GCO PUBLICATION No. 1/90

REVIEW OF DESIGN METHODS FOR EXCAVATIONS

GEOTECHNICAL ENGINEERING OFFICE
Civil Engineering Department
The Government of the Hong Kong
Special Administrative Region

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First published, March 1990
Reprinted, September 1991
Reprinted, November 1996
Reprinted, August 2000
Reprinted, February 2003

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FOREWORD

This document presents the results of a review carried out within the Geotechnical Control Office of the state of the art in the design of excavation support systems, and in the prediction of ground movements around excavations. Whilst the document is not intended to be a design guide, the information given here should facilitate the use of modern methods and knowledge in the design of excavation support systems. Practitioners are encouraged to comment to the Geotechnical Control Office on the contents of this publication.

The review has been carried out under the general direction of Mr J.B. Massey as a project in the Geotechnical Control Office's Research and Development Theme on Excavations. It was undertaken by Mr C.Y. Kwan, with checking, editing and production by Dr P.L.R. Pang. Numerous staff of the Geotechnical Control Office and the Housing Department, and several firms of consulting engineers, have contributed during the preparation of this document. In particular, Dr S. Buttling provided valuable comments in the final stages. The contributions of all those who have assisted are gratefully acknowledged.

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March 1990

CONTENTS

1. INTRODUCTION

This document presents a review of the state of the art in the design of excavation support systems in soils, and in the prediction of ground movements around excavations. It is written in the context of the dense urban setting of Hong Kong and its geological and topographical characteristics. While a fair proportion of the literature reviewed is based on British and North American work, relevant Hong Kong literature is also cited. A brief review of the design practice and geotechnical control of excavations in Hong Kong is given in Chapters 10 and 11 respectively.

In this document, the term "excavation" is applied generally to cover all basements, large pits and trenches, while the term "deep excavation" is reserved for excavations which are deeper than about six metres (20 feet), in conformity with the convenient distinction made by Terzaghi & Peck (1967). Descriptive terms used for rocks and soils are as defined in Geoguide 3 : Guide to Rock and Soil Descriptions (GCO, 1988).

2. TYPES OF EXCAVATION SUPPORT SYSTEMS

Excavations for urban building development and civil engineering works are often carried out up to the property boundary and usually require batters which are not stable without structural support. Some of the structural systems which have been used to provide lateral support to the ground are illustrated in Figure 1 (Institution of Structural Engineers, 1975), and their advantages and disadvantages are listed in Table 1.

The following retaining wall types are commonly used in Hong Kong to support excavations :

(a) sheet pile wall,
(b) soldier pile wall,
(c) diaphragm wall,
(d) caisson wall, and
(e) grouted pile wall.

The above retaining wall types can be further divided into the following three categories according to the form of support provided :

(a) cantilevered or unbraced wall,
(b) strutted or braced wall, and
(c) tied-back or anchored wall.

The factors to be considered in the choice of a support system for a deep excavation are given in Table 2.

The cantilevered wall depends upon its own strength and embedment to carry the earth and water loads, while strutted and tied-back walls transfer much of these loads to additional support elements. Cantilevered walls are limited to relatively shallow excavations unless designed to be massive and embedded in stiff soils. However, under such circumstances the strutted or tied-back wall is likely to be more economic than the cantilevered wall.

In a strutted excavation, there is an economic incentive to the contractor to excavate as far as possible below the supports. For tie-back systems, the excavation floor at any given time is a necessary and convenient platform for the installation of the anchors, which is usually carried out at the correct levels due to difficulties in providing access later. In this respect, tie-back systems have an advantage over strutted systems in that they automatically guard against over-excavation. Although an open excavation area is created by the tie-back systems, their adoption is not always possible due to encroachment into adjacent property. In the urban areas of Hong Kong, the strutted excavation is the most popular system. A summary of the design considerations for strutted and tied-back walls is given in Table 3.

3. CONSTRUCTION ASPECTS OF EXCAVATION SUPPORT SYSTEMS

3.1 SHEET PILE WALLS

The sheet pile wall is the most common type of temporary retaining wall system used in Hong Kong. When selecting sheet piles to be used, it is important to taken into account the drivability of the piles. The ability of a sheet pile to penetrate the ground depends on the section size of the pile and the type of pile hammer used, as well as the ground conditions. Gillespie (1983) considered that it is difficult to drive sheet piles through soils with Standard Penetration Test (SPT) 'N' values greater than about 50, and that the penetration through zones with N values of 80 or more is very limited. For heavy pile sections, however, the corresponding SPT values could be up to 80 and 120 respectively. It should be noted that hard driving conditions will necessitate the use of pile sections larger than those required to resist bending moments due to lateral earth pressures.

It is not uncommon in Hong Kong for sheet piles not to reach design penetration. At an estimated 75% of sites where sheet piles were used (Malone, 1982), this problem was sufficiently extensive to necessitate a significant design change after the piles were driven. In weathered rocks and colluvium, hard corestones and boulders are the obstacles, whereas in fill, the problems are boulders, old foundations and seawalls. These obstructions are normally removed in advance of the main excavation by means of a local cofferdam, and then the sheet piles are redriven to the required depth of penetration. In some cases, even the use of a powerful pile hammer and heavier steel sheet pile sections is unable to either split or push aside the boulders or corestones. Special measures such as the provision of grout curtains behind, and driving of H-piles in front of the inadequately penetrating sheet piles may be necessary. Sometimes, part of a line of sheet piles is replaced by a group of soldier piles installed in pre-drilled holes through the obstruction.

Seepage may be expected to pass through interlocking steel sheet piling which supports a difference in hydraulic head. In compressible soils, the drop in piezometric levels may induce consolidation. Tearing of interlocks under hard driving conditions may cause ground loss due to groundwater infiltration through the torn interlocks. Extraction of steel sheet piles from cohesive soils often results in removal of soil at the same time, leading to settlement of adjacent ground.

Driving steel sheet piles in loose sandy soils can result in settlements in adjacent ground. The effects of noise and vibration caused by sheet piling operations have been discussed by Clough & Chameau (1980) and Sarsby (1982). In Hong Kong, legislation was introduced in 1988 to control noise from construction and other activities. Under the Noise Control Ordinance (Government of Hong Kong, 1988), construction noise permits must now be obtained from the Environmental Protection Department prior to the use of percussion piling during specified periods. Information on 'quiet' construction equipment and working practices, including percussion piling methods, is given by Environmental Protection Department (1989).

Malone (1982) reported six sheet pile wall collapses and two cases of gross movement of sheet pile walls (Figure 2), and listed the factors present in these eight cases as follows :

(a) inadequate toe support to piles which do not penetrate below bottom excavation level (Figure 2(a)), four known instances,

(b) buckling of struts due to inadequate horizontal or vertical stiffening (Figure 2(b)), three known instances,

(c) raking struts too steep and improperly fixed to piles (Figure 2(a)), two known instances,

(d) raking struts inadequately founded (Figure 2(b)), one known instance,

(e) over-excavation in the passive soil buttress or its complete premature removal prior to installation of struts (Figure 2(c)), two known instances.

(f) In one case, translational shear failure appears to have taken place along a surface at about toe level in a layer of clay, presumed to be a marine deposit, the low shear strength of which had not been appreciated.

It should be noted that in seven of these cases lateral support was supposed to be provided by raking struts, and that in at least five of the eight cases the piles involved had not been driven to the design level before excavation commenced.

Other case histories of sheet pile wall failures have been given by Sowers & Sowers (1967), Broms & Stille (1976), and Daniel & Olson (1982). These studies show that failures of anchored bulkheads and excavation supports are seldom the result of the inadequacies of modern earth pressure theories. Instead, they are caused by the neglect of surcharge loads, construction operations that produce excessive earth pressures, poorly designed support systems, inadequate allowance for deflection, deterioration and corrosion, and poor design and construction details.

The following comments were given by Beattie (1983) on the practical design of sheet piling under Hong Kong conditions :

(a) Sheet piles are rarely installed in a straight line or vertical at depth and the design of shoring and packing may need to be modified on site to cope with the deviations. The packing details should be adequate to take the design loads. Otherwise, yielding of the support will allow movement of the piles and retained ground.

(b) Guidance should be given on the drawings of actions to be taken if the sheet piles are prevented from reaching their design depths by boulders or other obstructions. Other differences between the designer's assumptions and the actual site conditions should be investigated.

(c) The designer should specify monitoring, the results of which should be made immediately available if they are to be of use in gauging the performance of the works.

(d) The site staff should take time to secure progress by ensuring proper installation of struts and support before proceeding with excavation.

(e) Regular site inspections by the designer and close cooperation with the builder are important to ensure the works comply with the intent of the design at all stages.

3.2 SOLDIER PILE WALLS

Soldier pile walls have two basic components, soldier piles (vertical component) and lagging (horizontal component). Soldier piles provide intermittent vertical support and are installed before excavation commences. Due to their relative rigidity compared to the lagging, the piles provide the primary support to the retained soil as a result of the arching effect. Spacing of the piles is chosen to suit the arching ability of the soil and the proximity of any structures sensitive to settlement. A spacing of 2 to 3 m is commonly used in strong soils, where no sensitive structures are present. The spacing is reduced to 1 to 2 m in weaker soils or near sensitive buildings. It is essential that the soldier piles are maintained in full contact with the soil. For this reason, they are either driven or placed in pre-drilled holes, which are backfilled to the ground surface with lean concrete.

The lagging serves as a secondary support to the soil face and prevents progressive deterioration of the soil arching between the piles. It is often installed in lifts of 1 to 1.5 m, depending on the soil being supported and on the convenience of working. There are four methods of transferring earth pressures from the lagging to driven soldier piles, as shown in Figure 3 (Peck, 1969b) :

(a) Lagging is wedged against the inside flanges (Figure 3(a)). In this method, the soil yields as it is trimmed to make room for the lagging. Most of the earth pressure is transferred through the soil to the pile and very little through the lagging to the pile. However, any remaining voids between the soil and the lagging could permit further soil movement.

(b) If large voids exist between the soil and the lagging in (a), they can be filled with grout or mortar to provide full contact (Figure 3(b)).

(c) Lagging set between spacers, the soldier piles and the soil permits the piles to be included as permanent reinforcement to the structural wall (Figure 3(c)). Earth pressure is transferred to the piles as the excavation is carried to the face of the piles, but the heavily loaded soil behind the piles is then cut away to provide room for the spacers and the lagging. As a result, the 'earth wall' moves into the excavation which can result in substantial surface settlement (Peck, 1969b). Therefore, this method should not be used unless large settlement adjacent to the excavation is acceptable.

(d) Contact sheeting is placed against the flanges of the soldier piles on the wall of the cutting, permitting a tight fit because the exposed soil can be trimmed cleanly to the edge of the flange (Figure 3(d)). It is the most satisfactory method for reducing lateral movements, and also serves to provide a form for concrete for the permanent wall.

If there is risk of groundwater building up behind a lagged wall, or of washing in of soil particles, drainage holes may be provided or small gaps may be left between the boards to allow drainage, and in such cases a filter material such as a synthetic fabric may be used to prevent loss of soil.

Suitable conditions for soldier pile walls are provided by over-consolidated clays, in all soils above the water table if they have at least some cohesion, and in homogeneous, free-draining soils that can be effectively dewatered. Unsuitable conditions include squeezing soils such as soft clays and running soils such as loose sands.

In Hong Kong, soldier pile walls are used mainly for small excavations. They are less popular than sheet pile walls, although soldier piles installed in pre-drilled holes are often used to overcome the problem of inadequate penetration of sheet piles in areas where rock is encountered at shallow depths below the excavation. They are generally unsuitable for excavations below the groundwater level, particularly where the retained materials are granular in nature. This is because dewatering may cause significant loss of material and unacceptable settlement to adjacent structures and land, and provision of a grout curtain behind and beneath the wall to strengthen the ground and to act as a groundwater cut-off is expensive. For driven soldier piles, driving of the H-sections may cause heave and settlement of nearby structures (D'Appolonia, 1971). In hard driving conditions caused by boulders or other obstructions, soldier piles can be redrilled if necessary, or relocated to avoid the obstruction, which gives them an advantage over sheet pile walls. Predrilling is often used in urban areas to avoid the noise and vibration of pile driving.

3.3 DIAPHRAGM WALLS

The diaphragm walling technique was first introduced to Hong Kong for the construction of the New World Centre (Tamaro, 1981). As a result of its successful utilization, the technique was used extensively in the construction of the Mass Transit Railway (MTR) underground stations (Openshaw & Marchini, 1980; McIntosh et al, 1980; Budge-Reid et al, 1984) and of deep basements for high-rise buildings (Openshaw, 1981; Chipp, 1983; Craft, 1983; Fitzpatrick & Willford, 1985; Humpheson et al, 1986).

The design and construction of diaphragm walls have been discussed in detail by Xanthakos (1979) and the practical problems associated with diaphragm walling under Hong Kong conditions have been discussed by Openshaw & Marchini (1980), Davies (1982), and Craft (1983).

An important local problem is the temporary stability of the slurry-filled trench. The stability of slurry trenches under Hong Kong conditions has been reviewed by Buttling (1983) and Wong (1984). Wong (1984) considered that both Huder's (1972) method and Schneebeli's (1964) method are conservative for cases in Hong Kong where the soil is fill, marine deposit or colluvium. The reason for this is possibly due to the local practice of not using the apparent cohesion component of soil strength in design. A design procedure was proposed (Wong, 1984) for predicting the stability of slurry trenches subjected to heavy surcharge load from adjoining and adjacent foundations, by taking into consideration the redistribution of surcharge load to soil adjacent to the excavation, and the redistribution of stresses in the building structure.

In reclaimed land in Hong Kong, the presence of randomly dumped loose bouldery fill is not uncommon. A method for dealing with this problem has been described by Craft (1983). The technique is to first excavate a trench (prior to guide-wall construction) along the line of the diaphragm wall by backhoe to below groundwater level, and to backfill it with a lean mix concrete mixed with bentonite. The lean mix design has to be wet enough to be tremied, strong enough to stabilise the boulders, yet weak enough to be excavated subsequently by clamshell, and of low permeability. The guide-walls are then constructed. Excavation by clamshell is carried out through the lean mix concrete and as far as possible into the underlying boulders. Excavation is usually terminated on the basis of declining net progress. At this point, lean mix concrete is tremied into the excavation. This operation is repeated until the bouldery fill stratum has been stabilised by the lean mix concrete.

Trench stability in soft clays can be improved by reducing the panel length or increasing the slurry pressure. The slurry pressure can be increased by either increasing the density of the slurry, or raising the level of the slurry above ground level by raised guide-walls; this latter method is not favoured by contractors. To increase the density, special additives can be used, but the properties of the slurry must not conflict with the requirements for placing the concrete. Buttling (1983) considered that viscosity is a more significant parameter than density with regard to displacement of bentonite by concrete, and with proper specification the two parameters can be independently controlled. In sands and other permeable soils, groundwater lowering can be used to improve trench stability, as stability depends on the net slurry pressure, i.e. the difference in head resulting from the difference in level between the slurry within the trench and the external groundwater (Fitzpatrick & Willford, 1985; Humpheson et al, 1986).

During construction of diaphragm walling in weathered granite in Hong Kong, significant ground movements have been observed causing large surface settlement extending a considerable distance from the wall (Davies & Henkel, 1980; Cowland & Thorley, 1984). It is considered that the horizontal movement adjacent to individual panels was a result of swelling of the weathered granite, which created a compressible zone around the diaphragm wall panel. On construction of adjacent panels the arching around the compressible zone broke down, and recompression occurred as the earth pressure built up. This caused horizontal ground movements to extend back from the wall (Davies & Henkel, 1980; Davies, 1982). The ground movements can be controlled by increasing the net slurry pressure supporting the trench, in a similar manner to that described for improving trench stability in marine clays and sands.

Corestones or 'bedrock' (generally taken to be a rock mass containing rock materials wholly or predominantly of grades I to III) encountered during the construction of diaphragm walls in weathered granite can be broken up by grab and chisel (Openshaw & Marchini, 1980). However, diaphragm walling in such cases is extremely expensive due to the cost of rock excavation. Therefore, it is common practice to stop the wall on top of bedrock. If the toe of the diaphragm wall is exposed as the main excavation continues into bedrock, the wall will need to be underpinned, and chemical grouting may need to be carried out to provide an effective cut-off against groundwater.

Diaphragm walling, although expensive when compared with other wall systems, frequently provides the best solution to many underground construction problems, and the technique can usually be adjusted to cope with most conditions of difficult ground and adjacent surcharge by adjustment of the panel length, properties of the slurry, level of slurry in the trench and groundwater lowering.

3.4 CAISSON WALLS

Hand-dug caisson retaining walls, often referred to as caisson walls, are widely used in Hong Kong. The caissons are usually dug in stages of about one metre depth, by husband miners with their wives serving as winch operators at the ground surface. Each stage of excavation is lined with a minimum of 75 mm thickness of insitu concrete using a tapered steel shutter suitably supported and designed for ease of striking. The shutter remains in place to provide support for the fresh concrete and surrounding ground while the next stage of excavation proceeds. Submersible electric pumps are commonly used for dewatering at the base of the caissons. Excavation through corestones and other obstructions is carried out by pneumatic drilling. Pope (1983) considered that the main advantages of a caisson wall over a conventional retaining wall are :

(a) It can be constructed without temporary soil cuts or shoring.

(b) It requires a small plan area and can be used close to site boundaries and existing structures.

(c) It can act as both temporary and permanent retaining structures.

(d) Obstructions, e.g. corestones and boulders, can be overcome without too much difficulty.

It should be noted that some of the above advantages also apply to other wall types described in this Chapter.

Caisson walls were used in the construction of the MTR underground stations (Benjamin et al, 1978; McIntosh et al, 1980) and of deep basements for multi-storey buildings (Beattie & Mak, 1982; Addington, 1983). The caissons can be contiguous, or closely spaced at about two-diameter centres with in-filled concrete panels.

Base heave failures have been experienced with hand-dug caissons in Hong Kong (Chan, 1987). The mechanism of heave is poorly understood, but it is believed to be the result of slaking and swelling, and hence loss of strength, of residual soils and saprolites due to heavy seepage and stress relief. The presence of weak layers such as clayey marine and alluvial deposits may also bring about such failures. The possibility of base heave renders the construction of a deep hand-dug caisson extremely hazardous, particularly when it is carried out below the groundwater table. Moreover, groundwater drawdown for caisson excavation can result in ground settlement and damage to adjacent structures (Morton et al, 1980a & b; 1981). Precautionary measures such as the provision of grout curtains behind the caissons to reduce the effect of drawdown can be expensive. Nevertheless, these are commonly applied whenever unfavourable ground conditions are anticipated. The choice of grouts for hand-dug caisson construction has been discussed by Shirlaw (1987).

Another problem with caisson wall construction is the installation of back drainage. Various forms of drainage system have been discussed by Malone (1982) and these are shown in Figure 4. Type I suffers from the defect that at each stage of excavation the work is undermined. In type II, the no-fines concrete is generally not an adequate filter for residual soil/weathered rock. Types III and IV involve placing filter media in a deep, narrow borehole. Type IV also involves the installation of perforated pipes from the caisson shaft as excavation proceeds. Type V requires the installation of conventional horizontal drains. The long-term performance of these drainage systems will depend greatly on the care taken in design, specification, construction, monitoring and maintenance.

3.5 GROUTED PILE WALLS

The grouted pile walling technique was introduced to Hong Kong in the construction of the Tsim Sha Tsui MTR Station (McIntosh et al, 1980). The most common form that has been used is the "Pakt-in-Place" or PIP wall system. A PIP pile is formed by boring with a continuous flight hollow shaft auger to the design depth. As the auger is withdrawn, cement-sand mortar is injected through the auger shaft, leaving a formed mortar pile. A reinforcement cage is then lowered into the mortar to form a complete reinforced PIP pile. To form a continuous pile wall, PIP piles (A piles) are formed in alternate pile positions in the above manner. The infill piles (B piles) are then constructed in the same way as the A piles, except that during the auger withdrawal and mortar placement, a high pressure plunger pump is used for injecting cement paste through the pile, which effects a vertical mortar cut-off between the B piles and the adjacent A piles.

A recent innovative form of construction, known as 'pipe pile walling' has been used to overcome the problem of inadequate penetration when driving sheet piles through weathered rock with corestones and through old foundations. A pipe pile is formed by installing a perforated steel casing in a drillhole greater in diameter than the casing. As the drill assembly is withdrawn through the casing, suitable grouts are injected through the perforated casing into the surrounding soil. The forming of a pipe pile wall is similar to that of a PIP wall.

The advantages of the pipe pile walling technique are as follows:

(a) The installation of the piles involves little noise and vibration.

(b) The wall can penetrate through boulders, corestones and hard obstacles.

3.6 STRUTTED OR BRACED WALLS

Different schemes for strutting are shown in Figure 5. These include the cross-lot strutting and raking strut support schemes illustrated in Figures 5(a) and 5(b). The diagonal strutting technique shown in Figure 5(c) is more suited to small and preferably square excavations, e.g. shafts. The disadvantage of cross-lot strutting is that the working space can be severely restricted. Diagonal strutting is sometimes used near the corners of wide excavations and serves to leave a large portion of the excavation open. Because only a few struts are used, these structural elements can be very large (typically, pipe sections are used), and they can be subject to large variations in loads due to temperature fluctuations. In some cases, the struts are covered or painted with a reflective paint to minimise temperature effects.

Various methods have been devised to expedite the construction of deep basements through the use of the permanent structure to support the excavation during construction. These methods have the advantage of affording savings of both time and resources, but they do demand a high degree of coordination between the designer and contractor, and tight control of the sequence of operations. Figures 1(e), 1(k) and 1(m) show examples of such methods.

Beattie & Yang (1982) described a project in which an industrial building was constructed using the 'top down' method. During construction, the partially completed building was used to support an excavation up to 15m deep. Garrett & Fraser (1984) described how the top down method (in which the columns of a building below ground level are first constructed and then the permanent structure is built progressively downwards, supporting the excavation walls as they are exposed), the bottom up method (in which the excavation is first completed and the permanent structure is built from the bottom upwards), and a combination of top down and bottom up techniques, have been used successfully in the construction of the MTR underground stations.

Connections and details are of critical importance in an internally strutted excavation. Shoring plans should show :

(a) appropriate means for positioning and fixing of struts and walings, and for bracing of struts in both vertical and horizontal planes to provide lateral stability,

(b) details of web and connection stiffeners, and brackets,

(c) details of welding and other means of structural connection,

(d) details of connections between raking struts and the foundation,

(e) sequence of installation and dismantling of all elements of the structure, and

(f) provisions, if any, for wedging and jacking of struts to prevent horizontal movement.

Typical connection details are shown in Figures 6 and 7. Procedures for prestressing, wedging or jacking to maintain tight contact for all support members and to provide for distribution of load to struts and walings are also very important. Typical prestressing details are shown in Figures 8 and 9. Flat jacks are also commonly used in Hong Kong for prestressing strutted excavations. Preloading to about 50% of the design load is common practice in areas where displacements are of concern. Strut removal can give rise to additional displacements, and these may be controlled by a combination of well-planned re-strutting and effective compaction of the backfill between the wall and the structure.

For walls supported by raking struts, the Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 1985) recommends a detail which involves the construction of trenches in the passive earth buttresses to accept the raking struts and their footings. It also recommends that the distance of a footing from the wall face should be more than 1.5 times the length of the wall embedded below the toe level of the passive earth buttress. These measures are intended to limit the lateral movement of flexible walls.

3.7 TIED-BACK OR ANCHORED WALLS

Some tie-back or anchored systems are shown in Figure 10. These include 'deadman' type anchorages, driven piles and grouted anchor systems. In Hong Kong, prestressed ground anchors are commonly used for retaining excavations. The connecting elements between anchorages and anchor heads are either tie rods or cable strands. Because these elements are commonly made of high strength steel, usually only a small area of steel is required to give the necessary capacity; as a result, the longitudinal stiffnesses of the cables or tie rods are relatively small compared to those of struts. Double corrosion protection of these important structural elements is now standard practice in Hong Kong (GCO, 1989).

Because of their relatively low cost and ease of construction in confined areas, soil nails are being used increasingly in Hong Kong for supporting excavations (Massey & Pang, 1989).

4. LIMIT STATE METHOD OF DESIGN

4.1 GENERAL

In this document, the approach to geotechnical design is discussed using limit state terminology. A limit state is a performance criterion related to a limiting condition beyond which a structure or an element is assumed to become unfit for its purpose. The limit state approach to design requires the designer to check the adequacy of the structure against possible limit states (e.g. ultimate and serviceability limit states) that could be of importance during the design life of the structure. There is nothing new about the limit state approach; international standards on structural use of concrete based on such an approach were available in the early 1970's. Limit state design is, however, unnecessarily associated with the use of probability theory (in the assessment of risk), statistics of variation (in the assessment of loads and material properties) and partial safety factors. The application of limit state principles to geotechnical design does not demand the use of these, either singly or in combination. Nevertheless, the possible inclusion of these additional considerations in future geotechnical codes of practice has caused a great deal of controversy (see, for example, Boden, 1981; Bolton, 1981; Ovesen, 1981; Semple, 1981; Simpson et al, 1981; Smith, 1981). Limit state design of earth retaining structures is advocated in the Draft Model for Eurocode EC 7 : Common Unified Rules for Geotechnics, Design (European Geotechnical Societies, 1987), which gives a full account of the use of the underlying limit state principles.

A structure should be so designed and constructed that its functional requirements are fulfilled throughout its design life. Normally this requirement is satisfied by demonstrating that the structure has the necessary safety against ultimate failure (ultimate limit state) at all times, and that the predicted movements and deformations can be tolerated (serviceability limit state). These two so-called 'functional requirements' mean that two separate calculations are often performed for each individual structure, viz. an analysis against ultimate states of failure, and a deformation analysis of the states of normal use (working conditions). In practice, however, which of these analyses should be used is often dictated by experience; the other analysis may often either be omitted completely or limited to a rough check.

4.2 ULTIMATE LIMIT STATES

An ultimate limit state of a structure is deemed to have been reached when sufficient parts of the structure, the soil around it, or both have yielded to result in the formation of a failure mechanism in the ground or severe damage (e.g. yielding or rupture) in principal structural components. Design against ultimate limit states includes checking against modes of failure such as sliding, overturning, bearing capacity, uplift, and hydraulic failure.

Theoretical methods for analysing structures at collapse limit state have been developed within the framework of plasticity theory. There are two common approaches, the kinematic method proposed by Coulomb and the static method advocated by Rankine. When dealing with perfectly plastic materials, these two approaches provide upper bound and lower bound estimates of the collapse load respectively. Static methods, such as stress analysis, are known to be pessimistic when compared with kinematic techniques, such as wedge or slip circle analysis, which are known to be inherently optimistic. Kinematically admissible mechanisms are useful in determining the possible degree of conservatism in their static counterparts. Where statically admissible stress fields cannot be easily derived, it is necessary to optimise the geometries of different presumed mechanisms to achieve a reasonably low upper bound solution.

4.3 SERVICEABILITY LIMIT STATES

A serviceability limit state of a structure is deemed to have been reached with the onset of excessive deformation or of deterioration. Design against serviceability limit states includes checking against unacceptable total and differential movements, cracking, and vibration.

Serviceability limit state calculations for earth retaining structures involve solution of soil-structure interaction problems, which require the use of deformation parameters. Finite element methods incorporating appropriate soil models are well suited to this type of analysis. However, they are presently too cumbersome and costly for general application, and accurate determination of the required soil parameters is difficult.

Conventional 'working stress' design methods often contain large safety factors against collapse which are also intended to limit the extent of yielding in the soil; elastic soil strains are generally considered to be less damaging than plastic soil strains. However, this approach does not yield the order of the likely deformation of the structure and the adjacent ground. Design methods are considerably clarified if the prediction of deformations is kept separate from calculations for checking against collapse.

5. AN OVERVIEW OF ANALYTICAL METHODS

5.1 TYPES OF METHODS

Methods for analysing earth retaining structures can be broadly classified into the following types :

(a) limit analysis,
(b) associated stress and velocity fields,
(c) beam on elastic foundation,
(d) boundary element method,
(e) finite element method.

These methods are discussed in the following sections.

5.2 LIMIT ANALYSIS

5.2.1 General

The methods of Coulomb and Rankine are the forerunners of the limit analysis approach. As discussed in Section 4.2, they provide upper bound and lower bound estimates of the collapse load respectively. Therefore, they are well suited to the analysis of ultimate limit states. However, they cannot predict deformations associated with the collapse load nor can they yield information prior to the limit state being reached.

5.2.2 Upper Bound Methods

An upper bound solution is obtained by optimising the collapse load obtained from presumed mechanisms with kinematically admissible velocity fields. The velocity fields have to satisfy the velocity boundary conditions, and have to be continuous except at certain discontinuity surfaces, where the normal velocity must be continuous but where the tangential velocity may undergo a jump on crossing the boundary. The trial wedge method is normally used to calculate earth pressures acting on an earth retaining structure (GCO, 1982). Janbu (1957) developed a generalised procedure of slices method which can be adapted for earth pressure computations.

Methods of slices such as that of Janbu (1957) will give upper bound solutions for velocity fields which are kinematically admissible. They have advantages in that layered soil deposits can be analysed, drained and undrained soil strengths can easily be accommodated, and the mechanics of the methods are known to most geotechnical engineers. Also, when programmed into a computer, these methods can greatly facilitate the calculations. The disadvantages, which are common to all limit analysis procedures, are :

(a) No information on deformations can be obtained.

(b) The effects of interaction of materials with disparate stress-strain characteristics are not considered.

(c) The influence of construction sequence or unusual initial stress conditions are not accounted for.

(d) A trial procedure must be employed to locate the most critical failure surface.

5.2.3 Lower Bound Methods

A lower bound solution of the collapse load is determined from a statically admissible stress field which satisfies the stress boundary conditions, is in equilibrium, and nowhere violates the yield condition. If the upper bound and lower bound solutions coincide, then the result is the exact solution for the problem considered.

Numerical analysis using lower bound limit procedures was proposed by Sokolovski (1965) following a finite difference scheme. In the Sokolovski approach, the equations of equilibrium in two dimensions are combined with the Mohr-Coulomb failure criterion to give a pair of differential equations for the stresses along stress characteristics (i.e. lines along which the Mohr-Coulomb criterion is satisfied). The resulting differential equations can be solved by using finite difference techniques for stresses in the soil mass and along the soil-structure interface. The solution yields a prediction for the collapse load, the earth pressure distribution on the structure and the stress distribution in the soil mass at the assumed limiting condition.

Lysmer (1970) developed a lower bound analysis somewhat similar to the finite element method, which is more flexible than Sokolovski's technique. In Lysmer's approach, the soil mass adjacent to the structure is modelled as a series of discrete elements connected at nodal points, which are the corners of the elements. Stress boundary conditions are specified at the nodes along a boundary, and yield criteria (in the form of constraint equations) are assumed. By means of linear programming techniques, optimum lower bound values of stresses within the elements are obtained. Because the method generates its own stress field, it would appear to be ideally suited for use in analysis of more complex problems. However, Lysmer noted that the cost of the analysis increases very rapidly with the size of the mesh configuration, and its application has currently been limited to simple problems only.

Since the lower bound methods suffer from many of the disadvantages of the upper bound approaches, and are more difficult to apply, their role in design will probably be small.

5.3 ASSOCIATED STRESS AND VELOCITY FIELDS

A plasticity method proposed by Roscoe (1970) and extended by James et al (1972) and Serrano (1972) involves the development of consistent (associated) stress and strain fields for fixed increments of structural deflection or load. To begin, a load increment is specified, and an initial stress field is predicted by the method of Sokolovski (1965). Next, an initial strain field is developed, as described by James et al (1972) and Serrano (1972), using assumed soil deformation parameters. After determination of the strain field it will generally be found that the assumed soil parameters are not consistent with the calculated stresses and strains. An iterative procedure is then adopted to generate stress and strain fields which are consistent with all parametric assumptions. At convergence, a complete distribution of stresses and strains in the soil is obtained, as well as earth pressures on the structure.

Fundamental questions remain concerning the appropriateness of the assumed stress-strain laws and the effect on the predictions caused by non-coincidence of principal stress and strain increment directions. Also, questions relating to practical applications, e.g. how wall friction distributions can be assigned, and how account can be taken of construction sequences, seepage loadings, structural flexibility, etc., have yet to be answered. As the method is not fully developed, it is difficult to evaluate its potential as a design tool. Nevertheless, the method does provide the geotechnical engineer with a powerful tool to enhance his understanding of the processes involved in soil-structure interaction.

5.4 BEAM ON ELASTIC FOUNDATION

5.4.1 General

The assumption of a beam or slab on an elastic foundation has found application in numerical analysis of sheet piles and strutted excavations. Power series, finite difference, distribution and discrete element methods have been employed for the solution of the governing differential equations. In each case, the elastic foundation is assumed to generate a reactive pressure proportional to the deflection (Winkler's (1867) hypothesis) and, in essence, consists of a bed of springs. The soil response is usually characterised by a spring constant, which is related to the 'coefficient of subgrade reaction'. The coefficients of horizontal subgrade reaction recommended by Terzaghi (1955) are normally used. The following Sections give a brief review of work based on the Winkler model.

5.4.2 Power Series Method

Rowe (1955a) used the power series method to solve the flexure equation for cantilevered and anchored sheet pile walls and presented the results in the form of design charts suitable for design office use.

5.4.3 Finite Difference Method

Palmer and Thompson (1948) presented a finite difference method for solving the flexure equation for a sheet pile wall. The solution is of the iterative type and requires a knowledge of the coefficient of subgrade reaction and boundary conditions at the extremities of the sheet pile. An important characteristic of this method is that the coefficient of subgrade reaction need not be a constant but can be a function of depth or pressure.

5.4.4 Distribution Method

Turabi and Balla (1968a) presented a distribution theory of analysis for the solution of the flexure equation based on the approximate modelling of the soil below excavation level by five linear springs. The formulae obtained are in a form suitable for general use, and tables and charts are available for design purposes.

5.4.5 Discrete Element Method

Haliburton (1968) presented an efficient discrete element solution for a flexible wall on nonlinear foundation. The wall was divided into short equal lengths of beam elements, each of which could rotate without bending. The elements could transmit transverse forces and couples, and therefore transverse and rotational restraints could be applied. Each element could also sustain an axial force that contributes only to the bending moment but does not produce any axial deformation. The earth pressure-deflection relationship of the soil is assumed to be nonlinear and a function of depth. For simplicity, the portions of the curve between at-rest and passive states and between at-rest and active states were approximated by two straight lines with different slopes (Figure 11). Independent curves for the soil on the active and passive sides of the wall were developed, and the two curves were then superimposed to obtain a combined curve (Figure 12). With the use of a computer, repeated trial and adjustment procedures were carried out until there was no significant difference between two subsequent deflected shapes of the wall.

Rauhut (1969), in a discussion of Haliburton's paper, showed that in general Haliburton's method did not give good agreement with experimental results due to the omission of wall friction and the linear distribution of deflection.

Bowles (1988) also modelled the sheet pile wall as beam elements, but utilised the concept of subgrade reaction for soil below the excavation level. The matrix stiffness method of analysis was used to obtain the solution. Ng (1984) demonstrated how a program for the matrix method can be implemented on a programmable calculator.

5.4.6 Advantages and Limitations

An advantage of the 'beam on elastic foundation' approach over limit analysis is its ability to account for structure flexibility and soil stiffness. Thus the effects of stress redistribution in soil as a result of differential structural deflections are accommodated. Although the magnitude of the shear forces and bending moments in the wall and the strut or anchor loads are not very sensitive to the values of spring stiffness (constant) used in the analysis, the predicted deformations of the supporting wall are.

The limitations of the 'beam on elastic foundation' approach are :

(a) There is inherent difficulty in determining the appropriate spring stiffnesses (constants) of the soil for analysis as these are not fundamental soil properties.

(b) The method cannot directly simulate construction sequence, unusual initial soil stresses, and development of wall friction.

(c) The method does not yield surface movements behind the retaining structure.

Haliburton's method of utilising the relationship between lateral earth pressure and displacement is a slightly better way of modelling soil-structure interaction. However, it should be noted that although the influence of the horizontal earth pressure at rest on the deformation of the wall is taken into account, the uncoupled nature of the soil model and the difficulty in establishing the relationship between lateral soil pressure and deformation limits the applicability of this method in practice.

5.5 BOUNDARY ELEMENT METHOD

Wood (1979) developed a boundary element computer program to analyse pile groups subject to lateral loads and subsequently extended the program to include the analysis of multi-propped sheet pile and diaphragm walls. The wall was assumed to be a series of discrete beam elements attached to the soil at common nodes. Only the compatibility of horizontal displacements between the wall and the soil was maintained at these nodes. The soil was modelled as an inhomogeneous elastic continuum utilising an approximate extension of Mindlin's solution (Mindlin, 1936), based on the assumption that the displacement is a function of the local values of the elastic parameters. The initial disturbing force was determined from a consideration of the coefficient of earth pressure at rest and the out-of-balance earth pressure on either side of the wall. The earth pressures were derived and compared with the active and passive pressure envelopes obtained from classical theory. An iterative procedure was adopted to ensure that the final soil reaction always lies within the active-passive range. No account was taken of arching. At each excavation stage the incremental movements were computed and the summation of the incremental movement results were given. The application of this program to the prediction of the performance of a diaphragm wall is given by Wood & Perrin (1984).

A similar program was also developed by Pappin et al (1985). The soil on each side of the wall was modelled as an elastic solid, the flexibility of which was generated either by way of the integrals of the Mindlin equations, or by interpolation and sealing of flexibility matrices calculated for a simplified soil profile using finite element methods. A semi-empirical formulation was also developed to allow for variations in soil stiffness with depth. The wall was modelled by a series of elastic beam elements. In addition, the earth pressures were limited to within active and passive limits. Arching is permitted by considering the soil forces acting on the wall compared with the forces consistent with possible failure surfaces within the soil. Other features that can be accommodated by the program include struts, anchors and the effects of surcharges. This program has been applied to the prediction of the performance of the basement for the new Hong Kong Bank headquarters building (Fitzpatrick & Willford, 1985; Humpheson et al, 1986).

The boundary element method developed by Pappin et al takes arching into account, and is more rigorous than Wood's approach in the formulation of the soil-structure interaction problem. It also appears to be able to overcome the shortcomings of Haliburton's method. Boundary element programs, although not as general as the finite element methods, have a significant advantage over the latter in that they are much cheaper to apply, and sufficiently accurate for most design problems. They are particularly suitable for parametric studies. The shortcoming of the boundary element methods is their inability to provide information on the surface movement behind retaining structures. Other methods are required to correlate the deflections of retaining structures with surface movements (see Section 9.3).

5.6 FINITE ELEMENT METHOD

A complete method for solving soil-structure interaction problems is the finite element method which incorporates a continuum soil model. This method offers the designer an analytical tool which can simulate almost all of the complex facets of the strutted or tied-back wall except unquantifiable variables such as workmanship or geological uncertainties. The techniques involved in the application of the finite element method to the analysis of soil-structure interaction have been discussed by Clough (1972a), Clough & Tsui (1977) and the Institution of Structural Engineers (1989).

Tsui & Clough (1974) showed that the assumption of plane strain usually adopted in finite element analysis of strutted and anchored walls cannot be arbitrarily extrapolated to many practical problems, particularly those involving soldier pile walls or light sheet pile walls. Diaphragm walls, however, were shown to approximately satisfy the conditions assumed in a plane strain analysis. For all types of walls that have prestressed supports, a chart (which is reproduced in Figure 13) can be used to provide a qualitative index to the applicability of a plane strain analysis. The deviation of three-dimensional pressures from a plane strain distribution is defined in terms of the soil stiffness, the wall stiffness and the tie-back spacing. The chart shows that the stiffer the wall, the closer the tie-back spacing, and the softer the soil, the more accurate is the assumption of plane strain conditions. To simulate discontinuous wall elements such as soldier piles, struts or tie-backs in a plane strain analysis, the bending stiffness of the soldier piles in the wall and the axial stiffness of the struts or tie-backs have to be represented on a 'unit length' basis. Analyses using this representation yield results that are characteristic of the average values between those at the supports and those at the mid-point between the supports.

Clough and Mana (1976) demonstrated that simulation of excavation in a finite element analysis may be satisfactorily accomplished by a number of different procedures, provided that appropriate input soil parameters are used. Both a nonlinear elastic and an elasto-plastic soil model can be made to arrive at the same predicted excavation behaviour. However, changes in excavation support design, which will bring about unanticipated performance, often take place during construction. Isolation of the finite element analysis from the construction stages can easily lead to erroneous performance predictions, because important later occurrences are not accounted for. Hence, the finite element analysis should be used in parallel with an observational procedure (Peck, 1969a), and allowance should be made for re-analyses to be performed to accommodate soil parameter re-estimates and construction changes. Updated soil parameters are best obtained by comparing early observed performance with comparable finite element predictions. The design engineer should preferably be involved during construction to monitor the performance of the excavation, verify the design assumptions, and to effect any necessary amendments to the design.

The two single most important variables required in a finite element analysis are the stiffness of the ground, which mainly affects the displacements, and the magnitude of the initial horizontal stresses, which affect both the displacements and the bending moments.

It has been found that the values of the soil stiffness determined from small-scale insitu tests or from laboratory tests are less than those back-analysed from field measurement of the performance of an excavation. The reduction in stiffness has been attributed to sample disturbance or ground disturbance upon insertion of the testing equipment. However, recent field and laboratory studies have demonstrated that conventional methods of strain measurement can lead to very significant underestimates of soil stiffness. Some soils have been found to exhibit nonlinear stress-strain behaviour, with a stiffness at small strain (about 0.01%) higher than that normally measured in conventional laboratory tests (at about 0.1% strain). The influence of this small strain nonlinearity in soil-structure interaction has been studied by Jardine et al (1986) using a finite element analysis. The nonlinear soil model that was used predicted a pronounced settlement trough close to a strutted excavation, which agrees well with that observed in the field. However, such behaviour cannot be predicted by the conventional linear elastic soil model.

Martin (1977) showed that the deformation modulus of soils derived from insitu weathering of igneous and metamorphic rocks can be approximated by the pressuremeter modulus, and that a linear relationship exists between the logarithm of Standard Penetration Test (SPT) 'N' value and the logarithm of pressuremeter modulus. A similar relationship has also been found for Hong Kong residual soils and saprolites. The Young's modulus, E, of such soils is usually correlated with the N values using pressuremeter tests (Chiang & Ho, 1980), plate loading tests (Sweeney & Ho, 1982) and full-scale tests. Endicott (1984) showed that the E/N ratio for completely weathered granite back-analysed from the performance of excavations is greater than that obtained from pressuremeter tests, which in turn is greater than that from triaxial tests on undisturbed samples of completely decomposed granite. Chan and Davies (1984) have given a chart which shows the correlation of soil modulus with SPT 'N' values (Figure 14). Ranges of E/N ratio have also been given by Fraser (1985), Ku et al (1985), Ku et al (1985) and Whiteside (1986).

The insitu horizontal stress in the ground is related to the insitu vertical stress by K_o, the coefficient of earth pressure at rest. From conventional soil mechanics, it is well known that K_o, being dependent on stress history, is not a fundamental soil property. For a soil in the normally consolidated (i.e. low K_o) state, the stress change required to reach the active state is small compared with that needed to mobilise passive resistance. The contrary is true for a soil in the overconsolidated (i.e. high K_o) state (Figure 15). Therefore, the horizontal displacements of the wall and surface settlements behind the wall are dependent on the magnitude of the insitu stress. The existence of a high insitu horizontal stress would likely result in large wall displacements and surface settlements. For excavated walls in soils with a high initial K_o value ($K_o \geq 2$), the prop forces and wall bending moments can greatly exceed those predicted by limit equilibrium methods (Potts & Burland, 1983b; Potts & Fourie, 1984).

Geoguide 1 : Guide to Retaining Wall Design (GCO, 1982) recommends that the value of K_o should be not less than 0.5 for the design of walls retaining Hong Kong soils derived from insitu rock weathering. This is despite a mean K_o value of 0.32 for 'undisturbed' completely decomposed granite obtained by Chan (1976) in the laboratory. It could be argued that such low values were due to the irreversible effects of stress relief. However, there is other evidence which suggests that K_o could be this low in residual soils and saprolites. From the observed performance of existing structures, Lumb (1979) suggested that active and at-rest pressures in Hong Kong residual soils and saprolites should be smaller than would be estimated by conventional calculations. Endicott (1982) obtained earth pressure coefficients of 0.19 and 0.17 for wall movements of 6 mm and 12 mm respectively in a completely weathered granite and deduced an at-rest pressure coefficient between 0.21 and 0.25. The weakening theory for residual soils and saprolites proposed by Vaughan & Kwan (1984) has predicted that K_o should be lower than that obtained from Jaky's classical relationship $K_o = 1 - \sin \emptyset'$. Howat (1985) measured very low minimum values of active earth pressure coefficient K_a in granitic soils and showed that K_o pressures may be estimated by the following formula (Howat & Cater, 1985), which takes into account the unconfined compressive strength, q_u :

$$\sigma_h - u_w = (\sigma_v - u_w - q_u)(1 - \sin \emptyset') \quad . \; . \; . \; . \; . \; . \; . \; . \; . \quad (1)$$

where σ_h = horizontal total stress,

σ_h = vertical total stress,

u_w = pore water pressure.

The above equation supports Vaughan & Kwan's prediction that K_o should be less than $(1 - \sin \emptyset')$.

Although the data available from the literature suggest that the at-rest earth pressure of Hong Kong residual soils and saprolites could be low, no direct measurements of K_o in the field have been reported. K_o values may be deduced by back-analysing carefully instrumented excavations, but such values are likely to be on the low side because, by nature of excavations, movements can take place even when very stiff supports are provided. The use of self-boring pressuremeters for measuring the insitu horizontal stress should be considered. However, it should be noted that the insertion of an instrument into saprolites may destroy any bonding and alter the insitu stresses. Therefore, the effect of disturbance should be assessed.

The advantage of the finite element method in the analysis of earth retaining structures lies in its ability to predict both earth pressures and deformations with a minimum of simplifying assumptions. Both structure and soil are considered interactively, so that the effects of structural flexibility are taken into account. Limitations in using the method primarily derive from the inability to prescribe appropriate constitutive laws for the soil, and to determine the parameters needed for the constitutive models. However, when utilising the observational procedure (Peck, 1969a), the finite element method of analysis can be expected to yield useful design information and accurate performance predictions for soil-structure interaction problems.

6. CANTILEVERED OR UNBRACED WALLS

6.1 GENERAL

A cantilevered wall derives its support from the passive resistance below the excavation level (or 'dredge line') in order to balance the active pressure from the soil above that level. This type of wall is economical only for moderate wall heights, since the required section modulus increases rapidly with increase in wall height (the bending moment increases with the cube of the cantilevered height of the wall). The lateral deflection of this type of wall can be relatively large. Lowering of the excavation level, e.g. by construction activities, erosion and scour in front of the wall, has to be carefully controlled, since the stability of the wall depends primarily on the passive resistance developed in front of the wall.

Cantilevered walls of the flexible type, such as sheet pile walls, are normally used as temporary supports, e.g. support for the initial stage of a strutted excavation. When used as permanent supports, they are normally of the rigid type; in Hong Kong, these usually take the form of caisson walls (see Section 3.4). When used to retain an excavation of substantial height, unpropped caisson walls are usually socketed into rock.

In the design of cantilevered walls, there are four primary areas of geotechnical consideration :

(a) ultimate limit states,
(b) serviceability limit states,
(c) earth pressures for structural design, and
(d) design water pressures.

In many cases, the consideration of serviceability limit states governs the geotechnical design.

6.2 ULTIMATE LIMIT STATES

6.2.1 Types of Limit States

The following ultimate limit states are usually considered in geotechnical design :

(a) Overall instability : the embedment depth of a cantilevered wall should be adequate to prevent overturning of the wall.

(b) Foundation failure : the cantilevered wall should penetrate a sufficient depth to prevent base heave in cohesive soils.

(c) Hydraulic failure : the depth of penetration should be adequate to prevent hydraulic failure (i.e. piping or heave) in cohesionless soils.

6.2.2 Overall Stability

The overall stability of cantilevered walls is usually assessed using limit equilibrium methods of analysis in which the conditions of failure are postulated, and a factor of safety applied to ensure that such a failure state does not occur.

A cantilevered wall cannot be in equilibrium without its lower end being fixed, as shown in Figure 16(a). For a cantilevered wall rotating about a point near its toe, the theoretical pressure distribution is of the fixed-earth form, as shown in Figure 16(b), when the wall is at the point of collapse. The pressure distribution shown is considerably idealised, particularly at the point of rotation, O, at which it is assumed that there is an instantaneous change from full passive pressure in front of the wall to full passive pressure behind the wall. The calculation of the depth of embedment corresponding to this pressure distribution involves equating the horizontal forces and taking moments about O to obtain two equations with two unknown depths, d and z (Figure 16(b)). The equations obtained are generally rather complicated, one being a quadratic and one a cubic expression in both d and z. The solutions for d and z are usually obtained by a process of iteration.

In view of the considerable algebraic complexity of the full method, the simplification illustrated in Figure 16(c) is widely used in practice. This simplification assumes that the difference between the passive resistance at the back of the wall and the active pressure in front acts as a concentrated force, R, at the toe. By taking moments about the toe (thereby eliminating the force R from the equation), the depth of embedment in the simplified model, d_o, is found. Because of this simplification, the value of d_o is slightly less than the value of d obtained from the full method and is more likely to be nearer to d-z/2. To account for this, the usual practice is to increase d_o by a small amount (up to 20%). A simple check is usually made to ensure that the additional embedment is sufficient to provide a force at least as large as the assumed force, R. This can be achieved from a simple consideration of horizontal equilibrium. In most cases, a 20% increase in embedment will be conservative. This small additional increase in the value of d_o is specifically to account for the simplification, rather than to provide a factor of safety.

There are a number of ways of applying the factor of safety for the overall stability of cantilevered as well as propped cantilevered walls embedded in soils. These are :

(a) Factor on embedment depth. A factor is applied to the calculated embedment depth at limiting equilibrium, as explained in the US Steel Sheet Piling Design Manual (United States Steel Corporation, 1975) and the BSC General Steel Piling Handbook (British Steel Corporation, 1986).

(b) Factor on moments based on gross pressure. This method is described by the Institution of Structural Engineers (1951) and NAVFAC (1982b), and is given in Geoguide 1 (GCO, 1982) for checking the adequacy of penetration of a strutted excavation. It consists of factoring only the gross passive pressure. The water pressure is not factored.

(c) Factor on moments based on net total pressure. This method is advocated by the BSC General Steel Piling Handbook. The net horizontal pressure distribution acting on the wall is derived, and the factor of safety is defined as the ratio of the moment of the net passive forces to the moment of the net active forces.

(d) Factor on moments based on net available passive resistance. This method was developed by Burland et al (1981), and is analogous to the calculation of the bearing capacity for a strip load. The factor of safety is taken as the ratio of the moment of the net available passive resistance to the moment activated by the retained material, including water and surcharge.

(e) Factor on shear strength on both active and passive sides. The shear strengths of the soils are reduced by dividing c' and tan Ø' by a factor of safety. These reduced (or mobilised) shear strength parameters are then used to calculate the active and passive pressures.

(f) Factor on shear strength on passive side only. This method is recommended by CIRIA (1974), and is given in Geoguide 1 (GCO, 1982) for design of cantilevered walls. The shear strength of the soil on the passive side is factored, but no factor of safety is applied on the active side.

A comparison of the factors of safety defined by methods (b), (c), (d) and (e) was carried out by Potts & Burland (1983b) and Symons (1983). Also, the factors of safety defined by methods (a), (b), (d) and (e) were compared by Padfield & Mair (1984). These comparisons reveal that there is no unique relationship between the results obtained by different definitions of factor of safety. The choice of method is largely one of convenience, and the factor of safety is related to the method used, provided that the methods are applied consistently. Factors of safety appropriate to methods (a), (b), (d) and (e) are recommended by Padfield & Mair (1984), and these are shown in Table 4.

The following are specific comments related to each method of defining factor of safety. Method (a) is empirical and should always be checked against one of the other methods. Method (b) may give excessive penetrations at lower angles of shearing resistance ($Ø' < 20^{o}$). Therefore, different factors of safety are recommended for different ranges of Ø'. Method (c) gives very high factors of safety compared with other methods, and is not recommended for design unless an exceptionally high factor of safety is specified. Compared with other methods, method (d) appears to give a consistent lumped factor of safety throughout the practical range of soils and wall geometries. Unlike method (d), the same conceptual framework for slope stability analysis can be applied if method (e) is adopted. However, an anomalous situation may occur in methods (e) and (f) where sloping ground occurs. Owing to the different factors of safety adopted for slope stability and for overall stability of the wall, the angle of shearing resistance of the soil, after factoring, may be less than the angle of the sloping ground. This can give rise to difficulties in the evaluation of active and passive pressures. For method (f), Geoguide 1 (GCO, 1982) recommends that a safety factor of 3 should be applied to the shear strength of soil on the passive side. This large factor of safety on shear strength, which is generally considered to be conservative, is not related to the stability criterion. Instead, it is applied for the purpose of limiting wall deformation.

Malone (1982) showed that, for the case of level ground on both sides of a cantilevered wall with zero water pressure, the safety factor on shear strength ($\emptyset'=35^{o}$) on the passive side can be doubled from 1.5 to 3.0 by a moderate increase in soil embedment ratio (= 1 + d/H, where d and H are as defined in Figure 16(a)) from 1.8 to 2.15. Pope (1983) showed for the same situation that a safety factor of 1.5 applied to shear strength ($\emptyset'=40^{o}$) on both active and passive sides (method (e)) results in smaller embedment depth than by applying a safety factor of 3 on the passive side only (method (f)). The same conclusion can be drawn when the design water level is at one third the retained height of the wall.

Bica & Clayton (1989) compared a range of limit equilibrium design methods for cantilevered walls in granular materials. It was found that apart from the definition and magnitude of factor of safety, the assumptions made in the limit equilibrium analysis (e.g. wall friction and method of calculating earth pressures and their distributions) and the choice of method for determining soil and soil/wall shear strength parameters (e.g. use of plane strain test or triaxial compression test) can significantly affect the results of design.

For cantilevered walls socketed into rock, overall stability depends on the lateral load capacity of the rock socket. The methods used to assess the load capacity of the rock socket were reviewed by Greenway et al (1986). Failure of the rock socket by movement along discontinuities in rock should be considered, as well as bearing failure of the intact rock mass. The persistence of a rock joint is usually very difficult to assess during ground investigation. Therefore, it is usual practice to carry out rock joint discontinuity surveys during construction.

6.2.3 Foundation Failure

Bottom heave is a problem which primarily occurs in soft to medium clays. The failure is analogous to a bearing capacity failure, the difference being that stresses in the ground are relieved instead of increased.

Two types of analysis are available for calculating the factor of safety against base heave, as shown in Figure 17. The method of Terzaghi (1943) is applicable for shallow and wide excavations, whereas that by Bjerrum & Eide (1956) is suitable for deep and narrow excavations. Neither analysis is exact in that the effect of any wall segment penetrating beneath the excavation (thereby resisting soil movement) is not accounted for. The factor of safety against base heave is usually specified to be not less than 1.5. Field vane shear strength corrected in accordance with Bjerrum (1973) is often used in the analysis. If uncorrected vane shear strength is used, the actual factor of safety with respect to base heave may be very close to unity (Aas, 1985).

Where the safety factor falls below 1.5, substantial wall movement can occur as a result of yielding in the subsoils (Clough et al, 1979; Mana & Clough, 1981). Where limiting movement is the governing criterion, a safety factor not less than 2 is usually required (Clough et al, 1979; Mana & Clough, 1981). If soft clay extends to a considerable depth below the excavation, the beneficial effects of even relatively stiff walls in reducing deformation have been found to be small. However, if the lower portion of the wall is driven into a hard layer, the effectiveness of the wall in reducing deformation is increased appreciably (see Section 9.2.6).

A base heave analysis for wide excavation in anisotropic clay was given by Clough & Hansen (1981) and is illustrated in Figure 18. It appears that unless the 'passive strength' (undrained shear strength at 90^{o} stress reorientation) of a clay is less than 80% of its 'active strength' (undrained shear strength at 0^{o} stress reorientation), the base heave factor of safety will be within 10% of the value obtained by assuming isotropy with respect to the 'active strength'. The marine soils of Hong Kong appear to show only a slight amount of anisotropy (Lumb, 1977). Hence, anisotropy is normally not considered. The use of field vane shear strength, corrected in accordance with Bjerrum (1973), in an isotropic analysis is probably adequate in most cases.

Although it is rare, heave may also occur in cohesionless soils. The factor of safety against base heave in these materials can be estimated from Figure 19.

6.2.4 Hydraulic Failure

Where groundwater exists above the base of the excavation, and if the toe of the cantilevered wall does not penetrate into an impermeable layer, flow will occur under the wall and upward through the base of the excavation. Instability or gross deformation of the base, involving piping or heave, occurs if the vertical seepage exit gradient at the base of the excavation is equal to about unity. To prevent hydraulic failure, the wall must penetrate a sufficient depth below the base of the excavation. Design charts for wall penetration required for various safety factors against heave or piping in isotropic sands and in layered subsoils are given in NAVFAC (1982a). These charts are reproduced in Figures 20 and 21. A design chart is also given by the Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 1985) which shows that the seepage exit gradient for a circular excavation, in the middle section of the sides of a square excavation and in the corners of a square excavation, is 1.3, 1.3 and 1.7 times that for a strip excavation respectively. For clean sand, exit gradients between 0.5 and 0.75 will cause instability of the base. To avoid this, wall penetration for a safety factor of 1.5 to 2 against piping or heave is usually provided.

6.3 SERVICEABILITY LIMIT STATES

The design of cantilevered walls is currently based on limit equilibrium calculations incorporating a high factor of safety to account for both failure and deformation considerations. This approach does not necessarily achieve the aim of limiting deformation (see Section 4.3).

The 'beam on elastic foundation' approach is usually employed in deformation analyses. Bowles (1988) modelled the sheet pile wall as beam elements and the soil as elastic springs (the springs can be nonlinear). Pope (1983) proposed a model for deformation analyses in which the ground pressure/displacement curve is approximated by a series of nonlinear springs and the wall as a series of linear elastic beam elements. He showed that, by careful selection of the coefficient of earth pressure at rest and the spring stiffnesses (constants), good agreement with the observed deflected shape could be obtained. Pope also showed that there may be a substantial component of rotation for a caisson wall that is not socketed into rock. He considered that for a design based on stability considerations alone, even by applying a factor of safety of 3 on soil strength on the passive side, unacceptably large deformations may still occur if the caissons are not socketed into rock.

The boundary element method (see Section 5.5) and the finite element method (see Section 5.6) can also be used in the deformation analyses. These methods are superior to those using the subgrade reaction approach, because soil-structure interaction effects can be better taken into account. The lateral displacements of the wall predicted by linear elastic spring models can only be regarded as rough estimates, and need to be checked by field measurements.

6.4 EARTH PRESSURES FOR STRUCTURAL DESIGN

In the structural design of the cantilevered wall, which involves calculation of bending moments and shear forces, the magnitude as well as the distribution of earth pressures corresponding to structural failure are required. For this purpose, classical active and passive earth pressure distributions, which correspond to the limit state condition, are usually modified in some arbitrary manner (e.g. by the design safety factor against overall stability). There are three alternative methods of calculating the design bending moment :

(a) The maximum bending moment in the wall is calculated at limiting equilibrium using unfactored soil parameters (Figure 22(a)). The depth of embedment in the design configuration is greater than that corresponding to the limiting equilibrium condition with unfactored parameters, but the additional depth is not used in the calculations. The bending moment distribution at limiting equilibrium is then asssumed to be equal to that acting in the 'working condition', at which a factor of safety is provided against structural failure (Figure 22(b)).

(b) An earth pressure distribution, which is usually obtained by applying a safety factor to the earth pressures at limiting equilibrium, is assumed to act on the design configuration of the wall. The wall bending moment distribution is then calculated by considering moment and force equilibrium. Passive pressure at excavation level is, however, not fully mobilised in the analyses and hence the real pressure distribution is not properly modelled. The different methods of applying factor of safety to obtain an earth pressure distribution are discussed in Section 6.2.2.

(c) The earth pressure distribution which acts on the design configuration of the wall is calculated by using analytical methods such as those based on the 'beam on elastic foundation' approach (see Section 5.4), and the boundary element and finite element methods (see Sections 5.5 and 5.6).

Methods (a) and (b) are relatively simple for manual computation. However, these methods do not simulate the real soil-wall system, unlike the methods in (c) which take into account the soil-structure interaction. Another advantage of the methods in (c) is that the lateral displacements of the wall required in the displacement analyses are also calculated.

If wall deflection is not the prime consideration, method (a) is preferable to method (b) because the passive resistance in front of the wall close to excavation level is likely to be fully-mobilised even under working conditions, and the maximum bending moment is strongly influenced by this (Padfield & Mair, 1984).

When method (a) is applied, the design depth of embedment which exceeds that required to give limiting equilibrium is neglected in the calculation of bending moment, as shown in Figure 22 (Padfield & Mair, 1984). For a reinforced concrete cantilevered wall, the reinforcement should not be curtailed at the point where the calculated bending moment is zero. It should be taken to the bottom of the cantilever, and on both faces, to allow for the small reverse bending moments which may occur near the toe.

Another problem with the application of method (a) concerns the assumption on water pressures. Since the design embedment depth is greater than that required to give limiting equilibrium, the seepage path round an impermeable wall with the smaller depth of embedment assumed for the bending moment calculation differs from the seepage path corresponding to working conditions. Strictly speaking, the entire embedment depth of the wall should be used in the calculation of seepage pressures, even for the bending moment calculation. However, this tends to be somewhat cumbersome and is usually not allowed for. In general, the error in neglecting the additional seepage length for the calculation of water pressures is rather small. Considerations relating to design water pressures are given in Section 6.5.

Bica & Clayton (1989) found that, if plane strain ϕ' values are used for granular materials, many limit equilibrium methods can give unsafe predictions of maximum bending moments when compared with experimental observations. If the more conservative triaxial compression ϕ' values are used, then a few methods can give reasonable estimates of the moments. It is of interest to note that the available test results show an increase in maximum bending moment as the depth of embedment is increased beyond that required purely for equilibrium.

In the structural design of caisson walls, because of difficulty in establishing the 'true' rockhead levels at design stage, calculations of bending moments and shear forces are often carried out for a range of depths to rockhead. Different reinforcement details are shown on the drawings for different socket lengths, the dimensions of which are determined by inspection of the materials exposed during caisson excavation.

6.5 DESIGN WATER PRESSURES

Proper evaluation of water pressure and its effects is of utmost importance in the design of excavation support systems.

For a retaining wall which is impermeable, either by nature of the wall material or as a result of the provision of a groundwater cut-off by means of grouting, a difference in water level may exist behind and in front of the wall. Where a relatively impermeable soil layer (e.g. a marine clay stratum) or an unweathered rock mass of low permeability is present immediately below the toe of the retaining wall, a hydrostatic water pressure may act on the wall. Groundwater flow underneath the wall can give rise to different loading conditions.

The groundwater flow pattern around an excavation, which can affect the water pressures, the earth pressures on the active and passive side of the retaining wall, and the piping and heave potential of the excavation, depends on the ground conditions. Figure 23(a) shows a typical flow-net around a retaining wall in homogeneous soil under steady-state seepage condition. The groundwater table shown in this figure is close to the ground surface, a situation which is commonly encountered in excavations carried out in the reclaimed areas of Hong Kong. A simplified water pressure distribution, as shown in Figure 23(b), was proposed by Padfield & Mair (1984) for the design of cantilevered and propped walls. This simplified method assumes the hydraulic head varies linearly down the back and up the front of the wall. This greatly facilitates the computation of both the active and passive pressures acting on the wall. Potts & Burland (1983a) found that the results of a soil-structure interaction analysis based on such a simplified method were similar to those from an analysis in which the water pressures had been derived from a flow-net.

Kaiser & Hewitt (1982), in their discussion of the effect of groundwater flow on the stability and design of excavations, illustrated the influence of soil layering, hydraulic anisotropy and continuous and discontinuous lenses of low permeability material on the groundwater flow pattern. The resultant water pressures in these cases can exceed that in the homogenous soil condition (Figure 24). A knowledge of the soil fabric was considered to be of utmost importance by Padfield & Mair (1984), who showed that pervious silt or sand partings within a clay stratum may convey water at hydrostatic pressure to the base of the wall.

The work described above demonstrates the need for proper ground investigation in order to gain a good understanding of the ground and groundwater conditions for the design of excavations. For conditions other than that of a homogeneous soil, the simplified water pressure distribution shown in Figure 23(b) may not be sufficiently accurate, and a proper flow-net analysis using established techniques described in treatises on groundwater flow (e.g. Cedergren, 1977) should be carried out. The active and passive earth pressures should be evaluated using limit equilibrium methods (e.g. the trial wedge method and the generalised procedure of slices (Janbu, 1957)), with due account taken of the piezometric pressures on the potential failure surface and the soil/wall interface. It should be noted that the presence of water pressures (which reduce effective stresses in soil) will lead to an increase and decrease in active and passive earth pressures respectively. Another adverse effect is the possibility of hydraulic failure (i.e. piping or heave) due to upward hydraulic gradients in front of the retaining wall (see Section 6.2.4).

For a retaining wall composed of vertical structural elements spaced horizontally apart (e.g. caissons, soldier piles and pipe piles), some throughflow of groundwater can occur. If the vertical elements are closely spaced, 'damming' of the groundwater can result in a rise in groundwater level. A method for estimating the effect of piles and caissons on groundwater flow was given by Pope & Ho (1982). Drainage provisions behind an impermeable wall can also affect the flow pattern in the ground (Padfield & Mair, 1984) and their effects should be properly assessed.

Infiltration into the ground can sometimes cause a rise in groundwater level. This can increase the water load acting on the excavation support system in some situations. An example of this is where the rise in water level occurs within the retained height, which may take place if the original groundwater level is close to the ground surface or where an impermeable layer is present within the retained soil mass. Even for a retaining wall which has a vertical drainage layer at its back, steady-state seepage can still occur when there is sufficient water infiltrating into the retained soil, causing an increase in pore water pressures and a reduction in soil shear strength. The steady-state seepage condition is most likely to occur in a soil with a relatively low permeability, as rainfall of moderate intensity can supply sufficient water to sustain the seepage flow. The effect of infiltration should be taken into account in design.

Deformation of the ground adjacent to excavations may cause breakage of water-carrying services. In some cases, the amount of water released can adversely affect the stability of excavations. Therefore, it is important that the location, size and condition of such services in the vicinity of a proposed excavation are ascertained. In situations where significant water flow can occur in the event of breakage, it will be prudent to allow for the possible increase in water load in the design. Alternatively, consideration should be given to diverting water-carrying services prior to excavation.

7. STRUTTED OR BRACED WALLS

7.1 SINGLE-LEVEL STRUTTED WALLS

7.1.1 General

The behaviour of single-level strutted walls is similar to that of single-level anchored walls in which the anchors are installed horizontally. The effect of inclined anchors will be discussed in Section 8.1.1. The factors which influence the behaviour of an anchored flexible wall were reviewed by Bjerrum et al (1972). As a result of wall deflection, the earth pressures acting on a flexible wall such as a sheet pile wall are very much different from those predicted by classical earth pressure theories. On the back of the wall, the pressures at the top and bottom are increased due to the smaller outward deflections at these locations, while the pressures between the excavation and anchor levels tend to be reduced due to the relatively large outward deflections there. This redistribution of earth pressure around a flexible wall may be considered the result of arching. Rowe (1952) found in his model tests on sheet pile walls that the arching effects can be considered to consist of three parts : that due to flexure below the excavation level, that due to flexure above the anchor level, and that due to reduction in earth pressure between the excavation and anchor levels. The flexure below the excavation level is the most stable and reliable source of moment reduction. The flexure above the anchor level could be reduced or destroyed by additional anchor yielding after excavation, and thus cannot be relied upon in all cases. The moment reduction due to pressure reduction between the anchor and excavation levels is unstable and could be destroyed by anchor yield (or elastic shortening of the struts) and backfill settlement, except in the case of piles encastré at the anchorage.

As in the design of cantilevered walls, there are four primary areas of geotechnical consideration :

(a) ultimate limit states,
(b) serviceability limit states,
(c) earth pressures for structural design, and
(d) design water pressures.

The consideration of ultimate limit states rather than serviceability limit states usually governs the design.

7.1.2 Ultimate Limit States

(1) Overall Stability. The overall stability of strutted walls, as in cantilevered walls, is also assessed using limit equilibrium methods of analysis.

There is a choice between adopting a free-earth or a fixed-earth procedure for analyzing overall stability. For a propped wall, consideration of overall stability in terms of rotation about the prop position is only applicable to free-earth support conditions. Where fixed-earth support conditions apply, provided the wall is adequately propped and designed to resist the shear forces and bending moments acting upon it, there is no failure mechanism relevant to an overall stability check

(Padfield & Mair, 1984). Propped walls designed on the assumption of fixed-earth conditions are therefore not considered here. Assuming free-earth support conditions, the depth of embedment is usually determined by taking moments about the position of the prop (Figure 25). The different methods of applying factor of safety are discussed in detail in Section 6.2.2.

(2) Foundation Heave and Hydraulic Failure. Considerations of foundation heave and hydraulic failure are discussed in Sections 6.2.3 and 6.2.4 respectively.

7.1.3 Serviceability Limit States

As in the case of cantilevered walls, the design of propped walls is often based on limit equilibrium calculations, using a high factor of safety to ensure adequate stability and to restrict soil and wall movements to acceptable levels. A more detailed approach is to employ the 'beam on elastic foundation' approach (see Section 5.4) to estimate the lateral displacements of the wall. However, neither method takes into account the insitu stress state of the soil prior to excavation. Both the boundary element and finite element methods (see Sections 5.5 and 5.6) can be used to investigate the influence of initial stresses in the soil on the behaviour of single-level propped retaining walls.

An elasto-plastic constitutive law for the soil was employed by Potts & Fourie (1984) in a finite element analysis to model the behaviour of a propped wall. The results of their numerical analysis showed that, if a suitable factor of safety against overall stability is adopted for an excavated wall in soil with a low K_o value (K_o=0.5), the lateral wall movements are of small magnitude and therefore consistent with current design philosophy. However, for an excavated wall in soil with a high K_o value (K_o=2), relatively large lateral wall movements were predicted, even at shallow excavation depths with large factors of safety against overall stability. Therefore, it is important that the insitu horizontal stress of the soil be carefully assessed in the design of excavations. Where necessary, deformation analyses should be performed to evaluate the sensitivity of the design to the assumed value of K_o.

7.1.4 Earth Pressures for Structural Design

The structural design of a propped cantilever wall involves the calculation of bending moments and shear forces for the wall and the prop force from the distribution of earth pressures acting on the wall.

(1) Wall Bending Moment. There are three alternative methods of calculating the design bending moment :

(a) Assuming free-earth support conditions, the bending moment distribution is calculated at limiting equilibrium with unfactored soil parameters, as shown in Figure 26(a). As in the case of cantilevered walls, the design depth of embedment is greater than that corresponding to the limiting equilibrium condition with unfactored soil parameters, but the additional depth is not used in the calculations.

(b) An earth pressure distribution, which is usually obtained by applying a safety factor to the earth pressures at limiting equilibrium, is assumed to act on the design configuration of the wall (Figure 26(b)). This is in effect the 'fixed-earth support' analysis. The different methods for applying the factor of safety are discussed in Section 6.2.2. It should be noted that the fixed-earth support analysis, which was developed for flexible anchored sheet pile walls embedded in sand (Terzaghi, 1953), may be unsafe when applied to more rigid walls in clays due to the long-term deformation characteristics of such soils.

(c) The earth pressure distribution which acts on the design configuration of the wall is calculated by using analytical methods such as those based on the 'beam on elastic foundation' approach (see Section 5.4), and the boundary element and finite element methods (see Sections 5.5 and 5.6).

Rowe (1955a) used a power series solution to solve the flexure equation for sheet pile walls (see Section 5.4.2). On the active side of the wall, triangular and constant active earth pressure distributions are assumed above and below the excavation level respectively. On the passive side, a coefficient of subgrade reaction which increases linearly with depth is assumed. Curves showing the relationship between the degree of flexibility of a sheet pile wall and the reduction of bending moment, expressed as a ratio of the maximum moment calculated assuming free-earth support conditions, were established for both medium dense and very dense granular soils. Rowe's moment reduction theory was recommended in CIRIA (1974). Rowe then extended his flexibility method of analysis to the solution of sheet pile walls encastré at the anchorage (Rowe, 1955b), sheet pile walls at failure (Rowe, 1956), sheet pile walls in clay (Rowe, 1957a) and sheet pile walls subject to a line resistance above the anchorage (Rowe, 1957b). Rowe's work (1952-1957) was reviewed by Lee & Moore (1968). A similar method of analysis was also given by Richart (1955). Schroeder & Roumillac (1983) found that for flexible anchored walls with a sloping excavation line the calculated moments may be reduced by extrapolating Rowe's moment reduction curve.

A finite difference method of solving the flexure equation for sheet pile walls was given by Palmer and Thompson (1948).

Turabi & Balla (1968a) published a distribution theory of analysis by approximating the soil below the excavation level by five linear springs. A triangular active earth pressure distribution above the excavation level was assumed, similar to that assumed by Rowe (1955a). However, in contrast to Rowe (1955a), the application of a constant active pressure below the excavation level was considered not justified, as the pressure distribution below the excavation level can be obtained from the spring reactions. They later extended their theory by removing the assumption of linear pressure distribution above the excavation level (Turabi & Balla, 1968b). In this respect, their method is an improvement on Rowe's method.

Haliburton (1968) presented a discrete element method in which the nonlinear earth pressure-deflection relationship is approximated by two straight lines with different slopes. The important influence of the at-rest earth pressure on the deformation of the wall is taken into account. Popescu & Ionescu (1977) modified Haliburton's method by approximating the earth pressure-deflection relationship by two hyperbolic curves. However, Rauhut (1969) commented that, in general, the bending moments calculated by Haliburton's method were larger than those measured in Rowe's model tests due to the fact that wall friction and the linear distribution of deflections were ignored.

A discrete element method based on the 'beam on elastic foundation' approach was also proposed by Bowles (1988). Unlike Haliburton's method, the concept of subgrade reaction was used for the passive resistance only and, similar to Rowe's method, Coulomb's theory was used to calculate the earth pressure above the excavation level. However, constant active pressure below the excavation level was considered unnecessary, as in Turabi & Balla's method. Browzin (1982) commented that Bowles' hybrid approach is not legitimate as it combines conditions at failure (Coulomb theory) with conditions at working load ('beam on elastic foundation').

Maffei et al (1977b) proposed a model for earth retaining structures which simulates the step by step construction of an excavation. This model takes into account the elasto-plastic behaviour of the soil as well as its hysteresis. Winkler's hypothesis (Winkler, 1867) is adopted in calculating the displacements. Although this model is a better representation of the soil-structure system than the other 'beam on elastic foundation' methods, the uncoupled nature of the soil springs cannot model any arching effect originating from other parts of the wall.

Both the boundary element and finite element methods treat the soil as a continuum and give the best representation of the soil-structure interaction. The application of the boundary element and finite element methods to predict the performance of propped walls was carried out by Wood & Perrin (1984) and by Bjerrum et al (1972), Egger (1972), Hubbard et al (1984), Potts & Burland (1983a) and Smith & Boorman (1974), respectively.

Potts & Fourie (1984) used a finite element method to investigate the influence of type of construction and of initial stress on the behaviour of propped walls (see also Section 7.1.3). The results of their investigation indicated that, for walls in a low K_o (K_o=0.5) soil, the simple limit equilibrium calculations produce conservative values of prop force and bending moments. For excavated walls in high K_o (K_o=2) soils, prop force and bending moments greatly exceed those calculated using the limit equilibrium approach currently in use. Increasing the embedment of the excavated walls in high K_o soils does not reduce the prop force or bending moments, even though the factor of safety against overall stability is increased. Large zones of failed soil, especially in front of the wall, were predicted for excavated walls in high K_o soils, and the lateral earth pressures behind the wall differ substantially from the classical active distribution. Passive conditions in front of the wall are completely mobilised at small excavation depths and before active conditions are approached down the back of the wall. In contrast, walls in low K_o soils showed lateral pressures which were in agreement with the classical distributions. Active pressures down the back of the wall are mobilised at small excavation depths and before passive pressures fully develop in front of the wall. In all the cases considered, high mobilised angles of wall friction were predicted. There is therefore a need to assess the K_0 value in the case of excavated walls.

Smith & Boorman (1974) showed that Rowe's model test results can be reproduced using the finite element method. Potts & Fourie (1985) carried out a finite element analysis to investigate the effect of wall flexibility on the behaviour of propped walls. The results showed that, for flexible walls of the sheet pile type embedded in low K_0 soils, the bending moments are much lower than those given by the simple limit equilibrium predictions. As the wall stiffness increases, the bending moments approach the limit equilibrium value. These results are in full agreement with the model tests of Rowe (1952). However, for stiff walls in high K_0 soils, the bending moments greatly exceed the limit equilibrium values. A similar trend is observed for the prop force. Curves showing the variation in the ratios of the maximum bending moment and prop force to those predicted by limit equilibrium method with wall stiffness, for various insitu K_0 values, are given in Figure 27.

For excavated walls in high K_0 soils, it appears that the boundary element and finite element methods are the only suitable design methods. For walls in low K_0 soils, methods in (c) are also better than methods (a) and (b) because the lateral displacements of the wall as well as the wall bending moments and prop force can be obtained. As deformation is not usually the controlling criterion for propped walls, methods (a) and (b), which are conservative and suitable for hand computation, are normally adopted. In such cases, method (a) is preferable to method (b) because the earth pressure in front of the wall under working conditions is the fully-mobilised passive pressure, as revealed by the finite element method of analysis. When method (a) is used, the considerations regarding curtailment of the reinforcement and allowance for water pressure for cantilevered walls apply also to propped walls (see Section 6.4).

(2) Prop Force. There are two conflicting requirements concerning the prop positions, viz. the need to install supports at an early stage of excavation to restrict movement and the need to install them at a low enough level to limit the moment acting in the wall (Padfield & Mair, 1984). For greatest economy in the wall section, often props are placed so that moments are equal in the span below and in the cantilevered section above the prop. Shear forces in the wall near the position of the prop may be critical for concrete walls.

A factor of safety is usually applied to the design prop force. If method (a) is used for the design moment calculation (see Section 7.1.4(1)), resolution of forces allows a solution to be obtained for the horizontal prop load, which is generally increased by 25% to allow for the possibility of arching and stress redistribution behind the wall (Padfield & Mair, 1984). If method (b) is used for the design moment calculation, the value of the prop load obtained is very sensitive to the factor of safety used. The prop force should always be conservatively assessed because, unlike in the structural design of the wall, the consequences of failure are usually very serious. The system of props and waling should be able to withstand the loss of at least one prop.

Raking struts generally allow considerable movement, so it is acceptable to design for active pressure behind the wall, even with several levels of raking struts.

7.1.5 Design Water Pressures

Considerations relating to the evaluation of design water pressures are given in Section 6.5.

7.2 MULTI-LEVEL STRUTTED WALLS

7.2.1 General

The factors which influence the behaviour of multi-level flexible strutted excavation were reviewed by Bjerrum et al (1972). The loads which must be resisted by the struts in a strutted excavation depend on two entirely different factors.

The first factor is the magnitude of the total active earth pressure exerted by the soil behind and beneath the wall, down to the depth where soil movements are no longer significant. This depends exclusively on the shear strength and the unit weight of the soil behind the wall.

The second factor is the distribution of the earth pressure, which determines how much of the total active earth pressure will be carried by the struts. The distribution depends on the amount of arching and is controlled by the magnitude of the deformations in the soil beneath the excavation relative to those of the struts. These deformations depend on the modulus and the unit weight of the soil beneath the excavation and behind the wall, the width and depth of the excavation, and the depth to a firm layer. For soils with a high modulus, such as dense sands or soft rock, the deformations in the soil beneath the excavation level are insignificant and therefore the arching effect will be small. In these cases, the sum of the strut loads will be nearly equal to the theoretical Rankine value. However, the distribution of earth pressure will approach the shape as shown in Figure 28(a) : the earth pressure on the upper part of the wall is only slightly in excess of the theoretical Rankine value and the arching effect is mainly due to local redistribution of earth pressure between the lowest strut and the excavation level (Bjerrum et al, 1972). This is also the case for excavations in soils with a somewhat lower modulus if the width of the excavation is small compared to its depth. For wide excavation in soils with low modulus, such as soft clays, medium stiff clays, silts and loose sands, the deformations beneath the excavation level are large, the arching effect will be significant and loads will be thrown onto the struts such that the sum of the strut loads will considerably exceed the theoretical Rankine value. The earth pressure distribution will be close to the shape shown in Figure 28(b).

The design considerations for multi-level strutted walls are the same as those for the case of single-level strutted walls.

7.2.2 Ultimate Limit States

The wall penetration below final excavation level should also be sufficient to limit the bending moment in the wall. A check should be carried out by considering the equilibrium of the free-ended span below the lowest strut, assuming fixity at that strut (Figure 29). For the temporary (construction) condition, a factor of safety not less than 1.5 on passive resistance is usually applied.

The depth of penetration of the wall below the excavation level should be sufficient to prevent foundation heave in cohesive soils (see Section 6.2.3) or hydraulic failure in cohesionless soils (see Section 6.2.4).

7.2.3 Serviceability Limit States

Ground movements around an excavation are usually estimated using the methods given in Section 9.3. However, the lateral deflection of a multi-level strutted wall is of secondary importance and deformation analysis is usually not carried out. If required, the 'beam on elastic foundation' approach (see Sections 5.4), the boundary element and finite element methods (see Sections 5.5 and 5.6) may by be used to predict the lateral displacement of the wall.

A parametric study was carried out by Palmer & Kenney (1972) to investigate the behaviour of a strutted excavation with fully penetrating sheet pile walls through weak clay to bedrock. The results of their study showed that, of the parameters concerning soil conditions, the soil deformation modulus has the greatest influence. The influence of shear strength and soil/wall adhesion is of minor importance. This may be the case for an excavation with a fully-penetrating wall. However, in the case of a partially-penetrating wall, the other parameters might have greater influence. Potts & Fourie (1984) showed that the initial insitu stress has a considerable influence on the deformation of single-level strutted walls. It is considered that the same is true for multi-level strutted walls.

All of the parameters concerning support conditions are important, but of these, wall stiffness is the most important. The effective strut stiffness depends not only on the stiffness of the struts and walings but also on the construction methods used to eliminate 'slack' between the wall and the walings and struts (such as wedging or other means of filling any gaps between the wall and the walings, and prestressing the struts).

Where it is important to keep soil deformation, and especially surface settlement, to a minimum to prevent damage to neighbouring structures and public utilities, stiff sheet piles and a stiff support system (i.e. one with stiff walings and closely-spaced struts) have to be used. However, although these measures will be useful, some surface settlement is inevitable, primarily because of the amount of deformation which occurs below the bottom of the excavation.

Prestressing of struts for an excavation in weak clay will only reduce deformation by a certain amount. Significant deformation will still occur, and increased strut loads must be anticipated. It is suggested that a small amount of prestress could be beneficial in ensuring the effectiveness of the support system at an early stage. However, a large amount of prestress may not provide additional benefits (see Section 9.2.7).

The vertical spacing of struts affects the magnitude of strut loads and also of wall deflections, particularly where a flexible wall is used. A close horizontal strut spacing could provide a significant contribution to the effective stiffness at a particular strut level. A stiff waling has a similar effect.

7.2.4 Earth Pressures for Structural Design

The structural design of multi-level strutted walls is usually based on the semi-empirical apparent earth pressure envelopes developed by Terzaghi & Peck (1967), which was later summarised by Peck (1969b) (Figure 30). The following four important features of the Terzaghi & Peck method should be noted (Lambe, 1970) :

(a) It applies to a 'deep' excavation, deep being defined as "depths greater than about 6 m (20 feet)".

(b) The water table is assumed to be below the bottom of the excavation. Sand is assumed to be drained, with zero pore pressure; clay is assumed to be undrained and only total stresses are considered.

(c) The apparent pressure diagrams are not intended to represent the actual earth pressure or its distribution with depth, but rather envelopes from which safe estimates of the strut loads can be derived.

(d) The behaviour of an excavation in clay depends very much on a stability number, N_c, where $N_c = \gamma D/C_u$. "Extraordinary large movements appear when the depth of a cut in clay approaches that corresponding to N_c = 6 or 7 and when a considerable depth of similar clay also extends below the bottom of the excavation" (Peck, 1969b).

The Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 1985) has recommended that hydrostatically distributed water pressures should be added to the apparent pressure envelopes for sands and stiff to hard fissured clays, and that for soft to firm clays an apparent earth pressure smaller than the hydrostatic water pressure should not be used. For excavations in deep deposits of soft clay, Henkel (1971) proposed a rational method for calculating the resultant thrust on the sides of the excavation as an alternative to the apparent pressure envelopes (Figure 31).

There are also other semi-empirical methods of obtaining horizontal soil stresses and strut loads, e.g. the methods given in Teng (1962), Armento (1972), Tschebotarioff (1973) and the Japan Society of Civil Engineers (1977). The apparent pressure envelopes given in these references are generally conservative, but they are less widely used than those of Terzaghi & Peck. Swatek et al (1972), however, found reasonable agreement with the Tschebotarioff pressure envelopes in designing the support system for a 21 m deep excavation in Chicago clay. It appears that the Tschebotarioff method may be more appropriate when the excavation depth exceeds about 16 m.

It should be noted that none of these envelopes are based on measurements taken from saprolites, the material properties of which are significantly different from sedimented sands and clays, and the mass of which frequently contains discontinuities that act as preferential flow paths for groundwater and often as potential sliding surfaces. Preferential flow paths have an important bearing on the design against piping and heave at the base of the excavations. On the other hand, the presence of corestones can reduce the magnitude of earth pressures acting on the walls of the excavation (Massey & Pang, 1989).

The apparent pressure diagram derived by Massad (1985) and Massad et al (1985) for the Brazilian lateritic and weathered sedimentary soils show a maximum value of $0.08\gamma H$ (Figure 32) where γH is the overburden pressure. This is considerably lower than the corresponding value in the Terzaghi & Peck envelope for stiff sedimentary soils of other origins. Wirth & Zeigler (1982) found that the measured strut loads for the Baltimore Subway strutted excavation in soils derived from insitu weathering of gneiss-granite and schists were about 70% of the Terzaghi & Peck envelope for sands. Although there are no reported field measurements of strut loads for Hong Kong residual soils and saprolites, observations from the performance of existing structures suggest that active and at-rest pressures are smaller than would be estimated by conventional calculations (Lumb, 1977; Endicott, 1982; Howat, 1985). Thus, field measurements of strut loads should be carried out in order to establish the apparent earth pressure envelopes for residual soils, which could be much lower than those of Terzaghi & Peck.

In the determination of strut loads from apparent pressure envelopes, a reaction at the base of the excavation is assumed to exist. This reaction is provided by the passive resistance of the soil beneath the excavation. Figure 33 illustrates the method given by Goldberg et al (1976) for determining the depth of penetration in 'competent' soils (e.g. medium dense to dense granular soils and stiff to hard clays), which are capable of developing adequate passive resistance. In weak soils, such as soft clays, the passive resistance may never reach the value of the active pressure on the retained side, no matter how deep the penetration is. This may also apply to a deep excavation in competent soil where the pressure on the wall below the lowest strut level is in an active state due to a high lateral pressure resulting from surcharge loading. For such cases, Goldberg et al (1976) have recommended that the wall should be driven to a depth required to prevent bottom heave or piping, and be designed as a cantilever about the lowest strut level. This approach is similar to that given in NAVFAC (1982b) for limiting the bending moment in the wall (see Section 7.2.2).

Surcharge loadings should also be included in the apparent pressure envelopes to determine the strut loads. Support systems for deep excavations in urban areas are often subjected to asymmetric surcharges. An asymmetric condition occurs when the foundations of the buildings on the two sides of an excavation are of different types or when the buildings have a different number of floors or are at different distances from the excavation. Another condition of asymmetry is when the water table has been lowered more on one side of the excavation than the other. Analysis and field measurements by De Rezende Lopes (1985) showed that the wall opposite to the higher surcharges could be subjected to higher internal forces and could even be forced against the retained ground, developing passive pressure. Hence, both sides of the support system should be considered when the excavation is subjected to asymmetric surcharges.

All construction stages of a strutted excavation should be considered, including strut removal stages. The required penetration depth, and the sections to be used for the struts, walings and the wall should be adequate for all construction stages. A step by step procedure for designing strutted excavations is illustrated by an example given by NAVFAC (1982b).

Scott et al (1972) showed that the earth pressure distribution of a strutted excavation in dense sand can be approximated by the extended Dubrova's solution (Dubrova, 1963) for a given wall deformation. The four possible types of deformation of a strutted excavation for which a graphical solution have been given all result in parabolic lateral earth pressure distribution curves.

An empirical method for the design of multi-tied diaphragm walls was first introduced by Littlejohn et al (1971) and was presented again by James & Jack (1975) (see Section 8.2.4). It involves a procedure which calculates the position and magnitude of a resultant tie at any stage of excavation by treating the wall as a single-tied structure. It has the advantage of being a repetitive single-tied wall design and is amenable to varying soil layers. Wood (1983) developed a computer program using this method for the design of multi-strutted diaphragm walls.

Bowles (1988) used a discrete element method to simulate the stage by stage construction of a strutted excavation. Coulomb/Rankine theory was used to calculate the earth pressure above the excavation level, and the concept of subgrade reaction was used for obtaining the passive resistance below the excavation level. Maffei et al (1977b) proposed a model for analysing multi-level strutted walls which utilises the earth pressure-displacement relationship of the soil, as well as invoking Winkler's hypothesis (Winkler, 1867). All of these 'beam on elastic foundation' approaches can at best give approximate values for the strut loads, wall bending moments and wall displacements (see Section 5.4).

Both boundary element methods and finite element methods were used by Fitzpatrick et al (1985), Humpheson et al (1986) and Wood & Perrin (1984), and by Clarke & Wroth (1984), Eisenstein & Medeiros (1983), Fernandes (1985), Garrett & Barnes (1984), Hubbard et al (1984), Izumi et al (1976), Palmer & Kenney (1972), Roth et al (1979) and Tominaga et al (1985) respectively to predict the performance of strutted excavations. Qualitative agreement between predicted and measured earth pressure distributions was obtained. Good quantitative agreement, however, requires the choice of suitable values of deformation parameters. Both Eisenstein & Medeiros (1983) and Tominaga et al (1985) used Lambe's (1967) stress path method to determine the deformation parameters. Eisenstein & Medeiros found that the analysis, which used a soil stress-strain model based on plane strain test results, describes the field behaviour more closely than those based on conventional triaxial and stress-path triaxial test data. The calculated lateral pressure distribution was also found to differ from the semi-empirical pressure distribution diagrams.

Eisenstein & Negro (1985) proposed a theoretical framework for strutted excavation analysis (Figure 34). Pressure is uniquely related to displacement Δh through two main controlling factors : the at-rest pressure P_o and the slope of the ground reaction curve. The former is related to the insitu vertical stress by K_o, the coefficient of earth pressure at rest (see Section 5.6), and the latter is a function of the soil's stiffness and strength. The amount of displacement allowed is a function of the excavation method and supporting system used. In fact, the supporting system could be represented by a 'support reaction curve' as shown in

Figure 34. At point 'A' in this figure, support and ground would interact and find an equilibrium point, E. The construction technique would affect the amount of displacement, Δh_i, taking place prior to support installation. The wall and support stiffness would be represented by the slope, n, of the normal to the support reaction curve shown. Thus, a function could be postulated to represent the ground response :

$$G\left(\frac{\Delta h}{H}, \frac{P}{P_o}, \frac{z}{H}, K_o, \frac{Eo}{P_o}, \text{strength, construction technique}\right) = 0 \quad . \; . \quad (2)$$

Similarly, the support response function could be :

$$S\left(\frac{\Delta h}{H}, \frac{P}{P_o}, \frac{z}{H}, \frac{H}{D}, \frac{t}{H+D}, \text{support stiffness}\right) = 0 \quad . \; . \; . \; . \; . \; . \; . \; . \quad (3)$$

These functions could be obtained by numerical modelling, using an appropriate constitutive relationship for the soil. Eisenstein & Negro then applied this framework to explain that the reason for the low values of the lateral pressures observed in the Brazilian lateritic and sedimentary soils (Massad, 1985; Massad et al, 1985) are due to their lower K_o values.

Since the construction technique has a significant influence on the behaviour of strutted excavations, any application of the finite element method to practical cases should be used in parallel with Peck's (1969a) observational procedure, and allowance should be made for re-analysis to be performed so as to accommodate soil parameter re-estimates and construction changes.

The semi-empirical methods do not consider soil-structure interaction and are conservative, as they generally overestimate the strut loads. The analytical methods which take into account soil-structure interaction have a better prospect of giving strut loads closer to reality. Maffei et al (1977a) compared the 'beam on elastic foundation' approach (Maffei et al, 1977b) with the finite element method and found that the lateral stresses on the wall are not very sensitive to the coefficient of subgrade reaction, but the wall displacements are. In terms of professional time and cost to perform an analysis, the finite element method is the most expensive, and the 'beam on elastic foundation' approach is cheaper than the boundary element method. If the ground displacement is the crucial factor, the finite element method is the only method capable of making a reasonable prediction.

7.2.5 Design Water Pressures

Considerations relating to the evaluation of design water pressures are given in Section 6.5.

7.3 TEMPERATURE EFFECTS

Temperature changes may cause significant changes in strut loads. Chapman et al (1972) showed that, by using elastic theory, the deformation modulus of the soil behind a strutted wall can be estimated by comparing strut-load changes with wall deflections for a given temperature change.

By combining the relations for elastic displacement of a strutted wall with the effect of temperature on load and displacement of a strut, the following approximate relation was obtained :

$$\Delta P = A_S E_S \alpha \Delta T/(1+3nA_S E_S H/A_W EL) \quad . . . . (4)$$

where ΔP = load change due to a temperature change of ΔT,

A_S = cross-sectional area of strut,

E_S = Young's modulus of steel,

α = thermal coefficient of expansion for steel,

nA_S = total area of struts acting against the wall of area A_W,

H = height of excavation,

L = length of strut,

E = Young's modulus of soil.

Chapman et al (1972) showed that the above relation provides a reasonable means for estimating the strut load changes which result from temperature changes. Strut load changes will approach the load changes for a restrained strut if the soil modulus is high, the total area of the steel struts divided by the area of the strutted wall is low, and the length of the strut is large in comparison with the height of the excavation.

For a perfectly restrained strut, the increase in strut load due to a temperature change is given by :

$$\Delta P = A_S E_S \alpha \Delta T \quad . . . . (5)$$

Load changes resulting from large temperature increases should be added to the strut loads if the apparent earth pressures are based on data obtained for relatively constant temperatures, or small changes in temperature. The temperature effect on strut loads has also been discussed by Goldberg et al (1976) and Massad (1985).

8. TIED-BACK OR ANCHORED WALLS

8.1 SINGLE-LEVEL TIED-BACK WALLS

8.1.1 General

The factors which influence the behaviour of tied-back or anchored flexible walls were reviewed by Bjerrum et al (1972) and are discussed in Section 7.1.1. For anchored walls with sloping anchors, the effect of the vertical component of the anchor force should be considered in design (Figure 35). This vertical force may cause possible downward movement of the wall when the wall has not been driven to refusal and the point resistance is low (see Section 8.1.2).

In the design of anchored walls, there are four primary areas of geotechnical consideration :

(a) ultimate limit states,
(b) serviceability limit states,
(c) earth pressures for structural design, and
(d) design water pressures.

The consideration of ultimate limit states rather than serviceability limit states usually governs the design.

8.1.2 Ultimate Limit States

(1) Overall Stability. Possible failure mechanisms of anchored sheet pile walls are shown in Figure 36.

The penetration depth is not sufficient in Figure 36(a) to resist the axial force (due to the anchor) acting on the wall. The vertical component of the load in the inclined anchor forces the sheet piles downwards into the underlying soil. The wall will also move outwards because of the inclination of the anchors.

The sheet pile wall in Figure 36(b) rotates about the anchor level when the toe of the wall moves outwards.

The total stability of the cut is not sufficient in Figure 36(c). Failure occurs along a slip surface which passes below the wall.

The failure mechanism in Figure 36(d) is caused by rupture of the anchors or of the walings. The sheet pile wall will, in this case, rotate or fold around a point located just below the bottom of the excavation. This mechanism may develop even when a single anchor rod or strand ruptures. This is because arching is unlikely to assist in re-distributing the lateral earth pressure, unless the anchors are exceptionally close; a waling, if present, may assist. In any case, the load increase in adjacent anchors must equal the load lost in the failed anchor. Rupture of anchors is most likely to occur when anchor loads are increasing generally, in which case additional capacity may not be available and progressive failure may then take place. Although not specifically mentioned by Broms & Stille (1976), bond failure (anchor pull-out) is another mechanism which can be considered to have the same effects.

The flexural resistance of the sheet piles has been exceeded in Figure 36(e). One or several plastic hinges develop at the levels where the positive or negative bending moments reach a maximum. At failure, the wall folds around these hinges.

Broms & Stille (1976) reported failures of anchored sheet pile walls in the soft clays of Sweden. The failure mechanisms shown in Figures 36(a), 36(b) and 36(c) predominate. These indicate the importance of considering the vertical support of anchored sheet pile walls and other possible factors, such as reduction of shear strength of the clay due to excess pore water pressures caused by pile driving nearby, which can decrease the axial load carrying capacity of the sheet pile. The failure mechanism shown in Figure 36(d) has also occurred. Failure of the anchors was caused principally by the lateral expansion of the soil due to frost action, and was seldom caused by stress corrosion of the anchor bars or by slippage of the anchor bolts or the wedges. The latter generally occurred during the development stage of new anchor systems. The failure mechanism shown in Figure 36(e), caused by yielding of the walings or of the sheet piles, has not occurred in Sweden and also appears to be rare in other parts of the world. This indicates that the methods currently used for the design of walings and sheet pile walls are safe.

Daniel & Olson (1982) reported an anchored bulkhead failure in stiff fissured clays due to a lack of understanding of soil behaviour under conditions involving significant excavation near the bulkhead. The design was adequate for undrained conditions, but was inadequate for fully drained conditions.

In Hong Kong, there is no need to consider frost effects. The design of anchors and the structural design of sheet pile walls are discussed in Section 8.1.4. The deep seated failure in Figure 36(c) can be analysed using appropriate slope stability analyses, such as those discussed in the Geotechnical Manual for Slopes (GCO, 1984).

The toe failure mechanism, Figure 36(b), can be assessed using limit equilibrium methods of analysis, and the free-earth support method is preferred to the fixed-earth support method. The different methods for applying the factor of safety are discussed in detail in Section 6.2.2.

Rowe (1952) derived a flexibility number, ρ, for sheet pile walls :

$$\rho = \frac{L^4}{E_W I_W} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (6)$$

where L = length of sheet pile,

E_W = modulus of elasticity of sheet pile material,

I_W = moment of inertia of sheet pile cross-section.

It should be noted that ρ is not dimensionless, and is not applicable to cohesive soil. The values of ρ which delineate 'stiff' piles (whose bending moments can be calculated reasonably accurately using the free-earth support method) from 'flexible' piles (whose bending moments are smaller than those calculated using the free-earth support method) are given by $\log \rho = -3.5$ and $\log \rho = -4.125$ for loose sand and dense sand respectively. These values, however, are in the imperial units adopted by Rowe (viz. feet for L, lbf/in^2 for E_w and in^4/ft for I_w).

Browzin (1981) developed a dimensionless flexibility number, ρ_d, for cohesive frictional soils :

$$\rho_d = \frac{\gamma B L^4}{E_w I_w}\left(1 - \frac{2c}{\gamma L} K_a^{-0.5}\right) \quad . \; . \; . \; . \; . \; . \; . \; . \; . \; . \quad (7)$$

where γ = effective unit weight of soil,

B = width of sheet pile,

c = cohesion of soil in terms of total stress,

K_a = coefficient of active earth pressure.

For the free-earth support conditions, ρ_d must be less than five for loose soils and less than one for dense soils. The limiting length for the sheet piles can be calculated from these limiting values of ρ_d. When the sheet pile length is greater than the limiting value, the bottom end of the sheet pile can be considered to be fixed.

The stability against penetration failure (Figure 36(a)) is adequate if the side resistance and reaction at the base or toe of the wall is sufficient to overcome the vertical component of the anchor forces. Browzin (1977) showed that, for walls with inclined anchors in cohesionless soils, the required driving depth is nearly the same as for walls with horizontal anchors. Also, the skin friction capacity of the piles is much higher than the shear stresses generated by the vertical component of the anchor load. Browzin (1982) subsequently showed that the condition of sinking of piles in cohesive soils is not critical because of the lower anchor forces resulting from the great depth of penetration that is needed for stability against overturning.

Anchored walls with sloping anchors in cohesive soils should be designed to avoid the following three conditions that involve a single stability number, $4c/\gamma H$:

(a) overturning,
(b) sinking of the piles with sloping anchors, and
(c) heave at the bottom of the excavation (see Section 8.1.2(2)).

A graph presented by Browzin (1981) shows the relationship between the stability number and the parameter $\beta = 1 + H/d$ (where H is the retained height of the excavation and d is the depth of penetration) for the above three conditions of equilibrium (Figure 37). From the graph, which was derived for an essentially cohesive soil (ϕ=0^o-10^o), it can be seen that stability against overturning requires larger values of $4c/\gamma H$ than stability against sinking except when $\alpha = 0^o$, where the lines representing overturning and sinking conditions intersect at $\beta = 3.7$. This leads to the conclusion that stability against overturning is the governing condition for anchored walls with sloping anchors. The equation developed for stability against overturning also shows the high sensitivity of the required depth, d, to the assumed angle of internal friction, ϕ, a fact also pointed out by Kovacs et al (1974).

Browzin (1983) later presented universal design charts (Figures 38 to 40) for calculating the required depth of penetration for stability against overturning, using the free-earth support method. The depth of penetration is particularly sensitive to the assumed values of Ø and c.

Nataraj & Hoadley (1984) developed a pressure diagram to reduce the computational effort in the design of anchored walls in sands using the free-earth support method.

(2) Foundation Heave and Hydraulic Failure. Considerations for foundation heave and hydraulic failure are discussed in Sections 6.2.3 and 6.2.4 respectively.

8.1.3 Serviceability Limit States

Deformation considerations for anchored walls are usually of secondary importance and are similar to those for single-level strutted walls, which are discussed in Section 7.1.3.

8.1.4 Earth Pressures for Structural Design

The calculation of bending moments for anchored walls is similar to that for single-level strutted walls, which is discussed in Section 7.1.4(1).

As discussed in Section 8.1.2(1), bending failure of the walings or of sheet piles rarely occurs in practice. This indicates that the currently used design methods are adequate.

Similar to strutted walls, a factor of safety is applied to the design anchor force. Further allowances (e.g. sacrificial thickness) are required if corrosion problems are expected. Anchors are usually inclined, and this has three possible effects :

(a) The value of K_a is increased due to reversal of wall friction under working conditions (however, at failure, the wall friction reverts to the usual direction, so no change needs to be made to the calculation of K_a for overall stability).

(b) Settlement of the wall and the retained soil is increased.

(c) The anchor tension is greater than the equivalent horizontal prop force.

A comprehensive treatment of the design of anchors is given by Hanna (1980), British Standards Institution (1989) and Geospec 1 : Model Specification for Prestressed Ground Anchors (GCO, 1989). Factors of safety for anchor design in Hong Kong are given in Geospec 1.

8.1.5 Design Water Pressures

Considerations relating to the evaluation of design water pressures are given in Section 6.5.

8.2 MULTI-LEVEL TIED-BACK WALLS

8.2.1 General

The behaviour of multi-level tied-back walls is shown qualitatively in Figure 41 (Hanna, 1968). The use of inclined anchors causes vertical loading of the wall as construction progresses, and inadequate bearing capacity at wall base level can contribute to wall failure. As the wall is loaded, displacements of the wall relative to the retained soil mobilize friction on the embedded wall and strains within the soil mass. Vertical wall movements resulting from wall compression and settlement of the wall base are responsible for lateral movements of the wall system.

Observed behaviour of tied-back wall systems (Clough, 1972b) suggests that tied-back walls generally yield smaller deformations than strutted walls. Finite element analyses by Clough & Tsui (1974) have shown that much of the difference in behaviour is due to the fact that tied-back wall systems are usually prestressed, while strutted wall systems are usually not, and that over-excavation of the temporary excavation platform, a problem associated with strutted walls, can lead to substantial increase in movement.

The process of development of earth loadings for strutted and tied-back walls is fundamentally different. For a strutted wall, active earth pressure is partially mobilised behind the wall, and the loads which develop are primarily a function of soil type and strength. Because of the high stiffness of the struts and the sequence of excavation, the top of the wall becomes essentially fixed after the first strut is installed, and the wall rotates about the top strut as excavation proceeds. This leads to higher loads near the top of the wall than would be indicated by Rankine/Coulomb theory. A tie-back system, on the other hand, has two possible modes of behaviour, both of which differ from that of a strutted wall. In the first mode, the wall is prestressed with a resultant greater than that of the active or partially active condition. Under these circumstances the system responds more akin to a foundation slab on soil, and the earth loading on the wall is primarily dictated by the loading distribution assumed in the design prestress diagram and relative wall and soil stiffnesses. In the second mode, the tied-back wall is prestressed to lower levels, i.e. total resultant equal to or less than that of active conditions, and the loadings on the wall are developed in a manner similar to a strutted wall. However, the upward redistribution of loads which occurs in a strutted wall does not occur in the case of a tied-back wall because the tie-back elements are very flexible, and the top of the wall will continue to rotate outwards as excavation progresses. Thus, the loads developed resemble those of a conventional triangular earth pressure distribution, instead of those usually developed in a strutted wall.

The design considerations for multi-level tied-back walls are the same as in the case of single-level tied-back walls.

8.2.2 Ultimate Limit States

The failure mechanisms in Figure 36 are also possible for multi-level anchored walls.

The design of the anchors and the structural design of the wall are discussed in Section 8.2.4. The stability against the deep-seated failure shown in Figure 36(c) can be assessed using appropriate slope stability methods (GCO, 1984).

The vertical component of the anchor forces for multi-level anchored walls could be large. Stability against penetration failure is therefore an important consideration. The side resistance and the reaction at the wall base must be adequate to balance the vertical anchor forces. For this reason, it is common for multi-level anchored walls to be founded on rock.

Anderson et al (1983) reviewed four methods of checking the overall stability of anchored retaining walls, viz. the German method (German Society for Soil Mechanics and Foundation Engineering, 1980), the Ostermayer (1977) method, the French method (Bureau Securitas, 1972) and the method by Littlejohn et al (1971). They found that all four design methods resulted in stable systems, both at the end of construction and after backfill loading. The determination of anchor prestress loads by the method by James & Jack (1975) was found to be based on erroneous earth pressure assumptions (see Section 8.2.4). More consistent behaviour was found in tests designed using the method by Littlejohn et al (1971) for overall stability, particularly on surcharge loading, indicating that a logarithmic spiral method may be the most appropriate to use. However, Schnabel (1984) commented that the German and French methods are wrong, as they assume the tie-back pulls on the soil without pushing on the wall. He considered that only the Ostermayer method is correct because it considers the tie-backs as internal forces in the soil wedge between the wall and the anchors. The method by Broms (1968) also recognizes this. A comprehensive review of methods of analysing overall stability of anchored retaining walls is given in British Standards Institution (1989).

The depth of penetration of the wall below excavation level should be sufficient to prevent foundation heave (see Section 6.2.3) or hydraulic failure (see Section 6.2.4).

8.2.3 Serviceability Limit States

As in the case of multi-level strutted walls, deformation analysis of multi-level tied-back walls is of secondary importance and is usually not carried out. If required, the 'beam on elastic foundation' approach (see Section 5.4), the boundary element and finite element methods (see Sections 5.5 and 5.6) can be used to predict the lateral displacement of the wall.

Maffei et al (1977a) found that the wall displacement is very sensitive to the assumed coefficient of subgrade reaction. In their parametric study of the behaviour of tied-back walls, Clough & Tsui (1974) found that for a wall founded on rock, the wall movement and the soil settlement behind the wall can be substantially reduced by increasing the prestress load, the wall rigidity and the tie-back stiffness. The use of higher prestress loads can eliminate the wall movement at the wall top but is not able to prevent wall movement near the bottom of the excavation. As a result, soil settlements occur in all stages of the excavation. The decrease in wall movement is not in direct proportion to the increase in wall rigidity and tie-back stiffness. Stroh (1975) showed that the wall movement is more than twice as large where clay is present below the wall compared with the case where the wall is founded on rock, and that the wall movement decreases almost linearly as the length of anchors increases.

8.2.4 Earth Pressures for Structural Design

The earth pressures that act on an anchored wall depend on the wall stiffness relative to the soil, the anchor spacing, the anchor yield and the prestress locked into the anchors at installation.

Two conventional design methods which ignore the effects of wall rigidity, excavation depth and tie-back spacing on the earth pressure distribution acting on the wall are outlined below.

An empirical method first introduced by Littlejohn et al (1971) was presented again by James & Jack (1975). This method is also recommended by the Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 1985). Wood (1983) developed a computer program using this method (see Section 5.5). It can take account of the continuous wall construction, excavation and anchoring stages, and the procedure is applicable to varying soil layers. Certain basic assumptions are made in the method, e.g. the soil pressure distribution is assumed to be triangular and the wall is assumed to yield progressively as excavation proceeds. In addition, a point of contraflexure in the wall is assumed to occur at a location where the factor of safety against overturning is unity. Full-scale monitoring studies (Littlejohn & Macfarlane, 1975) have indicated that wall deflections and bending moments, which occur as excavation proceeds, follow a similar pattern to those predicted by this empirical design method. Moreover, the monitoring results suggest a triangular pressure distribution for a semi-rigid anchored diaphragm wall, which is different from the trapezoidal distribution assumed in the design of strutted excavations.

Anderson et al (1983) found from their laboratory scale tests that the determination of anchor prestress loads by the method of James & Jack (1975) is based on erroneous earth pressure assumptions. They suggested that the method should be modified so that only half of the passive pressure acts in front of the wall, and that the mean of the active and at-rest pressures acts behind the wall.

The Canadian Foundation Engineering Manual (Canadian Geotechnical Society, 1985) suggests three possible values of earth pressure acting behind the wall :

(a) If moderate wall movements can be permitted, or where foundations of adjacent buildings extend to below the base of the wall, the pressure may be computed using the coefficient of active earth pressure K_a.

(b) If foundations of buildings or services exist behind the top of the wall at a distance smaller than the height of the wall and not closer than half the height, the pressure may be computed using a coefficient $K = 0.5(K_a + K_o)$.

(c) If foundations of buildings or services exist at shallow depth, at a distance smaller than half the height behind the top of the wall, the pressure may be computed using the coefficient of earth pressure at rest K_o.

The Manual also suggests that the passive pressure should be reduced to account for the wall movement required to develop it.

Another empirical method often adopted is the use of apparent pressure diagrams similar to the Terzaghi & Peck envelopes used in strutted excavations. A considerable variety of methods for calculating anchor prestress loads exists (Table 5), and Figure 42 shows reported magnitudes of design prestress diagrams used in practice. Terzaghi & Peck envelopes for strutted excavations are also shown in the figure. It can be seen that, in most instances, prestressing loads applied to tied-back walls are higher than the upper limit values for strut loads.

Laboratory-scale tests on rigid multi-anchored walls by Hanna & Matallana (1970), Hanna & Abu Taleb (1971, 1972), Plant (1972) and Hanna & Dina (1973) showed that a trapezoidal earth pressure distribution, with the earth pressure coefficient equal to $0.5(K_a + K_o)$, is appropriate. For flexible multi-level tied-back walls, laboratory-scale tests by Hanna & Kurdi (1974) showed that a rectangular earth pressure distribution appears reasonable.

Clough & Tsui (1974) used a finite element method to investigate the behaviour of a tied-back sheet pile wall in clay. Their study showed that the calculated pressures are more triangular in shape than the design trapezoidal diagram, and the calculated pressures tend to concentrate slightly at the anchor levels, a phenomenon observed by Hanna & Kurdi (1974). The triangularization of these pressures is apparently caused by the movements of the flexible tie-back supports. Anderson et al (1983) found from laboratory-scale tests that a trapezoidal earth pressure distribution assumption is realistic if excavation is being carried out to wall base level. However, a triangular pressure distribution is more realistic if excavation terminates some distance above wall base level. In practice, excavation would usually stop some distance above the wall base to minimize base movements, and field observations at the Guildhall Re-development in London (Littlejohn & MacFarlane, 1975) would seem to confirm the latter distribution.

As discussed in Section 8.2.1, there are two possible modes of behaviour for tied-back walls, depending upon the prestress level employed. Ideally, prestress just sufficient to minimize settlements behind the wall should be used, bearing in mind that some settlement will always occur regardless of the prestress load, due to the unloading of the soil at the bottom of the excavation. Observed movements of actual tie-back systems suggest that the optimum level of prestress to minimize movements lies slightly above the load levels recommended by Terzaghi & Peck (1967) for strutted walls. Satisfactory performance can easily be obtained, however, using lower prestress levels if movements are not a primary concern (Clough, 1975).

Hanna & Kurdi (1974) found that the maximum bending moment in a multi-anchored wall decreases with increasing wall flexibility. If the wall height is taken to be the free span between two consecutive anchor levels, trends of reduction in bending moment with flexibility similar to Rowe's curves (Rowe, 1952) can be obtained.

Anderson et al (1982) found from laboratory-scale tests that in order to restrict movement of anchored walls supporting backfill with a uniform surcharge loading, a rectangular pressure distribution, and an earth pressure coefficient of K_o for the surcharge component appear reasonable. For a line or strip loading, they suggested that in order to restrict movement, the strip loading component of the rectangular pressure distribution should be 1.3 times the maximum horizontal stress predicted by elastic theory.

Analytical methods which take soil-structure interaction into consideration can be used to determine the magnitude and distribution of earth pressures acting on multi-anchored walls. The 'beam on elastic foundation' approach was applied by Bowles (1988) and Maffei et al (1977b), the boundary element methods by Wood (1979) and Pappin et al (1985), and the finite element methods by Clough et al (1972), Clough & Tsui (1974), Barla & Mascardi (1975), Murphy et al (1975), Stroh & Breth (1976), Rosenberg et al (1977), Simpson et al (1979), Stille & Fredricksson (1979) and Hata et al (1985), to model the wall/anchor/excavation system. Maffei et al (1977a) found that, using the 'beam on elastic foundation' method, the horizontal stresses acting on the wall are not very sensitive to the coefficient of the subgrade reaction when these stresses are compared with those obtained by the finite element method. As in the case of multi-level strutted walls, the finite element analysis (as well as other analytical methods) should be used in parallel with an observational procedure, and allowance should be made for re-analyses to be performed so as to accommodate soil parameter re-estimates and construction changes. Soil parameters are best refined by comparing the observed performance at early stages of excavation with finite element predictions.

Summarising, the available methods are, in increasing order of cost of performing an analysis, the method of apparent pressure diagrams, the semi-empirical method by James & Jack (1975), the 'beam on elastic foundation' approach, the boundary element and the finite element methods.

8.2.5 Design Water Pressures

Considerations relating to the evaluation of design water pressures are given in Section 6.5.

9. GROUND MOVEMENTS AROUND EXCAVATIONS

9.1 GENERAL

This chapter considers only ground movements caused by deep excavations. An excellent review of movements of excavation support systems in soft clay is given by Clough & Schmidt (1981). Ground movements caused by pile driving and dewatering are discussed in the state-of-the-art report by D'Appolonia (1971).

9.2 FACTORS INFLUENCING MOVEMENTS

9.2.1 Effect of Stress Changes

Figure 43 illustrates, in a qualitative manner, the stress changes and strains experienced by two soil elements near an excavation. The stress paths are typical for a normally consolidated plastic clay. Both the reduction in total vertical and horizontal stress, and the change in equilibrium pore water pressure, are important. The table in the figure lists the stress changes and the strains brought about by the excavation and the change in seepage conditions.

The critical factor that determines the inward movement of the sides of the cut below excavation level, and hence the magnitude of settlement, involves the proximity of the unloading stress path for element B to the failure envelope. If the effective stress points $\bar{B}_1$ and $\bar{B}_{ss}$, are well within the effective stress failure envelope, K_f, i.e. if local yield is avoided, then the heave will be small and the lateral movement of the soil below excavation level will be small. On the other hand, if the effective stress points for element B approach the failure envelope and passive local yield occurs, then the movements will be large.

9.2.2 Dimensions of Excavation

The deeper the excavation, the greater is the decrease in total stress, and thus the larger are the movements of the surrounding soil.

9.2.3 Soil Properties

In clays, it has been shown (Clough et al, 1979; Mana & Clough, 1981) that the maximum lateral wall movement could be correlated with the factor of safety against basal heave, which in turn depends on the shear strength of the soil. The rate and magnitude of movement increase rapidly as the factor of safety approaches one.

If a clay is presumed to be isotropic when it is in fact anisotropic, the basal heave factor of safety may be much lower and lateral wall movements as well as ground surface settlements may be larger than expected (Clough & Hansen, 1981).

Wall movements and ground settlements are smaller in stiff soils, such as granular soils and stiff clays, than in soft soils, such as soft and medium clays and compressible silts (Peck, 1969b).

9.2.4 Initial Horizontal Stress

For excavated walls in soils with a high initial K_0 value, large soil and wall movements are experienced even at shallow depths of excavation. The wall behaviour is dominated by vertical unloading caused by the excavation process, and large movements still occur even if the wall is fully restrained from horizontal movement. For walls in soils with a low K_0 value, the displacements are much smaller in magnitude (Potts and Fourie, 1984).

9.2.5 Groundwater Conditions

In practice, a perfectly watertight wall penetrating an impermeable soil layer at the bottom of the excavation does not exist. Where water flows into the excavation, a decrease in groundwater pressure will occur. This will cause an increase in effective stress and settlement of the soil surrounding the excavation. Where groundwater has not been brought under complete control, large, erratic and damaging settlement due to the flow or the migration of fines into the excavation are not uncommon.

9.2.6 Stiffness of Support System

The support system stiffness depends on the stiffness of the wall and its supports, the spacing between supports, and the length of wall embedded below the excavation bottom. A valuable study into the effect of wall stiffness and support spacing was carried out by Goldberg et al (1976), and the results are presented in Figure 44, in which the stability number $\gamma H/C_u$ is plotted versus the stiffness parameter $E_w I_w/h^4$, where $E_w I_w$ is the stiffness of the wall, h is the distance between supports, γH is the overburden pressure, and C_u is the undrained shear strength of the soil.

Various boundary lines are drawn to establish zones of expected lateral wall movements. These data suggest that an increase in $E_w I_w/h^4$ has a significant effect in reducing movements. The movement is also a function of the factor of safety against basal heave, being more significant at lower factors of safety than at higher ones.

Increasing the stiffness of the strut or the tie-back decreases movements, but this effect shows diminishing returns at very high values of strut or tie-back stiffness.

Movements are also reduced as the depth to an underlying firm layer decreases and when the wall toe is embedded into the underlying firm layer.

9.2.7 Effect of Preloading

Preloading removes slack from the support system, and thus eliminates this potential source of movement. Each application of preload reduces the shear stresses set up in the soil due to previous excavation activities. This means that the soil is partially unloaded, and its stress-strain response is stiffened until the next excavation step generates shear stresses that reload the soil beyond its maximum previous level of shear stress. This temporary stiffening of the soil also leads to reduced movements.

Use of preloads in the struts or tie-backs reduces movement, although there are diminishing returns at higher preloads. Very high preloads may, in fact, be counter-productive, since local outward movements at support levels can damage adjacent utilities.

Clough & Tsui (1974) showed that the prestress loads in tie-backs calculated in accordance with a triangular at-rest pressure diagram are less effective in reducing movements than those obtained from the trapezoidal pressure diagram of the shape recommended by Terzaghi & Peck (1967) for calculating strut loads for strutted excavations. Clough (1975) also found that the optimum effect of prestress loads in reducing system movements can be achieved by using levels slightly greater than those recommended by Terzaghi & Peck (1967). Model tests by Hanna & Kurdi (1974) have confirmed this conclusion. Clough & Tsui (1974) also showed that movements of the tied-back wall, and the soil settlements behind the wall, can be substantially reduced by a judicious choice of prestress load and support system stiffness.

O'Rourke (1981) considered that, in most cases, cross-lot struts prestressed to 50% of their design load will be sufficiently rigid to restrict further movement at the level of the supports, and sufficiently low in load to avoid being overstressed as additional excavation occurs.

9.2.8 Effect of Passive Soil Buttresses

The effectiveness of these in controlling movements in excavation in soft to medium clay was studied by Clough & Denby (1977) using the finite element method. They used the term "berm" for a passive soil buttress. Figure 45 shows the theoretical relationships between settlements behind bermed sheet pile walls and the stability number, $\gamma H/C_{ub}$, where C_{ub} is the undrained shear strength at the base of the excavation, for the condition of excavation after the berm has been removed and the rakers installed.

At low stability numbers, increases in berm size produce minimal movement reduction. For intermediate stability numbers, increases in berm size show a diminishing effect. At high stability numbers, increasing from an intermediate to a large berm leads to a large reduction in settlements. However, this reduction is an example of false economy, since at large stability numbers, large movements occur even with large berms because deep-seated movements take place beneath the berms. As a result, the effectiveness of the berms is diminished.

9.2.9 Associated Site Preparation Works

Site preparation may include the following activities :

(a) relocation and underpinning of utilities,

(b) dewatering of aquifers above and below the base of the excavation,

(c) construction of the excavation support system, and

(d) the installation of deep foundations.

In some cases, the movements associated with the site preparation works will exceed those which occur as a result of the excavation and support process. The relocation of utilities, for example, may have a locally severe impact on an adjacent property, especially when trenching is carried out close to pipelines and communication conduits.

Dewatering may consolidate the soil over an area which substantially exceeds the area affected by excavation-induced movements. Also, it often causes settlements well in excess of excavation settlements. However, in areas which have been subjected to earlier dewatering activities, settlements due to further dewatering are smaller than those in virgin ground due to the stiffer response of the preconsolidated soil.

Wall construction may require predrilling for soldier piles, the use of vibratory hammers to install sheet piles, or the excavation of slurry panels for concrete diaphragm walls. Each of these can cause permanent movements, the magnitude and distribution of which will vary according to the soil conditions, site history and details of the construction procedures.

9.2.10 Influence of Construction Factors

A review of the influence of construction factors on excavation movements was made by Clough & Davidson (1977), and useful specific case history accounts were provided by Broms & Stille (1976), Hansbo et al (1973), Lambe et al (1970), O'Rourke et al (1976), White (1976) and Zeevart (1972).

Additional movements of excavations, and even local failure, have been produced by late installation of supports, over-excavation, pile driving, caisson construction, loss of water through holes for tie-backs and joints or interlocks of slurry or sheet pile walls, remoulding and undercutting of clay berms, and surcharge loads from spoil heaps and construction equipment.

Lambe (1970, 1972) demonstrated that variations in wedging techniques between the walings and struts, and differences in excavation procedure, can result in doubling of the wall and soil movements. Clough & Tsui (1974) used a finite element analysis to show that over-excavation could lead to a 100% increase in movement.

Because these factors cannot always be quantified, it is difficult to make accurate prediction of movements. However, many of the undesirable effects of these factors have been identified, and they can therefore be anticipated and controlled through good specifications, well-planned construction procedures and close supervision and monitoring. The designer must consider how the excavation and subsequent construction should be carried out, identify critical construction factors and, where possible, allow for them in performance estimates and specifications.

9.3 PREDICTIVE METHODS

9.3.1 Types of Methods

Prediction of soil movements behind a supported excavation can be made by the following methods :

(a) empirical methods,
(b) semi-empirical methods,
(c) the method of velocity fields, and
(d) the finite element method.

9.3.2 Empirical Methods

Settlement behind excavations can be estimated based on data obtained from previous excavations in similar soil conditions. Peck (1969b) summarised data on settlements behind excavations to produce the useful chart shown in Figure 46. Settlements presented as a percentage of the excavation depth were plotted against distance from the excavation divided by excavation depth. Three different zones, as defined in the figure, were identified. Generally, where workmanship is average or above average and soil conditions are not especially difficult, settlements should not exceed one percent of the excavation depth. In cases where seepage can occur and soil consolidation results, the one percent figure can be exceeded (Lambe et al, 1970). It should also be noted that construction technique can have a strong influence on movements of strutted systems (see Section 9.2.10).

O'Rourke et al (1976) compiled settlement data associated with soldier pile and lagging excavation in the dense sand and interbedded stiff clay of Washington, U.S.A. (Figure 47). Expressed as a percentage of the excavation depth, the surface settlements were equal to or less than 0.3% near the edge of the cut and 0.05% at a distance equal to 1.5 times the excavation depth. A comparative study of strutted excavations in the soft clay of Chicago using temporary berms and raking struts was also carried out by O'Rourke et al (1976). The data are plotted in dimensionless form in Figure 48. Three zones of ground displacement can be distinguished and related to the salient characteristics of construction. These zones approximate to the three zones of settlement delineated by Peck (1969b), with the exception that the widths of the settlement zones are notably shorter. This indicates that the settlements associated with the Chicago excavations are confined to areas that are comparatively nearer the edges of the excavations.

Based on these studies, O'Rourke (1981) noted that the strutted excavations were generally carried out in three prominent stages (Figure 49) :

(a) Initial excavation before strutting. The excavation was deepened before supports were installed. Thus, deformation of the wall occurred primarily as a cantilever-type movement. The horizontal strains produced in this mode of deformation form a triangular pattern of contours that decrease in magnitude with depth and distance away from the wall.

(b) Excavation to subgrade after installation of upper struts. As the supports were installed, the upper portion of the wall was restrained from further lateral movement. In the deeper portion of the excavation, inward bulging of the wall caused tensile strains, the contours for which were inclined at approximately 45° to the vertical.

(c) Removal of supports. As the bottom struts were removed to build the underground structure, further inward bulging of the wall occurred. When the upper struts were removed, the wall was supported in its lower portion by the subway structure, resulting in a cantilever-type deformation at the upper portion of the wall. Consequently, the cumulative strains were a composite of the horizontal distortion associated with inward bulging at depth and the distortion associated with cantilever movement at the upper levels of excavation.

Thus, the soil strains are closely related to the modes of wall deformation. A quantitative relationship between the displacement pattern of the wall and movement of the ground surface adjacent to the excavation was developed by O'Rourke (1981) and is shown in Figure 50. A coefficient of deformation, C_D, was defined as the ratio of the cantilever movement of the wall, S_w, to the sum of this cantilever component and the inward bulging component, S_w', of the displacement. It should be noted that the data summarised in Figure 50 pertain to excavations where the walls have been driven to a stiff soil layer, and thus do not have significant movement at the bottom.

The field data in Figure 50 show that the limits for the ratio of horizontal to vertical movement (settlement) are approximately equal to 0.6 and 1.6 for wall deformation caused solely by inward bulging (i.e. with the wall firmly supported at an early stage of excavation) and solely by cantilever movement, respectively. These bounds compare favourably with the limits determined from model tests in sands performed by Milligan (1983) (see Section 9.3.4).

Clough et al (1979) and Mana & Clough (1981) reviewed case histories of sheet pile and soldier pile walls in clays supported primarily by cross-lot struts. The results are summarised in Figure 51, which shows that the maximum lateral movement can be correlated with the factor of safety against basal heave defined by Terzaghi (1943). The movements increase rapidly below a factor of safety of 1.4 to 1.5, while at higher factors of safety the nondimensional movements lie within a narrow range of 0.2% to 0.8%. Moreover, there do not appear to be any significant differences in lateral movements between sheet pile walls whose tips are embedded in an underlying stiff layer and those whose tips remain in the moving clay mass.

Mana & Clough (1981) plotted maximum settlement data versus lateral wall movements, as shown in Figure 52. This figure shows that settlements range from 0.5 to 1.0 times the lateral wall movements. If settlements are conservatively assumed to be equal to lateral wall movements, then Figure 51 demonstrates the relationship between maximum ground settlements and the factor of safety against basal heave.

Clough (1972b, 1975) summarised the data from available literature on tied-back wall movements, as shown in Figure 53. Generally, the wall movements and settlements are well below one percent of the excavation depth, although in a few instances values above one percent appear. The data were re-plotted in Figures 54 and 55, where the percentage movement is plotted against the maximum ordinate of the design prestress diagram for sands and stiff clays respectively. The maximum prestress pressure is made nondimensional by the product of unit weight and excavation height. It can be seen that, for both sands and stiff clays, the movements decrease with increasing prestress. The design pressure levels suggested by Terzaghi & Peck (1967) are also shown in the figures. The data suggest that the optimum effect of prestressing in reducing movements is achieved by using pressure levels slightly greater than those of Terzaghi & Peck (1967). Further data on movements of the crest and settlement of the ground behind tied-back walls are given in British Standards Institution (1989).

A large amount of data have been obtained during the construction of the Mass Transit Railway (MTR) in Hong Kong. Stroud & Sweeney (1977) reported ground movements caused by the installation of a diaphragm wall test panel. Morton et al (1980a & b) found that the settlements induced by the MTR construction can generally be separated into those resulting from wall installation, dewatering and station box excavation. Settlements caused by the diaphragm wall construction were found to be significant. The reason for this was thought to be due to the breakdown of arching as the length of wall was constructed, and the high lateral swelling potential of the completely decomposed granite (Davies & Henkel, 1980). Cowland & Thorley (1984) also reported significant settlements due to slurry trench excavation up to a distance equal to the depth of the trench away. Significant settlements were measured at some buildings even when the whole of the base of their foundations was outside the theoretical active wedge. A relationship was found between maximum building settlement and the ratio of the foundation depth to trench depth (Figure 56). Dewatering was the prominent cause of building settlement at some of the MTR stations. Relatively large wall deflections occurred during excavation. However, the building settlements were usually low, the ratio of maximum lateral wall movement to building settlement being of the order of 4:1. While the ground conditions and the causes of settlement of adjacent buildings differed from site to site, the total settlements recorded were, surprisingly, found to be related to their foundation depths as shown in Figure 57.

Settlement data associated with the construction of the MTR Island Line were compiled by Budge-Reid et al (1984). The settlements caused by diaphragm wall installation, station box dewatering and diaphragm box excavation are shown in Figures 58, 59 and 60 respectively.

It should be noted that while Figures 56 and 58 give general empirical relationships for estimating settlement due to slurry trench installation, the fundamental factor affecting settlement is the effective stress change in the completely decomposed granite. Unfortunately, the only data relating settlement and the effective slurry pressure are those given by Davies & Henkel (1980).

Very few settlement monitoring data associated with deep excavations for basement construction in Hong Kong have been published. Morton & Tsui (1982) presented very limited ground movement data for the construction of the deep basement of the China Resources Building. Well documented settlement monitoring data for the construction of the new Hong Kong Bank headquarters building were given by Fitzpatrick & Willford (1985) and Humpheson et al (1986).

9.3.3 Semi-empirical Methods

The following semi-empirical methods have been proposed to relate the settlement of the ground behind an excavation with the lateral movement of the supporting wall.

Caspe (1966) presented a method of analysis which relates the settlement profile behind an excavation with the wall deflection. Three key assumptions were made in the analysis :

(a) There is a surface behind the wall which defines the limit of significant soil deformation.

(b) The variation in horizontal strain in the soil at any level between the above surface and the wall is assumed.

(c) At all points, the vertical strain is assumed to be related to the horizontal strain by the Poisson's ratio, ν.

Kane (1966) commented that the third assumption is incorrect, as the vertical strain should be related to horizontal strain by $\nu/(1-\nu)$ under plane strain conditions. Consequently, the predicted settlement profile no longer matches the measured profile. Bowles (1988) modified Caspe's method and obtained reasonable matching with the measured settlement profile.

Another method was proposed by Bauer (1984) to obtain a reasonable estimate of ground loss around excavations in sands, and this is illustrated in Figure 61.

9.3.4 Method of Velocity Fields

The method of velocity fields has been discussed in Section 5.3.

Bransby & Milligan (1975) showed that the deformations in the soil behind a flexible cantilever retaining wall could be predicted from the deflection of the wall using a simple velocity field. The method depends on a soil parameter, the angle of dilation, which can be related to the angle of friction using the stress dilatancy relationship of Rowe (1962). An estimate of the angle of dilation to within 5° is sufficiently accurate for all practical purposes.

For a wall with an anchor or support near the top, Milligan (1983, 1984) found that the pattern of displacements could be divided into a number of relatively simple zones, which could be related to the deformation of the wall, and these zones are linked by a more complex zone in which arching could occur. For the case in which the soil is deforming without change of volume, a simple velocity field could apply, irrespective of the mode of deformation of the wall.

9.3.5 Finite Element Method

Of the calculation methods that are available, only the finite element method takes account of the interaction between all the components within the retaining system (Institution of Structural Engineers, 1989). The inherent assumptions that have to be made in applying the finite element method to the prediction of ground movements were discussed by Burland (1978). The soil parameters required for a finite element analysis are discussed in Section 5.6.

Burland et al (1979) back-analysed measurements of movement using finite element techniques to provide insitu deformation parameters for London Clay, and used the derived parameters successfully in predicting and controlling ground movements around major excavations in critical locations in London.

Recent field and laboratory tests reveal that some soils exhibit a higher stiffness at small strain (see Section 5.6). Jardine et al (1986) used the finite element method incorporating a soil model that features small strain nonlinearity to predict the settlement behind a strutted excavation. Good agreement with the observed field behaviour was obtained.

Potts & Fourie (1984) used a finite element analysis which incorporated an elasto-plastic soil model, to investigate the behaviour of a single propped retaining wall. The results of their study showed that, for excavated walls in soils with a high initial K_o value, large soil movements occur even at shallow depths of excavation. This behaviour is related to the vertical unloading due to the excavation process. Large movements still occur even if the wall is fully restrained from horizontal movement. For walls in soils with a low K_o value, the soil movements are much smaller in magnitude.

Mana & Clough (1981) used a finite element model to verify the trend of field data, which indicate that there is a well-defined relationship between the maximum lateral movement and the basal heave factor of safety (Figure 62). The range of field data in Figure 51 are superimposed onto Figure 62, and these nicely bound the finite element trend curve. The results of the finite element analysis also showed that the maximum ground settlements range from 0.4 to 0.8 times the maximum lateral wall movements (Figure 63). However, the field data showed that the maximum ground settlements were from 0.5 to 1.0 times the maximum lateral wall movements (Figure 52). The slightly larger ratio of settlement to lateral movement observed for some of the field cases is probably due to the effects of small amounts of consolidation, or traffic or surcharge loading behind the wall.

Based on their finite element studies, Mana & Clough (1981) proposed the following procedures for estimating wall movements and ground surface settlements for a strutted excavation in soft to medium clays :

(a) Compute the factor of safety against basal heave using Terzaghi's approach at each construction stage where movements are desired. The factor of safety at each stage is to be used in the prediction procedures, except in the case where the factor of safety increases at later excavation stages due to the presence of an underlying strong layer. In this instance. the key factor of safety is the minimum which develops during excavation.

(b) Estimate the maximum lateral wall movements Δh_{max} between the factor of safety and wall movements, using the relationship in Figure 62. The maximum ground settlement Δv_{max} can also be estimated directly, assuming that it is in the range of 0.6 to 1.0 times the maximum wall movements.

(c) Based on the wall stiffness factor, $E_W I_W/\gamma h^4$, the strut stiffness factor, $S_k/\gamma H$, the depth to firm layer D_W, and the excavation width B, determine the influence coefficients α_W, α_S, α_D, and α_B, using Figures 64, 65, 66 and 67 respectively.

(d) Determine the influence coefficient for the design strut preloads using Figure 68.

(e) Select a value for the modulus multiplier M ($=E/C_u$), based on available test data or on information found in the literature. Then determine the modulus multiplier influence coefficient, α_M, from Figure 69.

(f) Using the value of Δh_{max} obtained in step (b) and the influence coefficients determined in steps (c), (d), and (e), a revised value for the maximum lateral movement is estimated as :

$$\Delta h^*_{max} = \Delta h_{max}\, \alpha_W\, \alpha_S\, \alpha_D\, \alpha_B\, \alpha_P\, \alpha_M \qquad . \; . \; . \; . \; . \; (8)$$

(g) A revised estimate for the maximum ground settlement is obtained by assuming that it is equal to 0.6 to 1.0 times Δh^*_{max}.

(h) The ground settlement profile is established using the value of the maximum settlement from step (g) and Figure 70.

Clough & Tsui (1974) carried out finite element analyses to study the difference in behaviour between tied-back and strutted walls in clay. The results of their study showed that, in the case of walls with 'equivalent' conditions, the tied-back wall moved more than the strutted wall, and settlements behind the tied-back wall were also larger. This was attributed to the movement of the anchorages in the retained soil mass. However, in practice, the tied-back wall is usually prestressed and the strutted wall is not, and over-excavation often occurs during construction of the strutted wall. In these cases, the finite element analyses showed that the tied-back wall moved less and ground settlements were smaller than those for the strutted walls.

Although the method of Mana & Clough (1981) for predicting movements is primarily designed for cross-lot strutted walls, it can be used in its present form for tied-back walls, provided that the anchors are embedded in an unmoving soil or rock mass. Should the anchorages be subjected to movement, the method is no longer applicable, since this will introduce additional displacements that are not considered in the method.

10. REVIEW OF LOCAL DESIGN PRACTICE

Local guidance on the design of excavation support systems and on prediction of ground movements around excavations is given in Chapters 7 and 10 of Geoguide 1 : Guide to Retaining Wall Design (GCO, 1982) respectively. These chapters of the Geoguide are based heavily on overseas publications from the 1970's. Some of the major factors which influence design, construction and control of excavations for basement construction in urban areas of Hong Kong are reviewed by Sivaloganathan (1986).

Strutted excavations are usually designed, or at least they are checked, using the apparent pressure diagrams given by Peck (1969b) and the Japan Society of Civil Engineers (1977). These pressure diagrams are known to be conservative. Apparent pressure diagrams, with maximum pressures less than those given by Peck, were derived for American and Brazilian residual soils (see Section 7.2.4). However, there are no reported field measurements of strut loads to establish the apparent pressure diagrams for Hong Kong residual soils and saprolites.

Despite its limitations (see Section 5.4), the 'beam on elastic foundation' approach is commonly used in Hong Kong for the design of excavation support systems. The values of coefficient of subgrade reaction recommended by Terzaghi (1955), although known to be conservative (Habibagahi & Langer, 1984), is normally employed in the calculations. The discrete element method utilising the relationship between lateral earth pressure and displacement has also been used. The boundary element method was employed to predict the performance of the basement for the new Hong Kong Bank headquarters building (Fitzpatrick & Willford, 1985; Humpheson et al, 1986). However, there are no reported case studies in which the finite element method is used for the design of excavation support systems.

In predicting ground movements around excavations, the settlement curves of Peck (1969b) and O'Rourke et al (1976) (which are given in Geoguide 1) are often used. It is doubtful whether these curves, which were compiled from data in North America, are directly applicable to Hong Kong residual soils and saprolites. A large amount of data have been obtained during the construction of the Mass Transit Railway (MTR) underground stations and other deep basement excavations. Some of the MTR settlement data have been published (e.g. Morton et al, 1980 a & b and Budge-Reid et al, 1984). It would therefore be useful to try to establish settlements curves from these data, similar to those derived by Peck, for Hong Kong residual soils and saprolites.

An important parameter that affects the magnitude of the strut loads, and the deformation of the soil, is the coefficient of earth pressure at rest, K_0 (see Section 5.6). Although there is evidence to suggest that the insitu horizontal stress in Hong Kong residual soils and saprolites is low, insitu measurement of K_0 is lacking.

Failures of sheet piling in Hong Kong were reviewed by Malone (1982). Case histories of excavation support failures in other countries are given by Sowers & Sowers (1967), Broms & Stille (1976), and Daniel & Olson (1982). These studies show that failures of the excavation support system are seldom due to inadequacies in design. Instead they are often caused by poor construction practice and inadequate site supervision. This indicates that current design methods are adequate.

11. GEOTECHNICAL CONTROL OF EXCAVATIONS IN HONG KONG

All private development in Hong Kong is controlled by the Building Authority under the Buildings Ordinance and its subsidiary Regulations (Government of Hong Kong, 1985). The geotechnical aspects of submissions are checked by the Geotechnical Control Office. Practice Notes listing the requirements on submissions are issued by the Building Authority to Authorized Persons and Registered Structural Engineers. The following Practice Notes are relevant to excavation submissions :

(a) PNAP : 74 - Dewatering in Foundation and Basement Excavation Works.

(b) PNAP : 78 - Requirements for a Geotechnical Assessment at General Building Plan Stage - Building (Administration) Regulation 8(1)(ba).

(c) PNAP : 83 - Requirements for Qualified Supervision of Site Formation Works - Building Ordinance section 17.

Practice Note PNAP : 78 lists the circumstances under which a geotechnical assessment of a site for the proposed works is required to be submitted to the Building Authority at the "general building plans" stage, i.e. the stage at which planning and fundamental approval for building is given. One of these circumstances is when the depth of the excavation exceeds 7.5 m. This requirement permits specialist geotechnical involvement at a stage when the developer's planning is still flexible, thus avoiding possible delays due to objections or constraints when detailed engineering plans are submitted. It should be noted that, irrespective of the depth of excavation, the Building Authority can, under the Buildings Ordinance, require the details of excavation and lateral support to be submitted for agreement or approval.

Where dewatering is to be undertaken to facilitate excavation works, the Practice Note PNAP : 74 lists the general requirements that should be considered when preparing foundation and basement excavation submissions to the Building Authority. These include the submission of the following :

(a) the details of the dewatering proposals,

(b) an assessment of the effects of excavation and dewatering,

(c) the method and sequence of construction,

(d) the location and details of instrumentation for monitoring the effects of the works on adjoining buildings, streets, land and underground services, the time interval between reading of instruments and the limiting criteria for movements and groundwater level drawdown,

(e) the ground investigation report, including laboratory testing results,

(f) shoring/underpinning details, and

(g) piezometric and settlement monitoring records.

Practice Note PNAP : 83 lists the requirements for three classes of qualified supervision of site formation and related works. Supervision by a full time resident engineer (viz. class (iii) supervision) is normally specified by the Building Authority for works which involve deep excavations in reclaimed ground, and excavation and lateral support works involving complicated working procedures and staging of works. The Authorized Person/Registered Structural Engineer is responsible for ensuring that the required supervision is provided.

It should be noted that at the time of publication of this document, legislation is being finalised to incorporate provisions in the Building (Administration) Regulations which refer specifically to the submission of an excavation and lateral support plan. When enacted, these regulations will empower the Building Authority to require the submission of comprehensive information relating to the design of excavations and the lateral support system. A Practice Note is also being prepared to give simple guidelines on the requirements for the submission of such information.

For excavations constructed for public authorities, checking of submissions is also carried out by the Geotechnical Control Office where this is warranted in the interest of public safety.

12. CONCLUSIONS AND RECOMMENDATIONS

A review of the state of the art of designing excavation support systems and of predicting ground movements around excavations leads to the following conclusions :

(a) Failures of excavation support systems are seldom due to inadequacies in design methods, but rather are caused by poor construction practice (e.g. inadequately planned working procedure and poor structural detailing) and inadequate site supervision. This indicates that current design methods are adequate.

(b) For the design of excavation support systems and the prediction of movements around excavations in Hong Kong, the recommendations given in Geoguide 1 (GCO, 1982), the Terzaghi & Peck (1967) diagrams for strut loads, and Peck's (1969b) settlement curves respectively, are often employed. The 'beam on elastic foundation' approach and the boundary element method are also used.

(c) Proper evaluation of water pressure and its effects is of utmost importance in the design of excavation support systems. The groundwater flow regime can have a significant effect on the water pressures, the earth pressures and the piping and heave potential of the ground. Proper ground investigation is necessary to obtain information on the ground and groundwater conditions for design. The location, size and condition of water-carrying services in the vicinity of the proposed excavation should also be established.

(d) Ground movements around excavations can be caused by many factors, some of which cannot be quantified with confidence. Therefore, it is essential that a monitoring system is implemented during construction to check design assumptions and to provide warning to allow precautionary measures to be undertaken where required.

(e) There are no reported field measurements of strut loads to establish the apparent pressure diagrams for Hong Kong residual soils and saprolites, which may be lower than those of Terzaghi & Peck (1967).

(f) Conservative values of coefficient of horizontal subgrade reaction recommended by Terzaghi (1955) are usually employed in the 'beam on elastic foundation' approach. Back analysis of monitoring data from excavations should be carried out to obtain more realistic values of ground stiffness for Hong Kong residual soils and saprolites.

(g) Although there is some evidence to suggest that the insitu horizontal stress in residual soils and saprolites is low, measurements of K_0 are lacking. Suitable methods for measuring K_0 in the field should be investigated.

(h) A large amount of settlement data has been obtained during the construction of the Mass Transit Railway underground stations and other deep basement excavations. It would be useful to try to establish settlement curves from these data, similar to those derived by Peck, for Hong Kong residual soils and saprolites.

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TABLES

LIST OF TABLES

Table 1 - Advantages and Disadvantages of Various Temporary and Permanent Support Systems (after Institution of Structural Engineers, 1975) (Sheet 1 of 2)

Types of Temporary and Permanent Support Systems	Figure	Advantages	Disadvantages
1. Temporary support strutted against central dumpling	1 (a)	(i) Suitable for large excavations in plan rather than in depth. (ii) Evades groundwater problems if sheet piling can effect seal in underlying stratum.	(i) Slow and radically constrains programme and access. (ii) Wall has to be self-supporting to withstand soil pressures when dumpling removed.
2. Temporary support by fully braced trench	1 (b)	(i) Suitable for excavations relatively large in extent rather than depth. (ii) Evades groundwater problems if sheet piling can effect seal in underlying stratum.	(i) Slow and restrains construction programme. (ii) Wall has to be self-supporting against soil pressures when basement area is excavated.
3. Fully braced temporary support	1 (c)	(i) Suitable for very deep excavations. (ii) Traditional. (iii) With incorporation of jacks preloading can be used where movements must be restricted to minimum.	(i) Slow, sure and very costly, particularly as width of excavation increases. (ii) Constrains construction programme greatly because of access difficulties.
4. Permanent wall constructed prior to excavation and braced from central dumpling	1 (d)	Variant of 1(a), somewhat cheaper, same advantages apply.	Same disadvantages as 1(a) apply, except that wall is not so massive, support usually derived from part of permanent work when dumpling removed.
5. Permanent wall constructed prior to excavation and braced from central permanent construction	1 (e)	(i) Good for relatively deep excavations provided that distance of perimeter wall from central construction is not too great. (ii) Can exercise control of movements satisfactorily with prestressing. (iii) Can exploit use of permanent work in temporary condition.	(i) Central structure must be massive. (ii) Passive soil buttress not so effective in restraining lateral movement of wall as a strut.
6. Ground anchors	1 (f)	(i) Provides clean unobstructed area for basement construction. (ii) Specially attractive when anchored into rock foundation.	Great care necessary to prevent movement of ground anchors and hence wall.

Table 1 - Advantages and Disadvantages of Various Temporary and Permanent Support Systems (after Institution of Structural Engineers, 1975) (Sheet 2 of 2)

Types of Temporary and Permanent Support Systems	Figure	Advantages	Disadvantages
7. Cantilevered walls	1 (g)	(i) Provides unobstructed area for construction. (ii) Quick to install.	Great care required during concreting of deep walls especially if water exclusion is also a requirement.
8. Caissons	1 (h)	(i) Rapid construction once started. (ii) Provides unobstructed working space inside. (iii) More suited to circular basements.	(i) Extremely careful construction required to achieve verticality and positioning. (ii) Boulders can retard construction. Hence, very careful ground investigation required.
9. Stabilization systems	1 (j)	Only used as last resort when other methods do not work.	(i) Usually costly and time consuming. (ii) May not eliminate groundwater problems.
10. Concurrent upward and downward construction	1 (k)	(i) Good for deep excavations. (ii) Affords speedier construction on superstructure.	Excavation and removal of spoil from enclosed area relatively difficult.
11. Long flying shores across excavations	1 (l)	Variant of 1(c), but suitable for narrower excavations.	(i) Impedes construction. (ii) Incorporation of monitoring jacks more difficult than for method 1(c).
12. Floors cast on ground with excavation continuing below	1 (m)	(i) Good method for deep excavation. (ii) Temporary strutting eliminated. (iii) Temporary passive soil buttresses eliminated.	Excavation under slabs and removal of spoil relatively difficult.

Table 2 - Factors Involved in the Choice of a Support System for a Deep Excavation (NAVFAC, 1982b)

Requirements	Lends Itself to Use of	Comments
1. Open excavation area	Tie-backs, raking struts or cantilevered walls	-
2. Low initial cost	Soldier pile or sheet pile walls; combined soil batter with wall	-
3. Use as part of permanent structure	Diaphragm or cylinder pile walls	Diaphragm walls most common as permanent walls
4. Deep, soft clay subsurface conditions	Strutted or raker-supported diaphragm or cylinder pile walls	Tie-back capacity not adequate in soft clays
5. Dense, gravelly sand or clay subsoils	Soldier pile, diaphragm or cylinder pile walls	Sheet piles may lose interlock on hard driving
6. Deep, overconsolidated clays	Struts, long tie-backs or combination tie-backs and struts	High insitu lateral stresses are relieved in overconsolidated soils. Lateral movements may be large and extend deep into soil
7. Avoid dewatering	Diaphragm walls, possibly sheet pile walls in soft subsoils	Soldier pile walls are pervious
8. Minimize movements	High preloads on stiff strutted or tied-back walls	Analyze for stability of bottom of excavation
9. Wide excavation (greater than 20 m wide)	Tie-backs or raking struts	Tie-backs preferred except in very soft clay subsoils
10. Narrow excavation (less than 20 m wide)	Cross-lot struts	Struts more economical but tie-backs still may be preferred to keep excavation open

Table 3 - Design Considerations for Strutted and Tied-back Walls (after NAVFAC, 1982b)

Design Factor	Comments
1. Water loads	Often greater than earth load on impervious wall. Recommend to install piezometers during construction to monitor water levels. Should consider possible lowering of water pressures as a result of seepage through or under wall. Dewatering can be used to reduce water loads. Seepage under wall reduces passive resistance.
2. Stability	Consider possible instability in any passive soil buttress or exposed batter. Sliding potential beneath the wall or behind tie-backs should be evaluated. Deep-seated bearing failure under weight of supported soil to be checked in weak soils. Stability calculation should consider weight of surcharge or weight of other facilities in close proximity to excavation.
3. Piping	Loss of ground caused by high groundwater table in silty and fine sandy soils. Difficulties occur due to flow beneath wall, through bad joints in walls, or through unsealed sheet pile handling holes. Dewatering may be required.
4. Movements	Movements can be minimized through use of stiff wall supported by preloaded tie-backs or struts.
5. Dewatering-recharge	Dewatering reduces loads on wall systems and minimizes possible loss of ground due to piping. May cause settlements, requiring recharge outside of support system.
6. Surcharge	Construction materials usually stored near wall systems. Allowance should always be made for surcharge.
7. Prestressing of tie-backs or struts	Useful to remove slack from system and minimize soil movements.
8. Construction sequence	The amount of wall movement is dependent on the depth of excavation. The amount of load on the tie-backs is dependent on the amount of wall movement which occurs before they are installed. Movements of wall should be checked at every major construction stage. Upper struts should be installed early.
9. Temperature	Struts subject to load fluctuation due to temperature loads; may be important for long struts.

Table 4 - Recommended Factors of Safety for Overall Stability of Cantilevered and Propped Walls (Padfield & Mair, 1984)

Method		**Design Approach A**		**Design Approach B**		**Comments**
		Recommended range for moderately conservative parameters (c', ϕ', or C_u)		Recommended minimum values for worst credible parameters ($c' = 0$, ϕ')		
		Temporary works	Permanent works	Temporary works	Permanent works	
(a) Factor on Embedment Depth, F_d	Effective stress	1.1 to 1.2 (usually 1.2)	1.2 to 1.6 (usually 1.5)	Not recom-mended	1.2	This method is empirical. It should always be checked against one of the other methods.
	* Total stress	2.0	-		-	
(b) Factor on Moments Based on Gross Pressure, F_p	Effective stress	1.2 to 1.5	1.5 to 2.0	1.0	1.2 to 1.5	These recom-mended F_p values vary with ϕ' to be generally consistent with usual values of F_s and F_r.
	$\phi' \geqslant 30°$	1.5	2.0	1.0	1.5	
	$\phi' = 20$ to $30°$	1.2 to 1.5	1.5 to 2.0	1.0	1.2 to 1.5	
	$\phi' \leqslant 20°$	1.2	1.5	1.0	1.2	
	* Total stress	2.0	-	-	-	
(d) Factor on Moments Based on Net Available Passive Resistance, F_r	Effective stress	1.3 to 1.5 (usually 1.5)	1.5 to 2.0 (usually 2.0)	1.0	1.5	Not yet tested for cantilevers. A relatively new method with which little design experience has been obtained.
	* Total stress	2.0	-	-	-	
(e) Factor on Shear Strength on Both Active and Passive Sides, F_s	Effective stress	1.1 to 1.2 (usually 1.2 except for $\phi' > 30°$ when lower value may be used)	1.2 to 1.5 (usually 1.5 except for $\phi' > 30°$ when lower value may be used)	1.0	1.2	The mobilised angle of wall friction, δ_m, and wall adhesion, c_{wm}, should also be reduced.
	* Total stress	1.5	-	-	-	

Legend :

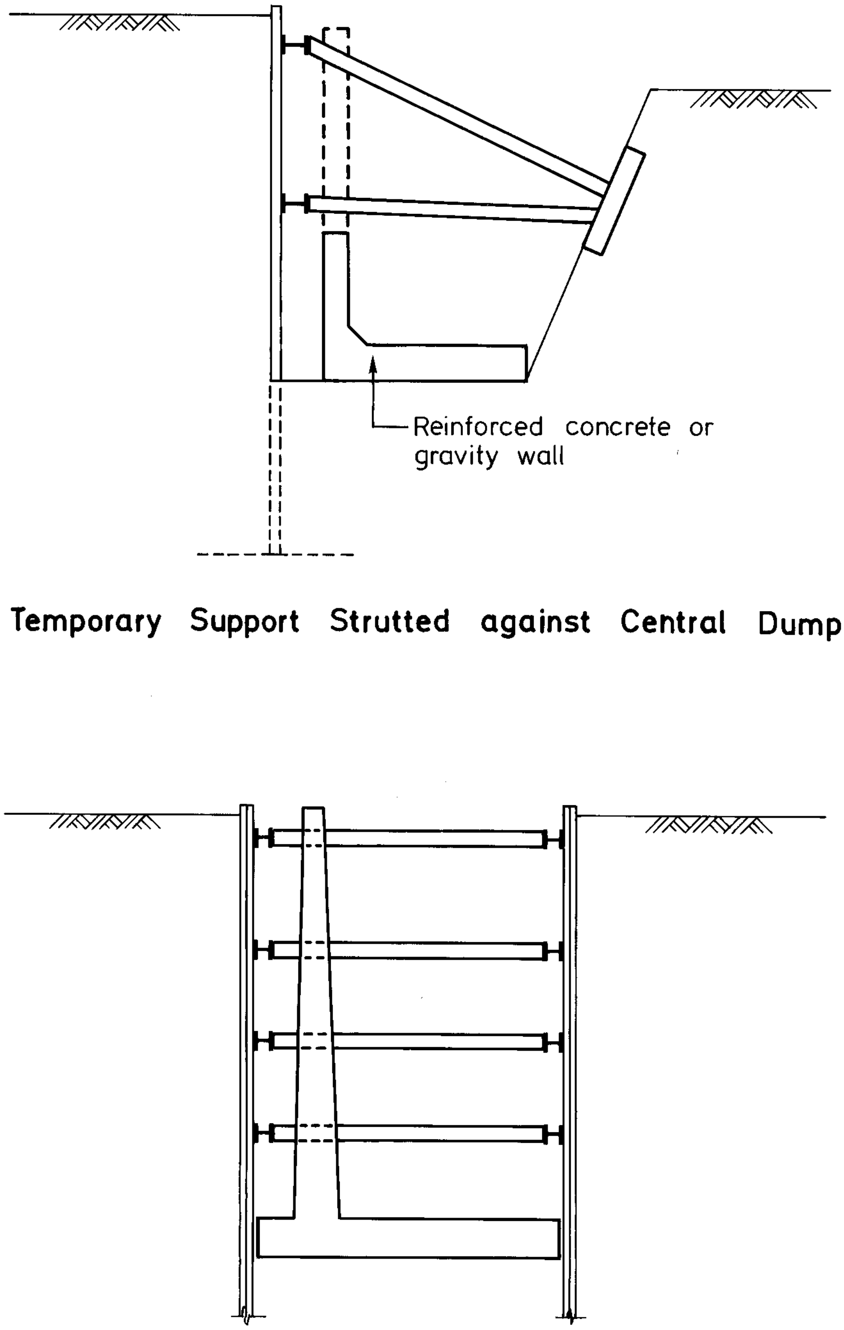
* Total stress factors are speculative, and they should be treated with caution.

Table 5 - Methods for Calculating Anchor Prestress Loads

Reference	Method
Clough, Weber & Lamont (1972)	Terzaghi & Peck rules (0.4 γH)
Grant (1985)	Terzaghi & Peck rules (0.25 - 0.3 γH)
Hanna & Matallana (1970)	Pressures halfway between active and at - rest
Larson, Willette, Hall & Gnaedinger (1972)	Pressures between active and at - rest
Liu & Dugan (1972)	15 × wall height (in psf)
Mansur & Alizadeh (1970)	At - rest pressures
McRostie, Burn & Mitchell (1972)	Terzaghi & Peck rules (0.4 γH)
NAVFAC (1982 b)	Sands : 0.4 - 0.5 $K_o \gamma H$ (rectangular distribution) Stiff to very stiff clays : 0.15 - 0.3 γH (rectangular distribution) Soft to medium clays : 0.5 - 0.6 γH (triangular distribution)
Oosterbaan & Gifford (1972)	Active pressures
Weber (1982)	Terzaghi & Peck rules (30 × wall height (in psf))

FIGURES

LIST OF FIGURES

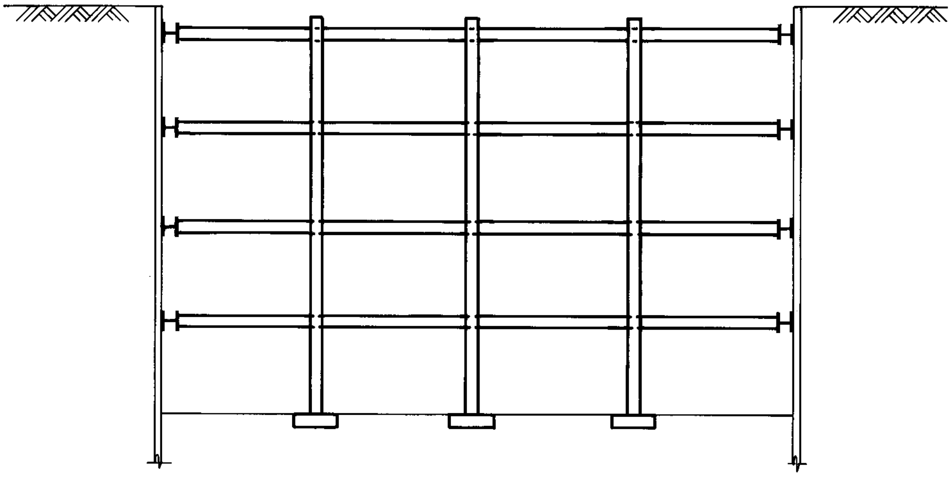


(a) Temporary Support Strutted against Central Dumpling

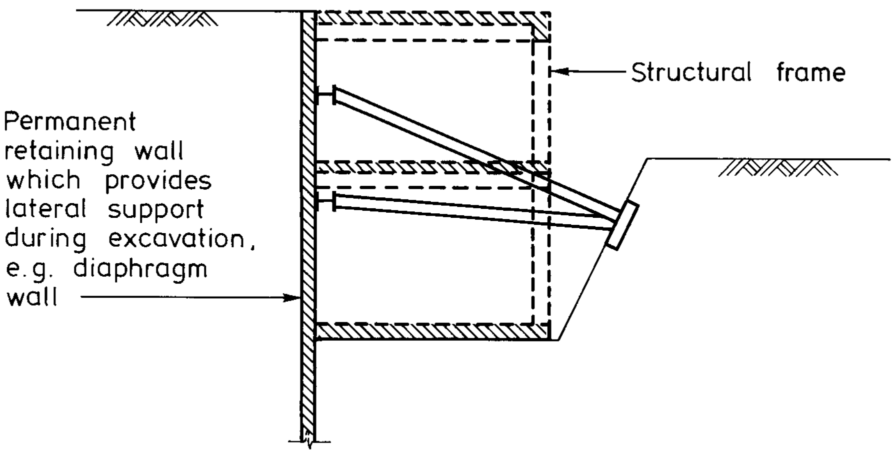
(b) Temporary Support by Fully Braced Trench

Note : Advantages and disadvantages of these systems are given in Table 1.

Figure 1 - Types of Temporary and Permanent Support Systems (after Institution of Structural Engineers, 1975) (Sheet 1 of 7)



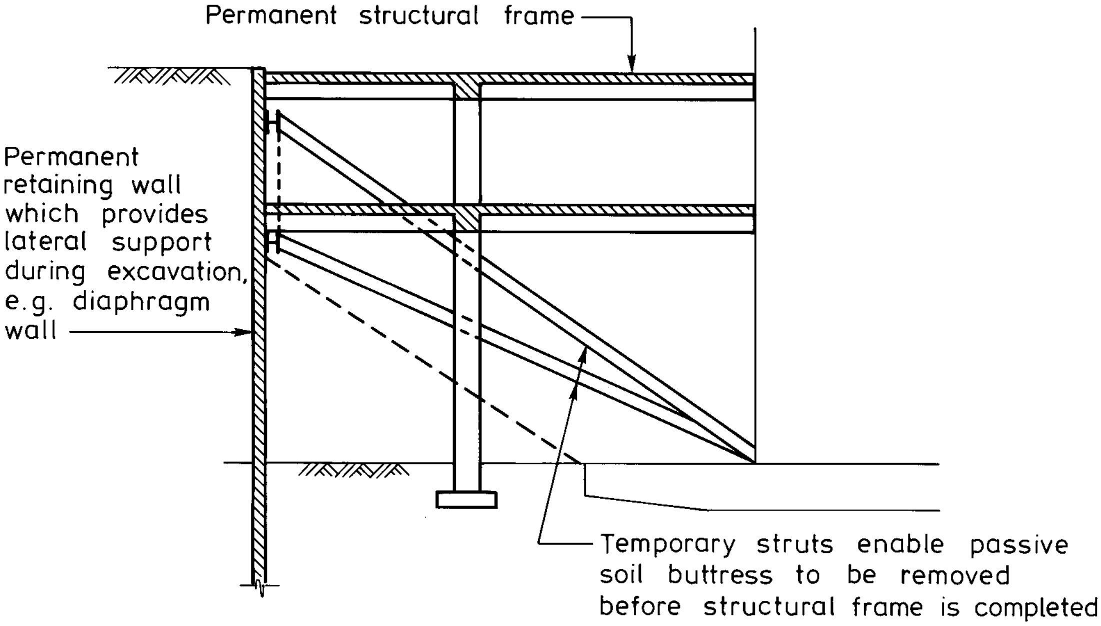
(c) Fully Braced Temporary Support



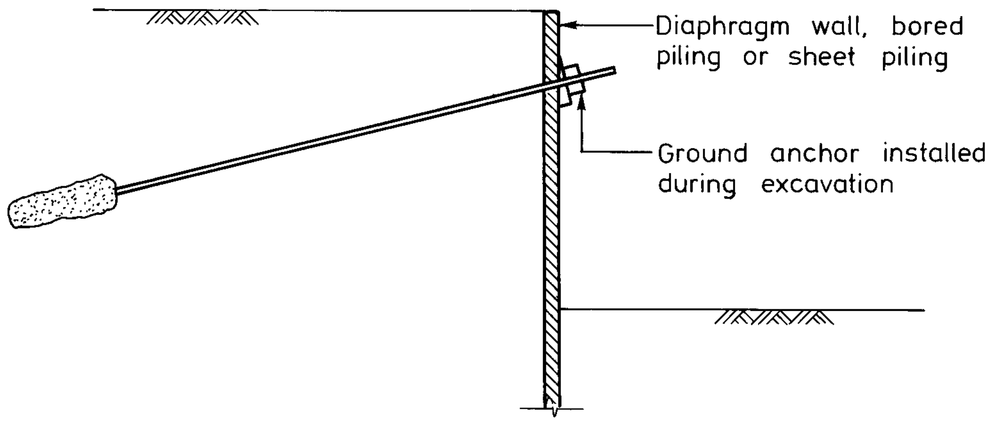
(d) Permanent Wall Constructed Prior to Excavation and Braced from Central Dumpling

Note : Advantages and disadvantages of these systems are given in Table 1.

Figure 1 - Types of Temporary and Permanent Support Systems (after Institution of Structural Engineers, 1975) (Sheet 2 of 7)



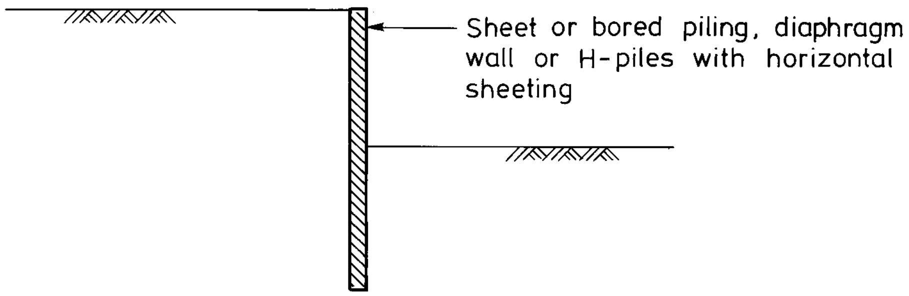
(e) Permanent Wall Constructed Prior to Excavation (Diaphragm Wall or Contiguous Bored Pile Wall) and Braced from Central Permanent Construction



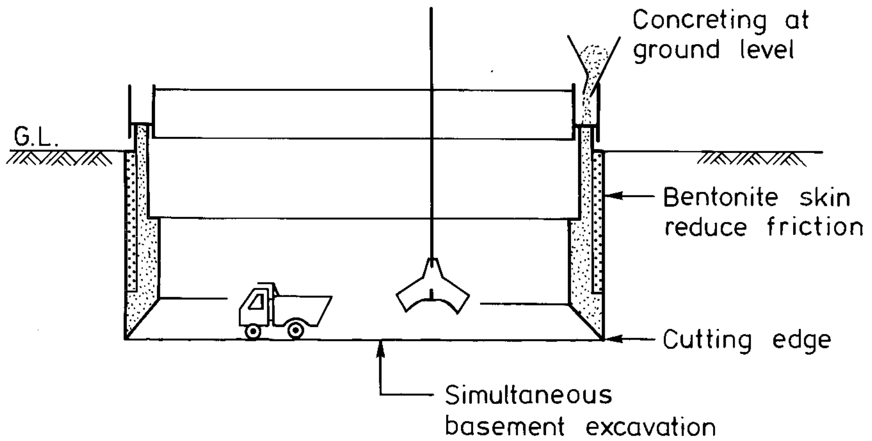
(f) Ground Anchors

Note : Advantages and disadvantages of these systems are given in Table 1.

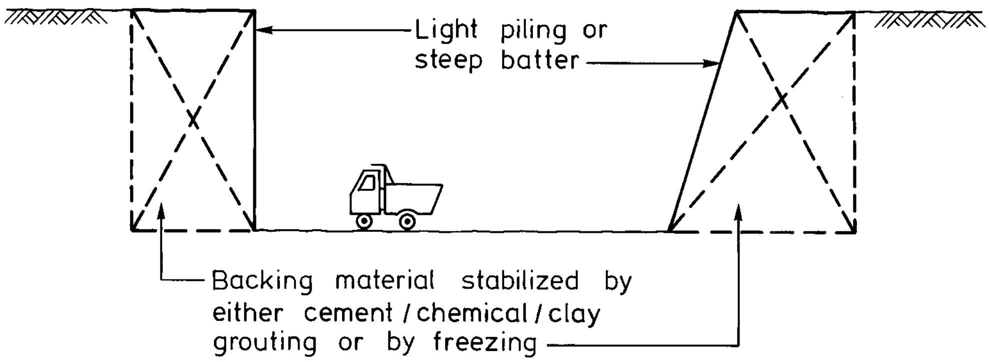
Figure 1 - Types of Temporary and Permanent Support Systems (after Institution of Structural Engineers, 1975) (Sheet 3 of 7)



(g) Cantilevered Wall



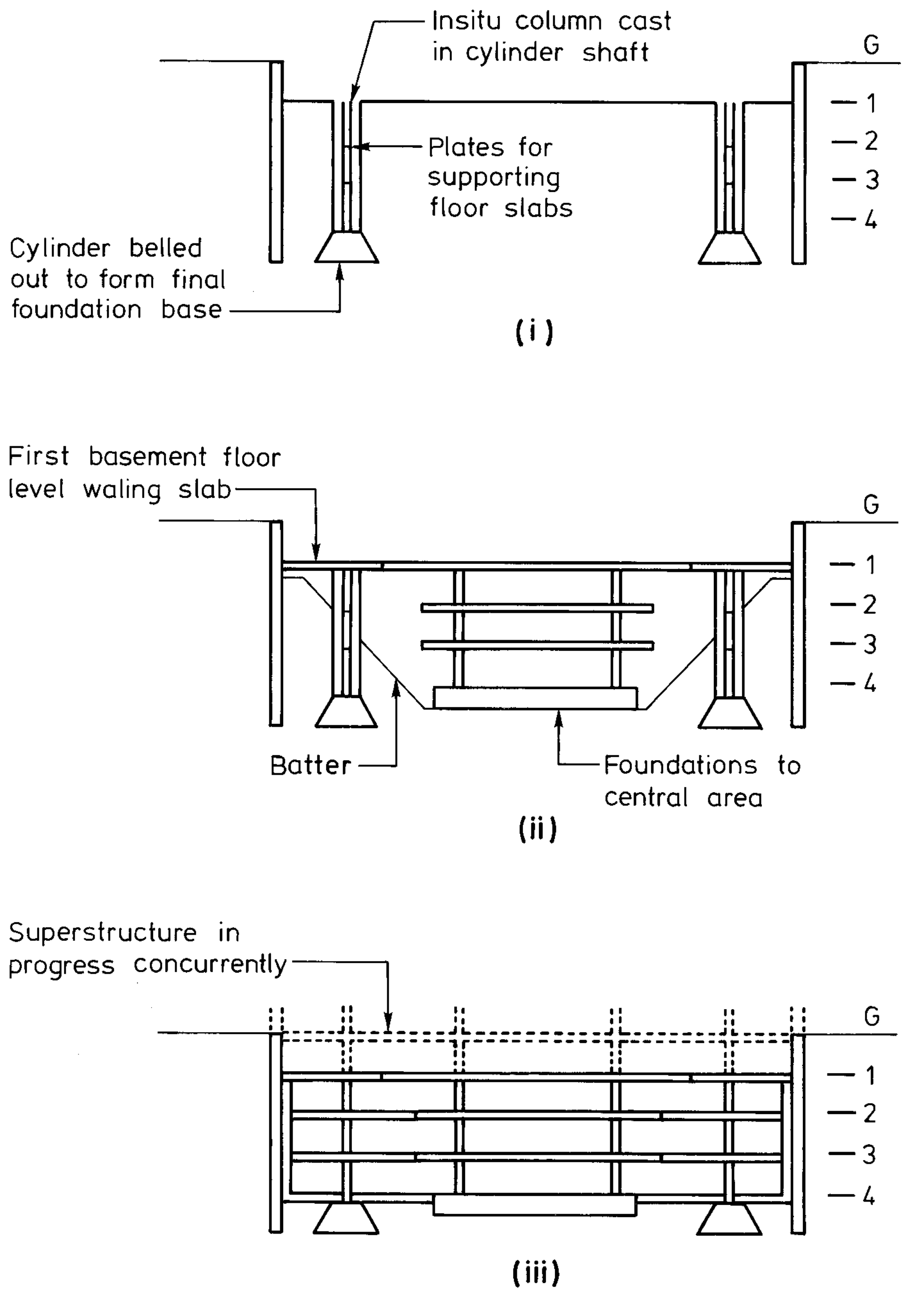
(h) Caissons



(j) Stabilization Systems

Note : Advantages and disadvantages of these systems are given in Table 1.

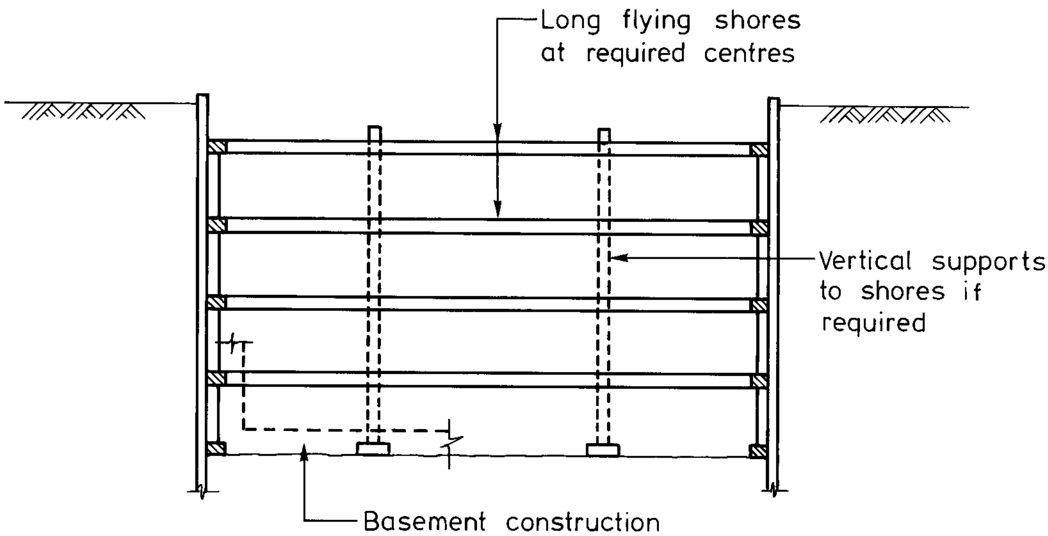
Figure 1 - Types of Temporary and Permanent Support Systems (after Institution of Structural Engineers, 1975) (Sheet 4 of 7)



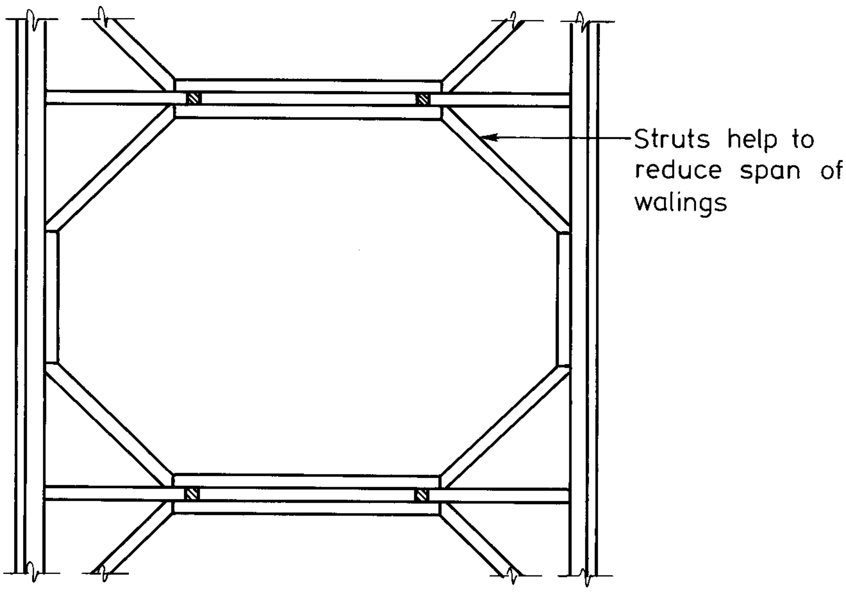
(k) Concurrent Upward and Downward Construction (Involving Progressive Underpinning)

Note : Advantages and disadvantages of these systems are given in Table 1.

Figure 1 - Types of Temporary and Permanent Support Systems (after Institution of Structural Engineers, 1975) (Sheet 5 of 7)



SECTION

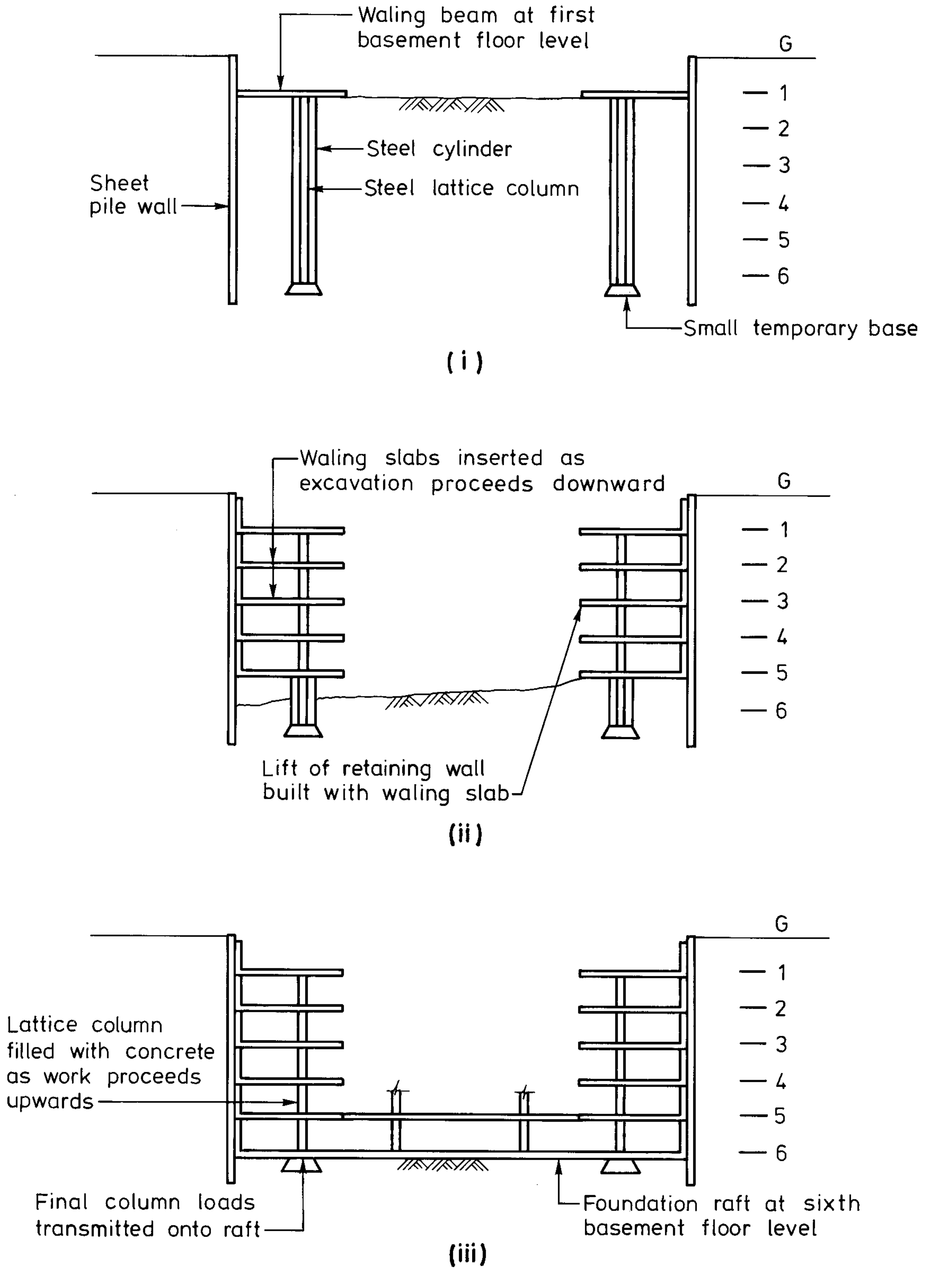


PLAN

(l) Long Flying Shores Across Excavation

Note : Advantages and disadvantages of these systems are given in Table 1.

Figure 1 - Types of Temporary and Permanent Support Systems (after Institution of Structural Engineers, 1975) (Sheet 6 of 7)



(m) Floors Cast on Ground with Excavation Continuing below

Note : Advantages and disadvantages of these systems are given in Table 1.

Figure 1 - Types of Temporary and Permanent Support Systems (after Institution of Structural Engineers, 1975) (Sheet 7 of 7)

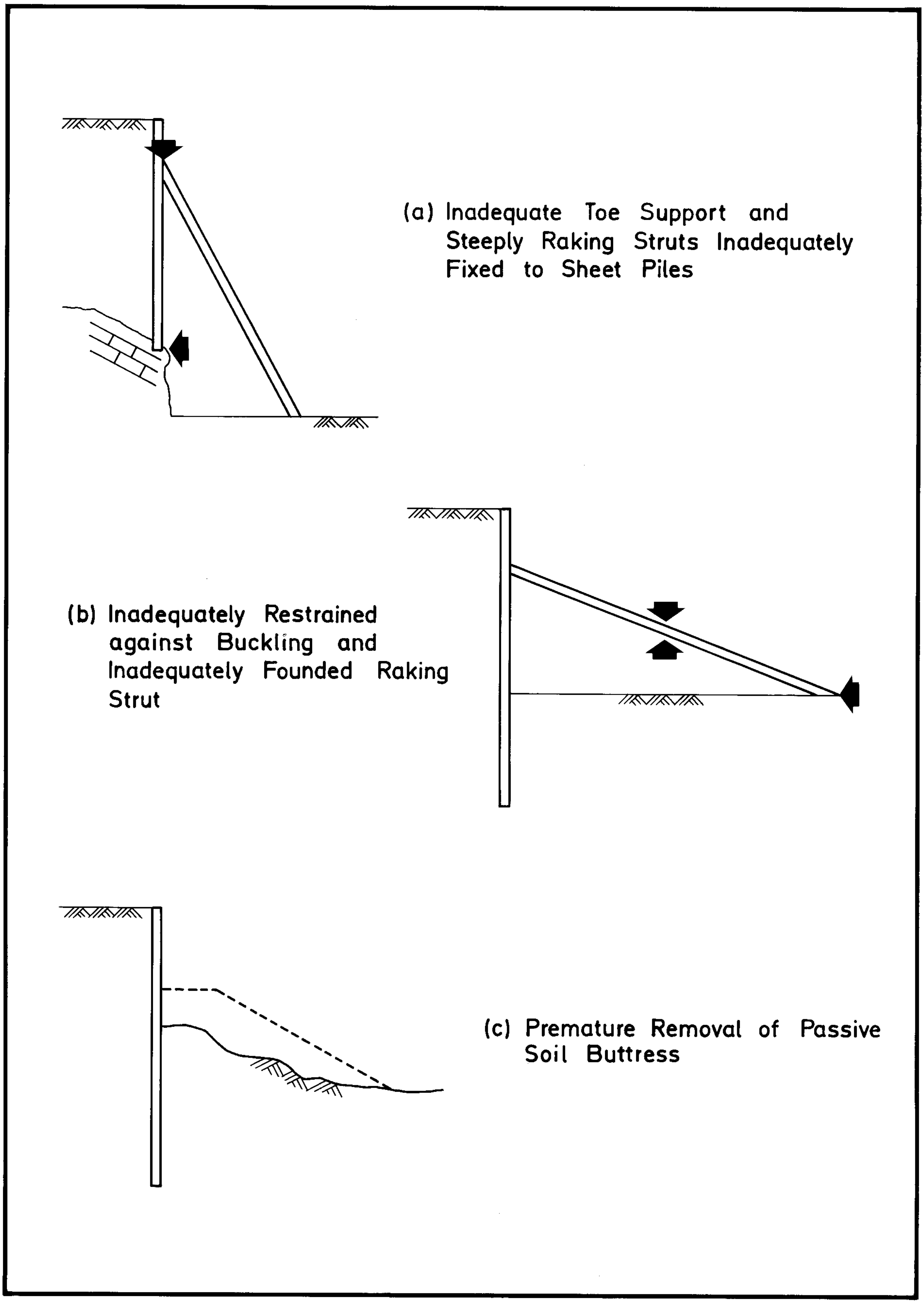


Figure 2 - Factors Causing Collapse of Sheet Pile Walls (after Malone, 1982)

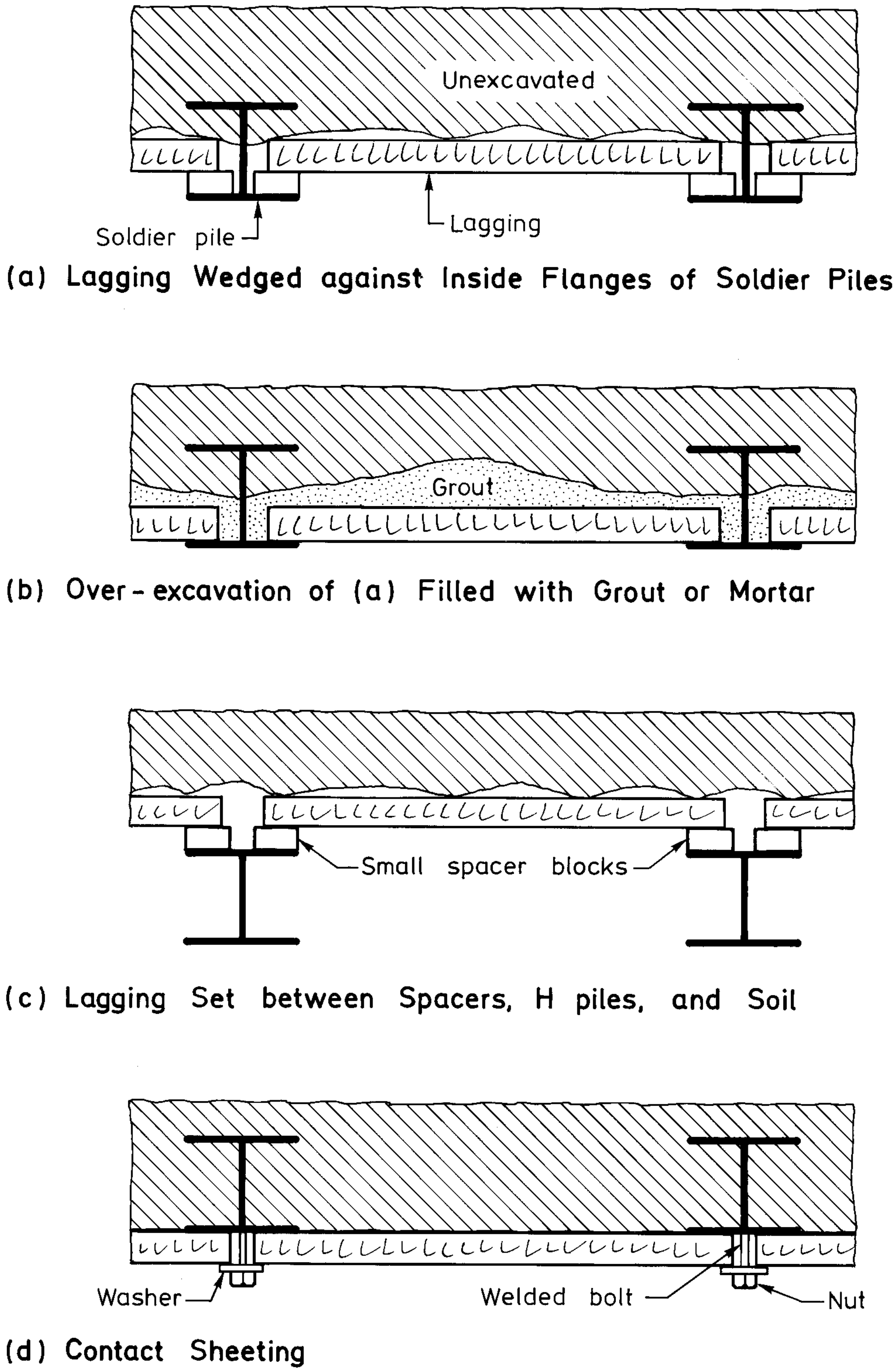


Figure 3 - Methods for Transferring Earth Pressure from Lagging to Soldier Piles (Peck, 1969b)

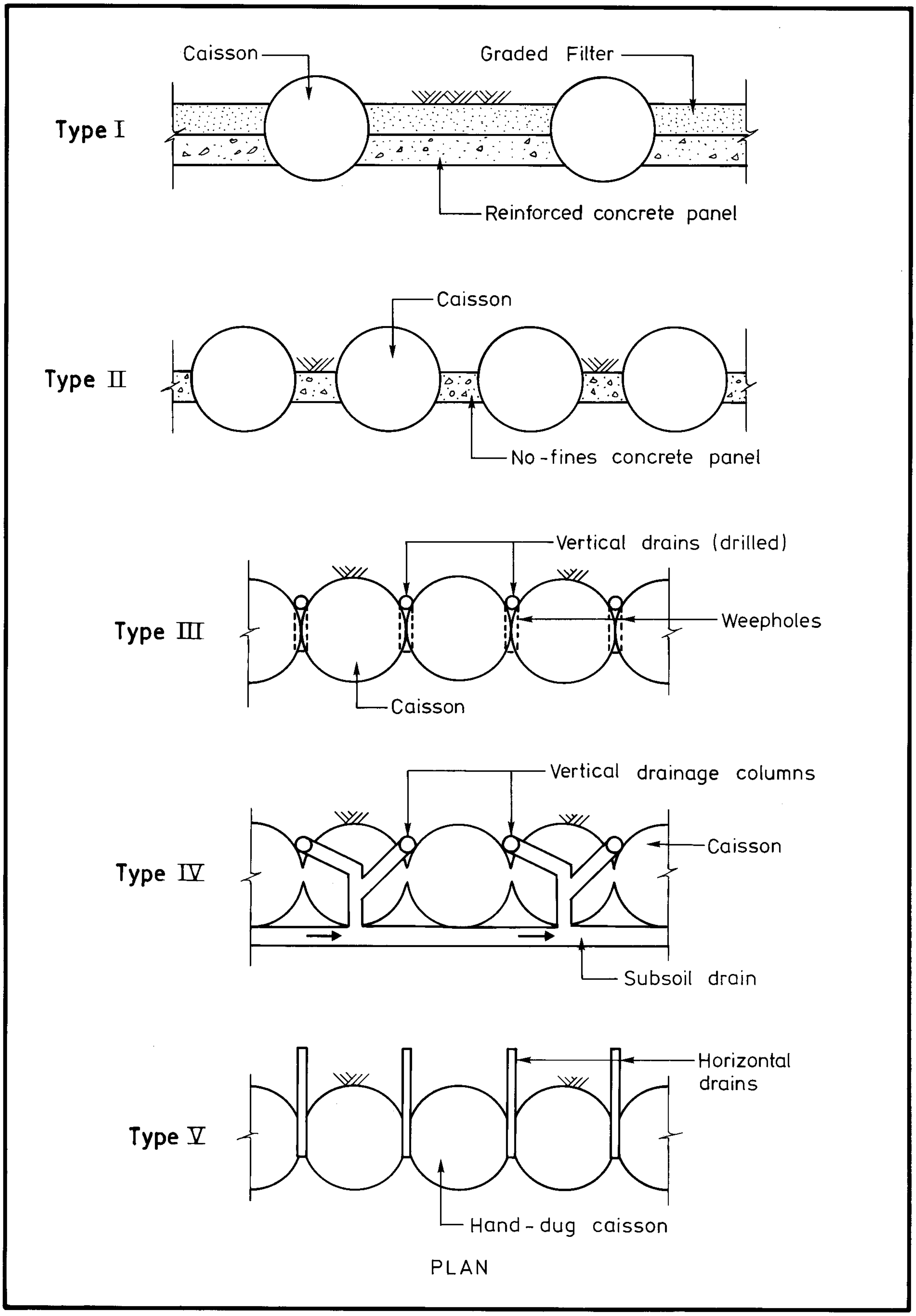


Figure 4 - Types of Caisson Wall Drainage System (Malone, 1982)

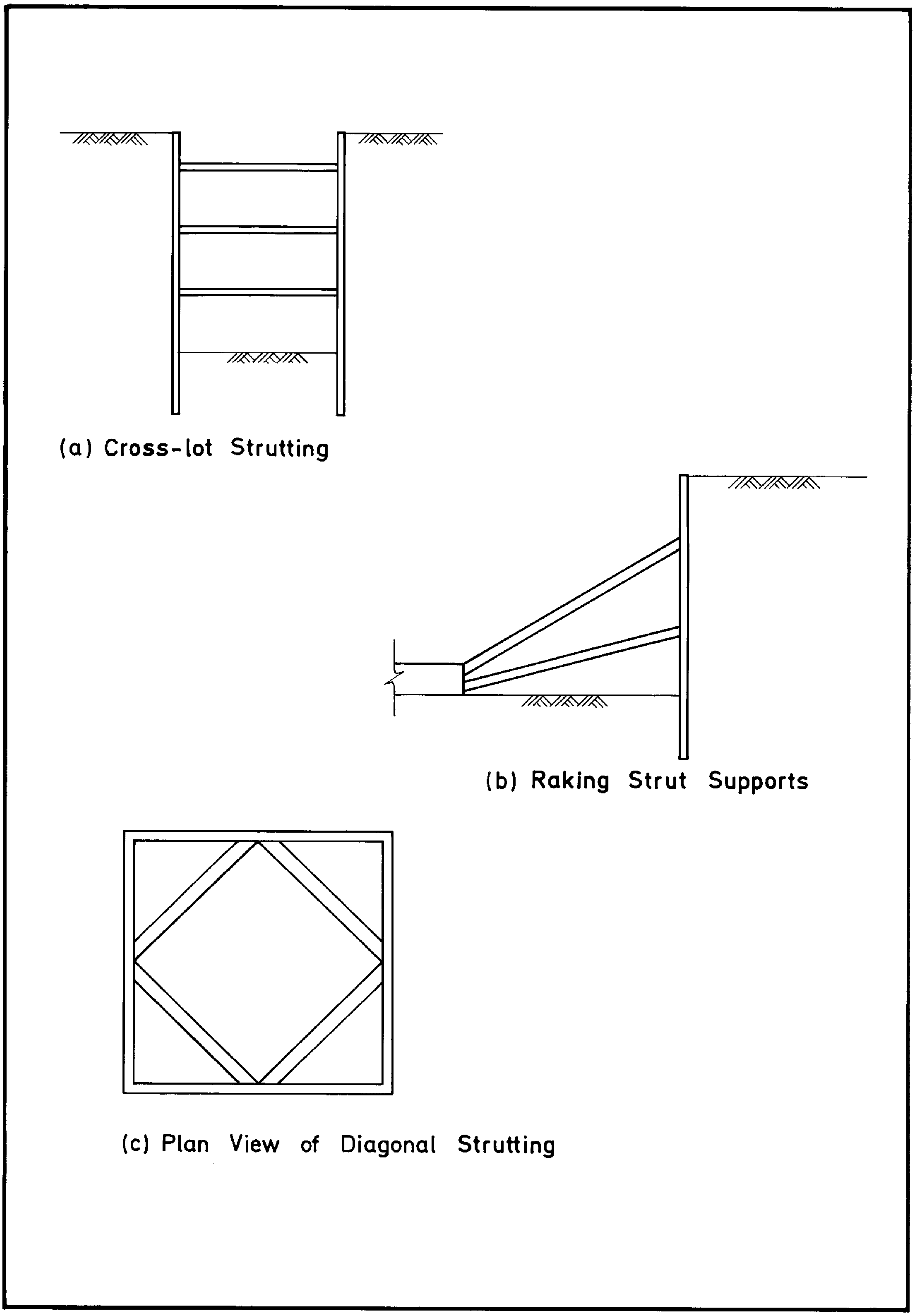
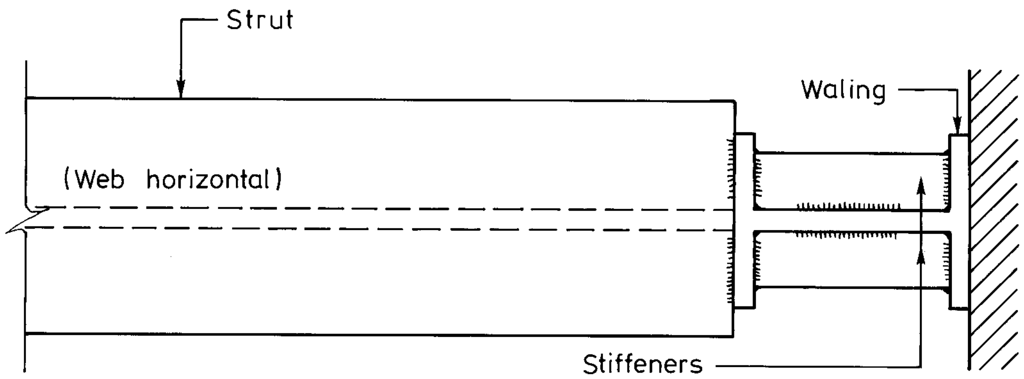
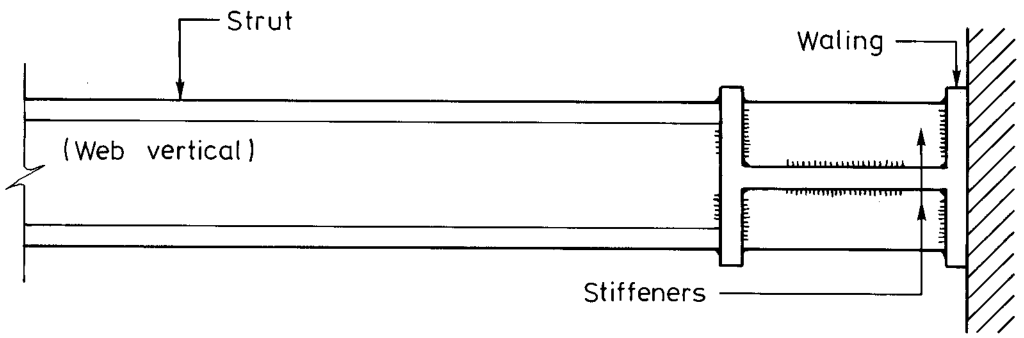


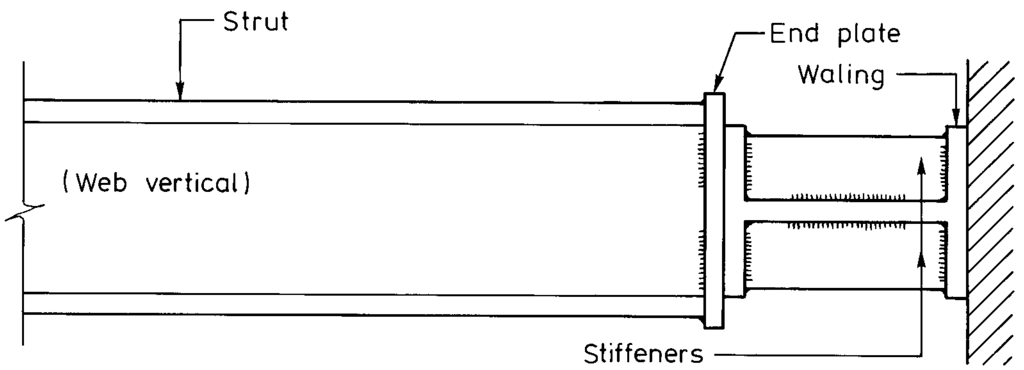
Figure 5 - Strutted Wall Support Systems (Clough, 1975)



(a) Arrangement with Strut Web in Horizontal Position



(b) Arrangement with Strut Web in Vertical Position and Strut Depth Smaller Than Waling Flange Width



(c) As in (b), But with Strut Depth Greater Than Waling Flange Width

Figure 6 - Typical Connection Details for Horizontal Struts (Goldberg et al, 1976)

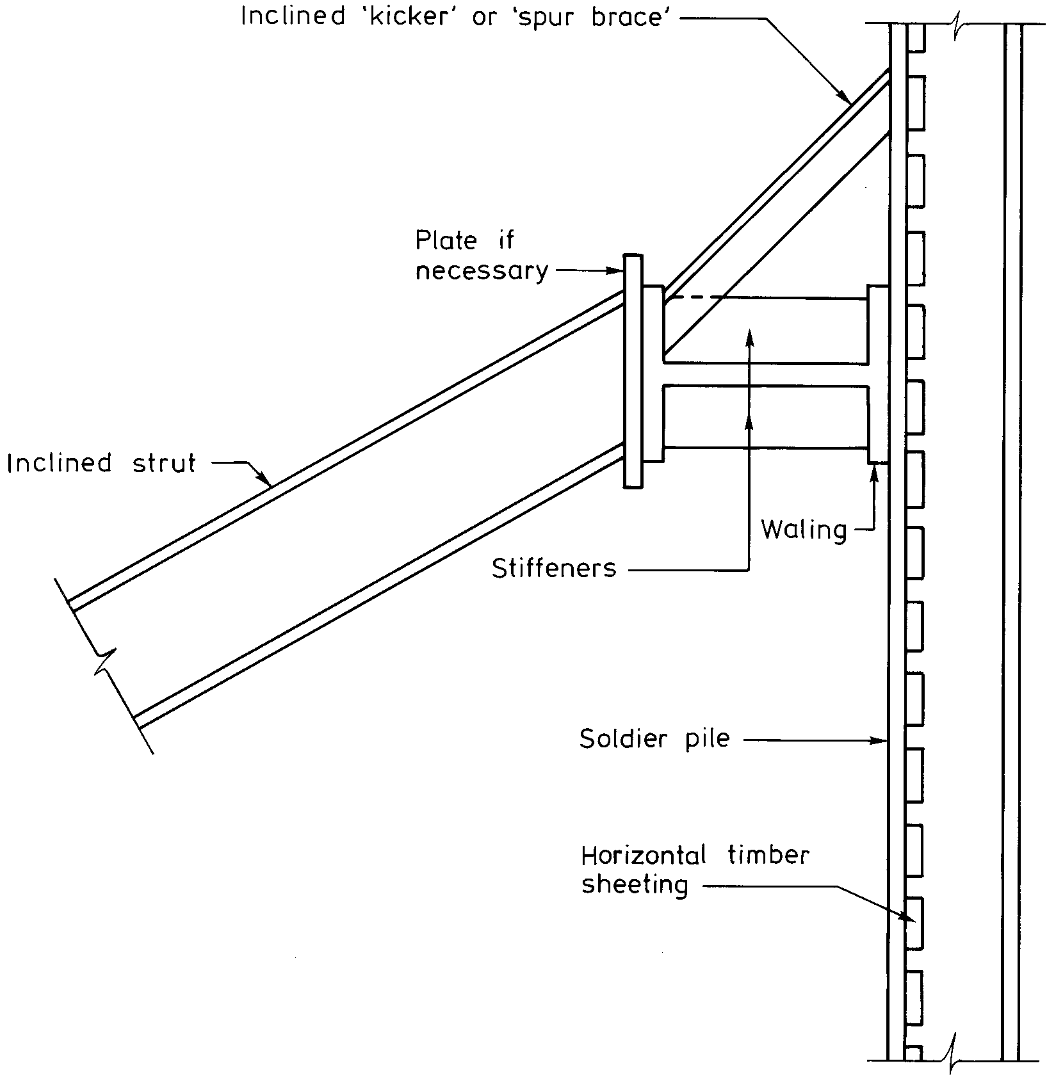


Figure 7 - Typical Connection Details for Inclined Strut and Horizontal Waling (Goldberg et al, 1976)

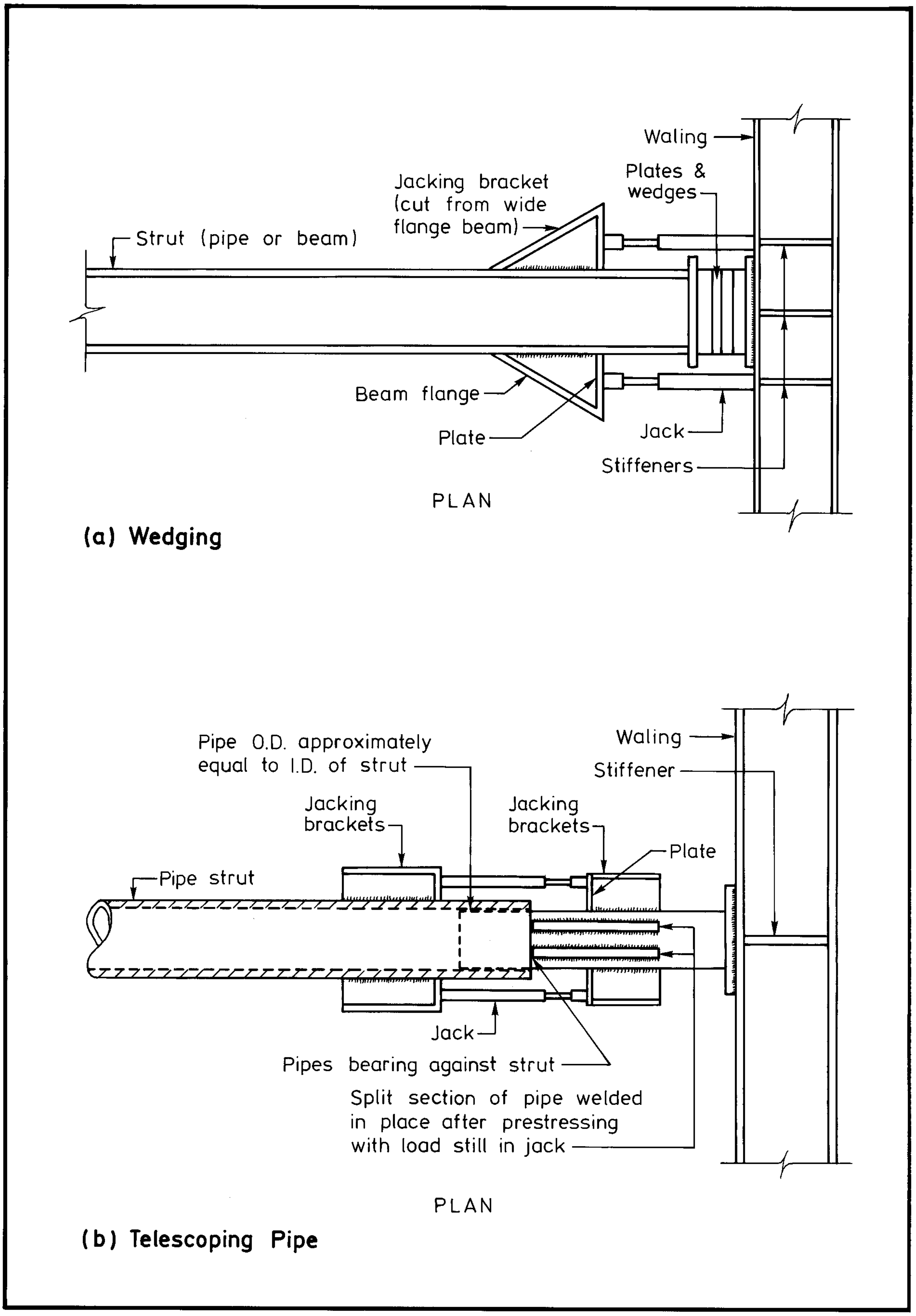


Figure 8 - Prestressing Details for Struts (Goldberg et al, 1976)

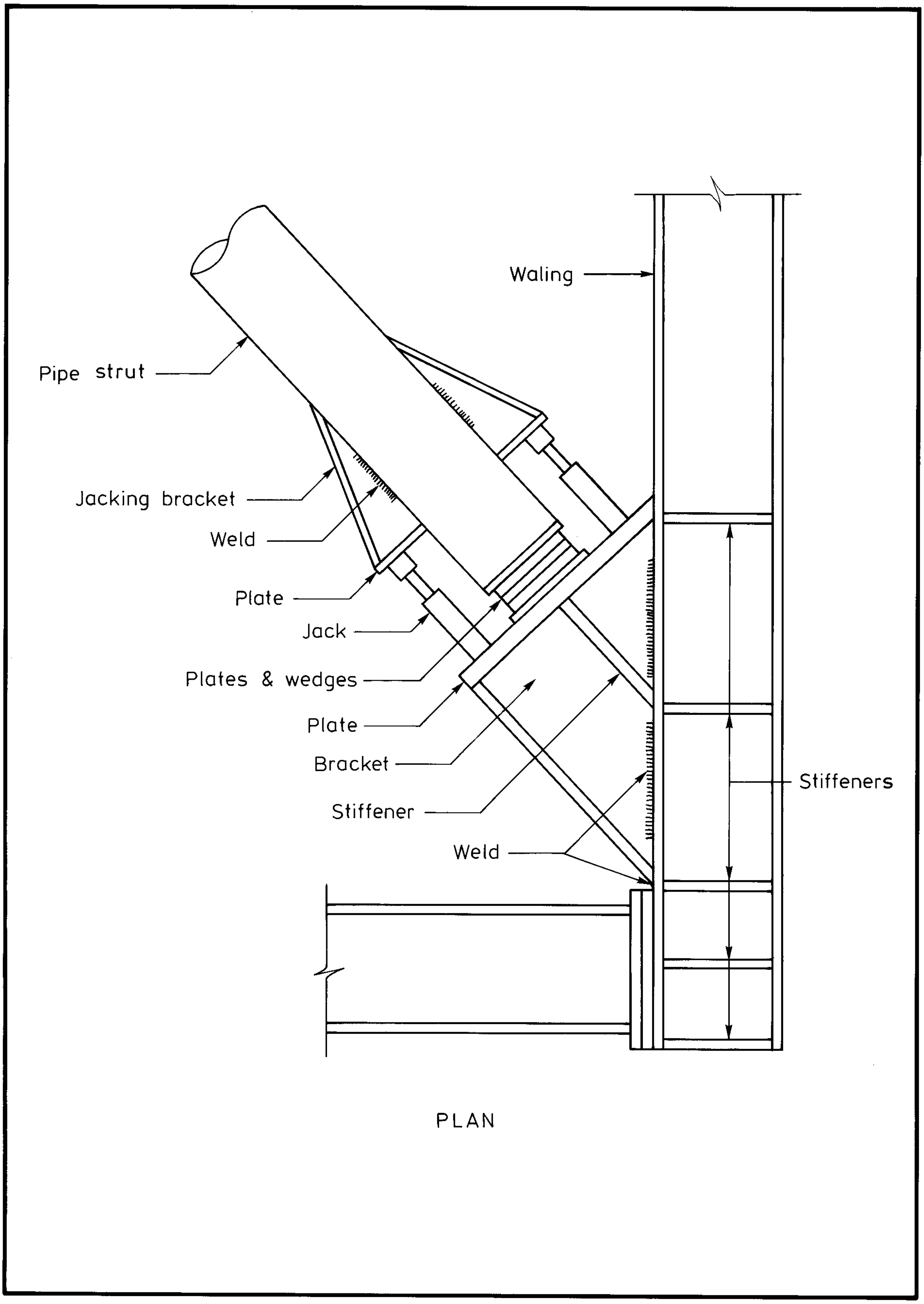


Figure 9 - Prestressing of Pipe Strut at a Corner Using Brackets as Reaction (Goldberg et al, 1976)

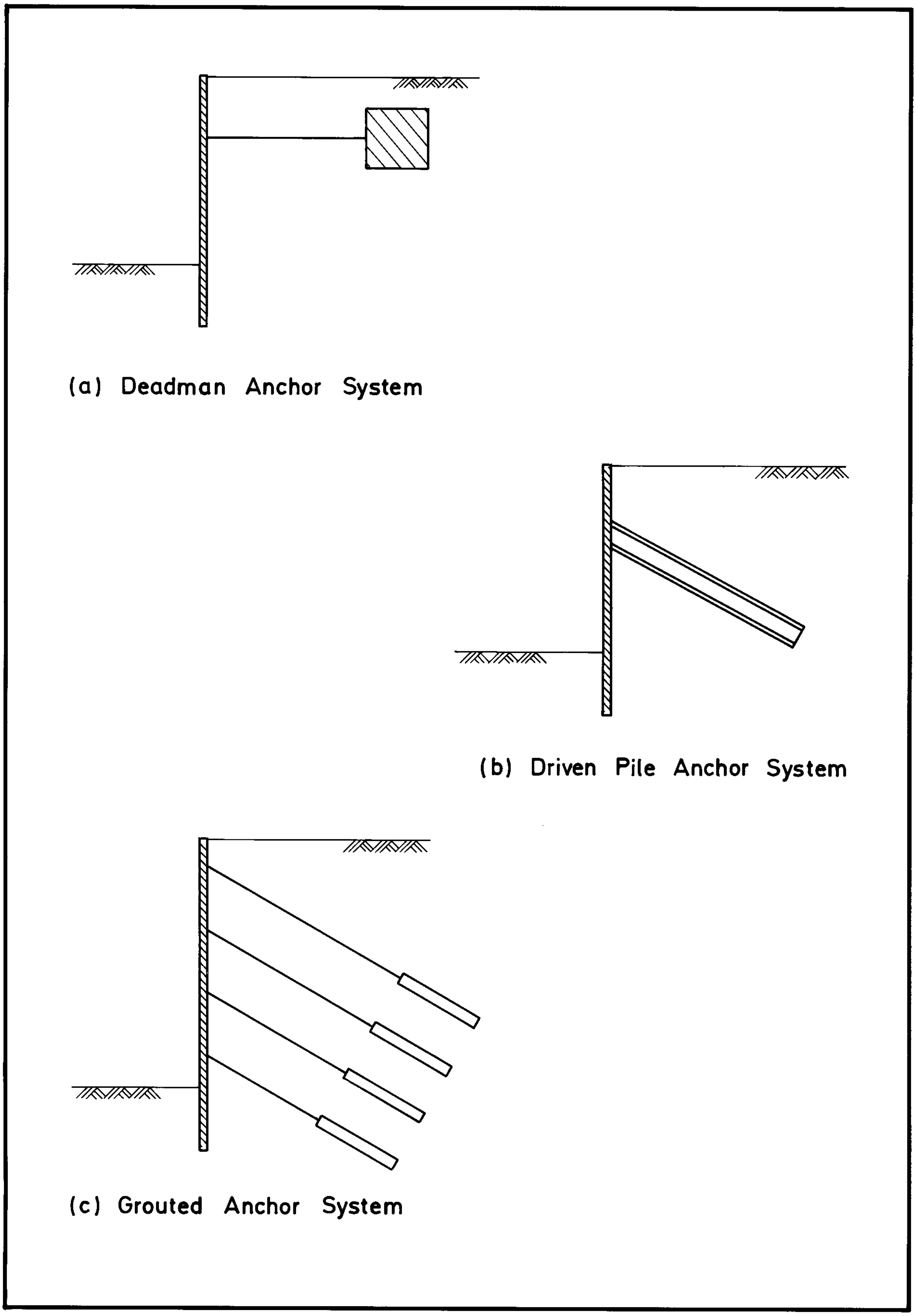
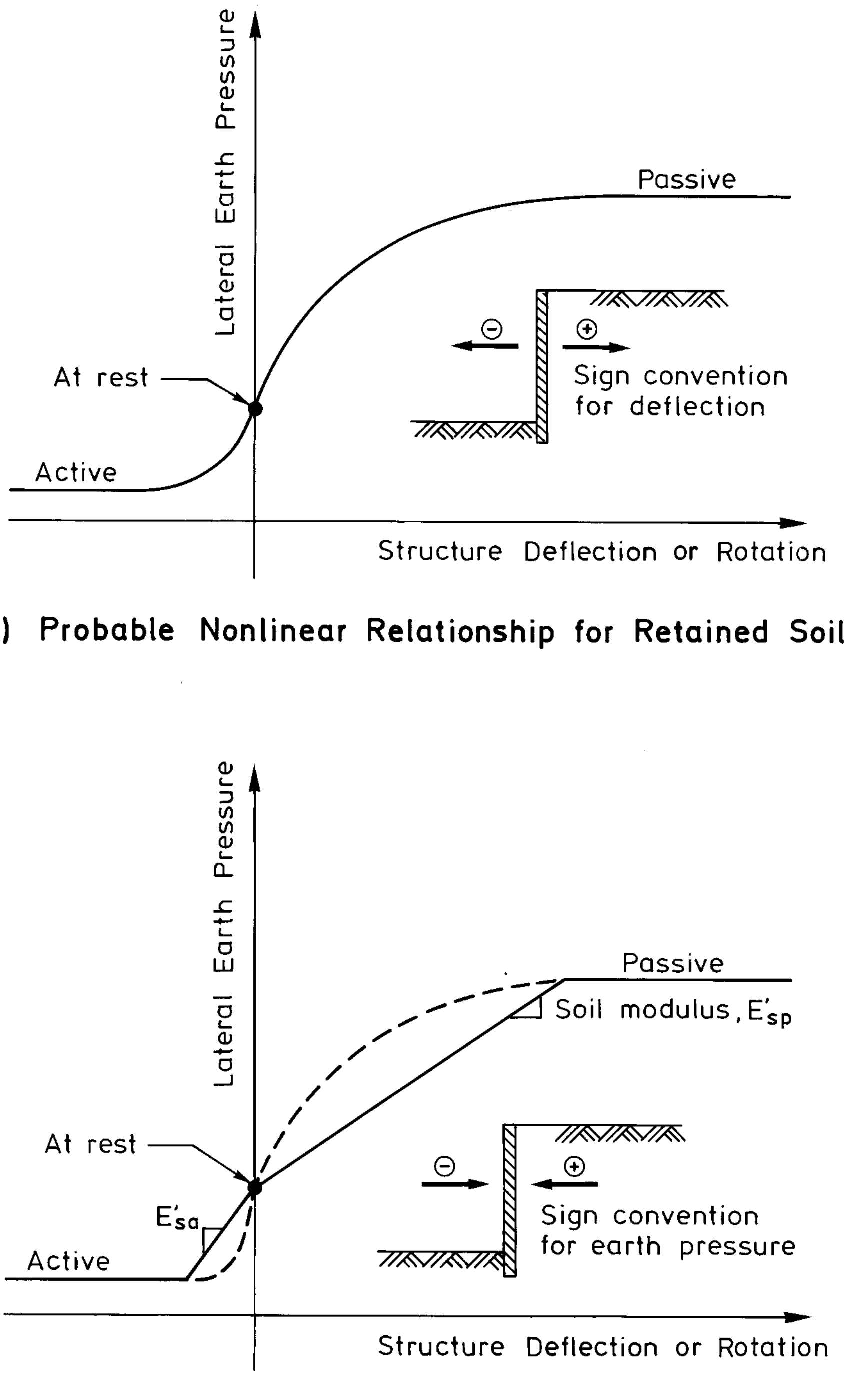


Figure 10 - Tie-back Support Systems (Clough, 1975)



(a) Probable Nonlinear Relationship for Retained Soil

(b) Simplified Nonlinear Relationship for Retained Soil

Figure 11 - Nonlinear Relationship for Soil behind Flexible Retaining Structure (Haliburton, 1968)

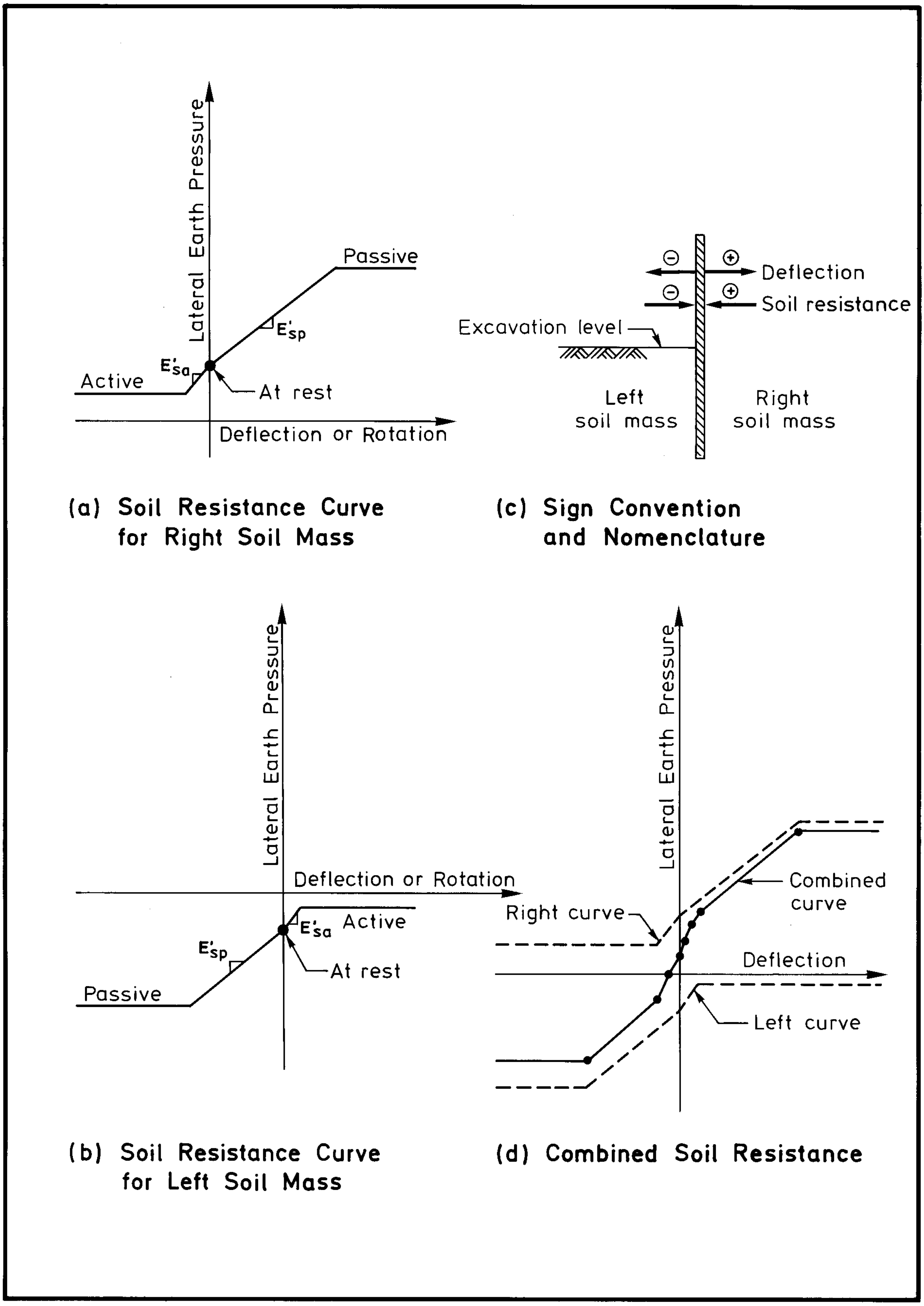
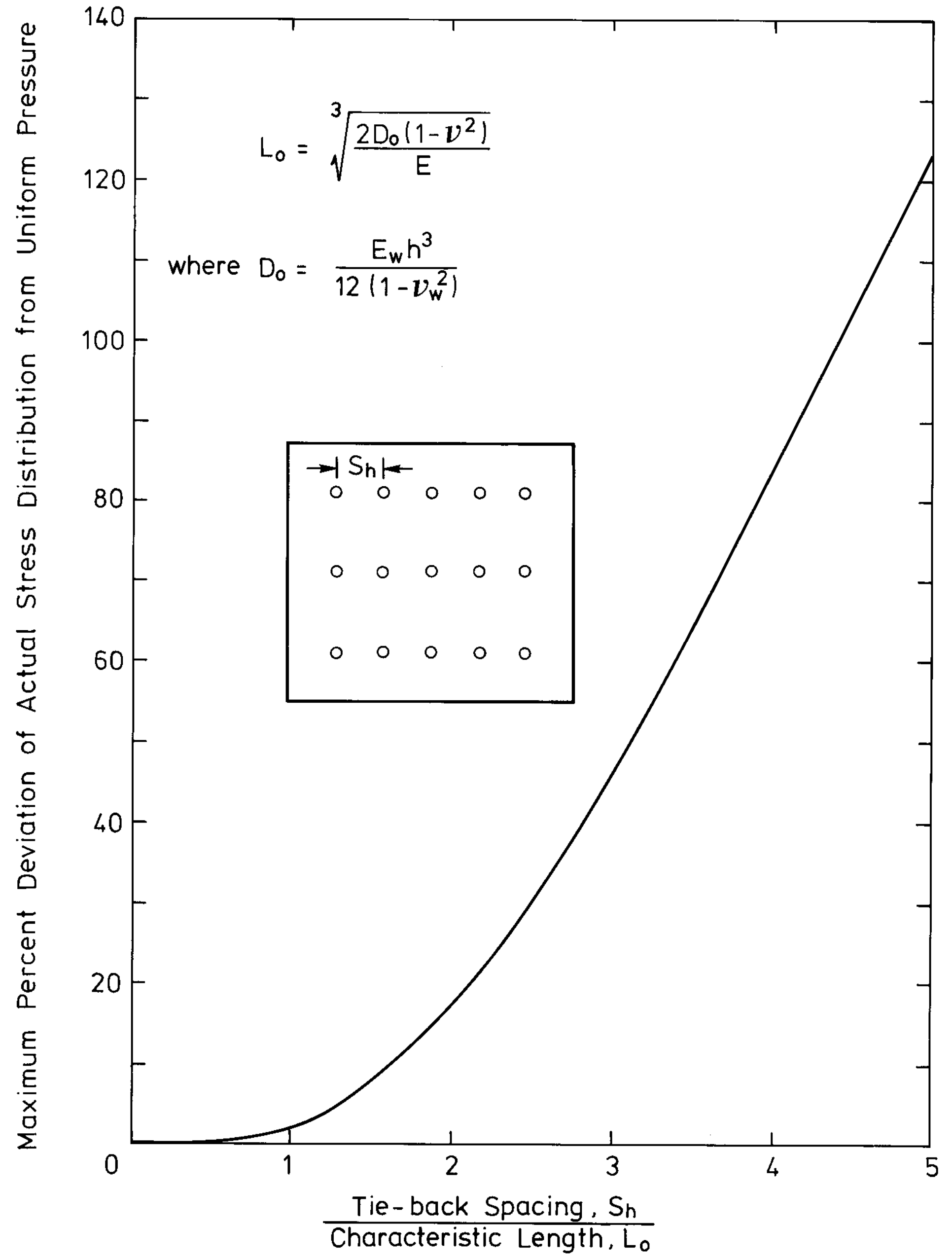


Figure 12 - Nonlinear Relationship for Soil below Excavation Level of Flexible Retaining Structure (Haliburton, 1968)



Legend :

E	Modulus of deformation of soil
ν	Poisson's ratio of soil
E_w	Modulus of deformation of retaining wall
ν_w	Poisson's ratio of retaining wall
h	Wall thickness

Figure 13 - Maximum Percent Deviation of Actual Stress Distribution from Uniform Pressure Distribution of Plane Strain Assumption (Tsui & Clough, 1974)

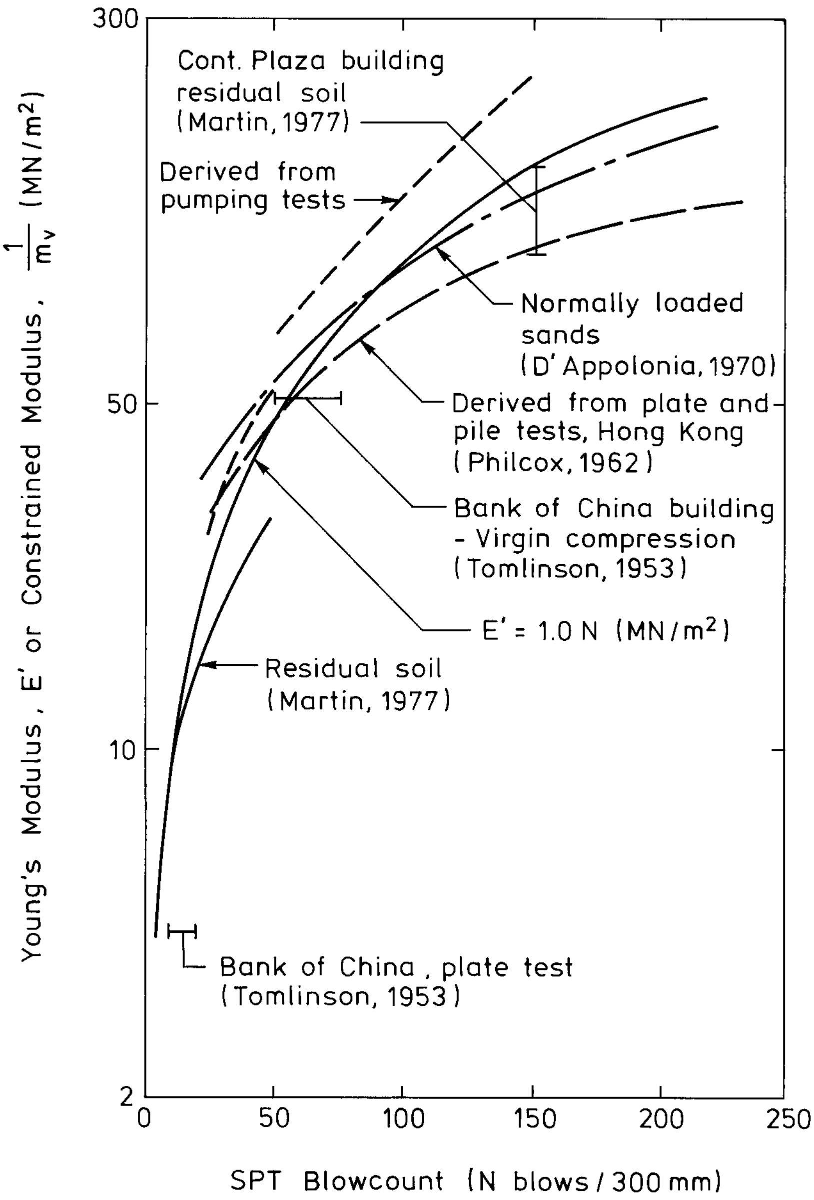


Figure 14 - Correlation between Young's Modulus and SPT 'N' Value (Chan & Davies, 1984)

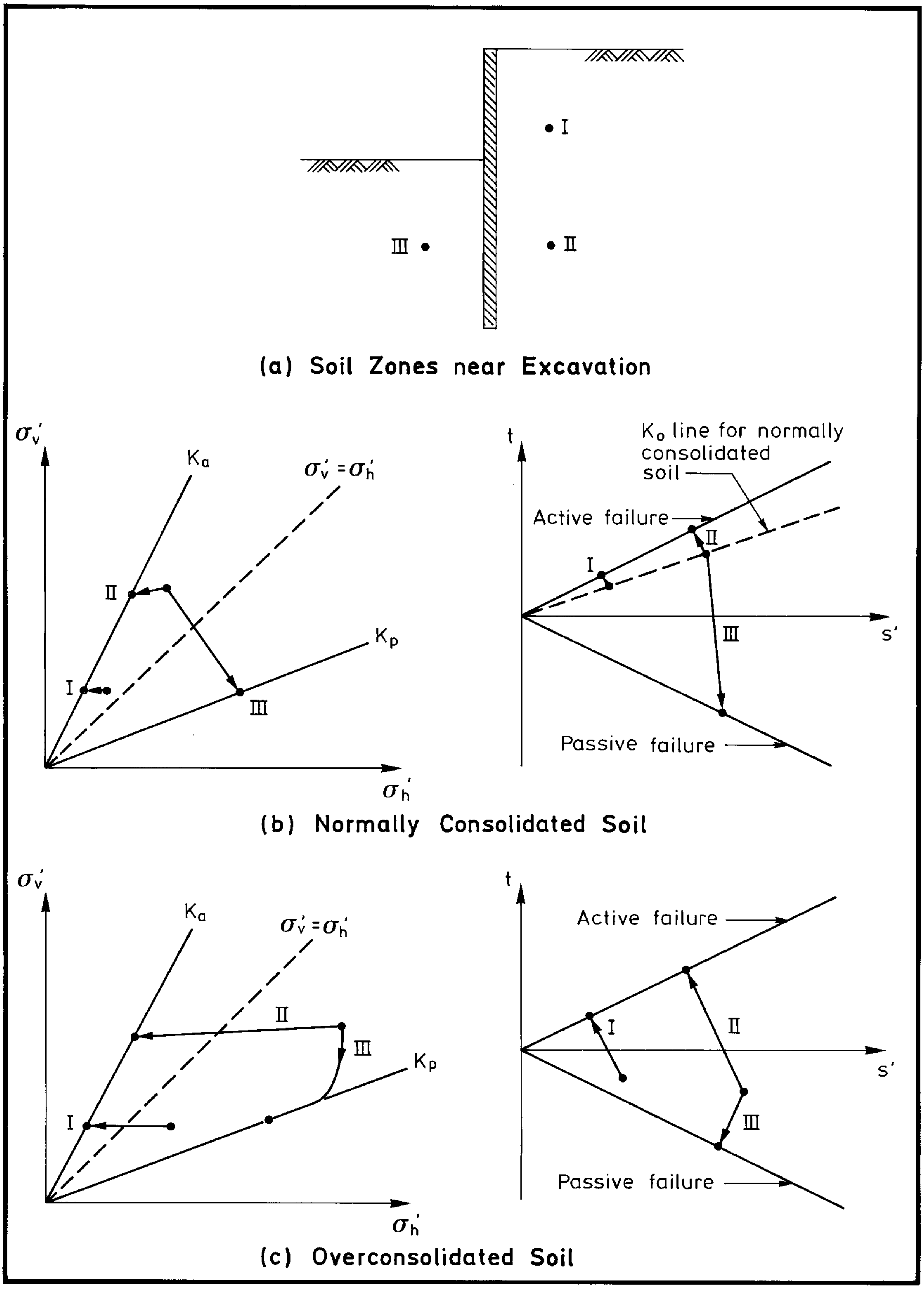


Figure 15 - Approximate Stress Paths Followed by Normally Consolidated and Overconsolidated Soil Elements Behind, and in Front of, a Retaining Wall (Padfield & Mair, 1984)

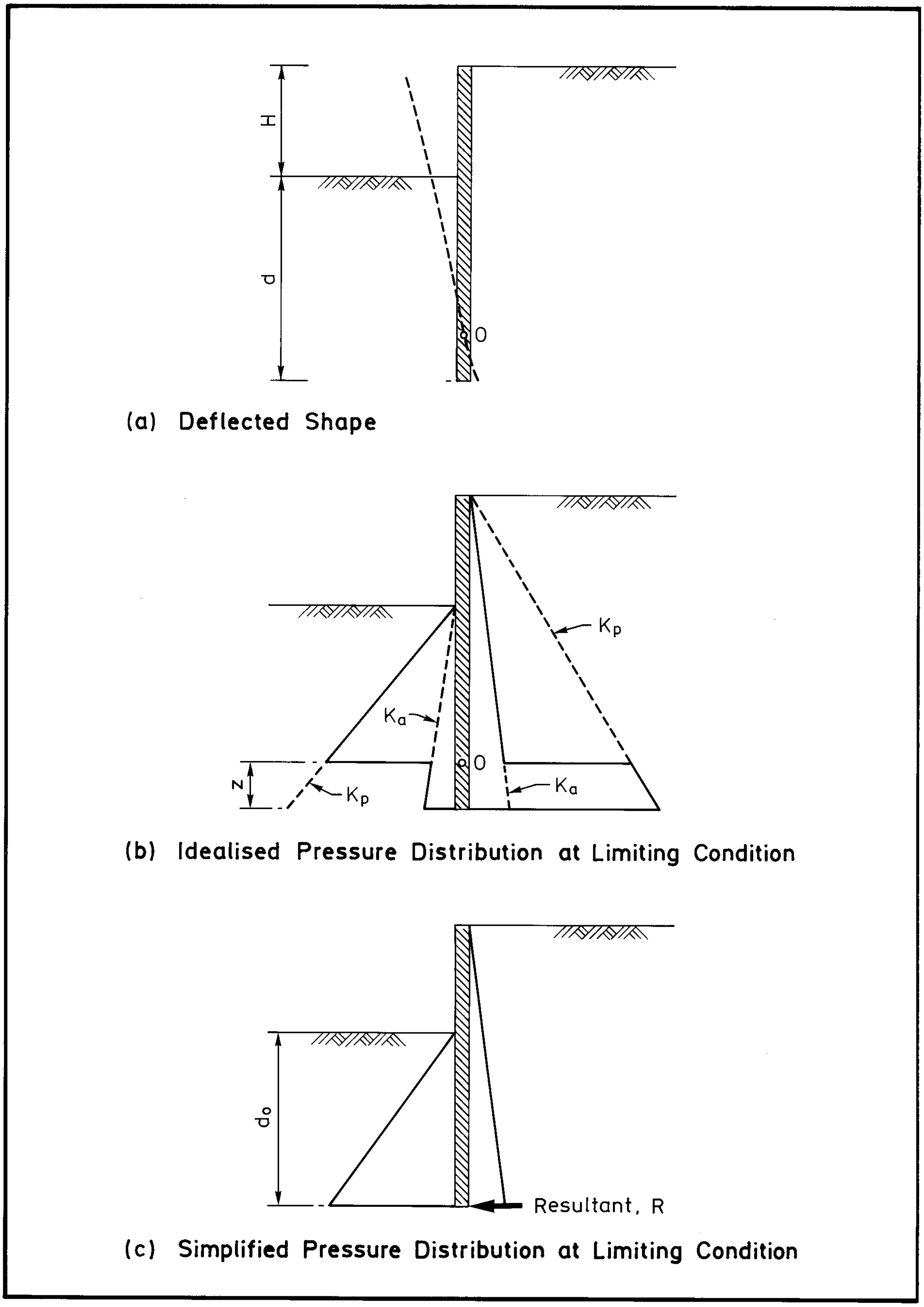


Figure 16 - Fixed-earth Support Conditions for a Cantilevered Wall (Padfield & Mair, 1984)

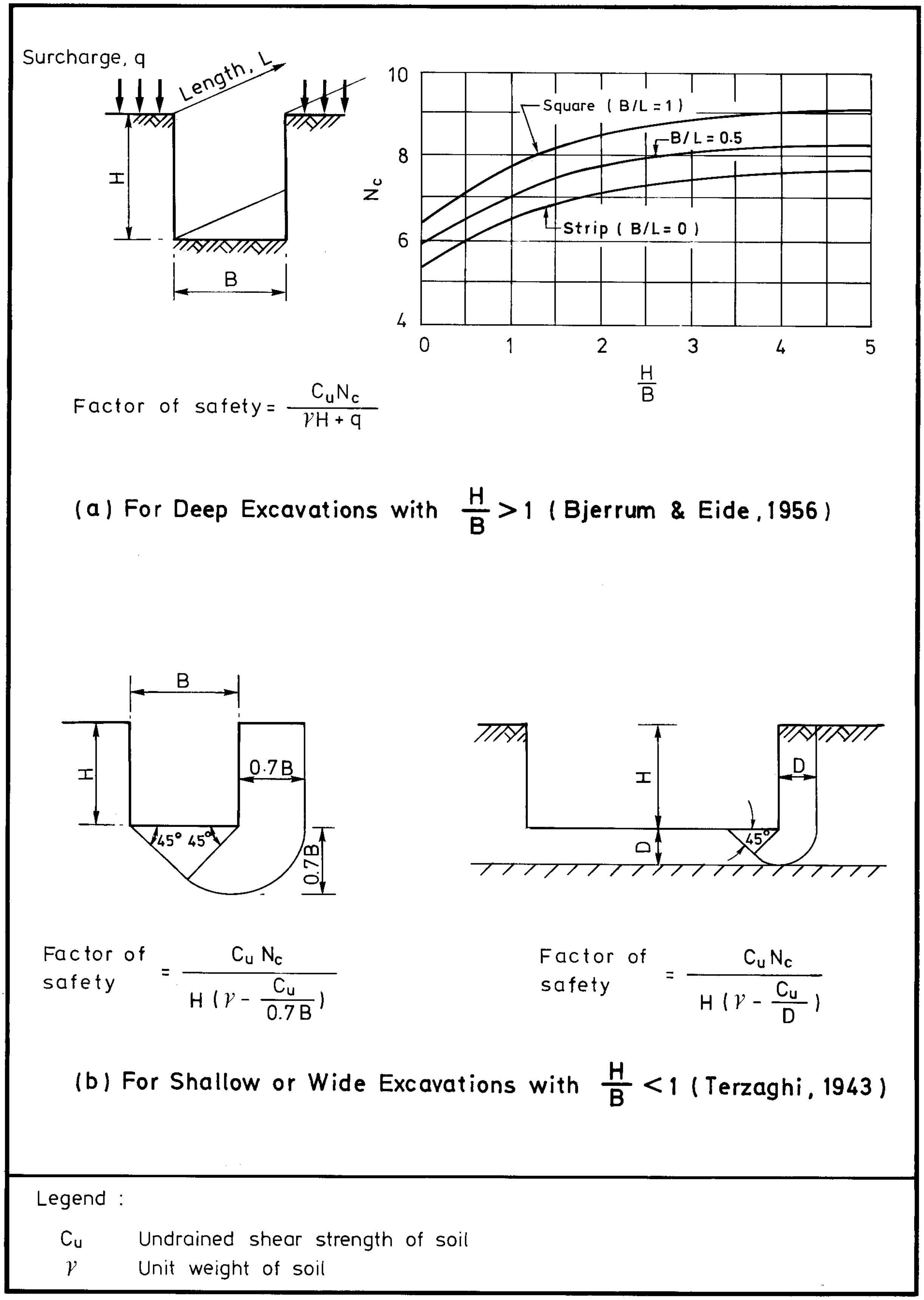


Figure 17 - Methods of Basal Heave Analysis in Cohesive Soils (Clough et al, 1979)

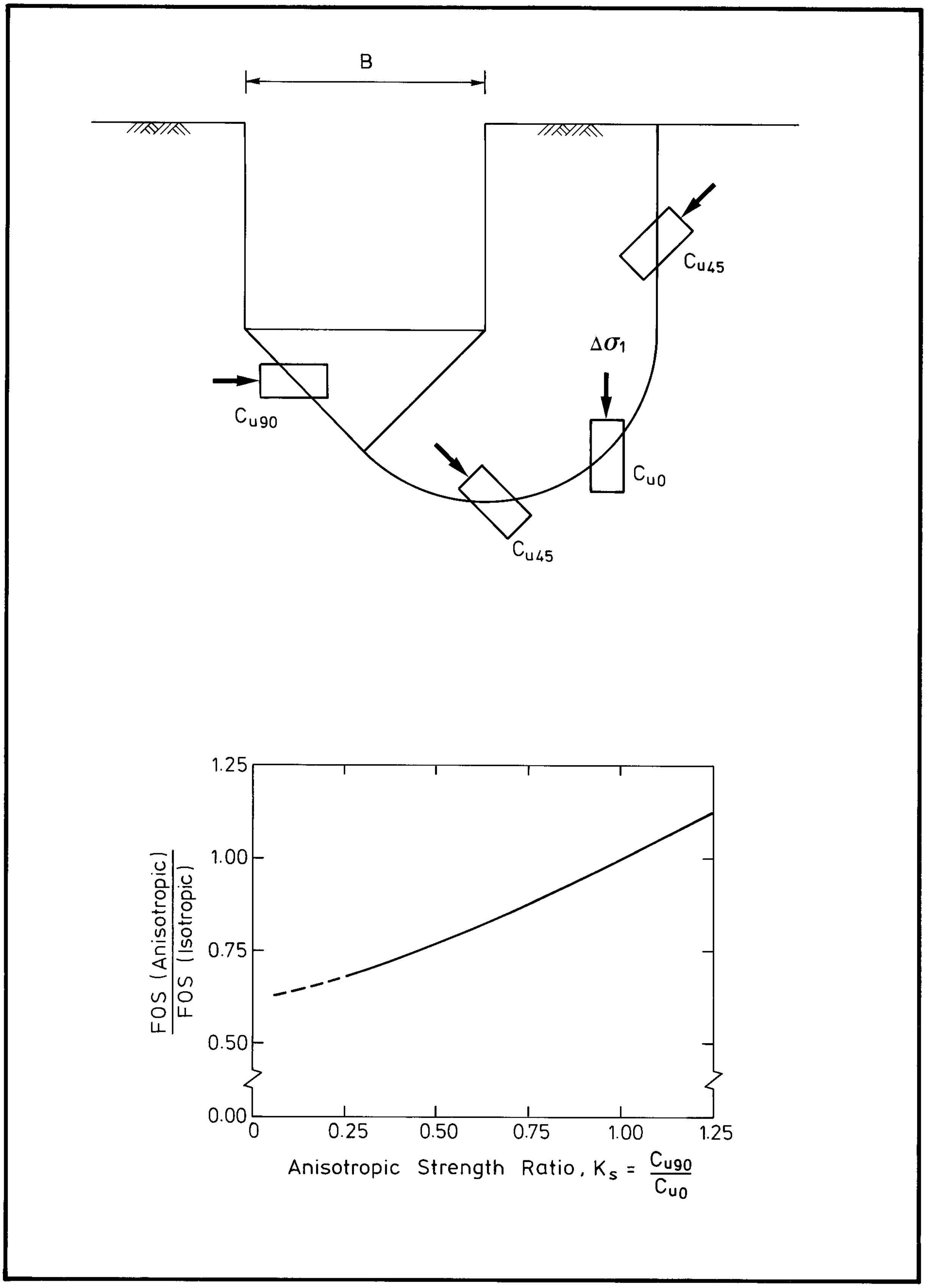
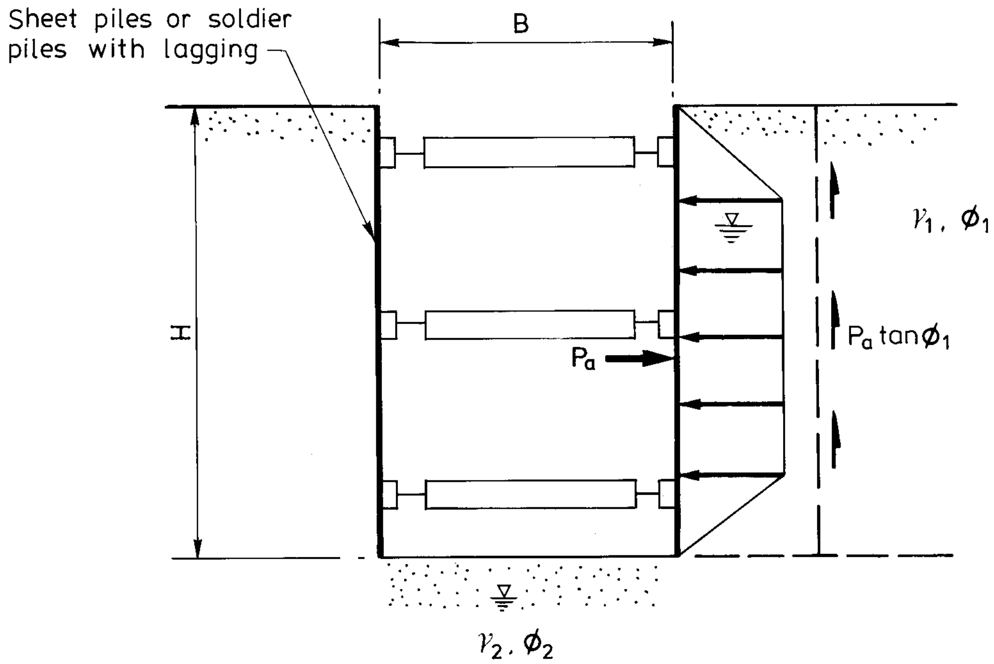


Figure 18 - Ratio of Factors of Safety against Basal Heave for Anisotropic Soil to that for Isotropic Soil for Wide Excavation (B > 15 m) (after Clough & Hansen, 1981)



Note : Stability is independent of H and B, but varies with γ, ϕ and seepage condition.

Safety factor, $F_s = 2N_{\gamma_2}\left(\frac{\gamma_2}{\gamma_1}\right)K_a \tan\phi_1$,

where N_{γ_2} = bearing capacity factor.

If groundwater is at a depth of B or more below base of cut :
γ_1 and γ_2 are taken as moist unit weights.

If groundwater is static at base of cut :
γ_1 = moist weight, γ_2 = submerged weight.

If seepage is moving upward to base of cut :
γ_2 = (saturated unit weight) - (uplift pressure).

Figure 19 - Basal Heave Analysis in Cohesionless Soils (NAVFAC, 1982b)

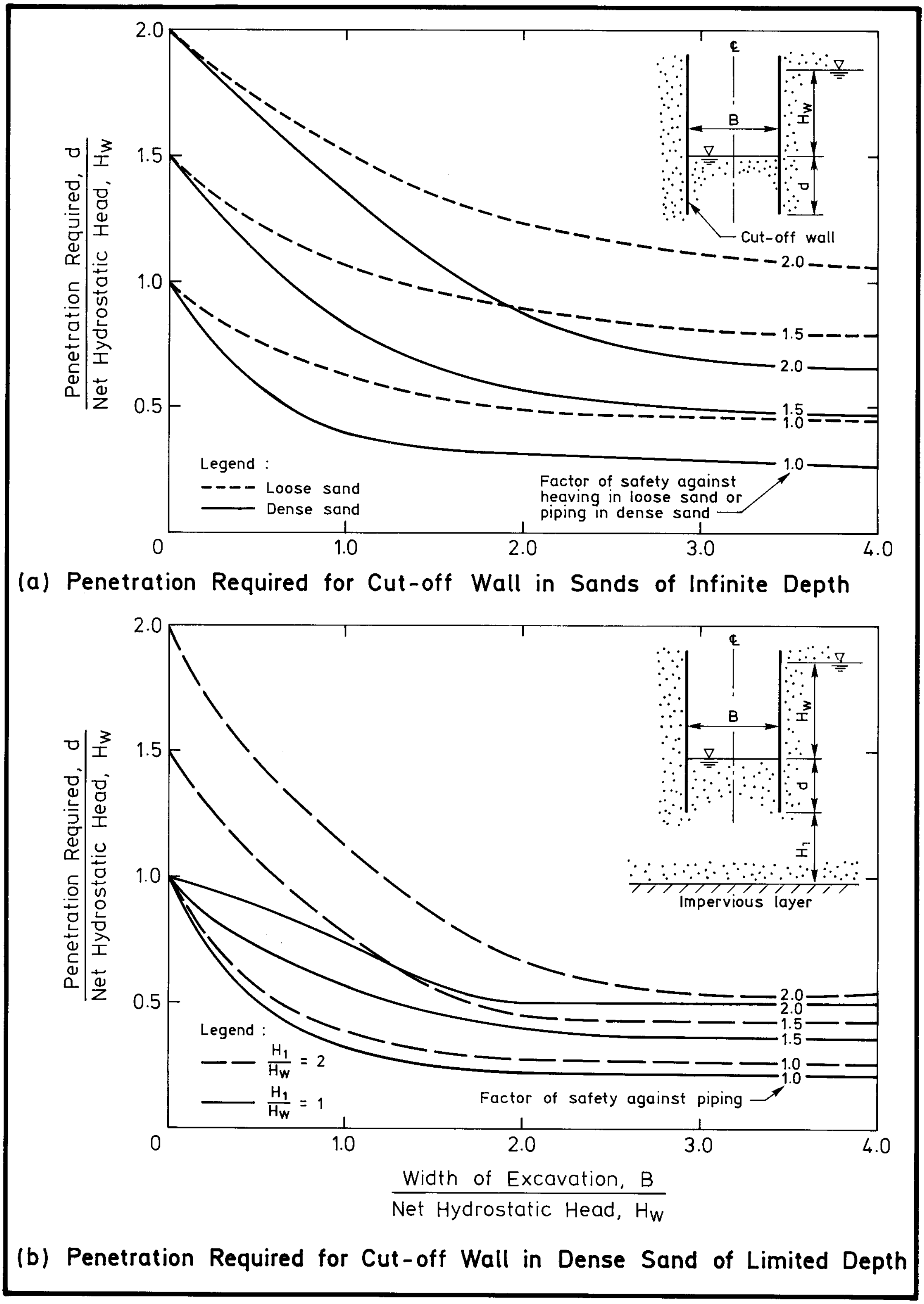
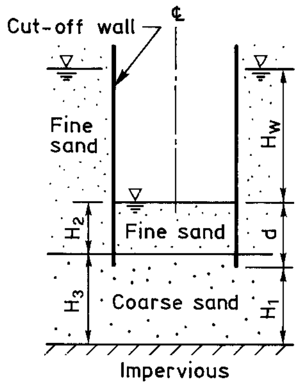


Figure 20 - Penetration of Cut-off Wall to Prevent Hydraulic Failure in Homogeneous Sand (after NAVFAC, 1982a)

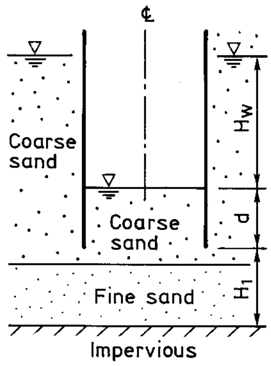


(a) Coarse Sand Underlying Fine Sand

Presence of coarse layer makes flow in the fine material more nearly vertical and generally increases seepage gradients in the fine material compared to the homogeneous cross-sections of Figure 20.

If top of coarse layer is below toe of cut-off wall at a depth greater than width of excavation, safety factors of Figure 20(a) for infinite depth apply.

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

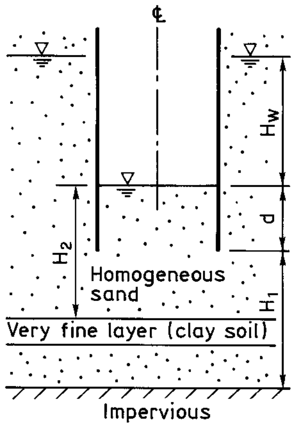


(b) Fine Sand Underlying Coarse Sand

Presence of fine layer constricts flow beneath cut off wall and generally decreases seepage gradients in the coarse layer.

If top of fine layer lies below toe of cut-off wall, safety factors are intermediate between those derived from Figure 20 for the case of an impermeable boundary at (i) the top of fine layer, and (ii) the bottom of the fine layer assuming coarse sand above the impermeable boundary throughout.

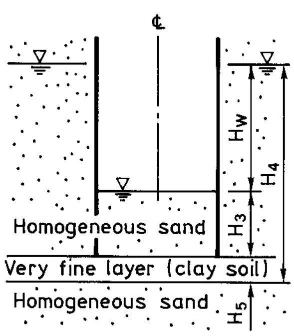
If top of fine layer lies above toe of cut-off wall, safety factors of Figure 20 are somewhat conservative for penetration required.



(c) Very Fine Layer in Homogeneous Sand

If top of very fine layer is below toe of cut-off wall at a depth greater than width of excavation, safety factors of Figure 20 assuming impermeable boundary at top of fine layer apply.

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



To avoid bottom heave when toe of cut-off wall is in or through the very fine layer, ($\gamma_S H_3 + \gamma_C H_5$) should be greater than $\gamma_W H_4$.

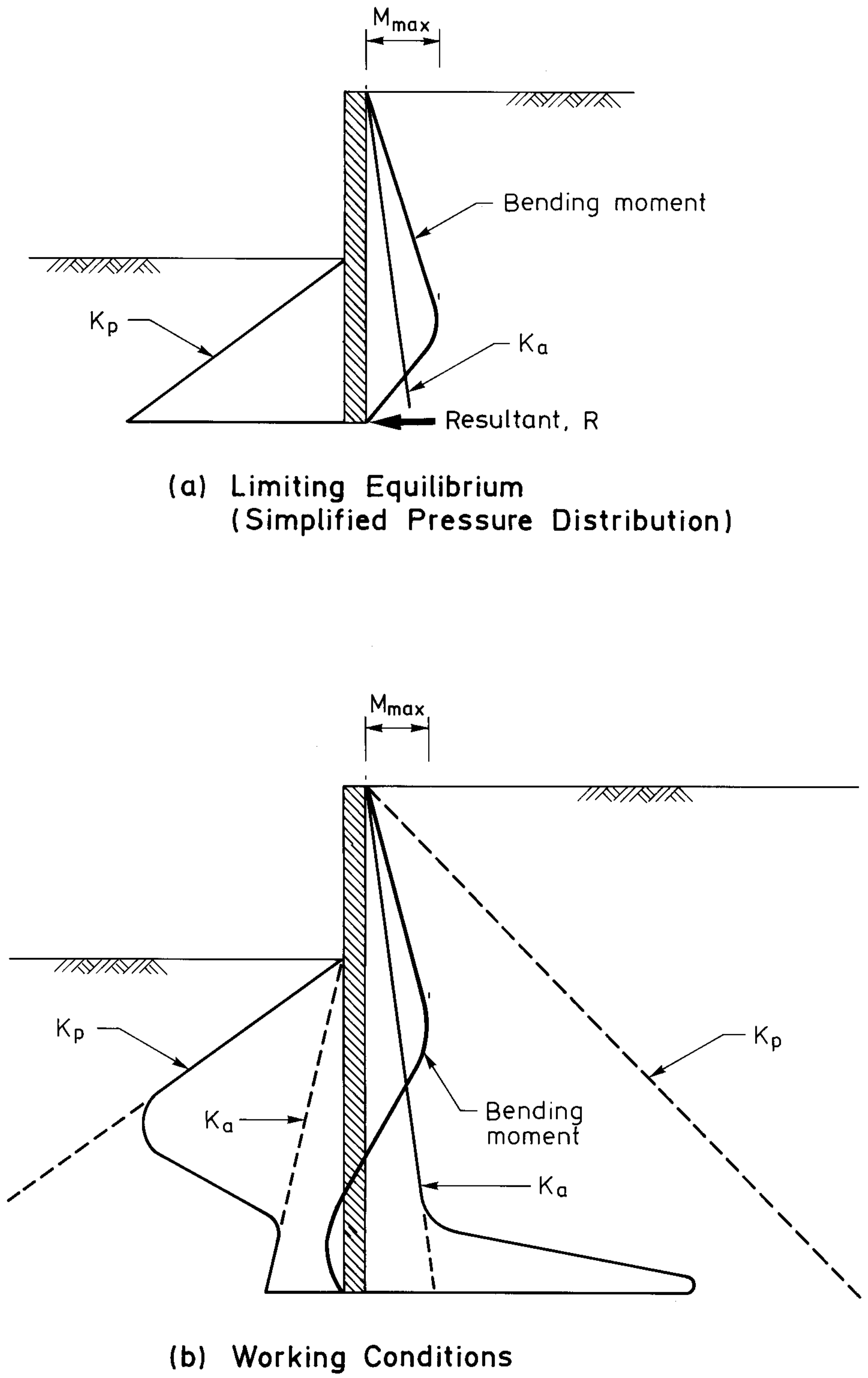
γ_S = saturated unit weight of the sand

γ_C = saturated unit weight of the clay

γ_W = unit weight of water

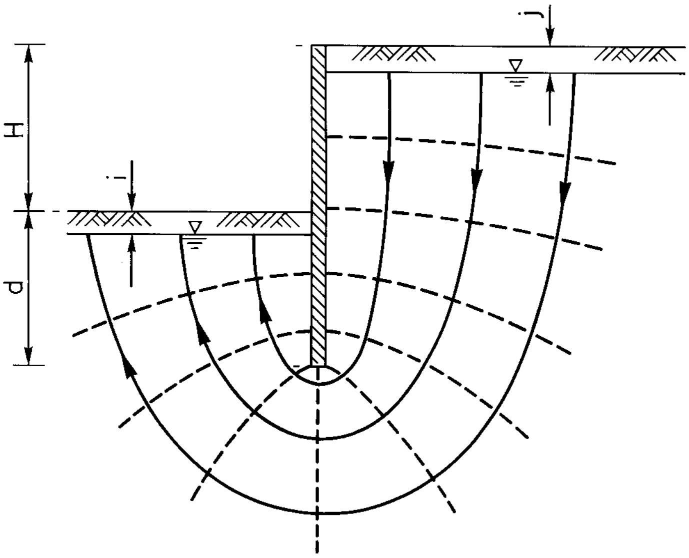
If fine layer lies above subgrade of excavation, final condition is safer than homogeneous case, but dangerous condition may arise during excavation above fine layer and pressure relief is required as in the preceding case.

Figure 21 - Penetration of Cut-off Wall to Prevent Hydraulic Failure in Stratified Soil (after NAVFAC, 1982a)

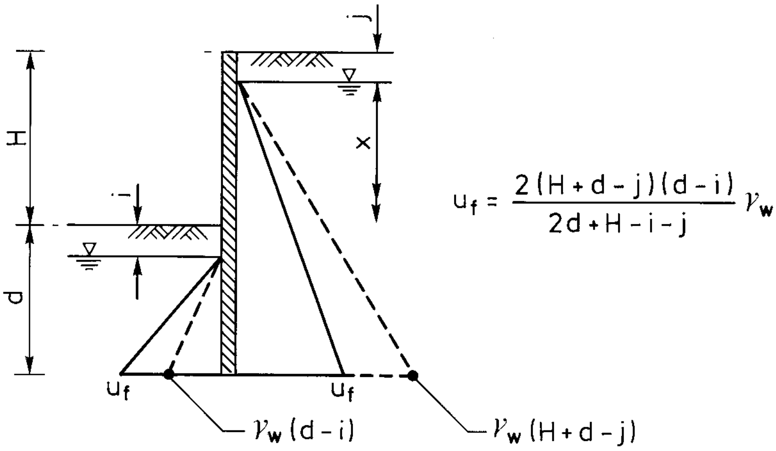


Note : Maximum bending moment, M_{max}, is largely unaffected by depth of embedment.

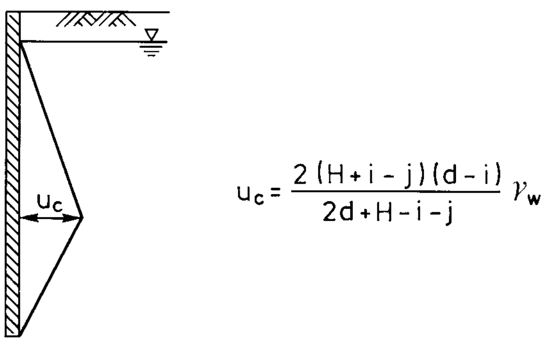
Figure 22 - Calculation of Maximum Bending Moment in a Cantilevered Wall (Padfield & Mair, 1984)



(a) Seepage Flow-net around a Retaining Wall in Homogeneous Soil



(b) Gross Water Pressure across a Wall with Seepage



(c) Net Water Pressure across a Wall with Seepage

Figure 23 - Pore Water Pressure Distribution across a Retaining Wall under Steady-state Seepage Condition (Padfield & Mair, 1984)

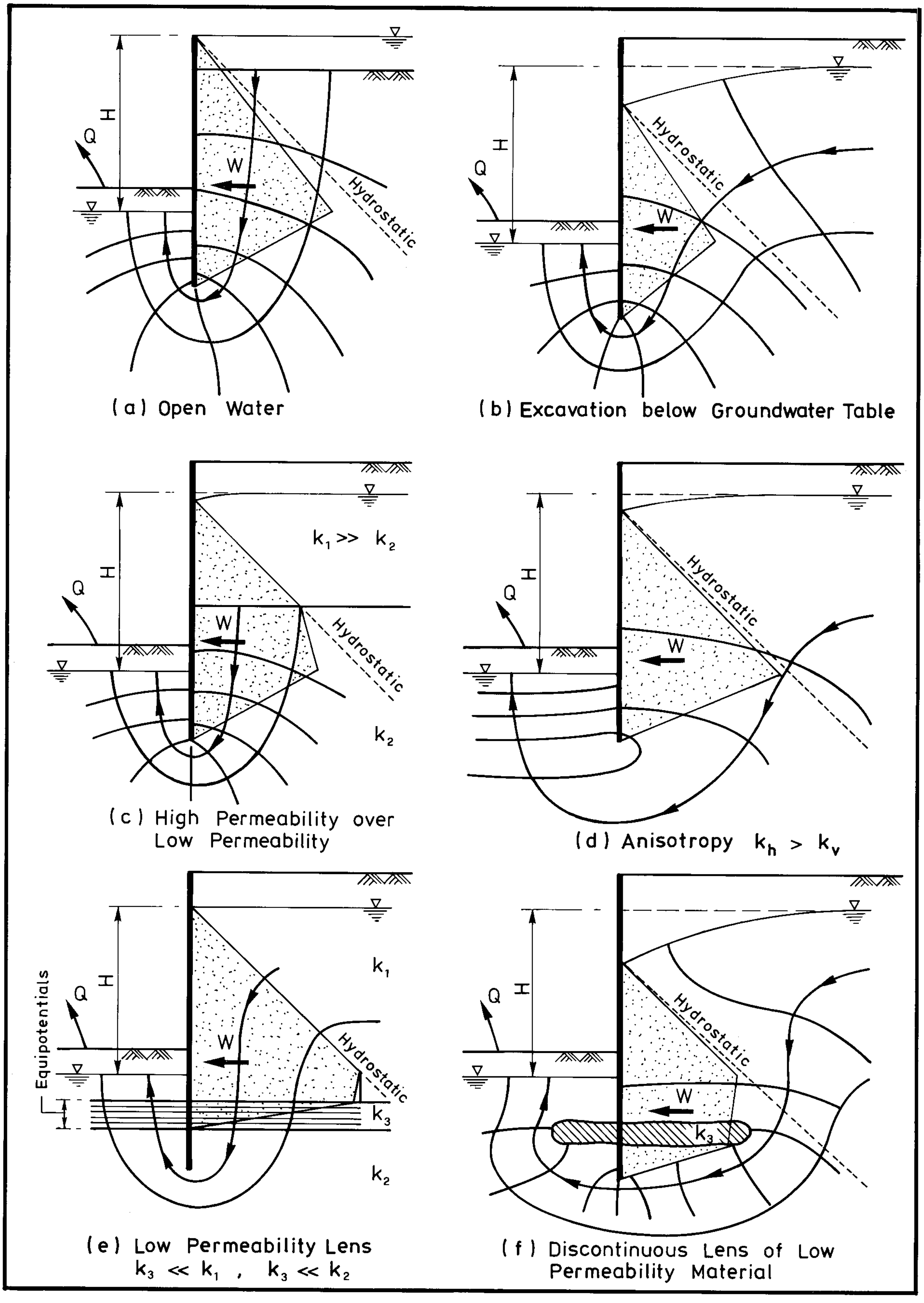


Figure 24 - Groundwater Flow Patterns and Resultant Water Pressures behind Excavations (Kaiser & Hewitt, 1982)

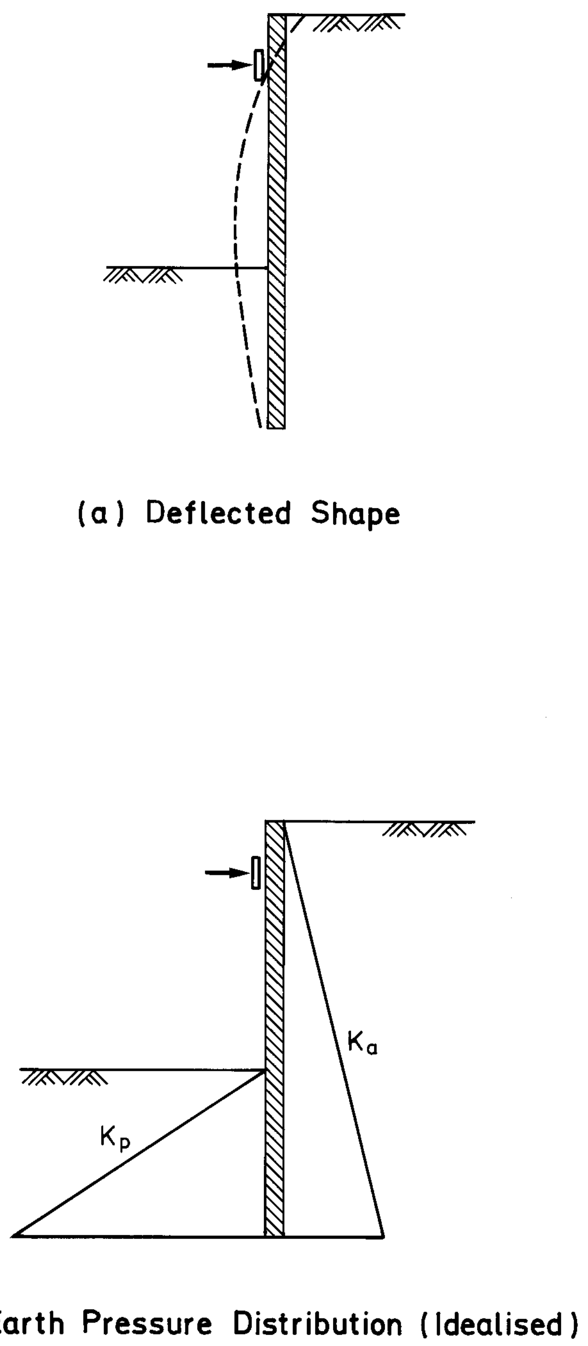
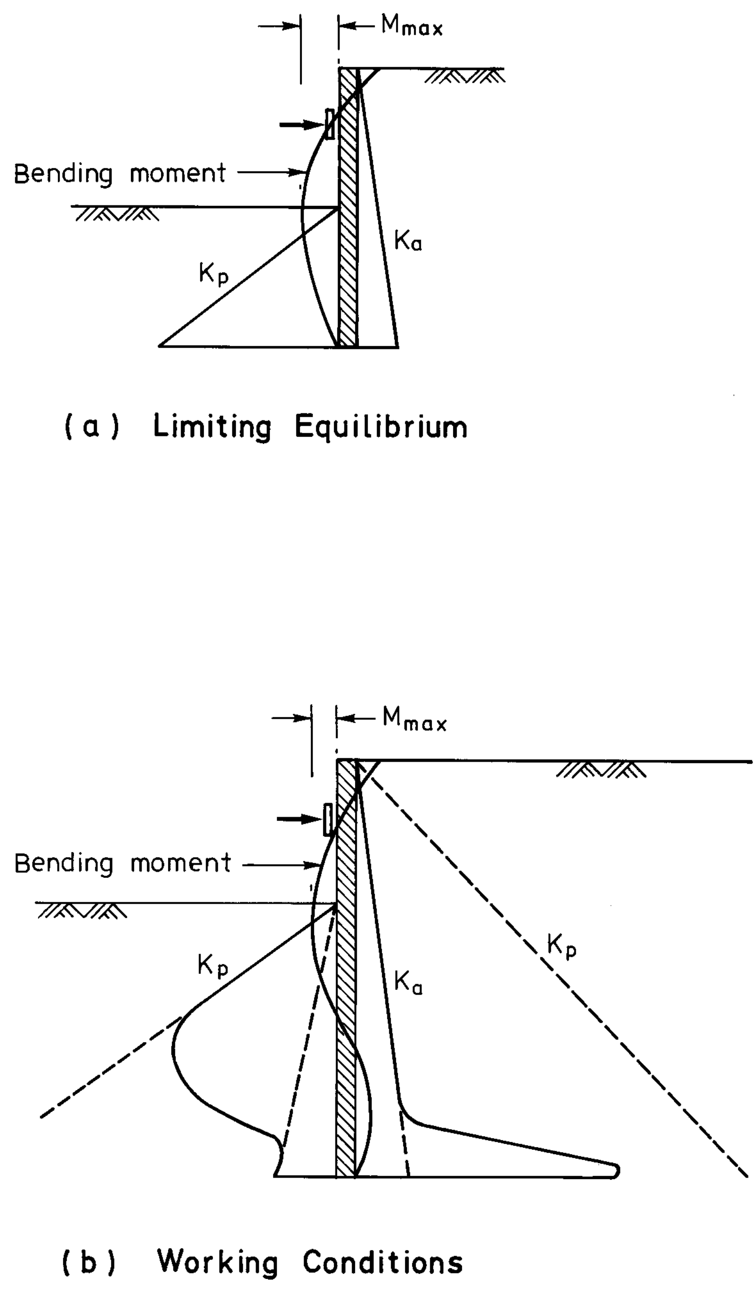
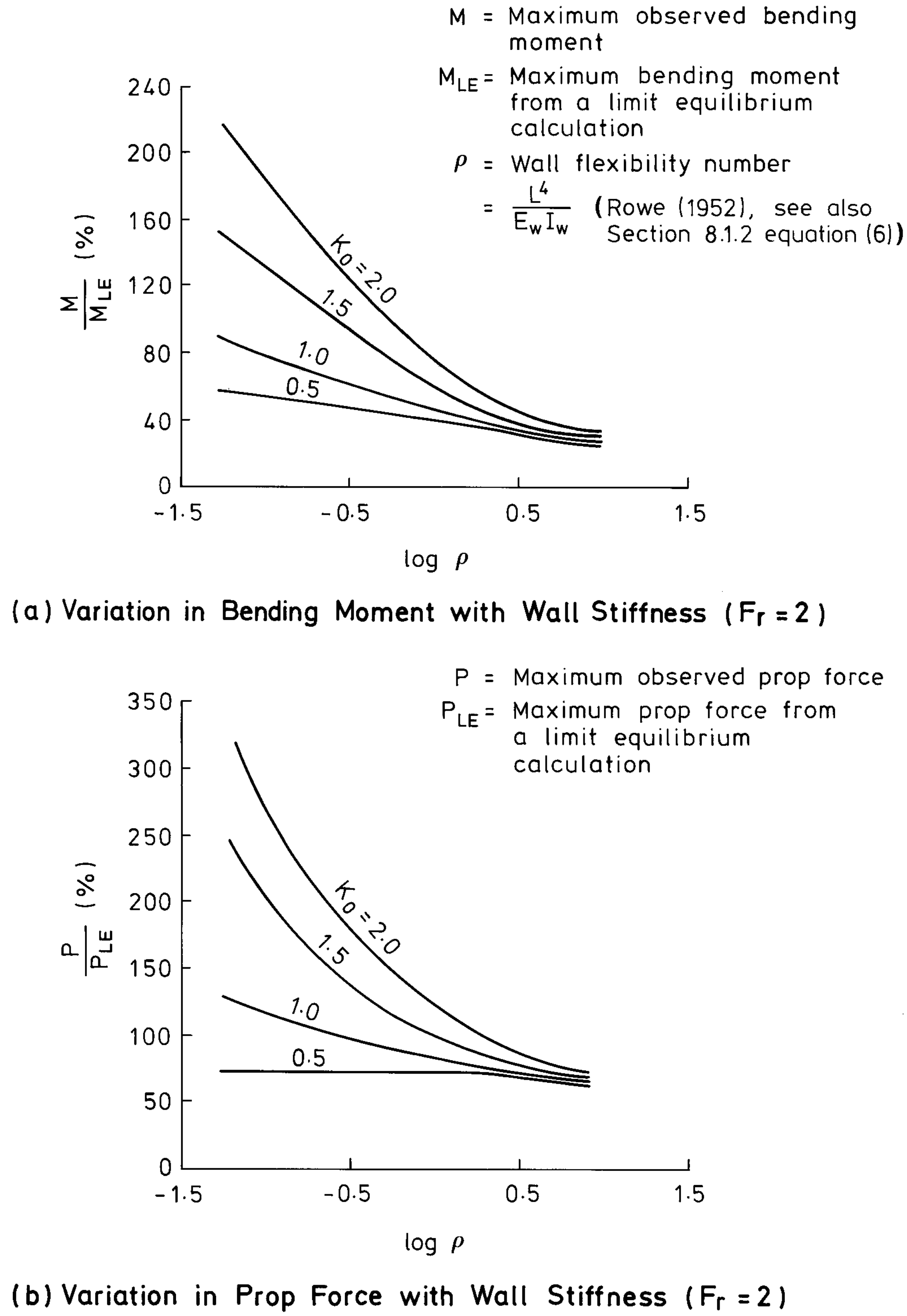


Figure 25 - Free-earth Support Conditions for a Propped Wall (Padfield & Mair, 1984)



Note : Maximum bending moment, M_{max}, is influenced by depth of embedment.

Figure 26 - Calculation of Maximum Bending Moment in a Propped Wall (Padfield & Mair, 1984)



Notes : (1) For definition of F_r, refer to Table 4

(2) ρ is in units of $\frac{m^3}{kN}$

Figure 27 - Effect of Wall Stiffness on Bending Moment and Prop Force of a Propped Retaining Wall (Potts & Fourie, 1985)

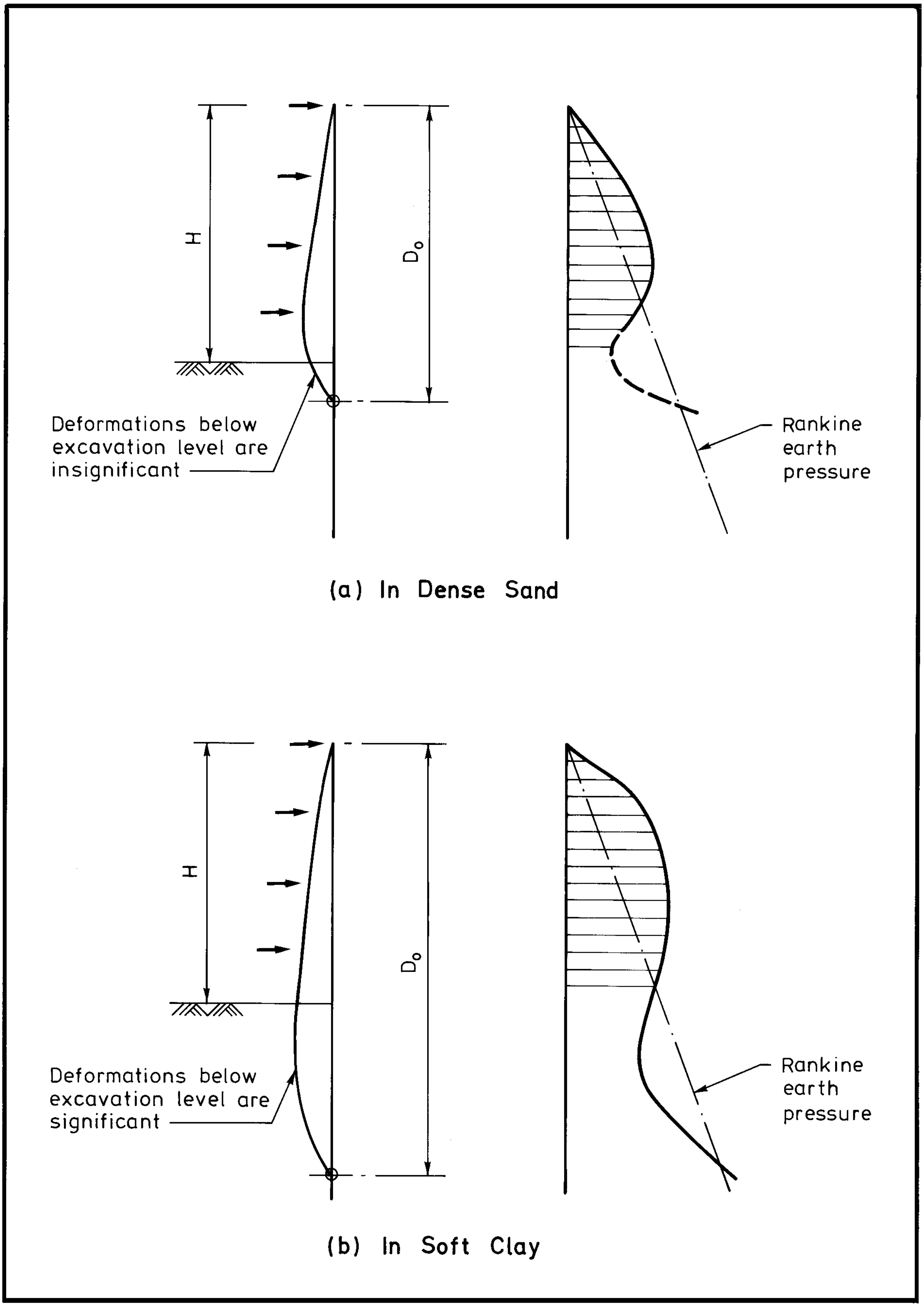
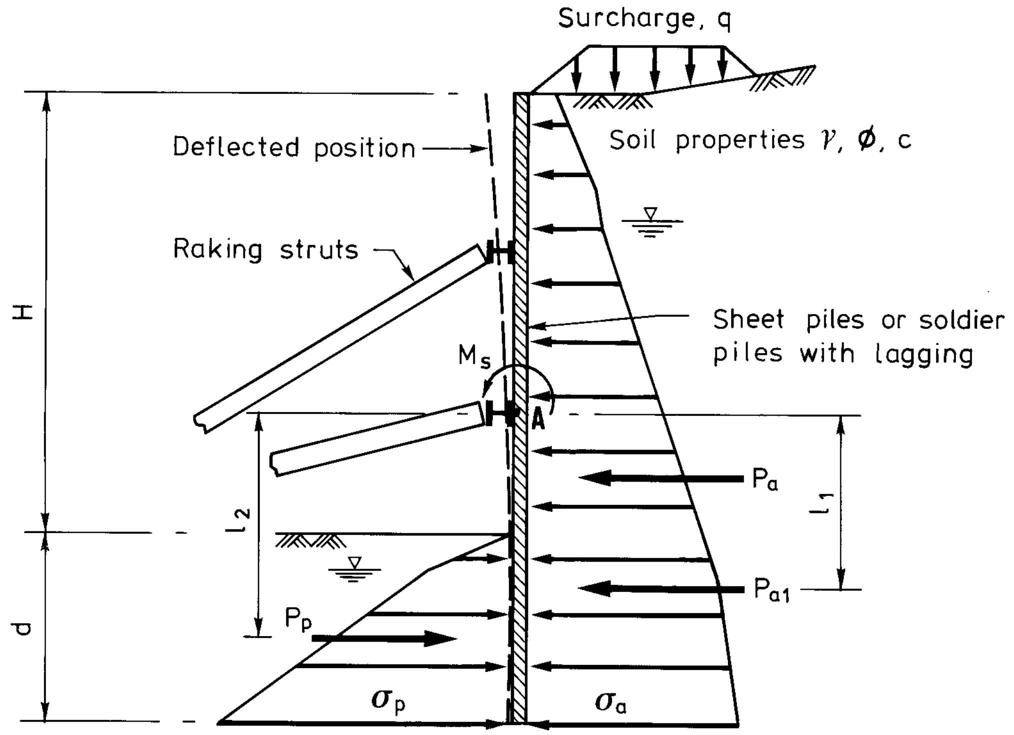


Figure 28 - Deflection of a Sheet Pile Wall and Re-distribution of Active Earth Pressure (Bjerrum et al, 1972)



Legend :

P_a	Resultant active pressure (including water pressure)
P_p	Resultant passive pressure (including water pressure)
P_{a1}	Resultant active pressure below point **A**
M_s	Allowable moment in sheet pile or soldier pile

Note : Required penetration of sheet pile or soldier pile is generally controlled by conditions at completion of excavation. Penetration required is determined from consideration of equilibrium of the free-ended span below point **A**, assuming fixity at **A** :

$$P_{a1}\, l_1 - \frac{P_p}{F_s}\, l_2 - M_s = 0$$

where F_s is the safety factor adopted in design.

Figure 29 - Calculation of Embedment Depth of Sheet Pile Wall below Excavation Level (NAVFAC, 1982b)

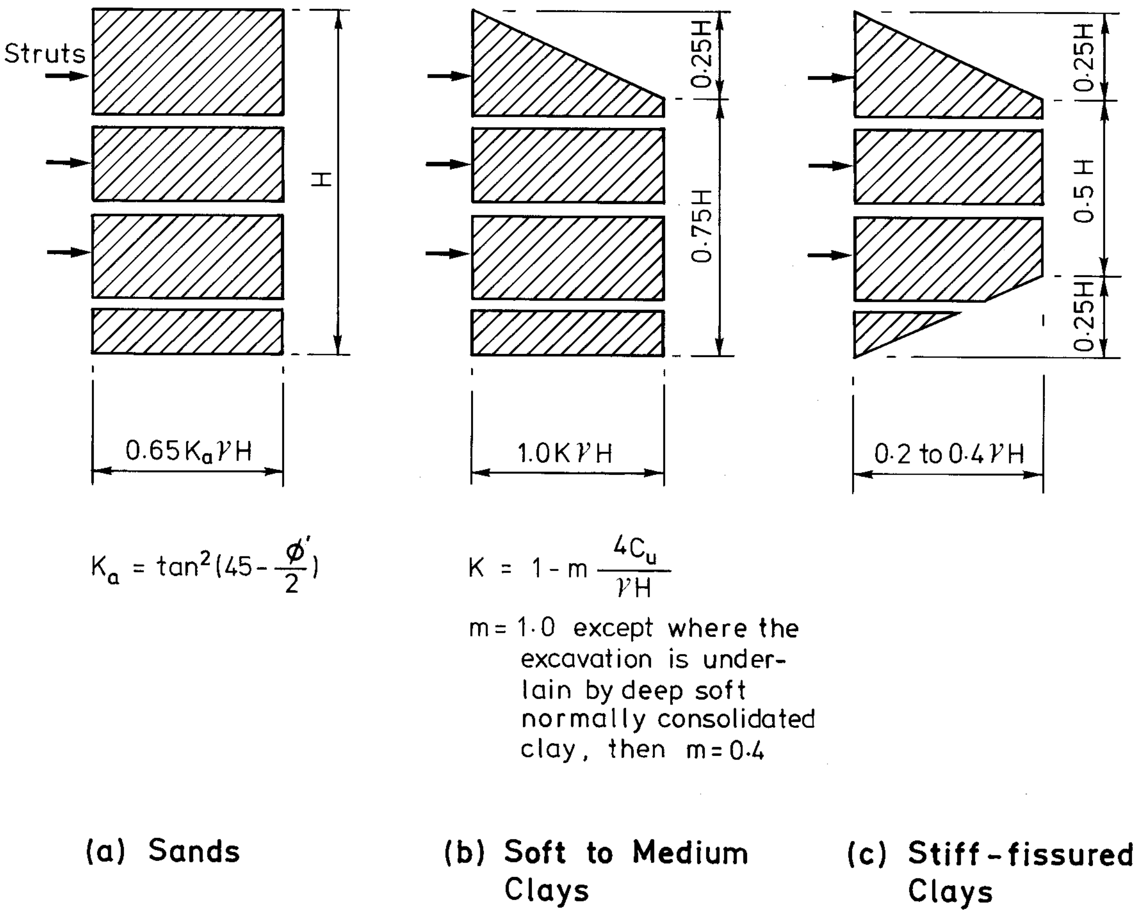


Figure 30 - Apparent Pressure Diagrams for Computing Strut Loads in Strutted Excavations (Peck, 1969b)

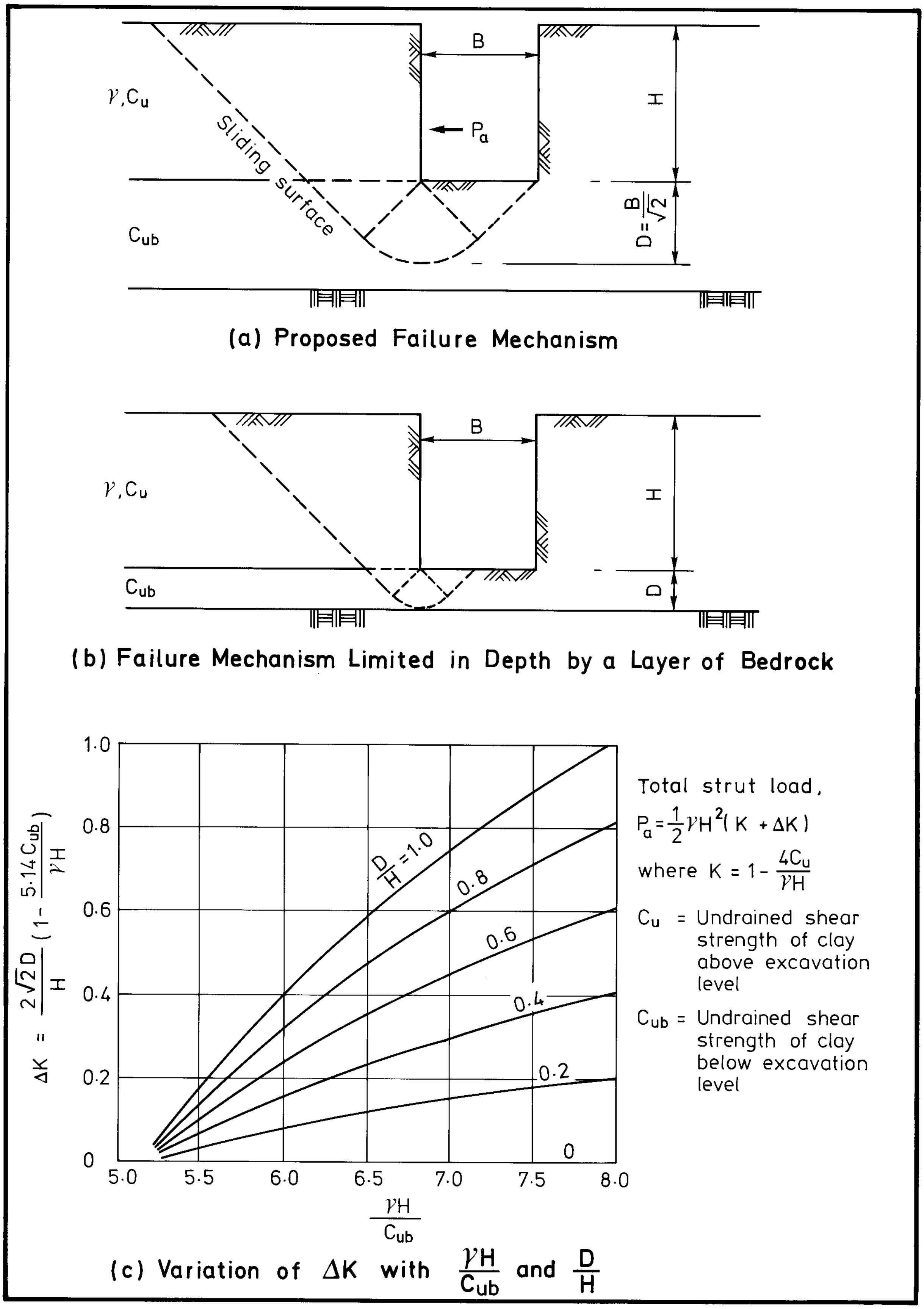
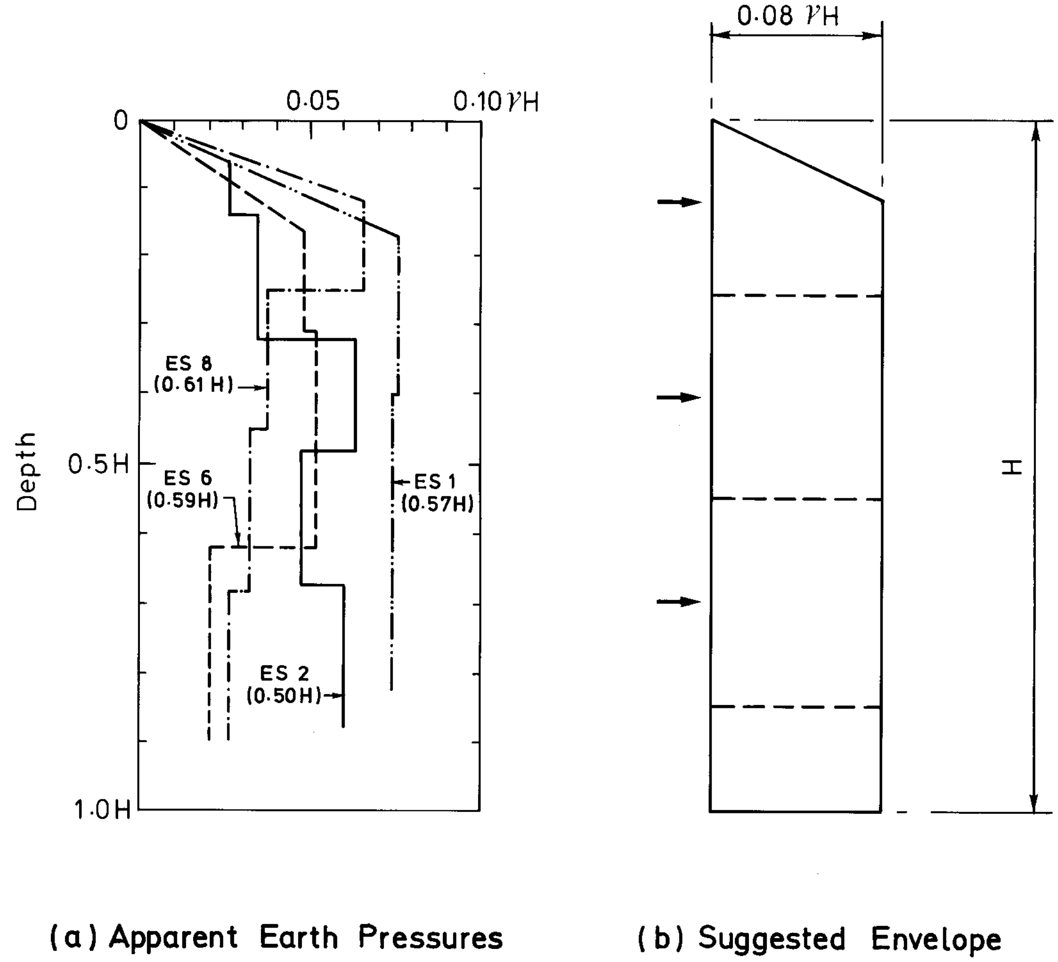


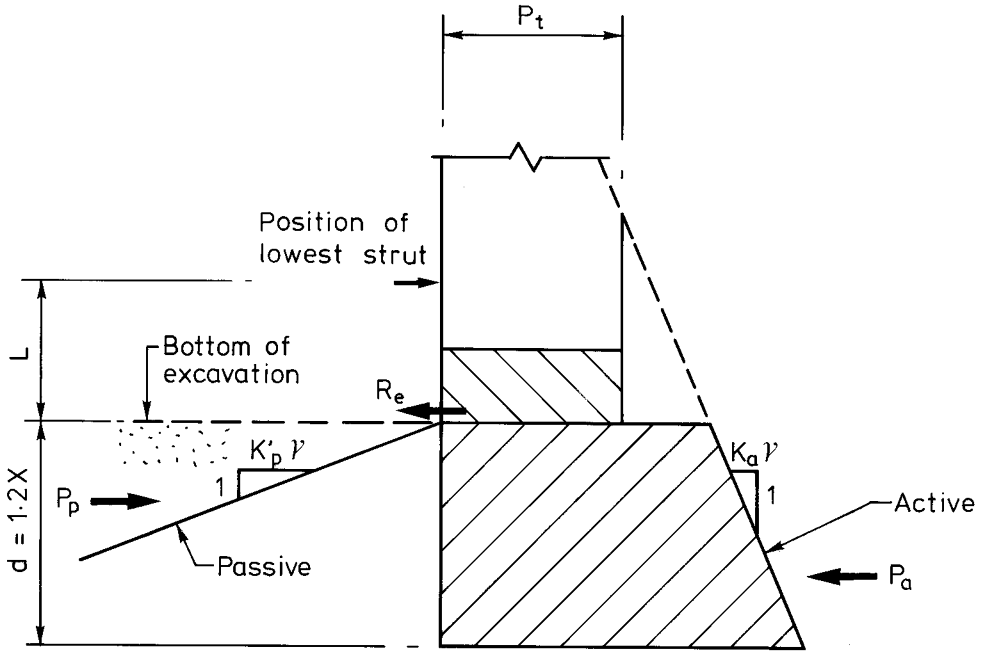
Figure 31 - Calculation of Strut Loads in Excavation in Soft Clay (Henkel, 1971)



Legend:

ES Experimental section
H Depth of excavation
( ) Distance of the centre of pressure from the bottom of excavation

Figure 32 - Apparent Earth Pressure Diagrams for Measured Strut Loads in Excavations in Lateritic and Weathered Sedimentary Soils (Massad et al, 1985)



Legend :

K_a	Active earth pressure coefficient
K_p	Passive earth pressure coefficient
P_t	From apparent earth pressure diagram (Figure 30)

Notes :
(1) Compute $R_e = 0{\cdot}5\,P_t L$.
(2) Compute depth X such that $P_p = R_e + P_a$, with minimum factor of safety of 1.5 for passive coefficient, i.e. $K'_p \leqslant \frac{K_p}{1.5}$.
(3) Check $M_{max} \leqslant$ yield moment of sheeting.
(4) Drive to depth $d = 1{\cdot}2\,X$.

Figure 33 - Calculation of Embedment Depth of Sheet Pile Wall in Relatively Uniform Competent Soil Conditions (Goldberg et al, 1976)

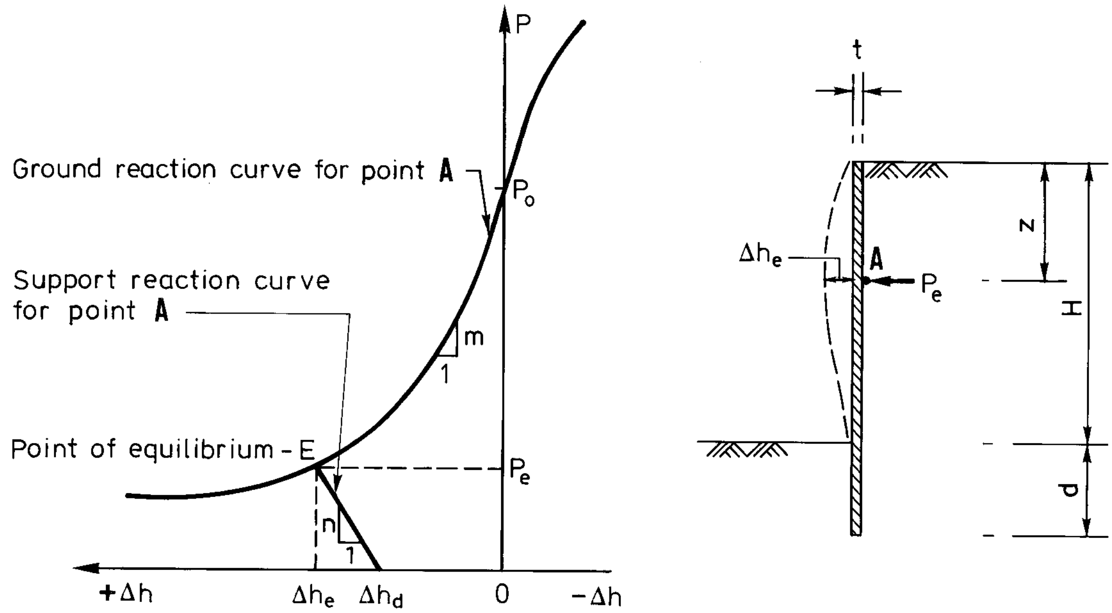


Figure 34 - Ground and Support Reaction Concept for a Strutted Excavation (Eisenstein & Negro, 1985)

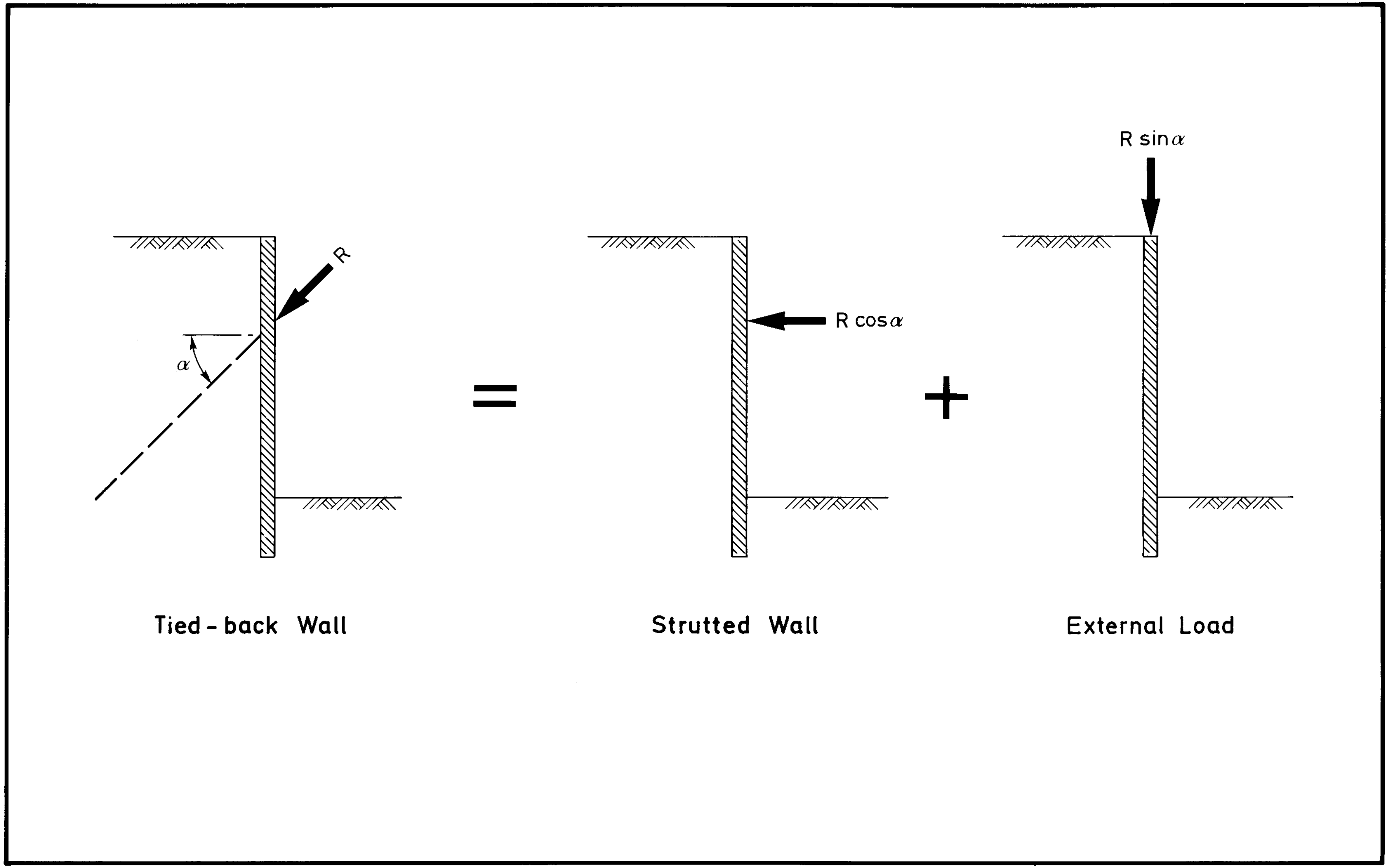


Figure 35 - Comparison between Tied-back and Strutted Walls (Broms & Stille, 1976)

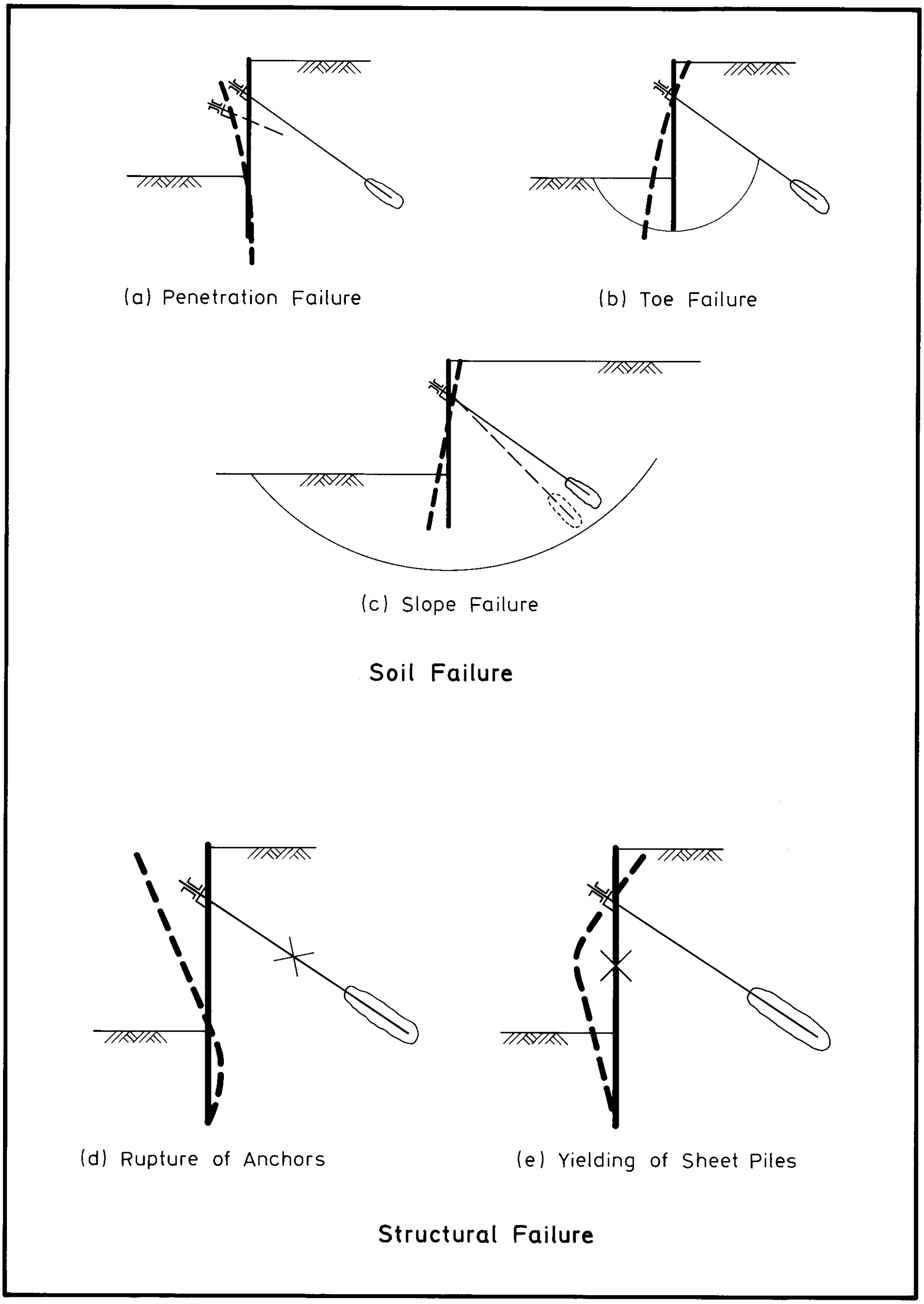


Figure 36 - Failure Mechanisms of Anchored Sheet Pile Walls (Broms & Stille, 1976)

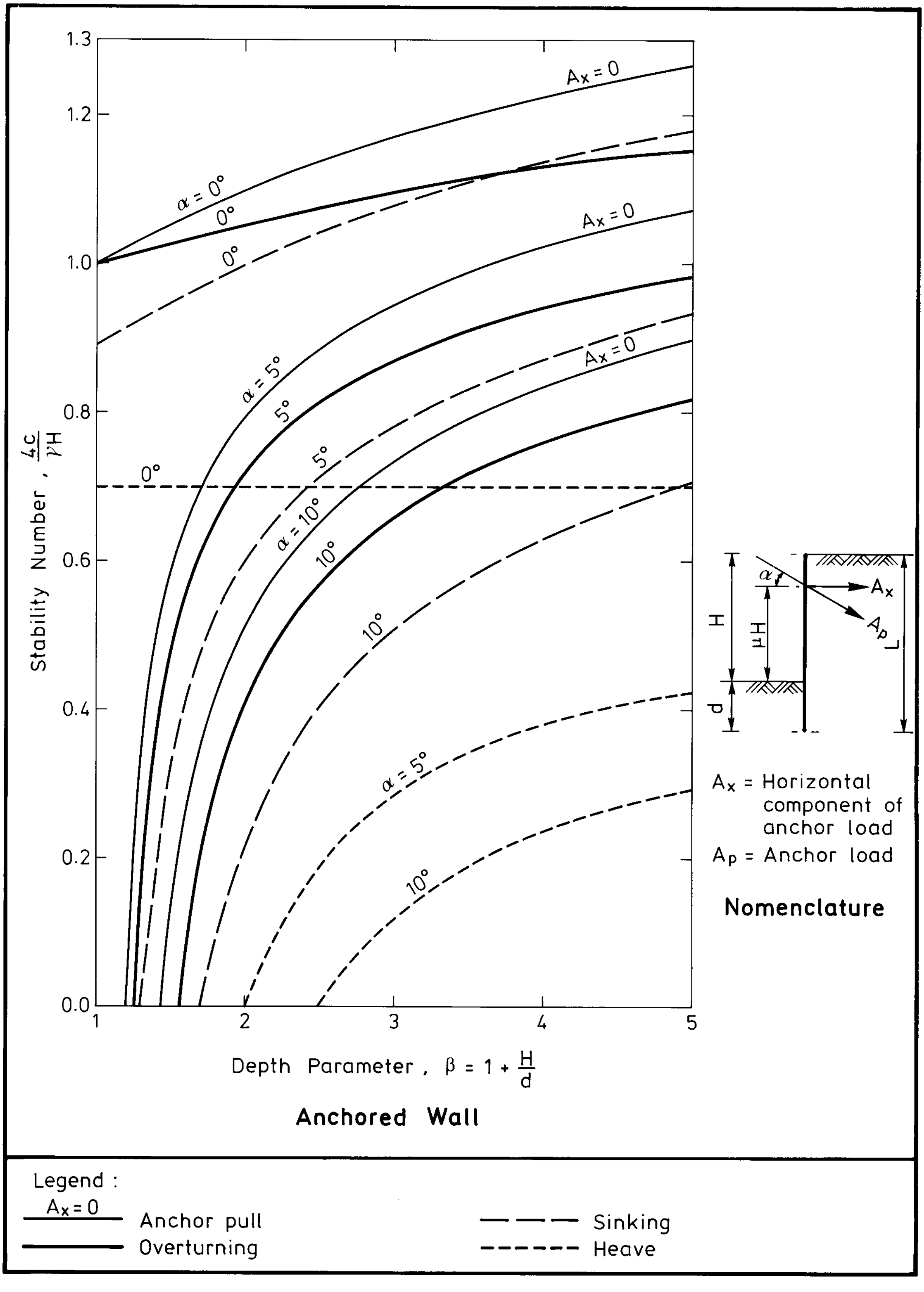
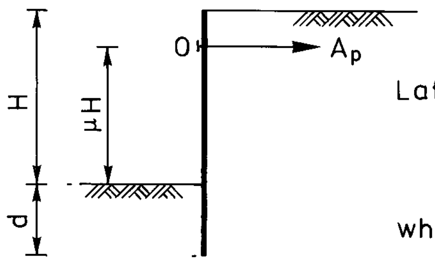


Figure 37 - Stability Number versus Depth Parameter for Three Conditions of Equilibrium : Overturning, Sinking and Soil Heaving (Browzin, 1981)



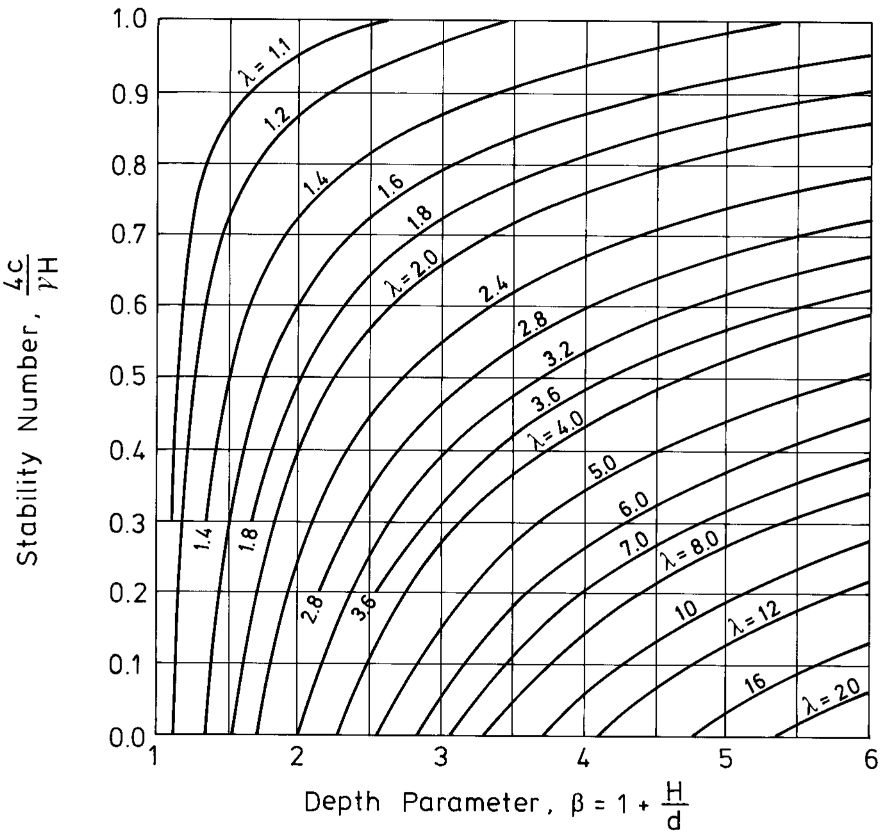
Nomenclature

Lateral earth pressure ratio,

$$\lambda = \frac{K_p^2}{F_s}$$

where K_p = Coefficient of passive earth pressure

F_s = Factor of safety on passive earth pressure



Procedure for using the chart,

Case 1 $\mu = 1$

Data : c, ϕ, γ, F_s, H

Calculate : K_p, λ, $\frac{4c}{\gamma H}$

From chart, obtain β then calculate d

Case 2 $\mu \neq 1$

Data : c, ϕ, γ, F_s, H, μ

Assume $\mu = 1$, calculate β (see Case 1)

From Figure 39, obtain k from λ & $\frac{4c}{\gamma H}$

Calculate $\beta_\mu = \beta \left(1 + \frac{k}{100}\right)$

then obtain d from $\beta_\mu = 1 + \frac{H}{d}$

Figure 38 - Design Chart for Calculating Embedment Depth of a Propped Wall in Cohesive Frictional Soils (Browzin, 1983)

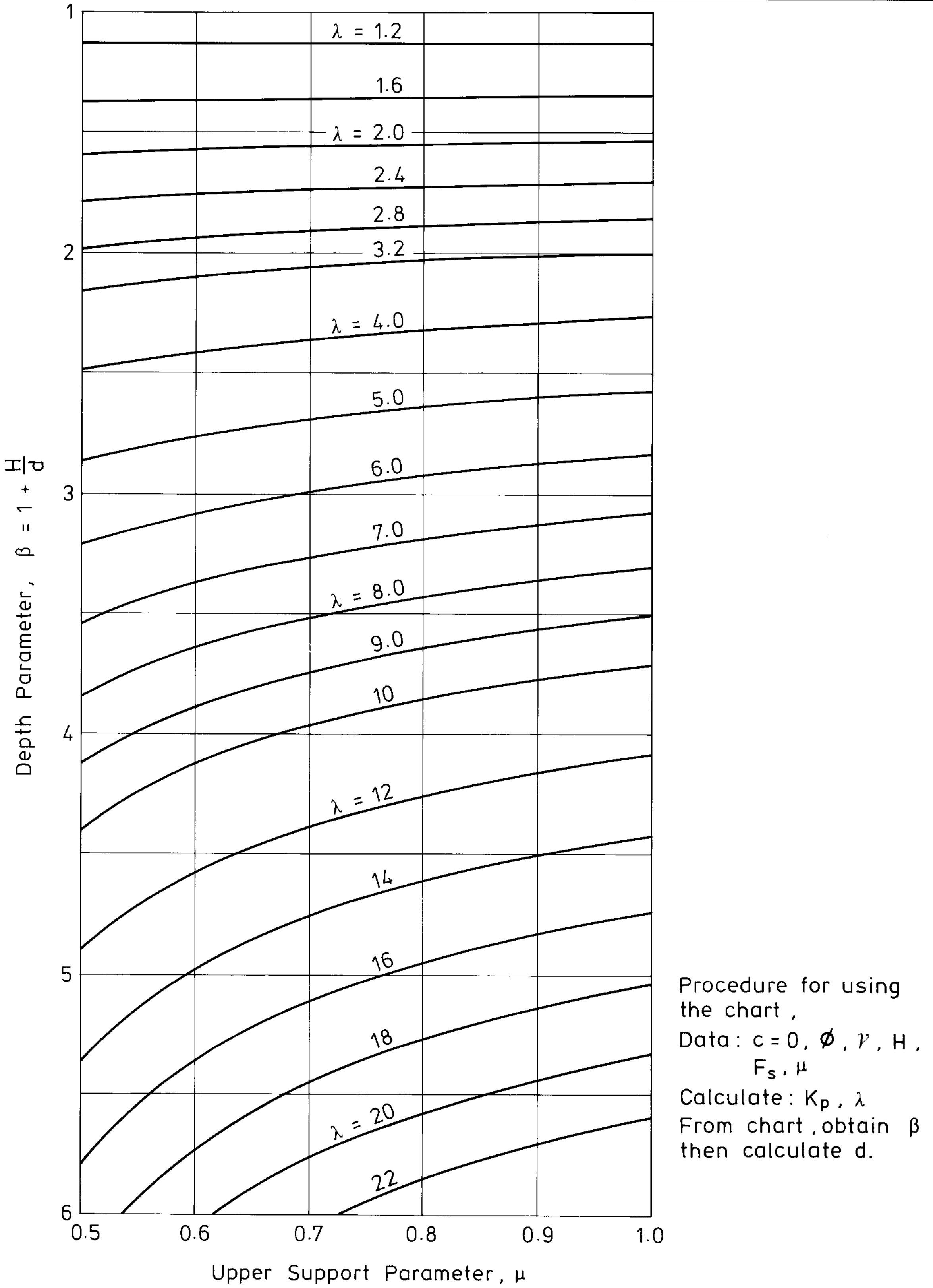
$\frac{4c}{\gamma H}$	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8
μ	λ = 1.2							
0.9	0							0
0.8						1	1	1
0.7					1	1	1	1
0.6				1	1	1	2	1
0.5	0	0	1	1	1	2	2	2
	λ = 1.4							
0.9	0							0
0.8	0	0	1	1	1	1	1	0
0.7	1	1	1	1	1	2	2	
0.6	1	1	1	2	2	2	2	
0.5	1	1	2	2	3	3	3	
	λ = 1.6							
0.9	0	0	0	1	1	1	0	
0.8	1	1	1	1	1	1	1	
0.7	1	1	2	2	2	2	1	
0.6	2	2	2	3	3	3	2	
0.5	2	3	3	4	4	4	2	
	λ = 1.8							
0.9	0	1	1	1	1	1	0	
0.8	1	1	1	1	2	1	0	
0.7	2	2	2	2	3	2	0	
0.6	2	3	3	4	4	3	0	
0.5	3	4	4	5	6	5	0	
	λ = 2.0							
0.9	1	1	1	1	1	1	0	
0.8	1	1	2	2	2	1		
0.7	2	2	3	3	3	2		
0.6	3	3	4	4	4	3		
0.5	4	5	6	6	6	4		
	λ = 2.4							
0.9	1	1	1	1	1	0		
0.8	2	2	2	2	2	1		
0.7	3	3	3	4	3	1		
0.6	4	5	5	5	5	2		
0.5	6	7	8	8	7	2		
$\frac{4c}{\gamma H}$	0.1	0.2	0.3	0.4	0.5	0.6		

0.1	0.2	0.3	0.4	0.5	0.6
λ = 2.8					
1	1	1	1	1	0
2	2	2	2	2	0
3	4	4	4	3	0
5	6	6	6	4	0
7	8	9	9	6	0
λ = 3.2					
1	1	1	1	1	1
2	2	3	2	2	
4	4	4	4	3	
6	6	7	8	4	
9	10	10	9	6	
λ = 3.6					
1	1	1	1	0	
3	3	3	2	1	
4	5	5	4	2	
7	7	7	6	3	
10	11	11	10	4	
λ = 4.0					
1	1	1	1	0	
3	3	3	2	1	
5	5	5	4	1	
7	8	8	8	2	
11	12	12	10	3	
λ = 5.0					
1	1	1	1	0	
3	3	3	2	0	
5	6	5	4	0	
9	9	8	6		
13	14	13	9		
λ = 6.0					
2	2	1	1		
3	4	3	2		
6	6	6	3		
10	10	9	5		
15	15	13	7		
0.1	0.2	0.3	0.4		

0.1	0.2	0.3	0.4
λ = 7			
2	2	1	1
4	4	3	1
6	6	6	2
10	10	9	4
16	16	14	6
λ = 8			
2	2	1	0
4	4	3	1
7	7	5	2
11	11	9	3
18	17	13	4
λ = 10			
2	2	1	
4	4	3	
7	7	5	
12	11	8	
19	18	12	
λ = 12			
2	2		
4	4		
8	7		
13	12		
21	19		
λ = 18			
2	2		
5	4		
8	7		
14	12		
23	19		
λ = 20			
2			
5			
9			
14			
24			
0.1			

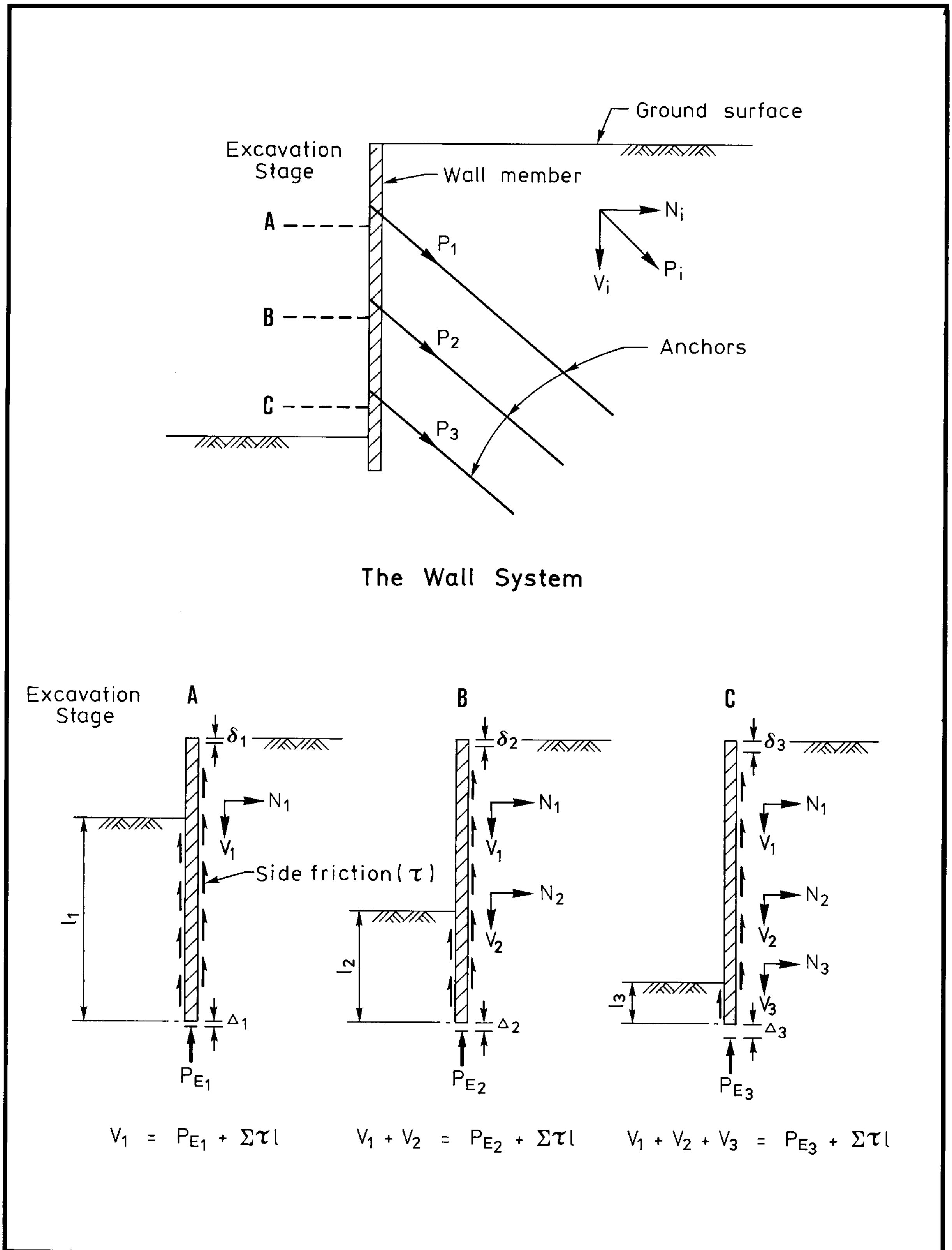
Note : See Figure 38 for nomenclature.

Figure 39 - Values of k for Correcting Depth Parameter β for Anchor Location μH (Browzin, 1983)



Note : See Figure 38 for nomenclature.

Figure 40 - Design Chart for Calculating Embedment Depth of a Propped Wall in Cohesionless Soils (Browzin, 1983)



Notes :
(1) $N_1 + N_2 + N_3$ = Area under design pressure diagram.
(2) Values of P_1, P_2 and P_3 may change as excavation continues.
(3) Test loading of anchor will temporarily modify load distribution.

Figure 41 - Idealised Force System on an Anchored Wall (Hanna, 1968)

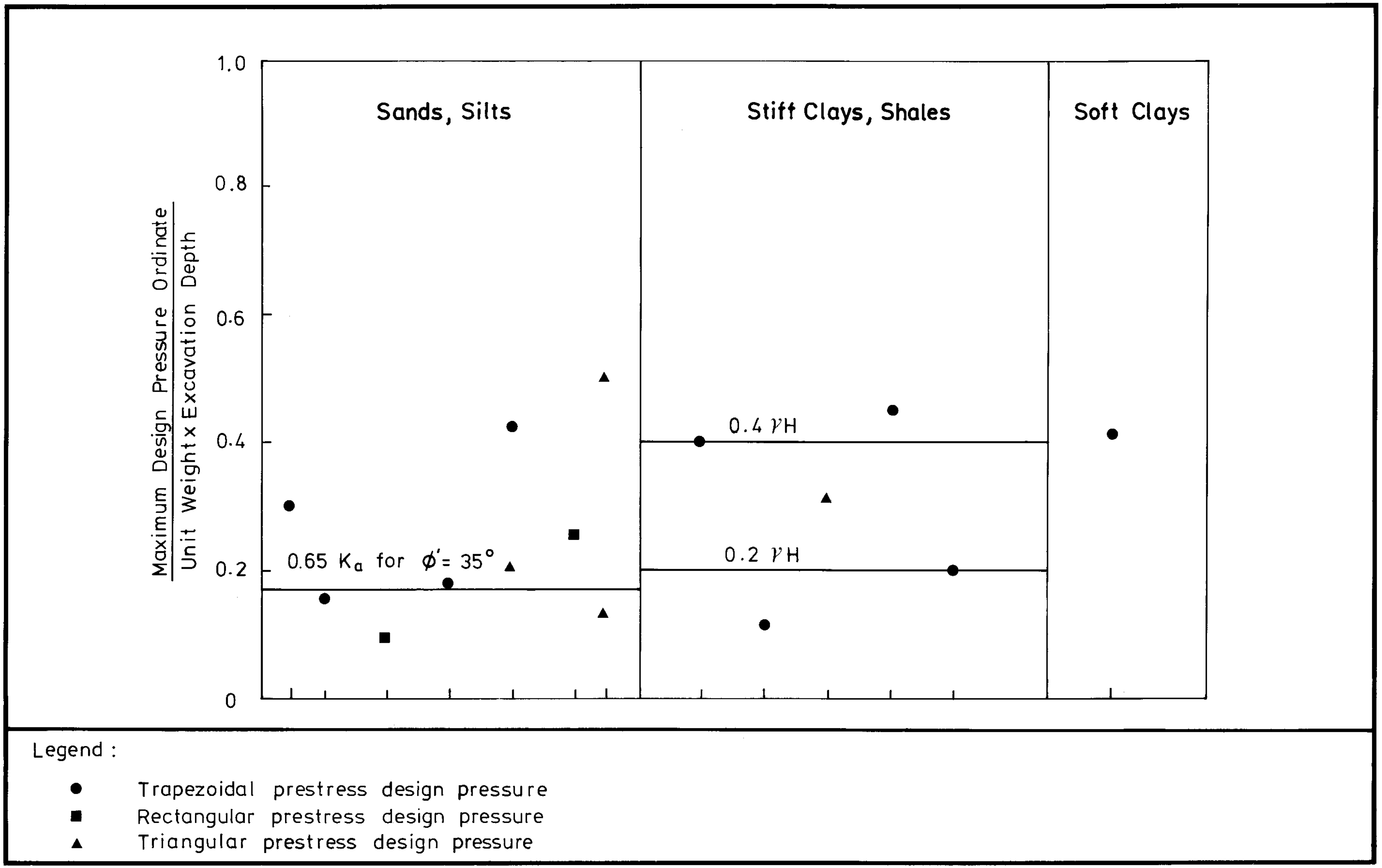
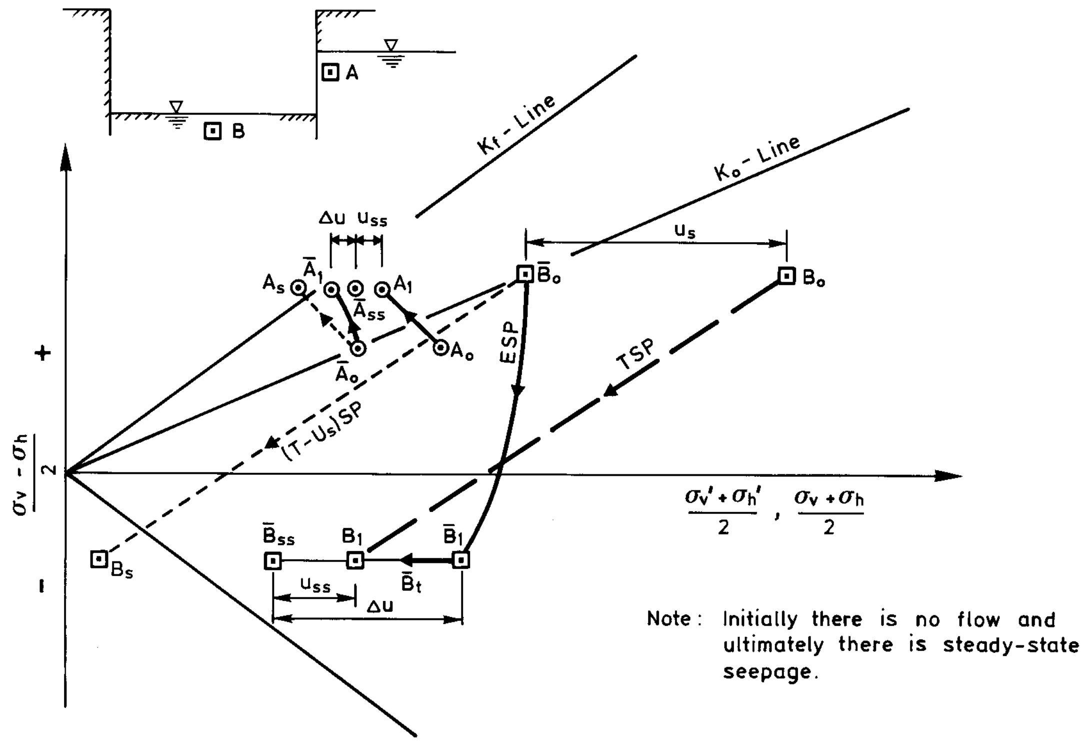
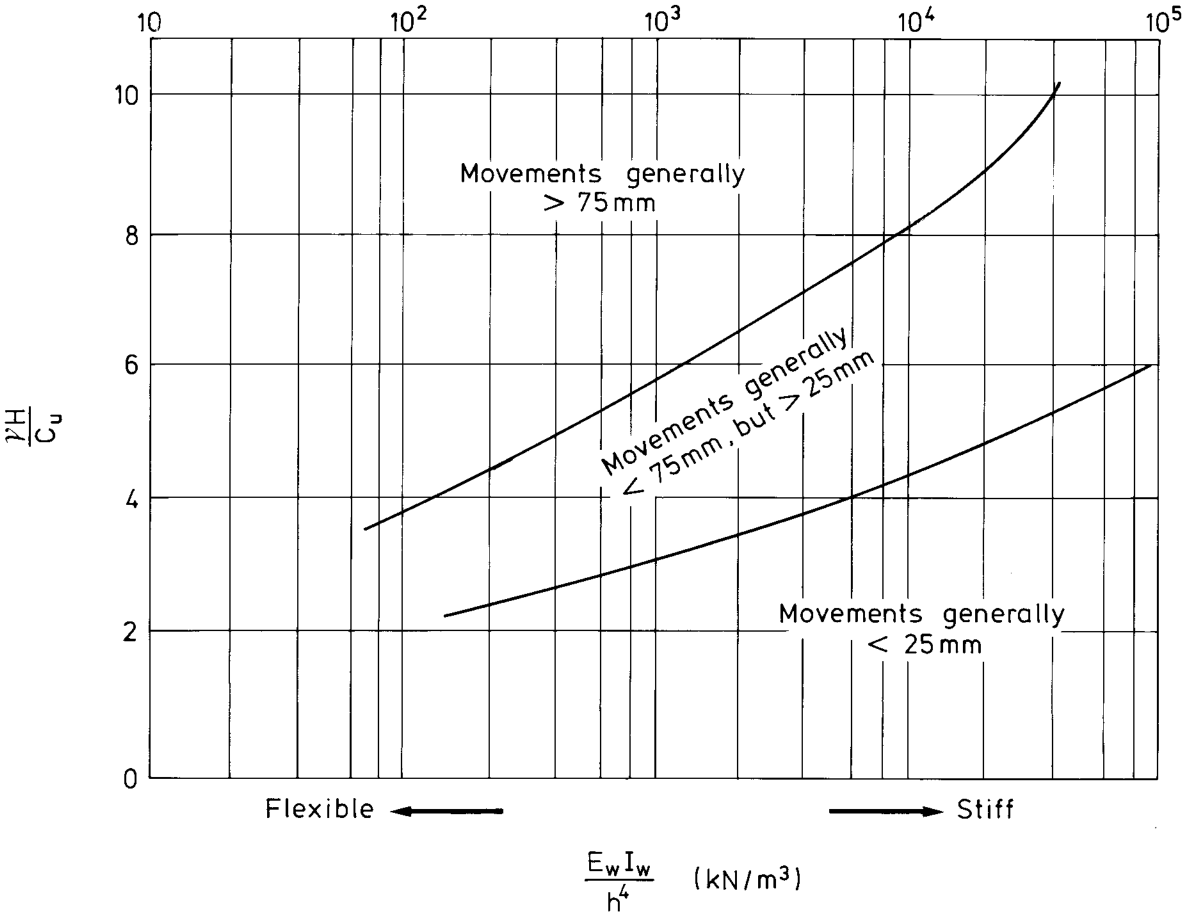


Figure 42 - Prestress Design Pressures Reported for Tied-back Wall (Clough, 1975)



Stresses and Strains for Soil Elements near an Excavation	Soil Element A	Soil Element B
Initial (Static) Pore Pressure, u_s	$A_o\bar{A}_o$	$B_o\bar{B}_o$
Pore Pressure at Steady-state Flow, u_{ss}	$A_1\bar{A}_{ss}$	$B_1\bar{B}_{ss}$
Pore Pressure upon Unloading	Decreases	Decreases
Pore Pressure during Consolidation	Decreases	Increases
Strain upon Unloading	Vertical compression	Vertical extension
Strain during Consolidation	Vertical compression	Vertical extension
Undrained Shear Strength during Consolidation	Increases	Decreases

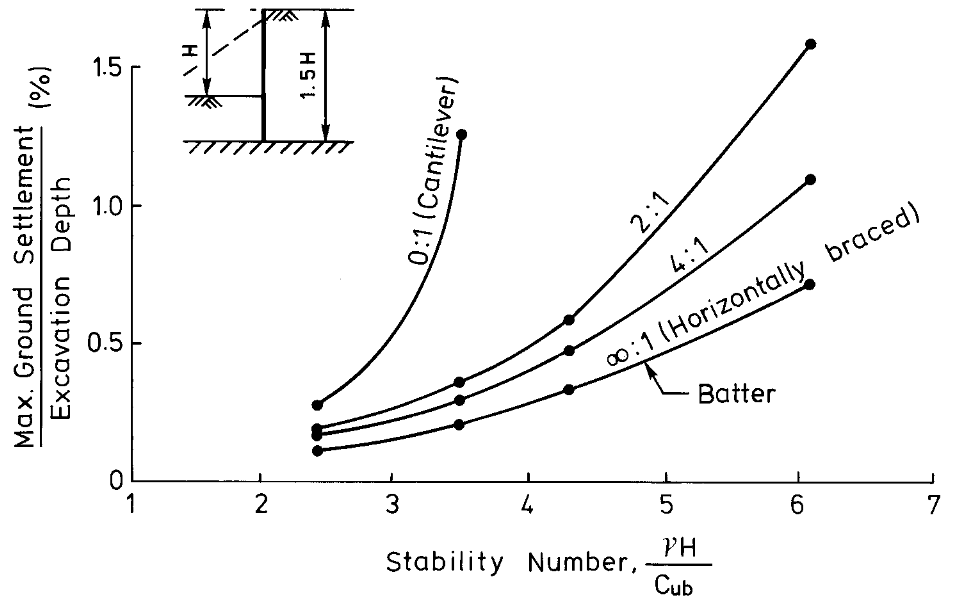
Figure 43 - Stress Paths for Soil Elements near Excavation (Lambe, 1970)



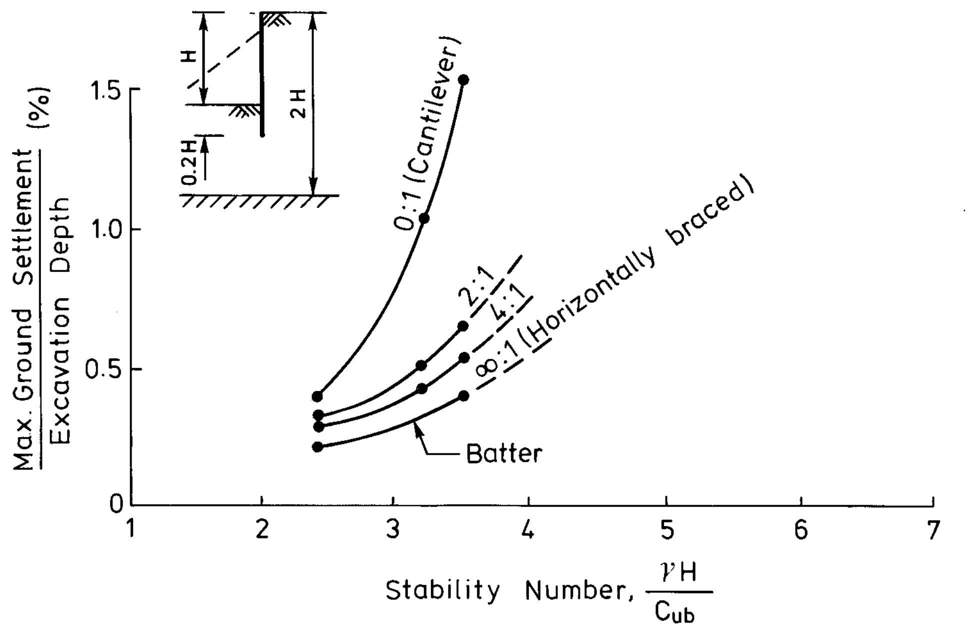
Legend :

h	Vertical distance between support levels or between support level and excavation base
E_w	Modulus of elasticity of wall material
I_w	Moment of inertia of wall per unit length

Figure 44 - Effects of Wall Stiffness and Support Spacing on Lateral Wall Movements (after Goldberg et al, 1976)



(a) Fully Penetrating, Fixed End, MZ-27 Sheet Pile Wall



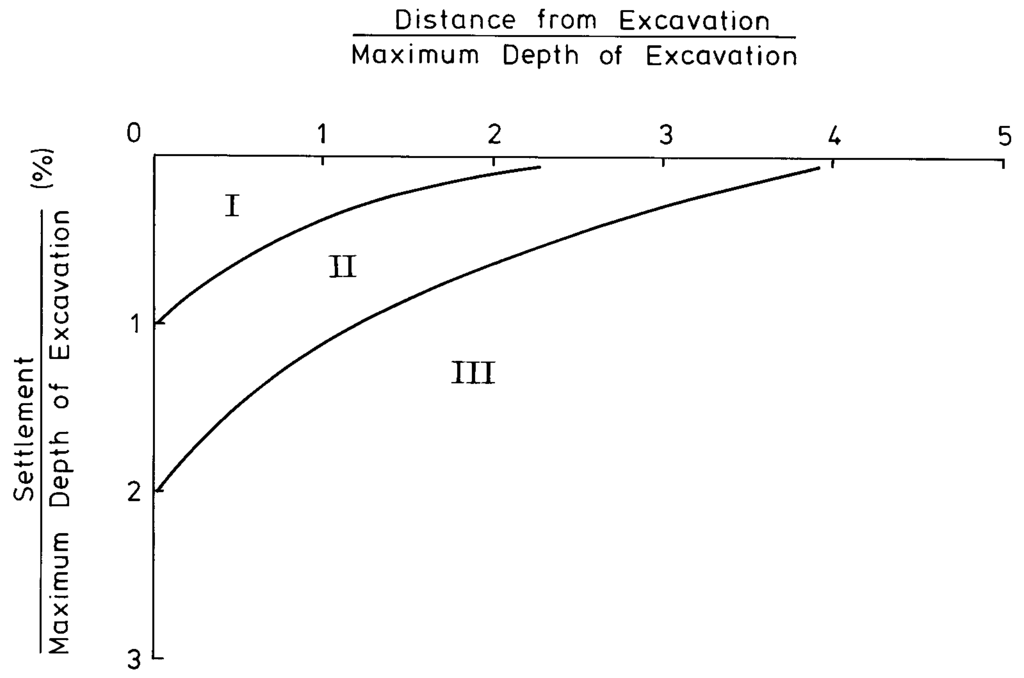
(b) Partially Penetrating, Free End, MZ-27 Sheet Pile Wall

Legend:

C_{ub} Undrained shear strength of clay at base of excavation

Note : Settlement is that at the end of construction after batter excavation and strut installation.

Figure 45 - Relationship between Maximum Settlement and Stability Number for Different Batters (after Clough & Denby, 1977)



Notes: (1) Zone I - Sand and soft to hard clay, average workmanship.

(2) Zone II - (a) Very soft to soft clay.

(i) Limited depth of clay below bottom of excavation.

(ii) Significant depth of clay below bottom of excavation but $N_b < 5.14$.

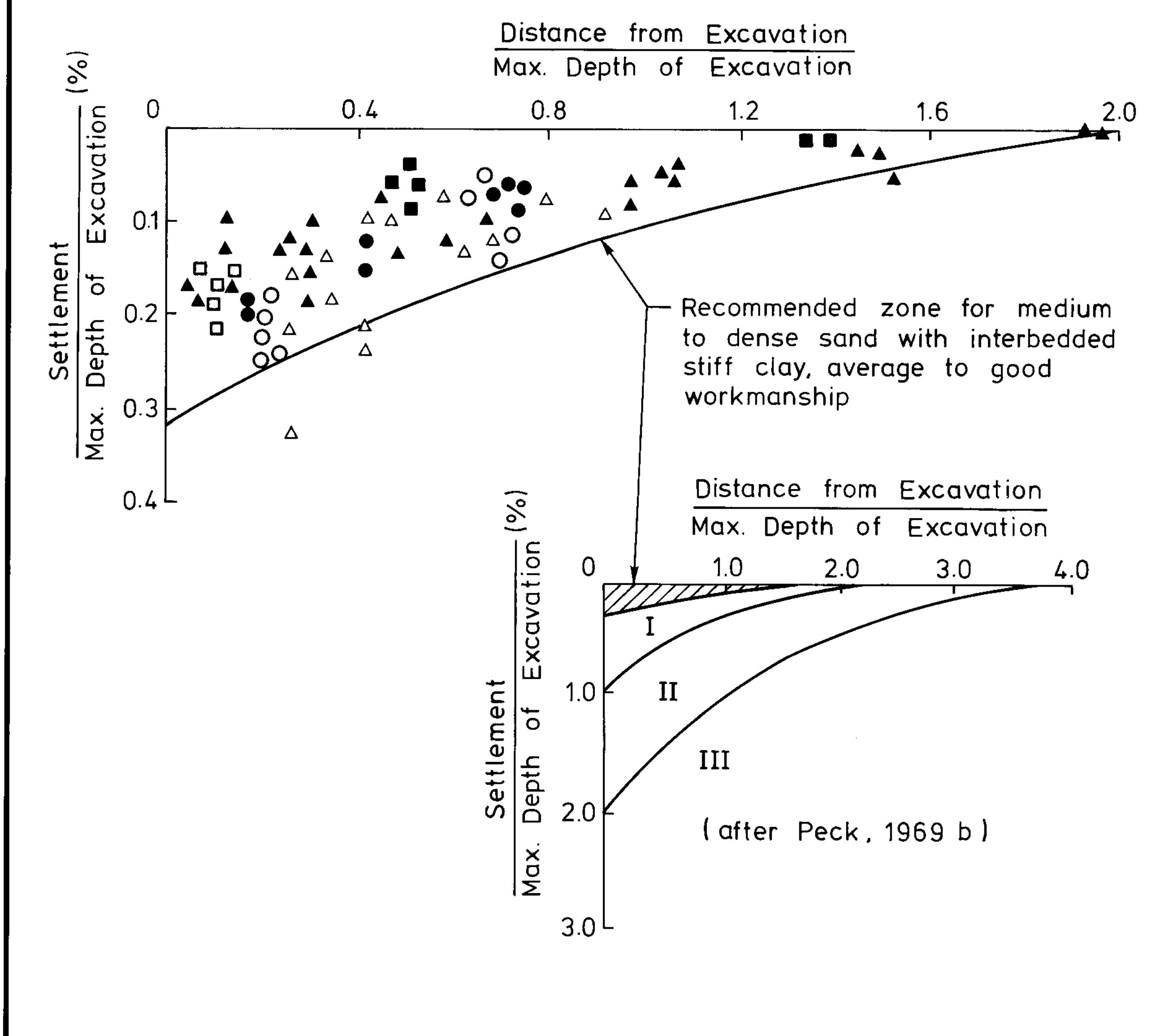
(b) Settlements affected by construction difficulties.

(3) Zone III - Very soft to soft clay to a significant depth below bottom of excavation and with $N_b > 5.14$,

where $N_b = \frac{\gamma H}{C_{ub}}$ and C_{ub} is as defined in Figure 31.

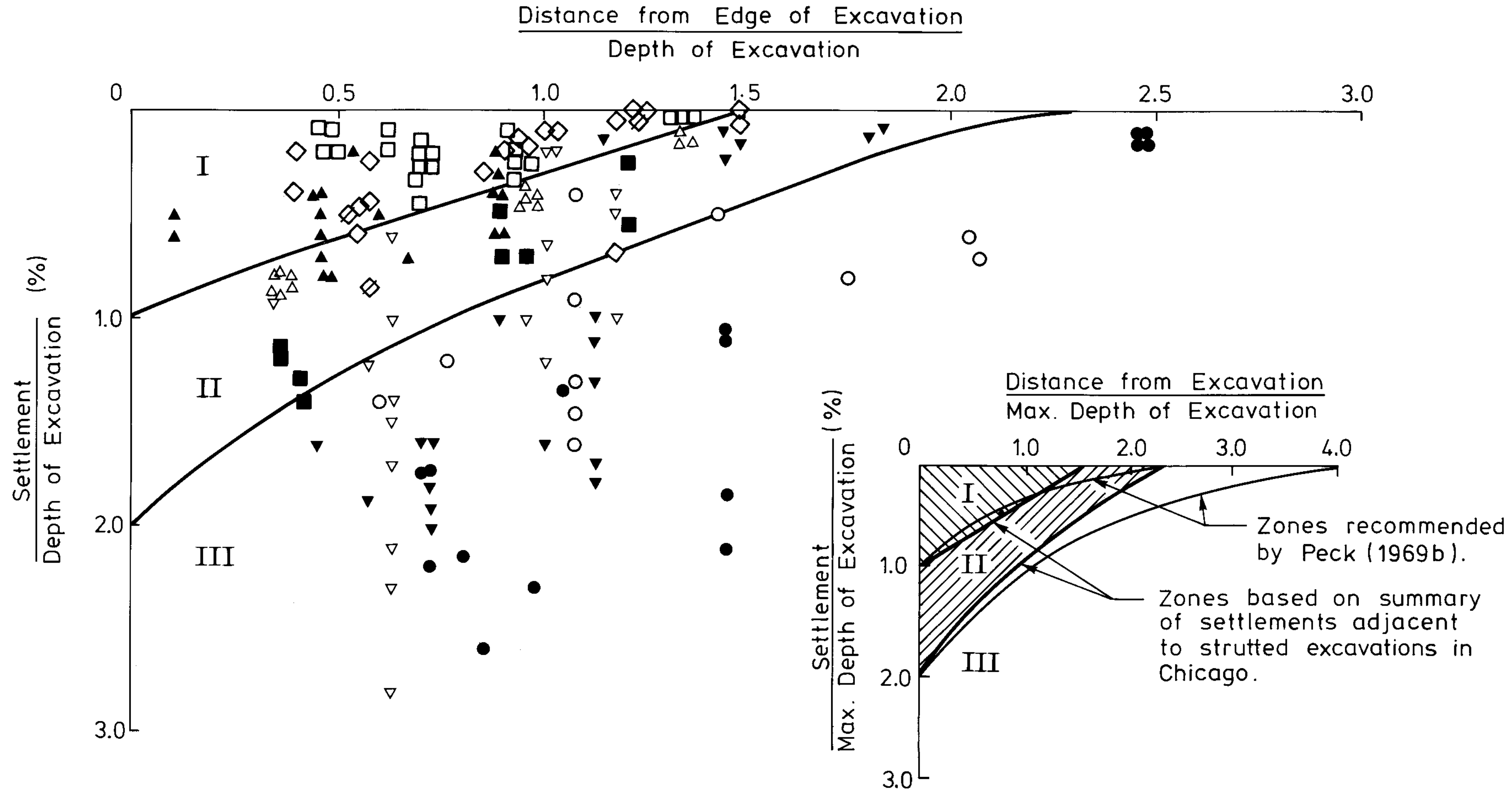
(4) The data used to derive the three zones shown in this figure are taken from excavations supported by soldier piles or sheet piles with cross-lot struts or tie-backs.

Figure 46 - Observed Settlements behind Excavations (after Peck, 1969b)



Case	Symbol	Location	Maximum Depth (ft.)	Support
1	●	7th & G St., N.W.	60	Soldier pile-lagging with cross-lot struts for all cases
2	○	9th & G St. N.W.	60	
3	▲	11th & G St., N.W.	55	
4	△	12th & I St., N.W.	60	
5	■	7th & G St., N.W.	80	
6	□	1st & D St., S.E.	80	

Figure 47 - Summary of Settlements Adjacent to Strutted Excavations in Washington, D.C. (O'Rourke et al, 1976)



Notes: (1) Zone I - Well-braced excavations with slurry wall or substantial berms left permanently in place.
(2) Zone II - Excavations with temporary berms and raking strut support.
(3) Zone III - Excavations with ground loss from caisson construction or insufficient wall support.
(4) In this context, the term 'berm' was used for a passive soil buttress.

Figure 48 - Summary of Settlements Adjacent to Strutted Excavations in Chicago (O'Rourke et al, 1976)

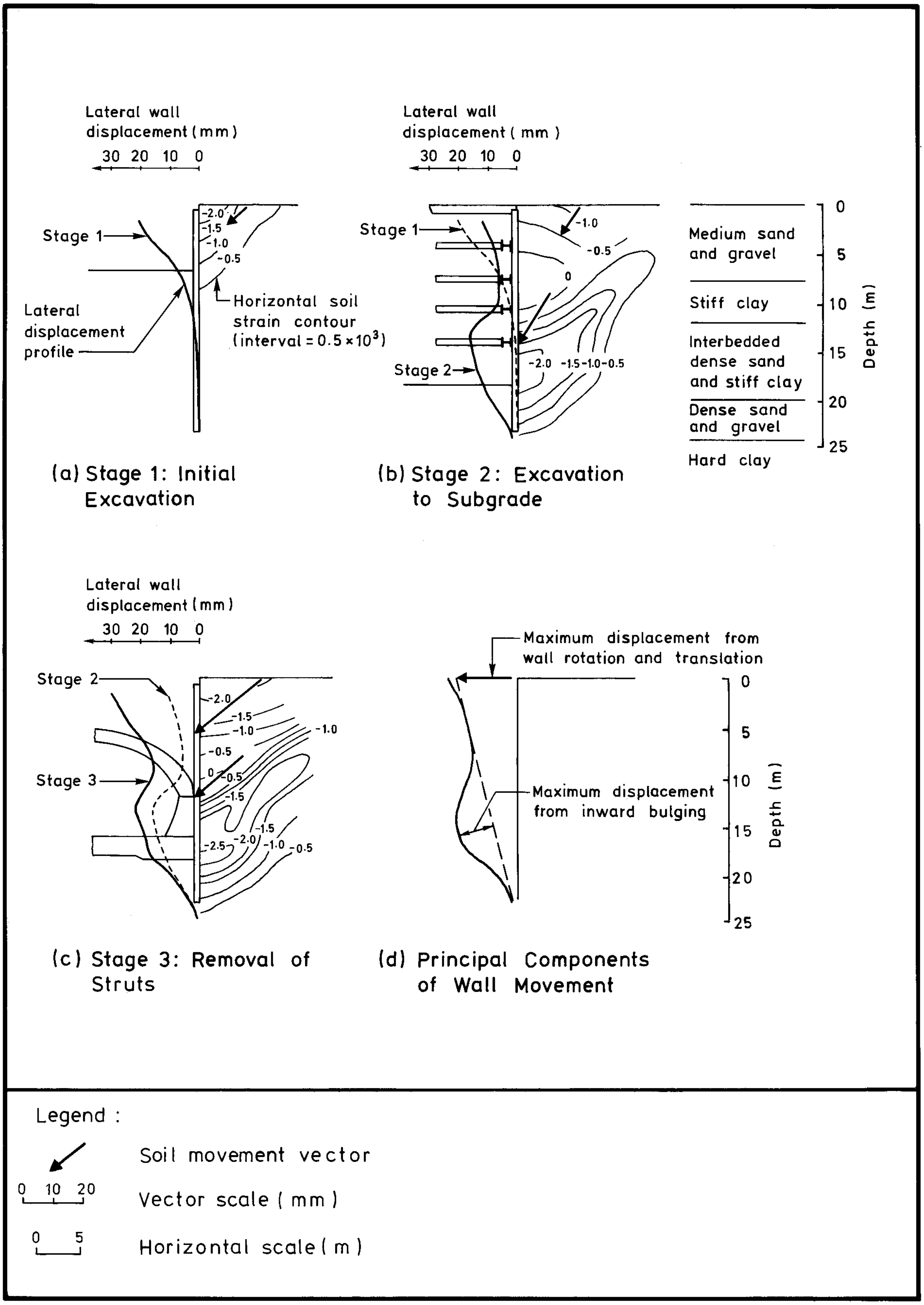
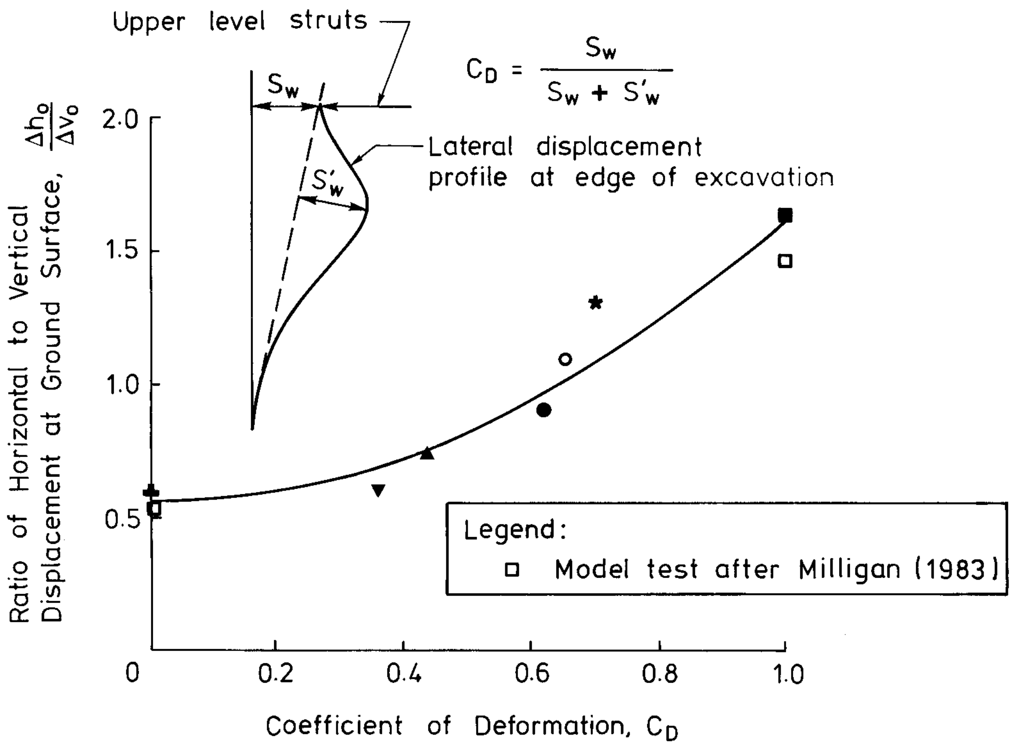


Figure 49 - Horizontal Strains Associated with Various Stages of Strutted Excavation Construction (O'Rourke, 1981)



Case	Symbol	Maximum Depth (m)	Support	Soil	Location
1	●	18	Soldier piles & lagging, 5 strut levels	Sand and stiff clay	Washington, D.C.
2	▲	13	Soldier piles & lagging, 3 levels of struts and rakers	Soft to medium clay	Chicago, Ill.
3	▼	8	Sheet pile, 3 raker levels	Soft to medium clay	Chicago, Ill.
4	■	13	Slurry wall, 2 levels of tiebacks and rakers	Soft to medium clay	Chicago, Ill.
5	*	9	Soldier piles & lagging, 2 raker levels	Soft to medium clay	Chicago, Ill.
6	o	8	Sheet pile, 2 raker levels	Soft to medium clay	Chicago, Ill.
7	✚	14	Sheet pile, 3 levels of struts and rakers	Soft to medium clay	San Francisco California

Figure 50 - Ratio of Horizontal to Vertical Ground Movement as Function of Coefficient of Deformation of Fixed-end Walls (O'Rourke, 1981)

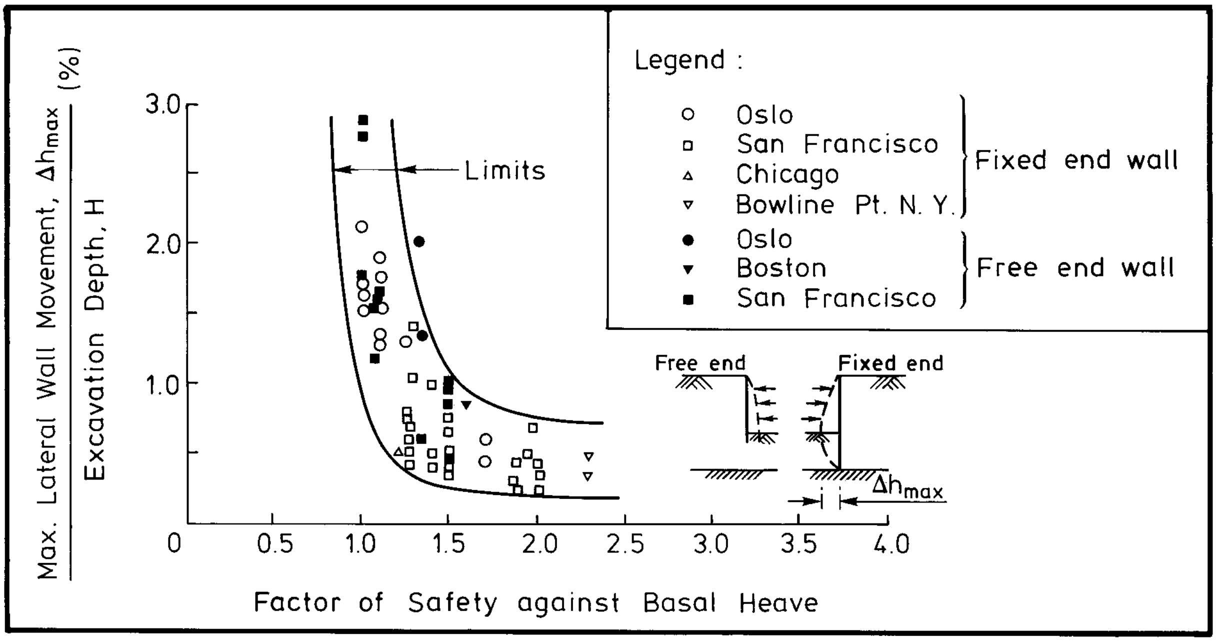


Figure 51 - Relationship between Factor of Safety against Basal Heave and Nondimensional Maximum Lateral Wall Movement for Case History Data (Clough et al, 1979)

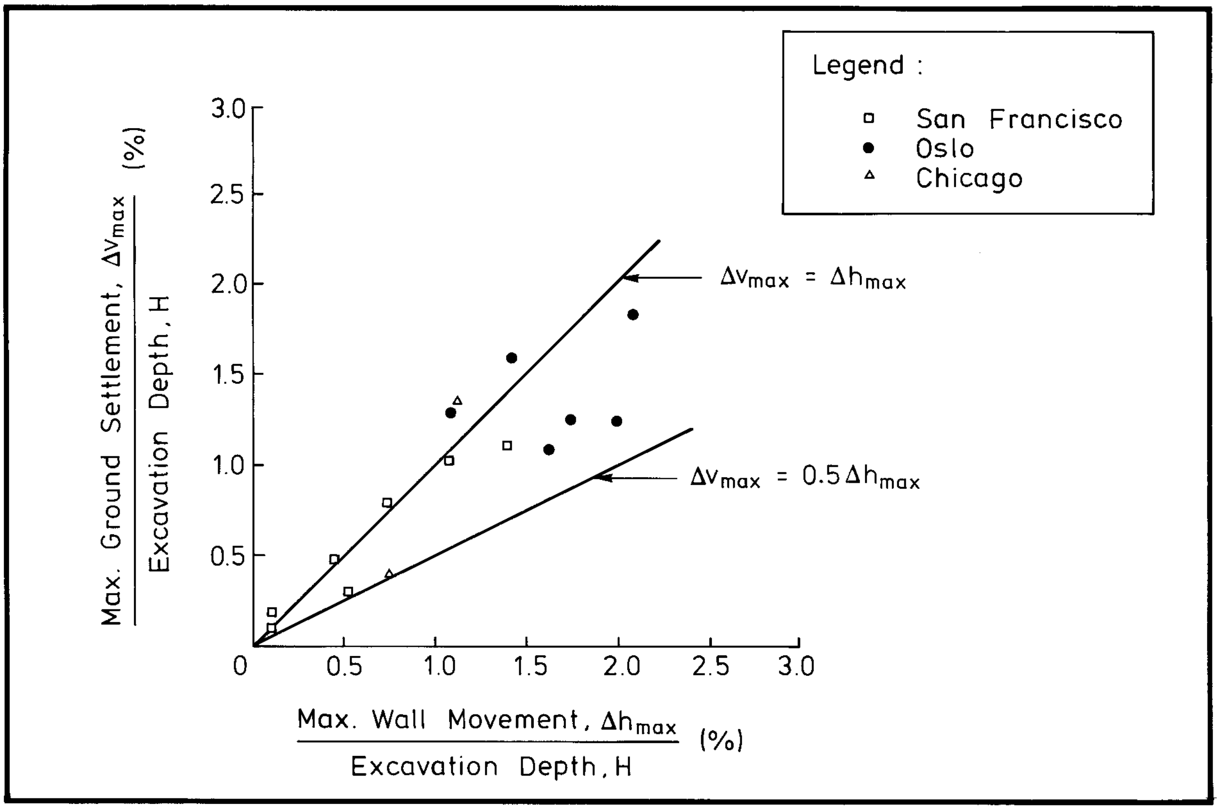


Figure 52 - Relationship between Maximum Ground Settlements and Maximum Lateral Wall Movements for Case History Data (Mana & Clough, 1981)

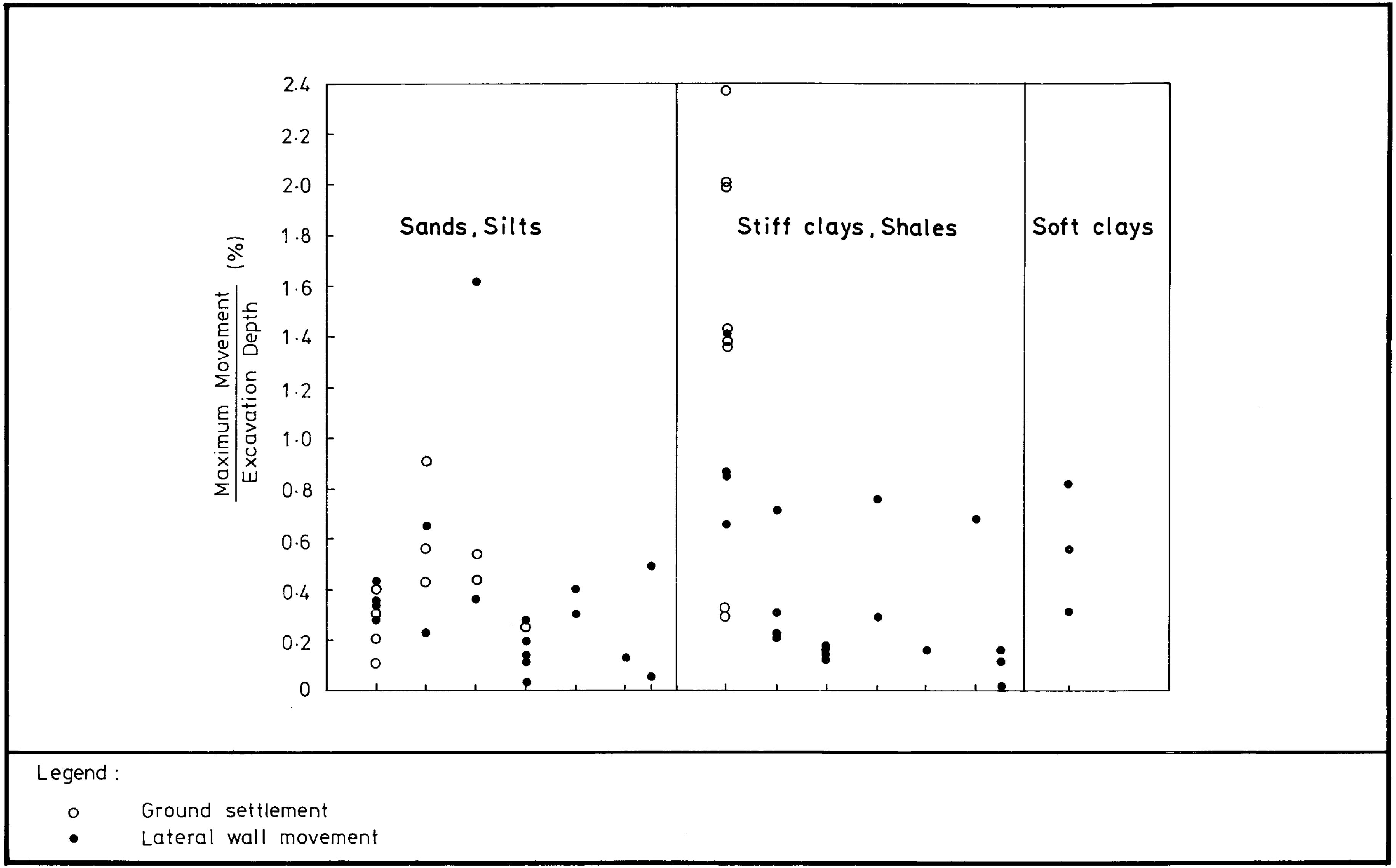


Figure 53 - Observed Movements of Tied-back Wall Systems (Clough, 1975)

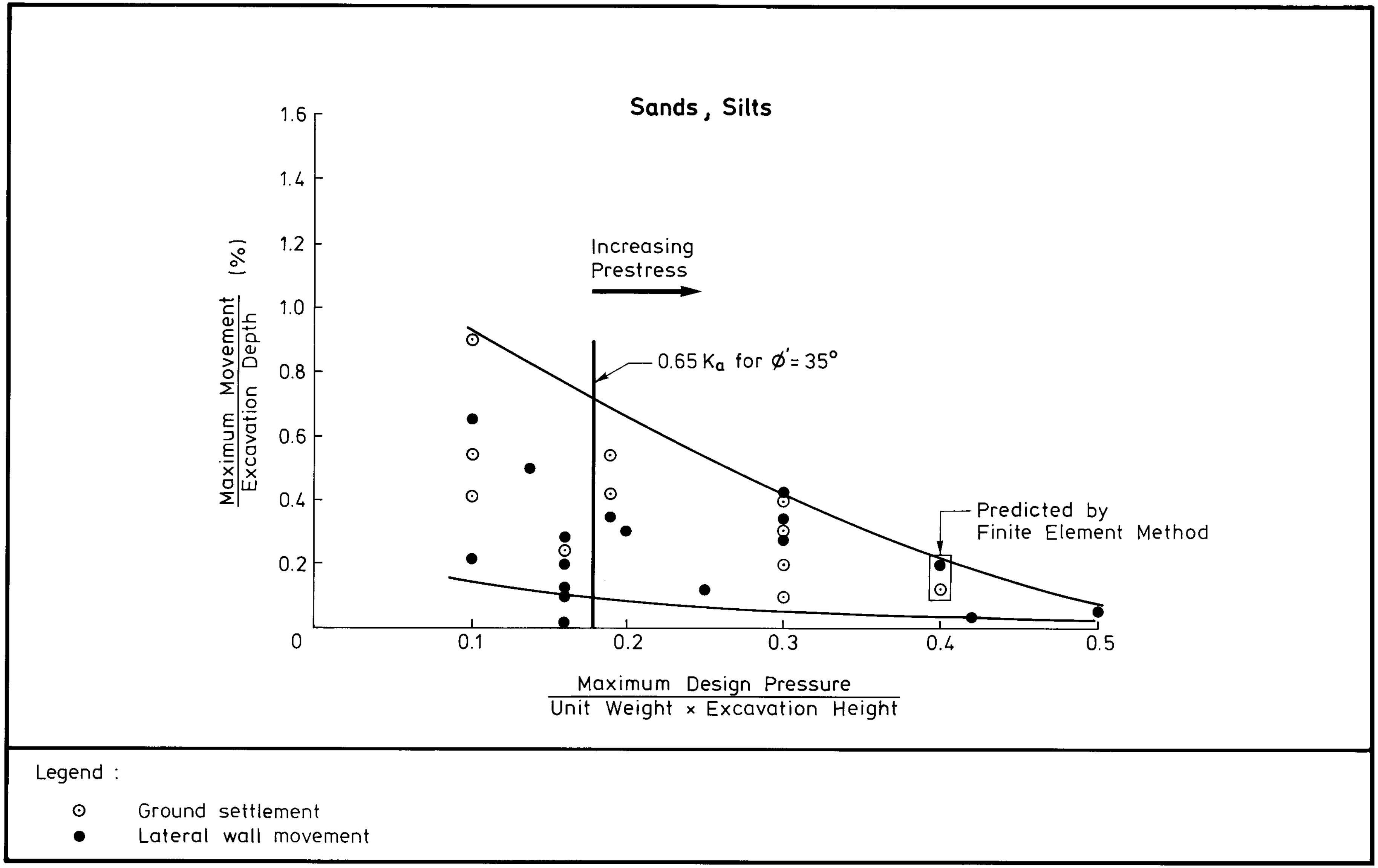


Figure 54 - Relationship between Prestress Pressure and Tied-back Wall Movement for Sands (Clough, 1975)

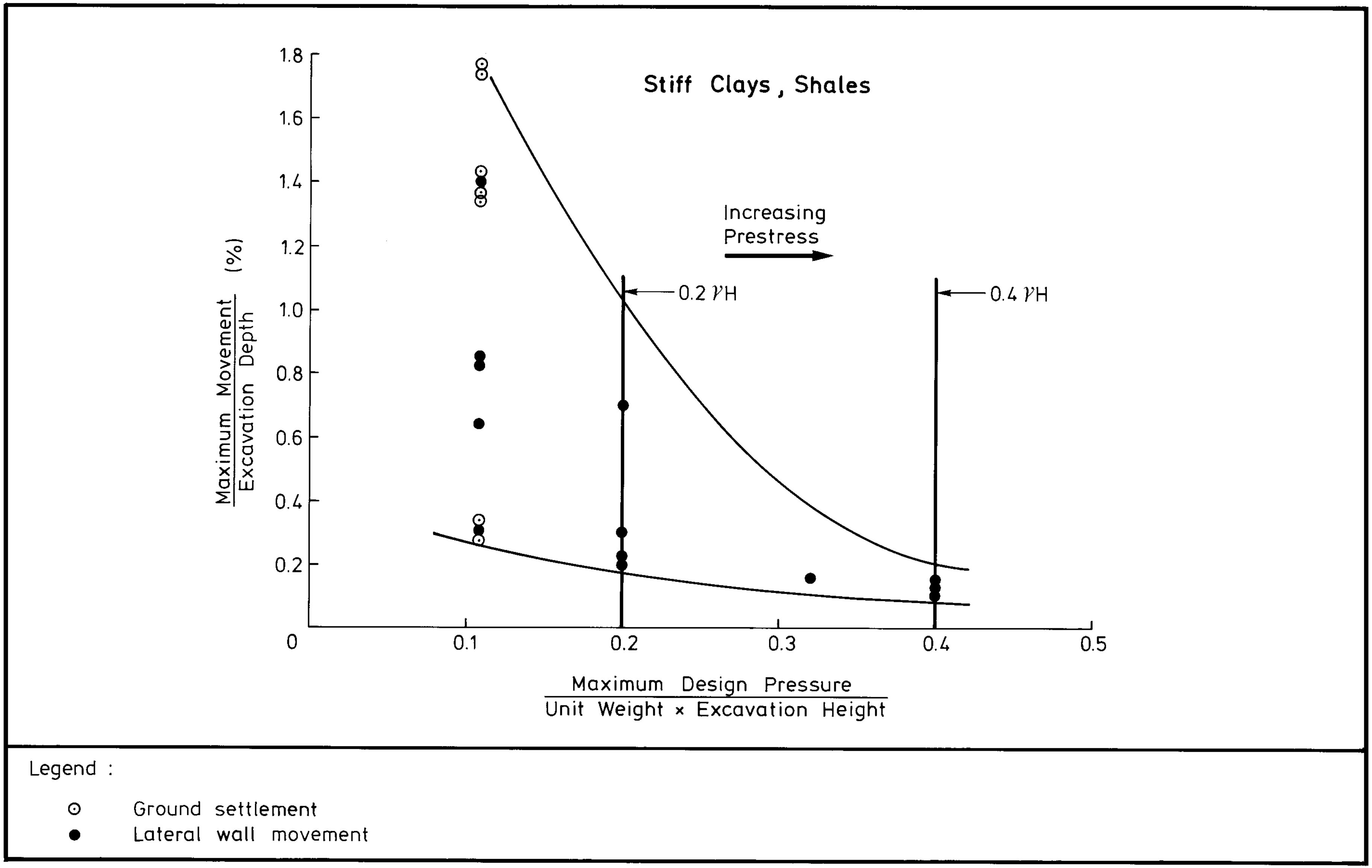
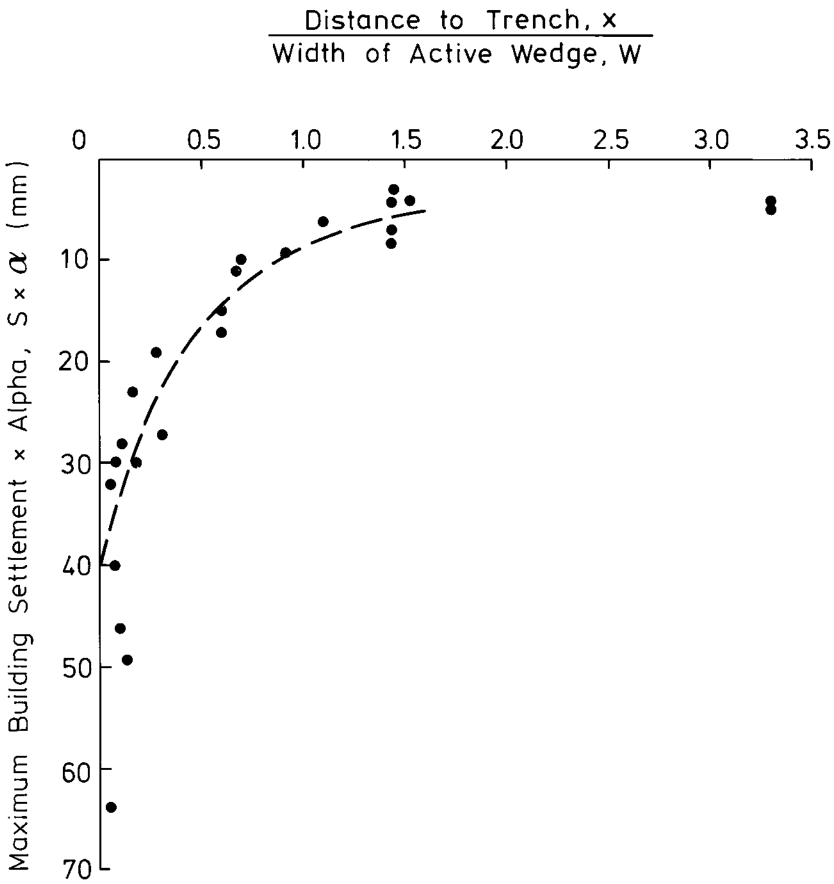


Figure 55 - Relationship between Prestress Pressure and Tied-back Wall Movement for Stiff Clay (Clough, 1975)



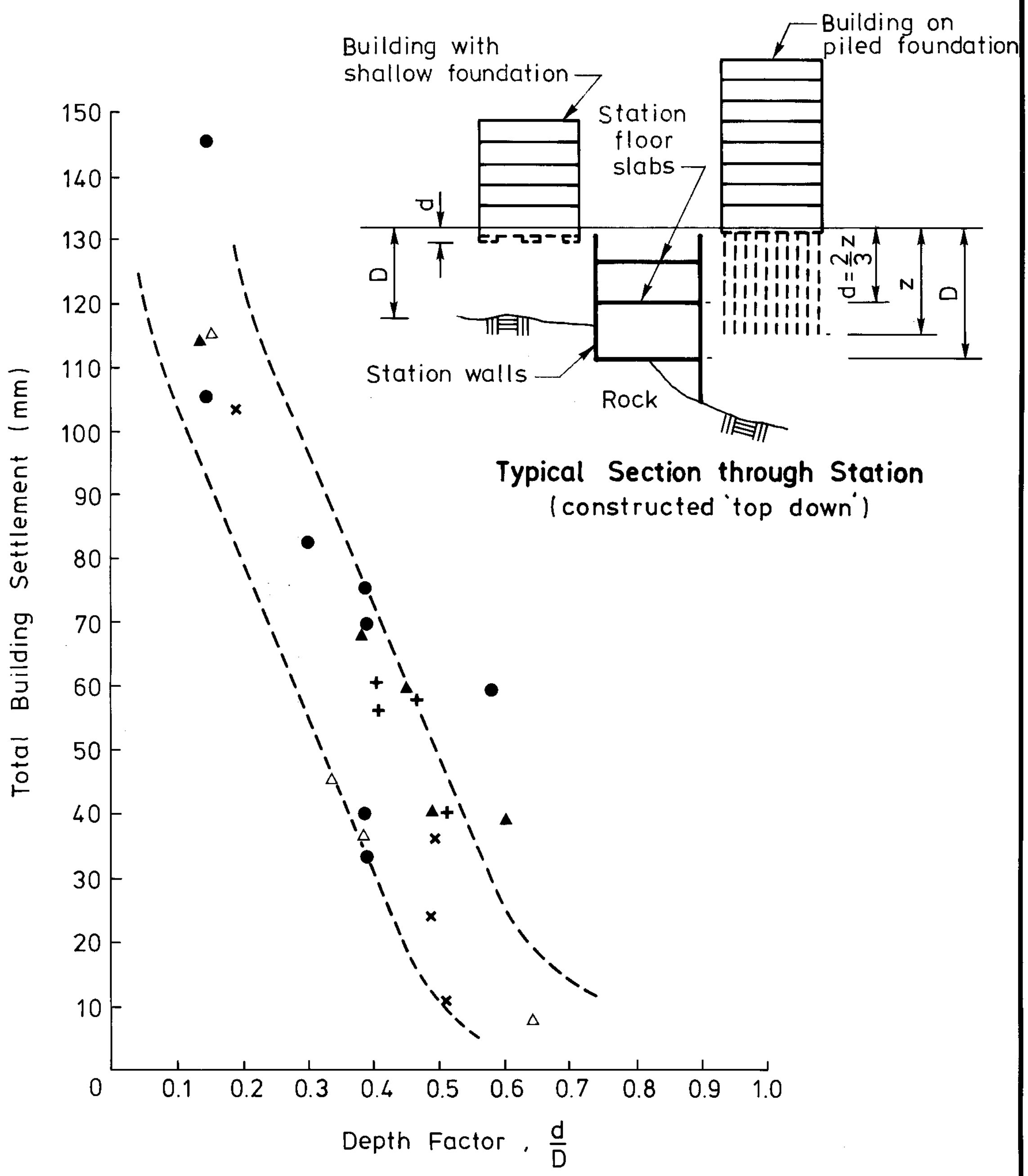
Note :

$$\alpha = 0.5 + \frac{d}{H}$$

where d = Foundation depth

H = Trench depth.

Figure 56 - Maximum Building Settlements due to Slurry Trench Excavation as a Function of Foundation Depth (Cowland & Thorley, 1984)



Legend :

● ▲ + △ × Denotes data derived from different Hong Kong Mass Transit Railway station locations

Figure 57 - Relationship between Total Building Settlements and Depth Factor at Hong Kong Mass Transit Railway Stations (after Morton et al, 1980a)

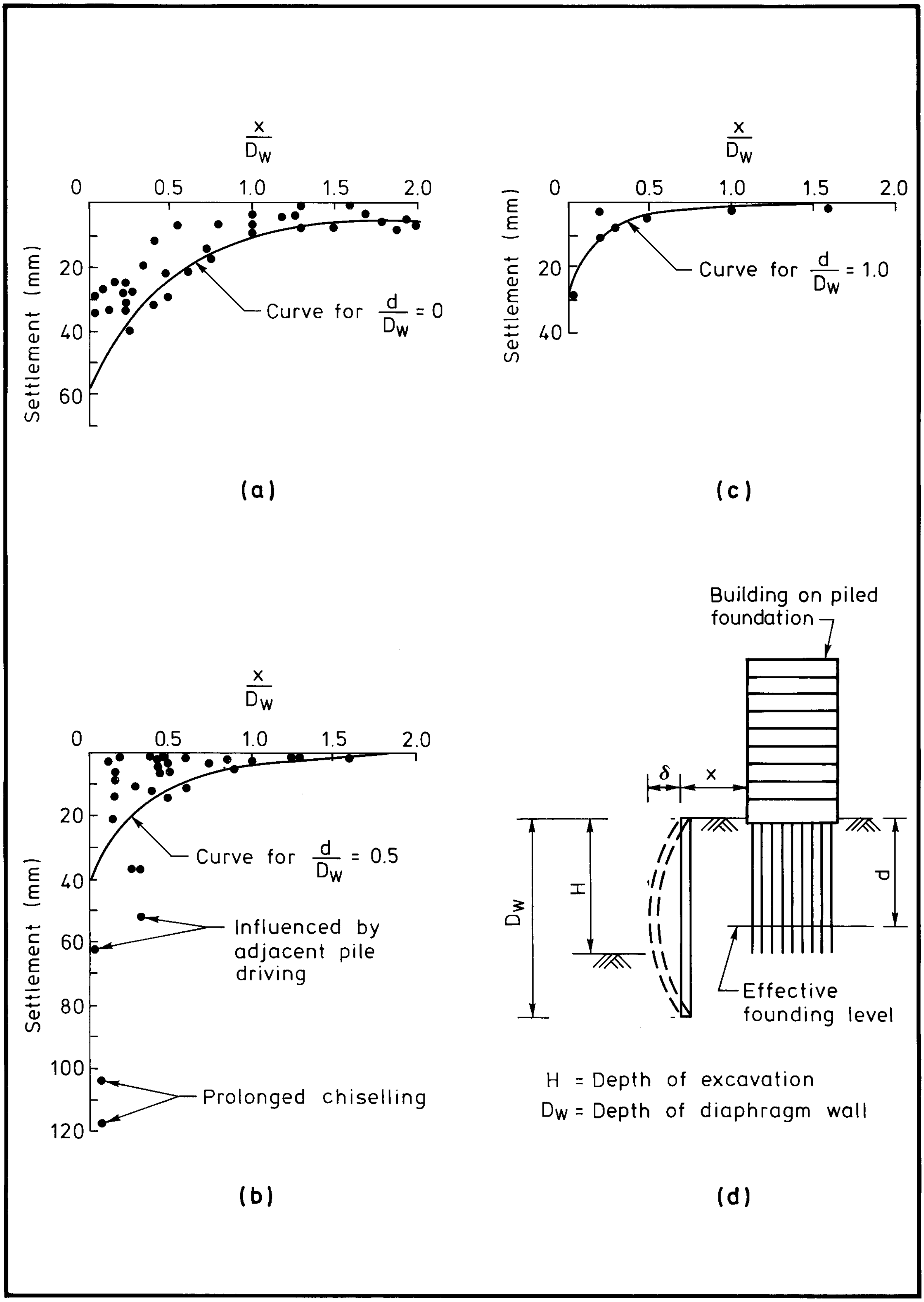


Figure 58 - Maximum Building Settlement due to Diaphragm Wall Installation (Budge-Reid et al, 1984)

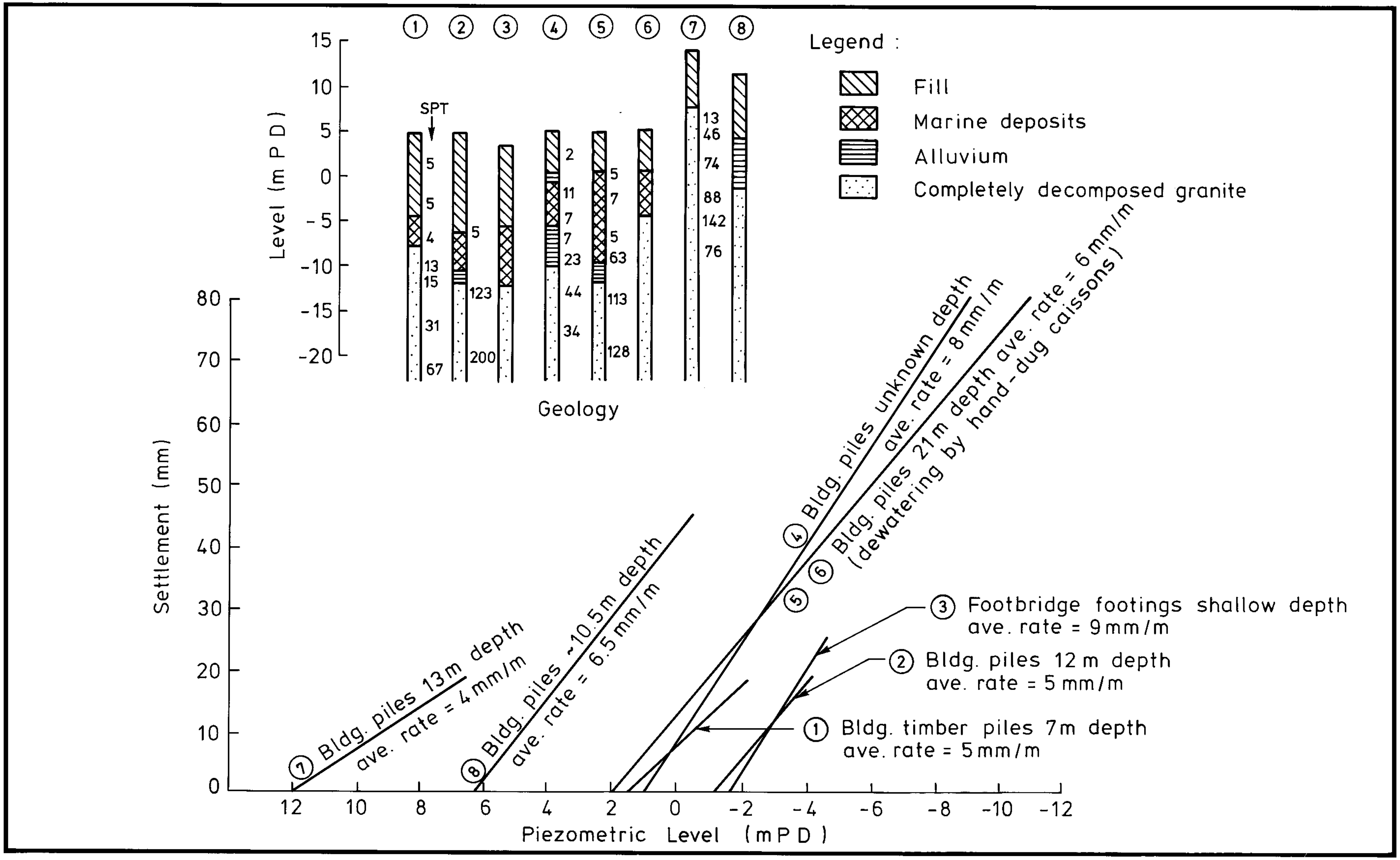
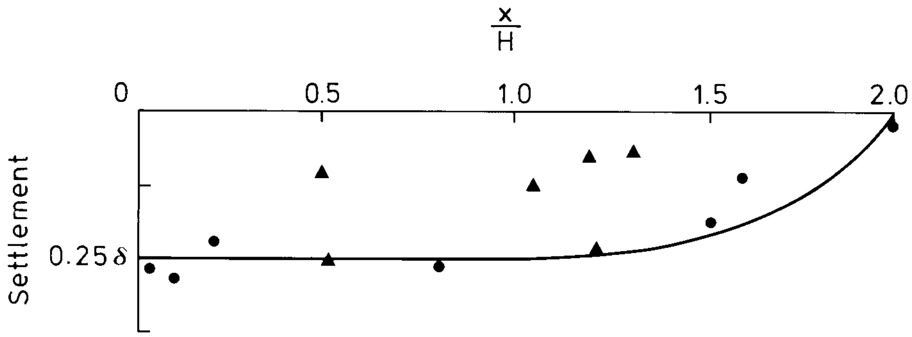


Figure 59 - Average Rate of Building Settlement for Each Metre Drop in Piezometric Level for Various Foundation Systems and Subsoil Conditions (after Budge-Reid et al, 1984)

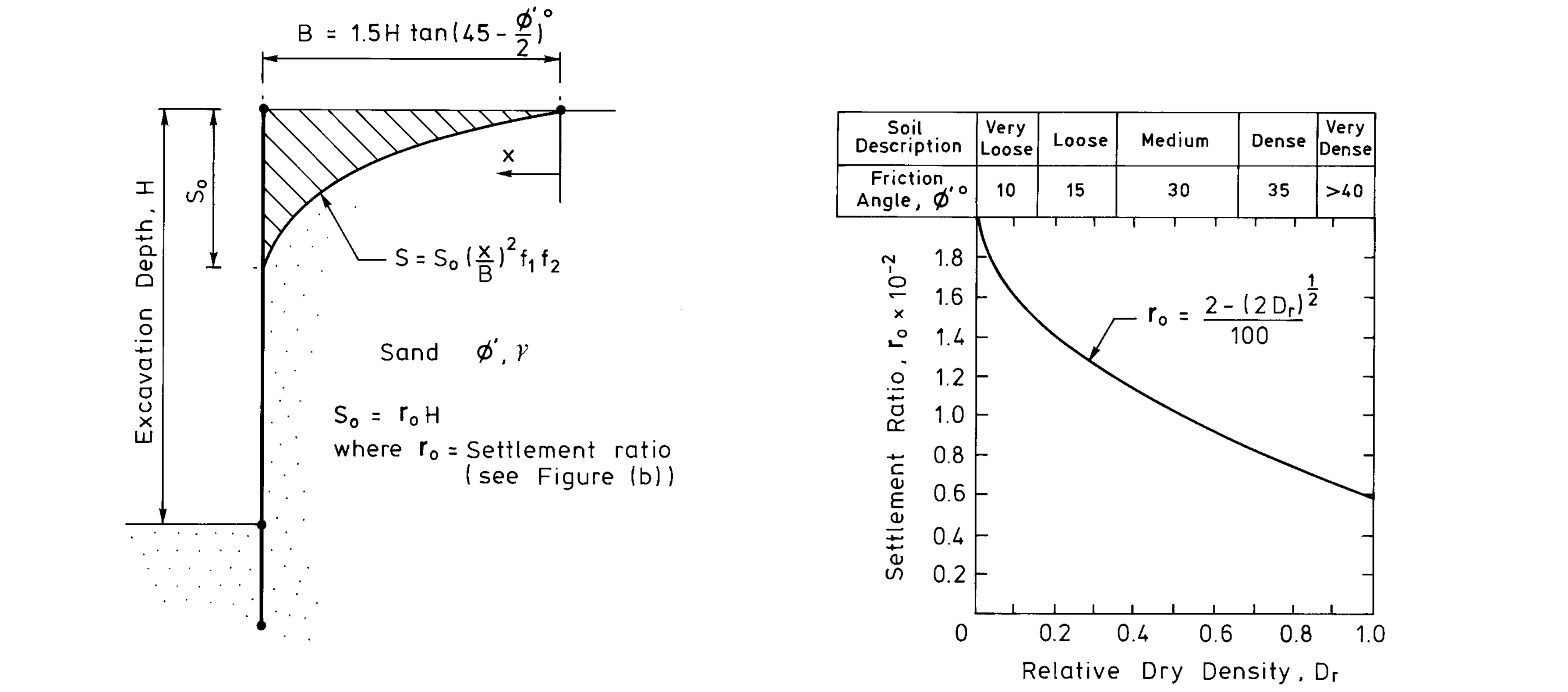


Legend :

▲ Piled foundations
● Ground or shallow foundations

Notes : (1) See Figure 58(d) for nomenclature.
(2) The curve is based on observations during construction and excavation of boxes utilizing diaphragm walls and top down sequence of construction which allows the floor slab strut to be installed as soon as possible.

Figure 60 - Settlement of Buildings during Excavation of Station Box Resulting from Lateral Wall Movement (Budge-Reid et al, 1984)



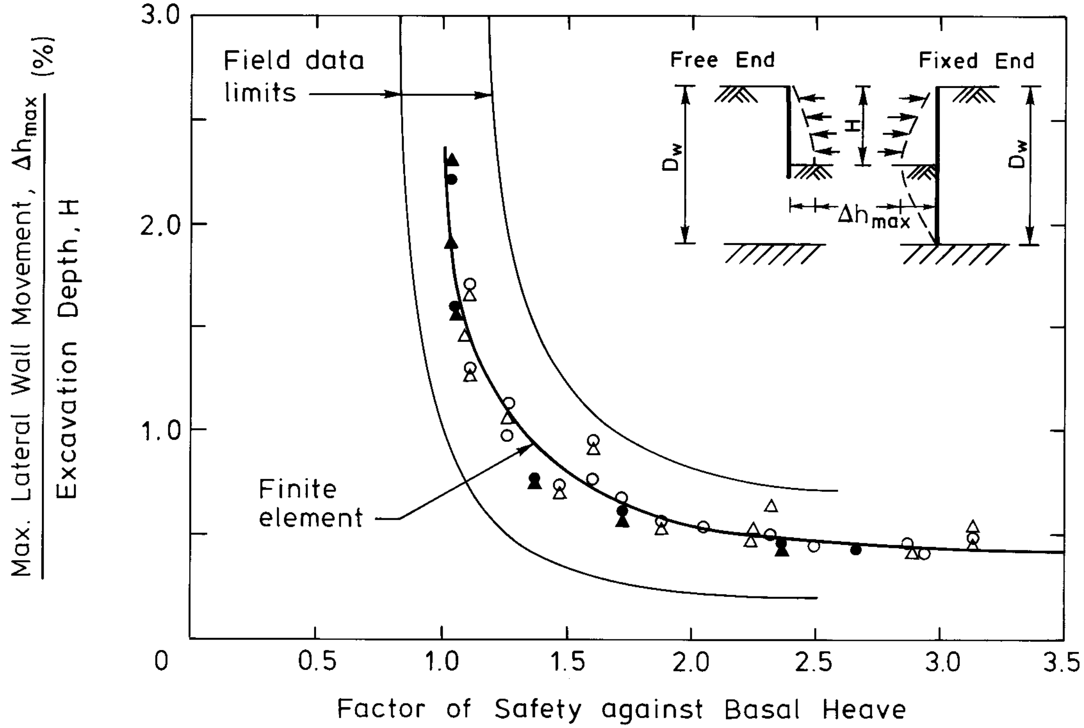
(a) Ground Settlement, S, Adjacent to Wall

(b) Variation of r_o with Soil Properties

Factor	Workmanship			
	Excellent	Good	Average	Poor
f_1	0.8	0.90	1.0	1.1

Factor	Construction Difficulty		
	None	Average	Severe
f_2	1.0	1.02	1.05

Figure 61 - Semi-empirical Method to Estimate Settlement (Bauer, 1984)



Legend :

▲ ● End of construction

△ ○ Intermediate stages

Note :

$E = 300\ C_u$

$D_w = 30$ m

Strut stiffness = 2000 t/m per m

B = 12 m

PZ - 38 sheet pile

Strut spacing = 3.5 m

Figure 62 - Analytically Defined Relationship between Nondimensionalized Maximum Lateral Wall Movement and Factor of Safety against Basal Heave (Mana & Clough, 1981)

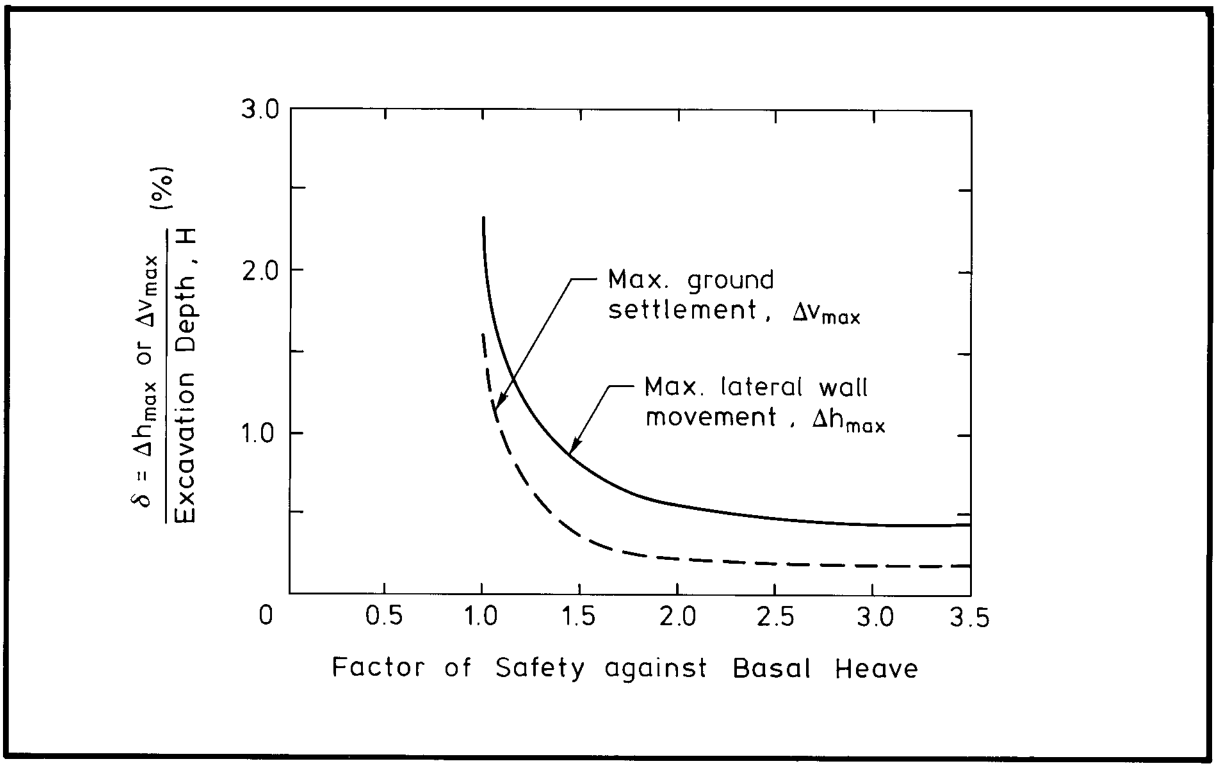


Figure 63 - Analytically Defined Relationship between Maximum Lateral Wall Movement/Ground Settlement and Factor of Safety against Basal Heave (Mana & Clough, 1981)

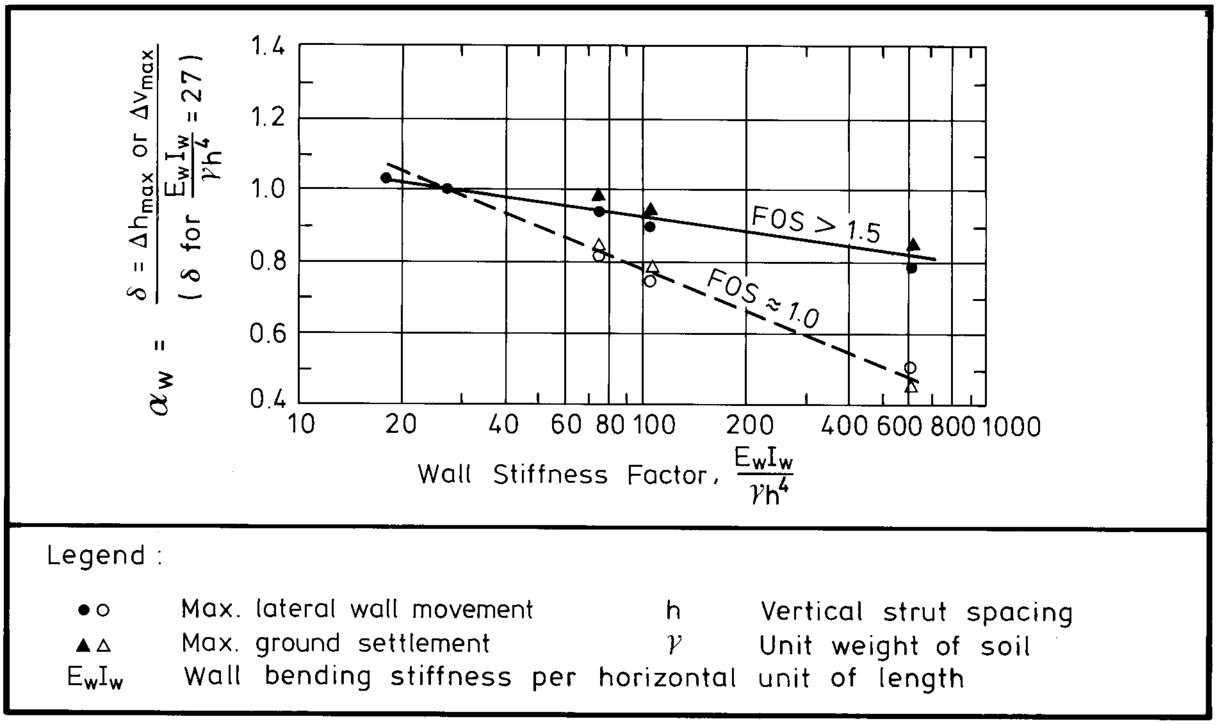


Figure 64 - Effect of Wall Stiffness on Maximum Lateral Wall Movement/Ground Settlement (Mana & Clough, 1981)

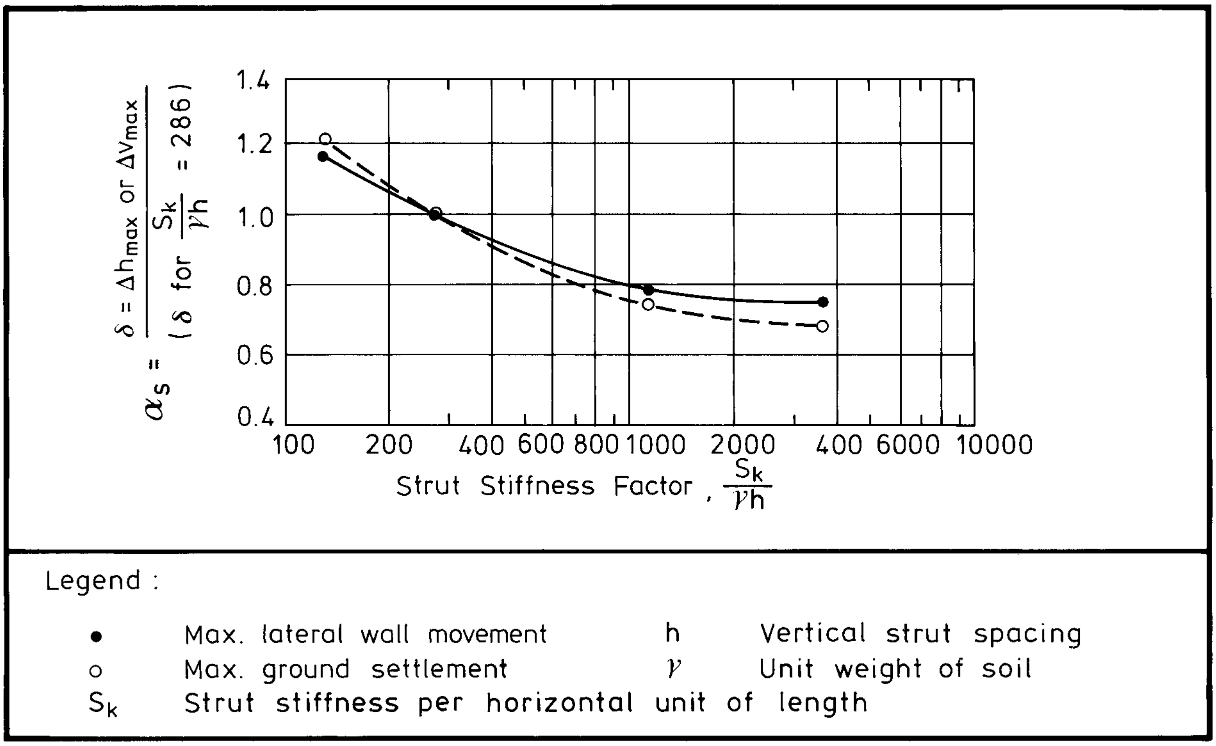


Figure 65 - Effect of Strut Stiffness on Maximum Lateral Wall Movement/Ground Settlement (Mana & Clough, 1981)

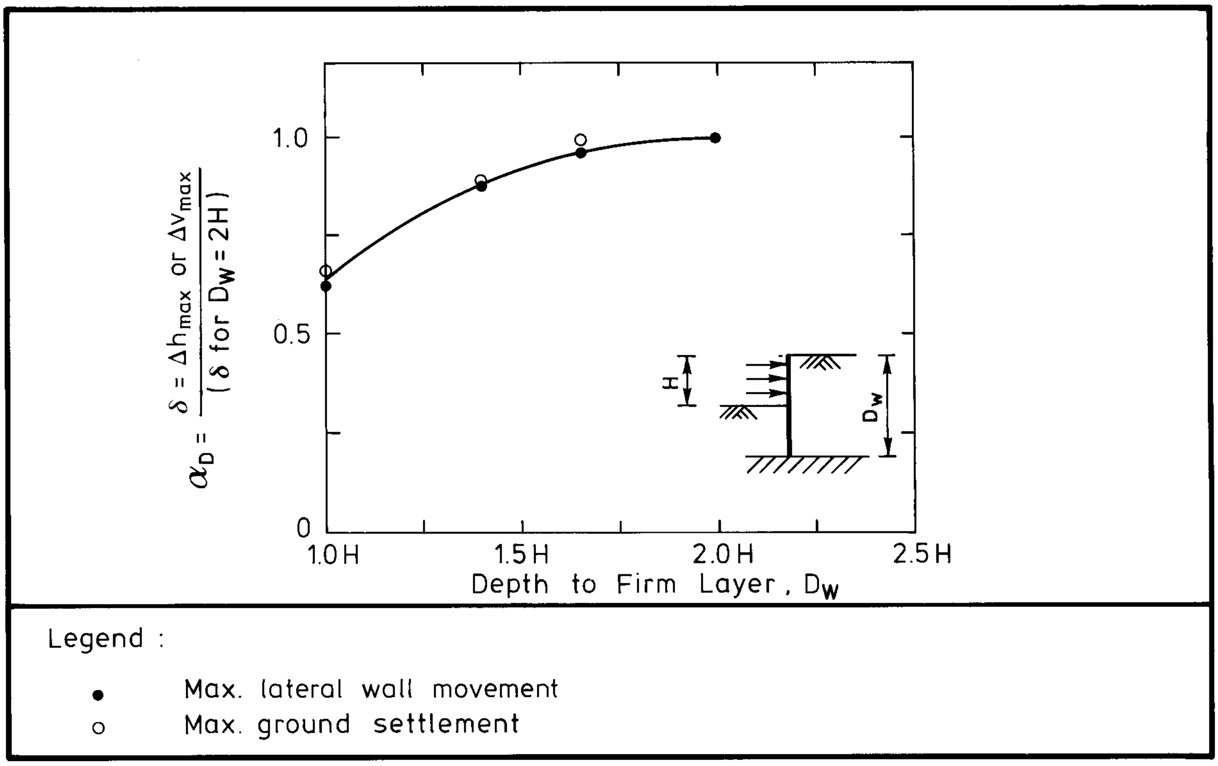


Figure 66 - Effect of Depth to Firm Layer on Maximum Lateral Wall Movement/Ground Settlement (Mana & Clough, 1981)

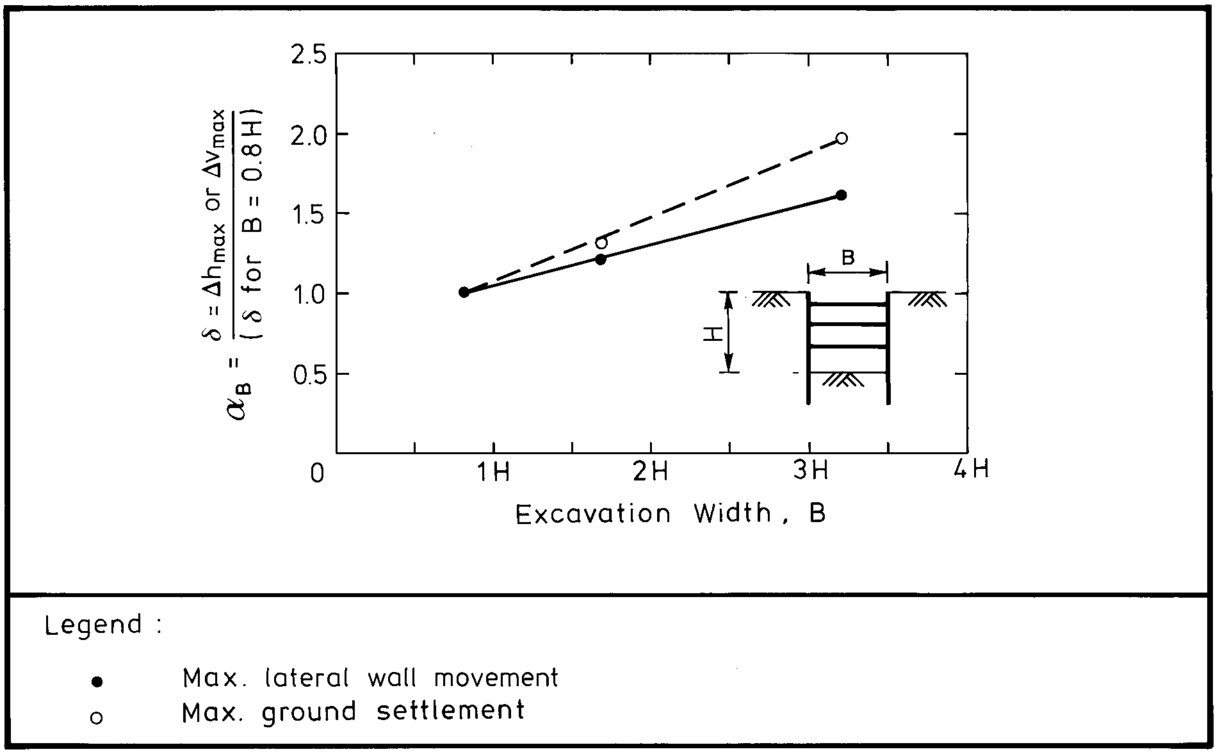


Figure 67 - Effect of Excavation Width on Maximum Lateral Wall Movement/Ground Settlement (Mana & Clough, 1981)

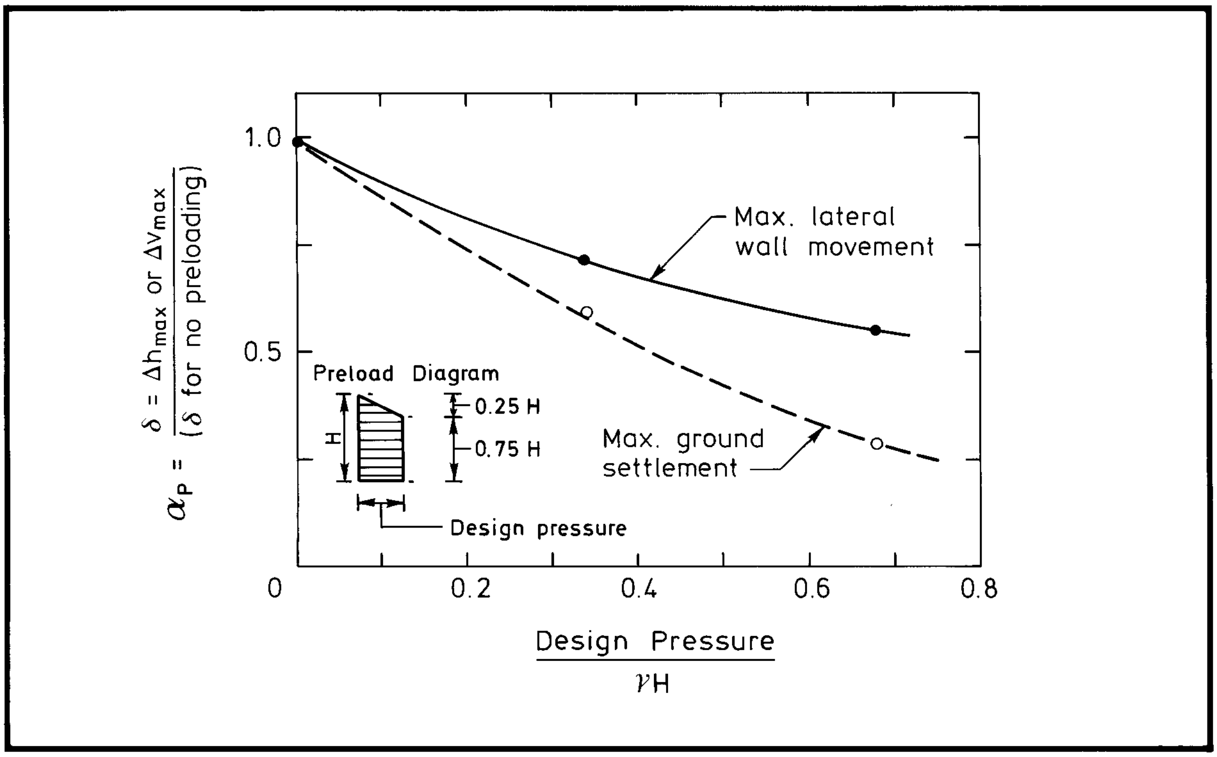


Figure 68 - Effect of Strut Preload on Maximum Lateral Wall Movement/Ground Settlement (Mana & Clough, 1981)

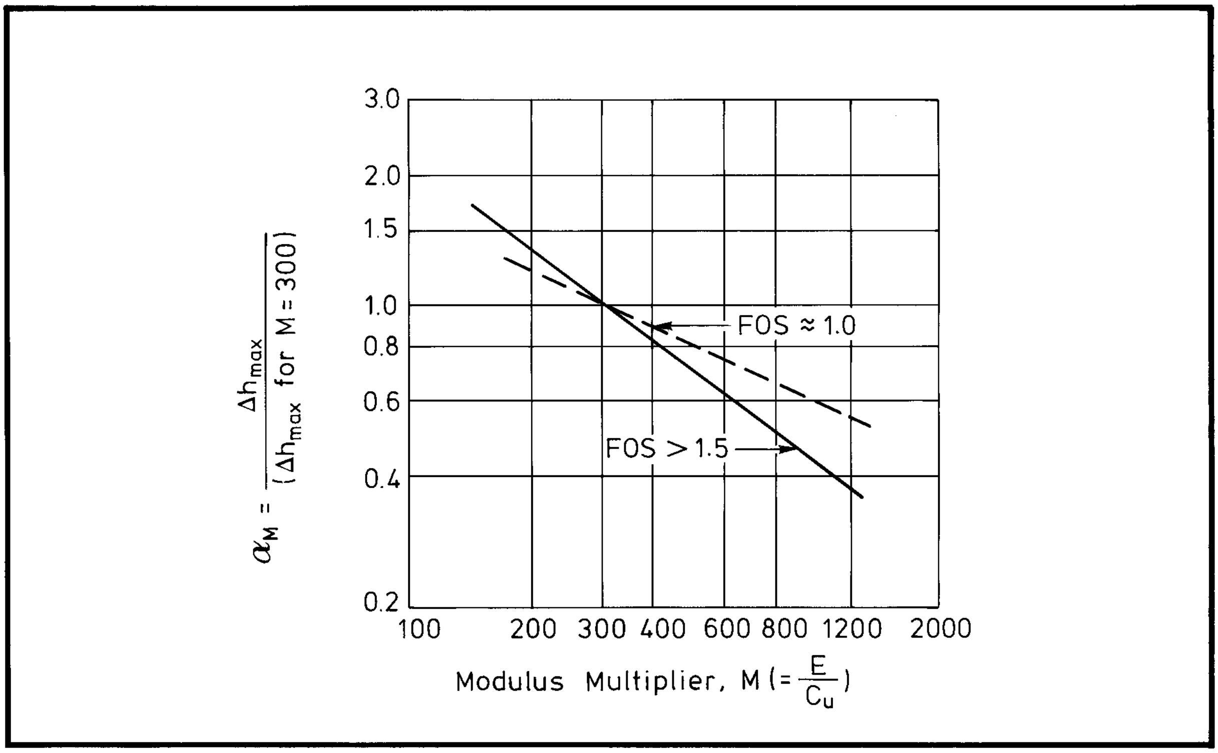


Figure 69 - Effect of Modulus Multiplier on Maximum Lateral Wall Movement (Mana & Clough, 1981)

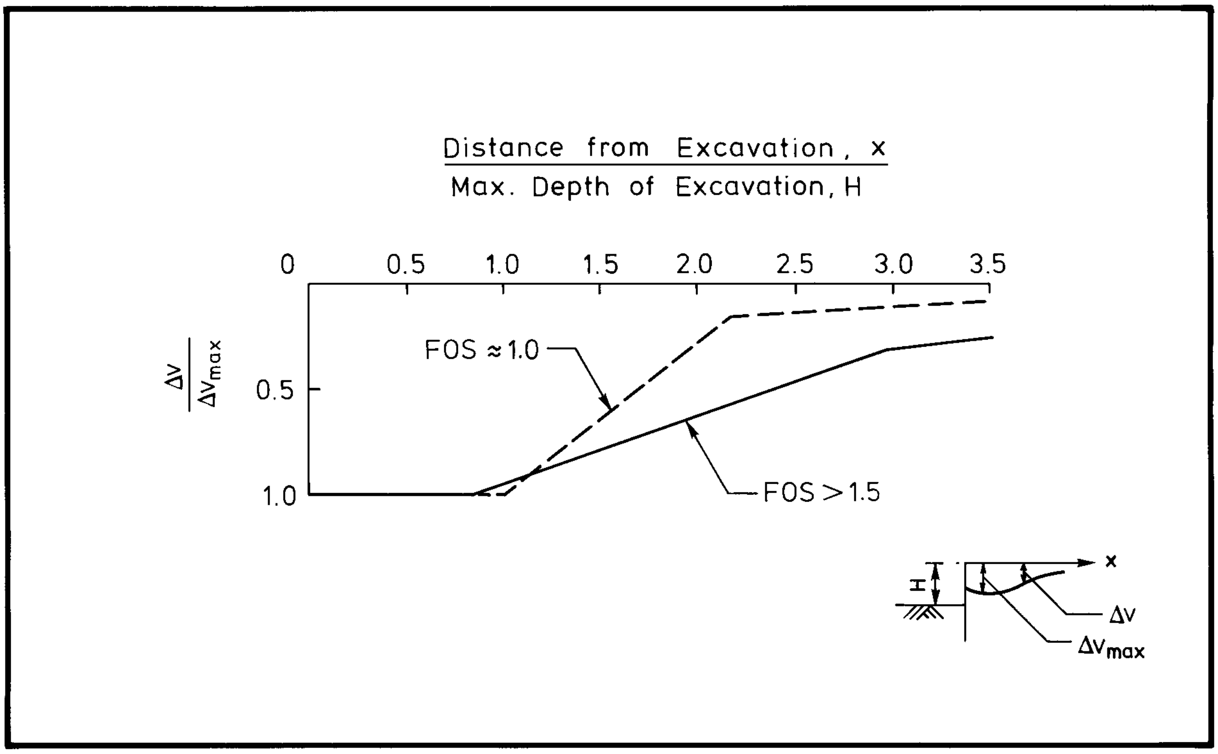


Figure 70 - Envelopes to Normalized Ground Settlement Profiles (Mana & Clough, 1981)