PROCEEDINGS

OF THE

SEMINAR ON

EARTH RETAINING STRUCTURES

MAY 27, 1989

COLOMBO - SRI LANKA

SRI LANKAN GEOTECHNICAL SOCIETY

SECRETARIAT: NATIONAL BUILDING RESEARCH ORGANISATION

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PREFACE

The Organising Committee appreciates the good response received from the invitees who were requested to present papers at this Seminar and wish to thank them for devoting their precious time to make the Seminar successful.

A total of nine papers have been received for presentation at this Seminar.

Discussions at the Seminar and the written discussions received till 20th June 1989 will be published in a separate volume.

The assistance given by Prof. A. Thurairajah and the members of the Organising Committee in organising the Seminar and the support given by the staff of the National Building Research Organisation in preparing this volume is deeply appreciated.

Kirthi S. Senanayake
Secretary
Organising Committee
SEMINAR ON EARTH RETAINING STRUCTURES

The statements and opinions expressed in the papers presented in this Volume are those of the authors and not necessarily those of the Organising Committee.

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PROVISIONAL STRUCTURES IN EXCAVATIONS

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1.0 INTRODUCTION

Construction activities are often associated with earthworks that involve making of embankments or excavations and cuttings in natural ground. Vertical faces or slopes thus formed, depending on the height and inclination and the soil conditions, may tend to collapse. In order to prevent such failure or to control excessive deformations in the ground it becomes necessary to provide supporting structures generally known as earth retaining structures.

Such earth retaining structures could be permanent structures e.g. retaining walls, bridge abutments, quay walls or diaphragm walls which are intended to be used on a long term basis as part or whole of the main structure. Or, they could be provisional structures erected temporarily to support the excavation or to provide dry work space during the execution of main construction work and therefore will remain until the main construction is completed. Provisional earth retaining structures are sometimes allowed to remain even after the completion of the main construction and, in this case, the role of provisional structure in earth retaining may be continuedly utilised or may be ignored totally or partly in the design of main structure.

In the planning of provisional earth retaining structures, approach can be made in two steps; in the first step a judgement is made on the suitability of the type of provisional structure and the method of its erection or construction. In the next step, several plans that are considered suitable are studied in detail and compared to arrive at the most practical and feasible type. Suitability of a provisional structure will be governed by the ground conditions and the environmental conditions within the site and its surroundings. The type of structure and the method of construction should be compatible with the ground conditions while the disturbances to the environment during and after the construction should be brought to a minimum. Safe and most rational type of provisional structure that satisfies the above conditions is then selected after comparative design. This design analysis should also deal with the apsects such as the procedure of erection of various components of the provisional structure and the stability of the excavated faces.

2.0 CLASSIFICATION OF PROVISIONAL EARTH RETAINING STRUCTURES

2.1 Major Types of Provisional Structures

Provisional structures that are required during execution of construction of a given main structure could be broadly classified into three groups as follows:

2.1.1 Provisional Structures for Earth Retaining

The main role of these structures is to prevent failure of the bottom of excavation and the surrounding ground. Major force the structure has to withstand is the earth pressure acting on its walls. These structures are sometimes refered to as timbering, although nowadays specially in the case of large excavation works, materials used in them are not limited to timber.

2.1.2 Provisional Structures for Waterfront Works

These are provisional structures erected in or adjacent to water bodies such as rivers, lakes or the sea in order to provide dry working area for the execution of main construction works. Major forces imposed on these structures are the hydrostatic pressures acting on the walls while wave forces could also be significant depending on the environment. In addition, earth pressures also need to be considered where excavation or dredging is required.

2.1.3 Provisional Structures for Earth-Water Retaining

These provisional structures possess the functions of both earth retaining and providing dry working area. This type may be required for excavations below ground water table or adjacent to water bodies. In provisional structures e.g.used for river works, major forces are the hydrostatic pressure acting on the wall on waterfront and the earth/ground water pressures acting on the wall on land side.

2.2 Main Aspects of Provisional Structures

2.2.1 Wall System

In all the above three types of provisional structures, the wall can be considered to be the most important structural member because it is the wall that plays the key role of withstanding earth pressures, hydrostatic pressures or a combination of these two. The type of wall is selected based on the strength required, water tightness, convenience of handling and installation, availablility of materials etc. Wall materials that are used in provisional earth/water retaining structures can vary from simple timber planks to steel or concrete sheet piles or to massive reinforced concrete diaphragm walls.

2.2.2 Method of Installation

Another major aspect need to be looked into in the construction of provisional structures is the method of installation of the walls. While manual installation of timbering is possible in shallow excavations, it may be necesary to use common machinery such as pile drivers or specialised and sophisticated equipment depending on the scale of the construction. Method of installation depends on the hardness of ground, type of wall material and specially the environmental conditions. Conventional methods of using drop hammers, diesel hammers or vibro hammers etc., for wall installations which cause environmental problems of noise and vibration are somewhat restricted in the populated areas and these are replaced by so called silent methods such as pre-boring or cast in situ wall techniques.

2.2.3 Method of Supporting

The third major aspect in provisional earth-water retaining structures is the method of supporting the wall erected. Sometimes it is not possible to freely support the walls depending on the magnitude of external forces of earth pressure and/or water pressure acting on them. In such instances, struts or anchors are used to push or pull the walls. Opposite walls of the structure or other suitable support within the excavation can be used to receive the reaction of the struts, while ground anchors or tie rods can be used to keep the walls in position. To support freely, the walls should be embedded into the ground adequately unless made stable as gravity structures as in the case of earth bunds or cellular cofferdams.

In a status-quo investigation of provisional structures used for excavation at 35 different construction sites within Tokyo Metropolitan area, Senanayake et al (1974) suggested their classification according to the combination of three elements viz. a) type of wall, b) method of installation and c) type of support (Fig. 1).

3.0 TYPES OF PROVISIONAL EARTH RETAINING STRUCTURES

3.1 Open Cut Excavations

Open cut excavations do not require any earth retaining walls and the cutting can be done vertically in stiff soils or by keeping a slope to suit the ground conditions. Critical depth of excavation is determined by the strength characteristics of the ground. When the depth of excavation exceeds 3m-4m, berms may become necessary to improve the stability. Working within open excavated areas is easy as there are no obstructions by structures. Compared to vertical cuts, slopes with berms take large space. Open cuts cannot be exposed for long periods where there is a tendancy for erosion or failure due to rains or high ground water table. Special care is needed in deep dewatering within open excavations.

3.2 Timbering

Timber planks of suitable thickness (35mm-50mm) and width (180mm-250mm) can be used either vertically as runners and poling boards or horizontally as laggings and sheeting for supporting vertical cuts. They can be arranged to make a continuous wall or can be placed at suitable spacings depending on the level of support required. Timber can be easily handled and fabricated to any required size at the site and moreover the initial cost of procurement is not high compared to steel or concrete. However, due to its low strength, use of timber may be usually limited to excavations less than 3m-4m deep. Moreover, as timber is easily subjected to damages and degradation, its reliability and durability are low. Self supporting of the wall is not advisable and often is not possible due to difficulties in driving timber boards into hard ground. Timber struts placed at suitable depth intervals are generally used with or without walings.

3.2.1 Timbering with Poling Boards

Excavations in stiff clays, soft rock strata and lateritic ground can usually stand unsupported for reasonable length of time thus simplifying procedures in timbering. However, to ensure safety of workmen against failures due to unexpected loads, vibration etc., and to prevent yielding or settlement in adjacent ground surface, some support would eventually be necessary. This support can be given by poling boards placed at suitable intervals (e.g. at 2m centres in open timbering for stiff clays and weak rock, at 1m centres in half timbering for fissured clays and compact cohesive sandy or gravelly soils or at 0.5m centres in quarter timbering for less stable materials) depending on the ground conditions (Fig.2). In very loose or weak ground or in dry sandy and gravelly soils it is advisable to provide a continuous wall with close timbering.

Poling boards are usually 1.2m-1.5m in length and therefore deeper excavations may require several settings of boards as in middle board system with waling at the middle of the board or in tucking frame system with walings at the ends of the board (Fig.3).

3.2.2 Timbering with Runners

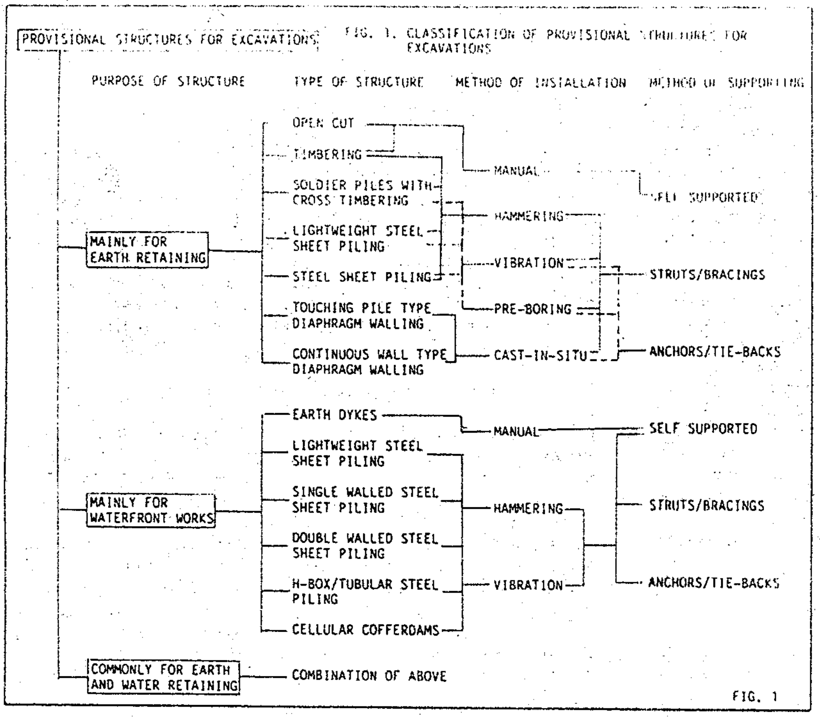
In timbering deeper excavations, timber runners, usually available in cross sections of 180mmx35mm - 250mmx50mm and length upto 5m are driven ahead of excavation and these will be supported by a system of walings and struts (Fig.4). For deeper excavations a second and third setting of timbering can be provided depending on the depth to be reached. However, the width of excavation gradually become narrower with the increased settings (Fig.5).

3.2.3 Timber Sheet Piles

Heavy tongue and groove timber sheet piles (100mm - 300mm thick, about 250mm wide and seldom available in lengths exceeding 12m - 15m) are used in provisional structures for deep excavations. Owing to construction problems encountered in driving and pulling out and, as large timber sections have become scarce, timber sheet piles are replaced by more convenient steel sections.

3.2.4 Soldier Piles with Horizontal Timbering

In this type of provisional structures the ground is supported by placing horizontal laggings or walings with vertical poling boards behind or between soldier piles driven in advance of excavation (Fig.6). Timber can be used for the soldier piles in shallow excavations but steel H-sections are commonly



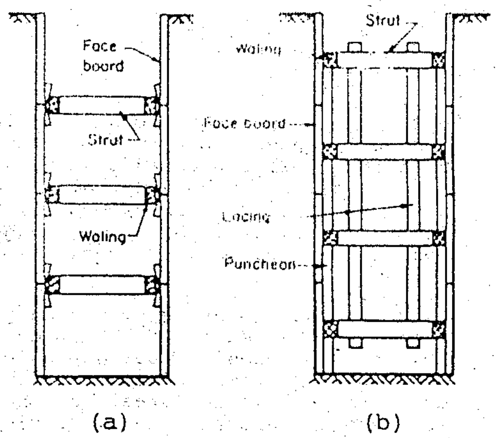


Fig. 3 Deep excavation with (a) tucking frame and (b) middle board systems

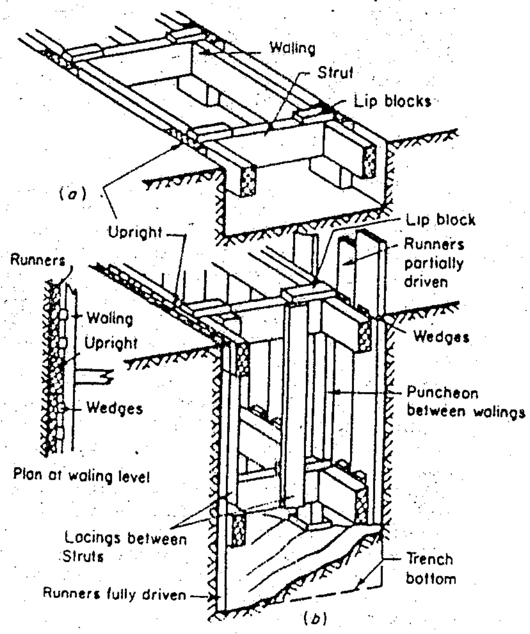


Fig. 4 Timbering with runners in a) first, b) second stages of excavation

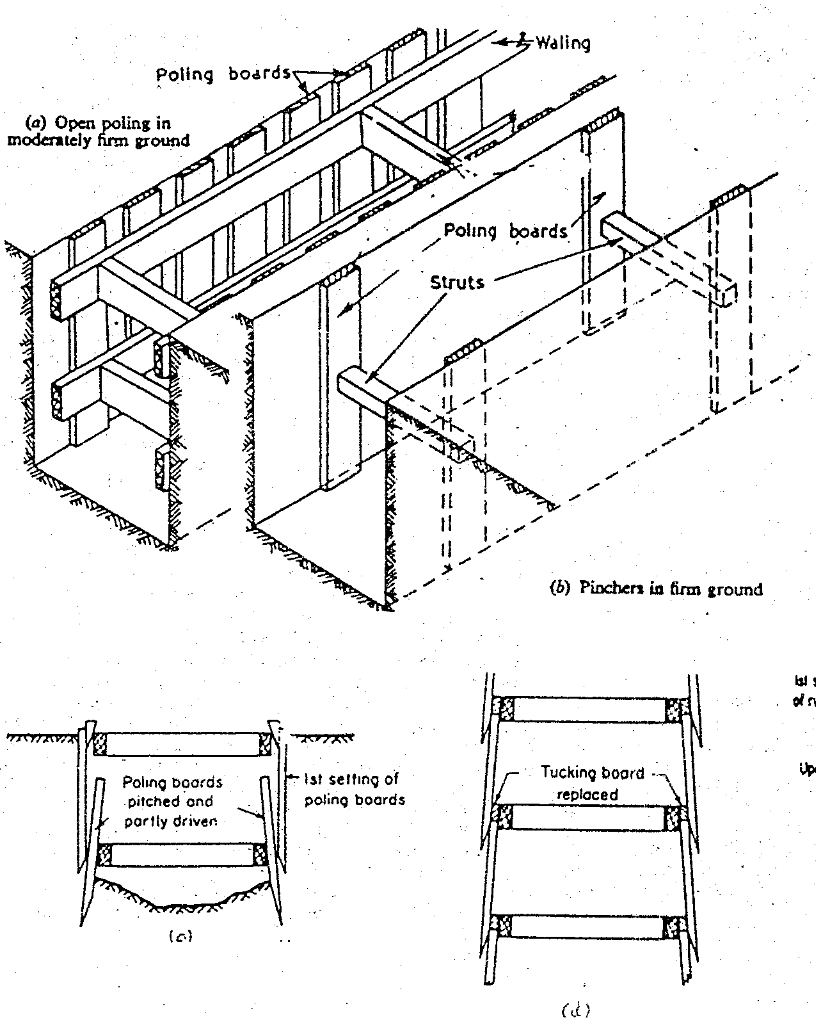


Fig. 2 Poling boards in shallow (a & b) and deep (c & d) excavations

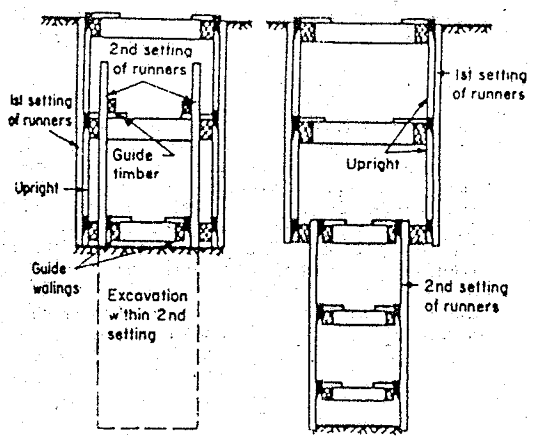


Fig. 5 Deep excavation with runners

used in deep excavations. Soldier piles are generally spaced at 0.8m to 3m centres and will be either self supported or otherwise supported by bracings or anchors depending on the scale of excavation and the arrangement of the work area. In shallow excavations timber runners or poling boards can be used in combination with short walings.

Soldier pile timbering system has the advantages such as the ease of erection and rémoval, ability to reach greater depths of excavation, adoptability where underground utilities etc. cross the excavation area. As the soldier piles are spaced apart, their driving cause less nuisance of noise and vibration than e.g. in the driving of steel sheet piles in continuous walls. As it is necessary to expose the ground at excavation level over a depth of two to three laggings for the installation of these horizontal boards, precautions are necessary against seepage that could lead to erosion of the soil. This system is not suitable where excavation bottom is liable to heaving or where ground water table is high in permeable soils.

3.3 Steel Sheet Piling

Steel sheet piling is a very common type of provisional structure because of following advantages; Steel sheet piles can resist high driving stresses developed in hard ground. Being reliable and durable they are economical and reusable. Pile length can be adjusted during installation by cutting or welding. As the piles are supplied in a variety of sections (Fig.7) from light weight to heavy duty, selection is easy. With suitable interlocking joints between adjacent piles, they can provide almost water-tight continuous walls. Steel sheet piles can be used as part of the main structure after they serve in the provisional structure. However, the genrally high initial investment on steel sheet piles is a disadvantage in addition to the construction difficulties and the noise and vibration nuisance caused in driving thorugh hard ground. Continuous sheet pile walls also pose a problem where underground utilities cross the work area.

With sufficient embedment into the ground, sheet pile walls can be self supported in excavations of limited depth. As the depth of excavation increases larger sections could be used for cantilevered sheet piling, or the walls can be supported with bracings or anchors installed at suitable depth intervals to minimise usually high deflections experienced in sheet piling.

3.3.1 Trench Piles

Light weight, simple shallow webbed or corrugated steel sheet piles (Fig7a) can be used in shallow excavations in the same manner as timber runners installed in open or close timbering. Shorter lengths can be easily handled manually and has a greater reliability than timber planks. Driving through stiff to hard ground is difficult and closer bracings are required as deflections could be high. Watertightness is not generally good but depends on the type of interlocking joint.

3.3.2 Deep Web Sheet Piles

Deep web sheet piles (Fig.7b) are required when it is necessary to resist large lateral pressures of water and/or soil either as cantilevered, braced or anchored vertical beams. U-shape or Z-shape sheet piles are the common types used in such situations. Two rows of sheet piling anchored together are sometimes used with the space in between filled with less permeable soil for improving the watertightness and stability.

3.3.3 Large-size Steel Sheet Piles

Where the work space is restricted and the wall should be as narrow as possible, stronger H-piles or tubular piles (Fig.7c) with interlocking joints can be used in place of deep web type sheet piles. Cement mortar is sometimes injected into the box or circular section to increase the stiffness of the wall. Although these piles can be reused, considering the very high cost of withdrawal, this type of sheet piles are more suited where the provisional structure can be effectively used in the main structure.

3.3.4 Cellular Cofferdams

When excavation has to be carried out in a large area overlain by water, cellular cofferdams are used to provide a dry work area e.g. in the construction of dams or waterfront structures. Several circular or diaphragm type cells are connected together along the perimeter of the excavation area (Fig. 8). Stability of these cells and hence the stability of cellular cofferdam is achieved by the gravity of soil mass filled in a cell enclosed by sheet piles, whereas the ring or membrane tensile stresses in the sheet piling effect a soil container. As the cofferdam behaves as a gravity structure no bracings or external anchorage would be necessary thus providing an obstruction free work area. Moreover, the cofferdam wall can also serve as an access road or working space. Geometry of circular cells has an advantage in stability over the diaphragm type where a series of circular arcs are connected with cross walls known as diaphragms made up of sheet piling.

3.4 Concrete Diaphragm walls

Environmental problems of noise and vibration associated with the conventional types of provisional structures in which driving of sheet piles or soldier piles are involved, specially in populated and built up areas, have lead way to the development of diaphragm walls as a suitable alternative in deep excavations. Diaphragm walls are cast-in-situ continuous walls constructed either as a row of touching piles or as a series of abutting rectangular wall segments. As the thickness of finished wall usually range from 300mm to about 600mm, a relatively high stiffness can be achieved from these reinforced concrete walls and are therefore suited for deep excavations. Recent developments in construction techniques have made it possible to construct diaphragm walls of

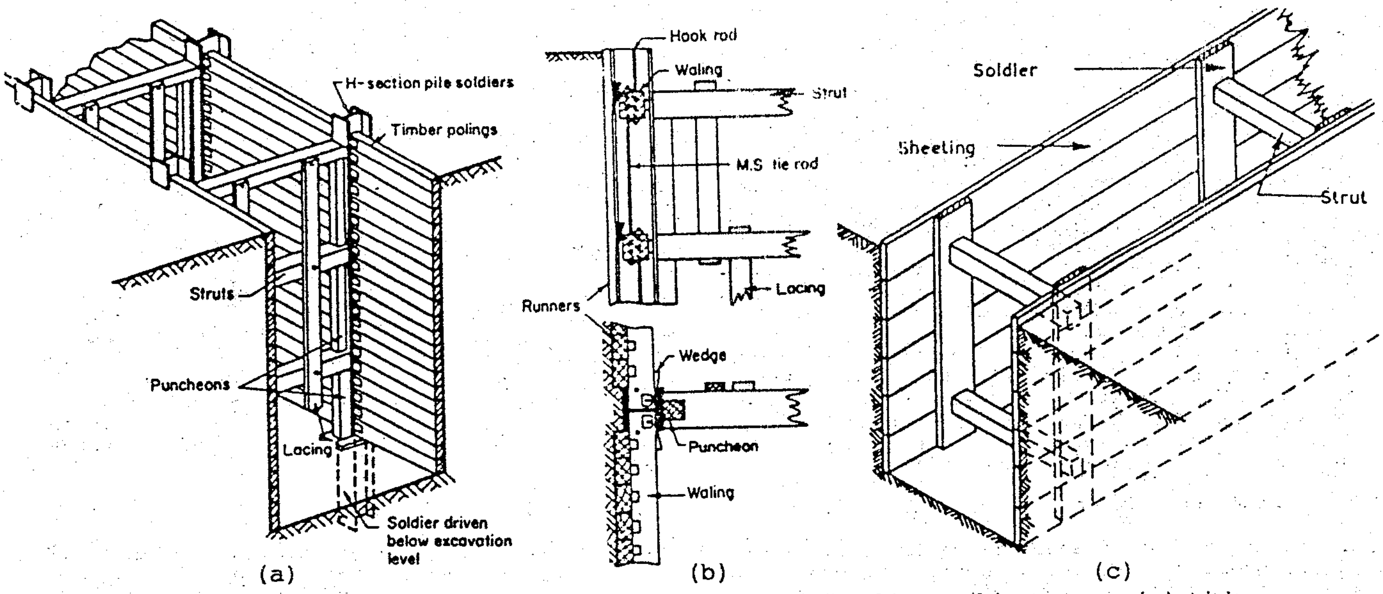


Fig. 6 Soldier piles with cross timbering H-sections with (a) polings, (b) runners, (c) timber soldiers with sheetings

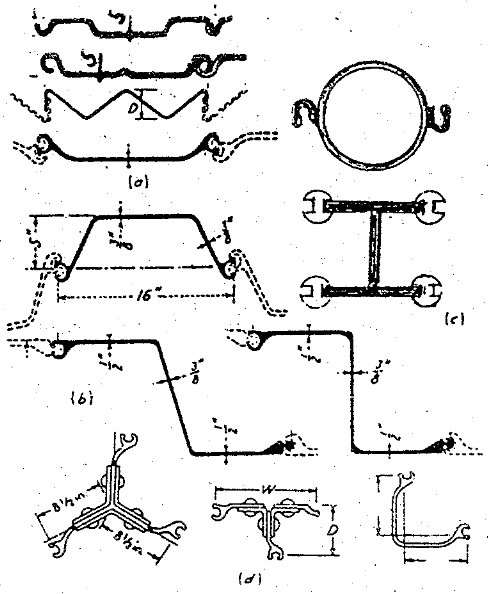


Fig. 7 Typical steel sheet piles
(a) lightweight/shallow webbed,
(b) deep webbed & (c) H-box and
tabular steel piles
(d) interlocking joints

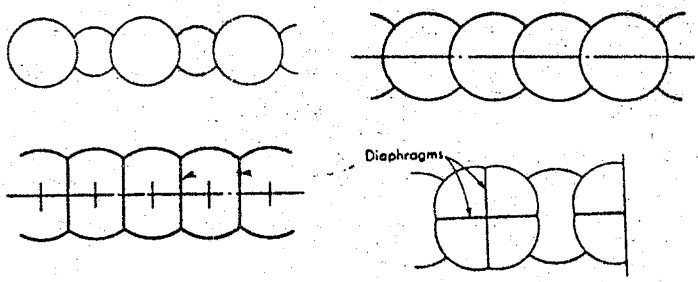


Fig. 8 Cellular cofferdams
(a) multiple circular cell type
(b) diaphragm type (c) cloverleaf type

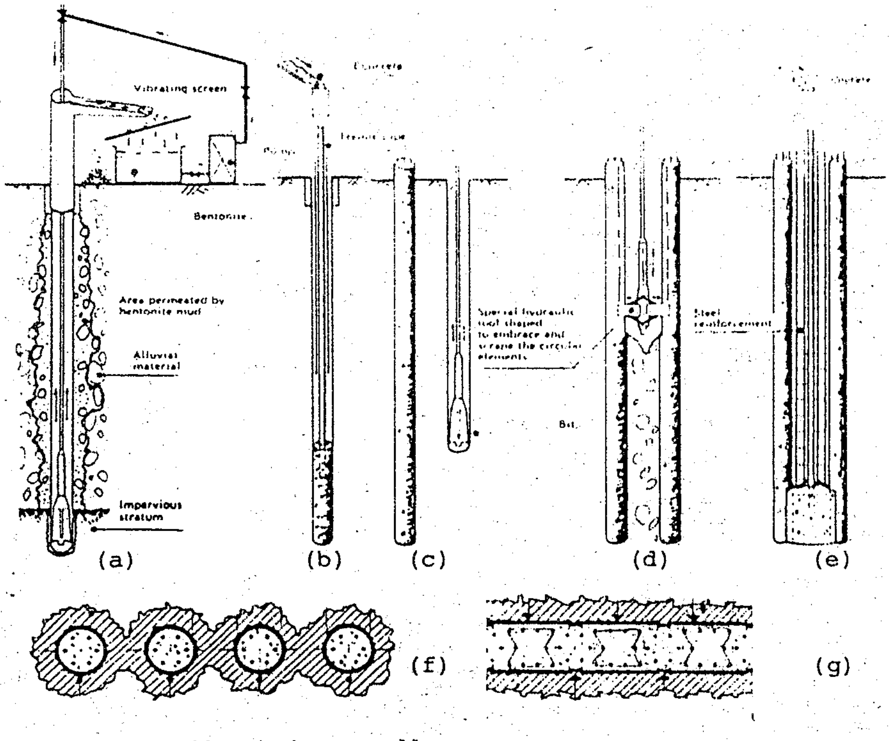


Fig. 10 Diaphragm walls
(a)-(c) drilling primary holes and casting elemants
(d)-(e) drilling and casting of secondary elements
(f) primary element cast (g) secondary element cast

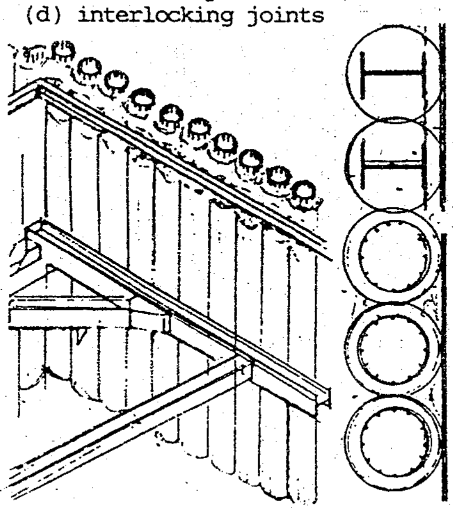


Fig. 9 Touching pile type diaphragm walls

high quality reliably and rather easily. However, 3 to 4 weeks required for achieving the desired design strength of concrete, high cost of construction, inability to remove and reuse etc. are some major disadvantages of this type. Therefore, diaphragm walls are most suited for where they can be continuedly utilised as part of the main structure. Since seepage through joints can be totally eleminated by suitably grouting, these walls can be used where watertight enclosures are required. However, these continuous walls are not suited where underground utilities exist. In deep excavations, when diaphragm walls cannot be self supported, bracings or anchors can be used as in the case of timbering or sheet piling.

3.4.1 Touching-Piles type Diaphragm Walls

In this type of diaphragm walls cylindrical piles, usually constructed in the same manner as the cast-in-situ concrete piles, are installed in a row so as to overlap or touch each other (Fig. 9). Different techniques using e.g. Benoto, earth drill, earth auger, rotary boring and percussion equipment are used in the casting of piles. After excavation of the borehole, reinforcement cages or steel sections are placed within the borehole and concreted. Where the strength of wall is not very important, piles, can be cast without any reinforcement by jet grouting or cement mixing in to the soil to form a soil-cement wall. To make the touching pile walls watertight, the joints or the gaps betwwen adjacent piles are sealed either by grouting or by casting additional piles to overlap. Touching piles have the advantage of casting several piles at the same time as the space required for the installation equipment is not very large.

3.4.2 Wall Type Diaphragm Walls

In this type, trenches are excavated to the required depth, using clam shell or special large size grabs, along the periphery of the excavation guided by pre-driven guide holes (Fig. 10). Collapse of trench walls during excavation is prevented by circulation of bentonite slurry. Rectangular wall elements are then either cast-in-situ or precast elements are placed within the trench. Integrity of the wall is achieved by grouting the joints of precast elements or by casting alternate elements to overlap. Unreinforced diaphragm walls are constructed by in-situ mixing of soil with cement or other grouting chemicals.

4.0 METHODS OF INSTALLATION OF RETAINING WALLS IN PROVISIONAL STRUCTURES FOR EXCAVATION

4.1 Classification of Methods

In a broad classification, the common methods used for the installation of walls in provisional structures can be divided into those executed manually and those operated using plant and equipment of varying scale.

4.2 Manual Methods

The manual methods can be e.g. construction of simple earth bunds or berms, placing of sand bags or gabions, setting of timbering in shallow excavations or in installing timber boards between soldier piles even at deeper elevations. Manual methods are favourable where space is limited for introducing machinery or where labour is cheap. In most circumstances, the efficiency is small and generally not suitable for large works.

4.3 Installation with Plant and Equipment

Plant and equipment are generally required for large works and the type of machinery depends on the type of wall to be installed. These include diesel drop hammers, vibro hammers or augering and press/push driving equipment for the installation of sheet piles etc., augering and drilling machines or grouting and excavating equipment for the installation of diaphragm walls.

4.3.1 Simple Drop Hammer

Drop hammers operated manually or mechanically are used in driving timber runners or sheet piles in shallow excavations. Limited space required, simplicity in operation and movement, low cost of initial investment etc. are some of the advantages.

4.3.2 Diesel Hammer

Diesel drop hammers are widely used for the driving of sheet piles and soldier piles. As the fuel is freely available and since driving can be done even in relatively hard ground diesel hammers can be used in any area. Availability of equipment in a wide range and the high efficiency are added advantages. However, nuisance due to noise and vibration is high although this can be reduced to some extent using special guards. Type of equipment should be selected after carefully considering the type of pile to be driven and the ground condition as the efficiency could decrease largely by wrong selection.

4.3.2 Vibro Hammer

Compared to diesel hammers, electrically or compressed air operated vibro hammers can not only drive but also can pull out piles efficiently through most soil types except in hard ground. Though noise and vibration levels are somewhat smaller than in diesel hammers continued operations for long durations cause serious nuisance. Auxiliary equipment such as air compressors, power generators are necessary.

4.3.3 Augering Equipment

Large diameter augering or boring equipment are used for pre-boring holes in the ground for installing piles by less noisy methods such as press-driving in an attempt to minimise noise and vibration problems. Piles installed in pre-bored holes are anchored to the ground by gently driving through some depth by drop hammer or vibro hammer and then grouting any cavities. Although this method is applicable to most ground conditions including relatively hard strata, cost and time required for construction become high due to the extra step of pre-boring operation. With the environmental regulations become strict, special equipment have been developed to curtail noise and vibration transmitted by pile driving. In populated areas, equipment which can press the piles into ground while the ground ahead is loosened, by the same equipment, by augering or water jetting are also used.

4.3.4 Cast-in-situ Installations

Walls are cast-in-situ with concrete or by grouting as in touching pile type or continuous type diaphragm walls. This method is suitable where ground conditions are poor and the noise and vibration should be controlled. Cost is very high and large working space is required for setting up of plant and equipment. A large number of methods and equipment are available for casting diaphragm walls by either method.

5.0 SUPPORTING SYSTEM

5.1 Methods of Supporting the Walls

The walls in provisional structures can be self supported, braced or anchored depending on the type of structure, scale of excavation and construction conditions.

5.2 Self Supported Walls

Cut slopes, earth bunds, sand bags or gabion walls, timbering, soldier piles, sheet piles or diaphragm walls sufficiently embedded in to the ground and the gravity type structures such as cellular cofferdams represent walls of provisional structures that could be self supported. Advantages of self supporting the walls are the availability of working space within the excavation area unobstructed by bracings etc., room for increased efficiency when using machinery and the time and cost savings in construction as the erection of the provisional structure could be done speedily. However, the sections and stiffness of the wall should be larger when compared to supported walls. Depth of excavation will be limited and moreover the wall need to be embedded deeper into the ground for anchoring than in the case of supported walls.

5.3 Struts and Bracings

These are supports given from the inner side of the excavation to withstand earth and/or water pressures acting on the walls. Type of support can vary from earth counterberms, shorings and struts to rater complicated bracing systems. The latter three types can be installed at one or several elevations depending on the scale of excavation. Order in which excavation and installation of supports is done and the position of support largely influence the design and cost of a provisional structure. To minimise time and cost involved in setting bracings, prefabricated bracing systems have been developed which can be easily mounted using jacking arrangements (Fig.11). As the supports in this type are erected within the excavation, no additional space is required. Bracings are reliable when properly designed and constructed. However, if the struts and other supporting members are not spaced suitably, efficiency of mechanised excavation and construction works will decrease.

5.4 Anchors and Tie Backs

Anchors or tie backs (Fig.12) can be used in excavation sites where space is limited for supporting otherwise with bracings. These supports can be either earth anchors or ties anchored on to piles or other suitable structures installed outside the work area. Efficiency of construction work, as a whole, would be high as machine operations within the work area are not obstructed. In installing earth anchors, care is needed to ensure that they donot intrude into other lands. Earth anchors may not suit where buried utilities, old wells or cavities exist. High level of quality control is necessary to improve the reliability of earth anchors. Type of soil and specially the presence of ground water affects the quality of these anchors.

High strength steel rods are used as tie rods for anchoring the walls. These are more suitable where the anchoring has to be done closer to the ground surface. In double walled sheet pile cofferdams the walls are anchored to each other to withstand pressures from the infilled soil, but this type remains a self supported provisional structure for the purpose of classification.

6.0 SELECTION OF PROVISIONAL STRUCTURES

6.1 General Concepts in Planning

Provisional structures are intended to be temporarily used and are usually removed after the completion of the main construction. Therefore, with comparison to the permamnent structures, some allowance for larger allowable stresses, deflections and deformations can be permitted in the structural members and also a high construction accuracy may not be required. However, when the construction work is large in scale and become complicated, work load, cost and time requirements of the provisional structure will cover a significant portion of the entire construction work. Depending on the construction environment e.g. when work has to be done in poor ground or adjacent to existing structures, over-emphasis of economy may lead to disasters which could be more embarrasing social and economical problems. Therefore, planning and design of provisional structures should be done with a very good understanding of the conditions of the work, site and its environment and giving sufficient attention to the importance of the provisional structure.

6.1.1 Basic Conditions for Consideraton

The basic conditions that need to be considered in the selection of a provisional structure can be classified into two groups, viz. construction conditions at site and environmental conditions. These include; necessity and importance of a provisional structure and its relationship and compatibility with main construction, topography and ground conditions at the site, scale of work, depth and width of excavation, duration of use of the provisional structure, work space, possibilities of mechanising construction operations and limitations to site activities, type of machines that could be used, procurement and handling of materials, environment and environmental restrictions, noise, vibration and distress to buildings and other structures, obstruction to traffic and communication, drying of wells etc., due to construction activities.

It appears possible to arrive at the best suited provisional structure by combining the most suitable systems of wall structure, installation procedure and supporting method for the given conditions. However, each system has many advantages and disadvantages and therefore selection of a structure that satisfies most or all of these conditions is practically not possible. In practice, it is seen that most provisional structures are chosen giving priority to safety, speed, economy and environmental protection.

6.2 Preliminary Investigations

In order to collect information required in the planning, design and construction of provisional structures, prior investigations should be carried out. Failure to do so may result in unanticipated problems of construction delays, damage to life and property etc.

6.2.1 Topography

Topography of the site and surrounding area should be studied to identify top soil type, undulations and slopes in land, unstable areas, and in waterfront works, river bed slope and obstacles, and to assess the flow velocities, wave forces etc.

6.2.2 Subsoil Investigations

Subsoil investigations are generally done at the design stage of the main structure, but usually with the objective of determining the foundation type and bearing capacity of the ground. As a result, the ground above excavation level is hardly investigated in detail if not neglected totally. For satisfactory planning and design of provisional structures, especially of a large scale, the physical, chemical and mecanical properties of subsoil should be examined so that these parameters will help in assessing the following:- Earth pressures, ground water pressures, stability against heaving, boiling and uplifting of excavation bottom, stability of side walls of excavation, method of excavation, seepage flow, dewatering requirements, extent of lowering of ground water table and ground subsidence in the vicinity, bearing capacity for intermediate (King) piles in bracings, construction of platforms and temporary roads etc., and necessity of ground improvement.

6.2.3 Adjacent Structures

When existing structures are situated close to the excavation, it is necessary to study the foundation type and the subsoil conditions below such structures and the mutual relationship or influence between existing and provisional structure, order of any settlements during lowering of water table. Where the existing structures are liable to distress, precautionary measures shall be taken by under-pinning or by other methods of strengthening. Possibilities of preventing damage during lowering of ground water table for eg: by recharging the ground, should also be considered.

6.2.4 Underground Utilities

Pipes for water supply, sewerage, gas etc., and cables for power supply and telecommunication are among underground utilities that may exist at the construction site. It is necessary to study the exact location, size, soil cover, degree of ageing etc., of such utilities and other underground structures in order to take suitable precautionary measures for their protection unless relocation is possible and necessary. When detailed and updated information cannot be obtained from the relevant authorities or when records of any past relocationing are not available, it would be necessary to probe or drive test pits for examination.

6.2.5 Land Use of the Environment

In the course of installing provisional structures and in executing other related construction works, nuisance to the neighbourhood, caused by noise, vibration, dust and mud & earth disposal etc., can be very significant. Therefore, suitable measures for environment protection should be considered. Where work cannot be carried out due to strict environmental regulations, it is necessary to introduce non-offensive construction equipment and techniques.

6.2.6 Work Schedule

Provisional structures are erected generally well ahead of the main structures. However, when changes are made in the work schedule it is necessary to confirm on regulations and conditions pertaining to the use of premises belonging to other authorities eg: use of river bed or roads etc., during to certain periods of time.

6.3 Design of Provisional Structures

Behaviour of provisional structures becomes complex with the progress of excavation. As the deflections and deformations in provisional structures for shallow excavations in good ground do not pose major problems to the safety, simple design methods would be adequate. However, in the design of deep excavations, specially in poor ground, it is necessary to consider the displacement patterns of the wall and the changes in ground behaviour from elastic to plastic. In other words, actual behaviour of the system should be taken into account in the design and the design conditions should not be ignored at the stage of construction.

Some important aspects that need to be studied in the design are:

1. Influence of excavation on the surrounding ground.
2. External forces, e.g. distribution of earth and hydrostatic pressures etc.,
3. Stability of the bottom of excavation that depends on the depth of excavation and depth of embedment of the wall etc.,
4. Stability against heaving and uplifting in clayey ground or against boiling in sandy ground.
5. Stresses and deformation of structural components such as wall, struts, walings, anchors, tie-rods etc.,

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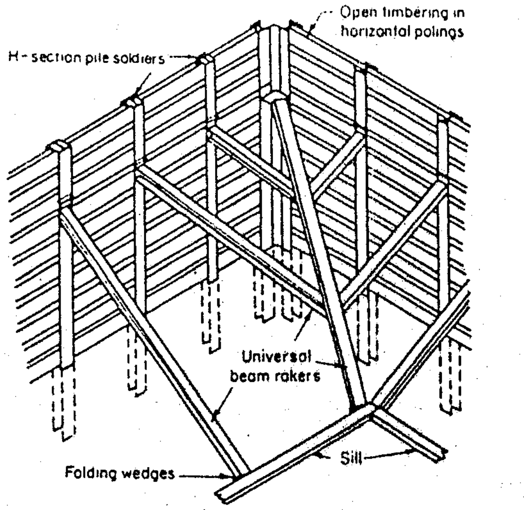

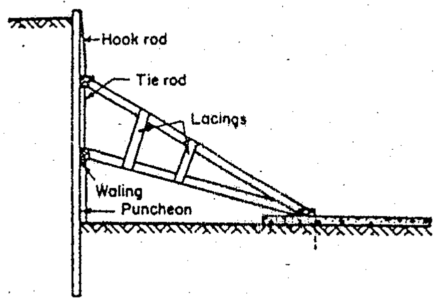

1. Waling, 2. Strut, 3. Bracing 4. Corner cover plate
2. Bracing support, 6. Waling cover plate
7. Strut cover plate, 8. Screw jack
9. U-bolt cross joint for struts
10. Strut bracket 11. Wailing bracket
12. Jack guard
13. Strut stiffener

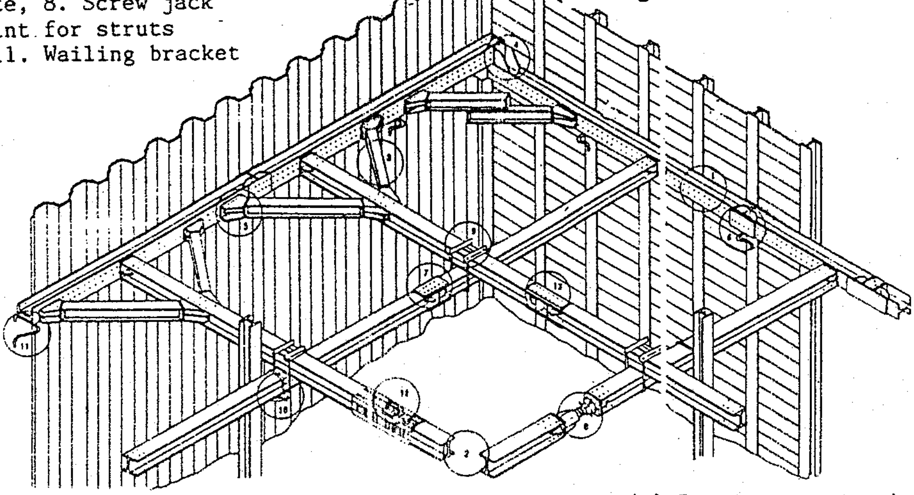

Fig.11 Shorings and Bracings for support of excavation (a) Shoring (b) Bracing system with prefabricated elements

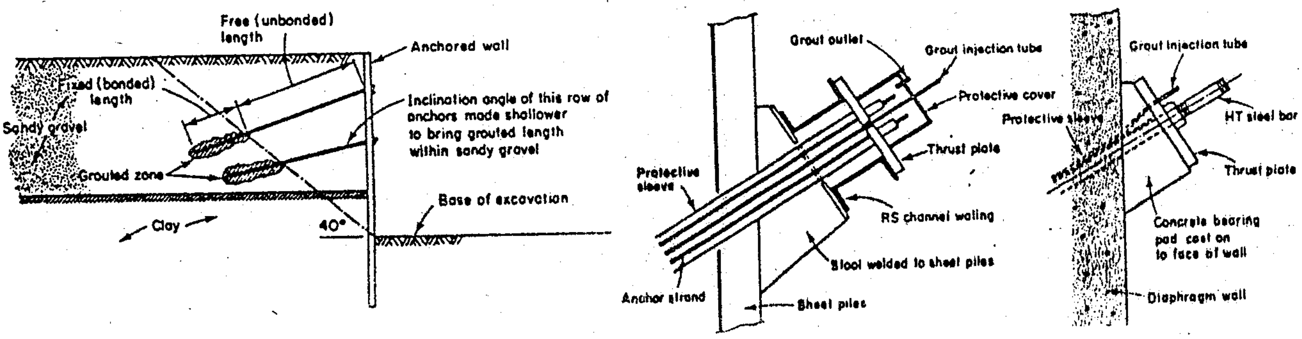

Fig.12 Ground anchors for support of excavation (a) Cable anchors in steel piling (b) Bar anchors in diaphragam walls

DESIGN OF FLEXIBLE WALLS

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1.0 INTRODUCTION

Flexible retaining walls are a very ancient form of construction used for foundation construction in water. FUGL-MEYER (1937) describes the Chinese bridge builders of the Han dynasty (206 BC - AD 221) constructing the piers of masonry bridges in circular cofferdams which consisted of double rows of bamboo piles sheeted with bamboo matting and with a clay infilling.

Today, flexible retaining walls usually consist of a single row of sheet piles with their lower ends embedded in soil. These sheet piles consist of special shapes of interlocking piles which are driven in close contact with each other to form a continuous wall or sheet. They may be fabricated from timber, steel or reinforced concrete. Additional support can be provided, if necessary, by anchor rods in which case the structure is called an anchored sheet pile. If anchor rods are not used, the structure is referred to as a cantilevered sheet pile.

The main uses of sheet piling are to

(i) resist lateral pressures caused by combinations of earth, water, and applied horizontal and vertical loads;

and/or

(ii) prevent or reduce the leakage of water into an excavation.

2.0 DESIGN PHILOSOPHY

Unlike in the case of gravity walls, flexible retaining walls carry load by bending. Therefore, in the design of anchored sheet pile walls, it is necessary to determine

(i) the depth to which sheet piles should be driven to ensure adequate lateral support;

(ii) the magnitude of the force which acts on the anchor; and

(iii) the maximum bending moment acting on the sheet pile.

3.0 EARTH PRESSURE DISTRIBUTION

As in the design of any earth retaining structure, the first step in design is to compute the earth pressure acting on the wall. The flexibilty of these structures prevent their rational analysis by procedures used for rigid retaining walls. Simplifications and assumptions concerning soil behaviour and structure deformation conditions have been made to allow efficient analysis of these structures.

Three types of approaches have been used for the study of lateral earth pressure distribution. These are

(i) methods of limit analysis;

(ii) semi-empirical methods of analysis;and

(iii) methods of elastic analysis.

3.1 Methods of limit analysis

These are the most common methods of analysis. The earth pressure distribution is determined at limit state assuming that the soil is about to fail. In these methods of analysis, the wall deformation is not considered. The classical methods of COULOMB (1776) and RANKINE (1857); the method of characteristics of SOKOLOVSKII (1965); are all examples of this method of analysis.

The Coulomb method and the Rankine method give hydrostatic type pressure distributions for both active and passive earth pressure for smooth vertical walls. Therefore, in many design offices, for convenience, hydrostatic pressure distributions are normally assumed for the active pressure behind the sheet pile wall and for the passive pressure below the dredge line.

3.2 Semi-empirical methods of analysis

In these methods, the stress distributions are first determined on the basis of limit analysis. Subsequently empirical correction factors are introduced to take into account observed behaviour. For example, the active pressure behind a retaining wall may be much higher than the pressure determined on the basis of limit analysis if the wall is prevented from yielding. Then an empirical correction factor may be introduced. However, these empirical corrections are those developed by practicing engineers based on their experience and may not be generally applicable.

3.3 Methods of elastic analysis

Soil structure interaction is an inevitable mechanism that occurs in flexible walls; i.e. the distribution of earth pressure acting on the wall depends on the geometry and yield of support, and on the relative stiffness of the pile and the soil. This can be simply illustrated with reference to anchor yield. Consider the example of a sheet pile where the anchor yields sufficiently to allow active pressure to develop in the backfill. Then the distribution of earth pressure may be as shown in Fig.1a.

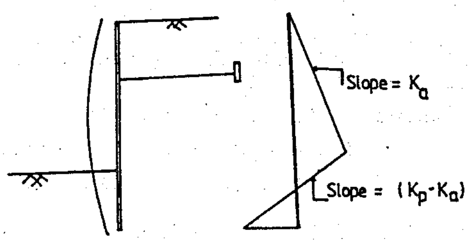


Fig. 1a - Earth pressure distribution with anchor yield

On the other hand, if the anchor is unyielding, then the distribution of earth pressure may change to that shown in Fig.1b.

Methods of elastic analysis can be used to study soil structure interaction. Two main categories exist :

(i) the finite element method; POTTS (1987)

(ii) the coefficient of subgrade reaction method. Examples of these are
   (a) Rowe's method where influence factors have been introduced to take into account pile flexibility, soil stiffness and anchor yield - ROWE (1952); and
   (b) the matrix method of analysis where the elastic beam method of solution is used - BOWLES (1974).

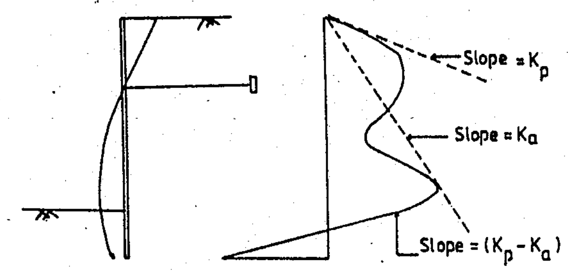


Fig. 1b - Earth pressure distribution without anchor yield

4.0 DEPTH OF PENETRATION

For deformation below dredge line, TERZAGHI (1943) identified two types of deformation conditions based on the depth of penetration. If the sheet piles are driven to shallow depths, then they are said to have 'free earth support'. On the other hand, if the sheet piles are driven to a considerable depth, they are said to have 'fixed earth support' as the lower end is more or less fixed in position.

4.1 Free earth support

Assuming adequate anchorage, sheet piles with 'free earth support' can fail either by bending of the piles or the sliding of the soil near the base of the pile. The deformations and stress distribution are as shown in Figs. 1a and 1b. As the depth of driving is relatively small, too much reliance cannot be placed on the development of the full passive resistance, and a factor of safety must be used on the available passive resistance. For example, when limit analysis is used, it is preferable to work with a reduced friction angle ϕ_R for the passive side only, where ϕ_R is given by

$$\tan \phi_R = \frac{\tan \phi}{F}$$

with F being a suitable factor of safety; say 1.5.

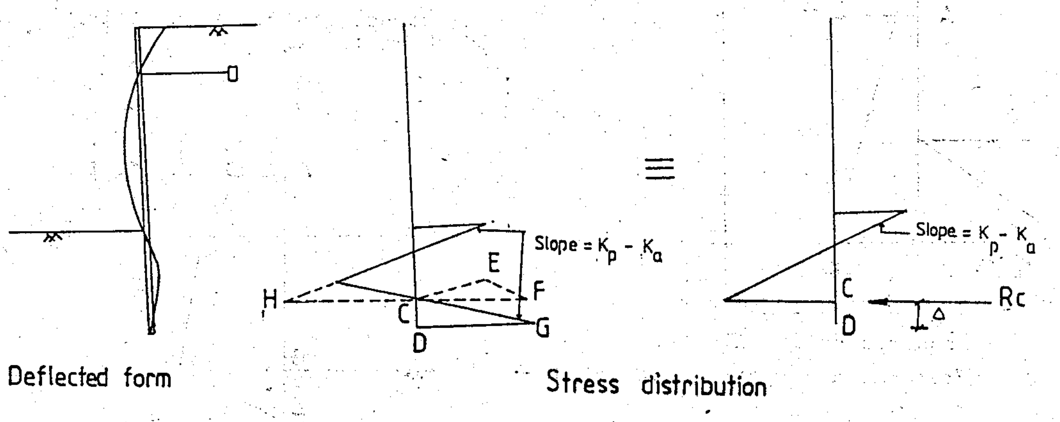


Fig.2 - Deflected form and stress distribution below dredge level for Fixed earth support condition

4.2 Fixed earth support

In this case, as the depth of driving is relatively large, when adequate anchorage is provided, the piles can fail only by bending. Hence, full passive resistance can be assumed for design. The usually assumed stress distribution (in limit analysis) on the length of pile beneath the dredge level is given in Fig.2.

Referring to Fig.2, it should be noted that the resultant force due to the two stress distributions CEF and CDG are given by a single resultant force R_c at C.

It has been shown that the length CD (i.e. Δ) can be determined from the equation

$$\Delta = \frac{0.45\, R_c}{\sigma_c}$$

where $\sigma_c = \overline{CH}$ = assumed equivalent stress at C.

4.3 Recommendations of the Codes of Practice

The British Code CP 2 (1951) on Earth Retaining Structures recommends that whereever possible to design sheet pile walls for the 'fixed earth support' condition because if the 'free earth support' condition is used it leads to large bending moments and results in uneconomic sections. However, when it is not practicable to drive the piles to a sufficient depth, then there is no alternative but to design for the 'free earth support' condition.

Similarly, in many European countries, the 'fixed earth support' condition is recommended. However, in USA the 'free earth support' condition is preferred.

5.0 SIMPLE DESIGN METHOD IGNORING SOIL STRUCTURE INTERACTION

Four cases have to be considered, the variables being

(a) for the sheet pile wall - cantilevered or anchored;

(b) for the depth of driving - free earth support or fixed earth support.

5.1 Anchored sheet pile wall with free earth support

The stress distribution acting on the wall is shown in Fig.3.

The unknowns are the anchor force T and the depth of driving D.

For the passive pressure distribution only, a reduced value of ϕ_R is used.

The analysis may be done either analytically or graphically.

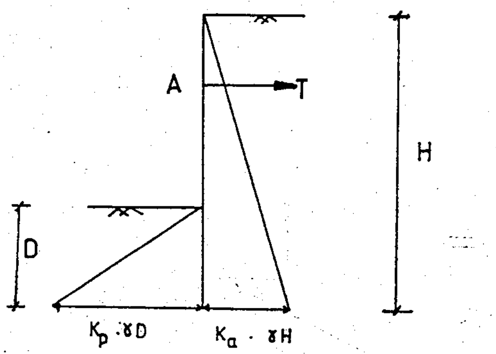


Fig.3 - Stress distribution for Anchored sheet pile for Free earth support condition

In the analytical method, the steps are

(i) take moments about A and determine D;

(ii) take moments about the toe and determine T ;

(iii) construct the BM diagram and determine the maximum bending moment.

Because of the possibility of loss of passive support if the dredge level is lowered due to scouring or any other process, it is conventional to consider D as the minimum depth of driving and in practice, to increase D by 20%.

The construction of the BM diagram and the determination of the maximum bending moment analytically can be tedious. Therefore, in many design offices a graphical procedure is commonly used. This procedure is illustrated in Appendix I with a worked example.

5.2 Cantilevered sheet pile wall with free earth support

The stress distribution acting on the wall is the same as that shown in Fig.3.

Again a reduced value of ϕ is used to determine the passive pressure distribution.

There is no anchor force, and hence the only unknown is the depth of driving D.

The depth of driving can be computed by equating the horizontal forces which gives a quadratic equation in D.

Now construct the BM diagram as before and determine the maximum bending moment.

5.3 Cantilevered sheet pile wall with fixed earth support

The stress distribution acting on the wall is shown in Fig.4.

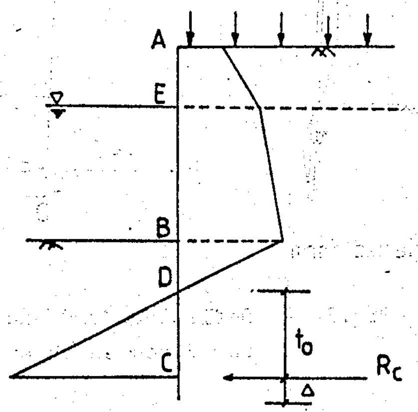


Fig.4 - Stress distribution for Cantilevered sheet pile for Fixed earth support condition

The unknown is the depth of driving D.

In this case,

(i) the full passive pressure can be taken into account; and

(ii) there is the reactive force R_c near the toe.

The analysis may be carried out either analytically or graphically.

In the analytical method the steps are :

(i) take moments about C and determine t_o . (Note that this gives a cubic equation in t_o and the most efficient method of solution is the iterative method for the solution of polynomial equations.)

(ii) determine the extended length Δ from the equation

$$\Delta = \frac{0.45\ R_c}{\sigma_c}$$

(iii) depth of driving = $\overline{BD} + t_o + \Delta$

(iv) construct the bending moment diagram and determine the maximum bending moment.

The alternate graphical procedure commonly used in many design offices is shown in Appendix II with a worked example.

5.4 Anchored sheet pile wall with fixed earth support

In this case as well, the analysis can be done either analytically or graphically. The presence of the anchor provides an additional constraint which makes the analysis more complex than for the cantilever wall.

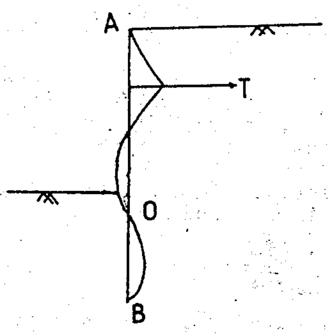


Fig.5 - Typical BM diagram for Anchored sheet pile for Fixed earth support condition

A typical bending moment diagram for this case is shown in Fig.5. It is seen that there is a point of contraflexure at the point O. The analysis is simplified if the position of O is known. BLUM (1931) has presented some results for determining the approximate position of the point of contraflexure and these are given in Table 1.

ϕ	20°	30°	40°
x/h	0.25	0.08	-0.007

where x = distance below dredge level;
and h = height of backfill.

Table 1

Then by considering the equilibrium of the part AO, it is possible to determine the anchor force as well as the shear force at O. Next the equilibrium of part OB is considered, and this gives the value for distance OB. Thus the depth of driving can be computed.

The bending moment diagram can then be constructed as before.

6.0 ROWE'S DESIGN METHOD

Although it is clear that pile flexibility, soil stiffness and anchor yield must influence the earth pressure distribution, the simple design method discussed in Section 5 has not taken these into account. ROWE (1952) was one of the first to consider the effect of soil structure interaction.

6.1 Parameters to take into account pile flexibility and soil stiffness below dredge level

The notations used by Rowe are given in Fig.

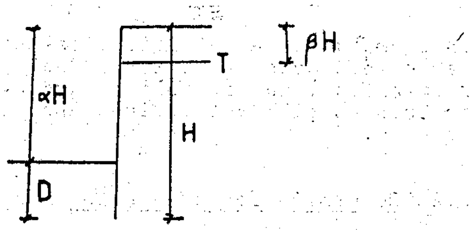


Fig.6 - Notations used in analysis of ROWE (1952)

The flexure equation below dredge level is

$$EI \frac{d^4y}{dx^4} = w = p_a - p_p$$

It is assumed that

$$p_p = \frac{mxy}{D} = \frac{mxy}{(1-\alpha)H}$$

where m is a measure of soil stiffness. (It should be noted that m has units of kN/m^2 per mm or kN/m^3, and it is therefore related to the coefficient of subgrade reaction.)

The values of m estimated by Rowe for some typical cohesionless soils is given in Table 2.

Soil	log m (kN/m^3)
Dense sand	5.2
Loose sand	3.9
Loose silt	3.2

Table 2

After some working, Rowe shows that the deflection of the pile below dredge level is a function of ($m\rho$)

where $\rho = \dfrac{H^4}{EI}$

Thus ($m\rho$) is a dimensionless number which takes into account soil stiffness and pile flexibility, and it is a parameter which influences pile deflections and bending moments.

6.2 Determination of bending moments without anchor yield

If M_{FES} is the maximum 'free earth support' or 'fixed earth support' bending moment as calculated earlier,

then the bending moment (M) taking into account pile flexibility and soil stiffness is given by

$$M = r_d \times M_{FES}$$

where r_d is a reduction factor which is a function of α, m and ρ.

The variation of r_d with $\log\rho$ for sand is given in Fig.7.

6.3 Influence of anchor yield

When there is anchor yield, there is in general an increase in the anchor load (as a result of straining of the tie bar), and a reduction in the maximum bending moment caused by the redistribution of stress.

$$\therefore \quad T = f_t \times T_{FES}$$

and $$M = r_t \times r_d \times M_{FES}$$

The correction factors r_t and f_t are given in Fig.8.

6.4 The operational curve of τ vs. $\log\rho$

Define $$\tau = \frac{M}{H^3} = \frac{r_d \times r_t \times M_{FES}}{H^3}$$

Since r_d is a function of ρ, and initially the pile section has not been designed, ρ is unknown.

Therefore, τ vs. $\log\rho$ can be plotted, and this is referred to as the operational curve.

6.5 The structural curve of τ vs. $\log\rho$

The structural curve is the relationship between τ and ρ, (τ and ρ being as defined earlier), where M refers to the resisting moment of the section.

For wood piles of rectangular section, the working shown in Appendix III gives

$$\tau = \frac{0.94}{\sqrt[3]{H\rho^2}}$$

The structural curve for the wood piles is shown in Fig.9.

Similarly, for steel piles, the working shown in Appendix IV gives

$$\tau = \frac{0.31}{\sqrt[3]{H\rho^2}}$$

The structural curve for the steel piles is also shown in Fig.9.

6.6 Summary of Rowe's design method

1. Calculate the depth of penetration, maximum bending moment, and force in the anchor rod ignoring soil structure interaction. This gives T_{FES} and M_{FES}.
2. Apply factor f_t to anchor load for normally yielding anchors.
3. Select r_t, and construct the operational curve of τ vs. $\log\rho$.
4. Construct the structural curve of τ vs. $\log\rho$.
5. Superimpose the structural curve on the operational curve as shown in Fig.10. The point of intersection between the two curves gives the maximum bending moment which would act at yield on the wall.

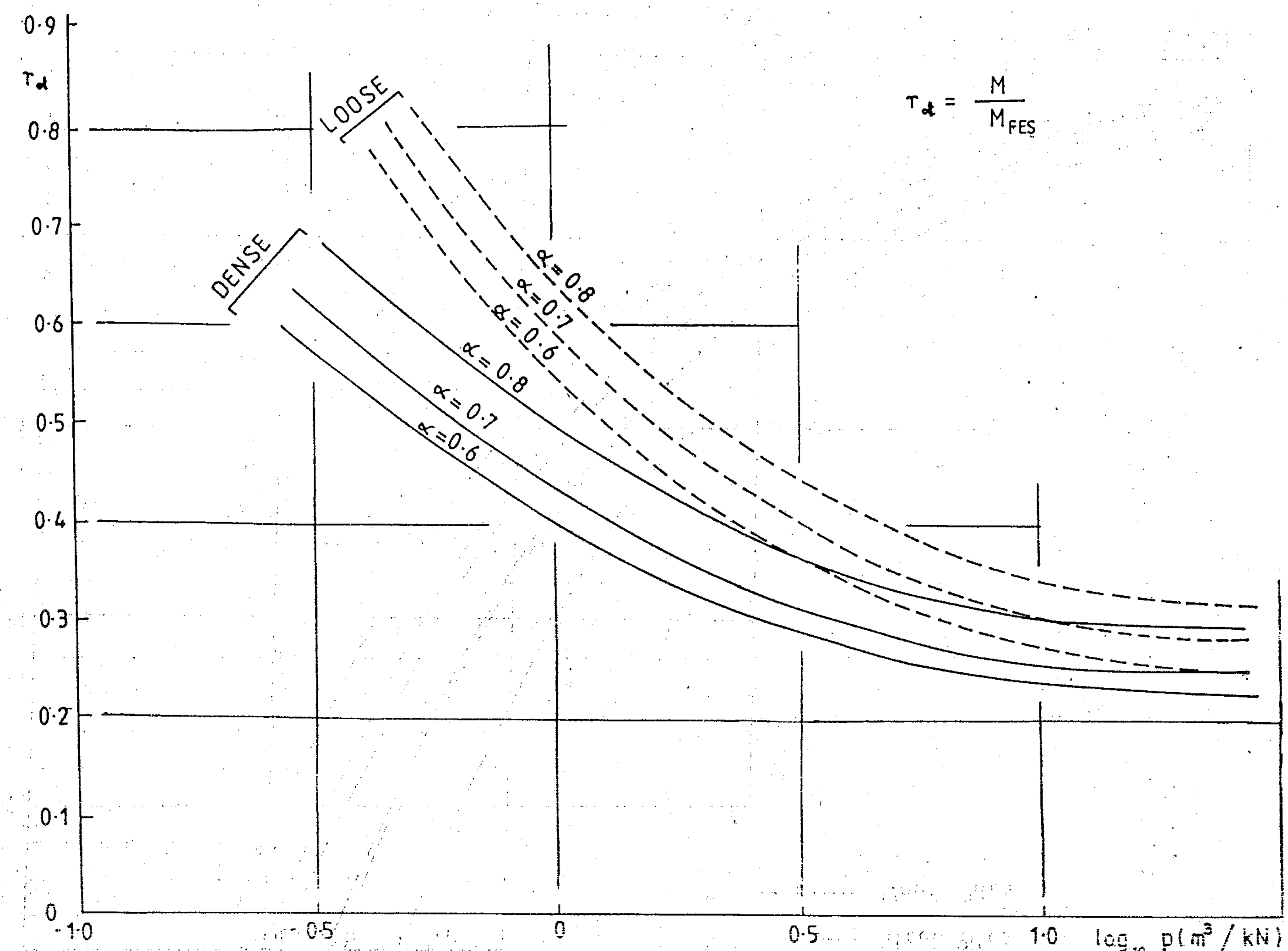


Fig. 7 - Variation of τ_α with ρ - Rowe analysis

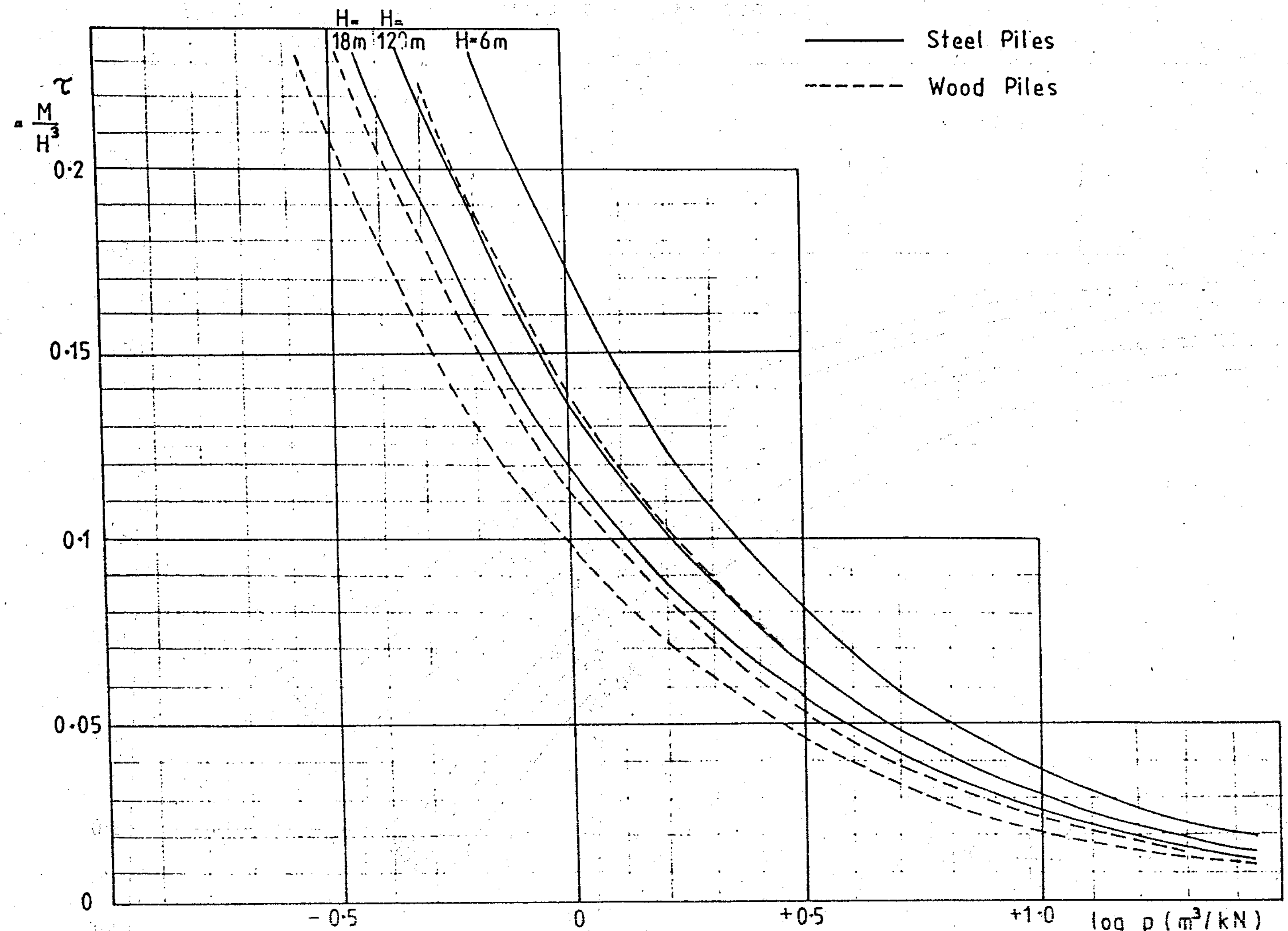


Fig. 9 - Structural curve - Rowe analysis

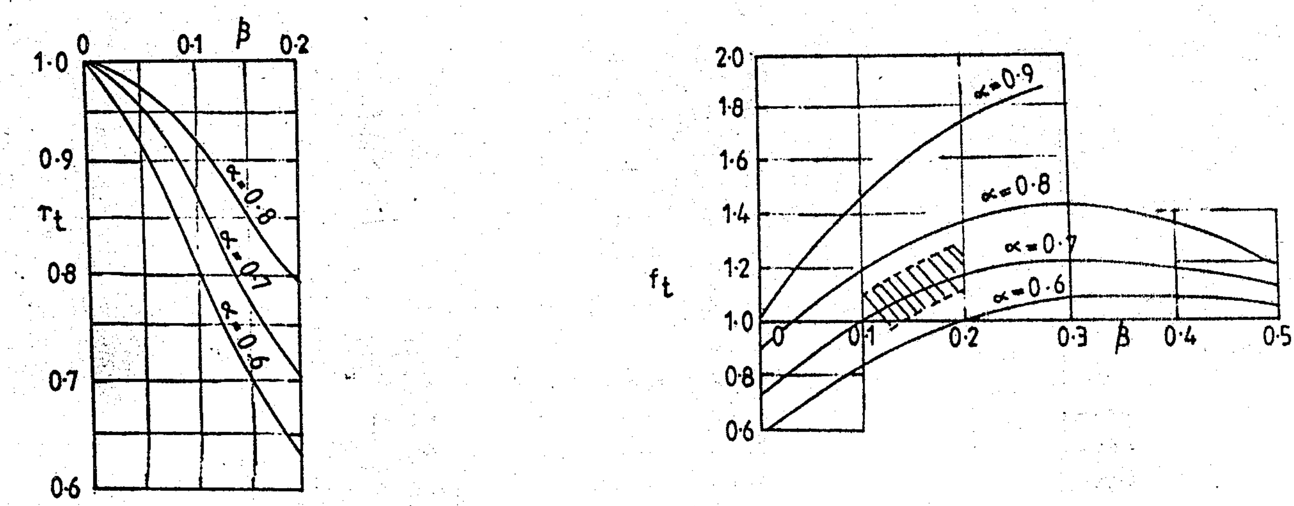


Fig 8 - Variation of τ_t , f_t with α and β - Rowe analysis

6. The required section modulus is then obtained from equation

$$Z = M_{max} \times \frac{F'}{f_y}$$

where f_y = yield stress of material

and F' = factor of safety.

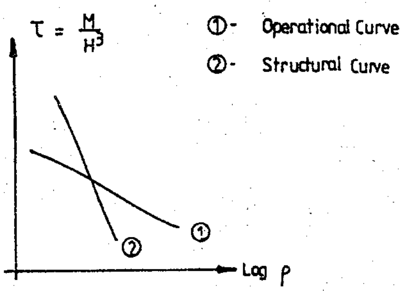


Fig.10 - Determination of maximum design BM by Rowe analysis

7.0 THE MATRIX METHOD OF ANALYSIS

The matrix method of analysis is based on the approach of 'Beams on elastic foundation' where the retaining wall below the dredge level is assumed to be supported on a series of springs.

Consider the example shown in Fig.11a.

The sheet pile is first divided into a series of elements by means of suitably placed nodes. In this example, the soil above the dredge level has been divided into 5 elements, and the soil below the dredge level has also been divided into 5 elements.

The external forces acting on the wall are transferred to the nodal points and are marked in Fig.11b as moments $P_1, P_2, \ldots, P_{11}$ and lateral forces $P_{12}, P_{13}, \ldots, P_{22}$.

These forces (P), give rise to external nodal displacements (X).

Use the coding $X_1, X_2, \ldots, X_{22}$ corresponding to $P_1, P_2, \ldots, P_{22}$.
(The sign convention for P and X are : (i) clockwise positive (ii) ⟵ positive).

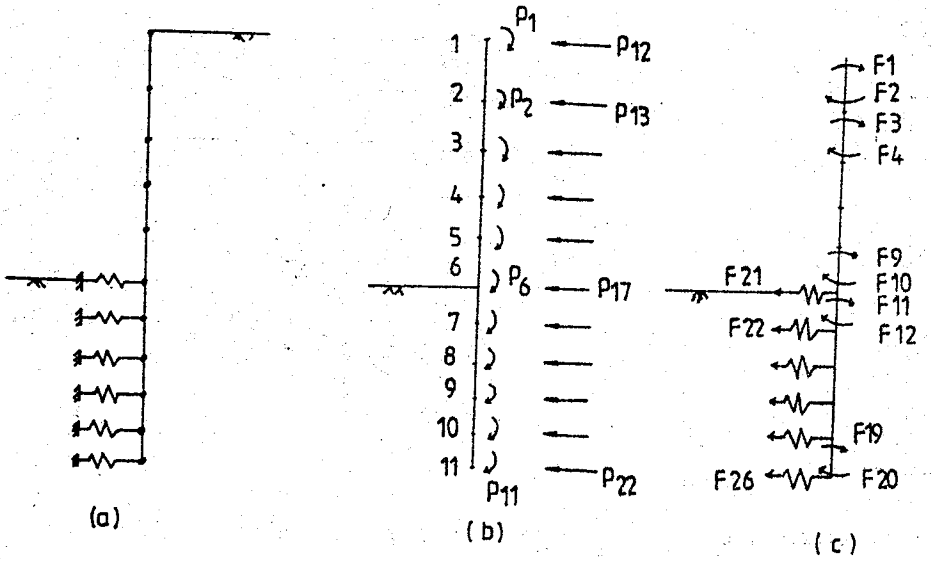


Fig.11 - Finite element formulation of sheet pile wall problem

The external forces are resisted by the internal resisting moments $F_1, F_2, \ldots, F_{20}$ of the elements, and the spring forces $F_{21}, F_{22}, \ldots, F_{26}$. These are shown in Fig.11c.

These internal forces (F) are set up as a result of internal deformations (e).

Use the coding $e_1, e_2, \ldots, e_{26}$ corresponding to $F_1, F_2, \ldots, F_{26}$.

Note that

(i) the forces F_1, F_2; F_3, F_4; etc. as marked in Fig.11c are the forces (moments) on the beam element. Hence, the forces on the nodes are equal and opposite to the forces on the element;

(ii) the soil spring forces are applied at the nodes.

The forces on the beam elements are as marked in Fig.12a; and the forces on the nodes are as marked in Fig.12b.

Consider the equilibrium of forces at the nodes.

For example, at Node 1

$$P_1 = F_1$$

and $$P_{12} = \frac{F_1}{a} + \frac{F_2}{a}$$

where a = length of element

At Node 2,

$$P_2 = F_2 + F_3$$

and $$P_{13} = -\frac{F_1}{a} - \frac{F_2}{a} + \frac{F_3}{a} + \frac{F_4}{a}$$

Then again at Node 6,

$$P_6 = F_{10} + F_{11}$$

and $$P_{17} = -\frac{F_9}{a} - \frac{F_{10}}{a} + \frac{F_{11}}{a} + \frac{F_{12}}{a} - F_{21}$$

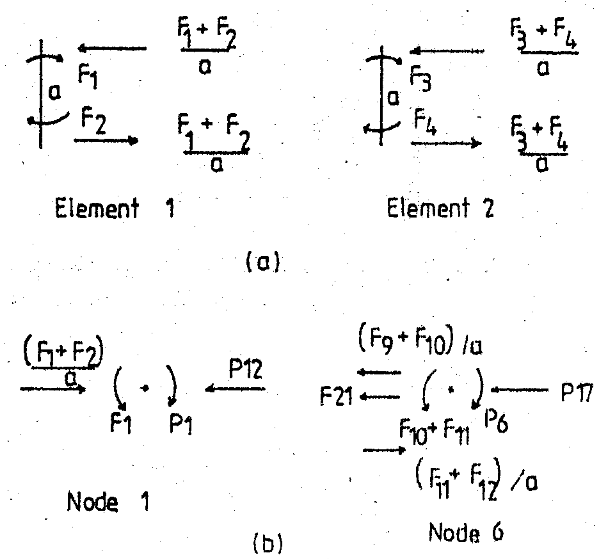


Fig.12 - Forces on beam elements and nodes

Therefore, by considering the equilibrium of forces acting at a node, a set of equations is obtained of the form

$$[P] = [A][F] \qquad (7\text{-}1)$$

where $[A]$ is given in Table 3.

By considering the relationship between external nodal displacements and internal deformation,

$$[e] = [B][X]$$

It has been shown that $[B] = [A]^T$

so that $$[e] = [A]^T[X] \qquad (7\text{-}2)$$

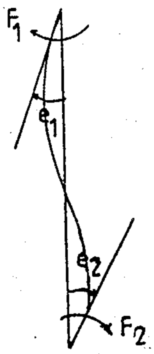


Fig.12c - Notations for Stiffness formulation

The internal forces (moments) set up in a member are related to the internal deformations through the stiffness matrix so that

$$[F] = [S][e] \qquad (7\text{-}3)$$

For example, for the topmost element of Fig.11 , the forces F_1 ,F_2 and the deformations e_1 ,e_2 are as shown in Fig.12c.

Then it can be shown that

$$F_1 = \frac{4EI}{a} \cdot e_1 + \frac{2EI}{a} \cdot e_2$$

and $$F_2 = \frac{2EI}{a} \cdot e_1 + \frac{4EI}{a} \cdot e_2$$

Then, for the spring at dredge level,

$$F_{21} = K_1 \cdot e_{21}$$

where $K_1 = a \cdot B \cdot k_s^*$

with k_s^* being the averaged modulus of subgrade reaction at the node at dredge level.

Hence the stiffness matrix can be built up as shown in Table 4.

Combining Equations (7.2) and (7.3),

$$[F] = [S][A]^T[X] \qquad (7\text{-}4)$$

Combining Equations (7.1) and (7.4),

$$[P] = [A][S][A]^T[X] \qquad (7\text{-}5)$$

Removing the brackets for convenience,

$$P = ASA^T X$$

$$\therefore [X] = [ASA^T]^{-1}[P] \qquad (7\text{-}6)$$

A computer program can now be written for which the flow chart is as shown in Fig.13.

Having inverted the ASA^T matrix,

$[X]$ is computed from Equation (7.6) , and

$[F]$ is computed from Equation (7.5).

Such a program has been developed by BOWLES (1974). This has been modified by the Author to be run on the IBM PC , and this is shown in Appendix V with some typical results.

[A] =

P \ F	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26
1	1																									
2		1	1																							
3				1	1																					
.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.	.
10																		1	1							
11																				1						
12	z	z																								
13	$-z$	$-z$	z	z																						
14			$-z$	$-z$	z	z																				
15					$-z$	$-z$	z	z																		
16							$-z$	$-z$	z	z																
17									$-z$	$-z$	z	z									-1					
18											$-z$	$-z$	z	z								-1				
19													$-z$	$-z$	z	z							-1			
20															$-z$	$-z$	z	z						-1		
21																	$-z$	$-z$	z	z					-1	
22																			$-z$	$-z$						-1

$z = \frac{1}{a}$

a = length of element

Table 3 - The A-matrix

[S] =

F \ e	1	2	3	4		19	20	21	22	23	24	25	26
1	A	B											
2	B	A											
3			A	B									
4			B	A									
19						A	B						
20						B	A						
21								K_1					
22									K_2				
23										K_3			
24											K_4		
25												K_5	
26													K_6

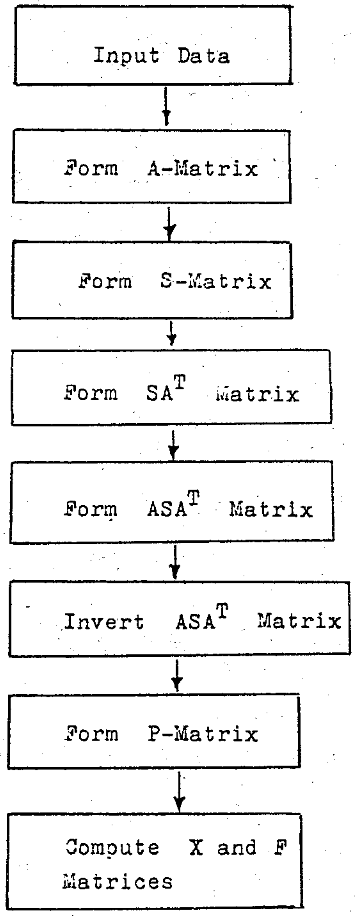
Where

$A = \frac{4EI}{a}$

$B = \frac{2EI}{a}$

$K_1 = aBk_s$

Table 4 - The S-matrix



The input data consists of

NABOV = No. of nodes above dredge line

NBELO = No. of nodes below dredge line

HWALL = height of wall above dredge line

HROD = depth from ground line to anchor rod.

ERN = moment of inertia of pile per meter length

ELAS = modulus of elasticity of wall material.

DEMB = initially assumed depth of embedment

HWAT = depth of water

GSAT = saturated unit weight of soil

GWET = wet unit weight of soil

PHI = angle of internal friction of backfill

DELTA = angle of wall friction

SCHGE = surcharge pressure

ARODK = K of anchor rod = $\frac{EA}{L}$

KS = modulus of subgrade reaction of soil.

I = segment number

H(I) = segment length

NODWAT = node locating water

NODAR = node locating anchor rod

Fig.13 Flow chart for Matrix method of analysis for sheet piles

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* * * * * * * *

APPENDIX I

Graphical procedure for analysing anchored sheet pile walls with free earth support

Consider the example shown in Fig.14 where an anchored sheet pile wall retains the bank of a canal.

Other data are :

height of soil behind wall above dredge level = 10 m

height of water in canal = 7 m

surcharge carried by soil behind sheet pile wall = 20 kN/m^2

Soil consists of a sandy soil with $\phi = 30$ and densities
(i) above W.T. $\gamma = 18 \; kN/m^3$
(ii) below W.T. $\gamma' = 11 \; kN/m^3$

(Suppose it is stated that in this case the full passive pressure can be relied on. This is usually not the case for the free earth support condition.)

First compute the earth pressure for which $K_a = \tan^2(45 - \phi/2) = 0.3$

and $K_p = \tan^2(45 + \phi/2) = 3.0$

Referring to Fig.14 (i),

at A, $\sigma'_v = 20 \; kN/m^2$

$$\therefore \sigma'_H = 20 \times 0.3 = 6 \; kN/m^2$$

at E, $\sigma'_v = 20 + 18 \times 3 = 74 \; kN/m^2$

$$\therefore \sigma'_H = 74 \times 0.3 = 22.2 \; kN/m^2$$

Similarly, at B, $\sigma'_H = 45.4 \; kN/m^2$.

(Since the water pressure is identical on both sides of the wall, the wall pressure is due to σ'_H only.)

Below the dredge level,

$$\sigma'_H = 45.4 - (K_p - K_a) \times 11 \times z$$

where z = depth below dredge line.

$$\text{i.e. } \sigma'_H = 45.4 - 29.7 z$$

$\therefore$ Position of D where $\sigma'_H = 0$ is given by

$$z_o = 45.4/29.7 = 1.53 \text{ m}$$

Measure t from D.

In the graphical procedure, the pressure diagram is divided into several slices and the forces are marked at the centre of each slice. (Refer Fig.14 (i) and (ii))

In Fig.14 (i), a length scale of 1 mm = p metres is chosen.

The line of forces is next drawn in Fig. 14 (iii) to a scale of 1 mm = q kN.

ab = 8.7, bc = 14.1, ..., lm = 34.7 for active side; and

mn = 3.7, ..., tu = 55.7 for passive side.

For the construction of the BMD of Fig. 14 (iv), it is necessary to select a pole in the line of force diagram of Fig. 14 (iii) distant s mm from the line of force.

(To avoid confusion of lines, two lines of force have been drawn - one for the active forces and the other for the passive forces. Correspondingly, two poles P_1 and P_2 have been used.)

The rays in Fig. 14 (iii) are then projected to the appropriate spaces of Fig. 14 (iv).

It is now necessary to construct the base line for the BMD and close it.

The BMD should have a shape as shown in Fig. 14 (v). Therefore, the closing line AC_o should satisfy 2 conditions :

(i) the closing line should intersect the tangent at the origin of the line of moments at the level of the anchorage A.

Referring to Fig. 14 (iv), A is the point of intersection between the tangent at A_o and the level of the anchor.

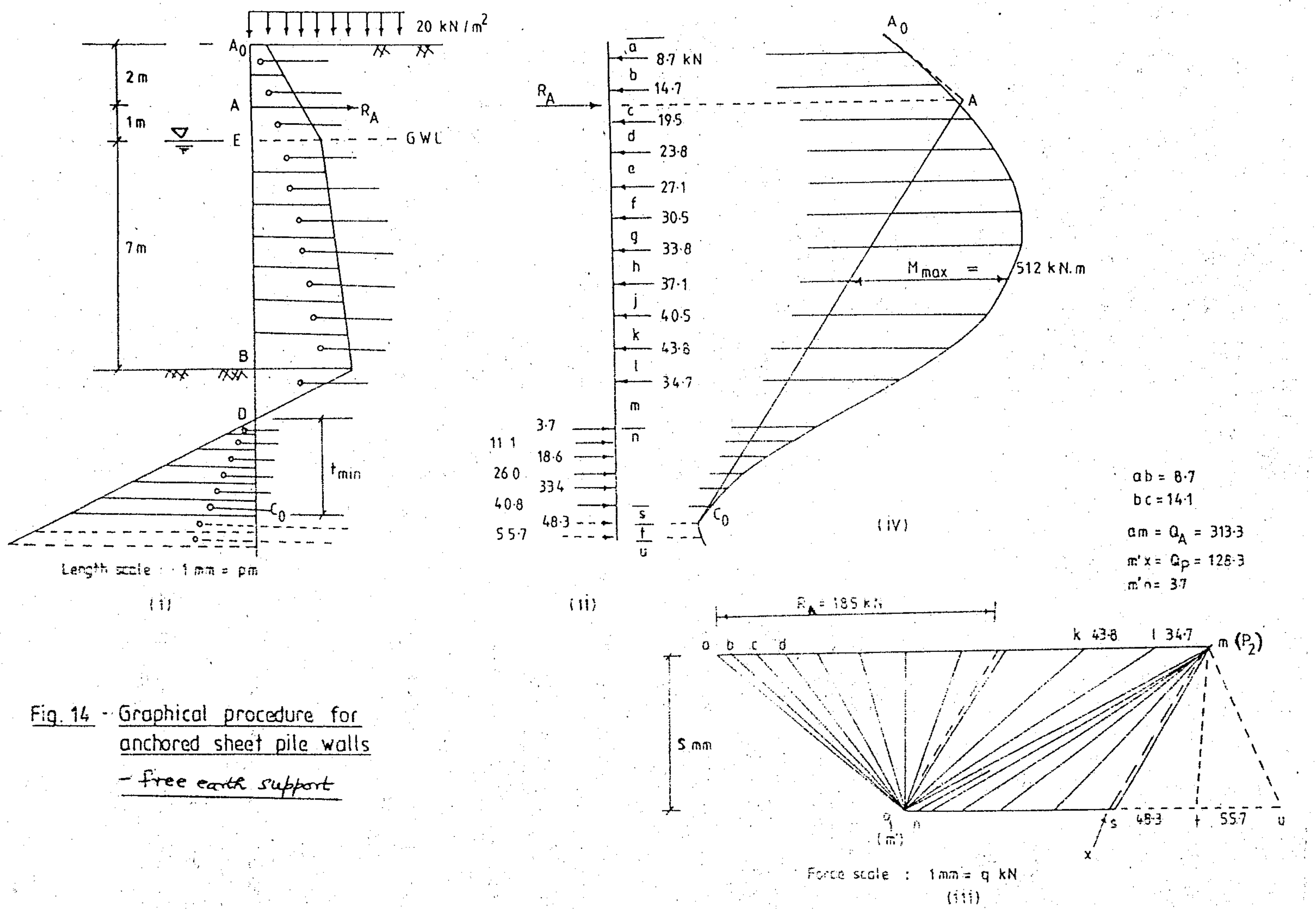


Fig. 14 - Graphical procedure for anchored sheet pile walls - free earth support

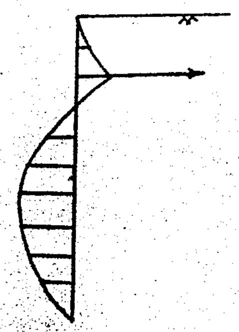


Fig. 14(v) – BMD for free earth support

(ii) the closing line AC_o should be tangential to the line of moments at the toe C_o. (This is because at the toe, the shear force = 0, hence $dM/dx = 0$.)

In Fig. 14(iii), a line drawn from P_1 parallel to AC_o gives R_A = 185 kN. Also, from P_2 draw line P_2X parallel to AC_o.

The force due to active pressure = Q_A = am = 313.3 kN

The force due to passive pressure = Q_P = mx = 128.3 kN

If the maximum bending moment read off the funicular polygon = k mm,

then maximum BM = k x (p x q x s) kN.m = 512 kN.m per m.

t_{min} can be read off from Fig. 14(i) as = 2.94 m.

$\therefore$ D_{min} = 1.53 + 2.94 = 4.47 m

$\therefore$ Design depth of driving = 1.2 D_{min} = 5.36 m

APPENDIX II

Graphical procedure for analysing cantilever sheet pile wall with fixed earth support

Consider the example shown in Fig. 15 where a cantilever sheet pile wall retains the bank of a canal.

Other data are :

height of soil behind wall above dredge level = 3.5 m

height of water in canal = 2.5 m

surcharge carried by soil behind sheet pile wall = 10 kN/m^2

soil consists of a sandy soil with $\phi = 30^\circ$ and densities of
(i) above W.T. γ = 18 kN/m^3
(ii) below W.T. γ' = 11 kN/m^3

Referring to Fig. 15(i), the stresses σ'_H at A, E and B are computed as before.

Below dredge level,

$$\sigma'_H = 18 \cdot 3 - 29 \cdot 4\, z$$

so that if t is measured from the point D, then the stresses below D are given by

$$\sigma'_H = 29 \cdot 4\, t$$

The construction of the pressure diagram and the BMD is done as before and shown in Fig. 15. It should be noted that in the construction of the BMD, because the shear force at the top of the wall is zero, dM/dx = 0 at the top of the wall. Hence if the pole P_1 is taken vertically below the commencement of the active force polygon as in Fig.15(iii), then the base line in Fig.15(iv) is also vertical.

The depth of penetration of the cantilever is then found as the intersection of this base line with the BMD, and this gives t_o = 3.75 m.

Also, Q_P = 206.7 kN ; Q_A = 46.5 kN

and hence R_C = 160.2 kN

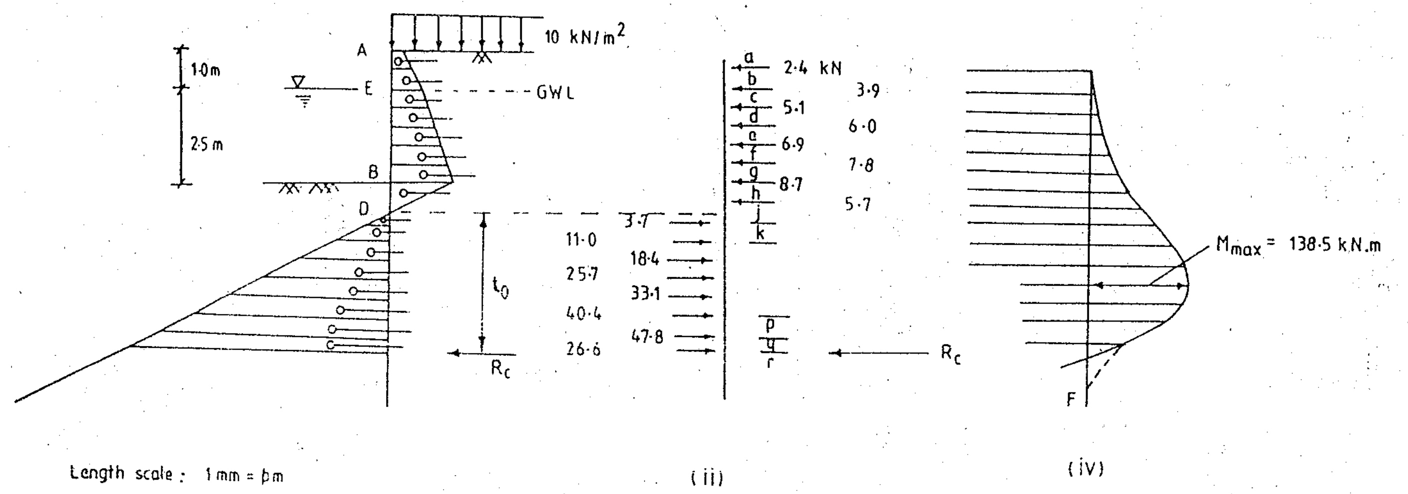
σ_C = 29.4 x 3.75 = 110.2 kN/m^2

so that the extended length of the sheet pile is

$$\Delta = \frac{0 \cdot 45\, R_C}{\sigma_C} = 0 \cdot 65 \text{ m}$$

This point is now marked as F in Fig.15(iv) and the BMD is adjusted so that the BM at F = 0.

The maximum bending moment is now read off the funicular polygon = k x (p x q x s) = 138.5 kN.m per m



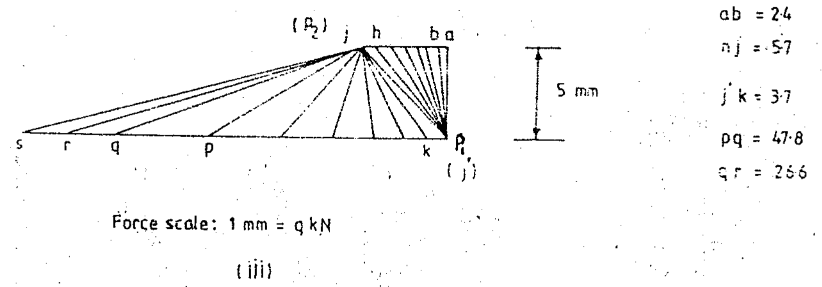


Fig. 15 - Graphical procedure for cantilever sheet pile walls
– fixed earth support

APPENDIX III

The Structural Curve of τ vs. ρ for wood piles of rectangular section

Consider a wall of width $b = 1$ m.

Since $I = \frac{1}{12} b d^3$, $d = (12\ I)^{1/3}$

$$M = \frac{I}{y} \cdot f = (d^3/12) \times (2/d) \times f_{max}$$

i.e. $M = (2/12) \cdot d^2 \cdot f_{max} = (2/12) \cdot (12 I)^{2/3} \cdot f_{max}$

i.e. $M = \frac{2}{\sqrt[3]{12}} \cdot I^{2/3} \cdot f_{max}$

Introducing parameters

$$\rho = \frac{H^4}{EI} \quad \text{and} \quad \tau = \frac{M}{H^3}$$

$$\tau = \frac{M}{H^3} = \left[\frac{2}{\sqrt[3]{12}} \cdot I^{2/3} \cdot f_{max}\right] \cdot \frac{1}{H^3}$$

$$\text{i.e. } \tau = \frac{2}{\sqrt[3]{12}} \cdot \left(\frac{H^4}{E\rho}\right)^{2/3} \cdot \frac{f_{max}}{H^3}$$

$$\text{i.e. } \tau = \frac{2}{\sqrt[3]{12}} \cdot \frac{f_{max}}{E^{2/3}} \cdot \sqrt[3]{\frac{1}{H\rho^2}}$$

For wood, taking $f_{max} = 50{,}000$ kN/m^2 ,
and $E = 10^7$ kN/m^2 ,

$$\tau = \frac{0 \cdot 94}{\sqrt[3]{H\rho^2}}$$

APPENDIX IV

The Structural Curve of τ vs. ρ for steel piles

Consider a simplified section shown in Fig.16.

I per unit length $= \frac{1}{12} \cdot KD^3$

where K = dimensionless shape factor.

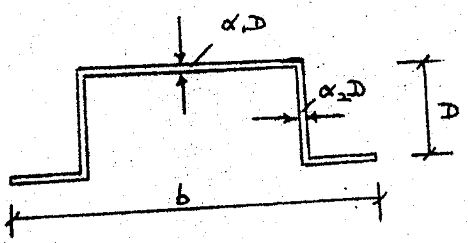


Fig. 16 - Simplified section for steel pile

By analogy with Appendix III ,

$$\tau = 2\sqrt[3]{\frac{K}{12}} \cdot \frac{f_{max}}{E^{2/3}} \sqrt[3]{\frac{1}{H\rho^2}}$$

Evaluation of K for steel sheet piles

The section modulus

$$Z = \frac{I}{(D/2)} = \frac{1}{12} KD^3 \times \frac{2}{D} = \frac{1}{6} KD^2$$

Tables for steel sections give Z and D from which K can be calculated for any section.

It is found that for most steel sections,

$$\sqrt[3]{K} \simeq 0 \cdot 53$$

For steel, taking $f_{yield} = 230{,}000$ kN/m^2 ,
and $E = 2 \times 10^8$ kN/m^2 ,

$$\tau = \frac{0 \cdot 31}{\sqrt[3]{H\rho^2}}$$

COST COMPARISON OF RETAINING WALLS FOR DIFFERENT GROUND CONDITIONS

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INTRODUCTION

Gravity walls, Cantilever walls, Counterfort walls and Buttressed walls are four types of commonly used rigid retaining walls. Of these, the most commonly used are the first three types, whereas the Buttressed wall is used only occasionally under special conditions. In Sri Lanka, it has been customary to build Gravity walls, and occasionally when the heights are large, Cantilever or Counterfort walls are constructed.

Prior to 1973, there was no published data as to the economic evaluation of the different types of walls. Tennekoon and Hemachandra (1973) provided some of this information using assumed soil and wall parameters along with unit costs of labour and material applicable at that time. The basis for the selection of these parameters has been dealt with by Hemachandra (1973).

This paper is an attempt to provide updated information based on current costs of labour and material and using the same soil and wall parameters. Total costs of Gravity, Cantilever and Counterfort walls are analysed for Ultimate Bearing Capacities of 8,6,4 and 2 Tons/sq.ft.

EARTH PRESSURE CALCULATIONS

The earth pressure calculations which are currently favoured are:-
(i) Coulomb's wedge theory,
(ii) Rankine's theory,
(iii) the method of slices, and
(iv) the analysis of Sokolovskii (1965).

The Coulomb and the Rankine theories are the classical theories, while the others are more recent. The method of slices, which is useful when the soil is stratified, is not used in this paper where uniform soil conditions are assumed. The Sokolovskii analysis is a very refined method, and its use is justified only when the other parameters such as the distribution of wall friction over the height of the wall are accurately known. Such distributions of wall friction have been measured in a few simple laboratory experiments e.g. James and Bransby (1970), but until accurate measurements under field conditions are made, the more sophisticated Sokolovskii analysis will not necessarily yield more accurate results than the Classical Theories. The Author has therefore kept to the conventional practice of determining the earth pressures using Coulomb and Rankine theories.

GRAVITY RETAINING WALLS

A typical section of a Gravity wall with broken backs is shown in Fig.1(a). For analysis they are generally assumed to be trapezoidal in shape, as shown in Fig.1(b). Because of their massive proportions and the resulting low stresses, the Gravity walls could be built either of low quality concrete (1:3:6), or with rubble.

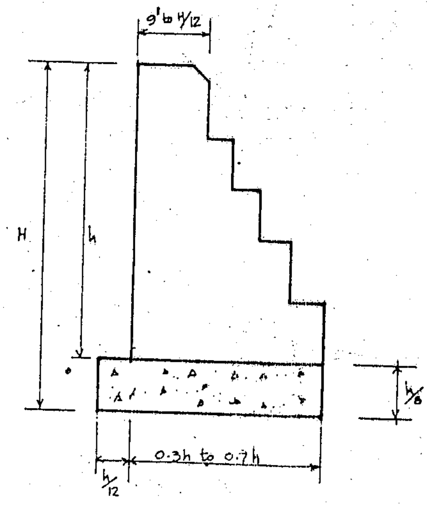


FIG.1(a)

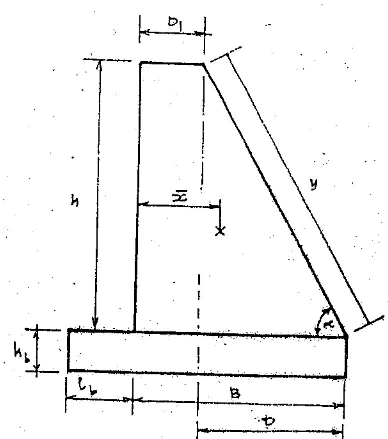
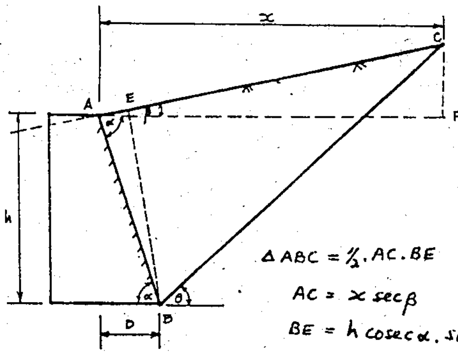


FIG.1(b)

The earth pressure calculations are done according to Coulomb analysis which assumes a straight line failure surface. Any trial failure surface [e.g. BC of Fig.2(a)] is selected, and the forces acting on the wall are computed by considering the equilibrium of the soil block contained between the failure surface and the wall. These forces are marked in Fig.2(b), and the polygon of forces constructed in Fig.2(c). The Coulomb analysis consists of the determination of the maximum earth pressure acting on the wall from several trial failure surfaces selected.



$$\Delta ABC = \tfrac{1}{2}.AC.BE$$
$$AC = x \sec\beta$$
$$BE = h \operatorname{cosec}\alpha . \sin(\alpha+\beta)$$

FIG. 2 (a)

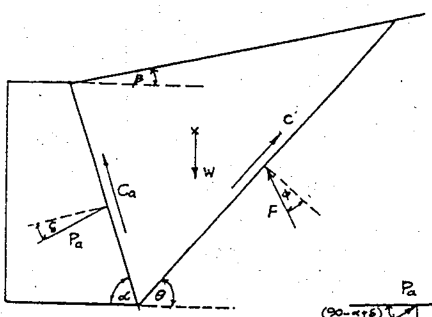


FIG. 2 (b)

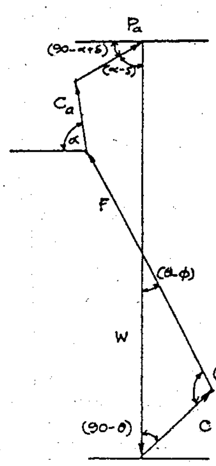


FIG. 2(c)

The forces acting on the wall are shown in Fig.3, and the wall is designed so that
(i) there is no tension developed at the base,
(ii) the wall is stable against overturning about the toe,
(iii) the wall is stable against sliding on the base, and
(iv) the bearing capacity of the soil is not exceeded.

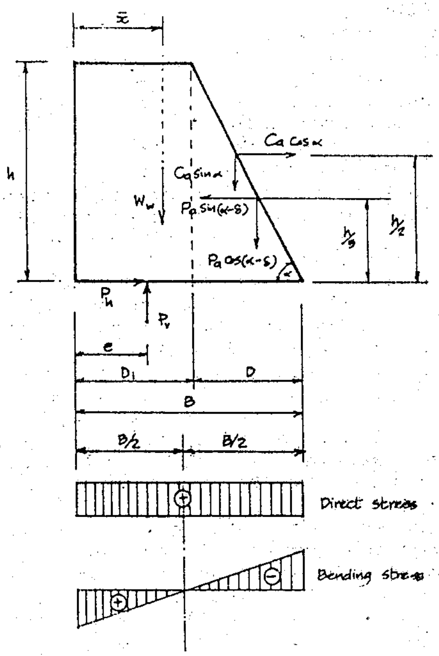


FIG. 3

The steps in the computer program for the design and cost analysis of any Gravity wall is given in Appendix 1. Although the program has been written very generally to be applicable for most situations, specific parameters are read in, in Steps 1 and 2, so that a comparison of the costs of different types of walls of any given height could be made.

Steps 4 to 6 consist of obtaining trial values for the dimensions of the wall; while in Steps 7 to 9 the forces on the block of soil considered are calculated. The check for the "no tension" condition, and the calculations of the different Factors of Safety are carried out in Steps 10 to 14. If it is found that tensile stresses develop at the base, or that the Factors of Safety against overturning or sliding are less than 1.5, or the Factor of Safety against bearing is less than 2.0, then the base width is increased and the calculations repeated until these design criteria are satisfied.

Finally, an estimate of the cost per foot length of the Gravity wall is made in Step 17. [The unit prices used for this calculation are given in Step 1. These may not be very accurate because of the rapidly changing prices today. Nevertheless, since the relative costs could be considered as fairly constant, it is possible to make a cost comparison among the different types of retaining walls.]

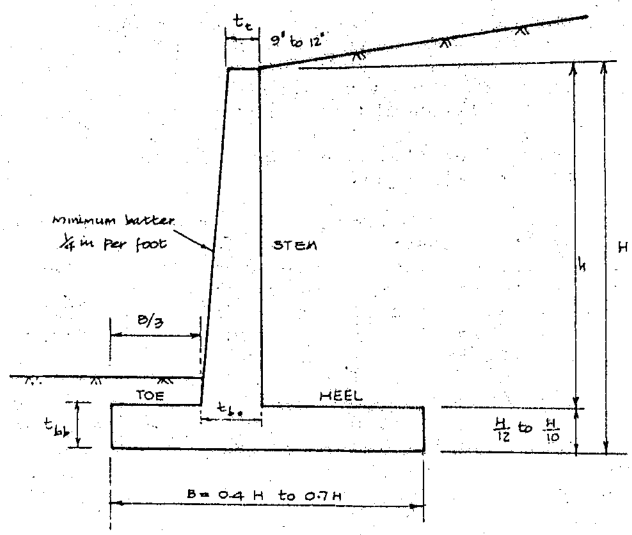


FIG 4

CANTILEVER RETAINING WALL

Tentative dimensions for a Cantilever retaining wall are given in Fig.4 [Bowles(1968)-Fig.7-3]. These are based on the history of satisfactorily constructed walls.

This type of wall could be thought of as three cantilever beams: the vertical stem, the toe projection, and the heel projection. The top of the stem is generally not made less than 9" so that proper placement of the concrete could be effected.

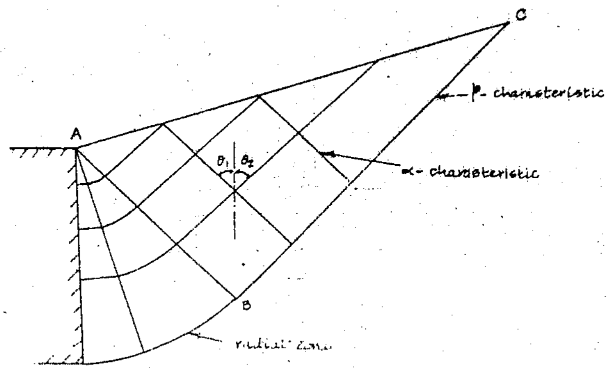


FIG. 5

The Rankine theory is commonly used to compute the resultant force on a vertical plane through the heel of the wall. The Rankine formula can only be used in regions where both sets of Stress Characteristics are linear. (The stress characteristics at a point in equilibrium correspond to the directions of the two failure planes through that point.) In the earth pressure problem of Fig.5, both sets of characteristics are linear in the region marked ABC. The inclination of these characteristics to the vertical can be shown to be equal to

$\theta_1 = 1/2(\pi/2-\phi) - 1/2(\Delta-\beta)$, and

$\theta_2 = 1/2(\pi/2-\phi) + 1/2(\Delta-\beta)$

where $\sin\Delta = \sin\beta / \sin\phi$

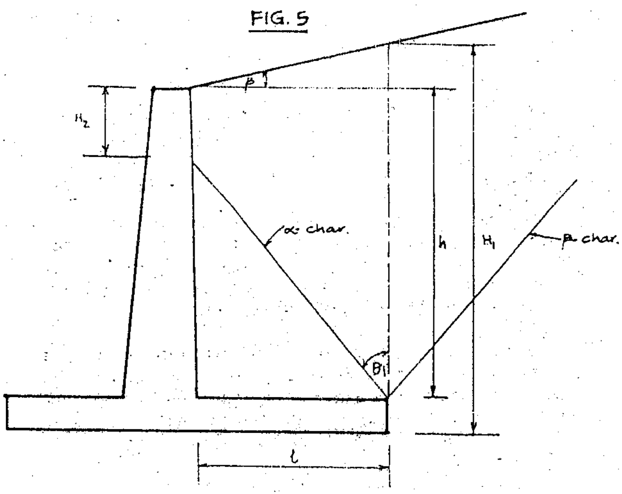


FIG 6

Referring to Fig.6, the earth pressure on the vertical plane through the heel is equal to that given by the Rankine formula only if the line drawn from the heel at θ_1 to the vertical does not intersect the stem of the wall. However, it has been shown that the use of the Rankine theory would not introduce any appreciable errors provided that if this line does intersect the wall, it does so at a point higher than the point given by 0.4 times the height of the soil above the extreme of the heel, measured from the top; i.e. in Fig.6, $H_2 < 0.4\ H_1$. [Huntington (1957) - Ch.2, Art.8].

The coefficient of earth pressure obtained from the Rankine theory can be calculated from a simple formula for a cohesionless material. Therefore, the soil which had strength parameters of c=150 lbf/sqft.(7.18 kN/sqm.). and ϕ =28 deg. in the Gravity wall calculation, has been considered equivalent to a cohesionless material of ϕ=35 deg.

The steps in the computer program for the design and cost analysis of the Cantilever wall are given in Appendix 2. The parameters used in the calculation are fed in, in Steps 1 and 2. It should be noted that the unit costs selected for concrete, labour and shuttering are higher than the corresponding values for the Gravity wall.

Steps 4 and 5 consist of the computation of earth pressure on a vertical plane through the heel. The overall dimensions of the Cantilever wall are then determined in Steps 6 to 8. In step 8(b), the dimension "a" of the base slab (see Fig.7) is calculated for a selected value of "b" from the formula given by Bowles (1968 - Fig. 7-6). This is an approximate formula for the resultant on the base slab to lie within the middle third. The "no tension' condition is checked in Steps 9 and 10.

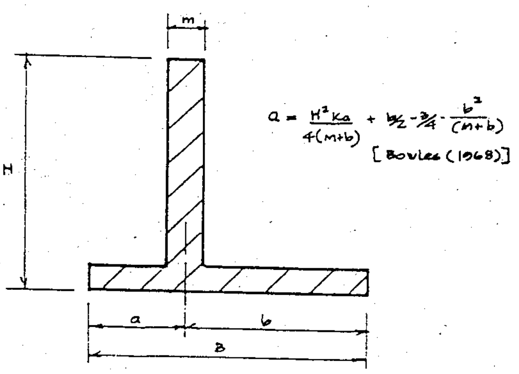


FIG.7

The calculation of the Factors of Safety against overturning, sliding and bearing are done in Steps 11 to 13. If the factors of Safety against overturning and bearing are found to be insufficient, the length of the toe is increased and the calculation repeated. But in the case of sliding, no change in the overall dimensions are made because the resistance to sliding can be increased by the provision of a heel projection.

The necessary steel reinforcements are then determined in Steps 14 to 22. The stress distributions used in the analysis are given in Figs. 8 and 9, while details of the bending reinforcements are shown in Figs. 10(a) and 10(b). In the last Step, the cost of steel, concrete, labour and shuttering per foot length of the wall are determined separately, and hence the total cost of the wall obtained.

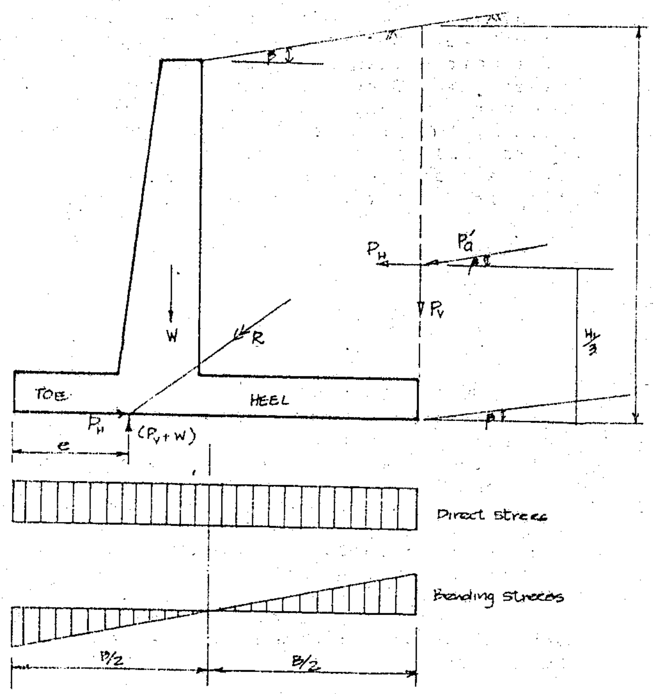


FIG.8

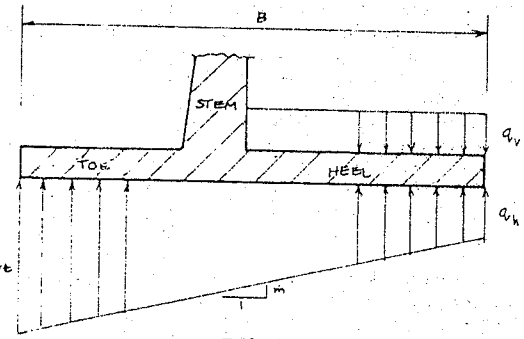


FIG.9

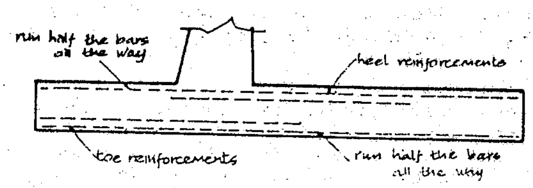


FIG. 10(a)

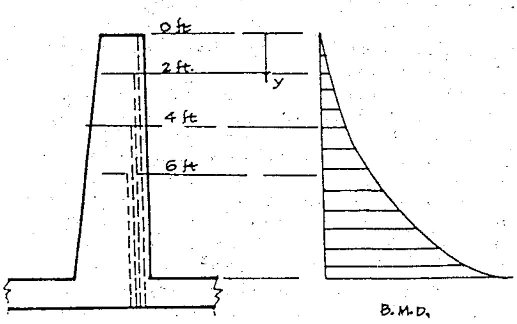


FIG. 10(b)

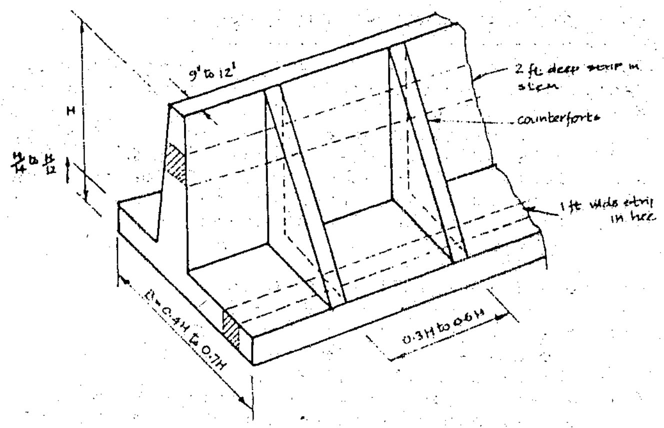


FIG. 11

COUNTERFORT RETAINING WALLS

Typical dimensions for a Counterfort wall are shown in Fig.11. The dimensions for the section of the Counterfort wall are calculated exactly as for a Cantilever wall. The differences in design lie in the calculation of the steel reinforcements in the stem wall and in the heel slab, and in the design of counterforts. An exact analysis of the stem wall would require the application of "plate theory", where the stem wall will have to be considered as a plate fixed at three edges and free at the fourth edge. A simplified method of design which has been used in this paper, is to consider the wall as a series of 2 ft.(0.6096 m.) deep strips which are continuous over the counterfort supports as shown in Fig.11. Nominal vertical steel is also provided at the front and back of the stem. (This method generally results in the wall being overdesigned.)

Similarly, the heel slab is designed as a series of 1 ft.(0.3048 m.) wide strips continuous over the counterfort supports. The stress distributions used in the design of the toe and heel slabs are shown in Fig.12.

The counterforts have been designed using a simple, but conservative method. The counterforts with the slab can be thought of as a very deep "T-beam". The inclined bars in Fig.13 are designed to carry the bending moment, and the horizontal tie bars are designed to carry the horizontal thrust against the wall.

The steps in the computer program are given in Appendix 3. The method of analysis is as stated above, with the additional criterion that the spacing of the counterforts has been varied until the minimum cost was obtained.

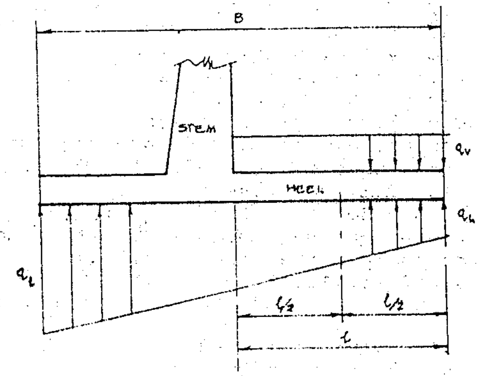


FIG. 12

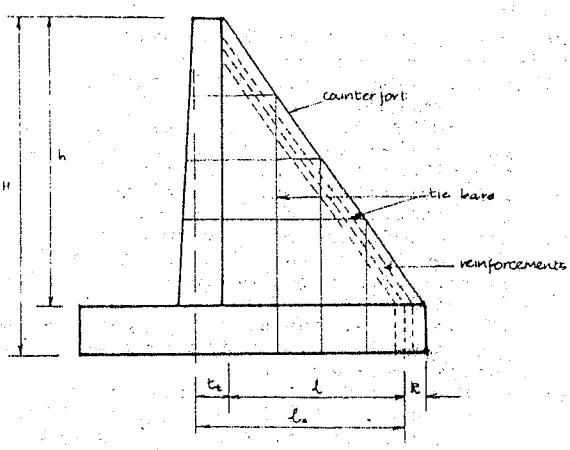


FIG. 13

COST COMPARISON OF RETAINING WALLS

The costs per foot length of the Gravity, Cantilever and Counterfort retaining walls for different heights of wall when ultimate bearing capacities are 8,6,4 and 2 tonf/sqft. (857, 643, 429 and 214 kN/sqm.) respectively are plotted in Figs.14 to 17. These diagrams show the following results for the soil parameters which have been assumed:-

For qD = 8 tonf/sqft.(857 kN/sqm.)

(i) the Gravity wall is about 50% more economical than the other two types for a height of 6 ft.(1.829 m.)

(ii) as the height increases, this percentage drops, and at a height of about 14-15 ft. (4.267 m.- 4.572 m.) all three retaining walls cost about the same.

(iii) beyond a height of 16 ft.(4.877 m.), the cost of the Gravity wall escalates much higher than the other two walls, and at a height of 30 ft.(9.144 m.) the Gravity wall is about 75% costlier than the other two walls (which are of equal cost at this height), and keeps on increasing beyond this height.

(iv) Cantilever wall is slightly more economical than the Counterfort wall for a height range of 6 to 30 ft. (1.829 m. to 9.144 m.)

(v) beyond 30 ft.(9.144 m.) height, Conterfort wall becomes a little more economical than the Cantilever wall, and at 40 ft.(12.192 m) height is about 6.5% less costly than the Cantilever wall.

For qD = 6 tonf/sqft.(643 kN/sqm.)

(i) to (v) same as for qD = 8 tonf/sqft. (857 kN/sqm.)

(vi) cost escalation of Gravity wall accelerates further from a height of 32 ft.(9.754 m.) upwards as the width of base has to be increased to satisfy the Factor of Safety against bearing.

For qD = 4 tonf/sqft.(429 kN/sqm.)

(i) to (iv) same as for qD = 8 tonf/sqft. (857 kN/sqm.)

(v) beyond 30 ft.(9.144 m.) height, Counterfort wall becomes a little more economical than the Cantilever wall, and at 40 ft.(12.192 m.) height is about 6.0% less than the Cantilever wall.

(vi) cost escalation of Gravity wall accelerates further from a height of 24 ft.(7.315 m.), [which is at a lower height than for qD = 6 tonf/sqft.(643 kN/sqm.)], as the width of base has to be increased to satisfy the Factor of Safety against bearing.

(vii) cost escalations of Cantilever and Counterfort walls also accelerate from a height of 30 ft.(9.144 m.) upwards as the width of base has to be increased to satisfy the Factor of Safety against bearing.

For qD = 2 tonf/sqft.(214 kN/sqm.)

(i) and (ii) same as for qD=8 tonf/sqft. (857 kN/sqm.)

(iii) beyond a height of 16 ft.(4.877 m.) the cost of the Gravity wall escalates much higher than the other two walls.

(iv) cost escalation of gravity wall accelerates from a height of 8 ft. (2.438 m.) upwards as the width of base has to be increased to satisfy the Factor of Safety against bearing.

(v) cost escalation of Cantilever and Counterfort walls also accelerate from a height of 18 ft.(5.486 m.) upwards as the width of base has to be increased to satisfy the Factor of Safety against bearing.

(vi) beyond a height of 22 ft.(6.706 m.) the Gravity wall requires very wide foundations which are not practical.

(vii) Cantilever wall is slightly more economical than the Counterfort wall for a height range of 6 to 30 ft. (1.829 m. to 9.144 m.)

(viii) beyond a height of 34 ft.(10.363 m.) both Cantilever and Counterfort walls also require very wide foundations which are not practical.

COST COMPARISON 1988 vs. 1973

Fig. 18 is a reproduction of a diagram showing total cost against height for qD=8 tonf/sqft. (857 kN/sqm.), presented by Tennekoon and Hemachandra (1973) assuming the same soil parameters which are considered by the Author.

The following observations could be made when Fig.14 is compared with Fig.18:-

(i) the total costs of the three types of walls have gone up by about 10 to 12 times during the period 1973 to 1988.

(ii) however, the relative cost structure of the three walls have not changed appreciably.

(iii) the "equal cost" factor is at a wall height of about 15 ft.(4.572 m.) in Fig.14 (i.e. in 1988) and is at a wall height of 16 ft.(4.877 m.) in Fig.18 (i.e. in 1973).

CONCLUSIONS

(i) For the soil parameters which were selected, (these could be considered as being typical of some of the soils in this country), the Gravity wall is shown to be the most economical upto a height of about 14 ft.(4.267 m.)

(ii) For heights from 14 to 17 ft.(4.267m to 5.182 m.) there is not much difference in cost among all three walls.

(iii) From a height of 17 ft.(5.182 m.) to about 30 ft.(9.144 m.), both Cantilever and Counterfort walls are equally more economical than the Gravity wall.

(iv) Beyond a height of 30 ft.(9.144 m.), Counterfort walls have been shown to be the most economical form of construction.

(v) When a soil having a low Ultimate Bearing Capacity is encountered as the foundation soil, there would be certain physical limitations to constructing retaining walls beyond a certain height. With a UBC of 2 tonf/sqft.(214 kN/sqm.), this height is about 22 ft.(6.706 m.) for Gravity walls and about 34 ft.(10.363 m.) for Cantilever and Counterfort walls.

(vi) Inflationary increase in cost of building material and labour, even over a long period, do not greatly affect the above conclusions.

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APPENDIX 1

Steps in the computer program for the cost analysis of Gravity Retaining Walls.

1. Values of Parameters used:

(i) density of wall material (γ_w)=150 lb/cuft
(ii) ult.bearing cap.of soil (qD)=8,6,4,2 tonf/sqft.
(iii) cost of 1:3:6 concrete (R1)=Rs.4000.00 per cube
(iv) cost of labour (R2)=Rs. 600.00 per cube
(v) cost of shuttering (R3)=Rs. 750.00 per square

2. Further parameters read in:

(i) height of wall (h)=6,8,...40 ft
(ii) angle of shearing resist.(ϕ)= 28 degrees
(iii) cohesion (c)=150 lbf/sqft
(iv) adhesion (ca)=150 lbf/sqft
(v) slope angle of backfill (β)= 0 degrees
(vi) angle of wall friction (δ)= 20 degrees
(vii) density of backfill soil (γ)=110 lb/cuft

3. ϕ, β, δ are converted to radians.

4. Top width of wall (D1) = 1/12 * height of wall, and select to the nearest 3", but with a minimum of 9".

5. Initially, width of base (B) = 1/3 * height of wall, and select to the nearest 3".

6. Referring to Fig.1(b), calculation of wall parameters D, α and y using geometry.

7. Calculation of forces on soil block in Fig.2(b).
x = horizontal distance from top of the wall to the intersection of assumed failure surface with the free surface.

Inclination of assumed failure surface to horizontal,
$(\theta) = \tan^{-1}[(x*\tan\beta + h)/(x - D)]$

Cohesive force (C) = $c(x-D)/\cos\theta$
Adhesive force (Ca)= ca*y
Weight of soil block (W) = $\gamma*x*(h+D*\tan\beta)/2$
Force on wall,

$$(Pa) = \frac{[W*\sin(\theta-\phi) - C*\cos\phi + Ca*\cos(\theta-\phi+\alpha)]}{\sin(\alpha+\theta-\delta-\phi)}$$

The maximum value of (Pa) is obtained by varying "x" in steps.

8. Computation of the weight of the wall (Ww), and the position of its centre of gravity.
From Fig.3, (Ww)= γ_w*h*(D1+B)/2

$\bar{x}$ = horizontal distance from toe to the centre of grav.
$= [D1^2 + D(D1+D/3)]/(D1+B)$

9. Computation of horizontal force (Ph) and vertical force (Pv) acting on wall face.

$$Ph = Pa*\sin(\alpha - \delta) - Ca*\cos\alpha$$

$$Pv = Ww + Pa*\cos(\alpha - \delta) + Ca*\sin\alpha$$

10. Determination of the eccentricity (e) on base (see Fig.3).
For overturning about the toe,
Stabilizing moment

$$(SM)=Ww*\bar{x} + Pa*\cos(\alpha - \delta)*[D1+2D/3] + y*Ca*\sin\alpha*[D1+D/2] + y*Ca*\cos\alpha*h/2$$

Overturning moment $(OM)=Pa*\sin(\alpha - \delta)*h/3$

Eccentricity $(e) = (SM - OM)/Pv$

11. If e < B/3, increase B by 3" and go back to Step 6.

12. Factor of safety against overturning
$(FSO) = SM/OM$

If (FSO) < 1.5, increase B by 3" and go back to Step 6.

13. Calculation of factor of safety against sliding (FSS).
Frictional force on base,
$(F)=Pv*\tan\phi$ for $\phi < 27$ degrees
$=0.5$ Pv for $\phi > 27$ degrees

$(FSS) = F/Ph$

If (FSS) < 1.5, increase B by 3" and go back to Step 6.

14. Calculation of factor of safety against bearing (FSB)
From Fig.3, direct stress $(fd) = Pv/B$
bending stress $(fb) = 3Pv*(B-2e)/B^2$

$(FSB) = qD/(fd+fb)$

If (FSB) < 2.0, increase B by 3" and go back to Step 6.

15. Volume of material per foot length of wall
$(V) = (D1 + B)*h/2$

16. Area of shuttering per foot length of wall
$(A) = 2h$

17. Calculation of cost of material, labour, shuttering and total cost.

APPENDIX 2

Steps in the computer program for the cost analysis of Cantilever Retaining Walls.

1. Values of Parameters used:

(i) all.comp.stress in conc.(pcb)=1000 lbf/sqin.
(ii) all.shear stress - conc.(pcs)= 100 lbf/sqin.
(iii) all.tensile stress-steel(pst)=20,000 lbf/sqin.
(iv) thickness of cover (tc)= 1.5 in.
(v) dia.of steel reinforce. (Dia)= 0.5 in.
(vi) density of concrete (γ_c)=150 lb/cuft
(vii) ult.bearing cap.of soil (qD)=8,6,4,2 tonf/sqft.
(viii) cost of steel reinforce.(R1)=Rs.1000.00 per cwt.
(ix) cost of 1:2:4 concrete (R2)=Rs.5400.00 per cube
(x) cost of labour (R3)=Rs. 600.00 per cube
(xi) cost of shuttering (R4)=Rs.1250.00 per square

2. Further parameters read in: -

(i) height of wall (h)=6,8,...40 ft
(ii) angle of shearing resist.(ϕ)= 35 degrees
(iii) slope angle of backfill (β)= 0 degrees
(iv) density of backfill soil (γ)=110 lb/cuft

3. ϕ, β are converted to radians.

4. Referring to Fig.5, inclination of α-characteristic to vertical

$(\theta_1) = 1/2(\pi/2-\phi) - 1/2(\Delta - \beta)$
where $\sin\Delta = \sin\beta/\sin\phi$

5. Coefficient of active earth pressure,

$$(Ka) = \cos\beta * \frac{[\cos\beta - \sqrt{(\cos^2\beta - \cos^2\phi)}]}{[\cos\beta + \sqrt{(\cos^2\beta - \cos^2\phi)}]}$$

6. Calculation of stem dimensions:-

Force acting on stem $(Pa)=\gamma*h^2*Ka/2$, parellel to slope.
Shear force on base of stem $(Q)=Pa*\cos\beta$
Thickness of base of stem $(tbs)=Q/(12pcs)$

The thickness of the top of the stem is calculated to have a value between 9" and 12" using a minimum batter of 1/4 inches per foot. If the thickness at the top is less than 9", the thickness at the base of the stem is increased so as to have the minimum thickness of 9" at the top.

7. Initially, thickness of base slab (tbb)= 1/10 * height of wall, and selected to the nearest 3" with a minimum value of 12".

8. Computation of lengths of base, heel and toe (B, b and a of Fig.7).

(a) Initially take B = 0.4 * height of wall.
(b) Take initial value of b = 0.6 * B
(c) Calculate "a" from the formula

$$a = \frac{H^2 * Ka}{4(m+b)} + \frac{b}{2} - \frac{3}{4} * \frac{b^2}{(m+b)}$$

(d) determine point of intersection of α - characteristic from heel with stem of wall, i.e. H_2 in Fig.6.

If $H_2 > 0.4H_1$, increase b by 12" and go back to Step 8(c).

9. Referring to Fig.8,

Force on vertical plane through heel

$(Pa') = \gamma * H_1^2 * Ka/2$
where H_1 = height of soil above extreme of heel.
Pa' has horizontal and vertical components

$P_H = Pa' * \cos\beta$, and $P_V = Pa' * \sin\beta$

For overturning of wall about toe,

Stabilizing moment (SM) = $W*\bar{x} + P_V*B$

where W = weight of wall
and $\bar{x}$ = distance of centre of gravity of wall from toe.

Overturning moment (OM) = $P_H * H_1/3$

Eccentricity (e) of resultant = $(SM - OM)/(W + P_V)$

10. If e < B/3 , increase b by 12" and go back to Step 8(c)

11. Factor of Safety against Overturning, (FSO) = SM/OM

If FSO < 1.5 , increase b by 12" and go back to Step 8(c).

12. Determination of Factor of Safety against Sliding (FSS).
Frictional force on base,
(F) = $(P_V + W)*\tan\phi$ for $\phi < 27$ deg.
= $0.5(P_V + W)$ for $\phi > 27$ deg.

FSS = F/P_H

13. Determination of Factor of Safety against Bearing (FSB).
Referring to Fig.8,
Direct Stress (fd) = $(P_V+W)/B$
Bending Stress (fb) = $3(P_V+W)*(B-2e)/B$

FSB = qD/(fd + fb)

If FSB < 2.0, increase "a" by 12" and go back to Step 9.

14. Computation of shear force and bending moment at critical sections of base.
Referring to Fig.9, the critical locations for the toe and heel are at the junctions to the stem.

15. (a) Check for bending and shear in 'toe' portion of base slab.
(b) Check for bending and shear in 'heel' portion of base slab.

If in either case, base thickness (tbb) is found to be insufficient, increase 'tbb' by 3" and go back to Step 9.

16. Calculate steel areas required in toe and heel to carry bending stresses.

17. Calculate total volume of steel in base, with the curtailment of bars carried out as shown in Fig.10(a).

18. For the stem, calculate the bending moment at intervals of every 2 ft. along the stem [see Fig.10(b)]. Determine the volume of steel.

19. Determine volume of tensile steel in stem continued into the base.

20. Minimum longitudinal steel for shrinkage = 0.30% of sectional area of wall (CP 2).

21. Minimum vertical distribution steel in front of stem = 0.5" dia. bar per foot length of wall.

22. Calculate total weight of steel per foot length of wall.

23. Calculate separately the cost of steel, concrete, labour and shuttering; and hence the total cost.

APPENDIX 3

Steps in the computer programme for the cost analysis of Counterfort Retaining Walls.

Steps 1 to 17 are the same as for the Cantilever Wall. (see Appendix 2).

18. Select initial spacing of counterforts, (z) = 1/4 * height of wall.

19. Calculate longitudinal steel in héel slab, assuming the slab to consist of 1 ft. wide strips continuous over the counterfort supports. (see Fig.11).

20. Calculate longitudinal steel in wall stem, assuming that the wall stem consists of 2 ft. deep strips continuous over the counterfort supports. (see Fig.11).

21. Minimum vertical steel in stem (front and back) = 2 Nos. 0.5" diameter bars per foot length of wall.

22. Minimum longitudinal steel for shrinkage in toe = 0.30% of sectional area of toe (CP2).

23. Design of counterforts. (see Fig.13).

   (a) Select thickness of counterforts = 1/25 * height of wall, with a minimum value of 9 inches.
   (b) Total force on wall per counterfort, (Qr) = $Ka * \gamma * h^2 * \cos\beta * Z/2$ acting at a depth of 2h/3 from top of wall parallel to backfill.
   (c) Inclined steel in counterfort designed to carry bending moment due to "Qr".
   (d) Horizontal tie bars designed to resist horizontal component of "Qr".
   (e) Vertical tie bars = 2 Nos. 0.5" dia. bars per foot.

24. Determine cost of steel, concrete, labour and shuttering; and hence the total cost.

25. Increase spacing of counterforts by 1 foot and repeat Steps 19 to 24 until minimum cost is obtained.

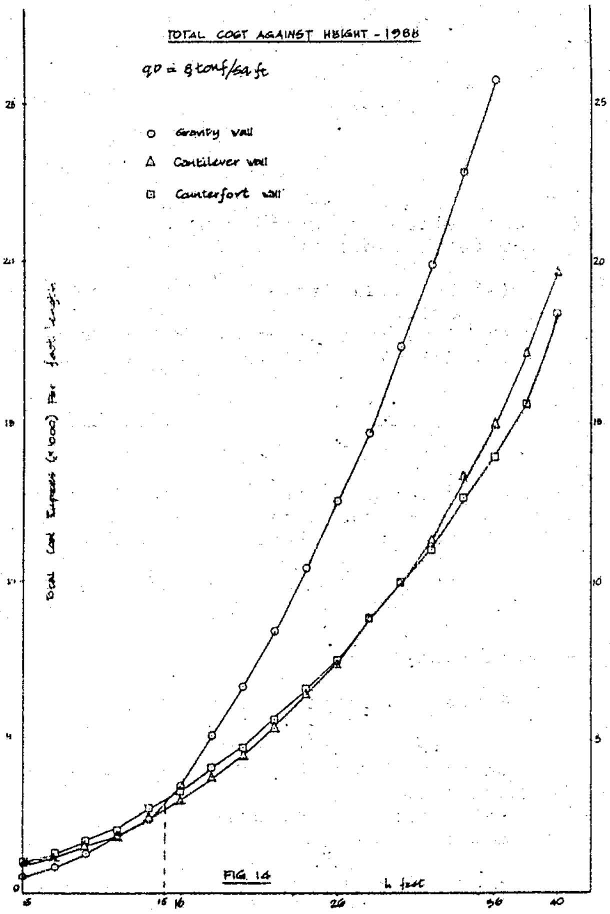


FIG 14

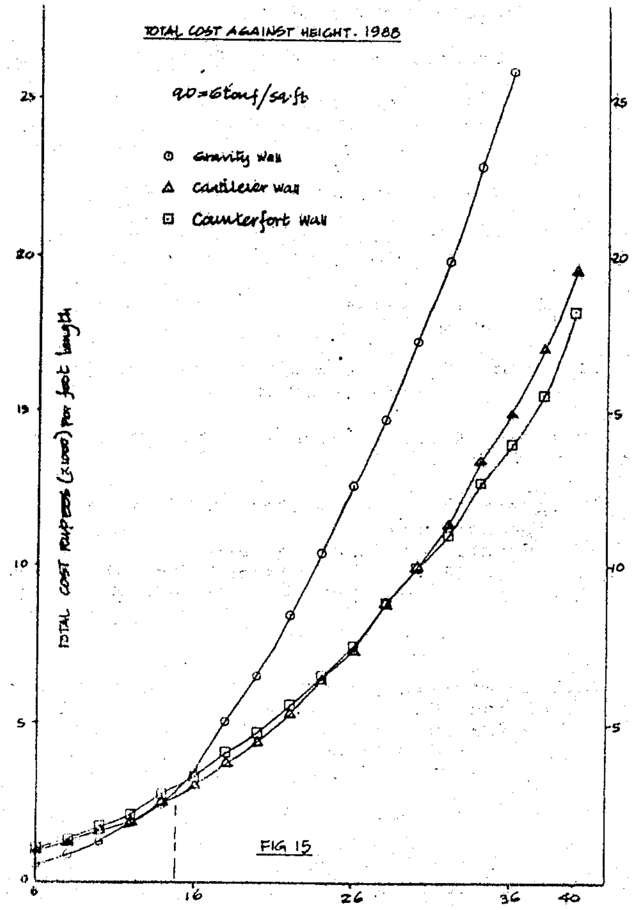
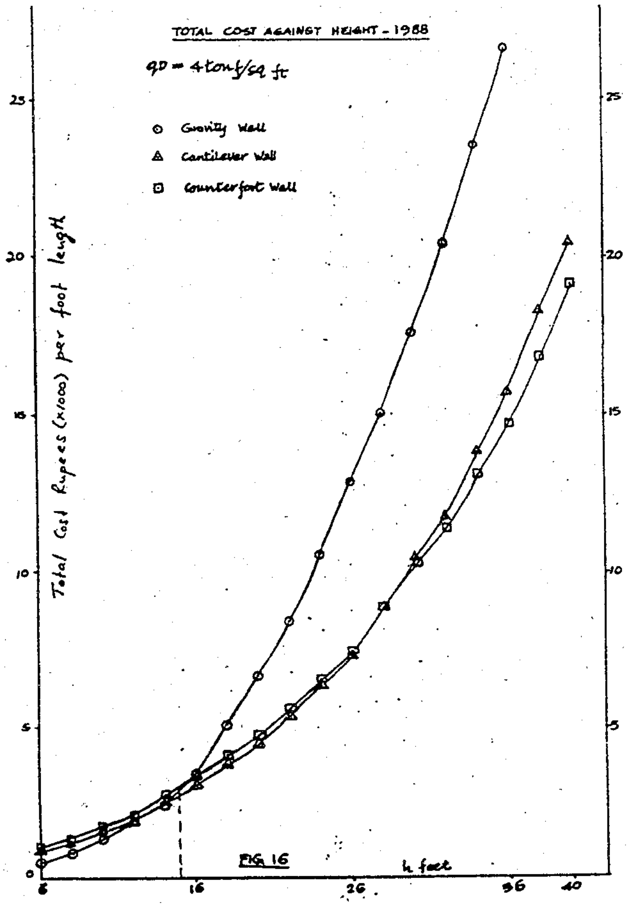


FIG 15



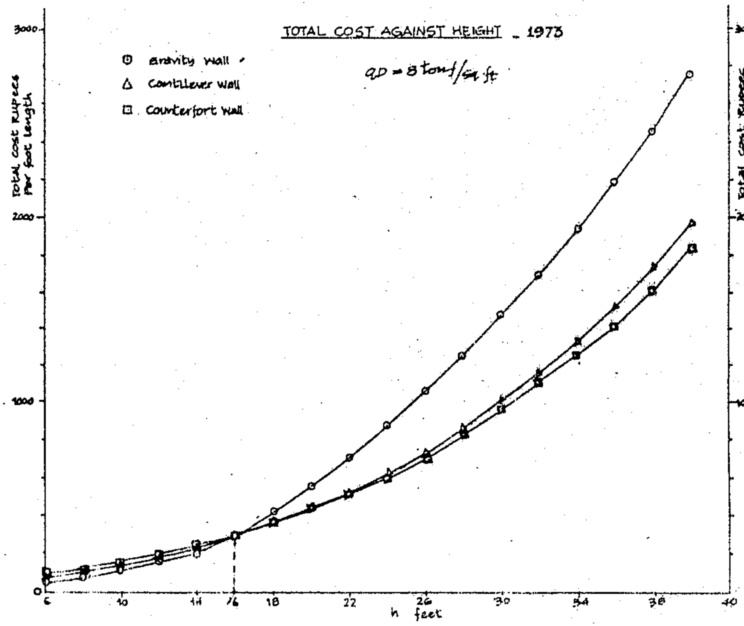
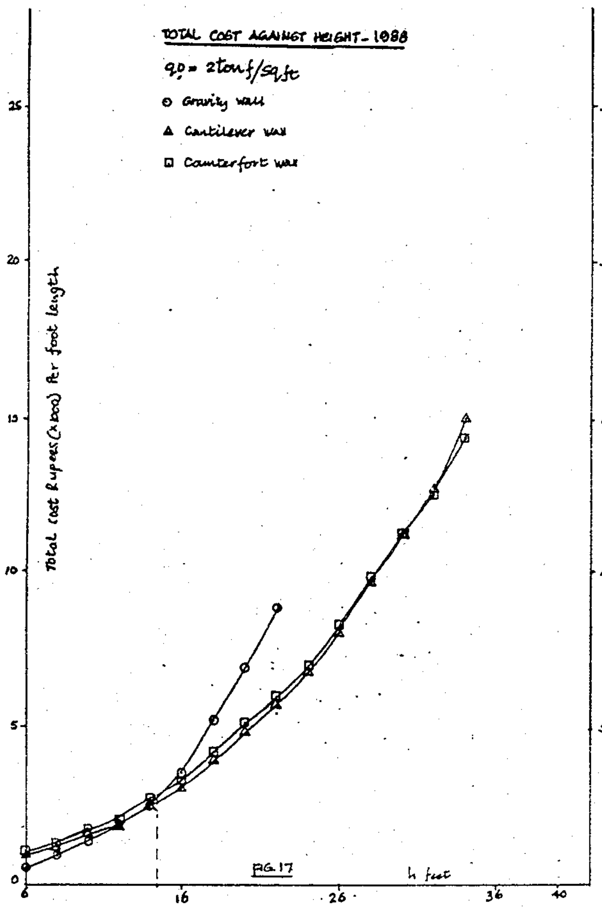


FIG. 18



Rates applicable in 1973

Gravity wall

1:3:6 CONCRETE - Rs. 300 Per cube
Labour - Rs 200 " "

Cantilever and counterfort walls

Steel Rs. 100 Per cwt
1:2:4 CONCRETE Rs. 400 Per cube
LABOUR Rs. 300 Per cube

DESIGN AND CONSTRUCTION OF ANCHORS AND TIE-BACKS

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1. INTRODUCTION

Anchors and tie-backs are widely used nowadays for providing both temporary and permanent support to a wide range of structures. An anchor is basically a structural tension member used to resist an applied load by transmitting it to a distant (generally deeper) soil or rock strata. A tie-back is a special form of anchor used to support ground retaining structures such as sheet piling, soldier piles and, insitu diaphragm walls.
Anchorage systems are widely used for,
a) Holding down slabs subject to hydro-static uplift forces.
b) Support of pylons and towers subjected to overturning.
c) Strengthening of existing foundations.
d) Stabilization of slopes.
e) Support of temporary and permanent walls as tie-backs.
f) Support of wharf walls and other maritime structures.
This paper is confined to the design and construction of anchors used for earth retaining structures. As an example Fig. 1 illustrates the use of a multi-anchorage system in support of a deep excavation in Belgium.

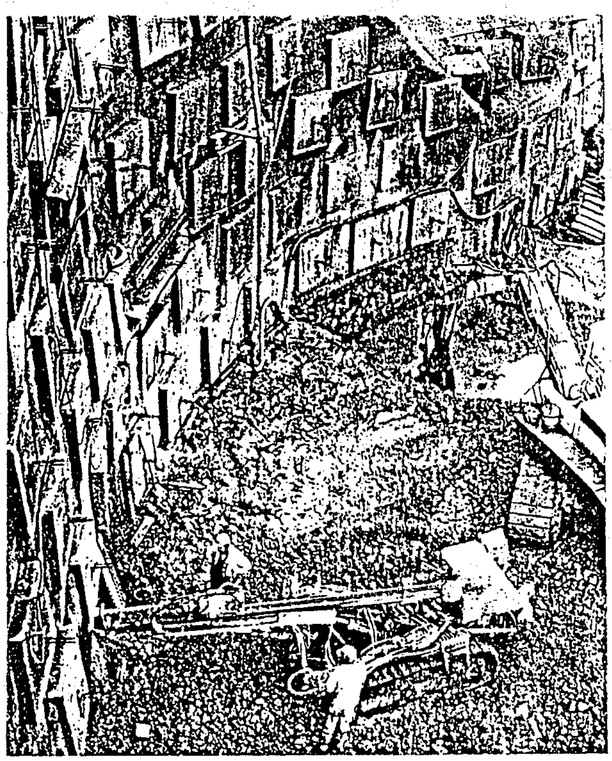


Fig. 1 A multi-anchorage system

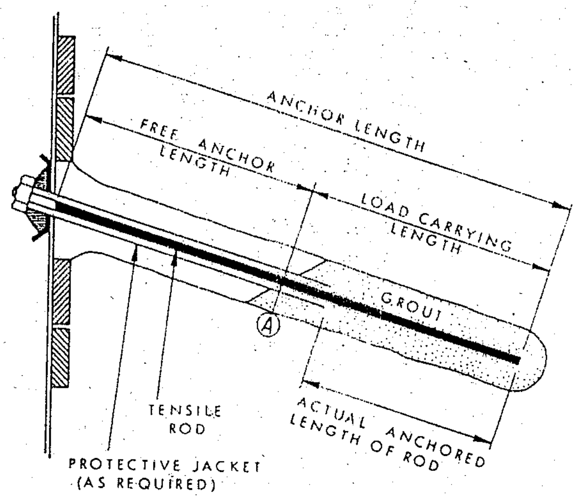


Fig. 2 Details of a typical anchor

2. CONSTRUCTION ASPECTS

2.1 installation

Fig. 2 illustrates a typical side loaded ground anchor. It is installed by drilling a borehole of suitable diameter, homing the loading tendon into the borehole and grouting the tendon in place. The borehole can be lined or unlined to suit the stability condition.
Tremie grouting is common in rock or very stiff to hard cohesive soils. In some cases the borehole is under-reamed at one or several places. Under-reaming is effective in stiff to hard cohesive soils. It utilises the end bearing capacity than side friction to resist pull-out forces.
Low pressure grouting via a lined tube or an insitu packer can be used to increase the effective diameter of the anchor above the borehole diameter. Low pressure normally implies injection pressures less than the total overburden stress at the anchor. The grout permeates through the pores or natural fractures causing minimum disturbance to the surroundings. Low pressure grouting is popularly used in soft fisured rocks and coarse alluvium. It can also be used in fine grained cohesionless soils where the borehole expands under pressure though the grout penetration through pores is negligible.
High pressure grouting is used to increase the effective diameter of the anchor by hydro-fracturing of the ground mass to produce an artificial root or fissure system for grout permeation. This method is primarily employed in cohesionless soils; some success is also reported in stiff cohesive soils.

2.2 grouting

Grouting is usually carried out in stages working backwards from the toe of the anchor. Tube a manchette is a popularly used stage grouting system. Where a casing is used it is gradually withdrawn as grouting proceeds. Care must be taken to ensure that high grout pressure would not cause damage to adjacent anchors and, other structures or services.
The grouts used vary with the technique and the system. Usually they are cement-based though in alluvial deposits more sophisticated chemical grouts are employed. In large bore-holes concrete can be used instead of grout. The grout compressive strength should be sufficiently high to carry the tendon load. Sufficient time, varying from one day to a week, must be allowed for the grout to set properly.

2.3 tendon system

The tendon usually consists of 12 to 16 mm diameter high tensile single, multistrand or bundled multistrand prestressing strands. A factor of safety of 1.6 (temporary anchors) or 2.0 (permanent anchors) is normally used in the design.
Tendons are protected against corrosion. The degree of protection varies according to the working life of the anchor , the environment and the possible damage to the protection system during installation. Temporary anchors are normally protected only over the free length by grease covered with plastic tape. Permanent anchors and temporary anchors in corrosive environments are protected over the whole length using grease and plastic materials.
Post-tensioning is carried out after curing of the grout by applying a controlled tension pull on the tendon using a hydraulic jack. The techniques of loading vary particularly with respect to the rate of loading and stressing beyond the working load (see sections 3.3 and 3.5). The movement of the top of the anchor is recorded along with the applied load. Post-tensioning stage is very important as it provides a method of assessing the workmanship and a means of confirming the capacity of the individual anchor to perform its function.

3. DESIGN OF SINGLE ANCHORS

3.1 introduction

For the design purposes, anchors may be divided into three groups based on ground conditions namely,
a) cohesive soils
b) non-cohesive fine or coarse grained soils
c) soft or hard rocks.
The other factors affecting the anchor design are effectiveness of installation, dimensions of the anchor system and mechanical properties of the anchor components.
The anchor dimensions are also influenced by ground conditions. The load carrying capacity of an anchor can be increased by increasing the length or the 'effective diameter'; effective diameter is greater than the bore-hole diameter in cases in which the borehole expands or grout permeates into the surroundings during installation.
There are many empirical or semi-empirical methods available for the design of single anchors. However, they are mostly based on a particular anchor system than on fundamental concepts.
Also the carrying capacity of an anchor is influenced substantially by the construction techniques and the quality of workmanship. Fortunately however, it is possible to test load the anchors during post-tensioning stage at a low additional cost. This enables the designer to rectify any shortcomings of the design at the construction stage.
Design methods discussed here are for temporary anchors. For permanent anchors a larger factor of safety, at least 30% higher than what is given here is required.

3.2 Allowable anchor loads in soils and/rocks

From limit analysis it can be shown that the pull-out capacity (T_c) of an anchor is given by,
$T_c = \pi.D.L.\tau_{ult}$ where,
D is the effective diameter of the anchor, L, fixed anchor length and, τ_{ult} ultimate bond or skin friction at the rock(soil)/grout interface. Note the followings:
a) Trnsfer of load from the fixed length of the anchor to the ground is assumed to produce a uniformly distributed stress over the whole perimeter.
b) In sound rock the effective diameter is equal to the borehole diameter. However, in soft rocks or in soils when pressure grouting is carried out, the effective diameter increases due to grout permeation into surrounding ground and/or due to borehole expansion. The effective diameter in these cases should be estimated from the grout intake during construction and the porosity of the ground.
c) The end bearing resistance of the anchor is ignored here. However, in designing under--reamed anchors and also in cases where the effective diameter is considerably larger than the borehole diameter, the end bearing capacity should also be considered. In fact for under-reamed anchors end bearing resistance is more dominant than the side friction component.
For cohesionless soils T_c can be estimated from the equation,
$T_c = \sigma'_z.\pi.D.L.K_f$ where,
σ'_z is the effective vertical stress at the mid-point of the load carrying length and K_f, an anchorage coefficient dependent on the soil type and condition as given below.

VARIATIONS IN K_f

Soil Type	Condition		
	Loose	Compact	Dense
Silt	1	4	10
Fine Sand	1.5	6	15
Medium Sand	5	12	20
Coarse Sand, Gravel	10	20	30

A more accurate determination of T_c should involve the shear strength parameters of the soil.

However, any form of analysis may be only approximate due to disturbance to the ground during installation and also due to the difficulties in accurate determination of D. It may be prudent to use a simple analysis until gaining experience with a particular system of anchors.
Anchors are not recommended in soft or firm clays (undrained shear strength < 50 kPa.) or in sensitive clays because of the large deformation which can occur, both at and subsequent to loading. In stiff to very hard cohesive soils the anchor load capacity can be estimated from the equation,
$T_c = \pi D.L.c_u.\alpha$ where,
c_u is the undrained shear strength of the soil over the fixed length and, α , the coefficient of adhesion. A design relationship between α and c_u is given in Fig. 3.
Allowable anchor loads in soils are determined by dividing the calculated anchor load capacity, T_c, by a factor of safety. In cases where no pull-out tests are carried out use a minimum factor of safety of 3.
Anchorage design in sound rock is based on an allowable grout to rock bond stress s_b acting over the fixed anchor length. s_b should not exceed either of the following;
a) 1/30 of unconfined compressive strength of the surrounding rock,
b) 1/30 of unconfined compressive strength of the grout,
c) 1300 kPa.
Using this criteria the allowable anchor load T_a is given by the equation,
$T_a = \pi D.L.s_b$. In soft or weathered rocks, Standard Penetration Test Value (N) may be useful in the design. Based on data from Japan on weathered granite an allowable bond stress $s_b = 0.0023N + 0.04$ (N/mm^2) is recommended.

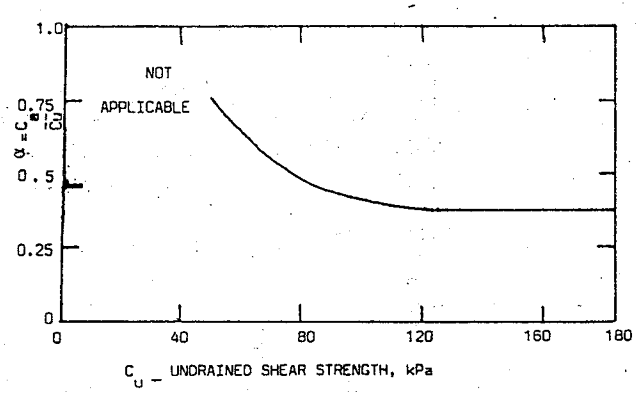


Fig. 3 Relationship between α and c_u

3.3 Anchor load capacity from pull-out tests

Where the load capacity of anchors are to be determined by pull-out tests, at least one anchor in ten of those actually used in the project, with a minimum of three in each soil or rock type encountered, be tested. The pull-out capacity is defined as the load at which withdrawal of the anchor begins. If this load is not clearly apparent from the test data, the pull-out capacity is taken as the maximum load at which withdrawal is still tolerable for the structure. If an ultimate capacity is not reached, or no withdrawal is observed in the test loading, the greatest applied test load should be assumed as the pull-out capacity of the anchor.
The allowable anchor load is determined by dividing the pull-out anchor load by a factor of safety. Required mimimum values of factor of safety vary between 1.5 - 2.0 depending upon the anchor inclination (see Fig. 4); linearly interpolate from the figure for in between inclinations.

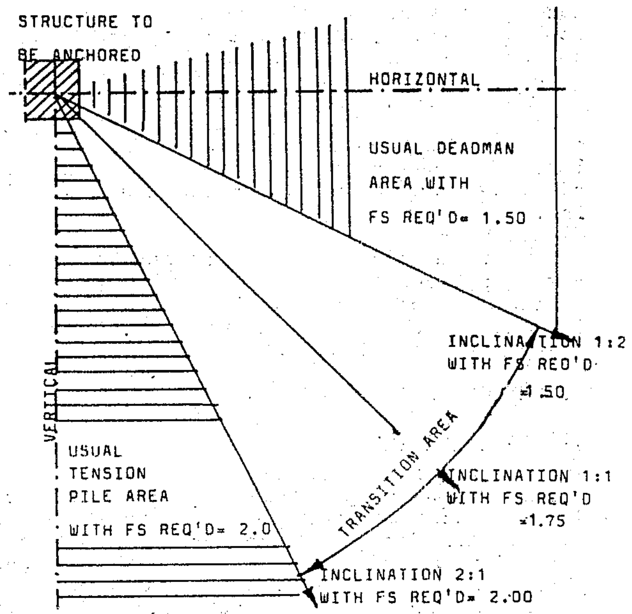


Fig. 4 Required factor of safety

3.4 dimensional requirements and location

The minimum fixed anchor length used, L is 3 m. The generally accepted fixed length to diameter ratio is 5 in cohesive soils and 10 in cohesionless soils. In sound rocks or in cohesive soils under low pressure or in tremie grouting, the effective diameter, D, is essentially equal to the borehole diameter, d. In other cases D depends on grout permeation ,local compaction and borehole expansion. Typically observed D/d ratios are, 3-4 in coarse sands and gravels with grout permeation, 1.5-2 in medium dense sand and 1.2-1.5 in very dense sand with no permeation and local compaction only. D can be approximately estimated by porosity measurements of the surrounding soil and by grout intake.

The depth of overburden above any anchor should not be less than 5 m in soil or in weathered rock and 1.5 m in sound rock. When multiple anchors are used the minimum horizontal or vertical spacing between them should be 4D. In anchored walls all anchors should be at least 0.15 times the wall height, distance away from the classical Coulomb failure plane for the wall without anchors (see Fig.5).

3.5 stressing and proof loading

Each installed anchor should be stressed and proof loaded at least upto 1.33 times the allowable or design working load for the anchor. The following procedure can be used for proof loading.

a) Test load the anchor to 80% of the ultimate tensile strength of the tendon, maintain the load for five minutes and then unload completely. The working load should not exceed 60% of this ultimate tensile strength.

b) Restress the anchor to 110% of the required working load while recording the tendon movement at the ram as the load is incrementally applied. The load extension graph obtained during this loading should compare closely with that estimated for only the free length of the tendon. Lock off the anchor at this load.

c) Check the anchor after 15 minutes. If a loss of pre-stress in excess of 5% is recorded, restore to 110% working load by shimming.

d) Repeat c)

e) If a further loss of pre-stress is recorded, reduce the anchor load until creep ceases. A safe working load for the anchor is then equal to 60% of the load showing no creep after 15 minutes.

4. DESIGN OF ANCHORED WALLS

4.1 introduction

Anchoring of retaining walls or diaphragm walls, either permanently or temporarily during construction, is very frequent nowadays. The design of anchors in such cases will depend on the actual earth pressures which act on the wall. These earth pressures depend on many factors including wall stiffness relative to the surrounding soil, anchor spacing, anchor yield and the pre-stress locked into the anchors at installation.
The use of Rankine's and Coulomb's methods of analysis is described here. However, if the installation and deformation conditions of the wall approximate those for an strutted excavation, the anchors should be designed using pressure distribution diagrams for strutted excavations. The anchor groups should also satisfy the dimensional and space requirements given in the last section. The design capacity of the anchors should be verified either by test loading to failure or at least by proof loading as described before for the case of single anchors.

4.2 Rankine analysis

Fig. 6 illustrates the use of Rankine's method in calculating the anchor forces. The pressure diagrams are assumed to be triangular in form

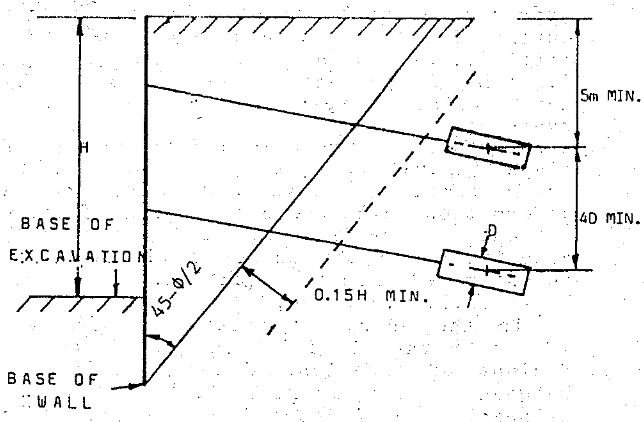
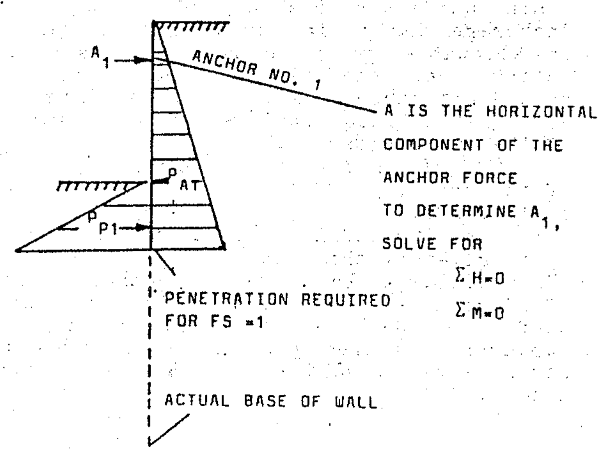
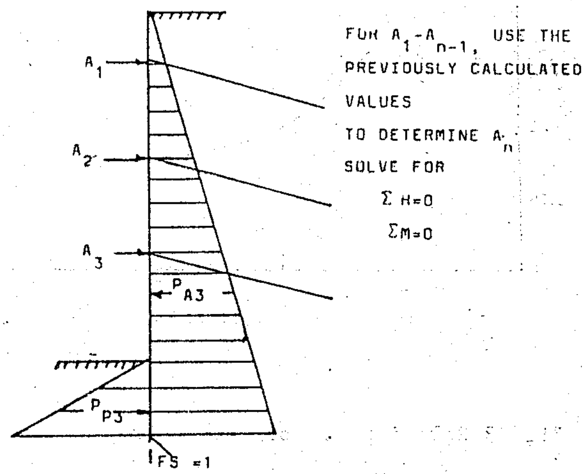


Fig. 5 Minimum spacing and depth of anchors



(a) ANALYSIS FOR FIRST ANCHOR



(b) ANALYSIS FOR INTERMEDIATE ANCHORS

Fig. 6 Rankine's analysis of anchor forces

as in the classical case. Active pressures acting on the wall are calculated using the coefficient of active earth pressure K_A if moderate wall movements are permitted. K_A is also used when foundations of buildings and services exist at depths below the base of the wall or at shallow depths but at distances greater than the height of the wall. If foundations of buildings or services exist at shallow depth at, a distance less than 0.5H (H, height of the wall), active pressures should be computed using the coefficient at earth pressure at rest K_0. For in between cases use a coefficient of earth pressure equal to $0.5(K_A + K_0)$.
Passive pressure should be computed using a reduced coefficient of passive earth pressure with a factor of safety not less than 1.5. In cohesive soils total stress analysis using undrained shear strength with zero angle of internal friction may be used. However, wherever possible it is better to use effective stress analysis. In both cases the possiblity of tension cracks being filled with water should be considered. The design method which is illustrated in Fig. 6 is outilined in the following steps.

a) Determine the earth pressure diagrams allowing for the effect of surcharge and the water table.

b) Assume that the highest load on the n^{th} anchor occurs just before the installment of the (n+1) anchor and draw the excavation cross-section for that condition.

c) For all anchors other than the lowest, determine the depth of penetration of the wall required to establish a unit factor of safety against rotation using the pressure diagrams previously established . Use the anchor forces of previously installed anchors for this calculation.

d) Determine the required force in the n^{th} anchor for stability of the wall based on overall horizontal equilibrium.

e) For the next to lowest anchor check that the required depth of penetration as indicated by the analysis is in fact available.

f) For the lowest anchor take the depth of penetration at the proposed design value and calculate the anchor force from horizontal force equilibrium.

g) Check the bending moments that will develop in the wall at each stage of construction. Critical conditions will occur immediately before each anchor is installed.

h) In general, where the lowest anchor is more than a metre away from the bottom of the wall, the wall should penetrate below the base of the cut at least to the depth at which the computed resultant earth pressure is zero.

4.2 graphical analysis

Figs. 7 and 8 illustrate the graphical analysis of an anchored retaining wall in a one layer and two layer soil. The analysis follows the general Coulomb type of analysis used for the design of retaining walls. It is necessary to carry out this type of analysis for each anchor level of the anchoring system starting from the top anchor. At each level the required anchor force is the sum of all anchor forces above the relevant lower failure plane.

The location of failure planes for the three normally used anchorage systems are shown in Fig. 9. The graphical analysis is an alternative to Rankine's method and is capable of handling complex ground and loading conditions.

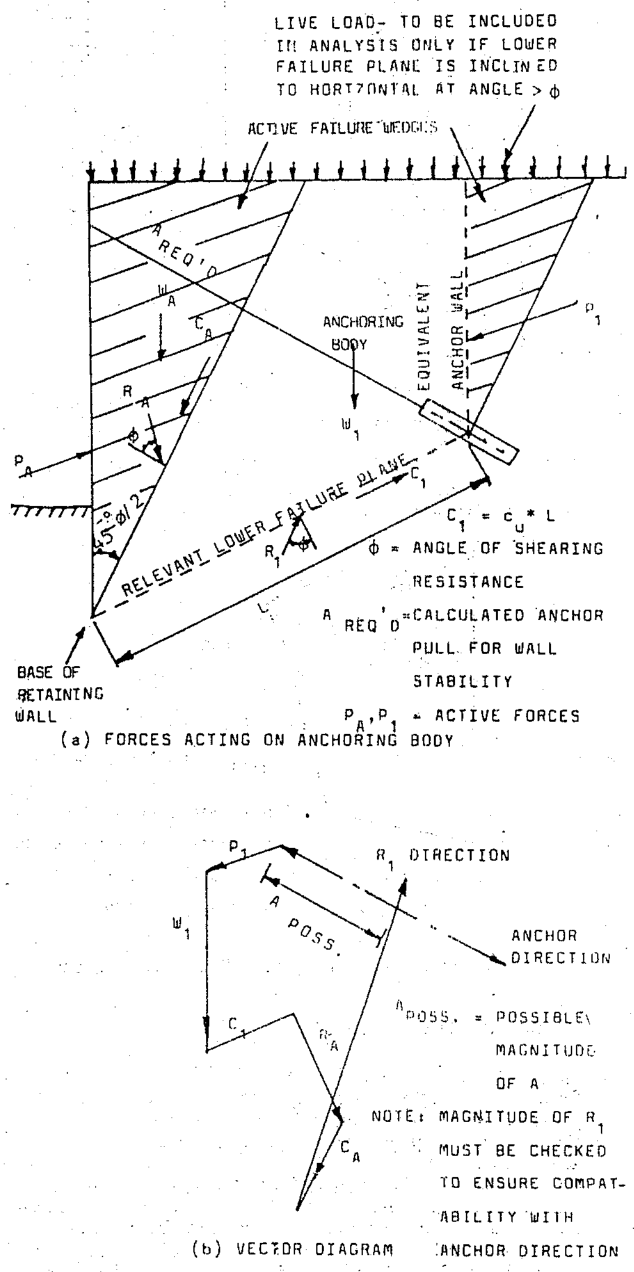


Fig. 7 Graphical analysis of anchored walls (single layer)

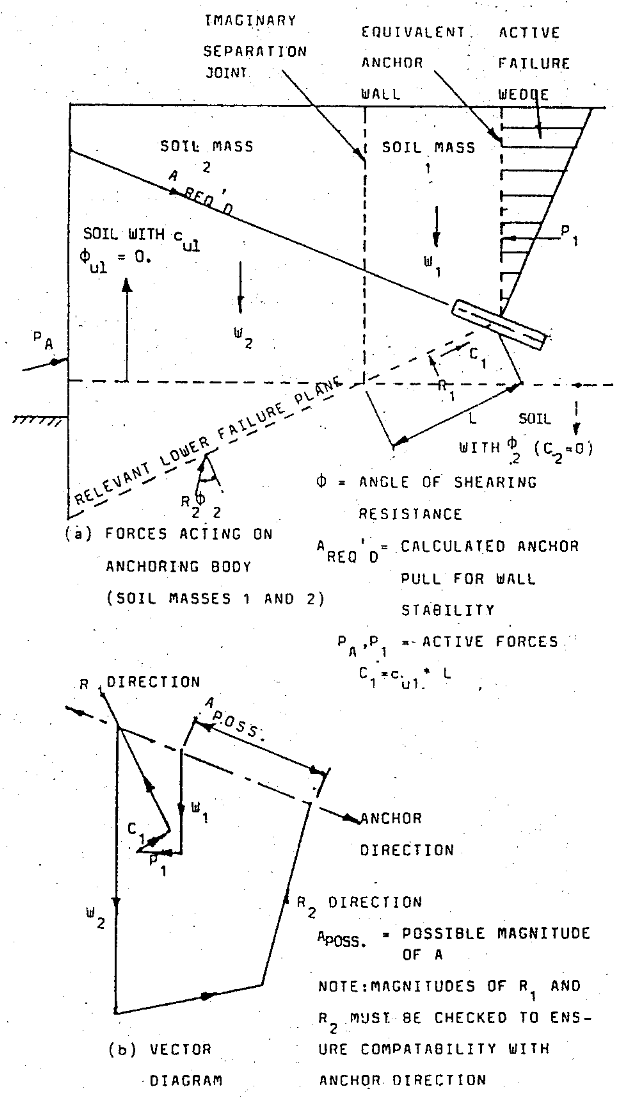


Fig. 8 Graphical analysis for two layers

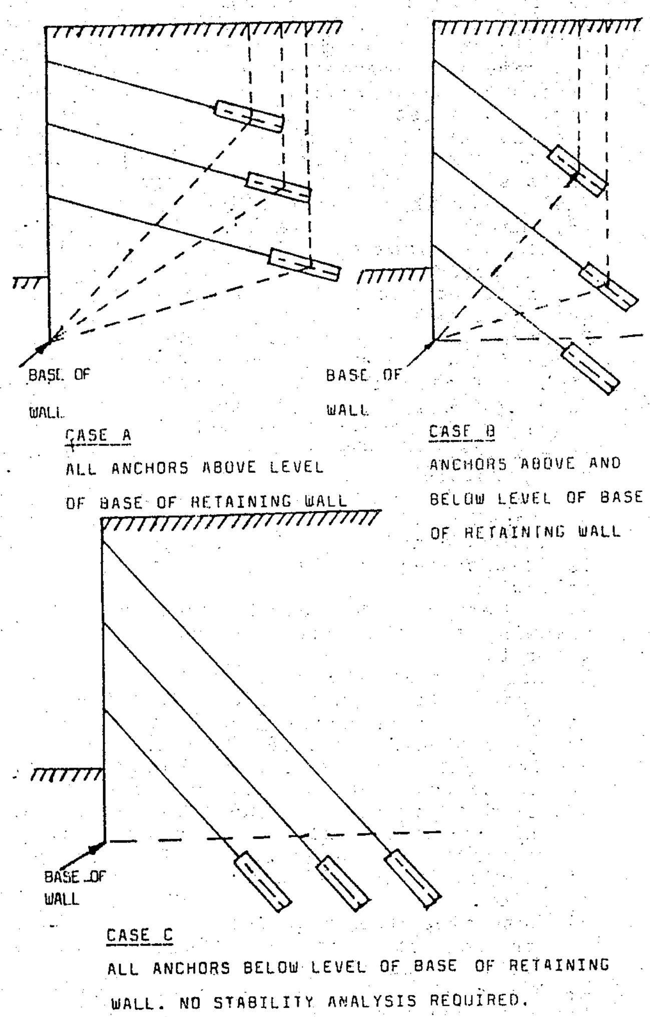


Fig.9 Potential failure planes - anchored walls

5. CONCLUDING REMARKS

The design methods outlined here are taken from the references given at the end of this paper. They are based on the design and construction experience gained mainly in North American and European countries. One should be cautious in applying them to local conditions. It is necessary to gather information on locally constructed ground anchors so that design and construction techniques better suited to local conditions can be adopted.

Acknowledgements

The author wishes to thank Mr. M.S.M. Ilmudeen and Ms. C. Ramachandran for their assistance in literature survey and in drawing diagrams for this paper.

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DESIGN AND CONSTRUCTION ASPECTS OF DIAPHRAGM WALLS

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1. INTRODUCTION

The use of diaphragm walls (also called slurry walls) for civil engineering purposes has increased tremendously during the recent past. A 1977 assessment of the total area of diaphragm walls built throughout the globe is in excess of 10 million square metres, excluding cutoff walls, bored piles and similar installations.

Although the supporting action and the stabilizing effect of slurries must have been known since 1900, the first publication on the subject did not appear until 1913. Bentonite was first introduced in slurry systems in 1929, but significant progress in preparation and control of slurries probably did not occur until after the second world war. However, this technique has not been in wide use in Asia and Africa until recent times. Japan and South Africa are exceptions to the above. The technique of diaphragm wall construction remains basically the same although there exist many slight variations in details of design, construction and equipment because of,

(1) different soil conditions
(2) geographically constrained evolvement and
(3) legal aspects such as patent laws etc.

It is believed the predecessors to diaphragm walls are Drilled Contrete Piers (bored piles), contiguous Drilled - Pier (bored pile) walls and Slurry - Trench - Cutoff walls.

The main uses of diaphragm walls are as retaining structures and load-bearing elements for deep building basements, traffic underpasses, underground mass transit stations, cut and cover tunnels, underground parking garages, underground industrial facilities, docks and waterfront installations, waterworks and miscellaneous foundations. In general three factors have contributed to the expansion of application of the diaphragm walling technique;

(1) commercial availability of bentonite

(2) ability (of method) to accommodate difficult conditions such as urban construction and problematic soils.

(3) Improvements in excavating techniques including new equipment.

continuous diaphragm wall formed and cast in a slurry trench, was first conceived by Verder in 1938 and can be looked upon as a combined development from two related systems; the mud-filled borehole and the contiguous boredpile wall. The construction sequences for a continuous diaphragm wall using modern methods and equipment is shown in Figure 1.

This paper also covers some of the applications of diaphragm wall systems and its design and construction aspects. The paper also summarizes some of the advantages and disadvantages of diaphragm walls.

2. DESIGN ASPECTS OF DIAPHRAGM WALLS

The design of a diaphragm wall must include the following three important aspects;

(1) stability and settlement problems associated with deep trench excavation.

(2) structural behaviour of a diaphragm wall to be subjected to various loading patterns during the performance of the desired underground structure and

(3) structural behaviour of a diaphragm wall as a load bearing part of the final structure in service.

Prior to the above design aspects the engineer must first ensure that the proposed construction method is technically feasible

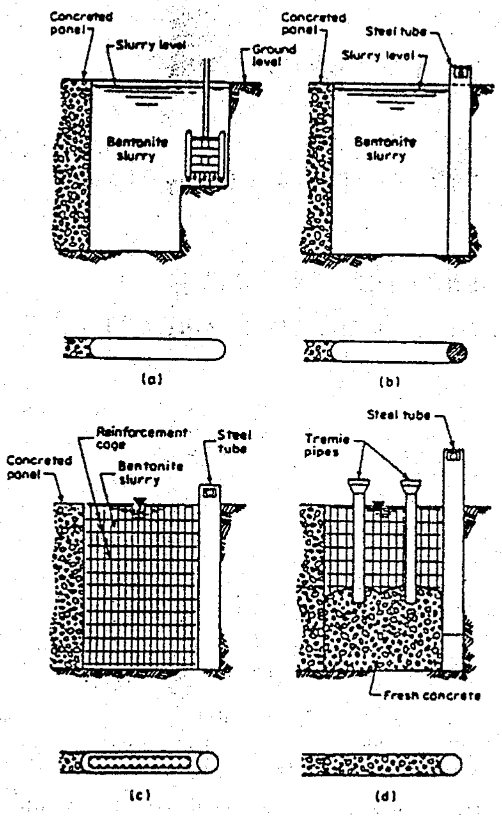


FIGURE 1 : Typical construction sequence of a diaphragm wall

(a) excavation, (b) insertion of steel tubing
(c) placement of reinforcement cage,
(d) placement of concrete.

and economically acceptable. Then the probable effects on the surrounding should be predicted. The analysis and design steps may be carried out thereafter according to the following sequence

(1) Analyse the stability of the excavation based on soil properties revealed by a comprehensive field and laboratory soil investigation program. This analysis shall consider all possible stages of excavation incorporating the effects of bentonite slurry, by known method of analysis. It is important to choose proper soil parameters and correct concepts (total stress or effective stress) of analysis to obtain realistic predictions.

(2) Predict the pattern, distribution and magnitude of ground movement due to excavation, and establish the corresponding probable settlement of adjacent buildings and utilities.

(3) If limitations must be imposed on the permissible or acceptable movements determine the constraints on the construction method and sequence of construction and therefore on the design.

(4) Estimate the magnitude and distribution of Lateral stress as a function of displacement and strain in various construction stages as well as after construction

(5) Analyse and design the structure and its parts including the bracing. The analysis may be based on semi-empirical methods, or it may be checked by numerical techniques.

(6) Consider the merits and the feasibility of observing and monitoring the first section of the project to check the predictions and the accuracy of the analysis and if necessary to modify the initial design.

2.1 Stability of Trenches in Cohesive Soils

2.1.1 Validity of $\phi = 0$ Concept (total stress analysis)

The $\phi = 0$ analysis is valid if the soil is saturated and is stressed under undrained conditions. The above conditions are satisfactorily met by cohesive clayey soils originally saturated, because the trench excavation in diaphragm wall construction is temporary. However, the $\phi = 0$ analysis does not establish the failure surface since $\phi = 0$ always implies a critical plane inclined at 45° with the horizontal. This discrepancy is rectified by noting that in a triaxial test it is not necessary to know the exact locations of the failure plane. The $\phi = 0$ analysis is valid as long as failure is assumed to occur when the maximum shear stress reaches the maximum shear existing at failure in a corresponding triaxial test. Thus it is possible to analyse the stability of a trench without really knowing the exact inclination of the failure plane. The choice of method for short - term problems in saturated soils therefore, becomes a matter of convenience, and the $\phi = 0$ concept is preferred for its simplicity.

2.1.2 Invalidity of $\phi = 0$ analysis

For conditions other than undrained shear the $\phi = 0$ method can lead to unrealistic predictions. The concept similarly loses its meaning whenever field measurements of pore pressure are used as a control measure. The $\phi = 0$ concept clearly does not apply to partially saturated soils. In this case an approximate expression of undrained strength in terms of the two parameters C and ϕ is possible, but because of the many factors that influence these parameters, results from such analysis must be interpreted with caution.

Excavation of trenches in stiff fissured and weathered clays can give rise to special problems. Stress reductions due to unloading causes the fissures to open up and provide weak zones for the sliding surface to follow. This problem is more serious when the excavation is done without slurry.

2.1.3 Choice of Soil Parameters

It is well known that soil parameters obtained in the laboratory by various standard test procedures do not necessarily represent the actual in-situ values. Soil sampling procedure, sample disturbance, shearing rate, strength anisotropy are known among others to influence the strength of soil. Present semi-empirical procedures of analysis which have been time tested and revised over a considerable time span do apparently account for cumulative effects of such factors inherent in standard procedures. Therefore, one shall not deviate from standard procedures of testing even if the deviation makes the test superior as the implicit balance of empirical formulae may be disturbed.

2.2 Stability of Trenches in Cohesionless soils

Trenching in dry sand is relatively easy. Designing as well as construction is also more straight forward in dry sand. Stability shall be checked against simple wedge failure which depends on the soil properties and the weight of slurry.

However, where the free water level is near the ground surface, it is quite difficult to maintain trench stability. This may require one or all of the following measures; (1) lower the natural water table, (2) raise the slurry level in the trench, (3) use a heavier slurry, and (4) resort to other factors for stability. When the soil consists predominantly of coarse sand and gravel, some penetration of the adjacent zone by slurry will undoubtedly occur, generally above the natural water table, until the formation of the impermeable barrier at the interface restricts this flow. Disregarding the associated effects, one carries out the stability analysis assuming drained conditions; that is considering the effective stresses in the soil and the static pore pressures prevalent in the ground.

In addition to the above mentioned considerations the stability of a trench is also controlled by; (1) gel strength of slurry, (2) arching effect, (3) horizontal curvature of trench, and (4) dynamic loading.

2.3 Ground Movement and Settlement due to Excavation

For almost any type of ground support, the movement and settlement due to excavation depend upon the soil characteristics and the groundwater conditions; the size of the excavation, particularly its depth; the rigidity of the support system; the details, installation sequence, rigidity and preloading or prestressing of the bracing; and the general workmanship of the construction. As a result of the variety, number and diversity of the factors influencing movement the best practice has been to observe, measure and analyse ground movement around actual excavations and use this record to supplement theoretical predictions.

Typical patterns of movement are presented in Figure 2.

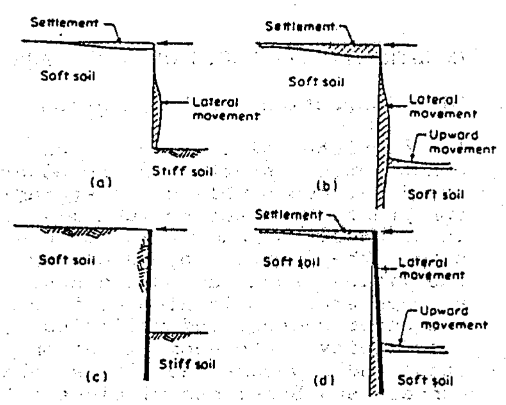


FIGURE 2 : Settlement, Lateral movement and Heave, (a) and (b) flexible wall (c) and (d) rigid wall

Prediction of settlement and ground movement prior to excavation by theoretical means is unrealistic as the principal cause of lateral movement (hence settlement) is highly dependent on workmanship, which is beyond the designers imagination. Therefore, it is important to control lateral movement of excavation support system (hence settlement) by;

(1) Monitoring lateral movement during excavation and construction of permanent support system.

(2) Improved workmanship to avoid delayed placement of lateral support (bracings, anchors etc.)

(3) By maintaining ground water table outside the excavation at normal levels.

There are several techniques of construction of permanent lateral support systems or temporary bracing, tie backs etc. in a diaphragm wall excavation. These techniques have their own merits such as vertual elimination of lateral movement to uninterrupted ground surface passage, while under the roof excavation is in progress. The following techniques are practiced in general;

(1) walls braced before excavation is begun
(2) walls braced with struts,
(3) walls braced with permanent floors in a downward process,
(4) Walls braced with tiebacks and ground anchors,
(5) free cantilevers, and
(6) walls supported by a combination of bracings, such as tiebacks, struts, rakers and berms.

3. CONSTRUCTION ASPECTS OF DIAPHRAGM WALLS

3.1 Site Inspection and Preparation

A complete site inspection before construction starts is necessary to avoid serious problems. Diaphragm wall construction sites are usually located in urban areas, and it is not unusual to have the site bounded by other facilities such as buildings, roads, subways etc. It is also quite common to encounter underground utilities (water, electricity, telecommunication mains etc.) and sewerage facilities, abandoned tunnels and basements within a site. An accurate prior knowledge on underground obstructions to diaphragm wall construction makes realistic scheduling possible. Else loss of time and money is most often unavoidable.

It is also important to gather other relevant information such as; (1) environmental regulations, (2) any restriction to install underground anchors or tiebacks across adjoining properties, (3) local traffic densities in relation to days of the week and hours of a day, also possibility of detouring traffic on boundary roads, and (4) overhead facilities which may hinder operation of construction equipment.

Site must be rid of all kinds of debris on the surface which may hinder smooth operations of diaphragm wall construction equipment. Sloping sites may need a lot of preparatory work prior to construction work. Once the layout of the diaphragm wall to be constructed is decided, layout of mud-circulation and preparation plant which includes slurry mixes, storage tanks, mud separation units and temporary storage of refuse is planned.

3.2 Guide Walls

As a part of site preparatory works guide walls are constructed along the proposed diaphragm wall layout. The guide walls may be avoided in sites with very stable ground conditions and low groundwater table. Design of guide walls depends on (1) ground conditions, (2) the groundwater table, and (3) anticipated panel length. Because of the above, the guide walls may be limited to simple sections with nominal reinforcement or heavily reinforced composite concrete sections. The walls may have to be erected above the ground surface to sufficient height to impound bentonite slurry.

The guide walls serve the following purposes;

(1) as the name implies, it provide a guide to trenching equipment (tools) hence ensure correct alignment,

(2) improves stability of ground, specially against failure due to rough insertion and withdrawal of excavating tools, and

(3) provids a means to impound slurry above the ground surface to enhance the trench stability.

The guide walls are temporary structures, which must be demolished after completion of the diaphragm wall.

3.3 Panel Dimensions

The panel dimensions and related details are mainly governed by site conditions. Thus a specified panel length often cannot be provided in the field because of restrictions on availability of construction space and time.

Average panel length is about 4m. Shorter panels are easier for concreting and better with respect to the stability of the trench. Longer panels cause less seepage and fewer alignment problems; also saves time and cost of joints. Reinforcement cages are best assembled in the shop and transported to the site. However, if this is not possible it can be assembled at site. Location of tiebacks, anchors or bracings is another controlling factor of panel length. Finally dimensions of excavating equipment available should also be a factor to be considered in deciding the panel length.

Panel depth is generally decided by project requirements. Excavating tools are generally capable of digging upto depths required in most jobs, hence it is not a controlling factor. A wall may be founded on rock or other firm material to transfer vertical loads; it may be extended into an impervious layer to protect the excavation from groundwater; or it may simply be embedded below excavation level for lateral stability and resistance to movement.

Panel width cannot be less than the range of the available excavating tools; also sufficient space for tremie pipes within the reinforcement cage shall be provided. Usual range of the panel width is 45cm to 1.5m. A frequent width is 60cm. It shall be noted that the minimum width of panel is not the most economical design as construction difficulties and slow speed of construction is associated with thin panels.

3.4 Slurry Quality

Quality of the slurry must be maintained so that it ensures maximum efficiency, since this is the most important material in the construction of diaphragm walls. On the construction site the slurry must be inspected periodically and the following measures taken to maintain its effectiveness;

(1) determination of the dry bentonite / water ratio and duration and intensity of the mixing process

(2) Determination of the swelling period, that is the time between completion of the mixing process and the use of the suspension for the trench excavation.

(3) The used slurry must be replaced by fresh material as soon as excavation progress slows down noticeably. Table 1 indicates the control limits for the properties of slurries.

3.5 Excavating Equipment

Excavation method and type of equipment for a given site shall not be restricted at design stage for the following reasons;

(1) number of bidders will be less due to lack of specified equipment as well as lack of experience in specified method.

(2) If unforeseen site conditions are encountered and the specified method and equipment are unsuitable for such conditions, large sums of additional cost will have to be borne by the Engineer. This also will cause considerable delays in completing the project.

Conversion of existing equipment for trench excavation shall not be allowed as (1) conversion of equipment is seldom, if ever, successful, and (2) converted equipment most often is a cause for slow excavation which is uneconomical and may become a threat to the stability of the excavated trench.

Trench excavating systems available at present can be classified as follows;

(1) conventional trench excavators, (2) machines of the bucket and grab type, (3) percussion tools, (4) rotary drilling equipment, and (5) reverse circulation machines, Backhoes and drag line buckets can be classified as conventional excavators. Bucket and grab type has a high weight - to - volume ratio to overcome drag and floatation effects on the slurry gel during excavation. Mechanical diggers, either back or forward acting; bucket excavators of special design; shovels and auxiliary tools such as augers; and special trenching grabs and clamshells. Percussion tools are heavy and rigid, often of special design; they are used to break rock or loosen hard ground and in situations where other types of equipment are not effective. Excavations with percussion tools is slow and therefore expensive. Rotary drilling equipment are also referred to as hollow - stem large diameter bit drilling rigs and machines. These equipment can perform either slot or circular excavations. Almost all rotary drilling tools and most percussion tools are equipped with reverse circulation. This process consists of using the drill stem and hydraulic pipeline for the direct excavation. Some widely used types of equipment are shown in Figure 3.

3.6 Excavation with clamshells and percussion tools

Single stage panel excavation is carried out in a series of passes (see Figure 4) and requires clamshells only. It is feasible in soils which are soft to medium hard, either granular or cohesive, but without boulders or other obstructions. The minimum linear element that can be excavated is the minimum grab bite (usually 2m) and the maximum panel that can be excavated in one pass is the maximum grab bite (about 6m).

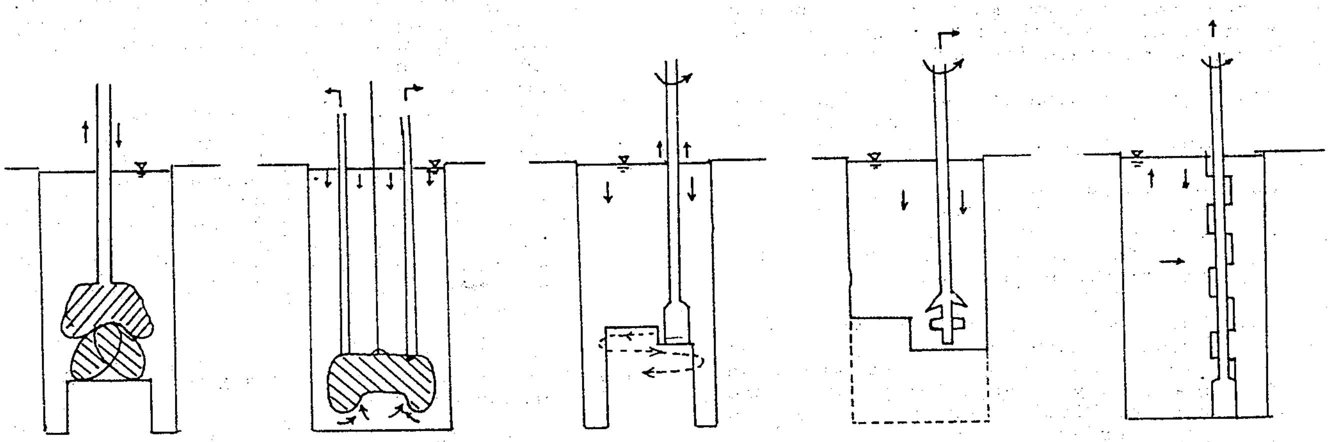
Average panels are usually excavated in three passes as shown below. The middle tongue is made 0.3m to 0.6m less than the grab opening to allow the bucket to embrace it. The first pass generally begins away from the last concreted panel to allow the concrete to harden. Round-end clamshells are used in

TABLE I

Control Limits for properties of slurries

	PROPERTY							
Function	Average bentonite concentration %	Density lb/ft^3	Specific gravity	Plastic viscosity CP	Marsh Cone viscosity	10 min gel strength lb/100ft^2	PH	Sand Content %
Face support	>3 - 4	>64.3	>1.03	-		-	-	>1
Sealing process	>3 - 4	-	-	-		-	-	1
Displacement by concrete	<15	<78	<1.25	<20		-	<12	<25
Separation of noncolloids	-	-	-	-		-	-	<30
Physical cleaning	<15	<78	<1.25	-		-	-	<25
Pumping of slurry	-	-	-	-		variable	-	-
Limits	>3 - 4	>64.3	>1.03	<20		>12 - 15	<12	>1
	<15	<78	<1.25	-		-	-	<25

* Average bentonite concentration should be expected to vary widely because of different bentonite brands.



(a) (b) (c) (d) (e)

FIGURE 3 : Equipment for Slurry-trench excavation

(a) clamshell bucket attached to a kelly (b) vertical percussion bit with reverse circulation

(c) percussion benching bit (d) rotary benching bit

(e) rotary bit with vertical cutter

conjunction with round-end tubes at the construction joints.

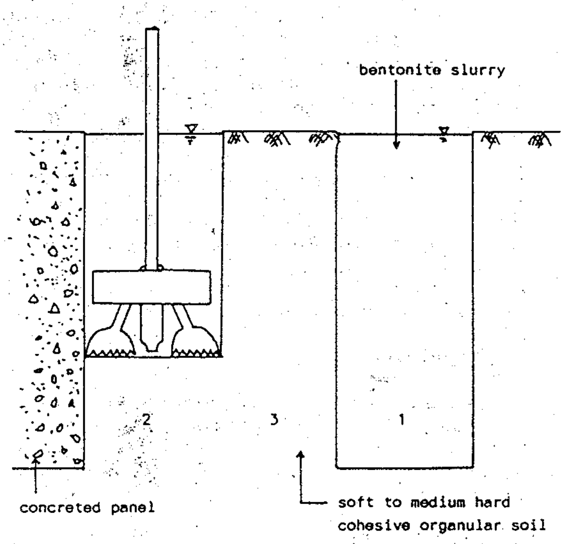


FIGURE 4 : Single-stage panel excavation with clamshell

Panel excavation can be performed using either 3, 5 or more passes. The alternate panels are first completed followed by the intermediate panels.

Other excavation systems can also be used for trench excavations based on soil conditions, applicability and availability of such systems.

3.7 Reinforcement Cages

The reinforcement cages are assembled on site or in the shop and transported to site. The cages can reach formidable dimensions, based on panel dimensions. Lifting up a reinforcement cage is not an easy task due to their bulky dimensions, weight and flexible nature. Careless handling can distort a cage in a matter of seconds which can complicate the whole operation of concreting a panel. Nevertheless, a distorted cage should never be inserted in a panel. A special lifting sling and 2 crane lines are used to prevent distortion of cage. In very deep panels two or more cage lengths may be used. When splices are necessary, the first length is left just projecting above the trench and the second cage length is welded or otherwise attached to it before entire cage is lowered into position. The minimum bar spacing should not be less than 150mm and preferably 225mm. The horizontal bars should never be closer than 300mm unless they constitute the main reinforcement. A generous concrete cover should be provided, especially for the earth side of the exposed wall, and the minimum clear cover should not be lass than 75mm Roller spacers are used to ensure correct alignment of the cage and minimum cover. Diaphragm walls usually need bracings, anchors, tiebacks, or floor slabs to be connected for lateral support. The reinforcement cages must be fitted with wooden boxes (or styrofoam flanks) to prevent fresh concrete from entering such areas. Within the boxed-out areas of the cages are bent dowels and other reinforcement which shall be structurally connected to the floor slabs, bracings etc.

3.8 Panel Joints

There are many systems of construction joints used in diaphragm wall technique. Most common is the round tube joint. However, depending on particular requirements it may be necessary to use more sophisticated joint systems which may allow better water tightness, transfer of shear forces, and continuity of horizontal reinforcement for bending resistance. The round tube joint cannot provide for continuous horizontal reinforcement, hence this joint cannot resist bending stresses. However, water tightness and shear resistance can be achieved sufficiently by good workmanship.

The round-tube joint is practicable for excavations carried out with rotary drilling equipment and/or round end clamshells. If cavitation or overwidth excavation occurs at the end, the fresh concrete may flow around the stop-end tube. Such over width concrete must be broken with chisels when excavating the adjacent section.

When concrete pour of a panel is completed and the concrete begins to set, the round-tube is slowly extracted. Initial extraction is accomplished by two hydraulic jacks. Once the movement of round-tube has begun, a heavy crane is used to remove the tube from its position. Removal of the tube leaves a half-round concrete key at the end of the unformed panel.

3.9 Concrete Placement

In general, it is advantageous to carry out the concrete placement operations soon after the reinforcing cage is inserted into the panel. The pour should be completed in the shortest possible time and without discontinuities or interruptions in order to avoid embedment of stop-end tubes, blockages in tremie pipes and cage flotation caused by the upward drag of rising stiff concrete. The most common method of concrete placement is by means of tremie pipes, which are withdrawn in stages as the concrete level rises. Tremie pipes shall have a minimum inner diameter of 8 times the maximum aggregate size.

When concrete is tremied in, a plug must be used in the pipe to revent freefall of concrete and mixing of concrete with water or slurry, a common source of dilution or segregation is due to the presence of water or slurry in the pipe before starting the pour. To ensure error free concreting the following sequence of measures is commonly used; (1) the pipe is lowered through the slurry until its tip rests firmly on the base of the trench, (2) the funnel hopper at the upper end is assembled, (3) a plug which floats in the slurry is placed inside the pipe. As concrete enters the tremie, the plug travels down under the weight of the mass of fresh concrete, and reaches the bottom. At this stage the tremie pipe is slowly lifted, allowing the concrete to push the plug out. Once this occurs concrete begins to discharge and fill the panel while the pipe is kept submerged and completely filled.

Once the placement begins, the process requires routine checking of the concrete level around the pipe and at the ends of the panel. Despite the practical advantages associated with the use of a single tremie pipe, as opposed to two or more, it generally is essential to specify the maximum tremie pipe spacing and therefore the maximum panel length that can be poured with one tremie pipe. Pipe spacing should not exceed 5m as a rule of thumb.

4. APPLICATIONS OF THE DIAPHRAGM WALL TECHNIQUE

4.1 Comparison with other systems

Technical feasibility of a construction technique does not conclude that, its the most suitable technique unless a comparison is made with the other techniques available for the same purpose. Therefore, it is necessary to compare other ground support systems such as; (1) soldier piles with timber lagging, (2) sheet piling, (3) continuous bored-pile walls, and (4) ground treatment such as grouting, freezing etc.

The direct cost of ground support varies according to the method used. For an average situation, soldier piles with lagging usually provide the least expensive system, followed by sheet piling, diaphragm walls, contiguous bored-pile walls and ground treatment. However, this comparison can be misleading unless it takes into consideration the many factors that determine the overall cost. For example a system that alleviates the need for underpinning and ground water control can result in an overall economy although the ground support system itself is not necessarily the most economical. In this light diaphragm wall technique can be singled out because it is not merely a ground support system but it can also be a part of the permanent structure carrying vertical as well as bending loads. In addition it can achieve near perfect water tightness.

With the evolvement of new excavation tools and refinement in slurry support and other processes it has been noted in many parts of Europe that the overall cost of diaphragm wall construction has gone down by as much as 50% despite inflationary pressures since the 1950s.

4.2 Applications

Diaphragm wall technique can be used in a range of applications namely: (1) cutoff walls (2) ground support systems (3) part of a permanent structure

4.2.1 Cutoff wall

In this application perimeter trench excavation around a site to be isolate from high ground water or along a core-line of an earth dam is carried out as described in section 3. Thereafter the trench is filled with bentonite - cement mixture which solidifies to attain a low strength (comparable to soil strength) plastic backfill.

Another special application is to isolate cities which have subsided due to excessive ground water pumping-out before recharging (to raise ground water level by pumping in water) to raise the ground surface.

4.2.8 Ground support system

Perimeter walls of basement excavation, cut and cover tunnel side walls, soil retaining walls of wide range, slope stabilization system, and access shaft wall are some of the many application of diaphragm walls as earth support systems, such applications can either be permanent or temporary depending on individual project requirements.

4.2.3 Load bearing elements of structure

Side walls and cross walls of deep basements of buildings which are designed to carry foundation loads. Underpinning of existing structures which are threatend by instability due to natural or manmade reasons. These diaphragm walls may or may not be extended upto bearing strata. If the bearing stratum is deep it is economical to install piles to support the wall through special access casings cast in to the wall.

5. ADVANTAGES AND DISADVANTAGES

Diaphragm wall construction in general provides maximum economy where it can transplant the temporary sheeting and the permanent ground support or where the excavation involves underpinning or groundwater control, the diaphragm wall can remedy this problem. Regarding the direct cost of its installation, the ability to produce a low cost structure will generally depend on many factors. Thus besides the physical dimensions and the configuration of the wall, its cost may be influenced by,

(1) the required embedment below excavation level for stability or seepage control,

(2) the nature of ground to be excavated and the presence of boulders or other hard obstructions.

(3) the associated stability requirements, and

(4) site factors such as mud disposal, availability of utilities, time restrictions and availability of working space.

5.1 Advantages

1. It is possible to produce time savings that satisfy the goal of rapid completion. This is true for examples with the under-the -roof method or by the downward construction process, whereby the excavation of the basement and the erection of superstructure can proceed simultaneously.

2. The method can be adopted under unfavorable soil and hydrological conditions and where other techniques many have limitations. The walls can be constructed to considerable depths ahead of the main excavation and act as structural underpinning to adjacent structures.

3. The installation is completed essentially free from noise and vibrations and can accommodate the requirements of dry excavation. Maximum usage of the site is possible by building walls along the site boundary and panel can actually be constructed against existing buildings under minimum clearance. The technique is not sensitive to irregular wall layout.

4. Ground movement and settlement can be eliminated or sufficiently controlled with bracing such as cross walls and permanent floors.

   The cross walls can be constructed by the diaphragm wall technique prior to the main excavation.

5.2 Disadvantages

1. Site congestion seriously impedes traffic, access to adjoining property, and other activities. In order to lessen this drawback, it may be necessary to (1) operate on one side only, (2) restrict panel size, mud plant, and reinforcement; and (3) operate during certain hours only.

2. The finished product is influenced by the type of soil, and the surface finish may be rough and thus require further treatment. Obstructions may cause concrete blisters and protrusions that must be broken out, and when working on slopes of hard formations there is a danger of deflections in the excavation with a corresponding loss of verticality.

3. The overall quality assurance depends on processes, methods, and operations carried out at the site; hence it need special field inspection and supervision.

4. The accuracy of the excavation to a certain extent depends on the skill and experience of the operator.

5. Disposal of used slurries and spoil material in urban areas may pose special problems.

5.3 Conclusions

Diaphragm walling is now a fully accepted construction technique in civil engineering specially in the developed world. In recent times diaphragm wall construction method has been successfully applied in many developing countries in Asia and other regions of the world.

Economic applicability of diaphragm walling in Sri Lanka need further research and analysis, especially because introduction of the technique essentially means import of equipment and know how. Further development in urban areas (such as basement car parks, multi-level basements of high-rise buildings and perhaps a subway system) in Sri Lanka could be the right time to introduce diaphragm walling technique to Sri Lanka.

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WATERFRONT EARTH RETAINING STRUCTURES

D. GODAGE SRI LANKA PORTS AUTHORITY

1. INTRODUCTION

An earth retaining structure as the term means is a structure to hold in place an earthfill or a vertical embankment of earth. It is a common civil engineering structure. A quay wall for berthing of vessels, lake or river embankment wall, drydock wall are among waterfront earth retaining structures. A waterfront earth retaining structure differs from an ordinary land-based earth retaning structure principally in the loads imposed on it and in its intended use. It has a head of water against the face of structure opposite to that retaining the backfill which means that the structure is submerged.

2. TYPE OF STRUCTURES

2.1 Classification by usage

By usage the waterfront earth retaining structures may be classified as permanent and temporary and some examples are given below:

a) Permanent Structures
- . Quay walls
- . Revetments
- . Seawalls
- . Walls for reclamation
- . Drydock walls
- . Lock walls

b) Temporary Structures
- . temporary retaining walls
- . Cofferdams

2.2 Classification by Structural type

By structural type, the earth retaining structures may be classified as:

a) Gravity type - principal examples being:
- . Caisson
- . L-shaped block
- . Rectangular mass-concrete block
- . cellular concrete block
- . Cast-in-place concrete or masonry

b) Sheet pile type
- -Cantilever
- -Anchored
  - - Tierod
    - .Vertical pile anchor
    - . coupled pile anchor
    - . deadman anchor
  - - Batter-pile anchor
  - - Relieving platform
- -Cellular
- -Other
  - - double sheet-pile wall
  - - multi-strut type.

3. EXTERNAL FORCES AND LOADS ON STRUCTURES

The external forces and loads acting on a waterfront earth retaining structure could be listed as follows :

a. Surcharge
b. Dead weight of structure
c. Earthpressure and residual water pressure
d. Buoyancy
e. Tractive force/impact force of vessels

Depending on the type of structure selected for design, some of the above listed forces and loads may be ignored or could be reduced. For instance, the dead weight is not considered in design of sheet-pile structure.

The surcharge in ordinary condition shall be determined with due consideration of the cargo handling practices in a quay apron or other load as the case may be. Generally a surcharge of $2t/m^2$ is used in harbour facilities. If there are other live loads such as crane wheel load, they shall be considered in addition.

The weight of a gravity type quay wall will be that of the portion in front of the vertical plane passing through the rear toe of the wall as shown in Fig. 1.

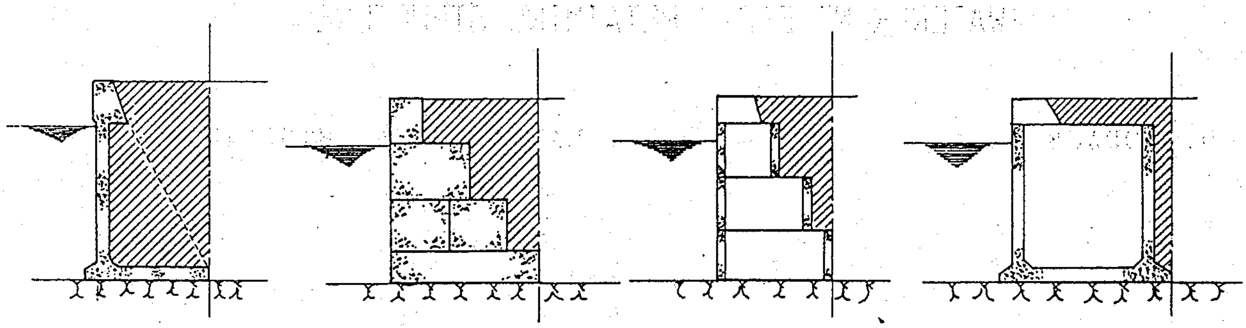


Fig. 1

The backfill behind the wall exerts lateral pressure on the wall which tends to push it away and the backfill is in active state of deformation. For an unyielding rigid wall, no soil deformation is possible and the backfill is said to be in a neutral or at-rest Condition to which as earth pressure coefficent value varying from 0.4 to 0.5 corresponds. In general, granular fills are used in waterfront retaining structures of harbour facilities and clay backfills are avoided. Earth pressure coefficients are available in tabular form in reference to soil characteristics shape and condition of structure, shape of backfill etc.

In the design of waterfront structures, residual water pressure should be added to the lateral earth pressure. The difference in waterlevel in the backfill and waterlevel in front of the wall is the reason for residual water pressure and it is determined considering the permeability of the wall and tidal range. Usually 1/3 ~ 2/3 of the tidal range is recommended in design and the pressure distribution as shown in Fig. 2.

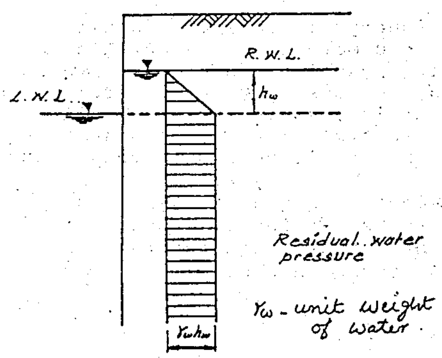


Fig 2. Residual Water Pressure

In case of steel sheet pile wall, 2/3 of the tidal range is used.

When a mooring post or a bollard is provided on the wall, the pull or tractive force of the vessel acting on the bollard is tranmitted to the structure. In the case of an anchored sheet-pile wall the tractive force is transmitted to the tie-rods.

4. CHARACTERISTICS OF EACH STRUCTURE TYPE

4.1 Gravity Type

Gravity walls are massive and they are satisfactory if the soil layers beneath base are firm. Frequently part of the soft natural bottom is dredged away and replaced with sand or granular fill forming a mattress to distribute the load of structure on the underlying soil. The use of a granular matress is advisable as it provides an even surface for seating of heavy structure above it and a higher resistance to sliding of the wall.

Gravity type walls are very frequently constructed using precast concrete units such as caissons, L-shaped blocks, rectangular mass-concrete blocks, cellular concrete blocks. Sometimes cast-in-situ concrete and masonry are also used in construction.

The design of gravity-type wall would follow the sequence given below :

a. determine design conditions
b. assume wall dimensions
c. determine external forces acting on the wall.
d. examine the structure for stability, in general for
   - sliding of wall
   - bearing capacity of foundation
   - overturning of wall
   - circular slip and settlement.

In case of concrete block walls it is necessary to examine the stability of the wall for different horizontal planes that constitute the joints. The presence of interlocking keys between blocks are neglected and the forces acting on the portion of wall above any horizontal section are used in this examination.

The factor of safety against sliding shall be more than 1.2 for satisfactory performance. Very frequently, improved coefficent of friction between the bottom of the wall and foundation is achieved by using a sand, crushed stone or graded rock mattress. Case-in-place concrete foundation is also used.

The factor of safety against overturning shall be 1.2 or more. In case of bottom opened cellular-block type wall the cellular concrete block and the infill shall be considered separately. The resisting moment due to vertical force of concrete block and wall friction forces due to filling shall be greater than the overturning moment due to external forces. If factor of safety is less than 1, then the cellular block will overturn leaving behind the filling.

In the examination for bearing capacity, the vertical loads and lateral loads acting on the wall shall be considered. Generally, the foundation of a gravity wall is of a two-layer with granular mattress on the subsoil. This characteristic of the foundation should be noted in the analysis.

If the foundation ground of a gravity type wall is soft clay, then it is necessary to examine the stability against slip failure and settlement.

When reinforced concrete pre-cast caissons are used in wall structure, they must be designed to be safe during launching, floating, towing and at different stages of construction. Once the caission is in place it is filled with sand and/or concrete, then it acts as a massive gravity retaining wall.

Examples of gravity type structures are shown in Fig. 3 - a mass concrete wall 3.6m (12 feet) high from Beira Lake, Fig. 4 - a concrete-block wall 14.3m (47 feet) high at Queen Elizabeth Quay, Fig. 5 - a reinforced concrete caisson wall 15 m high at Jaye Container Terminal.

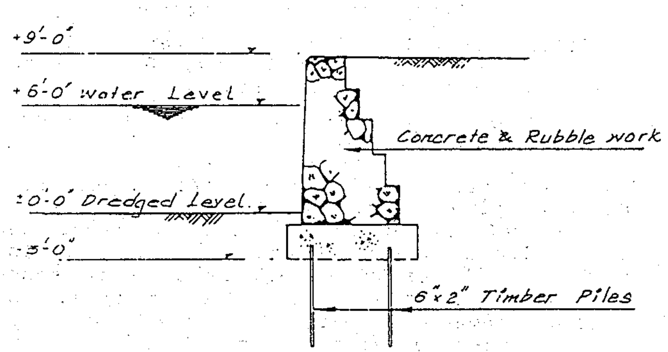


Fig 3:- BEIRA LAKE QUAYWALL

4.2 Sheet pile type

The sheet pile type structure is designed to resist external forces by flexural rigidity of the sheet piling, soil reaction of the embedded portion of the sheet piling and resisting force due to anchor or upper support if available. The design considerations of a sheet pile type wall, include:

a) Calculation of external forces and lateral pressures acting on the wall
b) determination of required pile penetration depth
c) Computation of bending moment in the wall
d) Selection of appropriate sheet pile and computation of maximum stress
e) design of anchor system.

4.2.1 Cantilever type

Cantilever type of sheet pile wall is designed to resist lateral forces by flexural rigidity of sheet piling and the soil reaction of the embedded portion of pile. This type is suitable for small-scale retaining structure and it is easy to build. Cofferdams are easily constructed using single line of steel sheet piling. Steel and reinforced concrete sheet piles are used in permanent constructions. While the steel sheet piles have interlocking arrangements, tongue and groove arrangement is used in reinforced concrete piles though it is less effective in sealing the backfill.

In case of sandy soil, a fictitious seabed is used instead of the actual sea bed. The fictitious sea bed is located a point where active pressure and residual water pressure equal passive earth pressure.

Two examples of cantilever wall, one with reinforced concrete sheet pile (Fig. 6) and other with concrete cylinder (Fig. 7) are shown.

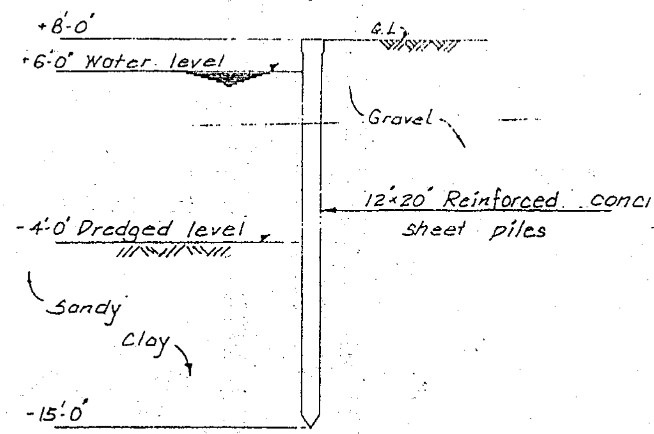
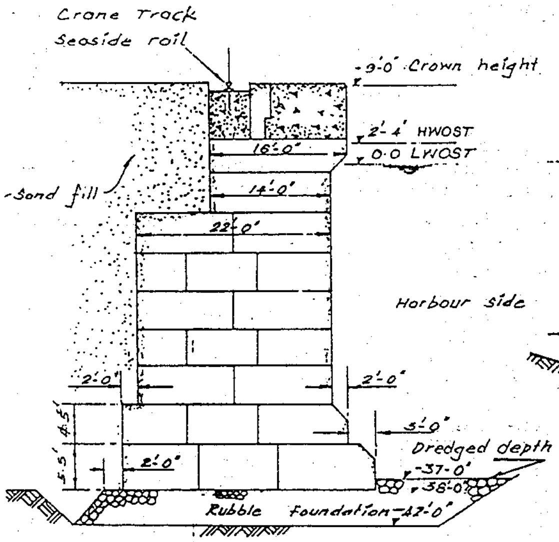
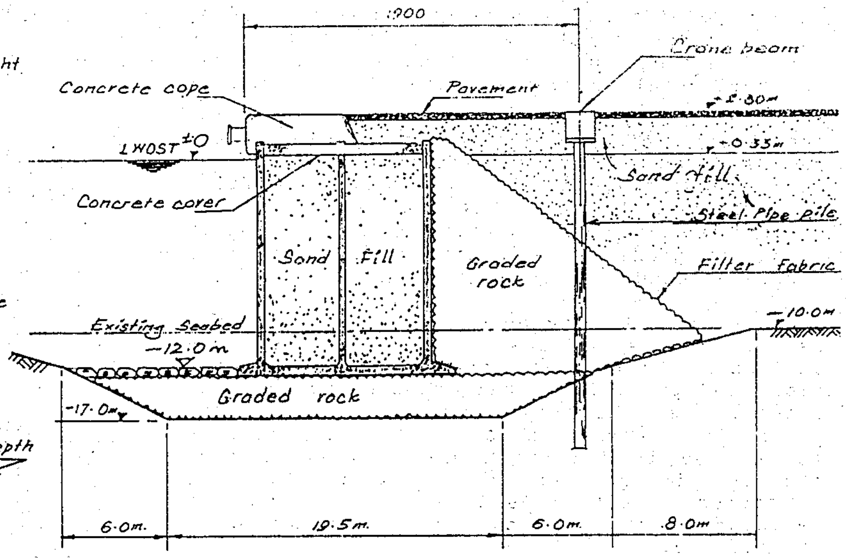


Fig 6:- R. C. SHEET PILE QUAY



Typical Section of Blockwork Quaywall

Fig 4 – QUEEN ELIZABETH QUAY



Cross - Section of Quay

Fig. 5 - JAYE CONTAINER TERMINAL

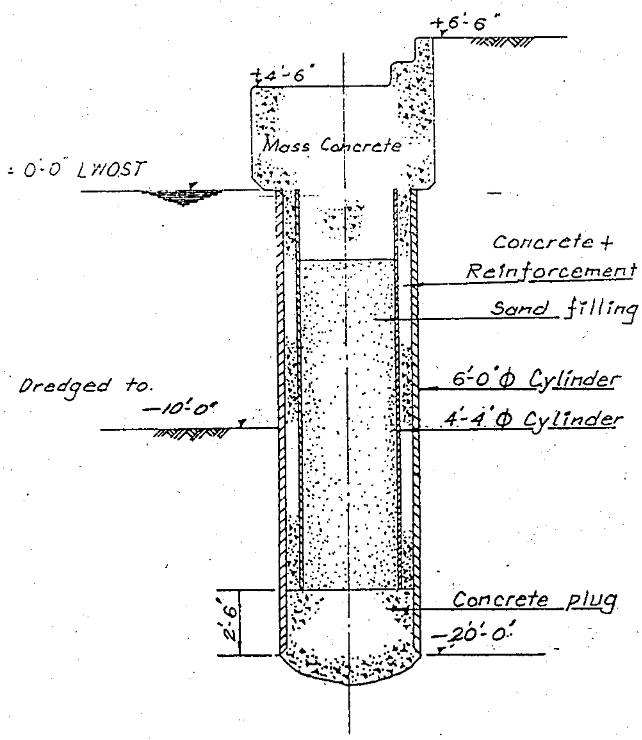


Fig 7 - GALLE LIGHTER QUAY

4.2.2 Anchored Type

Anchored type structure has an upper support in addition to the soil reaction of the embedded portion of sheet pile wall. Mainly, there are three types classified under anchored type of wall, viz. tie-rod type
batter-pile type
relieving platform type.

4.2.2.1 Tie-rod Type

The tie rod anchor type is very common. This type of structure is supported by the passive earth pressure in front of embedded portion of piles and anchor tie-rods near the top of piles. The design theories developed for the analysis of tie-rod anchor type structure may be as follows :

(i) Based on classical earth pressure theory
- Free Earth Support method
- Fixed Earth Support method

(ii) - Equivalent Beam Method (after Blum)
- Simplified Equivalent Beam Method (after Tschebotarioff)

(iii) Empirical rules based on experience (Danish rules)

In most design practices, the penetration depth of pile is calculated based on the equitibrium of moment at the tie-rod level due to passive pressure and active pressure, using a factor of safety of 1.5 or more in sandy soil and 1.2 or more in cohesive soil. The bending moment in pile is determined by assuming a virtual simplified beam supported at tie-rod level and at dredge line.

Fig. 8 shows pressure distribution in sandy soil on a sheet pile wall. Fig. 9 shows Imaginary Beam method for bending moment calculation.

The tie-rod can be a high tensile steel rod, a wire rope of high tensile steel or a pre-stressed concrete beam having high tensile wire. It shall be designed to take the reaction at tie-rod setting level. The force on mooring bollard of quay will be transmitted to the tie-rod.

The anchorage for tie-rod is generally provided by :

- Anchor wall
- Anchor pile or pile wall
- Coupled batter piles

The recommended location of anchorage works in each case are given in Fig. 10, 11 and 12. The external forces acting in the case of an anchorage wall are shown in Fig. 10A.

Two examples of tie-beam anchor quay walls are given from Galle Harbour and Colombo Harbour in Figs. 14 and 15.

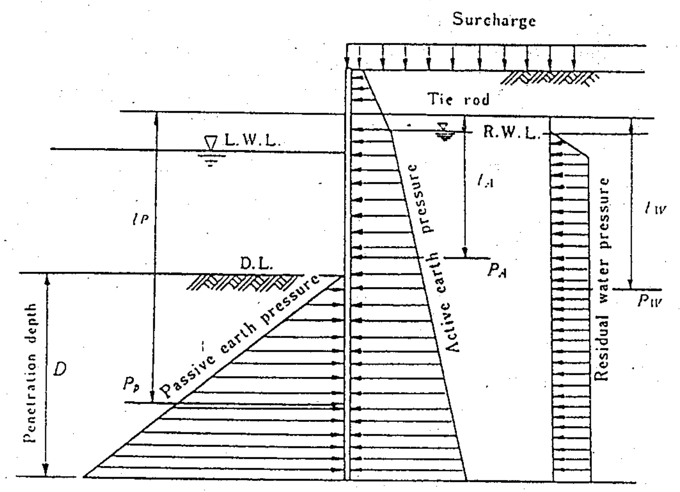


Fig 8. Earth and Water Pressures Acting on Sheet Pile Wall

4.2.2.2 Batter-pile Anchor Type

The batter-pile anchor type structure has very high resistance against wave pressure among those constructed using sheet piles. The heads of batter-piles and sheet pile wall are tightly connected to ensure stability, as shown in Fig. 16.

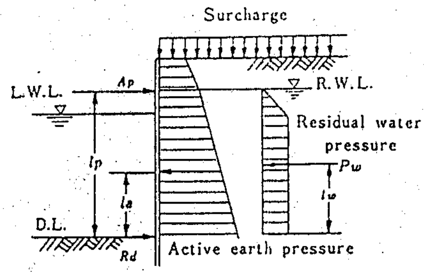


Fig. 9 . Imaginary Beam Method

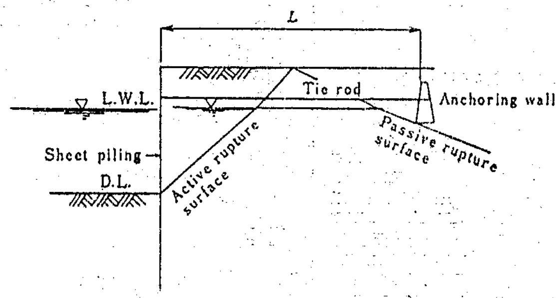


Fig. 10 . Location of Anchoring Wall

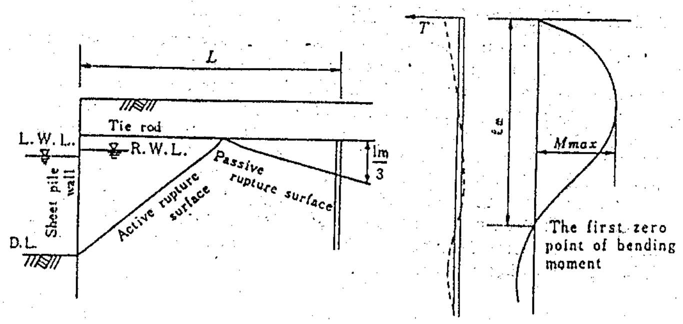


Fig.11. Location of Anchoring Pile

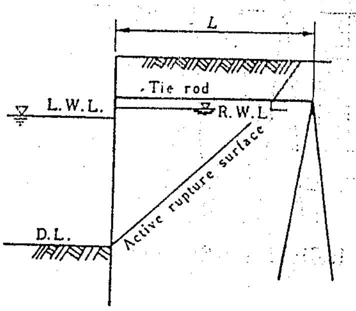


Fig. 12. Location of Anchoring Batter Piles

This type of structure is suitable for off-shore reclamation projects as it has relatively high self-standing stability. It is also suitable at sites where tie-rod installation is impossible due to buildings or other obstructions. Long batter piles are required as they are subjected to pull-out force. Application of this type of structure is not available locally.

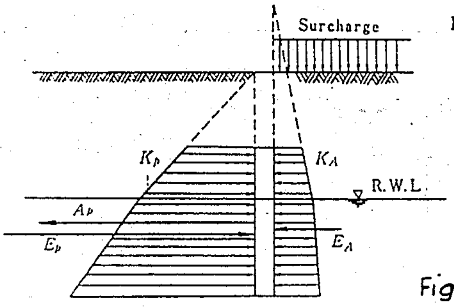
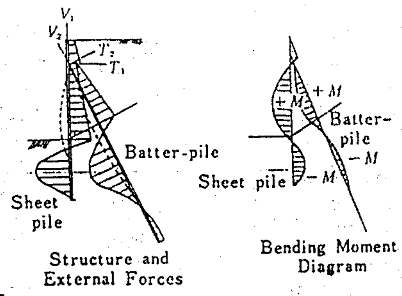


Fig.10A
External Forces Acting on Anchoring Wall



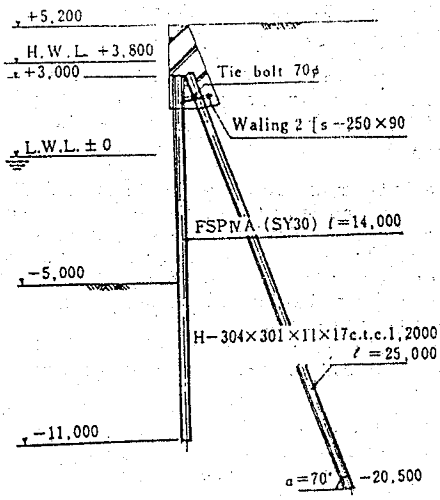


Fig. 16, Batter-Pile Anchor Structure

4.2.2.3 Relieving Platform Type

The structure with a heavy platform which carry surcharge loads and most of the fill above water level relieves earth pressure on the sheet pile wall. The load on the platform is transmitted to ground by piles thereby decreasing the lateral pressure of the fill on the sheet pile wall. Fig. 17 shows the pressure distribution on this type of structure. The horizontal force transmitted from sheet piles to the platform is the tie-rod reaction similar to that from tie-rod type structure.

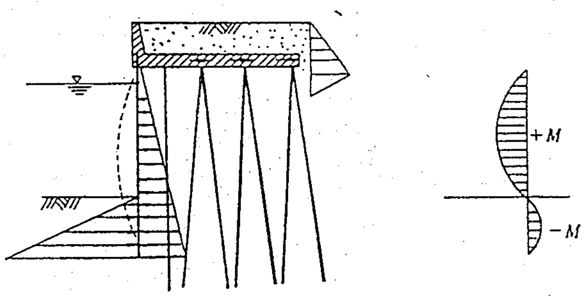


Fig. 17. Relieving Platform Type Structure

Relieving platform type structure is an expensive construction and it is generally suitable for deeper water depths than the ordinary sheet pile wall can serve. Application of type of structure is not found here.

4.2.3 Cellular Type

The cellular type sheet pile structure is constructed by driving interlocking straight-web sheet piles to form a circular cell and in-filling it with granular material such as sand or gravel. This structure behaves as a gravity wall having good stability. The individual cells are by connected by connection arcs and the spaces are filled with sand. Fig. 18 illustrates Cellular type wall.

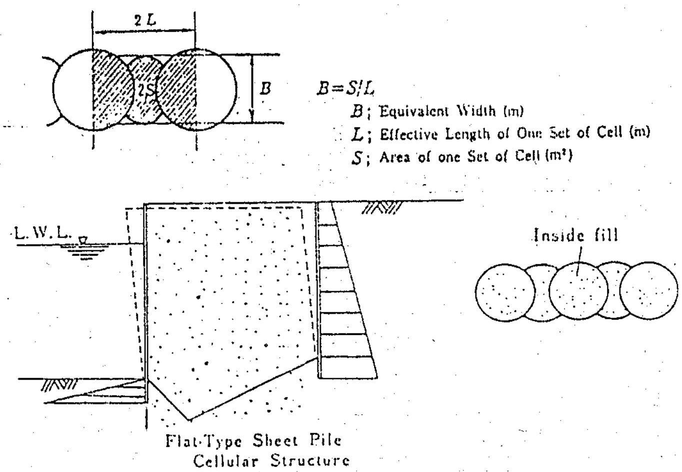


Fig. 18. Cellular Structures

Cellular type structure is used as permanent waterfront structure or as temporary cofferdam. This structure is applicable when subsoil is unstable at great depths or in deep water.

Individual cells are independently stable and each cell must be filled as soon as the driving operation is completed because the structure is not stable unless it is in-filled. The deformation or overturning moment due to lateral earth pressure is resisted by the resisting moment due to filling and due to

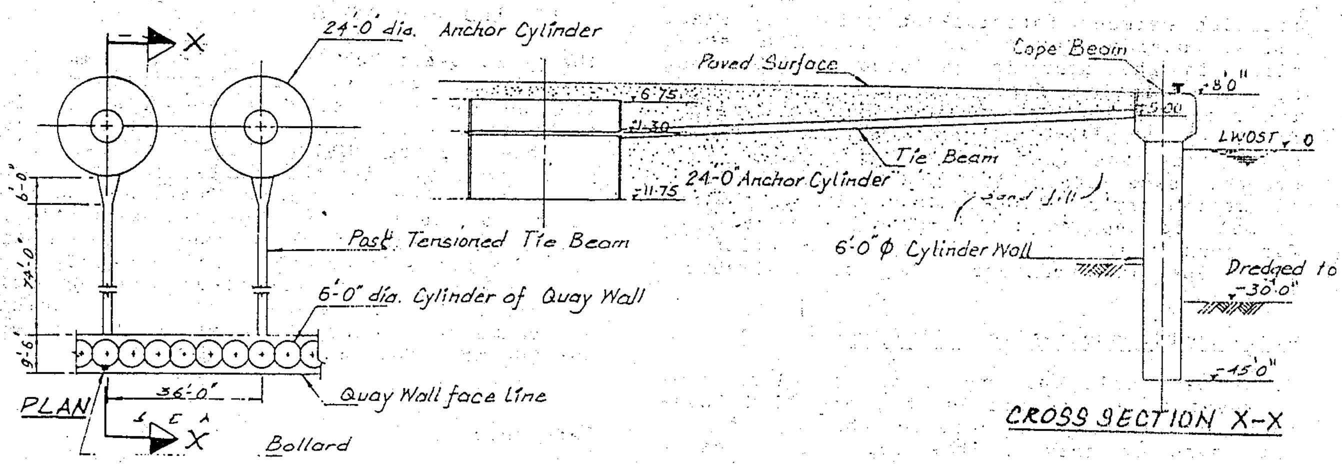


Fig 14 :- GALLE MAIN QUAY WALL

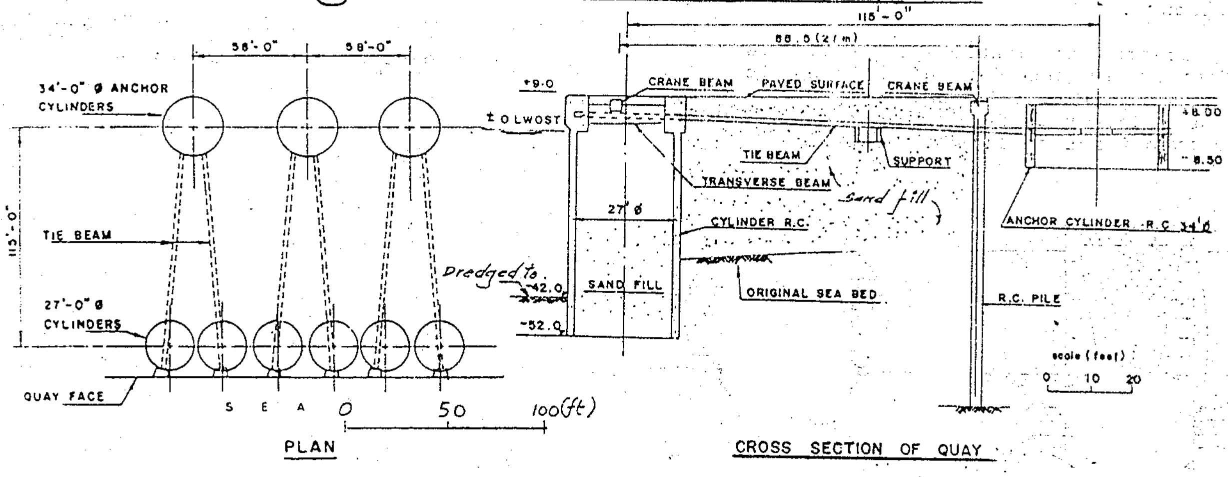
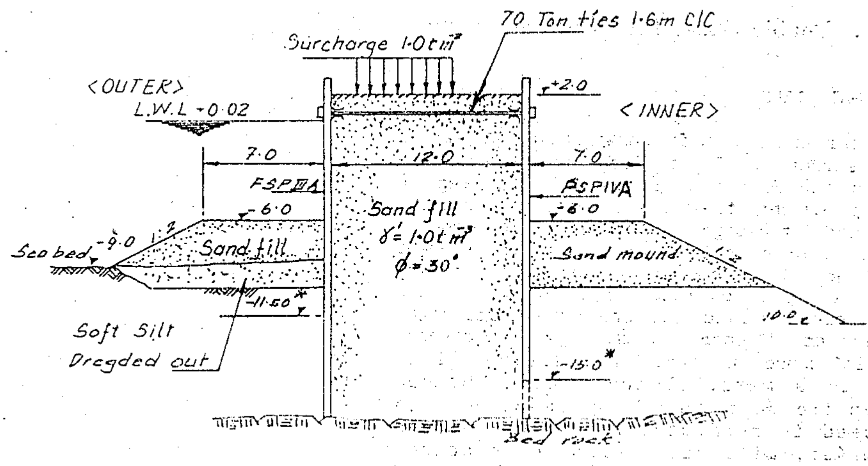


Fig 15 :- QEQ CONTAINER TERMINAL



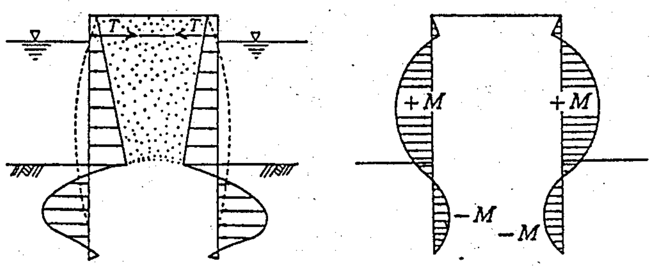
* required minimum for stability.

Fig 20 :- TEMPORARY COFFER DAM FOR 90,000 DWT DRY DOCK friction between interlocking joints of sheet piles. During deformation cell, the sheet piles in the rear is resisted from pullout by the friction forces of the backfill on the pipes to the full height. For this reason front sheet piles are embedded to a deeper depth than the rear piles. In case of soft ground, same depth is used for all piles. In design calculations, an equivalent width of wall is adopted. The cell wall is designed for hoop tension. Example of application of this type of structure is not available locally.

4.2.4 Double sheet-pile Wall Type

With the double wall type, two parallel rows of sheet piles are driven, they are tied together and the space between them is filled with sand or gravel. This type of structure is widely used for cofferdams, also appplicable for dock walls. See Fig. 19 for this type of structure.

An application of double sheet-pile wall for a cofferdam in the Port of Colombo is shown in Fig. 20. In this case, the two rows of piles were driven to bed rock for water sealing rather than for stability purposes.



Structure and External Forces

Bending Moment

Fig. 19. Double Sheet Pile Wall Structure

5. CONSTRUCTION MATERIALS

Concrete and steel are the main construction materials used in the construction of waterfront structures. The use of steel in the marine environment is limited in this country in permanent structures in view of corrosion problems. One important design factor to take into consideration in use of steel piling in aqueous environment is the effect of corrosion.There are marine steels having high resistance to corrosion that may be used. In view of extensive studies carried out and more information available, the effects of corrosion can be better assessed for their influence on structure service life. The concrete is a material that has withstood the marine environment for long years and is extensively used in such structures. Reinforced concrete, however, will have corrosion problems of steel reinforcement if proper design and quality control is not ensured.

6. CONCLUDING REMARKS

The paper describes the different types of Waterfront earth retaining structures with less design details. More emphasis is given to types of structures which has been applied in this country. While there are other examples available in the particular type only a few have been presented. There are no rigid rules to specify one type for a particular site but often more than one structure will satisfactorily meet the soil Conditions at the same site from a soil-engineering point of view. It is necessary also to combine environmentatl conditions as well and work out the more enconomical solution.

References

1. Technical Standards for Port and Harbour Facilities in Japan - OCADI of Japan.
2. Steel Sheet piling Manual - Nippon Steel
3. Soil Mechanics by Tschebotarioff.

EXCAVATIONS IN DEEP TRENCHES

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GENERAL

Project considered is the Greater Colombo Sewerage Project for the National Water Supply & Drainage Board. Work consisted of the laying of 21 km of forcemain and 60 km of gravity sewer. Pipe sizes varied from 200 mm to 1500 mm and laid at depths between 1.0 m and 9.5 m. This paper will deal with work ranging in depths from 6 m to 9.5 m and excavation done by the "Open Cut" method.

The contract involved laying of sewer pipes in highly residential areas, across and along busy streets, across water ways, under railway tracks etc.,

It is generally known in the construction industry relating to underground work, the costs of construction and efficiency of operations depend on the reliability and completeness of the information provided in the tender documents. When such information is lacking, there is a tendency for the contractor to "Pad" his price to include for unforeseen circumstances and thus resulting in higher prices.

During the construction stage, the Engineer must provide whatever assistance, within his means, to obtain proper co-ordination with utility owners to minimise damages and to co-ordinate repair work in the event of such damages as per contract specifications.

The successful completion of such projects will also depend on enforcing quality control, rates of progress as per agreed schedules, handing over and commissioning on an area basis to minimise inconvenience to the public. In this respect, the public must be warned and educated regarding the inconveniences they may have to encounter during work of this nature.

1. Design Consideration;

A successful design of a pipe line project depends not only on the efficient functioning of the system, but also on the selection of the routes to minimise inconvenience to the public, damages to utilities and properties. In projects requiring deep excavations, the latter considerations may decide the actual routing of the pipe line. In the selection of a route, the following considerations are suggested.

i) Utilities

Obtain accurate information from utility agencies of underground utilities and structures (Manholes, valve chambers, etc., as these are sometimes covered by road work contractors). Where accurate information is not available, determine the locations of utilities by tracing devices or series of trial excavation. As most utilities are usually located within 1.5 meters from the surface, this exercise can save large sums on subsequent costs.

ii) Soil Data

2" bore holes up to the depth of excavation at reasonable intervals to be taken and the information provided on the tender documents.

iii) As deep excavations require large and heavy equipment, avoid routing of the pipe lines through narrow streets, where building foundations can be undermined. Overhead cables, poles etc. can interfere unless these can be removed ahead of construction and re-installed after the work is completed.

iv) When sections of streets have to be closed for construction, ensure that alternative access can be provided.

v) If routes that can avoid the above mentioned problems are not available, methods of construction such as jacking or boring (tunnelling) must be allowed for in the design.

Having given regard to the suggestions outlined above, the pipe line route can be determined. Drawings must then be furnished to show the alignment with "tie measurements" at reasonable intervals. Where special techniques such as boring is anticipated, these areas must also be so indicated.

2. Contract/Specification Consideration;

The purpose of the Engineer giving information outlined in section 1 is to minimise the contractor unnecessarily "Padding" his price. Most specifications allow the owner/Engineer some room regarding the accuracy and /or completeness of the information provided in the documents (Clause IT-08 FIDIC). The lack of information may help the contractor to make subsequent claims for extra work and delays.

Specifications must adequately cover regarding the enforcement of quality control, progress of work, use of proper equipment, minimising of damages and repair to such damages without delays.

Adequate Insurance cover must also be specified and obtained.

3. <u>Construction Aspects of Project Under Study</u>

Basic difficulties encountered by the contractor during construction were due to;

i) Having to work in heavily built up areas.

ii) Inability to obtain accurate records of existing underground utilities from relevant authorities.

iii) Work in narrow streets, with heavy equipment.

iv) High water table in many areas requiring extensive dewatering.

These factors will be elaborated and the different stages of the work involved is discussed next.

4. <u>Preparatory Work:</u>

This work consisted of;

i) Study of the available work area and above ground obstructions.

ii) Study of the proximity of the buildings on either side and type of construction of the buildings. (ie. masonary, reinforced concrete etc).

iii) Study of the type of soils and of the water table.

iv) Trace of underground utilities and obtaining Engineer's instructions regarding alignment of sewer and removal (by the respective agencies) of utilities that would interfere.

v) Planning of traffic flow as extent of the work may make it necessary to close streets and provide alternate routes.

5. <u>Selection of Equipment and Method:</u>

The size of excavator required is primarily decided by the depth of excavation. However, due to restrictions such as availability of work area, a smaller machine may have to be used. If this machine cannot reach the full depth of excavation required, the lower-most excavations have to be done manually.

i) <u>Trench Box/Steel Plate Method</u> (see Fig-1)

Whatever possible, the method of shoring used was of the trench box/steel plate method.

This method works well where sandy soil with little or no water is encountered, or with clayey soil. Steel plates are driven

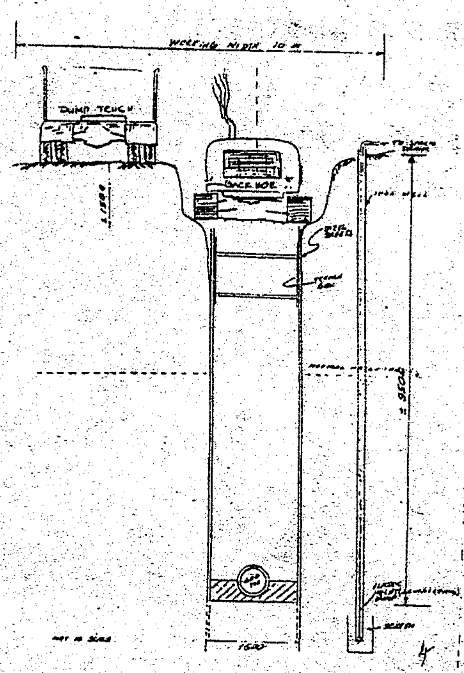


FIG. I

into the ground by the excavator. In instances where good laterite was encountered, 6 - 7 m deep excavations were carried out without a shoring system.

ii) <u>Interlocking Sheet Piling Method</u>

When depths of excavation exceeded 8 m sheet piling method was used. This method had to be used due to practical difficulties of handling and driving steel plates more than 6 m in length. Also in cases where high water table was encountered in sandy soils, interlocking sheet piles had to be used to prevent the flow of sand along with the water between steel plates.

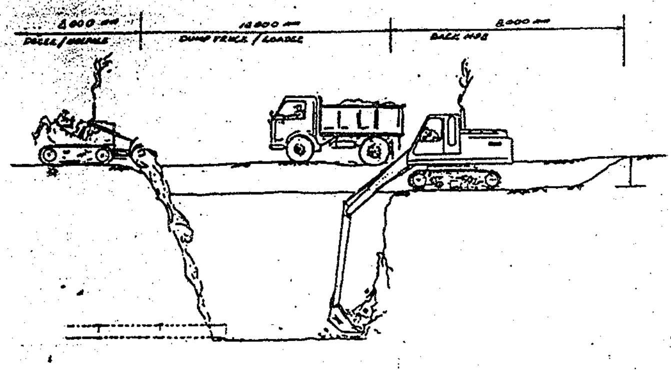
6. <u>Construction Procedures:</u>

For the two methods outlined, typical operations and procedures used during construction are depicted in Fig 2. The normal 'cut' excavated at a time was 6 - 7 m long to accommodate 4 Nos. PVC pipe x 1.5 m long or 1 No. GI pipe x 6 m long or 2 Nos. R.C. pipe x 3 m long.

<u>Trench/Box Steel Plate Method:</u>

Typical Equipment used are;

i) Excavator
ii) Loader
iii) Dump trucks
iv) Compactors (for back fill)
v) Generators and pumps (for dewatering)
vi) Trench box and steel plates



LONGITUDINAL SECTION OF PROPOSED TRENCH OPERATION

FIG. 2

Typical crew consisted of;

1 Foreman
1 Surveyor
2 Pipe layers
3 Operators
3 Drivers
1 Mechanic
1 Electrician (when generator and pumps are required)
1 Plumber (to repair damaged water mains for service connections)
3 Labourers

Procedure: Alignment of sewer is given by the Engineer with the measurements of manholes for gravity sewers and tie measurements at suitable locations for forcemain. Setting out with this information and levels given, is done by the crew Surveyor. Due to the non-availability of accurate records of the existing underground utilities about 1 m - 1.5 m is initially excavated to ensure that the given alignment can be maintained. If utilities interfere with the alignment, the Engineer will give appropriate instruction either to alter the alignment or remove the utilities (by the relevant authority) and replace after construction of the sewer. The trench box is then lowered into this cut and the steel plates are driven using the excavator to a depth below the invert of pipe.

The trench is then excavated to the required depth, pipe laid and backfilled.

ii) Sheet Piling Method:

In very deep excavations when the method of sheet piling is used to shore the trench, the sheet piles are driven, after the initial excavation of 1- 1.5 m is carried out to locate the underground utilities. The balance operations and dewatering done is similar to that of the trench box.

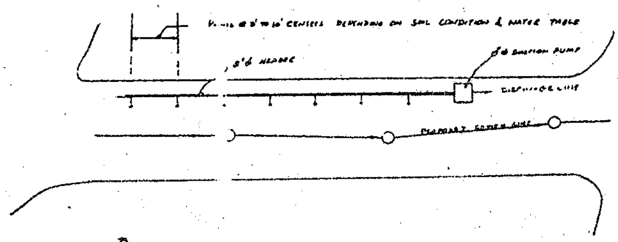
Additional equipment needed are;

crane and vibro hammer

DEWATERING

Trenches are required to be reasonably dry at the time of placing of bedding and pipes. In order to satisfy this condition, adequate number of pumps with required capacities are essential.

In sandy soils located in areas where the water table is very high (such as coastal areas of Dehiwela-Mount Lavinia) dewatering has to be done at least 2 weeks prior to excavation being commenced. This was achieved by using a system of "well points" (see Fig 3) upto 6 -7 m depths. Deeper excavations required a cmbination of "well points" and deep wells (usually located at 1 m to 4 m c/c). Construction of deep wells have to be carried out carefully with proper filters and screens to prevent the migration of "fine sand" from the soil.



PROPOSED WELL POINT INSTALLATION (TYPICAL)

FIG. 3

RETAINING WALLS IN ROAD WORKS

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1.0 Introductory Remarks

Retaining walls constitute essential structural elements in roads and functionally they could broadly be grouped as follows :-

(a) Structures supporting road embankments
(b) Structures supporting road cutting along hill side slopes
(c) Structures used as preventive measures against landslides
(d) Structures used as abutments and wingwalls of bridges

The last of the above is a subject for another paper and as such aspects regarding 1,2 & 3 only will be discussed in this paper.

As regards (a) above, the retaining walls will take the form of "toe walls" supporting a soil surcharge in most instances as indicated in figure 1 (annex 1). There could also be instances where the retaining wall will extend right up to the top of the embankment fill as indicated in figure 2 (annex 1).

Normally these retaining walls are first constructed and are backfilled with suitable soil. As such the structure could depend on the infill soil to provide for part of the stability against overturning or rotation.

In the cases of (b) & (c) above, the type of structure may have be such that during construction it will cause the least disturbance to the soil layers that the retaining wall is meant to provide the support for. As such it will not always be possible for these structures to depend on the supporting soil for stability. Such structures may get partial support from the soil infill as indicated in figure 3 (annex 1) or no such support at all as indicated in figure 4 (annex 1).

2.0 Types of Retaining Walls in use

Most of the retaining walls found along roads in this country are those that depend on their mass for stability and are mostly built of random rubble (RR), mass concrete or dressed stone. Of these the dressed stone type belong to the initial period of road construction. In present day context, however, RR predominates.

"Flexible" retaining walls using Gabions have been tried out on an experimental basis and found to be successful particularly in containing landslides. High initial costs however, has made this type of construction less attractive. Durability of the mesh used in gabions is another aspect that concerns the highway designer.

Very little consideration has been given to retaining wall construction using reinforced concrete (RC) although it could be argued that they will be highly competitive, if not more advantages, in the case of structures taller than about 5m (16 ft). It is fairly well realized that to retain a hill side cut, where the base of the structure has to protrude out, as indicated in figures 3 & 4 (annex 1), RC sections are more advantages. Some efforts have been made in recent times in the use of RC construction, under such circumstances.

3.0 Designs Considerations

Essential designs considerations regarding retaining walls are :-

- Stability of the structure against rotation and sliding
- Stability of the supporting soil against yielding.

3.1 The forces that have to be dealt with generally, regarding the stability of the structure are :-

(a) Destabilizing forces

(i) Active pressures of retained soils
(ii) Pore water pressures within the retained soils
(iii) Surcharge loading due to road pavements and traffic, normally considered as a uniformly distributed load (UDL) - see figures 1&2 (annex 1).
(iv) Surcharge loading due to other structures such as buildings see figure 5 (annex 1).

(b) Restraining forces against sliding

(i) Frictional resistance of the supporting soil at the base
(ii) Passive pressure of the soil in front of the wall

(c) Restraining forces against rotation

(i) The weight of the structure, and also the weight of the soil fill behind the wall where applicable
(ii) Forces due to any anchoring of the wall which could restrain both rotation as well as sliding - see figure 5 (annex 1).

Active and Passive pressures of soils

An accurate determination of active and passive ssures of the soils, acting on a retaining wall, will ually not be possible particularly because of the non formity of the soil type or types encountered and the iations of their denseness. However, assuming that the ls are reasonably uniform and could be categorised, and assuming that they are reasonably well compacted, earth ssure theories have been made use of to derive formulae their calculation.

The reader is referred to the Designer's Hand Book by nolds and Steedman[1] where such formulae are given in pect of several conditions of the backfill and in ation to several types of soil.

The reader is also referred to the charts given by zaghi and Peck[2], which combines theory and practice in semi-empherical manner to give a method of assessment of active pressures of the backfill in relation to the soil e and in relation to several conditions of backfill. For s purpose five types of soils have been identified.

Terzaghi and Peck[2], however, does not attach much ortance to the passive pressures of the soil located in nt of the wall and infact suggests that this could be lected in some instances.

In the use of these formulae and the charts an essential uirement is the identification of soil type encountered as such the importance of carrying out soil surveys or to the designing of retaining walls, has to be essed.

Pore Water pressure within soils

The pressures due to pore water in the retained soils y from season to season in addition to variations due to l type and to the denseness of the soils. These pressures ld reach serious dimensions in the case of hill slopes. such adequate preventive measures such as providing an regate filter layer behind the wall and weepholes in the l has to be given due consideration. Instances are many, retaining walls having been pushed out and/or tilted, and o collapsed, during rainy seasons, as a result of essive build up of pore water pressures in the retained ls.

Foundations of retaining walls

Terzaghi and Peck[2] states that most retaining wall lures are caused by inadequacies of foundations. This tement holds good in our context too, and instances are y of collapses of road side retaining walls due to the e reason.

Very little attention is normally paid to investigations assessment of bearing capacities of the supporting ls, particularly of the layers benieth. It has not become standard practice in this country to carryout sub-surface estigations prior to the construction of retaining walls ept under exceptional circumstances.

In the design of retaining walls, it is a standard ctice to ensure that the resultant of the forces fall hin the middle third of the base. This is a good practice particularly with gravity walls, as it ensures that no tension is developed at the base. However, in the limiting case of this consideration the stress at the toe becomes double that of the average, which is the value that needs to be watched in the design. Where this value of stress is not given sufficient consideration yielding of the soil at the toe takes place normally resulting in tilting and/or cracking of the wall.

For consideration of the reader some allowable bearing pressures depending on the soil type are given in table 1 (annex 2). However, these are meant only for small scale routine jobs and where the possible settlements are of a small order.

3.7 Safety against sliding

The sliding of a retaining wall is prevented by the friction between the soil and the base and also by the passive pressure of the soil in contact with the outer face of the wall foundation.

To ensure that there is no appreciable sliding of the wall a factor of safety of 1.5 is normally provided and the recommended values of frictional coefficient, by which the normal load should be multiplied to obtain the frictional resistance, varies between about 0.35 to 0.55 depending on the soil type and its degree of compaction.

Thus it would be seen that the selection of a proper coefficient of friction, depending on the type and condition of the soil, is an important consideration in the design of retaining walls.

4.0 Aspects of construction

As indicated earlier, most retaining walls along our roads are of gravity type and are normally constructed using RR. Some of these have also been constructed using mass concrete.

In the case of RR construction, it is a condition written into the new specifications[3] that through-stones should be provided at regular intervals. These stones have a tying-up effect of the wall, which is necessary particularly because of the poor jointing of the RR that is normally carried out.

Construction using mass concrete should be promoted wherever possible as mass concrete could be laid more uniformly unlike RR provided steel shutters are used and provided a reasonable control is exercised during mixing and laying of the concrete.

Flexible retaining walls, constructed using Gabions also should be promoted particularly for hill side slopes that are prone to slides. The cost effectiveness of the use of plastic coated mesh for the gabions has still to be established and this has be studied.

Cost effectiveness of RC construction for situations discussed in the paper has also to be studied in greater detail.

5.0 Concluding remarks

Retaining walls constitute "high unit cost" items in road construction and road rehabilitation and as such deserves a more careful investigation process and designing process than presently being adopted. There is no doubt that this could be a topic for a systematic research study by a selected group of persons.

REFERENCES

(1) Reinforced Concrete Designer's Handbook
by Charles E Reynolds & James C Steedman
9th Edition.

(2) Soil Mechanics in Engineering Practice
by Karl Terzaghi & Ralph B Peck
Second Edition.

(3) Standard Specifications for Construction and
Maintenance of Roads and Bridges
1989.

Annex 2

Table 1 - Bearing pressures of soils.

Type of Soils	Safe bearing pressure KN/m² (T/sq.ft)	
(a) Cohesion-less soils	Dry	Sub-merged
(i) Compact well graded sands and gravel sand mixes	400-600 (4-6)	200-300 (2-3)
(ii) Loose well graded sands and gravel sand mixes	200-400 (2-4)	100-200 (1-2)
(iii) Compact uniform sand	200-400 (2-4)	100-200 (1-2)
(iv) Loose uniform sand	100-200 (1-2)	50-100(1/2-1)
(b) Cohesive soils (other than lateritic soils)		
(i) Stiff clays and sandy clays	200-400 (2-4)	
(ii) Firm sandy clays	100-200 (1-2)	
(iii) Soft clays and silts	50-100 (1/2-1)	
(iv) Very soft clays and	0-50 (0-1/2)	
(c) Lateritic soils		
Hand Crust	400-600 (4-6)	
Intemediate layer	150-400 (2-4)	
Clayey layer	100-150 (1-2)	
(d) Gravelly soils		
Gravelly sand clay mixes, loose --	100-200 (1-2)	
Gravelly sand clay mixes, medium--	200-400 (2-4)	
Gravelly sand clay mixes, dense --	400-600 (4-6)	

Annex 1

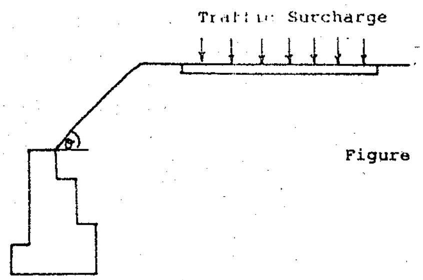

Figure 1

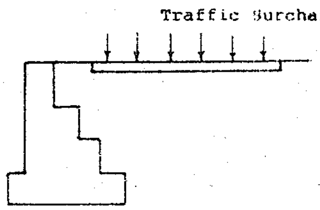

Figure 2

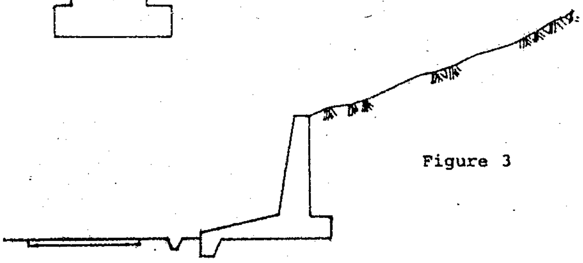

Figure 3

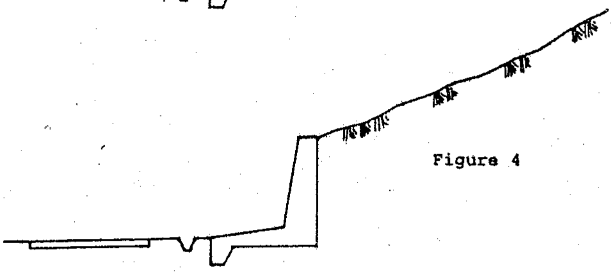

Figure 4

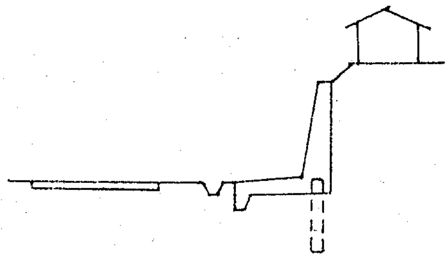

Figure 5