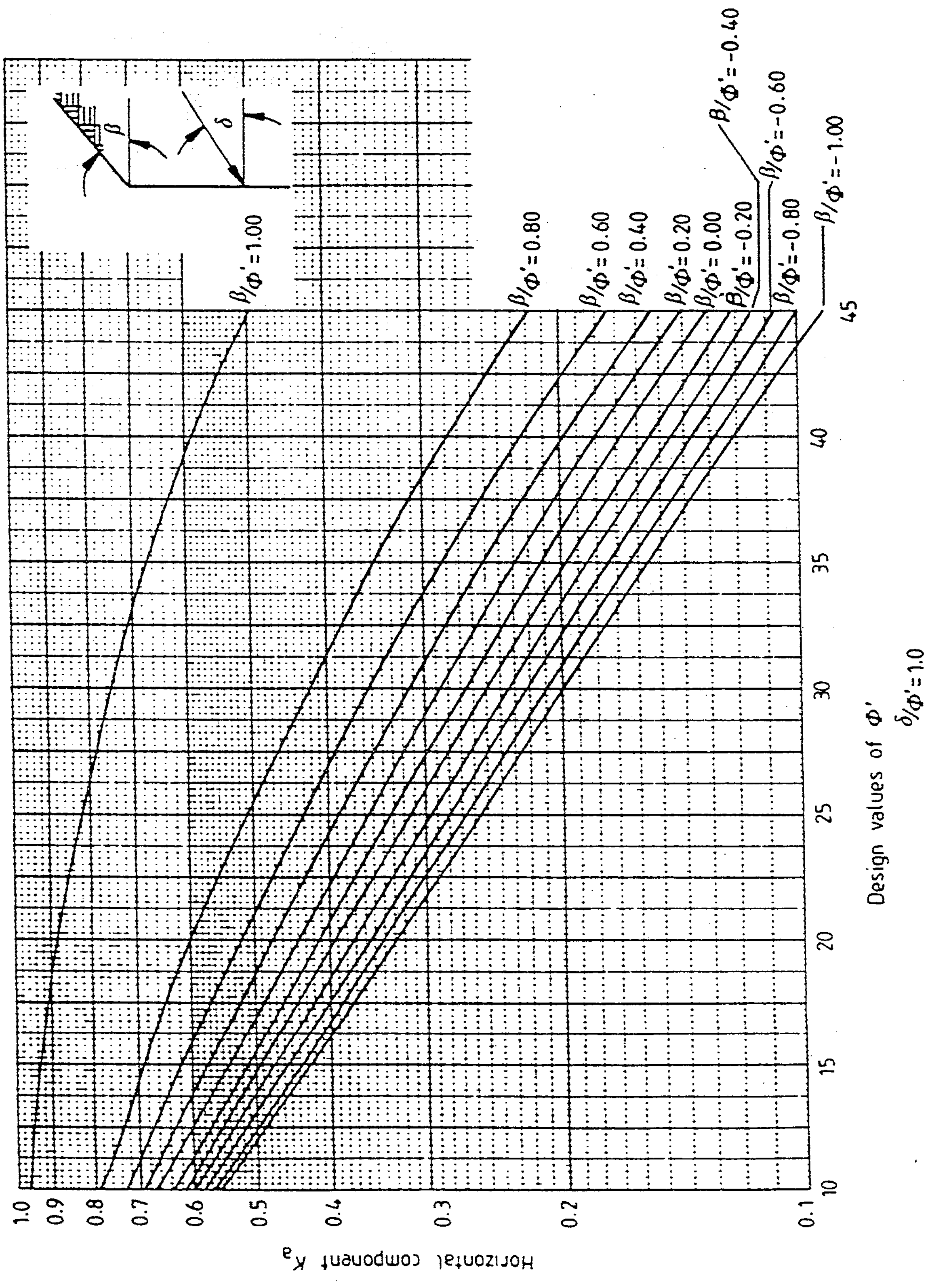


Figure A.5 Active pressure — Sloping ground surface behind wall: Values of K_a (horizontal component) (based on Kerisel and Absi, 1990)

A5

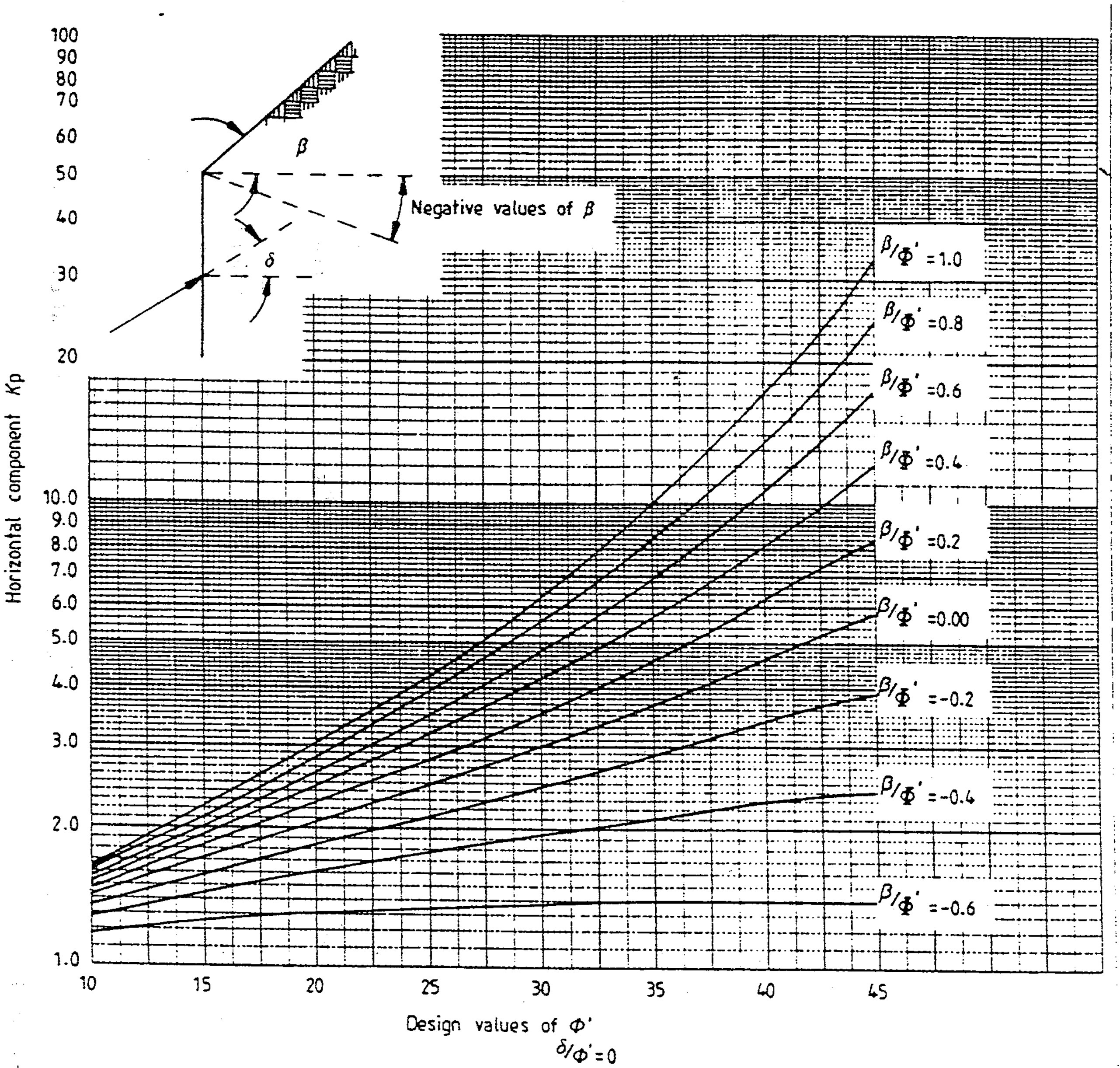


Figure A.6 Passive resistance — Sloping ground surface behind wall: Values of K_p (horizontal component)

A6

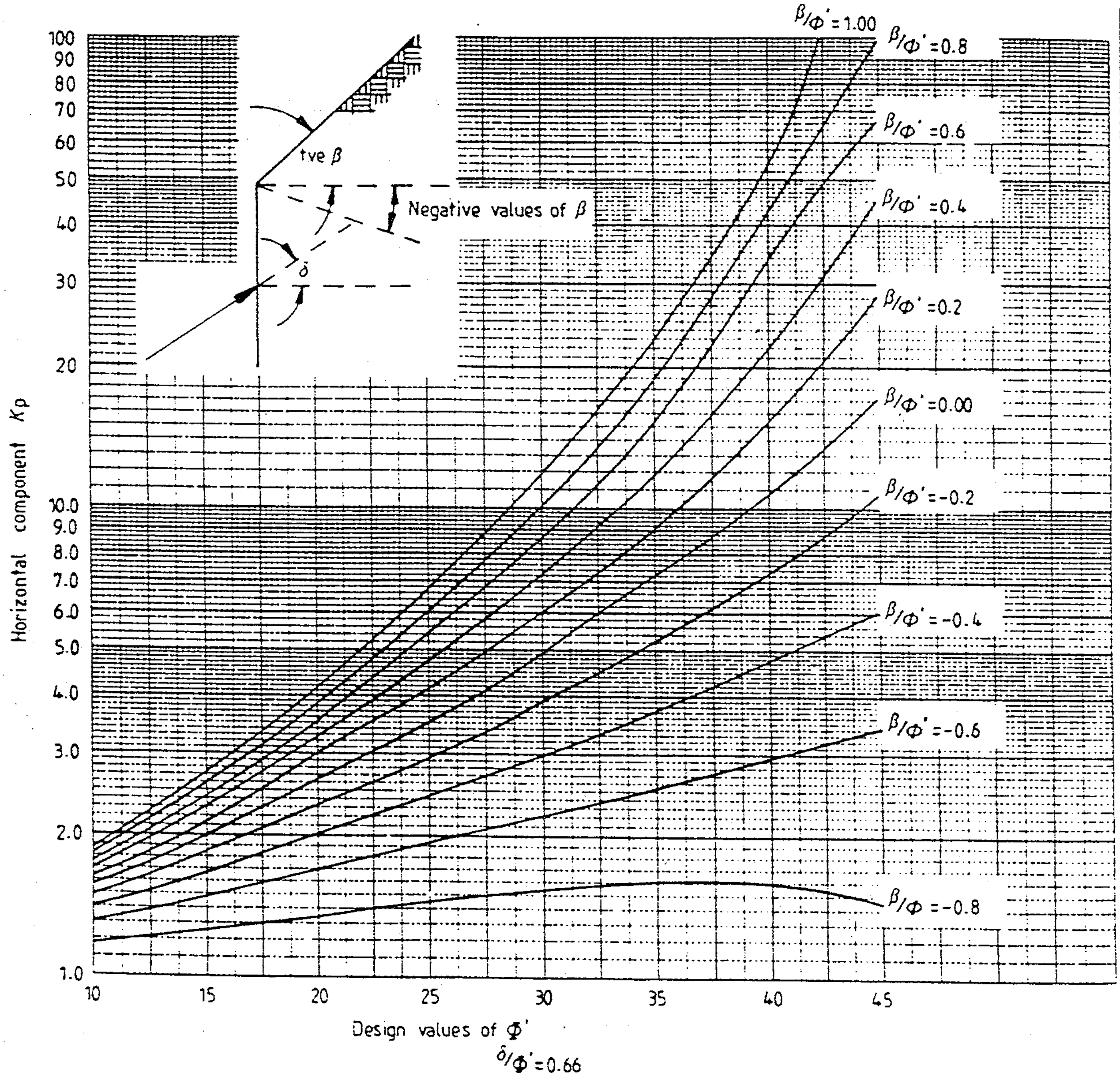


Figure A.7 Passive resistance — Sloping ground surface behind wall: Values of K_p (horizontal component)

A 7

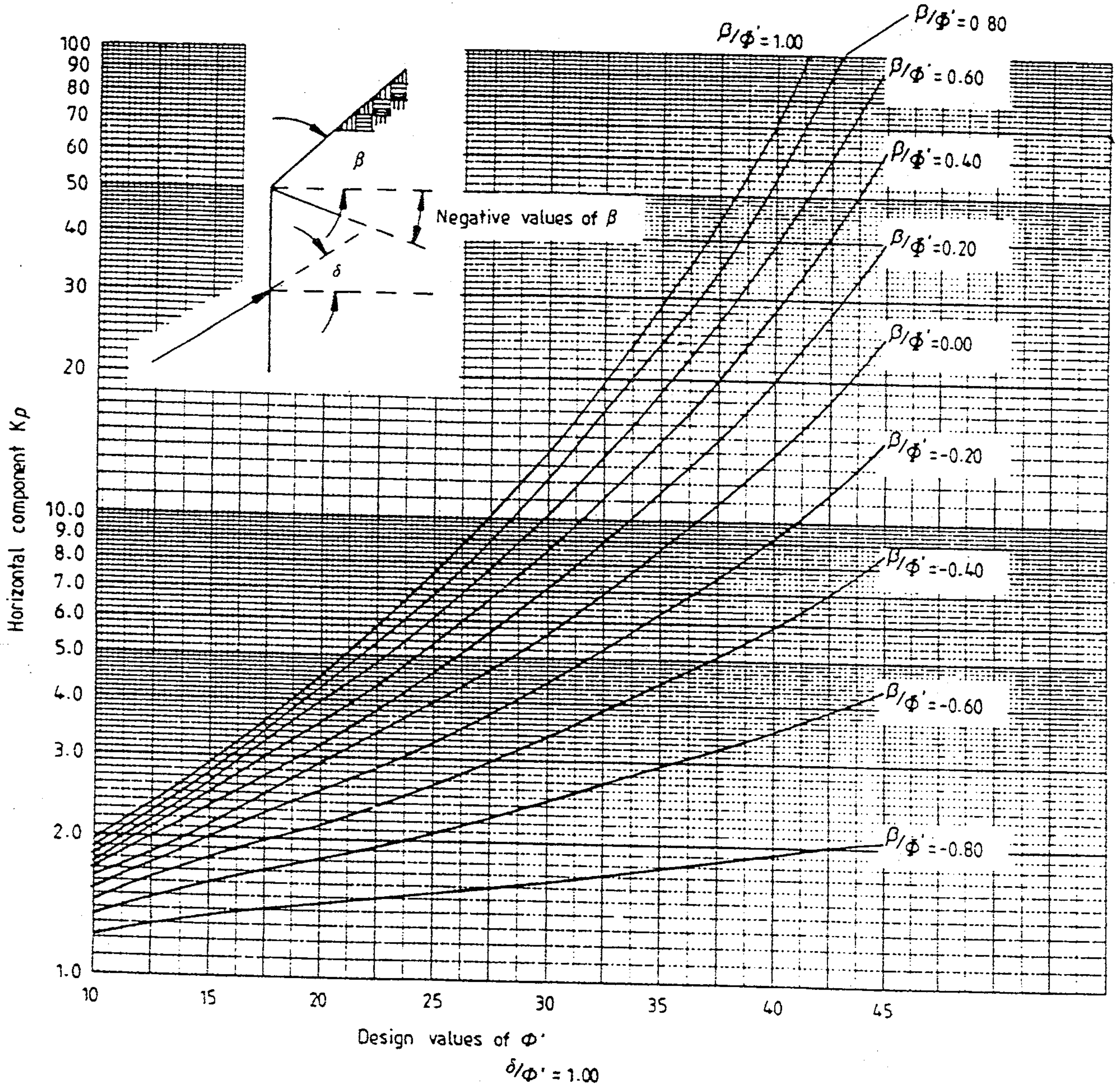


Figure A.8 Passive resistance — Sloping ground surface behind wall: Values of K_p (horizontal component)

A8

APPENDIX – B

A comparison between the design methods for earth retaining structures recommended by BS 8002:1994 and previously used methods

Proc. Instn
Civ. Engrs
Geotech. Engng,
1996, 119, Jan.,
29–34

Ground Board

Geotechnical
Engineering Panel
Paper 10939

Written discussion
closes 15 March 1996

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In May 1994 the new Code of Practice for Earth Retaining Structures BS 8002[1] was published and replaced the original Code of Practice CP2 published in the 1950s. Revision of this Code to include improved knowledge of the performance of soils and methods of analysis had been long awaited. In 1984 CIRIA report 104 on the design of embedded retaining structures in stiff clays had provided designers with guidance at an interim stage. This Paper makes a brief comparison between the scope of each Code, BS 8002 and CP2, and makes a critical review of the methods of earth pressure calculation and wall design recommended by each of them.

The revision of CP2

The preface to CP2 ended by stating that the Code would be revised as and when found necessary. While many would agree that CP2 had been a practical and useful document, particularly for retaining wall design, it was also held by many to be long overdue for replacement when draft copies for comment were circulated more than fifteen years ago. The slow acceptance of limit state design in foundation engineering with prescribed partial factors of safety may have been responsible for some of this delay.

2. The scope of CP2 was clear; the initial paragraphs stated that the Code dealt with design, construction and maintenance of all types of structure required to retain soils at a slope steeper than that which they naturally assume, or to protect soil banks against destructive agencies. The first chapter of the Code dealt with matters which affected all retaining structures in both soils and other retained materials other than liquids and included the estimation of forces in surrounding soil (or material) which threaten or promote stability, the various types of structure, the reasons for their choice and details of design, construction and maintenance to all types. Subsequent chapters dealt in turn with various types of structure, gravity walls, reinforced concrete walls, sheet pile walls, cribwork, revetments and walls subject to wave action. Appendices contained worked examples and descriptions of wall failures. Although the value of each section may have varied according to individual need and experience the reasons for the long delay in publishing even the first revision becomes more clear when the breadth of this contents list is considered. The descriptive text of CP2 contained good engineering common sense which prudently applied to either temporary or permanent works produced safe and economical earth retaining structures.

The scope of BS 8002 compared to CP2

3. The need to revise CP2 may have stemmed from new attitudes to estimating earth pressures on retaining structures, their change with time and their dependence on the relative stiffness of wall, wall support and soil. One would have expected therefore that the calculation of such pressures and their distribution would have been covered comprehensively in the new Code BS 8002. In some aspects this may be so; the foreword to BS 8002 promises a complete revision to CP2 and states that the main changes in design in the new Code are

(*a*) the recognition that effective stress analysis is the main basis for the assessment of earth pressures although total (undrained) stress analysis remains important for some walls during or immediately following construction

(*b*) the need to take account of movement (or lack of it) upon the resulting earth pressures on the wall. The largest earth pressures which act on a retaining wall occur during working conditions. These earth pressures do not increase if the wall deforms sufficiently to approach failure conditions. BS 8002 therefore takes into account the difference in retained soil strength mobilized by actual wall movement and that peak strength measured in a conventional triaxial test and further that when very large strains occur in the soil, these soil strengths

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may reduce to the residual shear strength value.

So far, so good. Even more promise is evident when it seems that the new Code incorporates limit state design philosophy for the design of the structural elements of the wall and interface is made with the relevant structural material codes. Compatibility begins to emerge between calculated maximum earth pressures at serviceability state and reduced earth pressures at ultimate limit state and the design of structural elements at these limit states in accordance with structural material codes particularly in concrete and steel.

4. BS 8002, however, does not attempt to specify the use of partial factors of safety applied to loads to take account of error in their estimation and substitutes a 'mobilization factor' to peak soil strength to approximate to the soil strength at the strain achieved in working conditions. No further partial factor of safety is applied prior to the design of the structural elements of the wall at ultimate limit state using ultimate design stresses for the element materials. The reduction in soil pressures on the wall from serviceability to ultimate limit state is considered sufficient to compensate for use of ultimate design strengths for materials. This ultimate design strength is specified in the material codes as the material characteristic strength reduced by a partial safety factor for materials γ_m and takes account of differences between actual and laboratory values, local weaknesses and (as quoted in BS 8110) the importance of the limit state being considered. The new code BS 8002 therefore uses a mobilization factor on soil strength, increasing load on the active side of the wall and reducing soil resistance on the passive side of the wall, but recommends that no further partial factors of safety are necessary prior to the design of structural members for the wall and its support using design strengths based on characteristic values reduced only by the partial factor for materials γ_m.

5. In a further innovation BS 8002 introduces a critical state approach for soil strength, that representative values should be assessed separately for peak strength and for the critical state strength of the soil. Means of estimating critical state values of angle of shearing resistance for clays and gravels are given in the Code.

6. BS 8002 also departs significantly from the scope of the previous Code CP2 by restricting its application to small and medium walls with a retained height of 8 m. In practice this proviso excludes its use for the design of many temporary cofferdams and braced walls which frequently exceed 8 m depth in civil engineering works. Although BS 8002 states that many of its recommendations are more generally applicable, the limit of retained height of 8 m must prove restrictive in the Code's use and the inclusion of design recommendations for braced walls, cellular cofferdams and double-wall cofferdams, all of which are typically of greater height than 8 m, appears contradictory and confusing. If the intention of the Code had been to address both the design of permanent walls such as reinforced concrete and gravity walls up to 8 m height and the design of temporary works to much greater depth, it should have clearly said so.

Wall design methods recommended by BS 8002 and CP2

7. The differences between the design methods for walls recommended by each code, BS 8002 and CP2, are significant, albeit that the principles described in BS 8002 apparently only completely apply to walls of 8 m height or so. In summary, the differences are as follows.

CP2: 1951

8. *Earth pressure calculation.* The basis is to apply limit active and passive pressures based on a total stress concept. No recognition is made of the reduction made to peak soil shear strengths by restriction of soil deformation in working conditions. The design values of limit active and passive pressure are therefore unconservative. Wall friction values are recommended related to wall material; concrete and brick, steel piling coated and uncoated. Wall adhesion values are recommended as a proportion of undrained shear strength. Earth pressures for cohesive soils are categorized under non-fissured clays ($\phi = 0$), silts and partially saturated clays ($\phi > 0$) and stiff fissured clays. Softening effects are discussed and for both non-fissured clays and stiff fissured clays design is based on $\phi = 0$ methods with values of c_u allowing for softening with time.

9. Tension crack depths in cohesive soils are defined as the calculated depth of negative pressure from the ground surface and use of negative earth pressure is not advised; use of the pressure from a water-filled crack is recommended, either to the calculated depths of negative pressure or half the wall height, whichever is less. Values of ranges for K_o are given for sands (0·4–0·6) and in normally consolidated clays at appreciable depth, of the order of 1·0. In the case of stiffer clays the value of total earth pressure is specified as not less than the horizontal pressure from a fluid with a density of 5 kN/m^3.

BS 8002: 1994

10. *Earth pressure calculation.* BS 8002 recommends that effective stress methods should be used to assess soil and water pressure on walls in the long term and can be used to assess short-term conditions in clays and

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soils of low permeability provided the change in pore pressures developed during construction are known. Total stress methods are alternatively recommended to assess short-term conditions in soft clays.

11. As previously described, BS 8002 relies on limit state design methods for retaining walls. The Code avoids the application of partial factors of safety to applied loads (or earth and water pressures). Instead soil strengths are factored by the introduction of mobilization factors to representative values of peak soil strengths in order to obtain a design soil strength which is typical of that soil strength likely to occur in working conditions of serviceability limit state. BS 8002 advises the value of mobilization factor M not less than 1·5 on the undrained strength of clays in total stress analysis and M not less than 1·2 on the drained strength ($c' + \sigma' \tan \phi'$) of any soil in an effective stress analysis. While M may resemble a partial factor its introduction here is linked to soil strain which is likely to occur and not to uncertainty in strength. The Code advises that no further factor of safety is necessary on loads and moments for design of wall members at ultimate limit state.

12. The Code gives recommended values for wall friction and wall adhesion in the absence of laboratory test results. The recommended values are proportions of design soil strengths (with the mobilization factor M applied to representative peak soil strengths), but are not related to wall construction materials.

13. Regarding tension cracks, BS 8002 recommends that in a retained clay soil the wall should be designed to withstand full hydrostatic water pressure from ground surface to base of the crack to a depth where the total soil pressure exceeds the possible water pressure. The Code proposes that this check should be applied to all clays using undrained clay strengths and hard clays and soft rocks should be similarly checked using drained soil strengths. BS 8002 most importantly advises on methods of assessing seepage flow under the wall where a difference in water pressures is likely to exist on opposite sides of the wall.

CP2: 1951

14. *Distribution of earth pressure.* For active pressure on walls, CP2 recommended that for gravity and cantilever walls a Coulomb distribution should be used while for strutted excavations a trapezoidal pressure distribution after Terzaghi should be used. CP2 fails to make clear that the trapezoidal diagram is in fact a strut load envelope and applies only to strut design. The envelope is shown in Fig. 1.

15. Passive pressure on vertical walls with horizontal ground surface in front of the wall is recommended to be based on Coulomb values, with linear increase in pressure with depth from the ground surface for all walls.

BS 8002: 1994

16. *Distribution of earth pressure.* BS 8002 advises the use of earth pressure diagrams for homogeneous and layered soils calculated from a Coulomb distribution, but using design soil strengths incorporating a mobilization factor. For embedded walls an annex is included with the Code which quotes CIRIA report 104 (Padfield and Mair, 1984).[3] This report had reviewed available methods for design of cantilever and single propped or anchored walls and the earth pressure distributions to be used in each method.

17. The Code recommends that where the retaining wall is supported by a multi-stage support system the strut loads may be derived from trapezoidal distribution of active pressure (after Terzaghi and Peck (1967) *et seq.*) as shown in Fig. 2. BS 8002 states that this diagram should not be used to calculate wall or sheeting bending moments which should be derived from earth pressure diagrams as reproduced in Fig. 3.

CP2: 1951

18. *Design of wall elements and support.* CP2 uses permissible stress methods of design for both temporary and permanent walls; only in 1972 was the principle of limit state design introduced to structural design. Factors of safety were applied, therefore, to ultimate material strengths to produce permissible stress values for design. Nevertheless this did not prevent designers from applying a further factor of safety to calculated loads, moments and shears and indeed CP2 suggested incorporation of these additional factors in the trapezoidal load envelope shown in Fig. 1 where the recommended pressure values contained an inbuilt increase of 44% in calculated active thrust. Other recommended increases were made in calculated values of anchor loads in anchored sheet pile walls.

19. The purpose of these increases was to

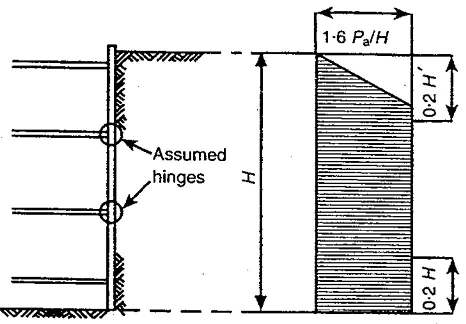

Fig. 1. Conventional pressure distribution on strutted excavation (after Terzaghi)

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reduce failure risk due to choice of unconservative soil parameters, to allow greater safety in members such as struts and bracing which were known to be a principal cause of overall failure; to allow for inequality of loading between individual members, to reduce the risk of progressive failure between frames of strutting and to reduce the risk of failure of occasional overload. The application of further factors of safety to calculated values in turn had the advantage of reducing soil and wall movement, a desirable matter for designers even though its accurate prediction was not possible at that time.

BS 8002: 1994

20. *Design of wall elements and support.* The earth pressures determined in accordance with BS 8002 are those presumed to be the most severe that will occur on the wall and accordingly the application of partial factors to bending moments, shears and thrusts derived from these earth pressures is not required. Where calculated bending stresses in sheet piling have been reduced to take into account sheeting flexibility the Code recommends that calculated waling loads, ties and strut forces should be increased by 25%.

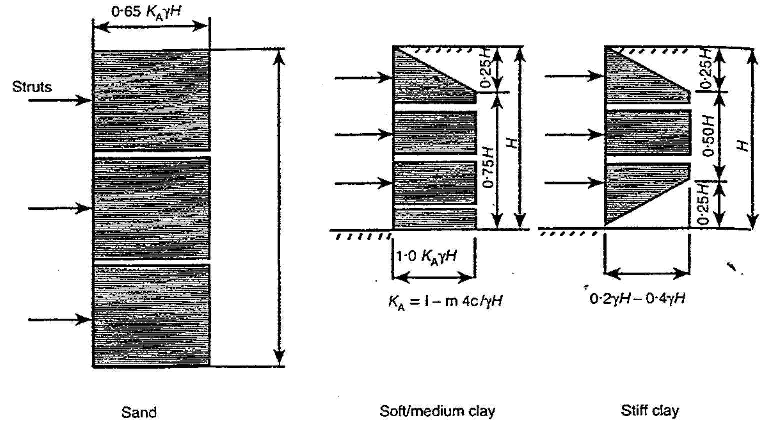

Fig. 2. Active pressure diagrams relating to maximum strut loads in braced earth retaining structures. (Source: Terzaghi, Soil mechanics in engineering practice, 2nd ed., 1967. Reprinted by permission of Wiley, New York)

21. BS 8002 requires that an obligatory design surcharge of 10 kN/m^2 is applied to the surface of the retained soil in the design of all retaining walls. The Code also requires that the design allows for a maximum extra depth of excavation of not less than 0·5 m and not less than 10% of the total height retained for

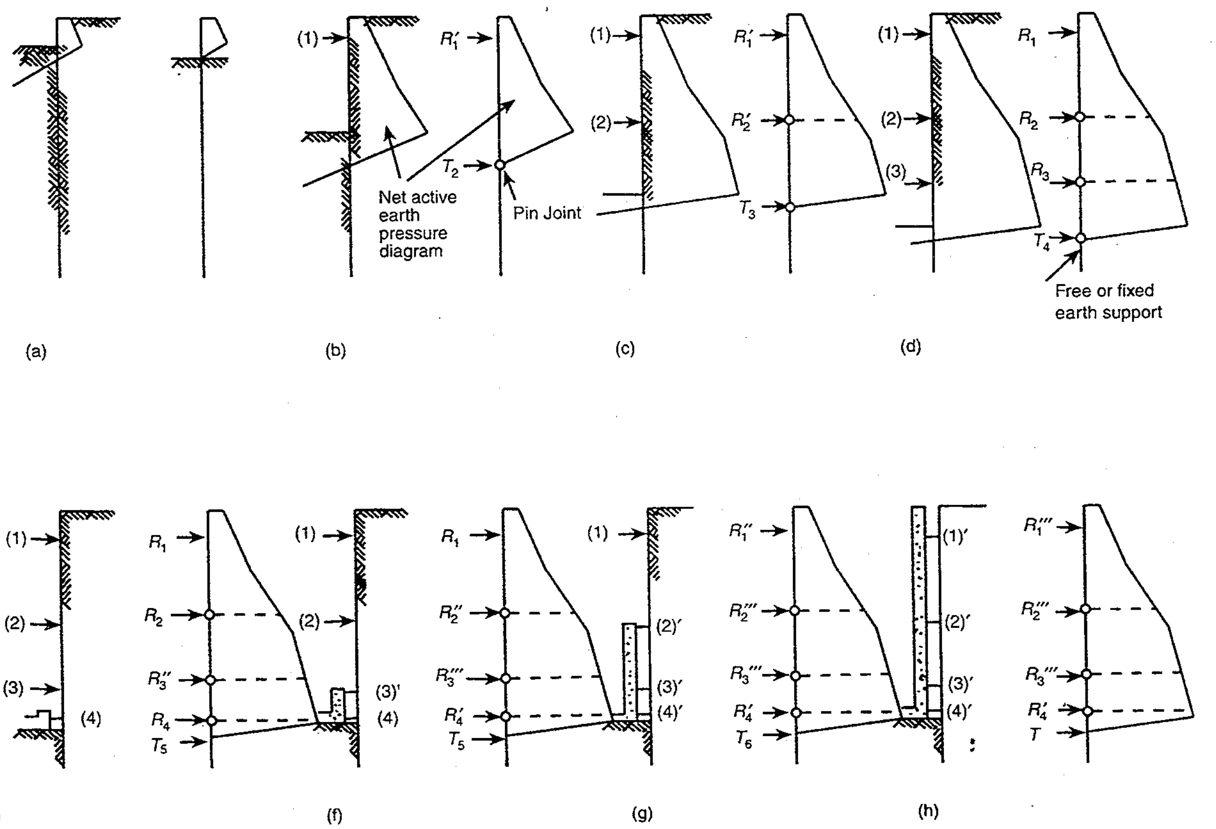

Fig. 3. Calculation method for braced excavations (Fig. 38 in BS 8002): (a) excavation for frame (1); (b) frame installed, excavation for frame (2); (c) frames (1) and (2) installed, excavation for frame (3); (d) frames (1), (2) and (3) installed, final excavation; (e) construction begins, frame (4) installed; (f) construction continues frame (3)' installed, frame (3) removed; (g) construction continues, frame (2)' installed, frame (2) removed; (h) construction continues, frame (1)' installed, frame (1) removed

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cantilever walls or of the height retained below the lowest support level for propped or anchored walls. This additional excavation depth is required for checking the stable equilibrium and soil deformation of all walls and according to BS 8002 makes allowance for unforeseen and accidental events.

Conclusions of the comparison of BS 8002 with previous practice

22. The conclusions of this Paper are as below.

(*a*) BS 8002 states that it is primarily applicable for walls down to 8 m depth but many of its recommendations are more generally applicable. In fact several of its recommendations, for example those applying to double-wall and cellular cofferdams principally refer to walls deeper than 8 m and in the case of multi-braced cuts many walls exceed 8 m depth. This restriction in scope of BS 8002 was not included in the Code which it replaces, CP2, and does not appear to be consistent in the new Code.

(*b*) BS 8002 does not define any difference between factors of safety required for permanent and temporary works since mobilization factors and hence safety factors are similar for both.

(*c*) The use of a mobilization factor to reduce peak shear strengths of soils to be compatible with the mobilized shear strength at the deformation which occurs within the soil at serviceability limit state as defined in BS 8002 may be logical in principle, but fails in application. This shortcoming applies to both rigid and flexible walls. For example, the Code minimum values of mobilization factor fail to take into account that the extent of strain needed to mobilize peak stress varies with soils. A stiff clay, therefore, would need only a mobilization of approximately 1·0 to mobilize peak stress whereas BS 8002 requires a value of 1·2 to be applied to representative $\tan \phi'_{max}$ and to representative c'. As another example, a constant specified value of mobilization factor is applied to representative strength irrespective of the extent of strain at any particular depth and since the deformed shape of the wall allows changes in strain with depth this assumption of constant mobilization factor is not correct. On the other hand rigid walls, for example gravity walls built from masonry, are relatively brittle and ultimate limit state would occur without passing through sufficient strain to exhibit a serviceability stage. The use of serviceability earth pressures for such walls as recommended by BS 8002 may prove unconservative. In a sense therefore the new Code is too prescriptive. While on the one hand it is desirable to set clear rules for the designer, it is not acceptable to achieve either a standard value for the mobilization factor by grouping together soils and walls of different deformation characteristics nor to presume a constant value of mobilization factor with wall depth either in homogeneous or layered soils. And yet on the other hand, if freedom of choice is given to vary the mobilization factor, the uniform avoidance of partial factors to bending moments, shears and thrusts in design of wall members loses credibility.

(*d*) The introduction of critical state values of soil shear strength in retaining wall design may prove constructive for future development of the Code presumably in parallel with its European counterpart in the short term. To civil and structural engineer designers, however, it may present added technical difficulty in the choice of accurate design parameters.

(*e*) The statement in BS 8002 that the Code is based on ultimate state design is not strictly correct. Using the Code method, the design earth pressures represent those that occur at serviceability state and these are subsequently used to represent pressures at ultimate limit state as the wall moves and collapses. An added proportion of ultimate earth pressure is included in design pressures at the ultimate stage to allow for a safety margin unconnected with the relationship of wall deformation and earth pressure. BS 8002 uses serviceability state earth pressures for design at ultimate limit state; the design method misrepresents actual applied loads at failure, the essence of ultimate limit state design.

(*f*) In terms of calculation method, the specified mobilization factor appears incompatible with the use of beam spring programs for cantilever, single propped and braced walls since the balanced earth pressures on active and passive sides as calculated from the spring system would not necessarily mobilize that proportion of peak soil strength required, as a constant value, as defined by the Code mobilization factor.

(*g*) BS 8002 does not allow the application of different factors of safety for varying risk on the design of structural components of a temporary wall. Whereas risk of strut collapse or passive failure is known to pose a greater risk of failure than flexural failure of the wall for embedded and braced walls, the application of a mobilization factor to soil strength does not recognize this. This omission presents a serious restriction to the designer in applying his practical knowledge of the likely cause of failure of cofferdams and other deep excavations and his use of measures to avoid such failure.

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(*h*) The accuracy of the assumption that the calculated depth of tension crack in cohesive soils is filled with water in both short- and long-term design periods is a critical influence in the final dimensions of structural members of many retaining structures, particularly those retaining stiff clays. There is a significant difference between the requirements of BS 8002 and CP2 in this regard; whereas BS 8002 requires water pressure within the crack to the full depth to where total soil pressure exceeds the possible water pressure, CP2 requires water pressure to the calculated depth of the tension zone or half the total wall height, whichever is less. In addition, CP2 retained a further safety measure for sheet pile walls, not included in BS 8002, that however low calculated active pressure may be, the lowest value to be used in design for the total horizontal thrust due to the soil should be that by assuming the horizontal pressure at any depth to be that due to a fluid with a density of 5 kN/m^3.

(*i*) The calculation of wall friction and adhesion as described in BS 8002 refers to proportions of peak soil strength of cohesive and cohensionless soils developed in wall friction and adhesion without reference to the need to obtain vertical relative movement to mobilize such resistance nor the extent of such movement. Design values of wall friction and adhesion are given without reference to the wall construction material. A capping value for wall adhesion in BS 8002 would appear appropriate in a similar way to that specified in CP2[2] and CIRIA Report 104.[3] An empirical approach on this matter appears highly desirable for each particular job both in terms of the probable extent of movement that would occur to mobilize wall friction or adhesion and its extent depending upon the wall material. Proportions of peak soil strength estimated from other considerations appear inappropriate.

(*j*) The obligatory use of surcharge loading of 10 kPa and a minimum overdig allowance to excavation in the front of earth retaining structures of 0·5 m as required by BS 8002 appears unrelated to practical conditions and presumes that the designer is unaware of standards of site supervision and risk of uncontrolled excavation.

(*k*) The publication of BS 8002 was made only a short time before the circulation of the final version of the European prestandard for Geotechnical Design Eurocode 7 in August 1994. This prestandard has an initial life of three years and presumably will then be amended after comment to form a standard document. Although the European and National Standards will run in parallel for some while, BS 8002 may therefore have a limited life, especially when compared to its predecessor, CP2. The proposed design methods for retaining walls of the Eurocode depend on limit state methods throughout and partial factors are applied to both characteristic loads and soil and water pressures based on characteristic soil strengths. A further partial factor is applied to thrusts, moments and shears for the design of structural elements. The draft Eurocode stipulates that mobilized soil strengths shall be related to wall and soil deformation at serviceability and ultimate limit states but avoids the specification of a mobilization factor. The successful application of partial factors to earth pressures in all cases may well prove to be a matter of considerable difficulty if the two Eurocodes, EC1 Basis of Design and EC7 Geotechnical Design are to be compatible but the introduction of the mobilization factor and its recommended values is avoided.

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Proc. Instn Civ. Engrs Geotech. Engng, 1996, 119, Jan., 35–48

Ground Board

Geotechnical Engineering Panel Paper 10940

Written discussion closes 15 March 1996

Comparative studies by calculation between design methods for embedded and braced retaining walls recommended by BS 8002: 1994 and previously used methods

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A parametric study of cantilevered walls of 4 m depth to formation level and of single propped walls of 8 m depth to formation level in sand and clay is made by methods described in CIRIA Report 104[1] and compared with the results of a similar study made by methods which comply with BS 8002.[2] The results of analyses on multi-braced excavations in homogeneous sand and clay are given for various methods based on empirical procedures, Winkler Spring theory and the requirements of BS 8002. A case study of calculations made for a deep multi-braced excavation in layered soil is presented to compare strut loading, wall moment and embedment depth using methods due to BS 8002 and those used by the authors previously. The Paper concludes with a summary of the findings of the study and recommendations for further investigations.

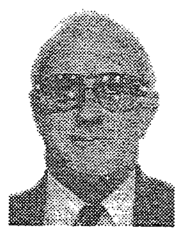
Introduction

1. In order to assess the effect of the provisions of BS 8002 on the established methods of calculation for cantilever, single propped and multi-braced earth retaining walls a series of calculations has been made, the results of which are presented in the following paragraphs. These studies are concluded by the application of both BS 8002 and previously used methods to a case study of a deep braced excavation with the occurrence of soft clays immediately below final formation level.

Cantilever walls

2. The results of comparative analysis for excavations to a depth of 4 m in dry sands and in dry sands overlying clay below final formation level are shown in Figs 1 and 2. In Fig. 1 showing excavation in dry sands, the angle of shearing resistance is varied, whereas in the Fig. 2, the dry overlying sand is maintained at constant shear strength and the undrained shear strength of the clay underlying the sand is changed in the analysis.

3. The traditional method of analysis as described in CIRIA Report 104 is used to compare wall dimensions with those calculated by BS 8002 methods. The CIRIA report analysis



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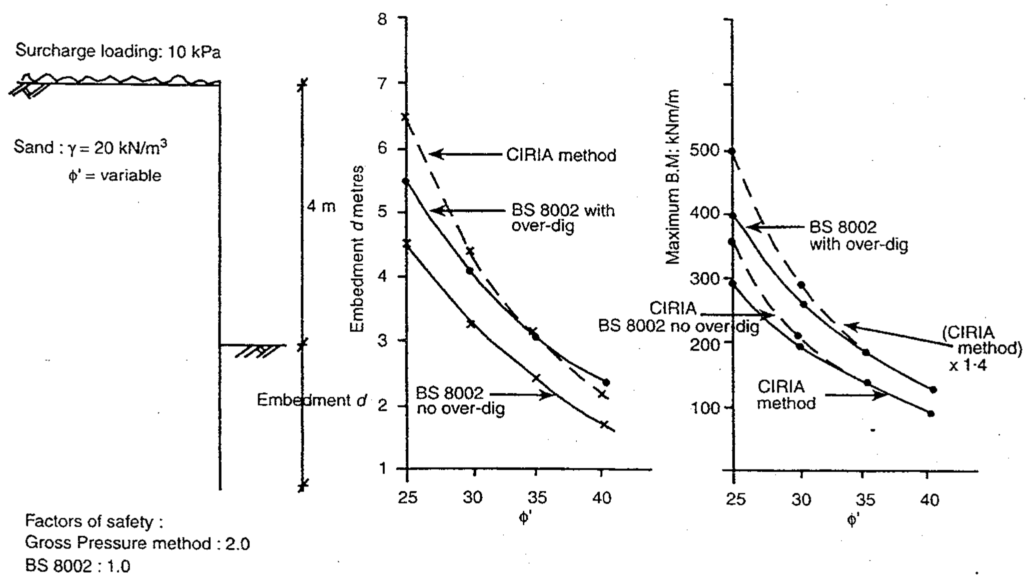

Fig. 1. Cantilever wall in dry sand: maximum B.M. and embedment

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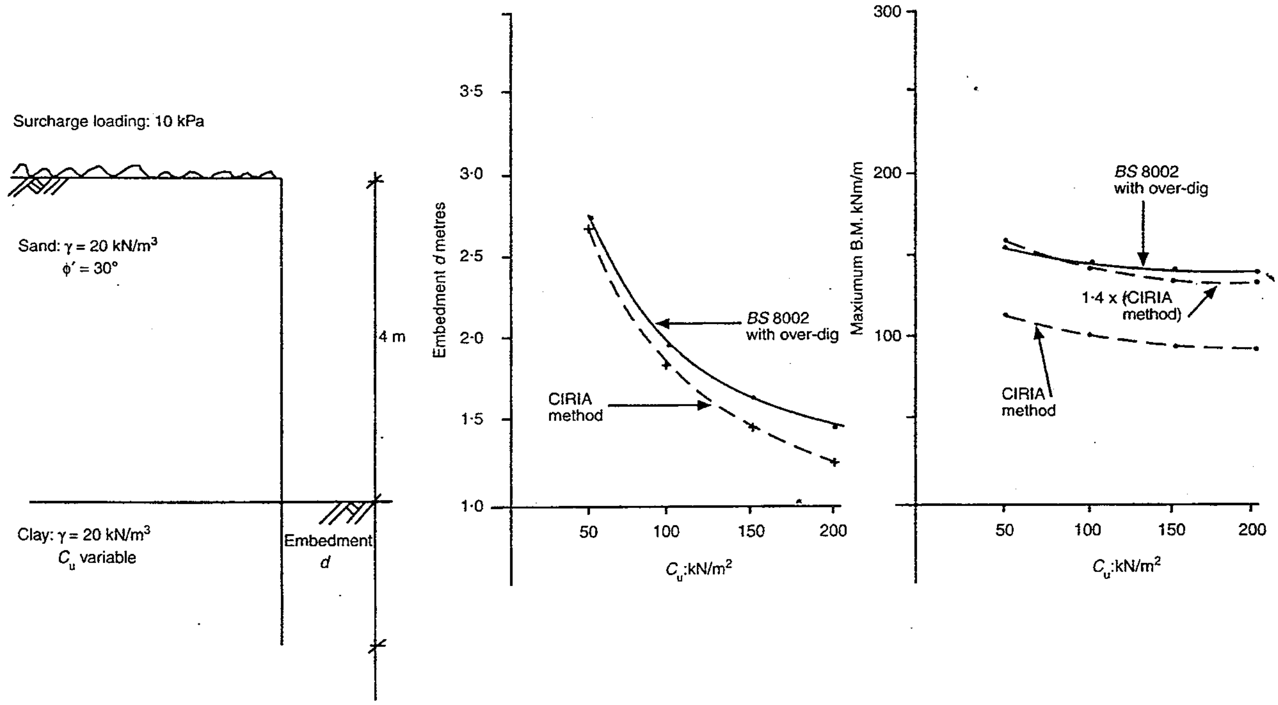

As Fig.1, factors of safety. Gross Pressure Method, CIRIA = 2·0, BS 8002 = 1·0. No water in tension crack.

Fig. 2. Cantilever wall in dry sand overlying clay: maximum B.M. and embedment

is based on the safety factor derived from the ratio of moments of active pressures and passive resistances from the gross earth pressure diagram. This method is in fact only one of several methods defined in CIRIA Report 104 for cantilever wall design, but originated from CP2[3] and is in frequent use. Other alternative methods referred to in the CIRIA report are based on factors of safety derived from increase in wall embedment, factored shear strength parameters and factored moments based on net earth pressure diagrams or areas of reduced passive pressure, and while the CIRIA report specifically addresses embedded walls in stiff clays the methods apply to both cohesive and cohesionless soils.

4. The analysis using the CIRIA report 104 method is based on a factor of safety of 2·0 with an angle of wall friction on the active side $\delta'_a = \frac{2}{3}\phi'$ and on the passive side $\delta'_p = \frac{1}{2}\phi'$ and wall adhesion = $0{\cdot}75c_u$. The analyses using the BS 8002 recommendations are based on a factor of safety of 1·0 and mobilization factors of 1·2 applied to representative values of $\tan\phi'$ and 1·5 applied to representative values of c_u. The values of active and passive wall friction used in the BS 8002 method are calculated from similar angles of friction δ'_a and δ'_p derived from $\tan\delta'_a = \tan\delta'_p = 0{\cdot}75\tan\phi_m$, where $\tan\phi_m$ = representative $\tan\phi$/mobilization factor M and wall adhesion = $0{\cdot}75c_{um}$ where $c_{um} = c_u$/mobilization factor M.

The second diagrams for each of Figs 1 and 2 include maximum bending moment values in walls as calculated by the CIRIA method factored by a factor of safety of 1·4 to obtain ultimate state design values which are compared with those values from the BS 8002 methods proposed by that Code as ultimate design values. The requirements of BS 8002 regarding minimum values of surcharge load and an over-dig to excavation depth are applied in the analyses.

5. The conclusions reached from these comparisons on cantilever walls are as follows:

> For the excavation in dry sand the embedment calculated by BS 8002 is generally less than the embedment calculated by CIRIA report 104, but the difference decreases with increasing sand strength and as ϕ' exceeds 35° the embedment due to the BS 8002 method progressively exceeds that due to the CIRIA report. The maximum bending moments in the wall calculated by BS 8002 are similar to those calculated by CIRIA report 104 except where ϕ' is less than 30° when BS 8002 values become increasingly less with reducing ϕ' than CIRIA report 104 values. For the excavation in dry sand underlain by clay at dredge level, the embedment calculated by BS 8002 is slightly higher than that calculated by CIRIA report 104, whereas the maximum bending moments calculated by each method are similar for the range of c_u investigated.

The similarity of results between factored moment values from the CIRIA report method and those from BS 8002 together with similar

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embedment depths for each method when the 0·5 m overdig requirement of BS 8002 is included is considered to be coincidental and such similarity may not necessarily occur for all wall depths or combinations of layered soils. Where either comparatively strong or weak soils occur immediately below formation level, similarity of calculated values may be difficult to find.

6. The principle of using a mobilization factor as defined in BS 8002 appears desirable but its application in the Code is flawed for a number of reasons (see Puller and Lee, 1995[4]). One of the most important of these is the presumption that a constant value of mobilization factor M can be applied with depth to retained soil irrespective of soil strength or stiffness. This assumption implies that while the cantilever wall deforms a maximum amount horizontally at the top of the wall, the constant value for M assumes incorrectly a mobilization of a constant proportion of peak shear strength from the top of the wall downwards. Where layered soils occur this assumption may cause greater error since the extent of soil deformation to achieve the development of peak shear may vary from stratum to stratum, depending upon soil type.

Single propped walls

7. Walls of constant depth to a final formation level of 8 m were analysed with a single prop at ground level, the soil profile being varied in a similar way to those for the analyses on cantilever walls. The first series of analyses was made on excavations in dry sand, the angle of shearing resistance being progressively varied from one analysis to the next, while the second series assumed a soil profile of dry sand

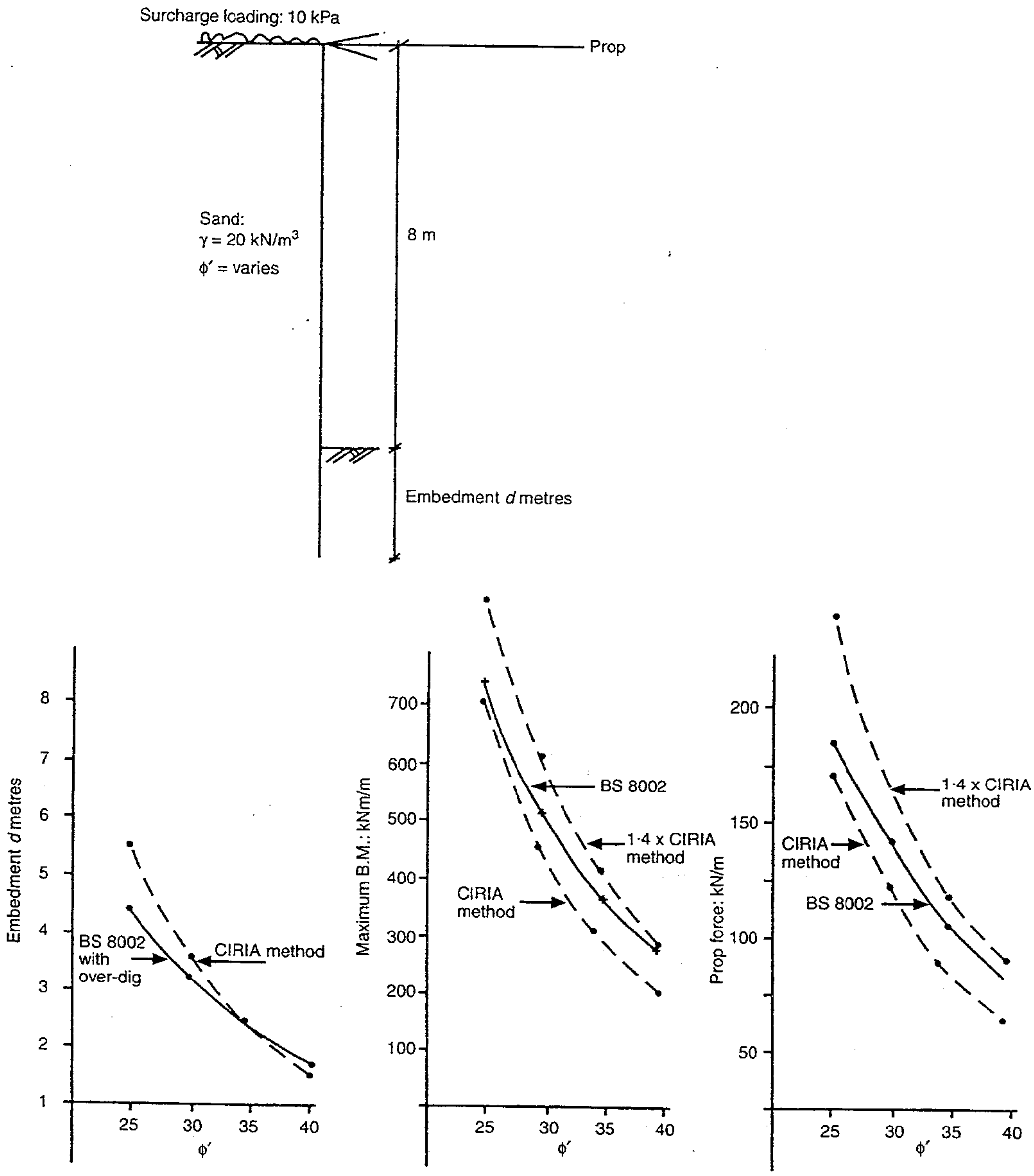

Fig. 3. Single propped wall in dry sand: maximum B.M., embedment and prop force

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of constant strength overlying clay, the undrained strength of which was varied from one analysis to the next. The values of wall friction and adhesion were as before and consistent with the requirements of BS 8002 or CIRIA Report 104. The requirements of BS 8002 regarding the minimum design values for surcharge loading and overdig for excavation depths were applied to the analyses of the single propped walls. The results of the analyses are shown in Figs 3 and 4. As before, values of maximum bending moment in the wall are also shown with a factor of safety of 1·4 applied; values of prop force are also shown similarly factored.

8. The basis of the CIRIA Report 104 method remained similar to that for cantilever walls; a factor of safety of 2·0 was maintained on the ratio of stabilizing moments to disturbing moments of the gross earth pressure diagrams using limit pressures, moments being taken about anchor level. The resulting wall embedment with the factor of safety applied achieves a wall fixity greater than that required for free earth support but not necessarily sufficient to obtain full fixity.

9. The conclusions reached from the comparative analyses are as follows:

- For the excavation in dry sand the embedment calculated by BS 8002 is less than that calculated by CIRIA Report 104 with ϕ' less than 33° but similar in sand of higher friction angle. When CIRIA report values of maximum bending moment and prop force are factored by 1·4 the values of both maximum moment

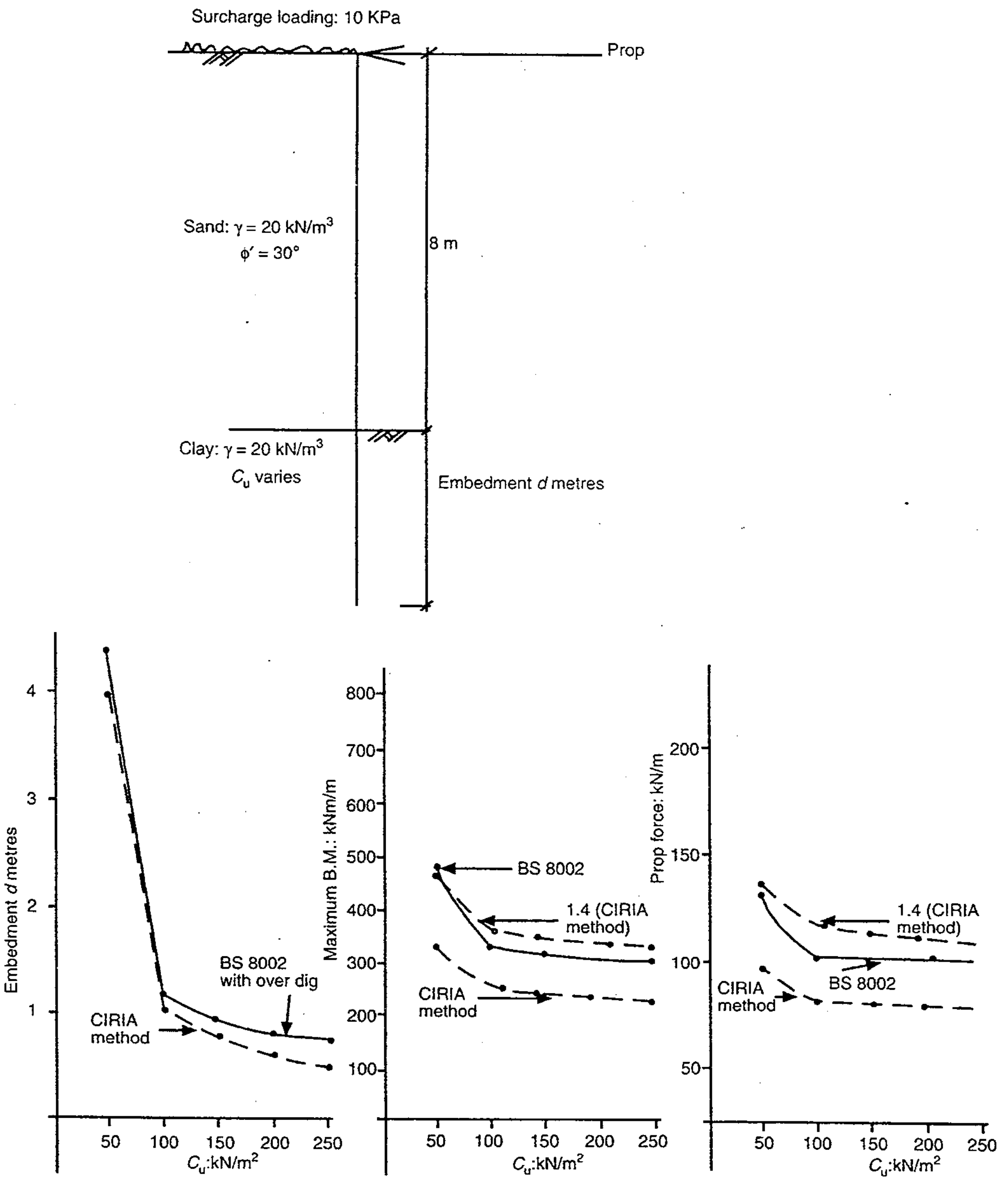

Fig. 4. Single propped wall in dry sand overlying clay; maximum B.M., embedment and prop force

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and prop force are greater than those using BS 8002 methods for all values of ϕ', although only modestly so in the range of ϕ' between 30° and 40°.

- For the excavation in dry sand underlain by clay at dredge level the embedment calculated by BS 8002 is slightly higher than that calculated by the CIRIA report method for the clay strengths in excess of 100 kN/m^2. Values of maximum bending moment and prop force calculated by the CIRIA report method and factored by 1·4 to give ultimate limit state values are only slightly higher for the range of clay strengths considered than those calculated using BS 8002.
- For the soil strengths and wall geometry analysed it appears that BS 8002 produces a slightly slimmer wall with a smaller section of prop than that calculated using the earlier CIRIA report method. These differences are small particularly in medium strength soils. They apply only for the range of soil strengths and the wall geometry analysed and do not result from logical differences between the methods but from coincidence in the application of particular mobilization factor values as a partial factor on peak soil strength, different wall friction values, an over-dig allowance of 0·5 m applied to the BS 8002 analyses, and a factor of safety of 1·4 applied to maximum bending moment and prop force for the CIRIA report method.

10. The inconsistencies which occur in the application of the mobilization factor using BS 8002 for single propped walls are similar to those for cantilever walls. The reductions to fully mobilized active and passive earth pressures with depth can be examined in a Winkler Spring analysis such as Lawall (see Wood[5]) and compared with the constant BS 8002 mobilization factor for that soil. The Winkler Spring values rarely compare well with the mobilization factor value and illustrate that the proportion of earth pressure resulting from peak shear values is dependent on wall and soil deformation at any particular depth. Because this movement is not constant with depth, a uniform mobilization factor is not appropriate; the mobilization factor should more logically vary with depth depending upon the anticipated deformed shape of the wall.

Multi-braced walls

11. The introduction of BS 8002 defines the scope of the Code to apply primarily to small and medium walls with a height of up to about 8 m. Section 4.4 of the Code, however, addresses the design of embedded walls including multi-braced walls which are often deeper than 8 m and, at times, to much greater depth. The methods recommended by the Code for calculating strut loads in multi-braced walls are the use of active and passive pressure diagrams using limit pressures and assuming pinned joints at all frame levels other than the top frame (see Puller and Lee[4]) and trapezoidal strut load envelope diagrams due to Terzaghi and Peck.[6] The Code recommends that bending moments in the wall should be calculated from the former method. Although not clearly stated, we presume that this should be done using limit earth pressure diagrams. If this is so, there is a departure in the consistency of the Code's advice; neither the earth pressure diagram nor the strut load envelope incorporates the use of mobilization factors and there is evidently some lack of fit between the initial design philosophy of the Code and the recommended methods of design for multi-braced walls.

12. In an attempt to identify a reliable method which does not contravene the requirements of BS 8002 a braced excavation was analysed in dry sand or clay of uniform strength by several methods including those referred to in the Code. The initial method used as a control was analysis by the Winkler Spring Theory using the Lawall program (see Wood[5]). This software uses an input of at rest earth pressures, based on peak soil strength parameters and then performs iterations to obtain the final balanced earth pressure values based on wall movement derived from spring stiffness obtained from soil stiffness parameters. It is the method of design for braced excavations used by both authors since 1986, and is based on similar theory to that of many programs used worldwide. More recently finite element programs have become well used for the same purpose, although in the uniform soil conditions used in this exercise the Winkler Spring Theory is considered an adequate control for comparison with the recommendations of BS 8002. As a means of avoiding gross error a check by hand method utilizing a derivative of the empirical method by Terzaghi and Peck[6] also is often used.

This derivative employs an imaginary prop to simulate the required net passive resistance below formation level and enables the calculation of a factor of safety against passive factors below final formation level as the ratio of available passive resistance, fully mobilized, to that calculated from the imaginary prop load. The method of analysis is described in Fig. 5.

13. The geometry of the wall analysed, together with the soil properties are shown in Fig. 6. The comparative methods of analysis used are as follows

(*a*) Winkler Spring Analysis using Lawall program as referred to previously. Maximum strut loads and wall bending moment derived from staged excavation. The retaining wall was assumed to be steel sheet

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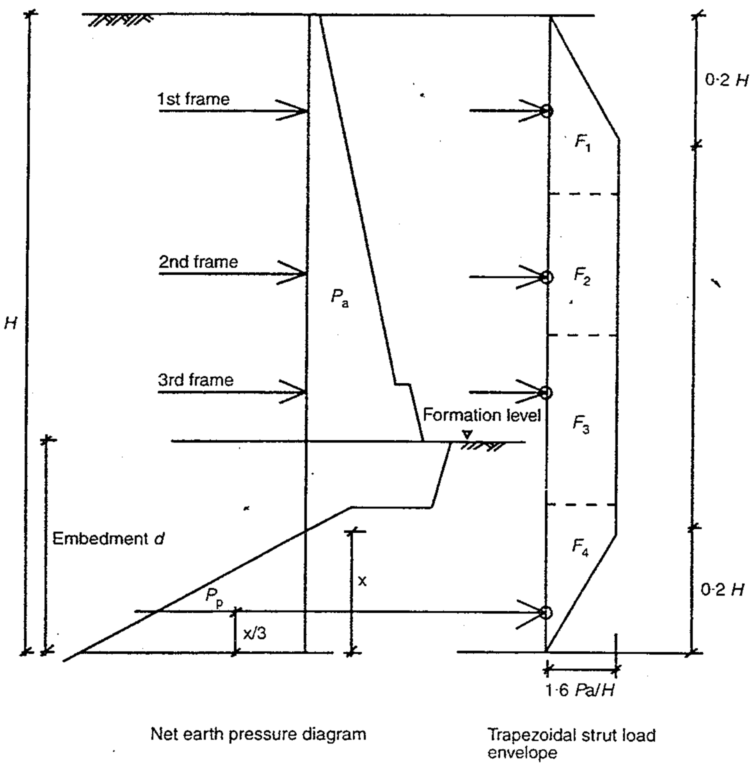
piling, Larssen 4A section, propped by a tubular steel system of strutting.
(b) Derivative of Terzaghi and Peck trapezoidal strut load envelope. Strut loads derived from full depth excavation; factor of safety against passive failure calculated.
(c) Terzaghi and Peck method strut load envelope as BS 8002; no mobilization factor nor over-dig applied. Strut loads derived for full depth excavation. No check on overall stability. Mean values of Terzaghi and Peck's strut load envelope of $0{\cdot}3\gamma H$ were assumed for the braced excavation in clay.
(d) Simplified equivalent beam calculation as shown in Fig. 38 of BS 8002 (see Fig. 3, Puller and Lee[4]). Staged excavation, strut loads and bending moments. No mobilization factor or over-dig allowance.
(e) Simplified equivalent beam calculation according to BS 8002 but incorporating mobilization factor and over-dig requirement as per BS 8002.
(f) Equivalent beam calculation using mobilization factor and over-dig requirements of BS 8002 but allowing continuity of beam over strut supports and assuming bottom support at toe of wall.

14. The conclusions reached from the analyses of the multi-braced walls shown in Table 1 are as follows:
In sand

- The distribution of Winkler Spring strut loading does not agree with values from the other analyses. The total of Winkler Spring strut loads was almost 50% higher than the average of the other methods.
- For the BS 8002 recommended methods there was generally poor agreement between strut load distribution by the trapezoidal envelope methods and the simplified equivalent beam method although the total of the strut loads showed reasonable agreement.
- The use of the equivalent beam method with $M = 1{\cdot}2$ gave higher values than the BS 8002 simplified beam approach but the agreement of strut load distribution with the trapezoidal envelope methods was poor.
- Maximum moment in the wall calculated by Winkler Spring, simplified equivalent beam, with $M = 1$ and safety factor = 1·4 were similar but moments calculated by simplified equivalent beam with $M = 1{\cdot}2$ and over-dig allowance and those by equivalent beam with continuity allowed were dissimilar to those by Winkler Spring and Lawall.

15. The authors' opinions from these findings are as follows:

(a) The Winkler Spring Analysis does not use limit pressures for earth pressures and



Steps in construction:
1. Construct net earth pressure diagram using limit pressures P_a and P_p
2. Calculate total active pressure from net diagram = P_a
3. Calculate trapezoidal strut load envelope co-ordinate = $1{\cdot}6\ P_a/H$
4. Calculate strut envelope areas F_1, F_2, F_3, F_4 (F_4 represents net passive resistance below dredge level and acts at centre of pressure of net passive diagram)
5. Calculate factors of safety, mobilized passive resistance = Net passive resistance P_p/calculated mobilized passive resistance F_4

Fig. 5. Construction of modified trapezoidal strut load envelope incorporating force due to passive resistance: 'SM method'

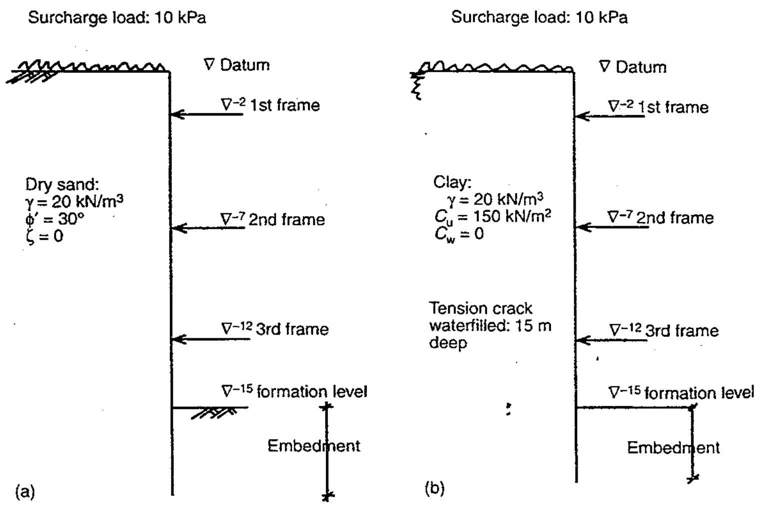


Fig. 6. Multi-braced walls: geometry and soil properties used in analysis for (a) when in sand; and (b) when in clay

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Table 1. Results of analyses of braced excavations in sand or clay

Braced excavation in sand						
	Winkler Spring Lawall Factor: 1·4 B.M. and strut	Derivative of trapezoidal envelope Factor: 1·4 B.M. and strut	Original trapezoidal envelope Terzaghi & Peck Factor: 1·4 B.M. and strut	Simplified beam BS 8002 Fig. 38 $M = 1$ Factor: 1·4 B.M. and strut	Simplified beam BS 8002 Fig. 38 $M = 1{\cdot}2$	Equivalent beam $M = 1{\cdot}2$
Prop force kN/m 1st frame	329	428	410	154	140	183
2nd frame	777	476	455	420	385	568
3rd frame	742	607	498	559	668	710
Max bending moment: kN/m	564	not calculated	not calculated	515	971	469
Embedment: m	7·3	6·5	not calculated	not calculated	not calculated	6·19
Braced excavation in clay						
	Winkler Spring Lawall Factor: 1·4 B.M. and strut	Derivative of trapezoidal envelope Factor: 1·4 B.M. and strut	Original trapezoidal envelope Terzaghi & Peck Factor: 1·4 B.M. and strut	Simplified beam BS 8002 Fig. 38 $M = 1$ Factor: 1·4 B.M. and strut	Simplified beam BS 8002 Fig. 38 $M = 1{\cdot}2$	Equivalent beam $M = 1{\cdot}2$
Prop force kN/m 1st frame	195	456	330	159	128	136
2nd frame	578	786	630	487	374	511
3rd frame	638	669	456	634	488	590
Max bending moment: kN/m	260	not calculated	not calculated	415	370	311
Embedment: m	1·7	1·7	not calculated	not calculated	not calculated	1·2

there appears to be no reason for agreement between calculated values from this method with those methods that do so, that is those methods in BS 8002 using a simplified beam.

(*b*) The Winkler Spring Analysis assumes elastic soil deformation which may not have been the case locally in the observed braced excavations on which the strut load envelope was based.

(*c*) The Winkler Spring Analysis assumed certain stiffness for soil, walls (Larssen sheet piles) and struts (tubular steel) which may not have been similar in the prototype walls observed by Terzaghi and Peck and formed the basis for the trapezoidal strut load envelopes.

(*d*) The simplified beam analysis contains assumption of pinned joints and point of support below formation level which are convenient short cuts in calculation but incorporate inaccuracy.

(*e*) The use of the mobilization factor as a uniform factor is not justified for a flexible wall and where wall deformation varies with depth.

(*f*) The assumption that the use of a mobilization factor allows wall moment and strut loads to be calculated without further factor of safety does not provide an adequate or uniform margin of safety for structural design of wall or struts.

(*g*) Estimation of safe embedment depth by Winkler Spring and the derivative of the trapezoidal envelope gave similar results for sand, and either method may probably be used for this purpose.

(*h*) The trapezoidal diagram and beam methods are not appropriate for analysis where post tensioned anchors are used for wall support.

(*i*) In the experience of the authors and others, the most prevalent modes of failure of multi-braced walls and sheeting are insufficient depth of embedment and inadequate strutting at the lowest and penultimate bracing levels. Flexural strength of the walling or sheeting is less important as a mode of failure. Use of the Lawall Winkler Spring design values appears to safeguard these most likely causes of failure more securely than the other methods used in the comparison.

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In clay

- The results for distribution of strut loads is not so dissimilar as those for analyses made in sand. The average total strut load of the methods used other than the Winkler Spring method is only approximately 10% less than the total for the Winkler Spring analysis.
- The maximum bending moments calculated by Winkler Spring Theory are less than those calculated by the equivalent beam methods.
- The load on the wall from the water-filled tension crack is high as a proportion of total load on the wall when the requirements of BS 8002 are used for calculation of water-filled tension crack depths.
- Because of the authors' experience of the most likely failure modes, that is, insufficient embedment and inadequate bracing at the lowest and penultimate frame levels, they

Soil parameters							
Undrained				Drained			
Bulk density γ kN/m²	C_u kPa	$\phi°$	E mPa	γ kN/m²	C' kPa	$\phi'°$	E' mPa
18	–	30°	10	18	–	30°	10
19	–	37°	20	19	–	37°	20
19	–	40°	20	19	–	40°	20
20	30 varies linearly 45	–	6 9	20	5	27°	5·2 varies linearly 7·8
20	60 120 160		12 24 24	20	10	25°	10·45 varies linearly 21·8

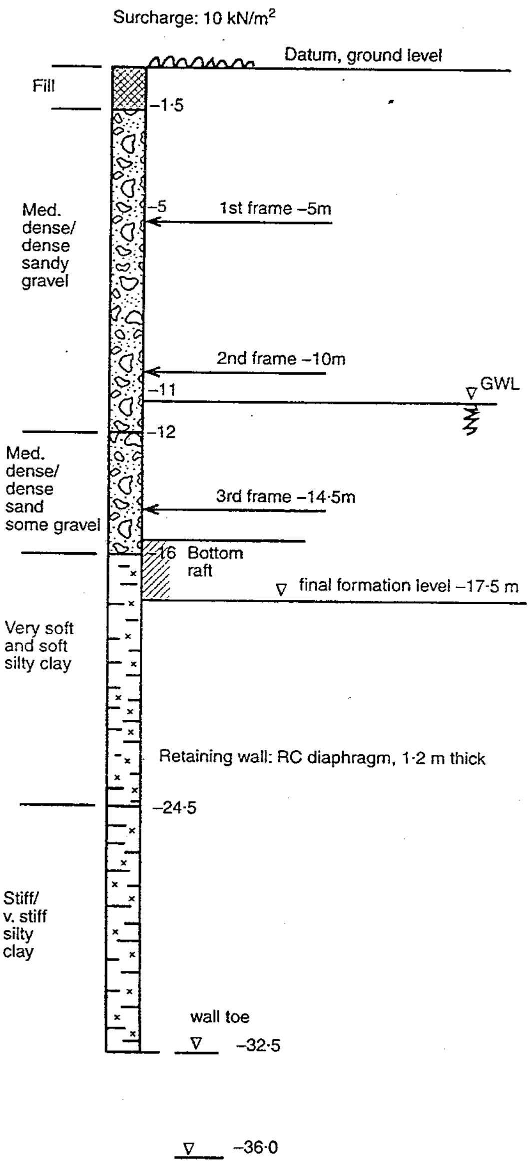

Fig. 7. Case study braced excavation, layered soils with soft clay: wall geometry and soil conditions

Prop. stiffness assumed: 1·2 m dia. tube, 20 mm thick, 10 m long, at 4 m centres horizontally
$E = 210\,000$ mN/m²

Excavation sequence
(i) Cantilevered wall excavation to –5·5 m
(ii) Install prop at –5 and excavate to –10·5 m
(iii) Install prop at –10 and excavate to –15 m
(iv) Install prop at –14·5 and excavate to –17·5 m

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would prefer to use the design values calculated by the Lawall program using Winkler Spring Theory.

Case study for multi-braced walls

16. A case study was undertaken on a deep braced diaphragm wall in layered soils to investigate the requirements of BS 8002 when applied to a deep braced temporary wall in varying soil conditions. The wall and soil conditions are shown in Fig. 7.

17. The soil conditions at this particular site, near Eastbourne in Sussex, are not ideal for economical soil support. Relatively dense granular soils overlie a very soft silty clay at final formation level. The effect of this soil sequence is to cause high bending moments within the sheeting or walling and high strut loads in both the lowest and middle frames of bracing because of low passive support for the sheeting below final formation level. Loads on the active side of the sheeting are increased due to groundwater pressure.

18. Comparison of results is made between the Lawall program based on the Winkler Spring Theory, analysis using the trapezoidal strut load envelope based on empiricism and a finite difference analysis using the FLAC program. The BS 8002 requirements used in the second Lawall run are as follows

(*a*) both effective stress and total stress analysis to be made where soft clays occur
(*b*) minimum surcharge of 10 kN/m^2 to be used
(*c*) minimum 0·5 m over-dig to be used in calculation
(*d*) mobilization factors to be used: 1·2 on values of $\tan\phi'$ for cohesionless soils, 1·5 on values of c_u for the very soft clay stratum and the stiff clay which underlies it. A mobilization factor of 1·2 is used on the low values of c' in both clay strata
(*e*) wall friction $\delta'_a = \delta'_p = 0{\cdot}75\tan^{-1}\phi_m$, where $\tan\phi_m = \tan\phi'$/mobilization factor and wall adhesion $= 0{\cdot}75c_m$, where $c_{um} = c_u$/mobilization factor.

Earth pressure diagrams based on limit active and passive pressures for both drained and undrained conditions are shown in Fig. 8, plotted as net horizontal pressure diagrams on the same axis. It is evident from this plot that the undrained conditions are more severe below final formation level, and because of the sensitivity of wall bending moments and strut loads to high earth pressures below formation level become the basis of wall design.

19. BS 8002 recommends that both drained and undrained analysis should be made for walls in clays unless short-term conditions can be assessed using effective stress methods together with knowledge of pore water pressure

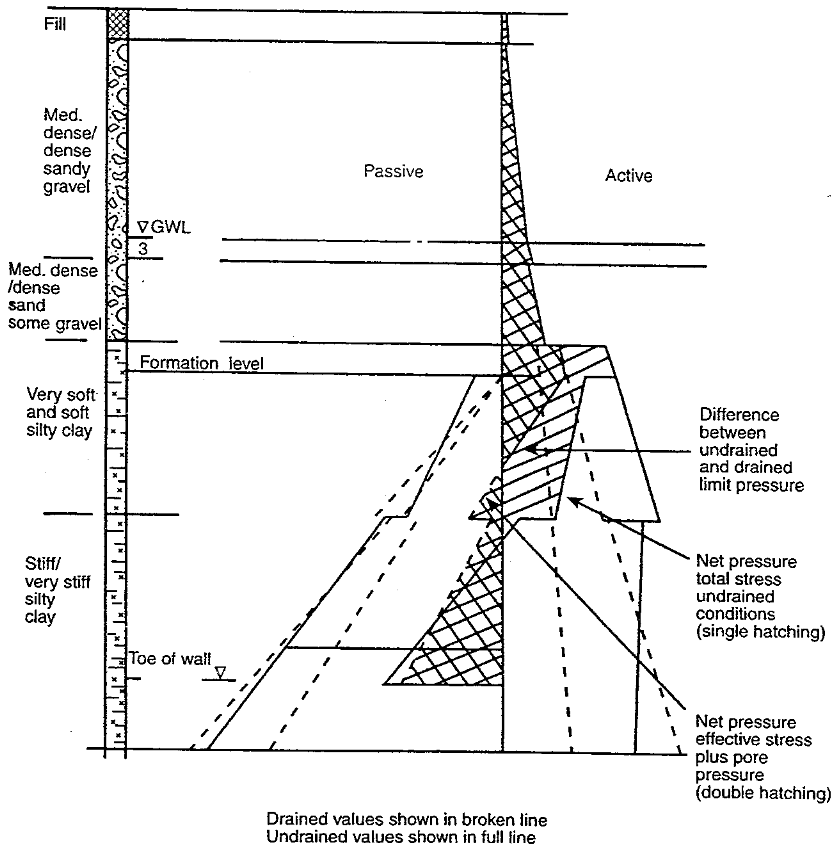

Fig. 8. Case study braced wall: earth pressure for drained and undrained conditions plotted as net pressure diagrams, limit pressures

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changes during construction. The FLAC[7] finite-element analysis on the other hand permits use of the assumption within the staged excavation of undrained excavation between prop levels with full dissipation of excess pore water pressure during prop installation prior to resumption of excavation to the next prop level.

20. The results of the analysis of the wall using the four methods, Winkler Spring, Winkler Spring using the requirements of BS 8002 for mobilization factor, surcharge and over-dig, the trapezoidal strut load envelope method and finite element analysis are shown in Table 2. Comparison between design strut loads and sheeting moments for the four methods is facilitated by the application of a partial factor of safety to the calculated values with the exception of those calculated using the recommendations of BS 8002, since the Code does not require use of such a partial factor. Referring to Table 2 design values of strut load, sheeting moment and embedment depth vary considerably and conclusions are difficult to draw, and are limited to the following

1. The design values calculated by finite difference analysis are significantly greater for prop design (and waling design) than Winkler Spring analysis using previous design assumptions.
2. There is no significant difference between strut loads using Winkler Spring with BS 8002 requirements or traditional assumptions, but only when the embedment is increased with BS 8002 assumptions to avoid passive failure.
3. The strut loads calculated by the trapezoidal strut load envelope method are very much higher for the lower two frames than values calculated by other methods.
4. Maximum bending moment in the sheeting is not dissimilar for the Winkler Spring methods, BS 8002 and traditional requirements, and finite element analysis.
5. Maximum deformation using finite element analysis is significantly less than that calculated from Winkler Spring analysis.

21. Comparison of the design values gives very little confidence in the security achieved by adopting any one of the four methods of predicting strut load in a staged excavation. In particular, the estimates by Winkler Spring

Table 2. Summary of case study of comparative analyses for a braced wall in layered soils with soft clay

	Winkler Spring. Staged excavation Lawall program. Traditional assumptions: Surcharge: 10 kPa No over-dig No mob. factor Undrained analysis, selected as more severe than drained analysis Partial factor: 1·4	Winkler Spring. Staged excavation Lawall program. As BS 8002; Surcharge: 10 kPa Over-dig: 0·5 m Mob. factors 1·2 applied to $\tan \phi'$ 1·5 applied to c_u Undrained analysis No partial factor applied	Trapezoidal strut load envelope. Two or three frames fixed Calculation by hand Surcharge: 10 kPa No over-dig No mob. factor Undrained, limit earth pressures, selected as more severe than drained analysis Partial factor: 1·4	Finite element analysis. Staged excavation FLAC program Surcharge: 10 kPa No over-dig No mob. factor Excavation undrained with full drainage allowed at each strut level Partial factor 1·4
Prop force: kN/m 1st frame	598	538	593	813
Prop force: kN/m 2nd frame	938	927	1729	1198
Prop force: kN/m 3rd frame	329	365	1395	734
Max. B.M. in sheeting: kN/m at depth	5020 @ 17 m	3900 @ 20 m	Not calculated	3820 @ 19 m
Max. wall deformation: mm at depth	155 mm @ 17 m	243 mm @ 20 m	Not available	60 @ 19 m
Embedment: m F of S. on passive failure	15 1·37	17·5 1·11	15 1·04	15 –

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analysis, including the analysis with the BS 8002 provisions show significantly smaller design strut loads particularly for the lower two frames than those calculated by the other methods. The finite element analysis, assumed by some designers to present more reliable results than either empirical methods or partly flawed Winkler Spring programs, shows values of strut load only 53% of the empirical trapezoidal envelope design value for the lowest frame and 69% for the same value for the middle frame. Despite the lack of agreement between the prototype soil conditions in the original observations by Terzaghi and Peck and those of the case study, it would appear foolhardy to neglect the extent of the differences between these design values, each based on well-regarded methods.

22. The differences between bending moment values calculated by Winkler Spring methods with or without the BS 8002 requirements and those calculated by finite element analysis are not excessive but comparison of maximum deformation values shows the relatively good agreement may be coincidental. In terms of embedment depth, the Winkler Spring analysis using the mobilization factors and overdig requirements recommended by BS 8002 shows that a deeper embedment, to 17·5 m, is required if the Code provisions are complied with.

23. It is possible that the somewhat unusual soil profile used in this case study serves to exaggerate the difference between the calculated values. Nevertheless, it can only be concluded that the adoption of any one method may appear hazardous and further that the advice of BS 8002 that use of a further, partial factor of safety to calculated values is unnecessary is particularly imprudent. The differences in design values as calculated draw attention to the advantages obtained by application of the principles of observational soil mechanics, particularly regarding prop loads in braced excavations.

24. The overall conclusion in these analyses is that the Winkler Spring Method, while itself flawed in terms of its assumptions regarding elastic deformation of the soil, generally gives higher estimates of strut loading and wall maximum moments and similar embedment depths than the methods used for comparison and poses an overall reliable method in the Lawall program. Neither Winkler Spring Methods nor Finite Element/Difference methods are referred to in BS 8002 and while this omission may be warranted for walls of shallow depth, it would appear necessary to include reference for deeper walls.

Effect of wall and strut stiffness

25. In 1969, Peck[8] concluded that for cuts in soft to medium clays the behaviour of the soil and the bracing depended on the stability number $N = \gamma H/s_u$ where s_u is the undrained shear strength of the soil at the side and below the excavation to that depth influenced by a shear failure of the excavation. For excavation in a clay with values of N not greater than four, Terzaghi and Peck recommended the use of the trapezoidal diagram with a maximum value of $0{\cdot}4\gamma H$. While the behaviour of clay in deep cuts or cuts in soft clays is influenced by plastic behaviour of clay near the base of the excavation in highly stressed soil zones, the influence of the wall and strut system stiffness was considered important in braced cuts where values of N are low and deserved more investigation. (For the example of the case study the value of N was 2·0).

26. The BS 8002 beam method assumes a flexible wall and rigid propping while the programs based on the Winkler Spring Theory require input of assumptions of wall strut and soil stiffness. Using the Lawall program a series of calculations for upper, middle and lower strut forces were made with constant soil stiffness for the soil conditions, strut location and excavation depth of the case study examples for either a flexible sheet pile wall or a relatively rigid diaphragm wall with a wide range of prop stiffness at each level. Plots of the results are shown in Fig. 9.

27. The calculations show that there is a threshold value of the ratio between strut and wall stiffness below which the prop force increases with reduced prop stiffness (i.e. the wall is failing) while above the threshold the prop force increases with increasing prop stiffness, (i.e. the wall is sufficiently stiff to redistribute the load to the props). The top and middle prop loads always increase with increasing prop stiffness, but this increase is not linear. For the walls studied, the variation in top prop force in the range of wall/strut stiffness ratios considered is from 150 kN/m to 300 kN/m. This considerable variation deserves further investigation for both soils of unchanged and varying stiffness. The methods of analysis which consider neither wall nor strut stiffness or use empirical results without considering either wall or strut stiffness in the prototype or the design wall appear flawed. Both methods recommended in BS 8002 (Terzaghi and Peck strut load envelope and the simplified equivalent beam methods) fall into these categories.

Conclusions

28. The conclusions of this Paper are as below.

1. The basis of wall design by BS 8002 is said to be based on ultimate limit state methods. The use of a mobilization factor introduces

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earth pressure at serviceability state for use at ultimate limit state. The use of a constant mobilization factor is flawed for flexible walls since the wall deforms varying amounts with depth. In layered soils the constant mobilization factor presumes incorrectly that different soils require similar movement to achieve peak strength. The use of a mobilization factor of similar value for different soils is also incorrect in homogeneous soil conditions.

2. For the embedment structures the comparative calculations showed that:
   (*a*) For typical cantilever walls of 4 m depth in dry sand within the range of ϕ between 30° and 40° there was little difference in values of embedment and maximum bending moment in the sheeting between methods due to BS 8002 and the previously used method due to CIRIA Report 104. The same agreement holds for a similar depth cantilever wall where dry sand overlies clay at dredge level and below for range of clay strength c_u between 50 and 150 kN/m^2. This apparent similarity in results may be particular to the wall depth and soil profiles chosen for analysis. The combination of mobilization factor, over-dig and wall friction requirements in BS 8002 appears to bring coincidental similarity to the results.
   (*b*) For single propped walls of 8 m depth in similar soil profiles, either dry sand or sand overlying clay the calculated values of embedment, maximum sheeting moment and prop force were similar for practical purposes for soils within the medium strength ranges, in dry sand for ϕ between 30° and 40° and for clay below dredge level where c_u is in the range between 50 and 200 kN/m^2. It appears that for both cantilever and single propped walls in these soils of medium strength the combination of factors, over-dig allowances, wall friction parameters and minimum surcharge loading allowances brings practical similarity to the BS 8002 and CIRIA Report 104 calculated values. This agreement would not appear to justify the reliability of the analytical methods on which the Code and CIRIA report requirements are based, and indeed for lower and higher strength soils above and below the medium strength zone some wider differences occur between calculated values.
   (*c*) The greater inexactitudes in the results of calculations of multi-braced walls is evident from the comparative calculations for braced excavations 15 m deep in dry sands and clay, each of medium strength. Analyses made by Winkler Spring and by both empirical and simplified beam methods recommended by BS 8002 show significant differences between prop force, maximum moment in the sheeting and embedment. In particular, the range of calculated forces shows that either dangerously low or uneconomically high design prop and waling loads would result from the choice of method of wall analysis. A case study of a diaphragm wall braced with three frames of steel tubular strutting in layered soils to a depth of 17·5 m further confirms this situation. Analyses were made using a trapezoidal strut load as advised by BS 8002, Winkler Spring Analyses were made undrained both with and without the Code over-dig and mobilization factor requirements and a finite difference

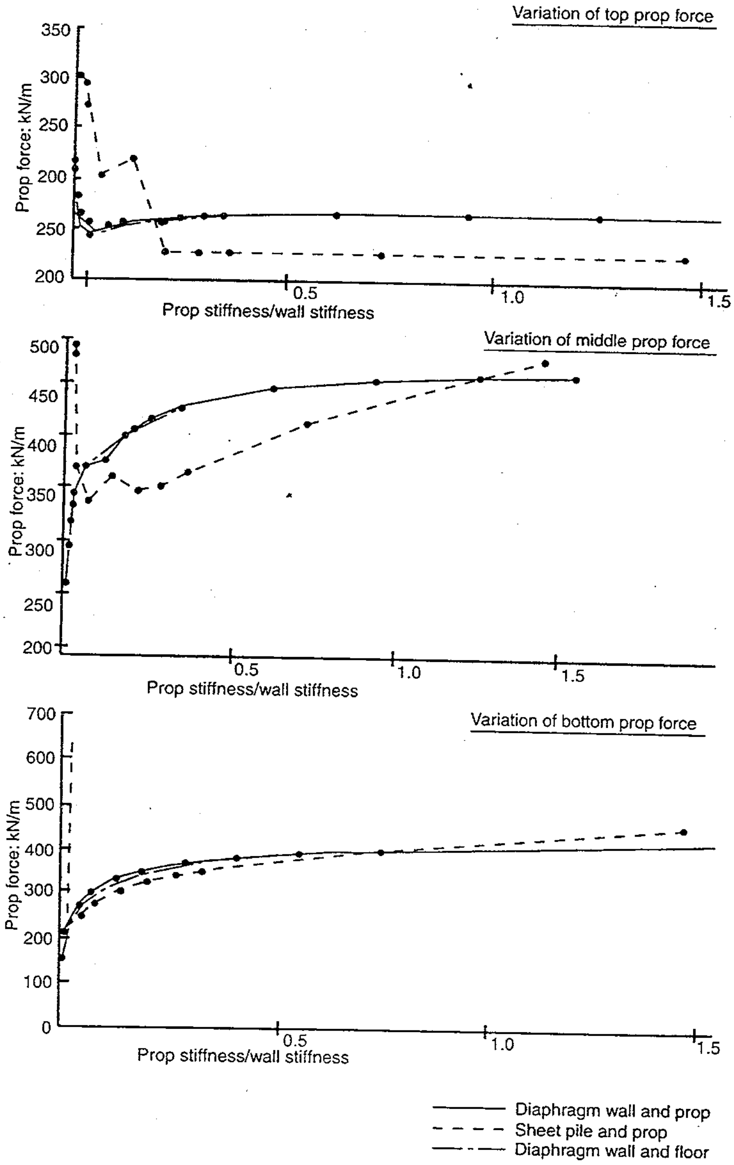

Fig. 9. Variation of prop force

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analysis was applied without the Code requirements but allowing dissipation of excess pore pressure at each strut installation stage. The range of calculated results was so wide for prop force and embedment that the choice of analysis method was shown again to be critical to avoid risk of either structural failure or grossly uneconomical design.

(*d*) It is evident that BS 8002 does not serve to resolve the difficulties of economical, safe multi-braced wall design in either uniform or layered soils. It would appear that Winkler Spring and finite difference methods in current use can produce widely different calculated values (e.g. the case study lowest frame strut loads). On the other hand, reliance on the traditional trapezoidal envelope method as recommended by BS 8002 can produce results greatly in excess of these from either Winkler Spring or finite difference methods.

(*e*) We conclude that for multi-braced excavations the Code should refer specifically to the use of Winkler Spring and finite element and finite difference methods (it ignores them) but should add the rider that calculated strut loads by either of these methods should be compared with values calculated using the empirical trapezoidal strut load envelope without over-dig or mobilization factor requirements. Drained and undrained comparisons are necessary in cohesive soils particularly where soft clays occur.

3. Other conclusions on BS 8002 which particularly apply to the relatively flexible embedded and multi-braced walls are:

(*a*) The Code BS 8002 does not identify any difference in earth pressure or earth pressure distribution between anchored and propped walls of similar depths and similar soil conditions. Where anchors are pre-stressed in cohesionless soils, reduced wall and soil deformation at serviceability state are likely to result and presumably increased values of mobilization factor as defined by BS 8002 should apply.

(*b*) The assumption regarding the depth of tension crack in clays and the extent to which it is filled with water is important in defining total pressure on walls and the resulting wall design. Where a tension crack is not filled with water, the minimum pressure as defined in CP2 due to a fluid of minimum density 5 kN/m^3 is not included in BS 8002.

(*c*) The critical importance of conditions of vertical relative moment of soil and wall needed to mobilize the wall friction should be emphasized in BS 8002. A capping value for wall adhesion should be applied in BS 8002 in a similar way to that specified in CP2 and CIRIA Report 104.

(*d*) BS 8002 should refer to methods of calculation for earth pressures on diaphragm walls which take into account soil deformation caused by the installation process.

(*e*) BS 8002 does not define different factors of safety for permanent and temporary works nor does it recommend the designer to apply increased factors of safety for structural members of the wall/bracing system where collapse risk is greater.

(*f*) Because of the presumption in BS 8002 of a constant mobilization factor for all soil, for all wall depth, for both homogeneous and layered soil conditions at the beginning of the analysis, the use of Winkler Spring programs which obtain a balance between active and passive pressure, mobilizing the minimum of peak strengths to maintain statical equilibrium, may prove incompatible to the principles of BS 8002.

(*g*) The relevance of varied stiffness of wall and bracing systems to calculated strut loads is illustrated in the Paper for soils of uniform stiffness. Further study requires to be made of the effect of varying soil, wall and strut stiffness on strut loads, wall moment and embedment for deep braced and cantilevered excavations.

29. It is the authors' view that unlike its predecessor CP2, the new Code has lost the opportunity to advance retaining wall design. The comparative calculations in the Paper show that wall design using the Code's recommendations for multi-braced walls produces significant differences in wall embedment, maximum moment and strut loads with design made by previous methods in varied soils and wall geometries. It appears that the designers' experience is still required to reject or vary calculated results to avoid risk of failure or lack of economy. The Code appears to do little to improve the previous complete reliance on empirical methods; the Code's innovations appear only to be detail changes which may be convenient to assist the application of limit state methods to both soil and structure but do little to improve the reliability of design. The Code does not advise on numerical methods of wall analysis, nor does it refer to users' experience which is now available. The restriction of the Code's specific recommendations to walls with a maximum depth of 8 m is particularly regretted and appears illogically to place walls of greater

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depth in a different design category. Some of the Code's recommendations appear too prescriptive (surcharge, over-dig, etc.), while its lack of direction on matters such as mode of wall failure and the necessary design remedies appears a serious omission.

Acknowledgement

30. The authors wish to express their gratitude to their colleagues in the Butler Puller Partnership for helpful discussion and, in particular, to Dr Wing Sun for his assistance in making the finite element analysis of the multi-braced excavation.

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B 20

APPENDIX – C

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APPENDIX – D

Presented by
Dr.Prasanna Rathnaweera

APPENDIX D: Representative Soil Strength (BS 8002):1994

Design Procedure

- Step I: Representative Soil Strength.
  - ⇒ is a conservative estimate (more adverse than likely values) of the mass strength of the soil (Cl. 1.3.2).
  - ⇒ obtained from Representative Soil Parameters obtained from in-situ and laboratory soil test data.
  - ⇒ In the absence of such data, from Tables 1, 2, 3, and 4.
- Step II: Design Soil Strength
- Step III: Design Earth Pressures

D1 Site Investigation

Assess the site to identify Representative Soil Parameters that may lead to Serviceability and Ultimate Limit States being reached.

Code of Practice for Site Investigation BS 5930 describes the general considerations to be taken into account and details the methods of site investigation available. Also refer ICTAD document on Specifications for Site Investigation and Typical Bill of Quantities for Building Works.

- Compile information on site conditions.
  - ⇒ General
    - ♦ topography and layout of the site.
    - ♦ details of adjacent foundations and services.
    - ♦ nature of ground and groundwater conditions.
    - ♦ tidal and seasonal variations.
    - ♦ strength and deformation properties of the soils: a) which will be retained, b) which will support the earth retaining structure.
  - ⇒ The relationship of the site to the overall geology should be established.
  - ⇒ The number of boreholes or other forms of investigation:
    - ♦ should be adequate to establish the ground conditions along the length of the wall and to ascertain the variability in those conditions. Varies 10-50m centres, along the length of the wall.
    - ♦ depth related to geology of the site and the type of wall.
      - a) for backfilled gravity or reinforced stem walls, depth below founding levels should be at least twice the proposed retained height.
      - b) where excavation will be carried out in front of the wall, the boring depth below excavation level should be at least 3 times the proposed retained height.

c) where type of wall and method of construction is uncertain, at the time of investigation, the boring depth below excavation level should be at least three times the proposed retained height.

d) ground anchorages are to be provided, investigation should be done taking the bond length into account.

⇒ Inspection should be carried out from time to time during construction to determine whether the conditions revealed are in accordance with the design.

- Information on groundwater
  - ⇒ requires the knowledge of
    - ♦ the groundwater levels
    - ♦ seepage pressures at the site
    - ♦ hydro-static uplift pressures.
    - ♦ possibility of flooding and its effect on groundwater condition.
  - ⇒ if permeabilities of strata differ, hydro-static levels within each stratum should be obtained. Standpipes and piezometers installed in accordance with BS 5930.
  - ⇒ if deleterious chemicals are present in the groundwater and soil, testing done in accordance with BS 1377: Part 3 Chemical and Electro-chemical tests.
  - ⇒ corrosion in steel and concrete structures should be studied.
- Climatic variations on the structure
  - ⇒ short term - long term rainfall variations and the effect on earth pressures of the resulting moisture content changes.
- Trees
  - ⇒ to minimise or eliminate the adverse effects of root penetration and changes in the moisture content of the soil and any associated desiccation and shrinkage of the soil.
  - ⇒ due to penetration of the root system.
  - ⇒ distance to retaining wall:
    - ♦ for trees and large shrubs, half the expected mature height.
    - ♦ for deciduous forest trees, mature height of the tree (BS 5837).

D2 Soil Test Data

Analysis

- usually involves an Effective Stress Analysis.
- A Total Stress Analysis is permitted when the soil is sheared at a rate

  a) sufficient to prevent or inhibit any significant pore water pressure dissipation so that C_u is the operative shear strength [Cl. 1.3.16].

  b) when data on drained and undrained conditions are available.

- In the first stage Representative Soil Parameters (to estimate Design Parameters Cl. 1.3.6) are selected. These should be conservative estimates (which are more adverse than most likely values Cl. 1.3.2) of the properties of the soil as it exists in situ (BS 1377 Part 1 to 9).

⇒ selection of soil parameter values which are pertinent to the behaviour of retaining structures. Assessment often dependants on the mechanism or mode of deformation being considered for the retaining structure. Conventional triaxial compression tests represent active conditions. Extension tests may be required if the behaviour of the passive zone is of particular concern.

⇒ A range of values should be considered particularly if the soil parameter values are likely to change during the lifetime of the retaining structure.

⇒ the selection of representative values of soil parameters should consider:

- ◆ geological and other background information.
- ◆ difference between in situ test conditions and the properties measured by field or laboratory tests.
- ◆ the effect of construction activities on the properties of the ground.
- ◆ changes which may occur in the field due to variations in the environment or weather.
- ◆ relevant data from previous projects and the performance of existing facilities.

⇒ Soil will operate at below peak strength conditions, under serviceability conditions. Under Ultimate Limit State conditions, soil may operate beyond peak strength and may dilate to approach the critical state values consistent with the strength envelope for loose or normally consolidated soils.

⇒ Critical analysis of raw data against consistency indices provide a useful correlation with soil strength and stiffness indices.

⇒ local variations of shear strength data obtained from different locations be scrutinised to determine their accuracy.

⇒ mean density of field data to be considered as representative value.

⇒ in the absence of data conservative bounds be applied to generally available parameters.

Clay Soils

- Representative soil parameters obtained from fully saturated samples.
  - ⇒ BS 1377 Part 7: Procedure to determine shear strength parameters for a total stress analysis without pore water pressure measurements.
    - ◆ Direct Shear Test: (small shear box): (cannot control drainage)
      Shear strength parameters: φ', c', φ'_{crit}, φ'_r, $c'_r \sim 0$
    - ◆ Direct Shear Test: (lage shear box for gravelly soils): same as (a).
    - ◆ Ring shear appratus: φ'_r, $c'_r=0$
    - ◆ Unconfined Compression Test: Undrained Shear Strength, C_u , $\varphi_u=0$.
  - ⇒ BS 1377 Part 8: Procedure to determine shear strength parameters for an effective stress analysis, with pore water pressure measurements.
    a) Consolidated Urained Triaxial Test: φ', c', φ'_{crit}, φ, c.
    b) Consolidated Drained Triaxial Test: φ', c', φ'_{crit}.
- Field tests (BS 1377: Part 9 and CIRIA Ground engineering report, 1987) can be used.
- Effects on drained shear strength parameters:
  - ⇒ at initial effective stresses closer to the effective overburden stress (i.e. OCR~1), soil mobilises φ'_{crit}

⇒ at lower initial effective stresses (i.e. at high OCR values), in soils of high sensitivity, the soil strength may drop beyond φ'_{crit}, to φ'_r. Cohesive soils with high clay contents exhibit the greatest fall from peak to residual strength, forming a polished rupture surface. At low residual friction angles φ'_r, previous shear surfaces, in plastic clays, may be reactivated. First time slides due to new construction have been found to mobilise mass strengths no lower than φ'_{crit}.

⇒ therefore the selection of representative values depend on the anticipated stress state.

⇒ if the stress-strain curve never reaches a peak during the maximum strain range achievable in the test, the largest strength mobilised is to be considered as φ'_{max}.

⇒ Take tangent parameters, φ' and c'; or secant value (conservative), over the stress range may be selected.

- Effects on undrained shear strength parameters:

  ⇒ Construction may result in changes in the strength of the ground in the vicinity of the wall. If the mass permeability is low changes may take place over a period; hence short-term (undrained) and long-term (drained) conditions should be studied.

  ⇒ Undrained strength is not a fundamental property of the soil. It depends on the type of test (triaxial compression, extension, direct shear, pressuremeter etc.).

  ⇒ for OCR<3 undrained shear strength increases with subsequent dissipation of pore pressures.

  ⇒ for OCR>3 undrained shear strength decreases caused by swell and softening in the long term. i.e. if C_u is used for temporary construction, soil should be free of permeable zones.

  ⇒ U100 sampling of stiff clays leads to partial remoulding causing excess negative pore pressures, hence the test gives a higher strength. To overcome this, it is recommended to consolidate the sample to its average in-situ effective stress.

  ⇒ Representative values of C_u for peak and remoulded strength should be obtained. C_u peak should make due allowance for the influence of sampling , method of testing, and likely softening on excavation.

- Stiffness parameters

  ⇒ conventional laboratory tests generally underestimates in situ values derived from back analysis of instrumented field structures.

  - ♦ BS 1377: Part 5: Compressibility, Permeability and Durability
  - ♦ BS 1377 Part 6: Consolidation and Permeability in hydraulic cells and with pore water pressure measurements.
  - ♦ BS 1377 Part 9: In-situ vertical deformation and strength tests.

Cohesionless Soils

- For siliceous sands and gravels estimated:

  ⇒ $\varphi'_{max} = 30 + A + B + C$

  ⇒ $\varphi'_{crit} = 30 + A + B$

  - ♦ A=angularity of the particles.
  - ♦ B=grading of the sand/gravel.
  - ♦ C=results of SPT; corrected for the effect of overburden pressure (Thorburn, 1963) and other correction effects.

(refer Table 3 BS 8002: 1994)

- Stiffness parameters
  - ⇒ conventional laboratory tests generally underestimates in situ values derived from back analysis of instrumented field structures.
    - ♦ BS 1377 Part 9: In-situ vertical deformation and strength tests.

Silts

- effective angle of shearing may be conservatively taken as φ'_{crit} in Table 3.
  - ⇒ inorganic siliceous silts can generate as much dilatancy as sands, at the same relative density, but them more easily soften to critical states in thin rupture bands.

Rock

- Site investigation should establish the strength and orientations of the dicontinuity planes, together with water pressures.
- to evaluate the strength and stiffness of certain weak rockmass:
  - ⇒ from correlations with Standard Penetration Test (BS 1377: Part 9)
  - ⇒ from compression and shear wave velocities.
  - ⇒ indicative values of the effective angle of friction (Table 4 BS 8002: 1994) conservatively treated as granular fragments with a RQD~0.

Lateritic Soils

- Hard Laterites: found above the zone of fluctuating water table.
- Soft Laterites: with in the region of fluctuating groundwater level.
- Decomposed rock: in lowest groundwater level.

Type	Average SPT	φ' (degrees)	c' (kPa)	q_{all} (kPa)
Soft	2-15	< 20	0-60	65-140
Hard	15-30	< 30	20-125	20-125
Decomposed Rock	> 30	< 35	50-100	> 200

Tennekoon, et al. (1986); A Review of Site Investigation Practices in Lateritic Soils for Building.

Fill

- types
  - ⇒ graded small rockfills, gravels, sands, certain cohesive fills. Peaty and highly organic soil should not be used as fill.
    - ♦ cohesive fill should be within the range suitable for adequate compaction.
    - ♦ placement moisture content should be close to the final equilibrium value to prevent swelling (if placed too dry) or consolidation (too wet). Volume changes may affect the pressure distribution, medium to long term. Recommended to limit Liquid Limit to 45% and Plastic Limit to 25% (DoT Specification for highway works, 1991).
  - ⇒ Compaction pressures [Cl. 3.3.3.6].
  - ⇒ due allowance be made in estimating the angle of friction, due to deterioration of granules caused by weathering.

Wall friction, base friction and undrained wall adhesion

- representative values obtained from undrained shear box tests.
  - ⇒ surface material (say rough concrete) at the bottom half and the soil on the top half.
  - ⇒ large shear boxes recommended to overcome side and end effects.
  - ⇒ tests be carried out over the range of normal stresses likely to exist on the wall during its life.
  - ⇒ test continued until a reduction in strength due to continued sliding occurs.
- in the absence of large shear box test results, the representative strength:
  - ⇒ in terms of effective stress, should not exceed
    - ♦ $\delta=\varphi'_{crit}$, for rough surfaces with a texture coarser than that of the median particle size; or
    - ♦ $\delta=20^{\circ}$ for smooth surfaces with a texture finer than that of the median particle size.
    - ♦ no effective adhesion c′ be taken for walls or bases in contact with soil. Hence representative wall friction is $\sigma_n \tan\delta$.
    - ♦ undrained shear strength is generally not used since drained conditions prevail at the interface. If such conditions prevail remoulded strength is recommended.

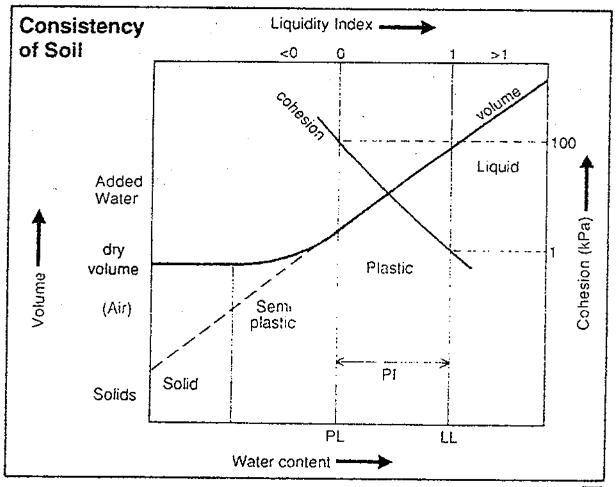
26 Soil Strength

Properties of a soil depend on the grain size, mineralogy and water content, all of which are inter-related.
Clay minerals can hold high water content; for fine grained soils, critical concept is consistency related to water content.

SOIL CONSISTENCY

With varying water content, a soil may be solid, plastic or liquid. Most natural clays are plastic.
Water content (w) = weight of water as % of dry weight.
Consistency limits (Atterberg limits) are defined as:
Plastic limit (PL) = minimum moisture content where a soil can be rolled into a cylinder 3 mm in diameter.
Disturbed soil at PL has shear strength around 100 kPa.
Liquid limit (LL) = minimum moisture content at which soil flows under its own weight.
Disturbed soil at LL has shear strength around 1 kPa.
Plasticity index (PI) = LL–PL. This refers to the soil itself and is the change in water content required to increase strength 100 times; high PI soils are less stable with large swelling potential.
Liquidity index (LI) = (w–PL)/(PI). This refers to a particular water content; it is a measure of the consistency and strength.

Clay Mineral	Activity	PI	ϕ
Kaolinite	0·4	30	15
Illite	0·9	70	10
Smectite	>2	200	5
PI values are for soil with 75% clay fraction			

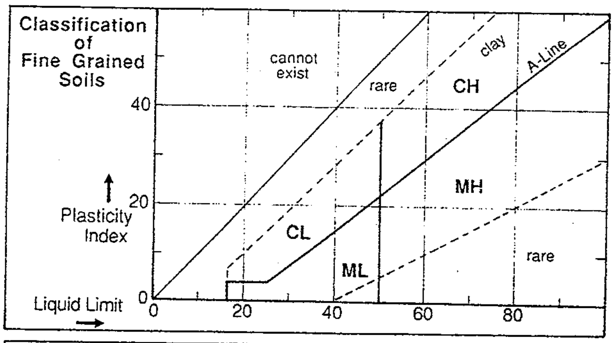


CLAY MINERALS

Plasticity and properties of clay soils depend on amount and type of clay minerals.
Soils with < 25% clay minerals are generally stronger, with low PI and $\phi < 20\%$.
Activity of clay = PI / % fines (< 0·002 mm diameter).
Soils with high clay fraction and high activity can retain high water content, giving them low strength, and also have low permeability.
Activity is mainly due to clay mineral type; smectite (montmorillonite) clays are the most unstable.

SOIL CLASSIFICATION

Soils are classified on grain size and consistency limits.
A-line distinguishes visually similar clays and soils.
More subdivisions exist in a full soil classification.



SHEAR STRENGTH

All soils fail in shear.
Shear strength is a combination of cohesion and internal friction; expressed by Coulomb failure envelope.
Cohesion (c) derives from interparticle bonds; significant in clays, zero in pure sands.
Angle of internal friction (ϕ) is due to structural roughness; higher in sand than in clay.

- Shear strength = cohesion + normal stress x tan ϕ

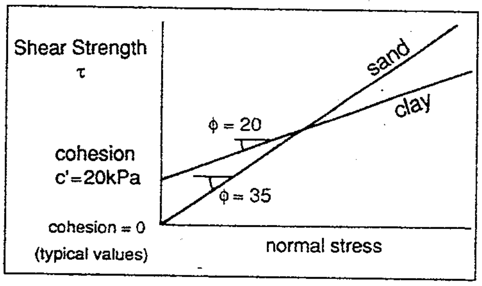
Normal stress is critical to shear strength but pore water pressure (pwp) carries part of overburden load on soil, thereby reducing normal stress.

- Effective stress (σ') = normal stress (σ) – pwp.

Shear strength is correctly defined in terms of effective stress, so that:

- Shear strength (τ) = $c' + \sigma' \tan \phi'$

Soil Classification		grainsize	typical values		
type	class	mm	LL	PI	ϕ
Gravel	G	2–60			>32
Sand	S	0·06–2			>32
Silt	ML	0·002–0·006	30	5	32
Clayey silt	MH	0·002–0·06	70	30	25
Clay	CL	<0·002	35	20	28
Plastic clay	CH	<0·002	70	45	19
Organic	O	–			<10



Properties of Cohesive Clay Soils							
Material	State	LI	SPT, N	CPT, MPa	C, kPa	m_v, m²/MN	ABP, kPa
Alluvial clays	soft	>0·5	2–4	0·3–0·5	20–40	>1·0	<75
	firm	0·2→0·5	4–8	0·5–1	40–75	0·3–1·0	75–150
Till and Tertiary clays	stiff	–0·1→0·2	8–15	1–2	75–150	0·1–0·3	150–300
	v. stiff	–0·4→–0·1	15–30	2–4	150–300	0·05–0·1	300–600
	hard	<0·4	>30	>4	>300	<0·005	>600

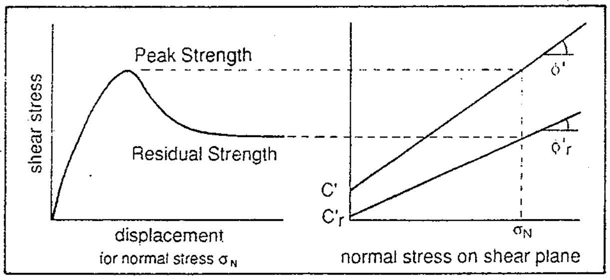
Cohesion (c) is equivalent to short term shear strength

STRENGTH DECLINE IN CLAYS

Drainage progress of a loaded clay is critical as any increase of pore water pressure may lead to failure; significant in new excavations and embankments.

Peak strength declines to residual strength due to restructuring, notably alignment of mineral plates, during dislocation along a plane. Change is due to almost total loss of cohesion and also reduction in friction angle. Significant in all clays, notably those with higher PI.

- Brittleness = % decline from peak strength.



Sensitive clays lose great proportion of their strength on restructuring of entire mass; they have high LI and small grain size, so cannot drain rapidly and load is taken by pwp; shear strength approaches zero.

- Sensitivity = ratio of undisturbed:disturbed strengths and relates to undrained brittleness.

CONSOLIDATION

This is decrease of volume, under stress.

Primary consolidation is large and fast; due to expulsion of water until excess pwp is zero.

Secondary consolidation is small and slow; due to restructuring and lateral movement; same as drained creep.

Normally consolidated clays are those compacted by their present overburden of sediments.

Over-consolidated clays are those more compacted in the past by overburden soils since removed by erosion (or by glacier ice); they can bear loading up to their previous overburden stress with only minimal compression and settlement.

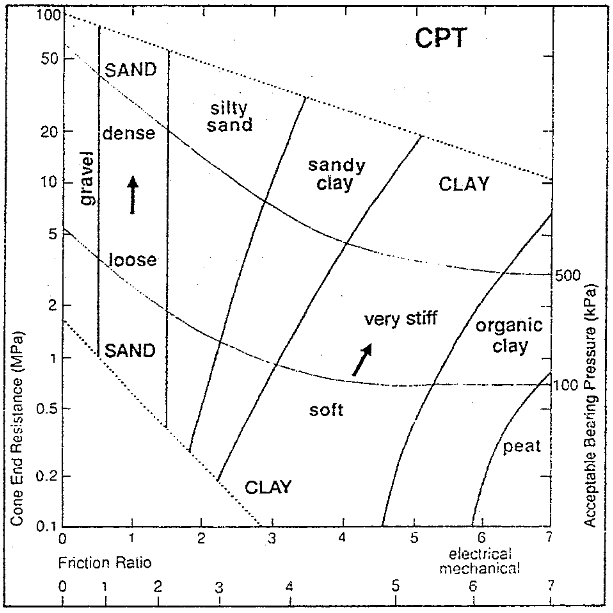
Compression coefficient = m_v = reduction of thickness with increase of stress; correlates closely with LL.

CONE PENETRATION TEST (CPT)

In a site investigation borehole, a 60° cone (= 36 mm in diameter) is driven into soil at 15–25 mm/second, followed by a concentric outer sheath.

End resistance and sheath resistance are measured: Friction ratio = (side friction/end friction) | 100; ratios on standard electrical systems differ on less commonly used mechanical systems.

Values relate to soil types and packing state, and give indication of Acceptable Bearing Pressure.



ACCEPTABLE BEARING PRESSURE

Values relate largely to soil water content and consolidation history.

Depend on SBP and acceptable settlement.

- Settlement = m_v × thickness × imposed stress.

Rate of settlement depends on permeability; slow in clay soils which cannot drain rapidly.

Settlements on clay may be large: then referred to as subsidence, along with other processes which affect clays (section 28).

Non-cohesive Soils

Sand soils, and gravels have no cohesion, except that derived from any clay matrix and water suction. Sand stands in steep slopes when wet due to negative pore pressure (critical in building sand castles), but will not stand when dry or saturated.

Strength, slope stability and bearing capacity all derive from internal friction; ϕ for granular soils (sands and gravels) range 30–45°; increases due to grading, packing density and grain angularity.

Settlement is small and rapid; not usually considered, except on very loose sands and artificial fills.

Properties are best assessed in situ by SPT; N values are a function of packing density and grading.

Bearing capacity of sandy soils may be improved by dynamic consolidation (with a 20 ton weight repeatedly dropped from a crane) or by vibrocompaction.

Properties of Sands

Packing	RD	SPT	CPT	ϕ	SBP
v. loose	<0·2	< 5	<2	<30	<30
loose	0·2–0·4	5–10	2–4	3–32	3–80
m. dense	0·4–0·6	11–30	4–12	32–36	8–300
dense	0·6–0·8	31–50	12–20	36–40	3–500
v. dense	>0·8	>50	≥20	≥40	≥500

STANDARD PENETRATION TEST (SPT)

In a site investigation borehole, a 51 mm split tube sampler is driven for 150 mm.

Using 64 kg hammer dropped 760 mm, number of blows (N) is counted to drive the tube the next 300 mm.

A simple, effective test; N values closely relate to sand properties; should be used with care in clay soils.

(At shallow depth N may be multiplied by empirical correction factor, F, to allow for low stress;

F = 350/(25D + 70), where D = depth in m.)

Relative Density is a measure of grain packing on a scale from loosest to densest possible states of compaction.

SPT refers to corrected N values.

CPT values are end resistances, in MPa, for fine sand; values are lower in silts and lighter in gravels.

Friction angles are for average sand; add 2° for angular grains; subtract 3° for rounded grains; add 5° for gravels.

SBP values, in kPa, are for foundations 3 m wide with settlement < 25 mm; multiply by 1·4 for strip foundations 1 m wide; values are halved for sand stressed below water table.