Sri Lankan Geotechnical Society

Seminar on

Earth Retaining Structures

At ICTAD Auditorium
on Thursday 24th September 1998

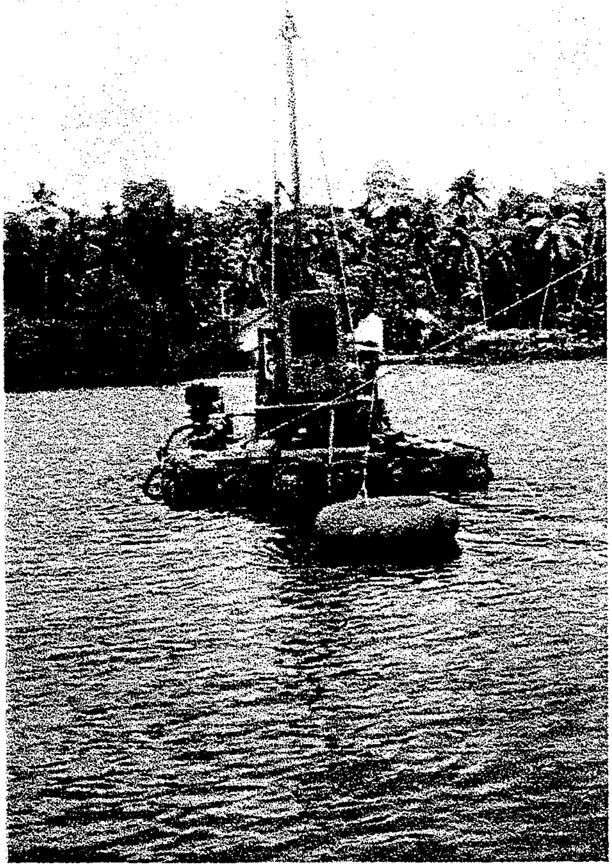
SRI LANKAN GEOTECHNICAL SOCIETY

Seminar On Earth Retaining Structures

24th September 1998 At ICTAD Auditorium

Agenda

9.00 - 9.15 AM	-	Registration
9.15 - 9.20 AM	-	Lighting of Oil Lamp
9.20 - 9.30 AM	-	Welcome Address Mr. D. P. Mallawarachchi – Vice President - SLGS
9.30 - 10.30 AM	-	Different forms of Earth Retaining Systems their Mechanisms, Methods of Design and Construction Dr. S A S Kulathilaka
10.30 – 11.00 AM	-	Tea
11.00 – 11.30 AM	-	Construction of Reinforced Earth Retaining Structures on Rathnapura – Wellwaya Road in Sri Lanka Mr. D P Mallawarachchi
11.30 – 12.00 AM	-	Stabilization of Slopes by Anchored Tyre Retaining Structures Mr. I H D Sumanarathna
12.00 – 12.30 PM	-	Model Studies on Anchored Tyre Earth Retaining Structures and Development of a Design Method Mr. S A S Kulathilaka
12.30 – 1.00 PM	-	Discussion
1.00 - 2.00 PM	-	Lunch
2.00 - 2.30 PM	-	Jet Grouted Piles as Earth Retaining Structures – A Case Study Mr. S A Karunarathna
2.30 - 3.00 PM	-	Experiences in use of a Anchored Bored Pile Retaining Wall in Central Bank Building Extension Project Mr. K L S Sahabandu
3.00 - 3.30 PM	-	Discussion
3.30 - 3.40 PM	-	Vote of Thanks Dr. P Rathnaweera
3.40 - 4.10 PM	-	Tea
4.10 - 5.00 PM	-	General Meeting of SLGS



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Proceeding of the SLGS Seminar on Earth Retaining Structures

Contents

Different forms of Earth Retaining Systems their Mechanisms, Method of Design and Construction

Dr. S A S Kulathilaka – University of Moratuwa

1. Introduction

Different forms of earth retention systems in use today can be separated into two broad categories as
(a). Externally stabilized systems and
(b). Internally stabilized system

Several hybrid systems which incorporate concepts of both systems are also developed in recent times. This paper will discuss the fundamental differences in these different systems, their supporting mechanisms, available methods of design and special features in the construction.

An externally stabilized system uses an external structural wall against which stabilizing and disturbing forces are mobilized. All the traditional retaining walls may be regarded as externally stabilized systems. Their supportive action could have been derived from their weight, their flexural rigidity or by a combination of the both. Number of retaining systems that can be rapidly constructed and are capable of tolerating significant differential settlements were developed in recent times.

An internally stabilized earth retaining system consists of reinforcements installed within the soil and extending beyond the potential failure mass. Shear stresses mobilized in the system to resist outward soil movements are transferred to the reinforcing elements developing tensile forces in them. Reinforcing elements should have a sufficient tensile strength to withstand these forces. Similarly pulling out of the reinforcements should be prevented by embedding them over a sufficient length beyond the potential failure zone or by providing suitable anchor elements.

The closely spaced reinforcements and the soil behaves as one structural unit. A facing is used simply to prevent local raveling, erosion and deterioration. It does not carry any significant structural load. The key aspect of an internally stabilized system is its incremental form of construction. In effect the soil mass is partitioned so that each portion receives support from a locally inserted reinforcing element. In contrast, with conventional externally stabilized structures pressures are integrated to produce an overall force to be resisted by the external structure.

2. Externally Stabilized Structures

2.1 Gravity Retaining Structures

Externally stabilized form of structures that rely completely on their weight to ensure stability are referred to as gravity retaining structures. They should be sufficiently heavy so that the resisting moments necessary to overcome the overturning effects of the backfill, and the resisting forces necessary to overcome the sliding forces exerted by the backfill, can be mobilized. It is also necessary to ensure that the bearing stresses exerted in the founding soil due to the combined effect of self weight forces and the backfill exerted forces are within allowable limits. They can be stepped or battered backs or fronts (Figure 1). A small amount of back batter (up to about 10°)

at the back face of the retaining wall can significantly reduce the earth pressure acting on the wall and even out the bearing pressure transferred to the ground.

Gravity retaining structures are traditionally made of mass concrete, or brick or random rubble masonry. These are fairly rigid walls and require some time for hardening after construction. Also they will show signs of distress due to differential settlements.

Gabion walls, crib walls and different forms of interlocking modular block walls are more flexible forms of gravity retaining structures. They also have the advantages of being able to construct rapidly and being usable immediately after construction. Their flexural nature and ability to accommodate significant differential settlements make them ideal solutions under unfavourable foundation conditions

Gabion Walls

The main unit here is the gabion. Gabions are large cages or baskets usually made of woven steel wire or square welded mesh. These cages are filled with rock fragments and are tied together. Cages are of sizes such as 1m x 1m x 1m, or 1m x 1m x 2m etc and are usually made of galvanized iron wires. Gabions to be used in the aggressive environments have a pvc coating on the galvanized wire. To prevent clogging of the gabion unit by fine particles in the backfill, a geotextile filter is often used at the back of the wall. Gabions are found to be an economical option in waterfront structures where foundation conditions are not so sound.

Gabions made up of wickerwork, bamboo slats and nylon or polypropylene were developed in recent times. But there are also reported instances of fire damages to gabion walls made up of these flammable material. Some forms of gabion units and gabion walls are depicted in Figure 2.

Crib Walls

Crib walls are built up of individual prefabricated units assembled to create a series of box like structure containing suitable granular free draining fill. Crib units are made up of either timber, reinforced concrete or Aluminum. Both timber logs and sawn timber are used for the construction. Timber used should be treated to improve durability.

The front face of a typical crib wall consists of a grid of units spaced at close intervals so that the infill does not spill through the units. Horizontal members of the grid are known as "stretchers". They are connected by transverse members known as "headers" to a similar grid of stretchers forming the back face of the wall. Crib walls can be of either open faced or close faced form. (Figure 3).

The width of a crib wall is dictated by the length of the header unit and minimum widths are around 1.2m. Walls of smaller heights may be vertical but walls higher than 2m are usually built with a batter.

Interlocking Modular Block Walls

Huang (1997) presented the construction and analysis of interconnected "H type" concrete modular block wall. The behaviour of the wall was analysed by the discontinuous deformation

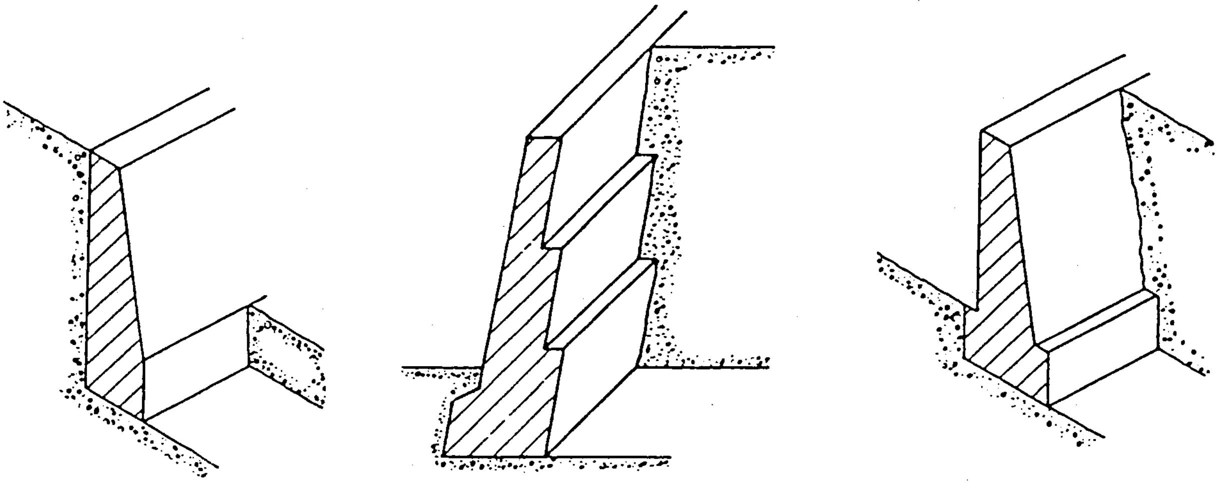


Figure 1 - Gravity Retaining Structures

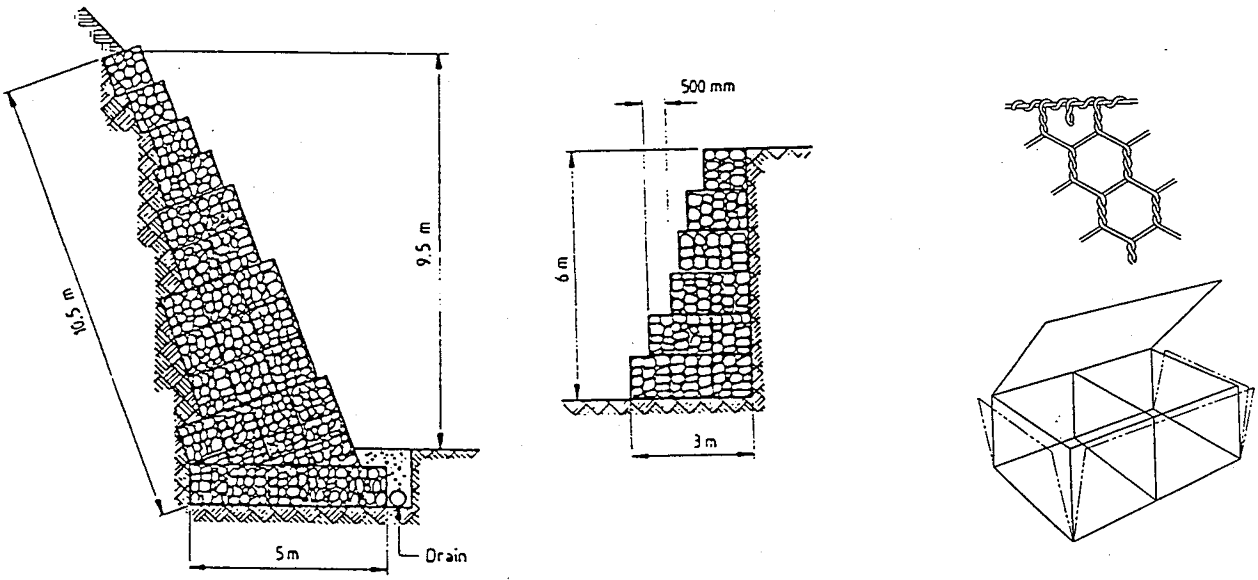


Figure 2 - Gabion Walls

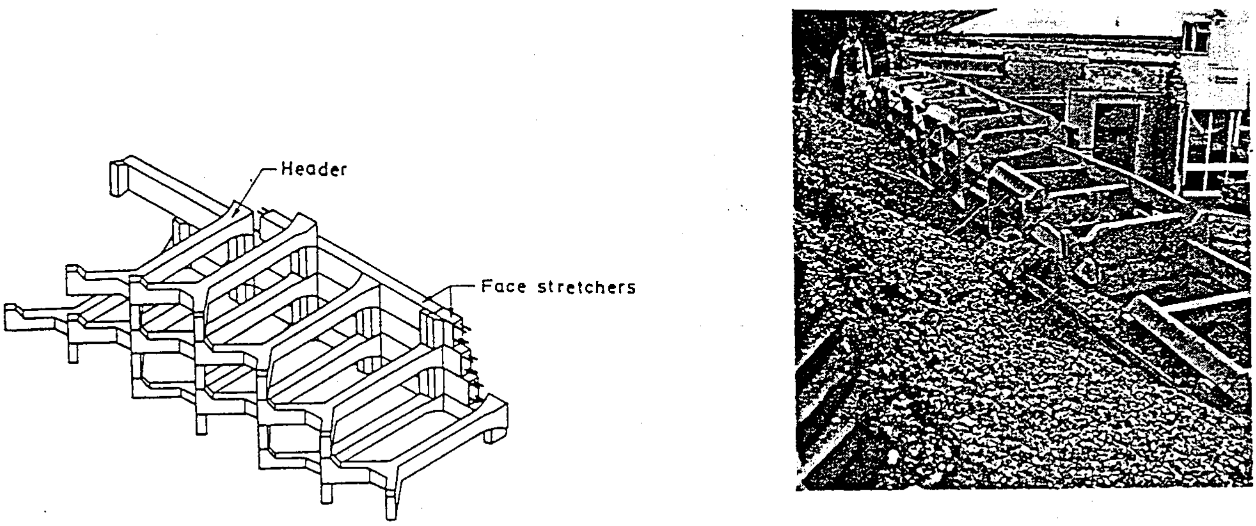


Figure 3 - Crib Walls

analysis (DDA) method which is another numerical procedure in the family of discrete element method (DEM). A 8.5m high and 35m long instrumented wall was constructed.

This construction incorporates two H type interconnected concrete blocks. Shearing and compressive resistance in the concrete blocks are developed during the stress transfer between blocks. Proper drainage, ability to accommodate large differential settlements, ease of construction and the cost effectiveness were listed as advantages in this system.

Sergent *et al* (1997) described the case study of providing temporary shoring using a 4m high interlocking modular concrete block wall.

Keystone walls are another commercially available modular retaining wall system. Walls up to 2m heights can be constructed in the form of gravity walls. Incorporating the principles of reinforced earth heights up to 12m can be achieved. Different forms of interlocking block walls are presented in Figure 4.

2.2 Embedded Retaining Structures

Gravity type retaining structures are not applicable for all situations. Vertical excavations required in basement construction etc. are supported by embedded retaining walls. They may also be used to stabilize earth movements and under difficult foundation conditions. They are generally thin structures embedded to a sufficient depth and can be with or without lateral support.

Cantilevered walls, that are not supported laterally derive their equilibrium solely from the passive resistance of the soil below the excavation level. Lateral support can be provided in the form of props from the excavation side or in the form of anchors behind the retained side. The lateral resistance from the support system will also help the equilibrium of the wall. The wall section used should be of sufficient flexural rigidity to withstand the bending moments generated due to the earth pressures acting on either side of the wall and forces from the lateral support system.

Cantilevered walls are economical only for small to moderate retained heights up to 3 – 4m. The required depth of embedment, the maximum bending moment in the wall and the lateral deflections increases rapidly with the height. Embedded retaining structures were traditionally referred to as sheet pile walls as they were formed by metal or timber sheeting. These sheet pile structures are mainly of temporary nature.

Advances made in the slurry trench excavation technique led to the construction of cast insitu concrete embedded walls that are referred to as concrete diaphragm walls. Embedded concrete retaining walls are also formed by a continuation of a line of bored piles. Concrete diaphragm walls and bored pile walls are of permanent nature and can be designed to be a part of the final structure.

Sheet Pile Walls

Sheet piles made of steel timber or precast concrete are driven to the desired depth from the ground surface and excavation is carried out to the desired levels. This method of construction does not demand good foundation conditions and the construction can be done underwater also. Boulders and stiff or dense soils encountered during driving and noise and vibration created are obvious disadvantages in this system.

Timber and steel sheet piles are generally used for temporary work and reinforced or prestressed concrete sheet piles are deployed for more permanent work. Pre cast concrete sheet piles with tongue and groove type arrangement can be made water tight by grouting. (Figure 5)

Use of prestressed concrete sheet piles will eliminate the development of tension in the pile and will prevent cracking. However the heavy weight of the generally thicker concrete elements and the imposed high lateral displacements are the disadvantages they have over steel sheet piles.

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

Insitu Concrete Diaphragm and Bored-pile Walls

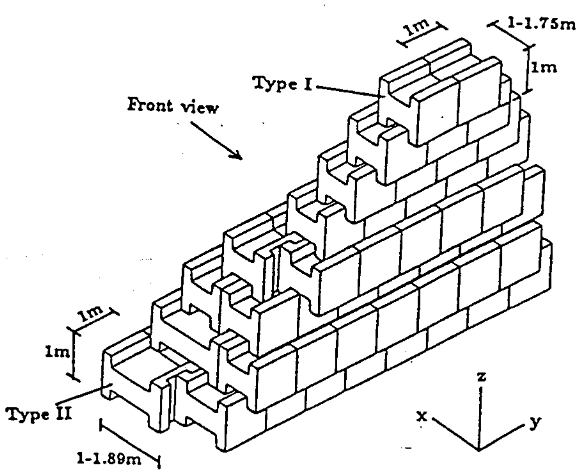
The use of diaphragm walling and bored pile wall techniques developed immensely over the last 2 to 3 decades due to the commercial availability of Bentonite, and new techniques developed with the experiences obtained. Diaphragm walling help to avoid the need for underpinning and ground water control and would allow maximum use of a small plot in an urban environment.

Conventional diaphragm walls are constructed in sections or panels (Figure 6). Excavation is made under Bentonite using a purpose built grab. Steel "stop – end tubes" are placed at the end of a panel prior to concreting. The shape of the "stop – end tubes" ensure that the adjacent panels interlock.

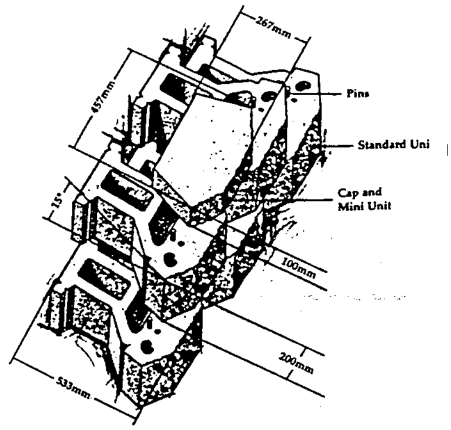
Bored pile walls have the advantage of being able to construct in almost any ground condition causing minimum disturbance to the surroundings. Bored pile walls may be ;
(a). Intermittent - centre to centre space exceeding diameter
(b). Contiguous - spacing = diameter - piles are in contact
(c). Secant - spacing < diameter - piles are interlocking,
depending on the support requirement of the wall and the need to control ground water. The top of the piles will often be capped by a reinforced concrete beam to distribute loads.

Lateral support for the concrete diaphragm wall or bored pile walls are provided by anchors placed at several levels or by propping via concrete basement slabs of the final structure. Lateral support is provided with the progression of the excavation in front of the wall. This method is quite widely used in the construction of the underground roads and other transport facilities. (cut and cover tunnels). Experiences in the construction of a secant pile wall that was laterally supported by several levels of anchors will be discussed in a separate presentation in this seminar.

In many practical situations it may be necessary to excavate to depths below the water level and to keep the excavation free of water. Dewatering inside the excavation will induce flow to the excavation beneath the toe of the wall causing lowering of the water table in the excavation side (Figure 8). Increase in effective stresses associated with this lowering could give rise to unacceptably high settlement in the structure at close vicinity to the wall. If it is found to be critical, concrete wall could be taken to the sound basement rock and socketed there to prevent seepage beneath the toe. This will result in the pore pressure distribution depicted in Figure 8 (b).

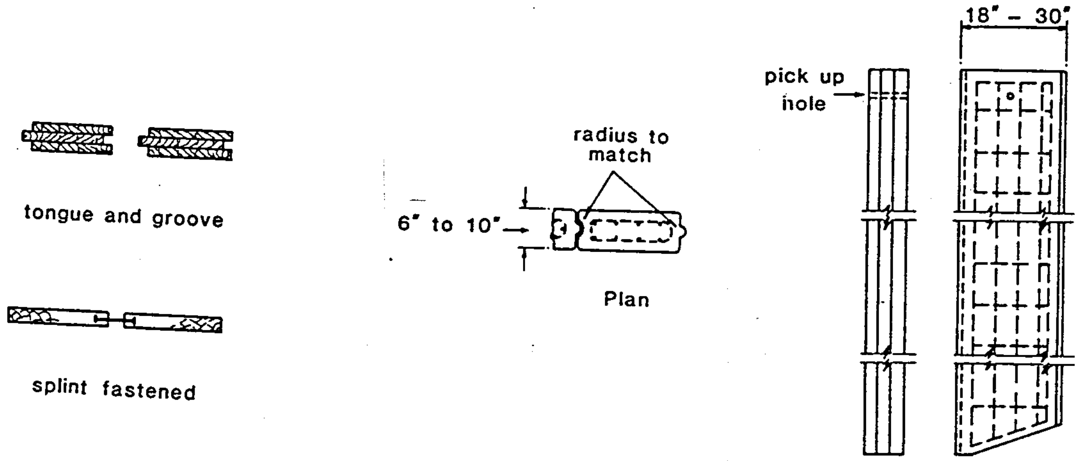


Typical H-Block Wall



KEYSTONE®'s patented interlocking system creates a strong, durable retaining wall.

Figure 4 - Interlocking Modular Brick Walls



(a) Wood sheet piles

side front

(b) Reinforced concrete sheet piles

Figure 5 - Standard Sheet Piles

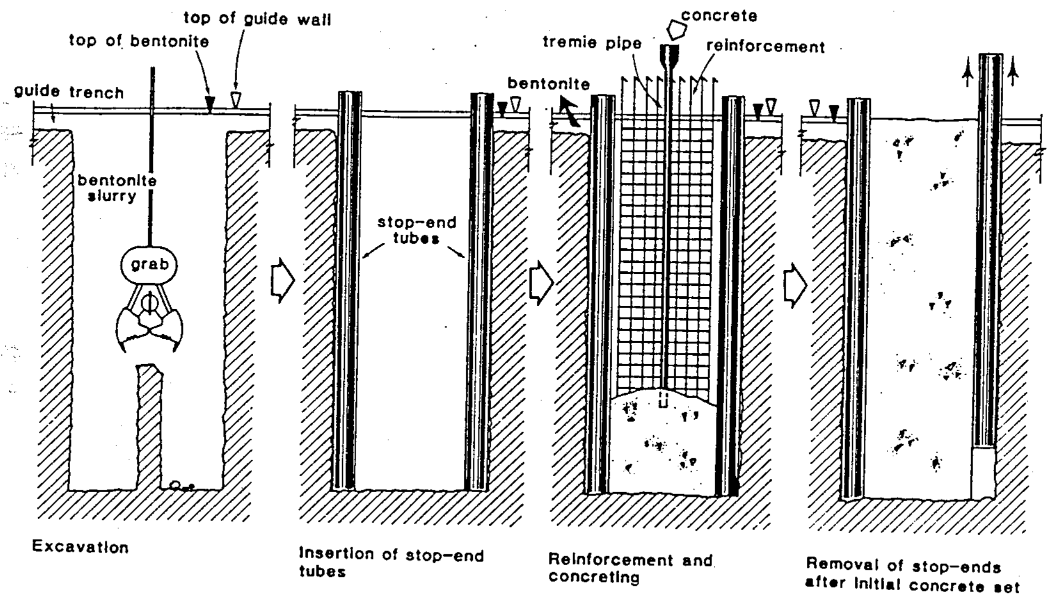


Figure 6 - Concrete Diaphragm Walls

Powrie and Chandler (1998) presented the case of supporting embedded retaining walls using a "stub prop" or "stabilizing platform" (Figure 9 (a)). This is expected to be more economical than providing an increased depth of embedement in the case of case of cantilever walls or installation of props reacting against the walls on the opposite side of a roadway.

The stabilizing platform is said to contribute to the stability by application of a point load to the wall and by application of a moment to the wall. The second aspect which results from the mobilization of the bearing capacity under the slab applies a moment to resist wall rotation. As this was a complex soil structure interaction problem it was studied through the finite element package CRISP.

The long term bending moment comparison for three unpropped walls of different embedment depths and the wall with a stabilizing platform are presented in Figure 9 (b). The effect of an increase in the length of the stabilizing platform to reduce the outward rotation of the wall about the toe is illustrated in Figure 9 (c). Hence they concluded that for the ground conditions analysed the provision of a stabilizing platform is more effective in reducing the wall movements than increasing the depth of embedment. The increase of the depth of embedment is found to cause substantial increase in the bending moments.

2.3 Walls Depending on Both Flexural Rigidity and Gravity

Reinforced concrete walls in the form of "L" or " inverted T", constructed in the cantilevered form or provided with increase in stiffness through "counterforts" or "buttresses" provide their supportive action through a combination of weight and flexural rigidity. Counterforts are provided at the back of the wall and buttresses are provided in front (Figure 10).

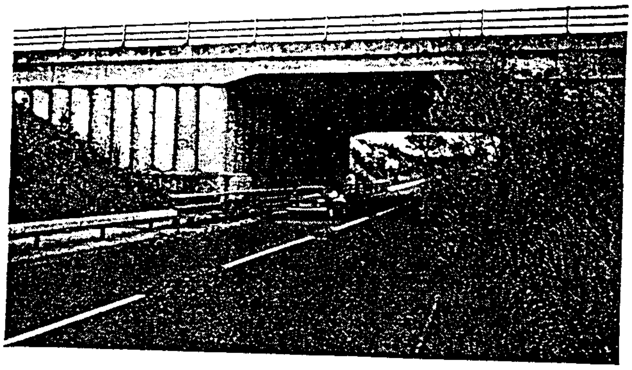
Stem of the wall is subjected to the bending moments due to the earth pressure from the backfill. Higher earth pressures that may exist due to backfill compaction should be considered in the design. If the wall is cantilevered the bending moments shall be computed accordingly. When lateral stiffness is provided through counterforts or buttresses stem can be designed as a continuous slab.

Weight of the soil resting on the heel portion of the wall acts together with the wall to provide stability against overturning and sliding. The vertical surface going through the heel of the wall is referred to as the vertual back of the wall. Overall stability analysis often shows that the sliding resistance is a problem. Sometimes it may be necessary to increase the sliding resistance by providing a shear key.

The form of a reinforced concrete wall section with a "Moment relief platform" (Figure 11) was found to be very effective in reducing lateral the earth pressures and therefore the bending moments in the stem.

2.4 Jet Grouted Pile Walls

A line of piles formed by the jet grouting technique can be designed to act as a retaining wall. In this technique grout injected at a high pressure liquefies the insitu soil and mix with it to form a grout pile which is of strength comparable with that of a soft rock or weak concrete. Most appropriate grout composition will depend on the soil type.



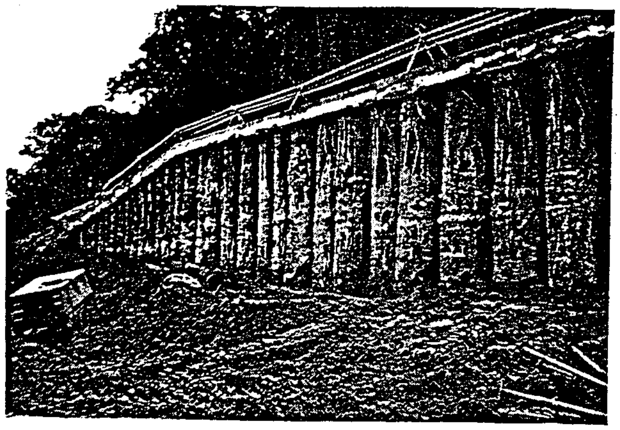


Figure 7 - Bored Pile Walls

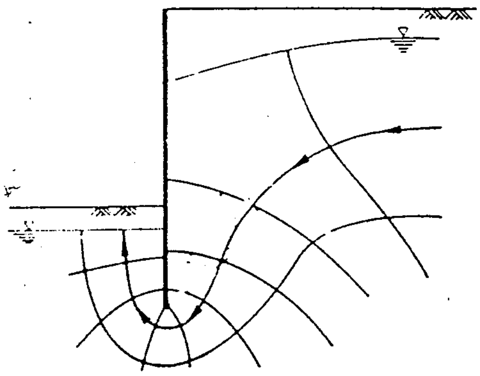
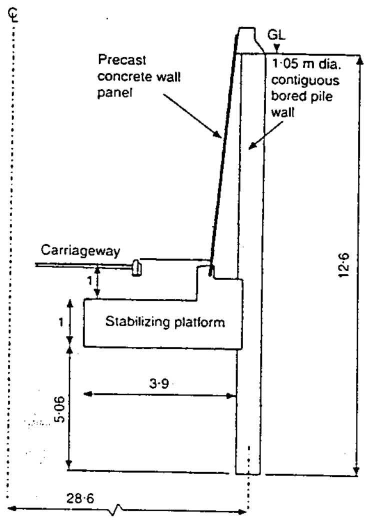


Figure 8 - Dewatering in front of Embedded Walls



.. . Cross-section through an embedded retaining wall with a stabilizing platform (dimensions in metres)

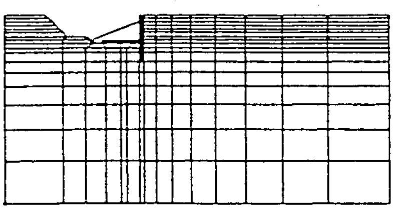
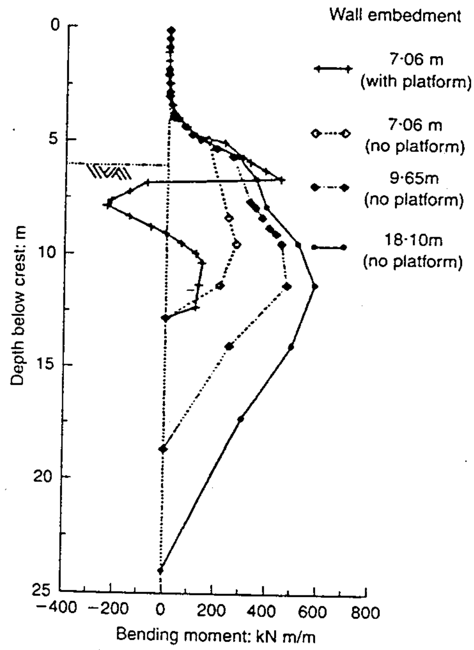
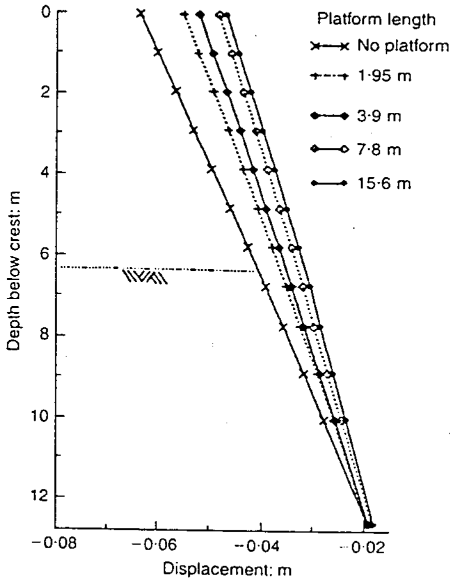


Figure 9 - Stabilizing Platform





Displacement profiles for walls of 7·06 m embedment with different platform lengths

They are generally of large diameters or several lines of them will be constructed to minimise the bending stresses developing in them. Flexural rigidity of the jet grouted piles can be enhanced by reinforcing them (usually with large H sections) at the liquefied state. The H section will simply go down due to its self weight.

A separate paper in this seminar will cover the experiences in design and construction of jet grouted piles in a building project in Colombo.

2.5 Solider Pile Walls

Solider pile walls with vertical solider piles of steel I sections installed at regular spacing and vertical sheeting and lateral supports provided with the progress of the excavation are widely used for excavations for basements of buildings. Normally wales are used at the levels of lateral support to provide a unified action.

2.6 Methods of Analysis and Design of Externally Stabilized Systems

Conventional Methods

Conventional design procedures for externally stabilized systems were based on classical earth pressure distributions and limit equilibrium type analysis. Retaining structure was assumed to move away from the retained side reducing the earth pressures to limiting active values in the retained soil. Dimensions of the gravity retaining walls are selected to ensure that overturning and sliding due to the back fill pressure is overcomed by the resistance to overturning and sliding developed due to the self weight. A sufficiently high factor of safety is employed. In the embedded retaining structures the depth of embedment required to overcome the disturbing effects of the retained soil was computed having used a sufficiently high factor of safety on passive resistance mobilization in front of the wall.

Internal stability of the structure is ensured by keeping the direct and shear stresses in the wall section within permissible limits. There are empirical formulae for allowable normal and shear stresses in Gabion walls depending on the properties of the cage.

Numerous experimental studies and finite element studies have revealed the displacement required for the mobilization of limiting pressure conditions around the retaining wall. Results of a study by Potts and Fourie (1986) is presented in Figure 12. Potts and Fourie (1986) showed that the initial earth pressure coefficient at rest k_o did not influence the limiting earth pressure. The movement necessary for mobilisation of fully active condition is much smaller than that required for the mobilization of fully passive condition. The movements required depended on k_o values. Numerical simulation of placement of fill behind a gravity retaining wall by Goh (1984) illustrated that due to the increased foundation stiffness outward movement of wall was inhibited and earth pressures remained much closer to at rest values. (Figure 13) Similar results were obtained by many other researchers.

The findings about the dependency of the stresses mobilized on the displacements were incorporated in the New British Code for Earth Retaining Structures Design BS 8002 (1994). It is based on the limit state concept and define "a ultimate limit state" that corresponds to a state of failure and "a serviceability limit state" that corresponds to a state of excessive deformation. For the retaining wall to function satisfactorily deformations should be within permissible limits. i.e.

It should not have reached a serviceable limit state. With the deformation allowed within a serviceable limit state Peak shear strength parameters of the soil will not mobilize. Therefore BS 8002 suggest to use a mobilization factor M and to reduce the shear strength parameters c, and ϕ by $c_{design} = c/M$ and tan $\phi_{design} = \frac{\tan\phi}{M}$. Thereafter stability is assessed through an equilibrium computation without the use of any more safety factors. However BS 8002 advises to do the design for the most adverse load condition that could exists and recommends the use of a minimum obligatory surcharge of 10 kN/m^2 in the retained side and an unplanned depth of excavation (10% of retained height or 0.5m which ever is greater) in front of an embedded wall. A mobilization factor of M = 1.2 was used on effective stress parameters and a value of 1.5 was recommended on total stress parameters.

Embedded retaining walls are often associated with different water levels on the two sides of the wall perhaps due to dewatering inside. CIRIA Report 104 (1984) and British Code BS 8002 suggest a simplified method to account for the seepage underneath the wall in computation of pore water pressures. (Figure 14)

BS 8002 does not use partial safety factors as in other codes under limit state concept. It argues that the characteristic strength and partial safety factors do not go along with mobilization of strengths in soils and employs the mobilisation factor.

Other recent codes such as Hong Kong Geoguide and Eurocode 7 uses partial safety factors on strength and loads. The magnitudes of these partial safety factors depend on the limit state under consideration.

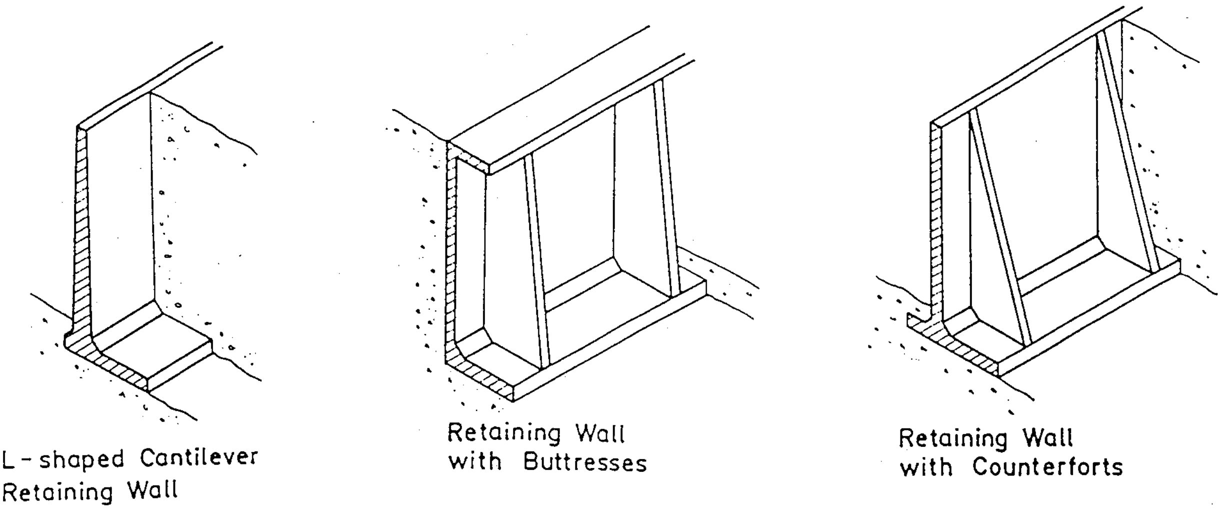
Use of Finite Element Technique

The conventional methods does not yield any information about likely displacements in the system. Finite element method is widely used for the computation of displacements around earth retaining structures. Verification of these computed deformations though sophisticated monitoring systems led to a wide understanding and enhancement in both computational and monitoring techniques.

Behaviour of the soil is generally modeled through an appropriate constitutive relationship such as non linear hyperbolic, elastic ideally plastic elastoviscoplastic etc. The relative displacement between the concrete or steel retaining wall and the soil is accounted for by deploying one dimensional interface elements at the boundary. Construction of the retaining wall and placement and compaction of the backfill is simulated by application of the corresponding forces at the appropriate nodal points. The deformation and the resultant stress changes in the system as a result of these construction activities are then computed.

Excavation in front of embedded walls can be simulated by applying appropriate upward vertical forces at the relevant nodes and reducing the stiffnesses of the removed soil. Methods are also now developed to numerically impose the reduction of lateral stresses (k_o values) that would occur during the excavation under Bentonite for diaphragm wall construction.

Comparison of data obtained by Kulathilaka (1990) via numerical simulation of the construction sequence of the Bell Common Tunnel (a cut and cover under ground road) and field instrumentation is presented in Figure 15. Displacements in the soil can also be graphically



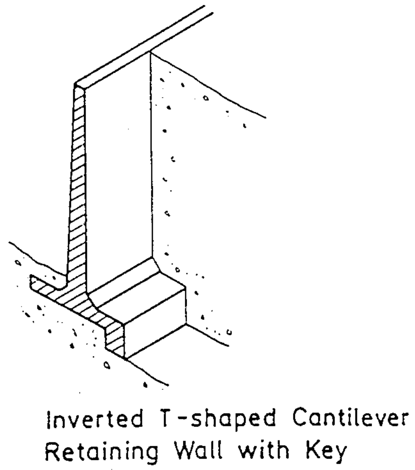


Figure 10 - Reinforced Concrete Retaining Walls

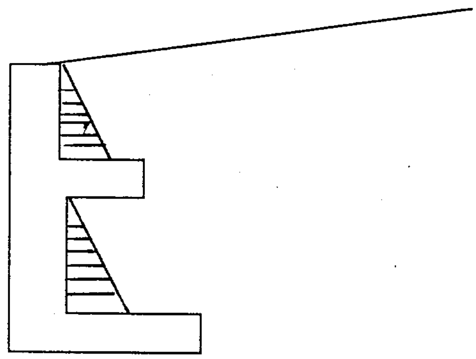
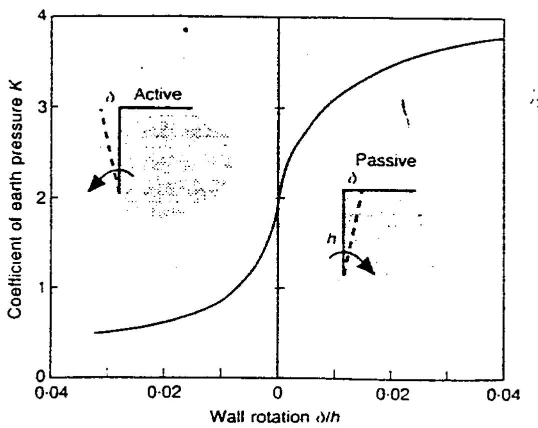


Figure 11 - Moment Relief Platform



Relationship between earth pressure and wall rotation computed by Potts & Fourie (1986) for over-consolidated clay

Figure 12 - Earth Pressure Variation with Displacement

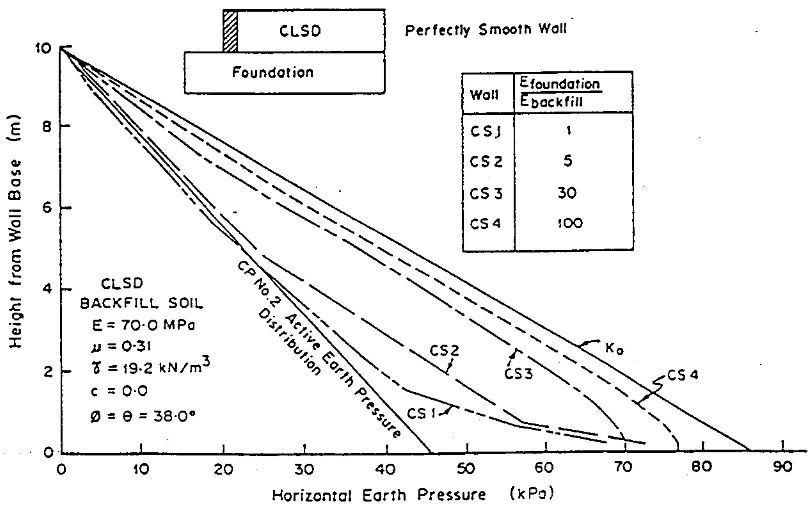


Figure 13 - Earth Pressure Behind Retaining Walls (Goh) (1984)

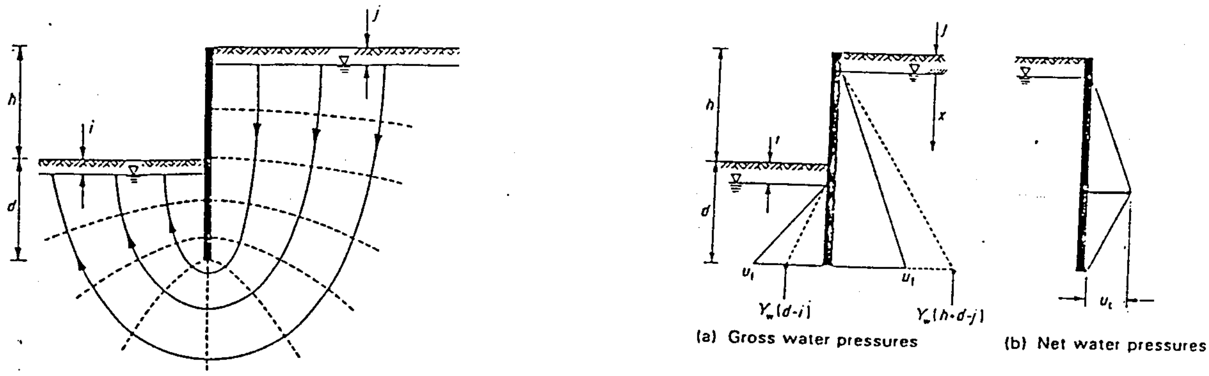
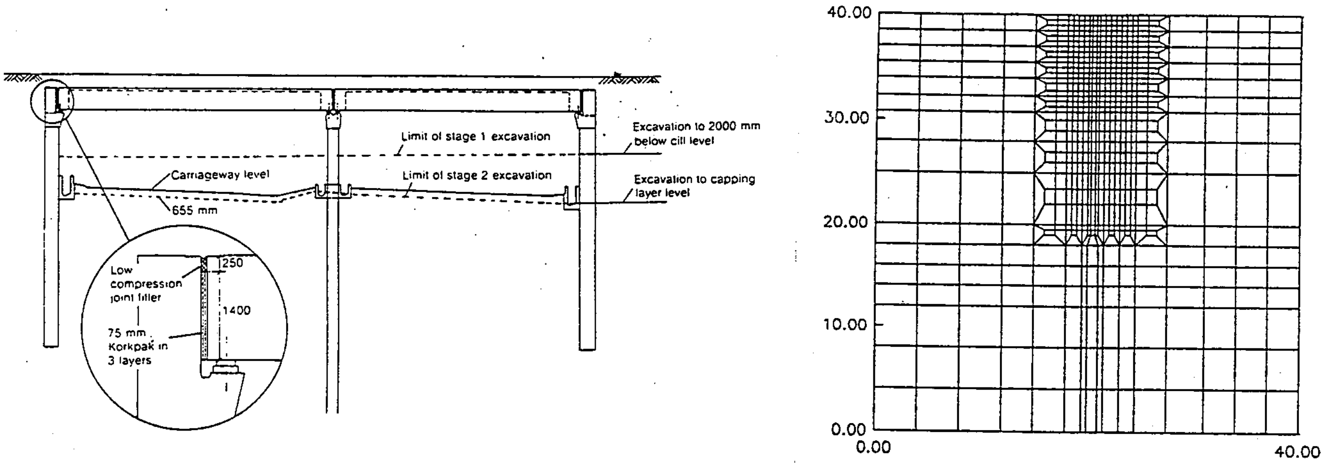
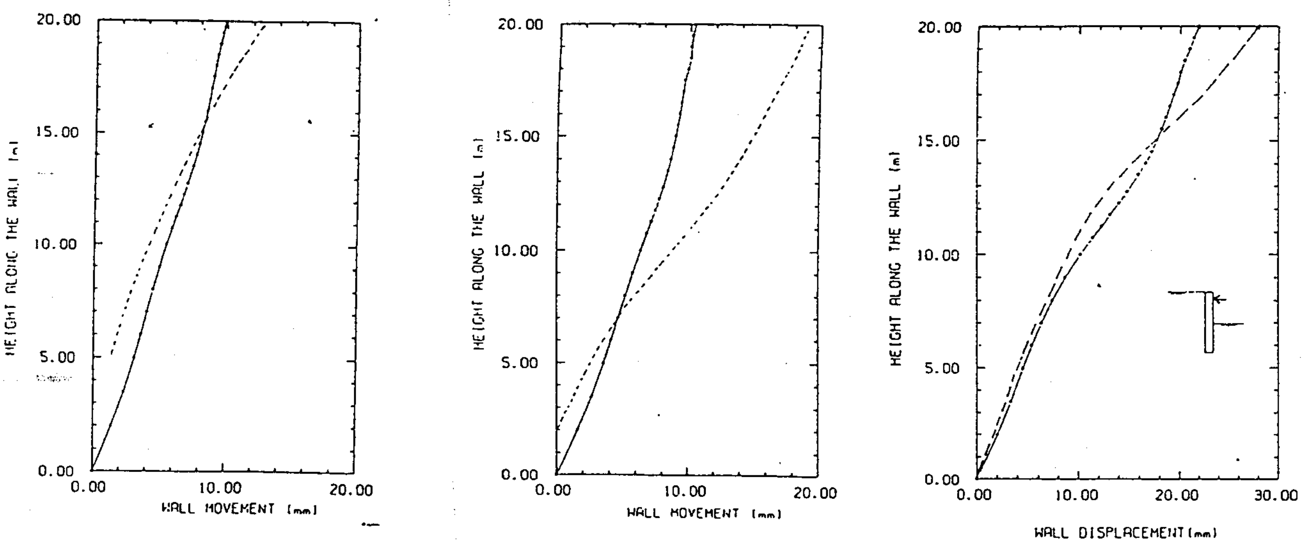


Figure 14 - Seepage Around Embedded Walls



Section through the Bell Common tunnel (after Hubbard et al,1984)



After excavation to 8.0m

End of construction

long term movements

Figure 15 - Bell Common Tunnel

presented in the form of displacement vectors. Results obtained through numerical simulation (Kulathilaka – 1990) of excavation in a c, ϕ soil is presented in Figure 16

Effect of compaction of fill behind retaining walls

Considerable research was done by Broms (1971), Transport and Road Research Laboratory (UK), (1977, 1980, 1989), Duncan and Seed (1986), Kulathilaka (1990) to study the influence of compaction behind retaining structures. Broms and Ingelson (1971) initiated this research following some fatal accidents associated with compaction of soil behind basement walls. Broms (1971) presented the first reasonable theory to model the compaction behind a retaining wall by taking the soil elements through an loading-unloading stress path

The resulting lateral stress distribution was trapezoidal and was significantly different from conventional triangular distributions. Brom's findings and further experimental evidence later clearly indicated that the compaction behind a retaining wall cannot be accounted for by increasing the k_o value (Figure 17).

Extensive experimental research was done at TRRL – UK in a large experimental setup compacting soil in layers according to the normal construction procedures in the fill. The setup shown in Figure 18 consist of a flexible metal wall and a rigid concrete wall. Three types of soils, a sandy soil, a silly clay and a highly plastic clay were used in three difference series of tests.

Study with the sandy fill indicated that lateral earth pressures much greater than k_o values will develop as a result of compaction. These values agree well qualitatively with the concept forwarded by Broms. However Broms prediction was for non yielding rigid walls and conditions in the flexible metal wall did not match with it. Finite element simulation of the procedure by Kulathilaka (1990), (Figure 19) where a loading–unloading-reloading compaction model was coupled with an elastic ideally plastic finite element simulation, resulted in close comparisons with the observed stresses and deformations. It was also found that a small, 2 mm (range) imposed outward movement is effective in bringing these compaction induced stress to very low values.

With the studies done with the silly clay it was found, large pressures measured as a result of pore water pressure development during compaction, reduced with time as the pore water pressure dissipated. Still the earth pressures were greater than that for a uncompacted fill.

Studies done with the highly plastic clay showed that if the soil is compacted at a high water content initially earth pressures are high due to developed pore water pressures and then they will reduce gradually with pore water pressure dissipation. However the compaction water contents are to be kept around the optimum water content to get good density and workability and soil will be partially saturated at that stage. When it is exposed to water (may be due to rain) subsequently, the highly plastic clay showed high affinity for water and swelled causing very large increases in the lateral earth pressures. (Figure 20). Therefore the use of highly plastic material behind retaining walls is not recommended and there are stringent criteria for selecting the fill material.

Loke (1994) presented the result of an experimental study where the use of a flexible foam type material had been effective in reducing the compaction induced lateral earth pressures on the wall.

Complicated state of stresses, that will develop with the simulation of compaction of fill behind retaining walls etc indicated the need of a new definition for the factor of safety. The use of

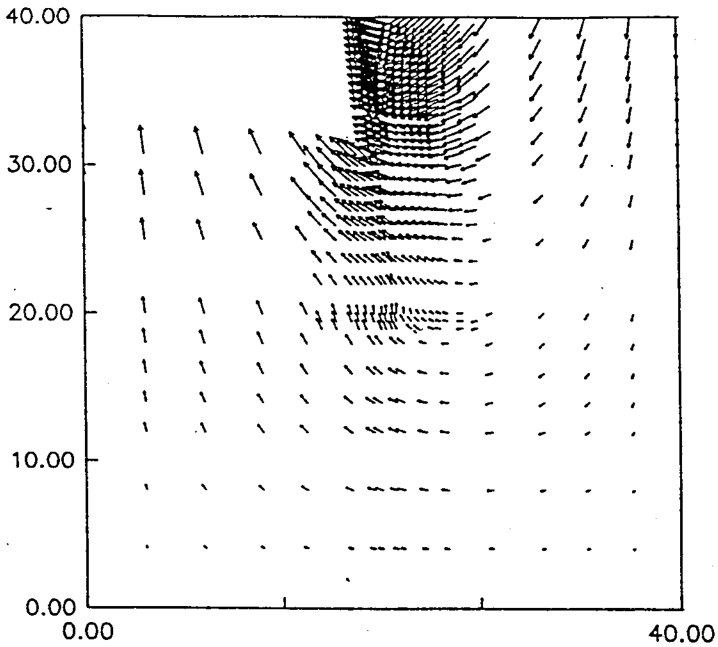


Figure 16 - Displacement Vectors

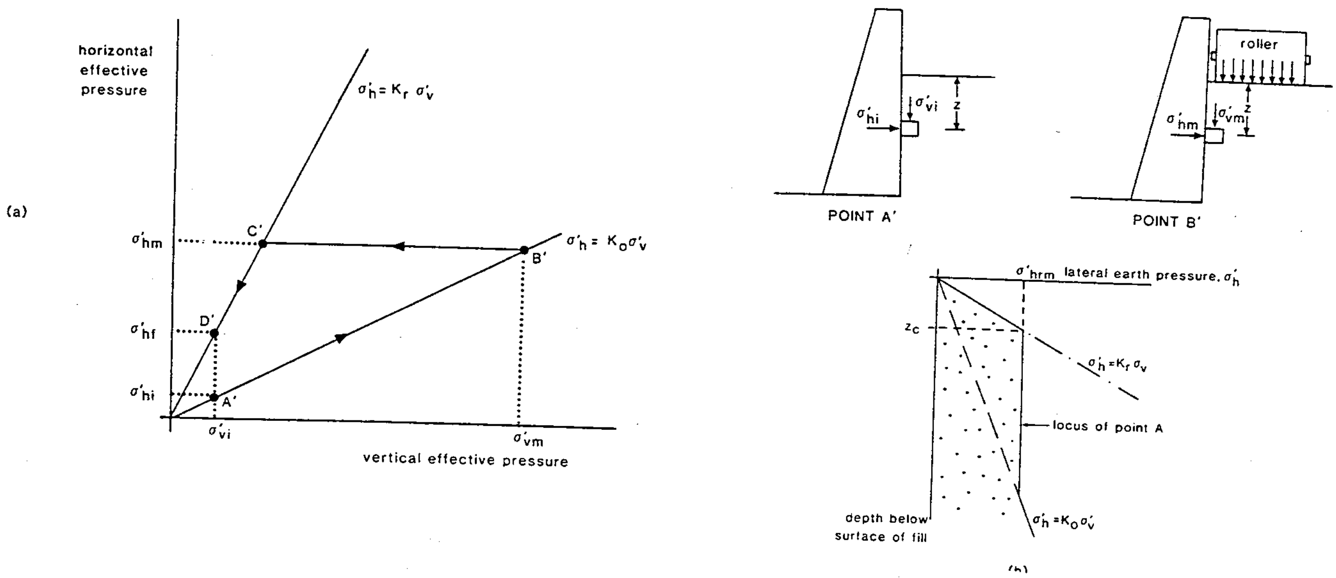


Figure 17 - Compaction Induced Lateral Earth Pressures

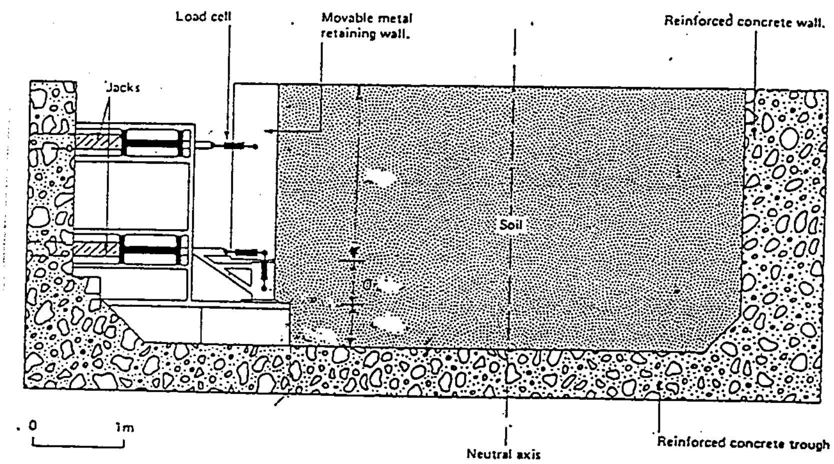
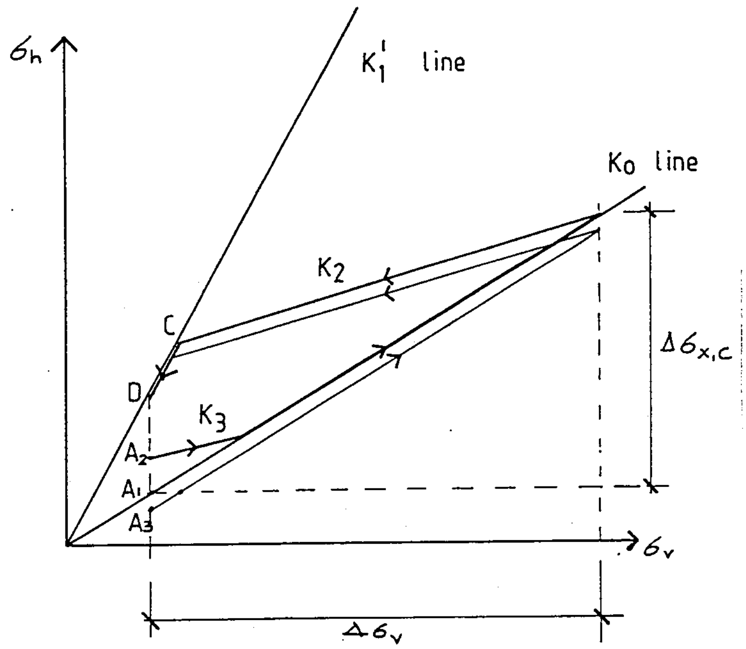


Figure 18 - TRRL Retaining Wall Set up



Compaction Simulation Model (Kulathilaka)

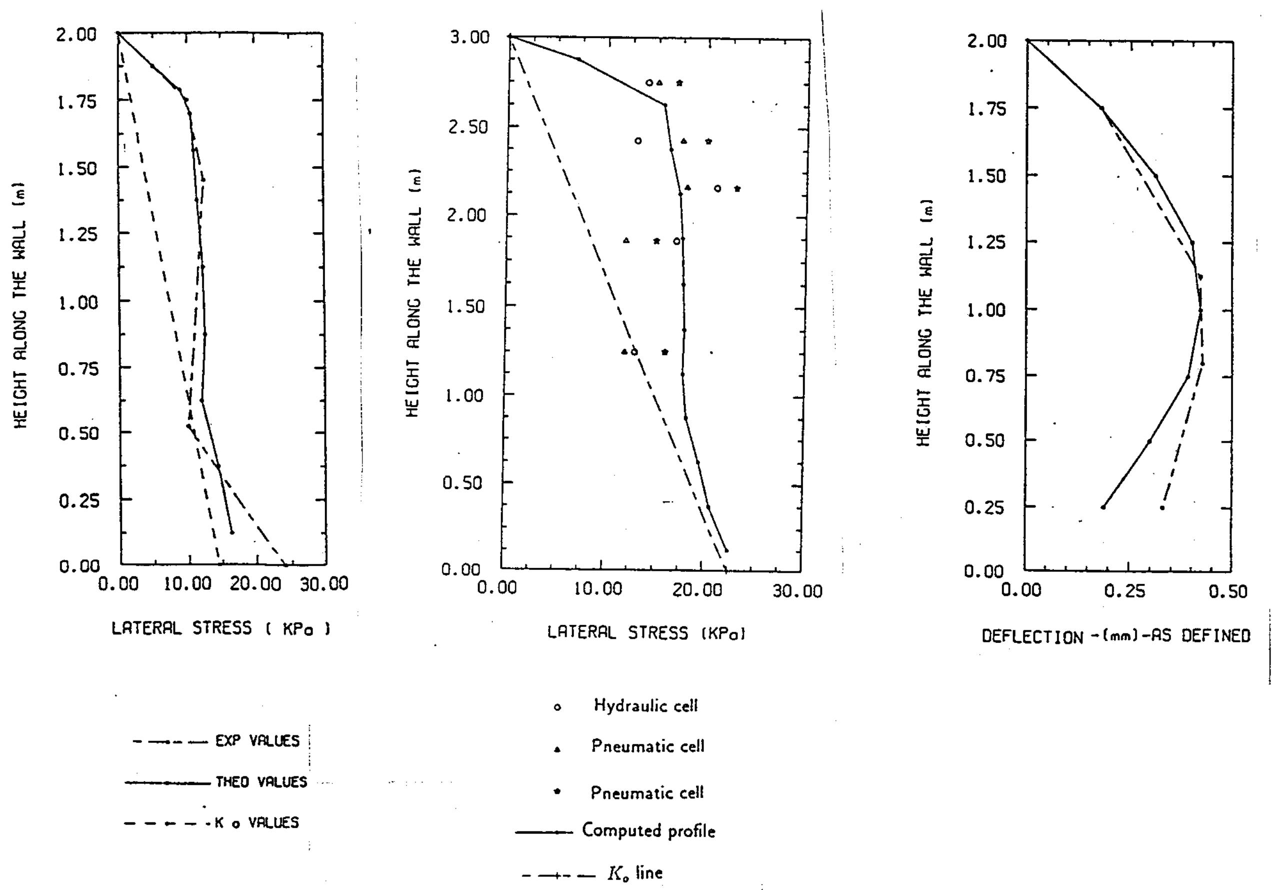


Figure 19 - TRRL Wall Comparisons

compaction induced stress profiles in a traditional factor of safety definition of sliding or overturning would indicate only the factor of safety on initiation of movement. With a little outward movement earth pressures would reduce. Hence a technique named as "Nodal Displacement Method" based on finite element analysis was developed Kulathilaka (1990).

This involves running the finite element model number of times with revised parameters and observing the deformations at a critical point (say toe of the retaining wall). The strength and stiffness parameters of the soil are reduced by a factor N and the deformation computed at the critical point in each analysis is plotted against N. This will indicate a gradual increase initially followed by a turning point and rapid increase of deformation with N thereafter. (Figure 22). The "Nodal Factor" corresponding to the turning point can be treated as an alternative to the conventional factor of safety. This method does not require assumption of any failure mode and considers stresses as well as deformations. Hence it would result in a more complete and comprehensive result. One major difficulty in applying it to day to day work is the excessive computational time/cost involved.

3. Internally Stabilized Earth Retaining Systems

3.1 Introduction

Internally stabilized earth retaining systems in use today can be separated into three basic categories depending on their major method of load transfer and method of construction.

The three groups are ;
(a). Reinforced Earth
(b). Anchored Earth and
(c). Soil Nailing

The basic components in these internally stabilized earth retaining system are facing elements, reinforcing elements and the soil. In the reinforced earth construction horizontal layers of reinforcements such as metallic strips, polymeric grids or geotextiles placed at designed vertical spacing are connected to a facing. Facing elements may be formed from metals, precast concrete, brickwork, gabions or geotextile. In anchored earth constructions facing elements of an appropriate form are connected by strips or ropes to an anchor element placed at a sufficient distance away from the potential failure zone. Both the reinforced earth and anchored earth constructions are carried out incrementally from bottom up.

Soil nailing is a situation where metallic bars or dowels are installed in an existing slope at regular vertical and horizontal intervals. A facing to integrate them is provided through shortcreeting.

Major advantages of internally stabilized earth retaining structures can be listed as

(a). They are flexible and can tolerate large differential settlements and lateral movements. Hence good foundation conditions or special preparation of founding soils is not essential.
(b). They can be constructed quite quickly and are operation as soon as constructed. Construction process is quite simple and does not require any special machinery.
(c). The cost saving can be up to 50%. The cost saving associated with rapid completion of the structure should also be added.
(d). There is no practical limit about height or width.

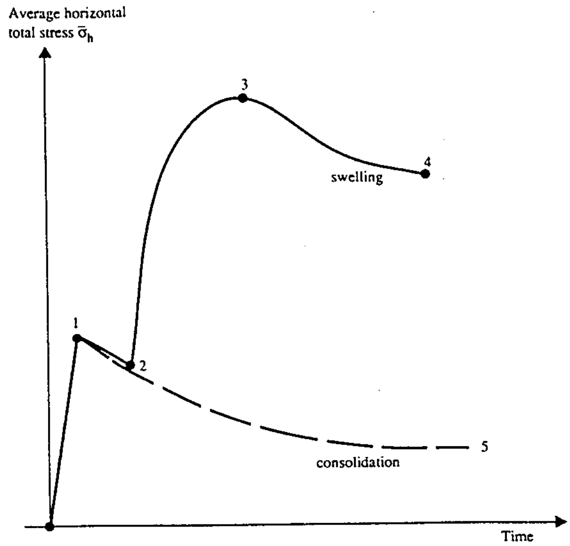


Figure 20 - Variation of Total Horizontal Pressure in Cohesive Backfill

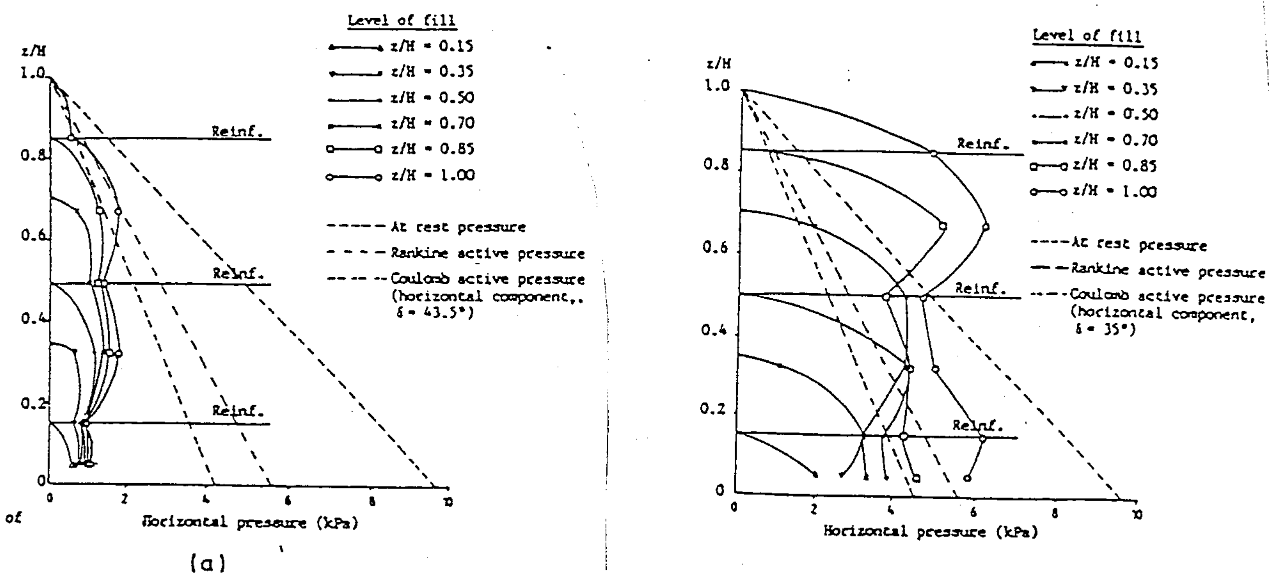


Figure 21 - Effect of Compressible Material at Wall Back

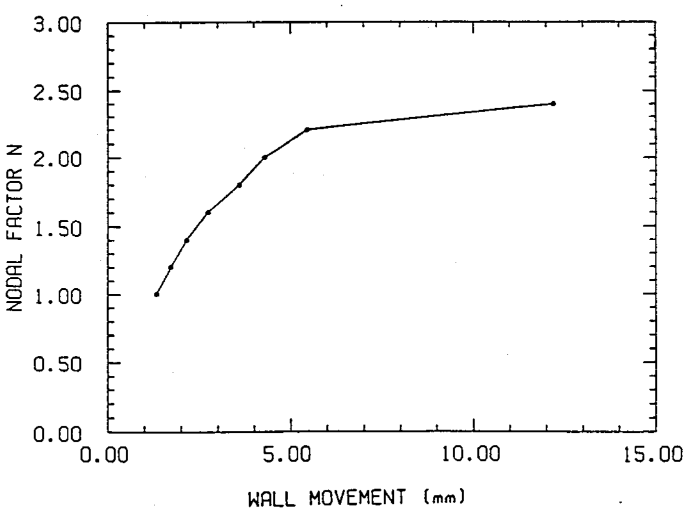


Figure 22 - Nodal Displacement Method

One of the largest applications of internally stabilized earth retaining system is in the construction of highway and bridge abutments. The speed of construction is a very special advantage specially in the case of repair of highways affected by landslides and in the case of widening of existing highway embankments.

The concept of internally stabilizing the earth structures is not new. There are historical records to indicate that this concept existed in the 5th and 4th Millennia BC. Great wall of china and Agar-Quaf structure are some examples. A significant contribution to the modern concept of reinforced soil structures were made by Munster (1925), Coyne (1927) (Figure 23). However the modern concept of reinforced earth came into wide usage after Vidal's introduction in 1960's.

Vidal's concept was for a composite material formed from flat reinforcing strips laid horizontally in a frictional soil. The interaction between the soil and the reinforcing members is due to interface friction. The material was described as "Reinforced Earth" and was patented. The first major retaining structure using Vidal's concept was built near Menton in the Southern France in 1968. The first structures used a surface cladding (facing) made up of horizontally laid U shaped sheet metal channel members. Plain metal reinforcing strips were connected to the facing elements. (Figure 24). In 1970's an alternate form of facing using a cruciform reinforced precast concrete members were introduced. (Figure 25). Concrete faced structure are widely used at present.

The concept of internally stabilizing the soil attracted many researches and research organisations over the last three decades. The understanding of the fundamental principles established by the early developers and users have been confirmed or further expanded through this research and number of innovative earth retention systems were developed.

Internally stabilized forms of earth retaining structures are very effectively used in active seismic zones. Many reinforced earth structures remained undamaged in the recent (1994) Kobe earthquake in Japan. In Turkey there is an example of a 3 tier reinforced soil wall road embankment retaining a height of 50 m from foundation to road in a active seismic zone.

3.2 Reinforced Earth Structures

After the Vidals introduction with plain metallic reinforcement strips and metallic or reinforced concrete facing element much progress was made and different forms of reinforced earth structures were developed. Reinforcements used in these soil structures should pocesses desirable qualities of adequate strength, high stiffness, low creep, good bond, and resistance to degradation.

Smooth steel (plain steel) strip reinforcements used in the earlier structures had low interface friction coefficients and thus the fill material used with them need to be of very good frictional properties. Subsequently the ribbed or deformed steel reinforcing strips with enhanced interface frictional qualities were developed. The steel reinforcements are subject to corrosion, the risk of corrosion being greater in silty or clayey soils. Protective treatment or alloying can reduce the rate of corrosion. Various forms of coating could also be helpful to some limited extent. A widely used design approach is to keep an allowance for the loss of cross section due to corrosion over the design life. Design is done based on the cross section expected to remain at the end of the design life. French Ministry of transport and British Department of transport have come up with sacrificial thicknesses for different combinations of metal types and fills. Darbin *et al* (1988) confirmed these values through study of 17 years old buried galvanized steel.

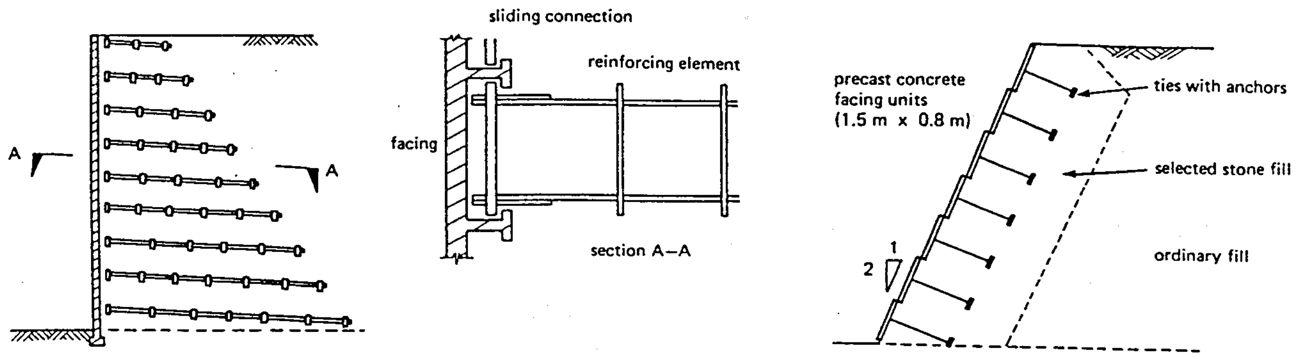


Figure 23 - Munster Wall & Coyne Wall

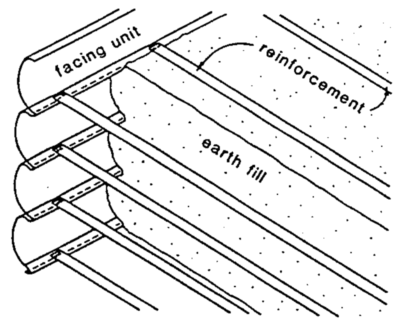


Figure 24 - Vidal Reinforced Earth Structure with Metal Facing

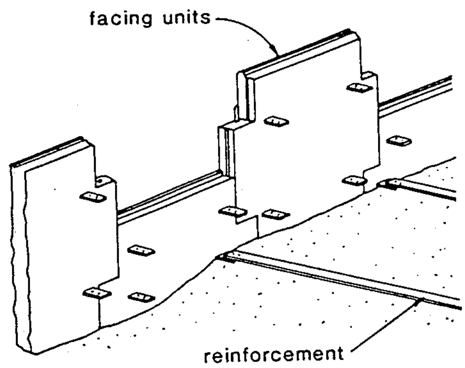


Figure 25 – Vidal Structures with R C Facing

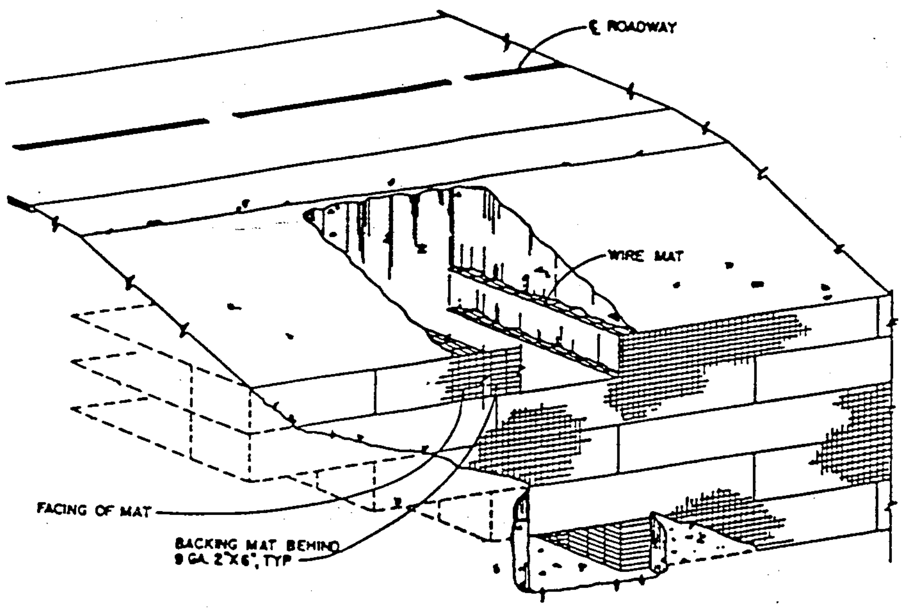


Figure 26 - Welded Wire Wall

Mesh and grid types of reinforcements made of metallic bars came in to reinforced earth structures from mid 1970's. Welded wire wall (1975), (Figure 26) VSL reinforced earth (1981) (Figure 27) are some examples. In addition to the interface frictional resistance developed between the longitudinal reinforcements and the fill, the passive resistance developing in front of the transverse members and the fill will be effective in these forms of reinforcements. Research studies have revealed that the contribution from the transverse members is much more significant (about 80 – 90%). With the introduction of these forms of reinforcing, the required frictional qualities of the fill could be relaxed.

More recently the polymeric reinforcements were also introduced into the reinforced earth structures. Differential forms of high strength polymer reinforcements of grid (geogrid, paragrid) and fabric (geotextile) forms are in use today. (Figure 28). Tensile strength of polymer reinforcement and deterioration when they are buried vary considerably. Mechanical damage during construction, exposure to UV radiation and action of chemicals in the soil found to be further critical reasons for deterioration.

High density polythene (HDPE) used in geogrids is highly resistant to more aggressive chemicals. Major problem in HDPE is high creep which may lead to outward movement of the retaining structure. Effects of long terms creep deformations are considered when the allowable tensile stresses in the polymer reinforcements are decided. When layers of geotexitiles are used as the reinforcing materials the facing can also be made from them by folding back. (Figure 29). Facing should be protected subsequently by use of bitumen, shotcrete or hydroseeding and vegetation to prevent exposure to UV radiation.

As discussed before facing of the reinforced earth systems can take a variety of forms. The tensile forces in the reinforcements are small at the facing and therefore the facings are not subjected to large loads. As such they can be relatively light. Their primary function is to stop erosion of the fill and ensure continuity while providing a suitable architectural finish.

After the construction of the reinforced earth wall, fill and subsoil could experience some settlement due to the self weight of the fill. If the reinforcements are rigidly connected to the facing and facing is unable to settle the differential settlement developed could increase stresses at the connection. This problem can be solved by keeping some packing between facing elements and removal of them subsequently. There are other construction details where the reinforcement layer is allowed to settle with fill at the connection through an appropriate sliding mechanism York Method (Fig. 30) is a good early application. Some further systems are depicted in Fig 31.

3.3 <u>Anchored Earth Retaining Systems</u>

Anchored earth retaining systems consist of interconnected facing elements and anchor elements (blocks) that are placed well beyond the potential failure zone. As in the case of reinforced earth structures they are constructed incrementally from bottom up without using any special equipment. They are also flexible and are operational soon after construction.

Soil in the failure zone tend to move outwards and exerts a force on the facing elements. As the facing elements are connected to the anchor elements the force is transferred to the anchor elements through the connecting strip or rod. Stability of the system depend on the pullout resistance of the anchor blocks and/or the tensile strength of the connecting wire/rod. The pullout

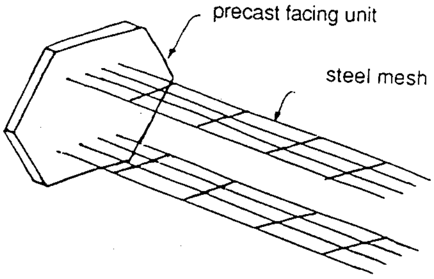


Figure 27 - VSL Reinforced Earth

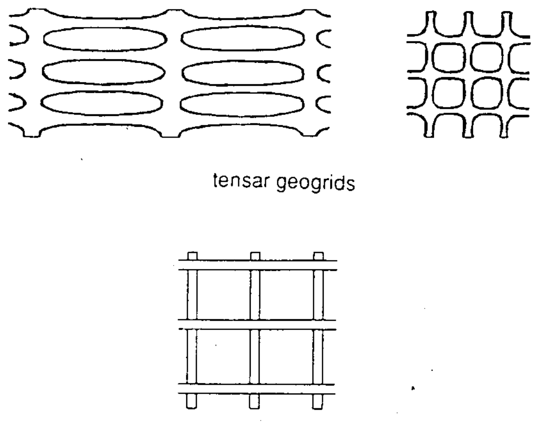


Figure 28 – Grid Reinforcements

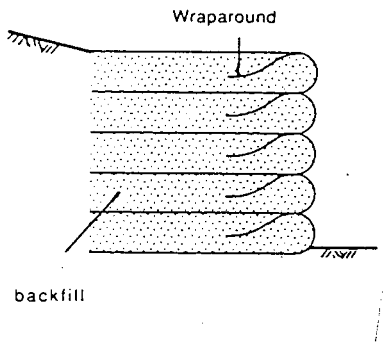


Figure 29 - Geotextile Reinforced Wall

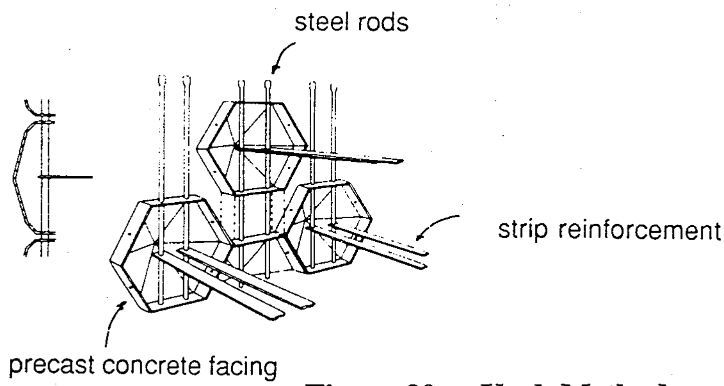


Figure 30 - York Method

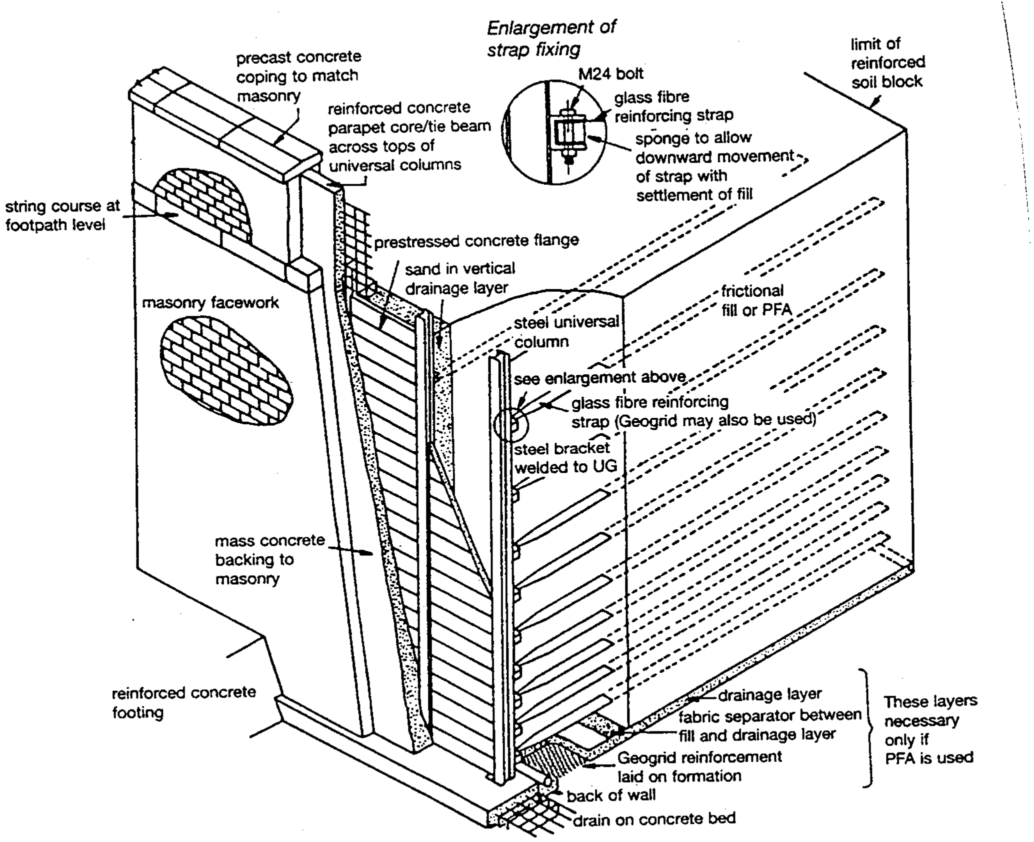


Figure 31 - Reinforced Wall Detail to accommodate fill settlement

resistance of an anchor block will depend on its shape, overburden stress and the shear strength parameters of the fill.

In reinforced earth (specially in strip reinforced structures) the resistance is developed primarily due to the interface friction between fill material and the strip. In anchored earth pullout resistance is developed mainly through the formation of a plastic failure mechanism in front of the anchor element. There is also some resistance developing via interface friction of the strip and top and bottom surfaces of the anchor. The pullout resistance (collapse load) and the mechanism will depend on the shape and size of the anchor and is much greater than that develop through interface friction. This also enables the use of lesser quality material as fill.

There is another advantage regarding the location of the anchor block. In reinforced earth constructions the reinforcing strip should be continued to a sufficient distance beyond the potential failure zone to develop the necessary pullout resistance. However with anchored earth constructions anchors can be placed just outside the failure zone leaving space for the formation of the plastic collapse mechanism. Thus the necessary width of the structure will be somewhat reduced. Also the cross sectional area of the connecting rod is not a controlling factor.

In 1980's techniques of using anchored earth form of structures evolved simultaneously in Europe, Japan, USA and Sri Lanka. Some of the systems in use today are;

(a). Loop anchored wall with concrete facing and concrete anchor blocks connected by polymeric ties in the form of a loop (Figure 32) – Brandl and Dalmatiner - Austria
(b). Multi anchored retaining wall using steel facing elements and steel anchor plates develop at Japanese Ministry of Construction (Figure 33) Okasa Kogyo -Japan
(c). Wall with concrete facing units and steel anchor bars bent to form triangular wedge shaped anchor developed at Transport and Road Research Laboratory - UK. (Figure 34).
(d). Anchored tyre retaining walls using discarded motor vehicle tyres for both anchors and facing elements – Developed in UK by Dalton and Hoban (1982) (Figure 35).
(e). Anchored tyre retaining system with discarded motor vehicle tyres as both anchor and facing elements – Developed by RC & DC – Sri Lanka – Sumanarathna (Figure 36).
(f). LIMI System in Finland (Figure 37)

3.4 <u>Soil Nailing</u>

Soil nailing is the other internally stabilized system of earth retention that started to evolve from early 1980's. This technique has been extensively used in stabilizing steep highway slopes.

In this technique in situ slopes are stabilized by reinforcing them by steel bars either driven (nailed) or placed in drilled holes and grouted into the slope. Driven nails are usually of diameters 12 – 25 mm with tensile strength around 100 – 200 kN. Nails used in drilled and grouted constructions are of diameters 12 – 40 mm and drill holes are of diameters 100 – 150 mm. Steel bars used are not subjected to any prestress and grouting is done under gravity. The method is suitable in forming steep (near vertical) cut slopes and stabilizing natural slopes showing signs of instability. In cut slopes nail installation can proceed in parallel with staged excavation. Number of nails are installed over the surface and are interconnected by providing a shotcrete surfacing after placing a wire mesh. Pre fabricated panels are also used as facings at present. The density of nailing is about 1 nail per 1.5m^2 to 8m^2 and the method is adaptable to site conditions. Staged construction permits the change of; inclination, density and dimensions of the reinforcement,

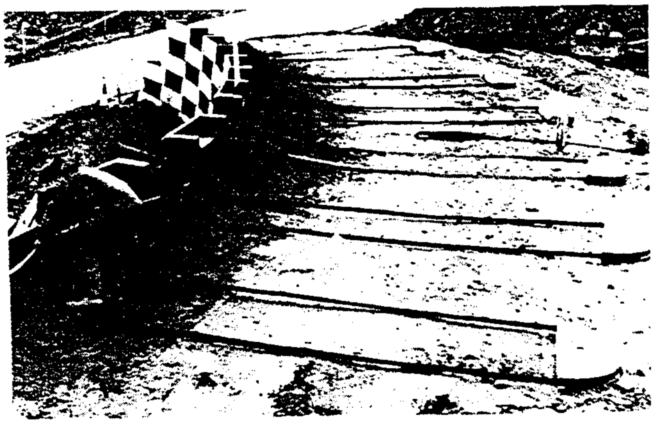


Figure 32 - Loop Anchored Wall

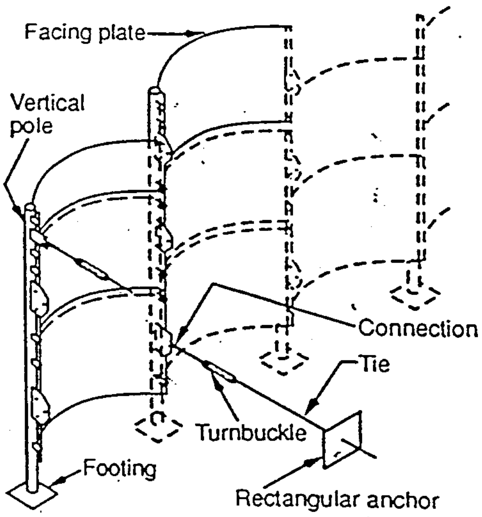
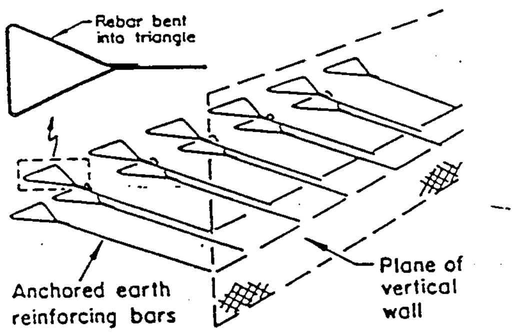


Figure 33 - Multi Anchored Wall



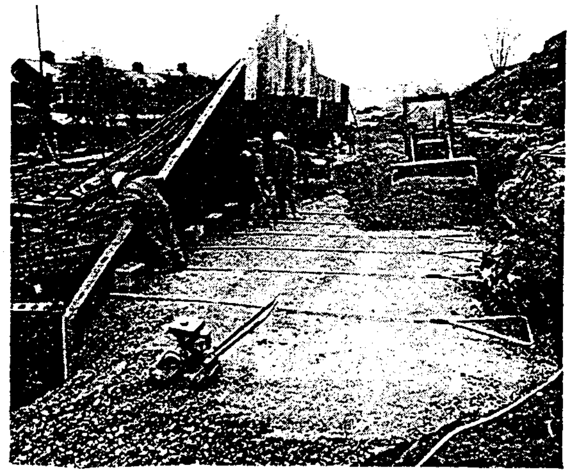


Figure 34 - TRRL Bent Reinforced Wall

depending on the site conditions and soil characteristics exposed at different levels during excavation.

The multiple levels of reinforcement installed in the soil mass should have gone pass the potential failure zone. The mechanism of stabilization is a combined form of interface shear, and shear and bending stiffness of the nails going across the failure surface.

In fine grained soils and in areas subjected to heavy rainfall it is advisable to ensure drainage of the embankment by installing inclined drains (by about 5 to 10 deg) at suitable vertical and horizontal spacing. Drains may be deep extending beyond reinforcement and made of slotted long tubes. Alternatively small steel or plastic pipes of 0.3 – 0.4 m length used to prevent accumulation of water behind facing. Soil nailing in excavation are still limited primarily to temporary structures, because of the difficulties associated with evaluating the corrosion rate of steel bars in the heterogeneous insitu ground and in producing low cost reinforcing elements with high resistance to corrosion. High adherence tube nails developed by Sorenfor is a significant development in this direction).

3.5 Hybrid Structures

Number of hybrid systems that utilizes characters of both externally stabilized and internally stabilized systems was developed within last decade. Tailed gabion are one such type and there are some applications of them in Sri Lanka. Tronderbock wall system is another form.

3.5 Methods of Analysis and Design of Internally Stabilized Systems

Soil elements in a zone closer to the free vertical face in an internally stabilized structure tend to move outwards. This was inhibited due to the interface friction mobilized between the soil and reinforcing elements placed at close intervals. This in turn will lead to the mobilization of a tensile force in the reinforcing element. The mobilization of shear strength (and therefore interface friction) and tensile force in the reinforcement with deformation is presented in Figure 39.

Behind an externally stabilized form of retaining structure the soil zone with outward moving trend can be well represented by Rankine type slip lines. They will get somewhat modified due to the roughness at the wall back. This zone is referred as the "active zone". Initially the active zone for a reinforced earth system was also thought to be similar making an angle of $45+\phi/2$ with the horizontal. However extensive research with model studies and instrumented real size structures indicated that the active zone in a reinforced earth system is more close to a log spiral shape as presented in Figure 40 (a). For the analysis purpose sometimes this is simplified into a trapezoidal zone (Figure 40 (b)).

For it to be stable reinforcements in the system should extend well beyond the active zone. The resistance to pullout is developed in the zone beyond the active zone and it is referred to as the "Resistant Zone". The tensile force mobilized in the reinforcement varies along the length. It starts with negligible values at the facing and reaches the maximum at the location of the log spiral failure surface. (Figure 41).

The reinforcements should be continued to a sufficient length in the "resistance zone" to develop the "demand pull out resistance". With strip like reinforcements resistance is developed solely from the interface friction. It is related to the friction angle of the soil and will depend on the

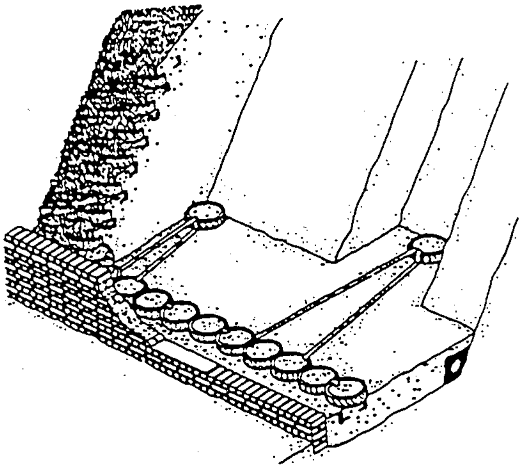


Figure 35 – Dalton Tyre Retaining Walls

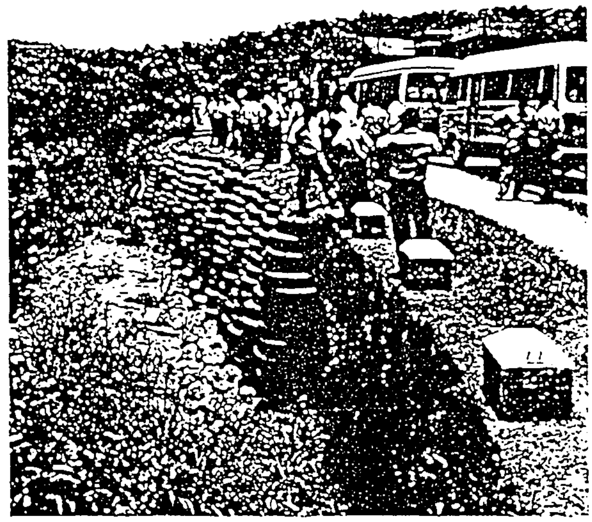


Figure 36 - RC DC Tyre Retaining Wall

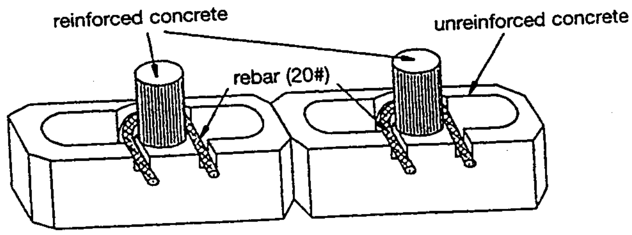


Figure 37 – LIMI System

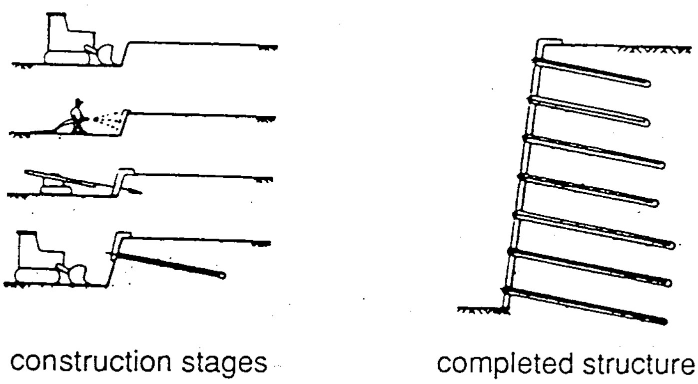


Figure 38 – Soil Nailing Examples

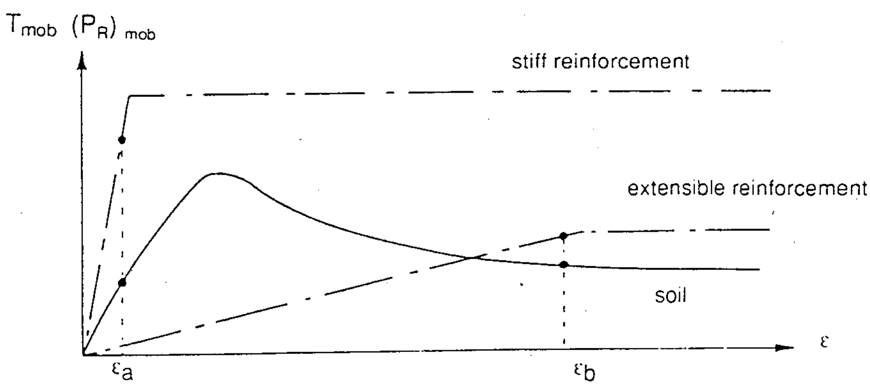


Figure 39 - Mobilization of Soil Shearing Strength and Tensile Strength

roughness of the strip. A coefficient μ is used giving the shear resistance as $\mu\sigma_v$. Alternatively μ is expressed as $\mu = f_b \tan\phi$ where ϕ is the friction angle of the soil and f_b is referred to as the bond coefficient. With grid and mesh reinforcements there are transverse elements and passive resistance developed in front of them will also contribute to the pull out resistance. Expressions are developed for them and usual practice is to provide an equivalent f_b value for them. Bond coefficient could be as low as 0.5 for strip reinforcements in clayey fill could be as high as 1.0 with polymer grid reinforcements.

Reinforcements are inserted at regular vertical and horizontal spacing requiring a given reinforcement to look after a zone of soil. In the case of grid or fabric reinforcements vertical spacing is the only variable. The tensile stress mobilized in the reinforcement depends on the vertical (and horizontal) spacing. In the design computations the spacing should be decided so that the tensile stresses developed in the reinforcements are within permissible limits. Reinforcements should be embedded to a sufficient length in the resistant zone to develop the required pull out resistance. Design of reinforced earth structures were initial done based on a limit equilibrium approach. More recently limit state concepts were introduced and the new British Code BS 8006 is based on that.

Limit Equilibrium Approach

In the limit equilibrium approach two forms of stability are considered. The two forms are
(a). External Stability and (b). Internal Stability

Under the external stability consideration the behaviour of the retaining structure as one unit (a gravity wall) is analysed. Overall width of the structure is evaluated so that the retaining structure can resist the overturning and lateral sliding caused by the backfill (with a sufficient safety margin). Usually the lateral sliding is critical.

Under the internal stability criteria necessary spacing (vertical and horizontal) and the length of the reinforcements to prevent tension failure and pullout failure is considered. Blanket or overall safety factors were used on both the tensile strength and the pullout resistance. Usually a factor of safety of 2 is used. Pullout resistance developing at both the upper and lower interface of the reinforcing material is considered.

Limit State Approach

As usual two limit states "ultimate limit state" and the "serviceability limit state" are considered. The limit states are defined in terms of "Limit modes of failure". Design computations should ensure that structure is stable against failure by the limit modes appropriate for the situation. In the case of vertical walls six limit modes are considered (BS 8006).

They are
- Limit Mode 1 - Sliding failure of the structure
- Limit Mode 2 - Bearing failure of the structure
- Limit Mode 3 - Tensile failure of the reinforcements
- Limit Mode 4 - Pullout failure of reinforcing elements
- Limit Mode 5 - Wedge/Slip circle instability
- Limit Mode 6 - Excessive deformations

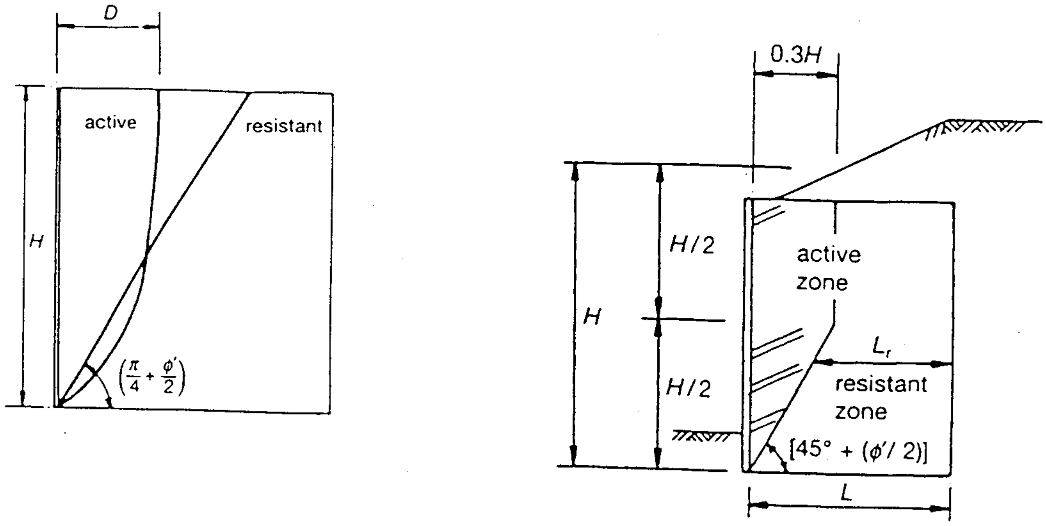


Figure 40 - Active and Resistant Zones

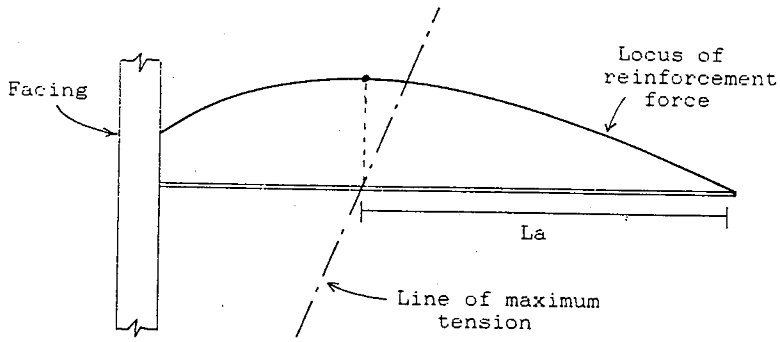
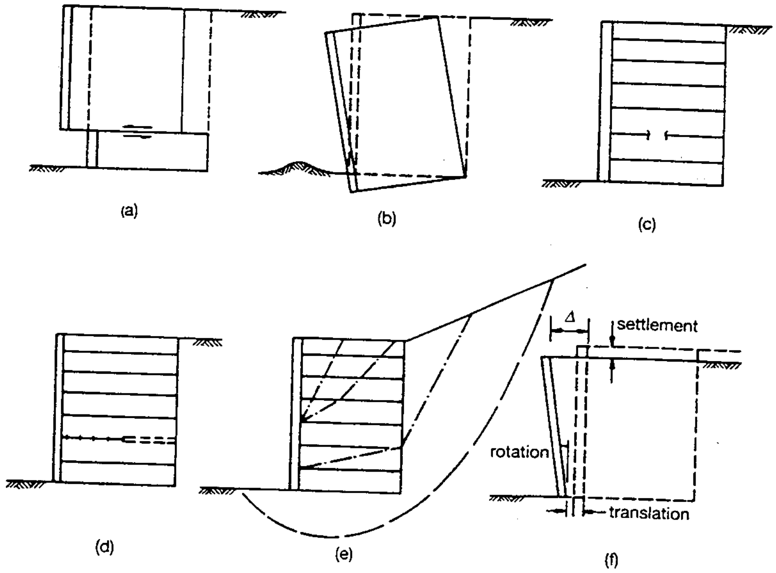


Figure 41 - Distribution of Maximum Tensile Force



Limit modes of failure of reinforced soil structures: (a) limit mode 1: sliding; (b) limit mode 2: bearing; (c) limit mode 3: element rupture; (d) limit mode 4: element pullout; (e) limit mode 5: wedge/slip circle stability; (f) limit mode 6: deformation

Figure 42 - Limit Modes of Failure

Except for the Limit Mode 6 all the other limit modes fall into the "Ultimate limit state" category. Partial factored are applied both on "loads" reinforcing material γ_m , soil material γ_{ms}, soil / reinforcement interaction forces (sliding and pullout) γ_s, γ_p, the foundation bearing capacity γ_{bc} and horizontal sliding on a soil/soil interface γ_{ss}. An additional factor of safety γ_{cc} related to the economic ramifications of the failure is also defined. Numerical value of the partial factor changed according to the criteria and whether it was a serviceable or ultimate limit state.

Design based on Limit Equilibrium Approach

In the proceeding discussion attention is paid to the design based on a limit equilibrium approach.

At the level of reinforcement considered, the vertical stress due to the self weight of fill, effect of backfill and any surcharge is evaluated. The tensile force the reinforcement is expected to carry is given by $T = k_A \ \sigma_v \ S_v \ S_H$. The ability of the reinforcement to withstand this without causing rupture or pullout is to be ensured. Inherited assumption in this is that the soil will move out sufficiently to mobilize full active conditions in the back fill. Hence this is suitable for more flexible forms of reinforcements. The failure zone is assumed be of a wedge shape and the adequacy of the total (summed up) pullout resistance to provide the force required for wedge equilibrium was checked after applying a necessary factors of safety. (Figure 43).

Coherent gravity hypothesis

The coherent gravity analysis relates to a reinforced earth structure constructed with a factor of safety and hence in a state of equilibrium. The observations made in model and real size studies, have indicated that the active zone is of log spiral shape. The design stresses in coherent gravity hypothesis relate to actual working stresses and not to failure stresses. (hence to a serviceability limit state). The lateral earth pressure coefficient k_A is taken to be in between k_o and k_A and depends on the depth. Also the coefficient μ used in computing pullout resistance is assumed to vary varied with the depth.

Mechanisms of Soil Nailing

The soil – reinforcement interaction in nailed soil structures can involve two fundamental mechanisms which are ;

- friction along the reinforcement and
- passive resistance against surfaces normal to the pullout direction.

Soil – reinforcement friction mobilization is found to be comparable with that observed with reinforced soil. The friction coefficient is found to depend on the type and density of the soil and also on whether the bars are driven or grouted in the pre bored holes. Although the tensile forces mobilizing in the reinforcements constitute the dominant reinforcing mechanism the passive lateral earth pressures are found to develop against the nail on either side of a potential failure surface if nails are rigid. (Figure 44). As illustrated in the figure a flexible reinforcement will deform until equilibrium is reached while rigid reinforcements resist the deformations. Consequently, passive lateral earth pressures will be mobilized at both sides of the potential sliding, surface, and shear stresses will develop on the cross section of the reinforcement to maintain the equilibrium. As such, depending on their orientation rigid reinforcements may have to withstand bending moments and shear forces along with tensile forces.

Jewell (1980) through an extensive test program showed that the maximum shear strength improvement in the soil can be achieved when bars are oriented in the same direction as the principal strain direction.

Also the laboratory model tests by Juran *et al* (1984) demonstrated that if the reinforcements are stiff the bending stiffness can have a significant effect on the failure mechanisms when the rotational kinematics of the active zone cause relatively large soil displacements perpendicular to the nails. The failure surface observed in the soil is similar for both types of reinforcements. In the case of flexible reinforcements potential rupture surface (maximum tensile stress location) for the reinforcement is the same as that of the soil whereas in the case of rigid reinforcements it is behind the soil failure surface.

Current Design Methods

Three design methods based on limit equilibrium analysis are in use today. They are

(1). Davis Method – Shen et al
(2). Gassler and Gudehus Method
(3). French Method (Schlosser et al)

First two methods consider only the tensile capacity and the last method accounts for shear and bending stiffness as well.

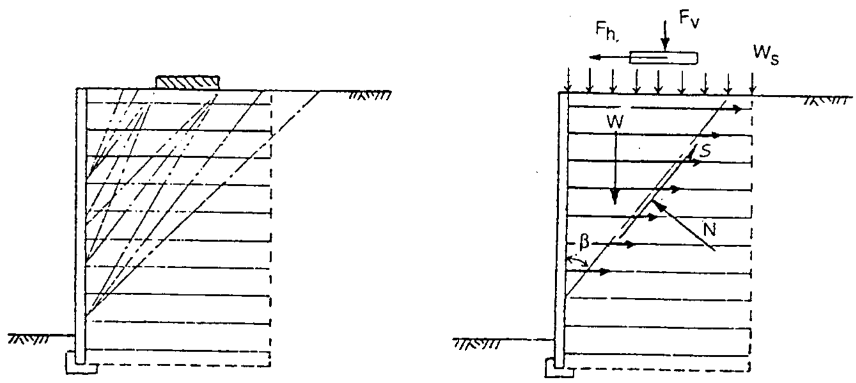
Most of the analysis used are based on the classical methods of slope stability analysis. An appropriate failure surface i.e wedge, circular or log spiral is assumed. The available shearing, tensile and pullout resistance of the reinforcements crossing the failure surface are considered in the analysis. The formulation of forces in the French Method is illustrated in Figure 45.

4. Concluding Comments

Many different forms of both externally and internally stabilized earth retention systems their mechanisms and basic design concepts were discussed.

The general trend was the development of more flexible systems that could be completed quickly and can accommodate considerable differential settlements. Considerable developments were taken place in the internally stabilized category developing new reinforcing materials and efficient systems of pullout resistance mobilization. An outline of application of different internally stabilized systems on various soil types is presented in Table 1. A comparison of cost for different forms of earth retaining structures provided by California Department of Transportation presented in presented in Figure 46.

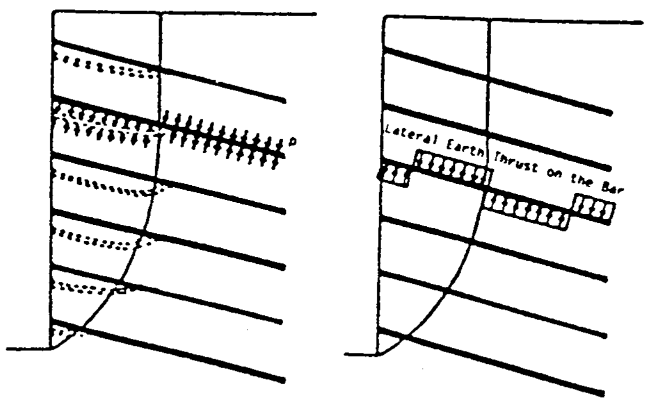
Advances were also made in the analysis and design of the earth retention systems. Numerical methods such as finite element method, boundary element method and discrete element method that can closely simulate the real conditions were used in the analysis of structures. These techniques also enabled the prediction of deformations. Instrumentation and monitoring systems were developed enabling accurate measurement of stresses and displacements. These were complimentary to the predictions made through advanced numerical analyses.



(a) trial wedges

(b) forces acting on typical wedge

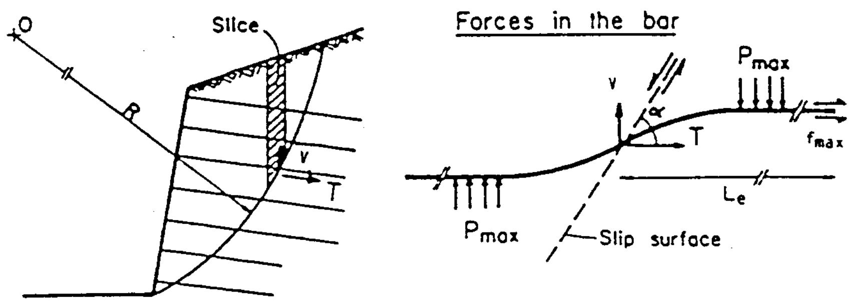
Figure 43 - Wedge Stability Analysis



A) Flexible Reinforcement

B) Rigid Reinforcement

Figure 44 - Effect of Rigidity of Reinforcement



- Failure criteria -

Shear resistance of the bar $T \leqslant R_n$, $V < R_c = R_n/2$

Soil bar friction $T \leqslant \pi D L_e f_{max}$

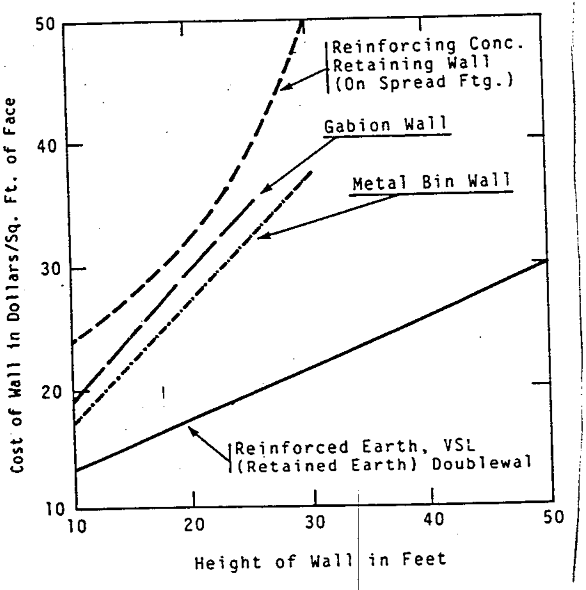
Normal lateral earth thrust on the bar $P \leqslant P_f$

Shear resistance of the soil $\tau < C + \sigma \tan\emptyset$

Figure 45 - Design of Nailed Soil Wall by - French Method

Table 1. Comparison of earth reinforcement systems. Note: Soil Type is based on stress transfer between soil reinforcement. Other criteria may preclude use of some soils for specific applications. (Adapted from Jewell, R.A. (1984), "Material Requirements for Geotextiles and Geogrids in Reinforced Slope Applications." *Proc.* 23rd International Man-Made Fibres Congress, Dornbirn, Austria)

	REINFORCEMENT TYPE		SOIL GEOMETRY Slope 30 60 Wall 90	SOIL TYPE Clay .002 Silt .02 Sand .2 Gravel 2mm	STRESS TRANSFER MECHANISM Surface Friction / Passive Resistance	REINFORCEMENT MATERIAL Metal	Non-Metal	PROPRIETY SYSTEMS/PRODUCT NAMES
IMPORTED EMBANKMENT TYPE APPLICATION	STRIP	Smooth	├──┤	├────┤	● (Surface Friction)	●		Reinforced Earth
							●	
		Ribbed	├──┤	├────┤	├──┤ (Passive Resistance)	●		Reinforced Earth
							●	Paraweb
	GRID		├───┤	├──────┤	●	●		VSL, MSE, GAS, RSE, and Welded Wire Wall
							●	Tensar Geogrids
	SHEET		├─┤	├─────┤	● (Surface Friction)		●	Geotextiles
	BENT ROD		├──┤	├────┤	● (Passive Resistance)	●		Anchored Earth
	FIBER		├───┤	├───┤	●	●	●	
IN SITU GROUND IMPROVEMENT APPLICATIONS	FLEXABLE, SMALL DIAMETER NAILS		◄───┤	IN SITU SOILS	● (Surface Friction)	●		
	RIGID, LARGE DIAMETER PILES		◄──┤	IN SITU SOILS	● (Passive Resistance)	●	●	



NOTES

(1) Costs shown are for wall materials and erection.

(2) Backfill and structural excavation costs are not included, except for Gabion and metal bin walls, where the costs include the cost of backfill placed inside the Gabion baskets and metal bins.

(3) For each individual project, backfill, structure and architectural treatment costs should be added to costs from the chart to make an overall cost comparison of wall types.

(4) Cost variations between RE, VSL Reinforced Earth, and Doublewal do not appear sufficient to justify separate cost curves for estimating purpose.

(5) Costs shown are intended for preliminary estimating and cost comparison purpose only.

(6) Costs shown are based on a combination of recent bid experience in FHWA Region 10.

(7) Chart was compiled by Ron Chassie, FHWA Region 10 Geotechnical Engineer, Portland, OR.

Figure 46. *Cost comparison of six wall types, 1981.* (Provided by California Department of Transportation)

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CONSTRUCTION OF REINFORCED EARTH RETAINING STRUCTURES ON RATNAPURA-WEWELWATTE ROAD IN SRI LANKA

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ABSTRACT
This paper describes the construction of reinforced earth retaining walls using treated bamboo strips as reinforcements and used tyres as facings, and also the combination of reinforced earth and anchored tyres for retaining structures. It also discusses the advantages of these methods of construction.

1 INTRODUCTION

In modern times reinforced earth structures developed in France is an invention of Henry Vidal and widely used in the world as an alternative earth retaining system. According to historical evidence, the principle has existed in countries like China in ancient times. The reinforced structure comprises of three principal components. A selected fill ideally a free draining well graded granular material strengthened by reinforcing elements and enclosed on the exposed faces by facing units/cladding. Strips of galvanised iron, aluminium or copper are used as reinforcements and at present polymer materials and geotextiles are also in use.

At the sites where these studies were carried out, valley sideslopes of the Ratnapura-Wewelwatte (R-W) road between culverts 19/6 & 19/8 and at 23/8 had collapsed in the late 1980^s due to stream cutting at the toe during the rainy season. In a normal situation in Sri Lanka, random rubble masonry walls would have been constructed at these sites in order to reconstruct and retain the road embankments. However at the first site, a reinforced earth retaining wall with treated bamboo strips as reinforcements and used tyres as facing elements, and at the second site a combination of reinforced earth and anchored tyre retaining wall were constructed on a trial basis in 1991 in order to try out the reinforced earth system and to determine whether treated bamboo strips can be used.

The technique of anchored tyre retaining wall was developed in about 1989 and over 40 structures were constructed using same. The technique involves the anchoring of a scrap tyre facing of a retaining wall to scrap tyres placed outside the active zone by tying the tyres together to form a combined system.

2 CONSTRUCTION OF REINFORCED EARTH RETAINING WALL BETWEEN CULVERTS 19/6 & 19/8 R - W ROAD

Figure 1 shows the cross section of the reinforced earth retaining wall.

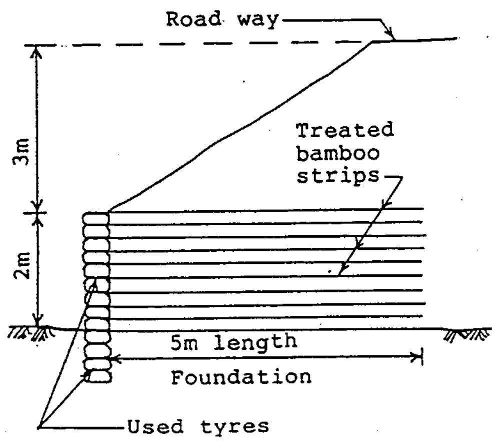

Figure 1. Reinforced earth retaining wall between 19/6 & 19/8 R-W Road

In this method, the bamboo strips were placed on a prepared foundation at intervals of about 0.3m extending from the boundary of the retaining structure towards the embankment to a length of about 5m. A set of scrap tyres were placed at the boundary of the retaining structure to stop erosion and connected at the bottom to the bamboo strips by using nylons strings of 6mm diameter. Clayey gravel with sand having a maximum dry density under standard conditions of compaction of 1600 Kg/m^3 was placed on the bamboo strips and compacted to a height of about 0.3m. This soil was also inserted fully inside the tyres and sufficiently compacted manually. Another similar set of bamboo strips were placed on the compacted soil and connected to the next set of tyres placed on top of the earlier set. This procedure was repeated to form a reinforced earth retaining structure to the required height.

A locally available most suitable material which is also inexpensive and readily available as reinforcing element in Sri Lanka is bamboo cut into strips. There are several varieties in many parts of the country which are quick growing and do not require much attention. Although there appears to be awareness of the potentialities of bamboo, very little published data is available.

Amaratunga (1970) has looked into the possibility of using bamboo strips as reinforcements in reinforced concrete in Sri Lanka and published the following data for common yellow (Dendrocalamus Strictus) and green giant bamboo (Dendrocalamus Gigan teus).

Tensile strength between nodes	90-140 MPa
Tensile strength at nodes	70-130 MPa
Tensile strength of strips	70 MPa

Using a factor of safety of 4, the permissible tensile stress of strips can be taken as 17.5 MPa.

In this retaining wall bamboo strips of about 50mm width x 4.6mm thickness were used. In order to preserve these bamboo strips, they were treated with copper sulphate solution of 3% by weight of water and keeping same immersed for minimum of 24 hours in a 5.5m long bath of the solution.

The construction of the retaining system was carried out by unskilled labour with the help of basic hand tools which are familiar to them. However to achieve the required degree of compaction of the soil layers of 100 percent maximum dry density under standard conditions of compaction for the top 450mm and 95 percent of this density below this level a plate vibrator or a vibrating pedestal roller was utilized. Although it is not theoretically required, distribution bamboo strips of the same cross section were tied to the structural reinforcing elements to form a mat. This method was adopted as a precautionary measure to avoid possible damages to main reinforcement element during compaction. The reinforcement mat was pegged to the compacted soil layer prior to placing of the subsequent layer of soil.

3 CONSTRUCTION OF COMBINED REINFORCED EARTH AND ANCHORED TYRE, RETAINING WALL AT CULVERT 23/8 R-W ROAD

At this site, a combination of reinforced earth and anchored tyre retaining wall of height varying from 2 to 3m were constructed to retain embankments of heights varying from 4 to 6m. The reinforced earth retaining wall was constructed as before. The construction of a typical anchored tyre retaining wall is described briefly in the ensuing paragraph. The details of this method will be given in a forthcoming paper (Sumanaratne, Mallawaratchie & Kulatileke, 1996).

The foundation of the anchored tyre retaining wall was taken to a suitable depth in stable ground. Scrap tyres of equal sizes were placed flat on the foundation of the retaining wall to interlock with each other and were tied together. Suitable soil was inserted fully inside the tyres and compacted sufficiently. The space between the tyres and the excavation was filled and compacted with suitable soil. This procedure was repeated by placing tyres one on top of the other, filling and compacting with soil until the foundation is firmly formed.

Another row of tyres was then placed above the ground level, tied with each other to interlock. Then they were tied with nylon ropes of suitable diameter to every other tyre and these ropes were tied to anchor tyres placed in a stable zone (passive zone). The number of tyres tied to an anchor was limited to 4. Back filling up to the top of tyre was then done using suitable fill material. This procedure is repeated until the required height was reached.

4 THE RESULTS OF TYPICAL DESIGN CALCULATIONS FOR REINFORCED EARTH RETAINING WALLS WITH BAMBOO STRIPS AND TYPICAL VALUES OF PROPERTIES USED IN SUCH CALCULATIONS.

In carrying out the construction of reinforced earth retaining walls using bamboo strips, rule of thumb methods and personal judgements were used. However, subsequently design calculations carried out in order to determine the stability of the structure proved that the factor of safety is above the recommended standard values. This is further confirmed by the present stable and intact condition of the structures after a period of about 5 years of their construction. In doing these calculations, the following values for the properties of soil and bamboo were assumed :-

Angle of internal friction of soil= 30^{0}
Cohesion of soil = 0^{0}
Bulk density of soil = $20KN/m^{3}$
Co-efficient of friction between bamboo strips and soil = 0.577
Tensile strength of bamboo strips = 17.5MPa

5 ADVANTAGES OF REINFORCED EARTH RETAINING WALLS

The major advantages of the use of reinforced earth as retaining structures are :-

1. It is very flexible. Large vertical settlements and lateral movements can be accommodated and good foundation conditions are not essential.
2. Retaining structure can be made simultaneously with the fill.
3. Construction process is easy and quick and does not require specialised machinery or labour. Only standard equipment and simple construction techniques combined with locally available labour are required.
4. The use of less cohesive fill material ensures that proper drainage exists.
5. Total cost can be 40% to 50% less than that of the conventional concrete walls, random rubble masonry and gabion walls etc.
6. Apparently there is no practical limit for height or width.
7. The wall will match with the surroundings as it gets covered with vegetation after sometime.
8. Reduction in construction time.

6. CONCLUDING REMARKS

The two structures described in this paper have been successfully constructed using locally available materials like bamboo strips and scrap tyres. These methods are labour intensive with many advantages enumerated in this paper. Design calculations carried out after construction indicate that the structures are safe with the assumption of values for properties of soil and bamboo strips. These methods could be adopted ideally in countries like Sri Lanka where similar conditions prevail and advanced technology and capital intensive equipment are lacking.

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Stabilization of slopes by anchored type retaining structures

Stabilisation de talus par des structures de soutènement du type ancrage

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Abstract

Anchored earth retaining systems constructed using discarded motor vehicle tyres are used in number of locations in the Sri Lankan highway network for restoration of failed slopes and embankments and for widening of roads

These structures are found to be very cost effective making savings of more than 60% from alternate conventional structures. They could be rapidly constructed and have a very high safety margin. Labour intensive construction process does not demand specialised machinery or skilled labour. As such this system is ideally suitable for a developing country like Sri Lanka.

1. Introduction

Number of Earth retaining structures were constructed at around 40 locations in the Highway network of Sri Lanka using discarded motor vehicle tyres. Usually the larger tyres used by lorries, motor coaches and trucks were incorporated. These structures were well distributed in various parts of the country. Large majority of them exist in the central districts of the country where hilly terrain prevails and frequent slope failures were observed. In most instances these structures were deployed to rehabilitate failed slopes and embankments. They were also used for the purpose of widening existing roads.

The concept of Anchored Tyre Earth Retaining Structures was developed by the first author and all the constructions were done by Road Construction and Development Company.

Tyre retaining walls could be erected very rapidly without the use of specialised equipment. Method of construction is generally labour intensive. The cost of the Tyre retaining structures were found to be less than 40% of alternate conventional gravity type structure.

All the structures constructed so far perform satisfactorily without a single record of failure.

2. Working Principle of Tyre Retaining Structures

Tyre retaining structures are essentially a type of anchored earth retaining systems. Therefore they will be referred to here as "Anchored Tyre Earth Retaining Structures". Thus it can be classified as an internally stabilised earth retaining system.

Both the facing elements and the anchor elements of the structure are made of discarded motor vehicle tyres. Several facing tyres are connected to an anchor tyre kept at a sufficient distance behind the facing.

When the soil embankment attempts to move outward it has to take the facing tyres with it. As the facing tyres are connected to the anchor tyres this will generate a tensile force in the connecting rope. If the ropes are strong enough so that they will not fail in tension and if the pullout resistance of the anchor tyre is greater than the developed tensile force, the structure will remain stable.

3. Construction of Anchored Tyre Retaining Structures

3.1 Foundation Conditions

Anchored tyre earth retaining structures are highly flexible and can accomadate considerable differential settlemnts . As such they do not require very sound foundation conditions. They can be erected on generally stable ground . If necessary natural ground may be compacted by several roller passes. If the structure is constructed adjacent to a stream or waterway sufficient precautions must be taken to ensure that no erosion will take place below or behind the facing tyres.

3.2 Placement of Tyres and Compaction of Fill

Placement of Tyres

Facing tyres are placed on the founding soil along the desired alignment. Each tyre is connected to its neighbour with nylon ropes. Ropes of diameter 8 - 10 mm are normally used. Alternate facing tyres are then tied to anchor tyres kept at a sufficient distance away from the facing. (Plate 1). Maximum of four facing tyres are connected to an anchor tyre. Figure 1 depicts a plan view of the arrangement of facing and anchor tyres. Facing tyres that are not connected to the anchor tyres are kept slighly behind the connected ones to have an interlocking effect.

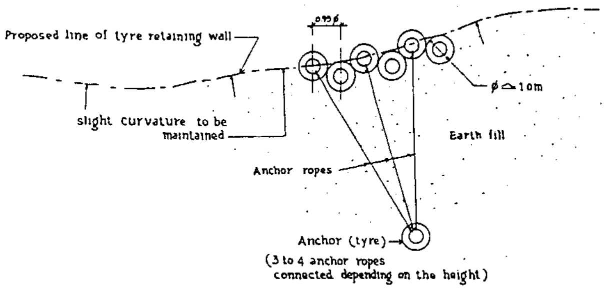


Figure 1 - Plan view of Anchor and Facing Tyre Arrangement

Plate 1 - Placement of Anchor and Facing tyres

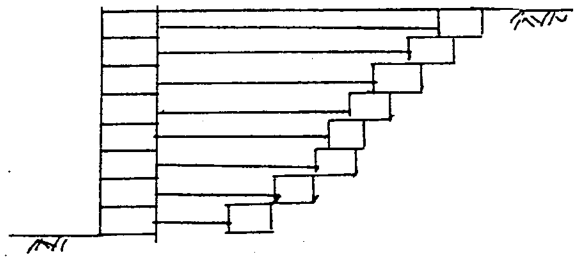


Figure 2 - Anchored Tyre Structure - Cross section

Plate 2 - Compaction of Fill Inside the Tube Space

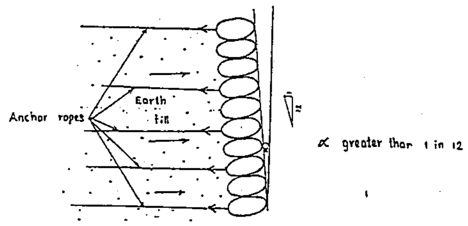


Figure 3 - Batter of the Facing Tyres

Inclination of the critical failure surface to the horizontal (θ) is greater than 45 deg for conventional gravity type structures. Hence the anchor tyres were placed at a location behind a 45 deg line drawn from the base of the wall. At a height H from the base therefore the anchor tyres should be at a distance greater than H from the facing. In practice at a height of H m the anchor tyres were placed at a distance equal to or greater than (H+1.0) m.

Compaction of Fill

Reddish brown lateretic fill material widely used in earth constructions in the country are used as the fill material. Soil with a Plasticity Index lower than 15 and a Proctor dry density greater than 1600 kg/m^3 are recommended for use in these structures.

After placement of a layer of facing and anchor tyres the tube space inside the tyre is filled with lateretic fill and well compacted manually using hand rammers. (Plate 2). If the structure is facing a stream or waterway tube space in the facing tyres are filled with a well graded gravely material.

Anchor tyre is pulled back to ensure that the ropes are well stretched and then the fill is properly placed. Fill is then compacted using rollers or rammers to a dry density above 95% of the standard Proctor dry density. Heavy compaction equipment were not brought very close to the facing tyres. In the close vicinity of the face hand held rammers or vibrating plates are used. Once the compaction of a layer is completed new layer of anchor and facing tyres were placed and the procedure is repeated till the desired wall height is achieved. As the level of the fill increases the necessary spacing between the facing and anchor tyres must be maintained. Although it is not essential, a batter of 1:12 is maintained in the structures. (Figure 3).

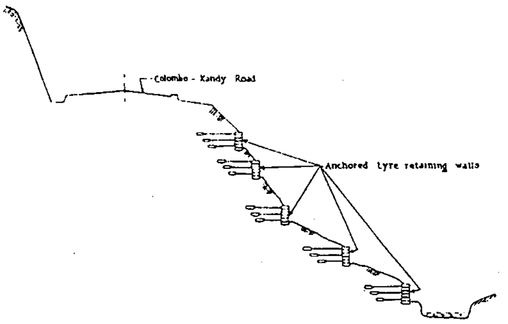
3.3 Multi tier Structures (Cascade Structures)

Although there is no limit to the height of the retaining structure that could be built by this method the height of a single structure constructed so far was limited to about 5.5 m. Where it is necessary to have a higher embankment, construction was done in several tiers. At Utuwankanda in the Colombo - Kandy road it was a three tiered structure.

4. List of Constructed Anchor Tyre Retaining Structures

Out of the more than fourty structures constructed to date details of 10 major structures are presented in Table 1. Table provides information such as,locations, length and height, construction period, date of completion, reason for construction, cost of the anchored tyre structure and estimated cost of an alternate conventional structure of random rubble masonry or mass concrete. In all these constructions tyres used were of diameter 1.0 m and nylon ropes used were of diameters 8 mm and 10 mm. Compaction was done with rollers and rammers.

Most of the constructions were done by the Rathnapura and Kegalla district branches of the Road Construction and Development Company in association with relevant district officers of Road Development Authority.



CROSS SECTION OF THE ANCHORED TYRE RETAINING WALLS ON 86th Km COLOMBO-KANDY ROAD

Figure 4 - Multi - tier (Cascade Type) Stucture

Plate 3 - Completed Anchored Tyre Stucture at Ranwala

Plate 3 provides a view of the completed structure at Ranwala in the Colombo - Kandy road and Plate 4 presents a view of the cascade type structure at Utuwankanda in the Colombo - Kandy road.

5. Factor of Safety Expression

As outlined already in section 2 of the paper the outward movement of the fill retained will generate a tensile force in the rope connecting facing tyres to the anchor tyres. Failure of a anchored tyre structure can happen either by;

- Failure of connecting rope in tension or
- Pulling out of the anchor tyre

Pull out resistance of an anchor tyre depends on the overburden stress at the level of the anchor and shear strength parameters of the compacted fill. Different pullout failure modes can be assumed for the derivation of pullout resistance. If the structure consist of n level of tyres, the pull out resistance available at each level can be computed and the

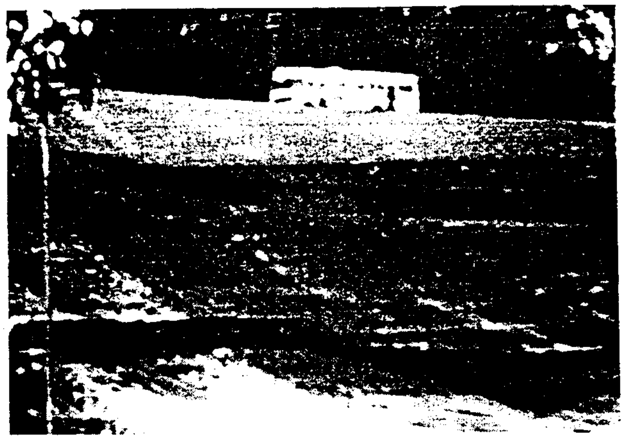


Plate 4 - Completed Anchored Tyre Structure at Utuwandanda

Total pullout resistance can be expressed as $T_{pullout}$, where $T_{pullout} = \Sigma T_{i,pullout}$. If the tensile strength of the wire if $T_{i,tensile}$ the capacity of anchor force due to tensile strength can be expressed as $T_{tensile} = \Sigma T_{i,tensile}$. The ultimate anchor force that can be provided by system is the lower value of the $T_{pullout}$ and $T_{tensile}$.

For Simplicity by assuming a wedge shaped failure zone (Figure 5) the anchor force T that is required for equilibrium $T_{eq'm}$ can be obtained. This depends on the assumed wedge angle θ and the critical $T_{eq'm}$ can be found. The Factor of safety of the structure can be defined as;

$$FOS = \frac{Anchor\ Force\ Available}{Anchor\ Force\ Required\ for\ Equilibrium}$$

$$FOS = \frac{Lower\ value\ of\ T_{pullout}\ and\ T_{tensile}}{T_{eq'm}}$$

Factor of safety of the anchored tyre retaining structure at Ranwala in Kegalle district was found to be 20.0.

6. Model Studies on Anchored Tyre Structures

Anchored tyre retaining structures are found to be a very cost effective alternative to the conventional type structures. As there is a need to understand the behaviour of these structures more closely a program of model studies is now being conducted at the University of Moratuwa. A model retaining wall is constructed with model tyres building up the structure in layers closely simulating the field procedures. The model structure is then loaded vertically and deformations and failure modes of the structure were monitored. Studies are also being carried out about the pullout resistance of the anchor tyres. Results obtained to date are very encouraging and indicate that there is a very high safety margin with these structures.

7. Conclusions

Large number of retaining structures were constructed in the Sri Lankan road network using discarded motor vehicle tyres. They are based on the anchored earth principle and all structures perform satisfactorily to date without the record of a single failure.

This is a very cost effective alternative to the conventional

Table 1 - **Major Anchored Tyre Retaining Structures**

Details of Some Major Anchored Tyre Earth Retaining Structures							
Location	Length (ft)	Height (ft)	Duration of Construction	Date of Completion	Cost of the structure	Approximate Cost of alt. str.	Reason for construction and special features
CRW Road - 170 km	80	17	2 months	Apr. 1990	Rs. 400,000/=	Rs. 1,500,000/=	To restore failed embankment
CRW Road - 169 km	100	10	2 months	Aug 1993	Rs. 300,000/=	-	Conventional wall not possible
CRW Road - 163 km	60	25	4 months	Aug 1996	Rs. 600,000/=	Rs. 2,500,000/=	Gravel filter provided
CRW Road - 164 km	20	15	1 month	Aug 1996	Rs. 100,000/=	Rs. 300,000/=	Extension of bridge abutment
CRW Road - 145 Km	50	13	2 months	Dec 1992	Rs. 60,000/=	Rs. 150,000/=	Restore failed embankment
CRW Road - 100 km	40	15	1 month	Dec 1994	Rs. 120,000/=	Rs. 300,000/=	To widen the existing road
Kahawatta	90	12	1 month	June 1992	Rs. 150,000/=	Rs. 300,000/=	To widen the existing Road Gravel packing is used.
Kalawana	150	15	3 months	June 1994	Rs. 350,000/=	Rs.1,000,000/=	Fd'n not suitble for other walls.
Utuwandanda	150	60	9 months	Dec 1990	Rs. 2,400,000/=	Rs. 6,000,000/=	Restore slope - Cascade structure
Ranwala	80	18	4 months	Apr. 1991	Rs. 600,000/=	Rs. 2,000,000/=	Gravel packing used in tyres

Remarks :

CRW Road - Colombo Rathnapura Wellawaya Road

Utuwankanda and Ranwala are in the Colombo - Kandy Road

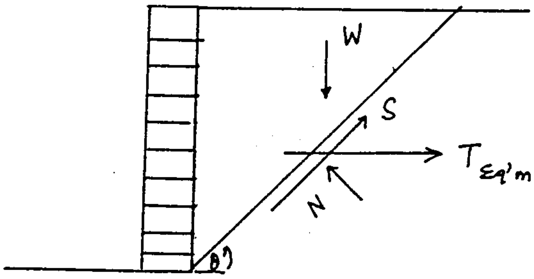


Figure 5 - Stability Analysis of Anchored Tyre Structures

type structures. As it is a highly flexible structure good foundation conditions are not essential. Also it is a permeable structure and water in the backfill will be drained easily. Method of construction does not demand any special machinery and is labour intensive. Hence it is ideally suitable for a developing country like Sri Lanka. These structure can be constructed very rapidly and are operational soon after the completion. Hence it is an ideal method for rehabilitating failed embankments. Once the structure is completed trees or grass that are indeginous to the respective locations were planted on the facing. This will make the structure blend harmoniously with the environment.

Nevertheless as the anchor tyres should be kept at a sufficient distance away from the facing, it should be possible to provide the necessary width. Construction activity such as removal of material that would be required to obtain the necessary width should not cause futher failures. That is the major limitation in this system.

8. Acknowledgements

The authors are indebted to Mr. M. B. S. Fernando, Chairman RDA/RC&DC and Mr. Densil Senanayaka former General Manager RDA for their inspiration direction and continued interest in the project. The whole hearted co-operation extended by Mr. S Mahendran Privincial Director is also acknowledged. The assistance rededered by the Engineers Mr. Dammika Karunasena, Mr. M. M. Thevarajah, and Mr. W. Wimalasiri in the construction is highly appreciated.

Model Studies on Anchored Type Earth Retaining Systems and Development of a Design Method

By Dr. S A S Kulathilaka – University of Moratuwa

1. Background

Anchored Earth Retaining systems constructed using discarded motor vehicle tyres were made at number of locations in the Sri Lankan Highway network by the Road Development and Construction Corporation Ltd., for the restoration of failed slopes and for the widening of road embankments. These structures could be rapidly constructed and are extremely cost effective and make savings of more than 60% when compared to alternate conventional gravity walls. More than 40 such structures constructed using larger tyres discarded by lorries, motor coaches and trucks are in successful operation since later 1980's.

These structures are made incrementally from bottom up. Tyres are used as both facing elements and anchor elements. Several facing tyres are tied up to an anchor kept at a sufficient distance away from the facing. Usually anchor tyres are kept outside the wedge shaped zone formed by drawing a 45° line from the toe. Apart from the above specification no design procedures were available to assess the stability of the structure.

This paper discribes the model studies that were conducted to understand the behaviour of these structures and the design method developed. Project was carried out at University of Moratuwa with the financial assistance from Road Development and Construction Corporation Ltd.

2. Preliminary Studies

Different forms of anchored earth retention systems developed in various countries are reported in literature. A system with bent reinforced bars as anchors and precast concrete elements as facings was developed by TRRL – UK (Murray and Irwing – 1981). Another system with concrete facing blocks and concrete anchor block connected by polymeric ties was developed in Austria (Brandl and Dalmatiner – 1986). Multi anchored retaining wall system developed by Japanese Ministry of Construction can also be classified as an anchored earth system. It was also found that a system using discarded motor vehicle tyres, very similar to the Sri Lankan System was developed in UK (Dalton 1982). They used a brick facing to cover up the tyres. (Figures of these structures are not presented in this paper as they were presented with the first paper in this Seminar). All the above reported systems were in a research stage and standard design procedures and guidelines were not available in literature.

A study on the behaviour of the anchored tyre earth retaining systems would require loading of the structure and measurement of deformations. The outward movements at the facing, the internal strain (and therefore stresses) in the wires and the outward movements of the anchors need to be measured. It will also be necessary to load the structure to failure to identify possible failure mechanisms. Preliminary computations indicated that massive loads would be required to induce collapse of a real size structure. Also the difficulties in attaching strain gauges to nylon wires to measure strains and likely experimental errors were recognized. Hence it was decided to carry out the study through model tests.

3. Model Studies

3.1 Preparation of the Model

A model tyre was casted from rubber to be used in the model. It was not a direct replica of a real tyre but a simple rubber ring reinforced with two steel wire rings. The out side diameter is 55 mm the hole is of diameter 35 mm and the tyre thickness is 15mm. These dimensions were scaled down from a real size tyre. In the construction of a anchored tyre wall in the field soil or rubble is filled inside the tube space and well compacted to form a stiff ring. Hence its modelling with a stiff rubber ring could be justified.

The model wall was formed inside a perspex box stiffned by slotted angle sections. The box had perspex only on three sides and the other side was left opened having stiffened by two lines of slotted angles.

Facing tyres were connected to each other by nylon wires and the same type of wires were used to connect the facing tyres to the anchor tyre. The basic unit used in the model structure consisted of three facing tyres connected to an anchor tyre as depicted in Figure 1. Adjacent facing tyre are connected to each other and the two outer ones are connected to the anchor tyre. The mode wall constructed was of total length 250 mm and two units were used for one layer of the structure. The structure was constructed incrementally from bottom up.

After laying the two units soil was filled and compacted with an energy equivalent to that in the standard proctor compaction test. Compaction was done by dropping a 5.5 kg square hammer over a height 0.56m and 22 blows were given for one layer. Amount of loose soil used for one layers was so selected to achieve a compacted layer at the top level of the tyres placed. The distance to the anchor tyre increased with the height. The model structure consisted of 13 layers of tyres.

Two series of model studies were done. A lateretic fill was used for the first series. The fill material had a maximum dry density of 1660 kg/m^3 at an optimum moisture content of 19.6%. The shear strength parameters of the as compacted material was determined through a slow direct shear test and values of $c' = 42\ kN/m^2$ and $\phi' = 37$ deg were obtained. In another series where shearing was done after saturating the samples in the shear box assembly $c' = 20\ kN/m^2$ and $\phi' =$ 34 deg were obtained.

The second series of tests were done with a sandy material. Since the concept of optimum moisture content was not applicable an appropriate water content to form a good workable material was determined by trial and error. Soil was compacted at that moisture content to a controlled density of 1900 kg/m^3 by taking a steel roller over the sand layer placed. As with the lateretic fill model structure was constructed incrementally from bottom up. The shear strength parameters of the sand compacted to the controlled density was determined through direct shear tests and values of $c' = 0$ and $\phi' = 34$ were obtained.

3.2 Loading of the Model

A self contained loading frame as depicted by Plate 1 was fabricated in the soil mechanics laboratory of the University of Moratuwa to conduct the model tests. Model was constructed inside the loading frame and a vertical load was applied through a Jack as illustrated in Plate 1.

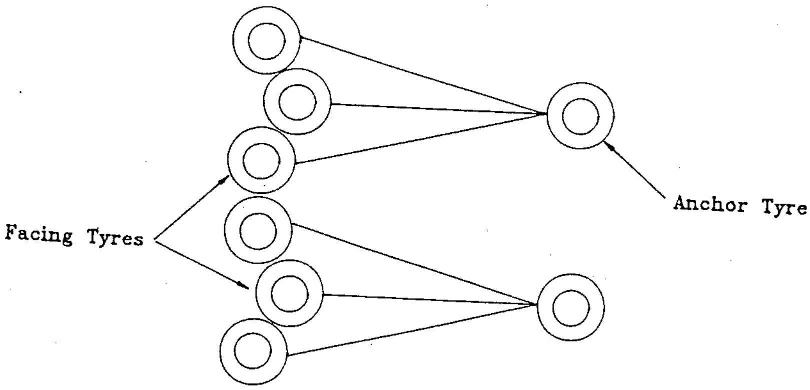


Figure 1 - Basic Unit of the Model Wall

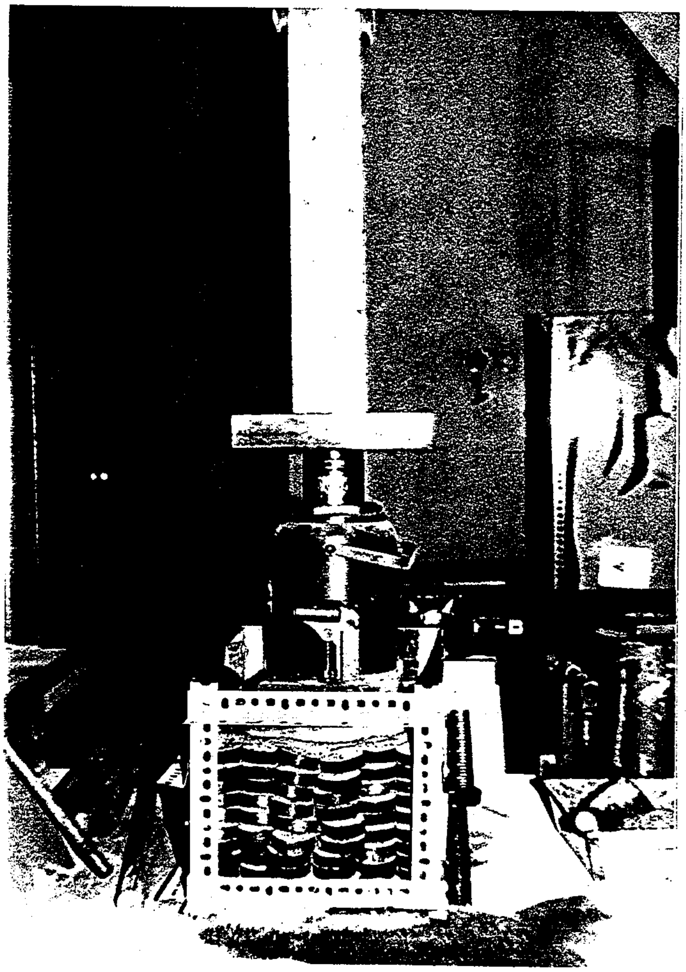


Plate 1 - Loading Frame with a Model Tyre Retaining Wall

The load applied at the top surface of the model was made uniform by keeping several stiff timber planks on top of the model surface and placing the Jack and the loading system on the timber planks.

The load applied was gradually increased through the hydraulic Jack and the resulting outward movements of the model tyres at the facing were measure at various stages of the loading. Also a close eye was kept on the sides of the perspex box to identify any possible failure surface formations.

4. Results of the Model Tests

4.1 Tests with a Lateretic Fill

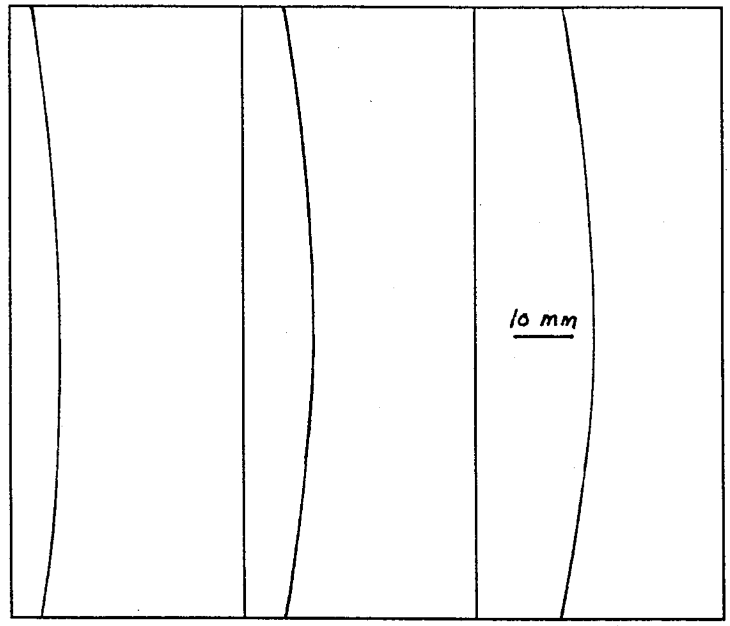
In the initial series of tests nylon wires of diameter 6 mm (larger diameter) were used to connect the tyres. Loading intensity was increased up to 660 kN/m^2 (a vertical load of 31 kN), without any signs of failure. At this stage wall facing has experienced an outward movement of around 18 mm. The defected shape of the facing is given in Figure 2. The vertical settlements closer to the facing were much larger than those further away from it. Due to this differential settlement there were difficulties in keeping the loading system vertical and it started to slip. Hence the loading had to be stopped. It should be noted that a loading intensity of 660 kN/m^2 will never be applied in the field on this form of structures.

However the deformations observed at this level indicates a strain of 9.0% . This is far above the acceptable serviceability limit state of a real structure. But the deformations occurred at the real loading levels were much less and were well within permissible levels.

Some isolated vertical cracks were observed through the sides of the perspex box perpendicular to the facing. A second series of tests were done where 3 mm diameter nylon wires were used to connect the facing tyres and anchor tyres. Load could be increased to a intensity of 176 kN/m^2 where maximum outward movements of the facing tyres were 25mm. Differential settlements at the surface caused the vertical loading to slip and further loading was not possible. However there were no signs of failure.

Model was excavated carefully layer by layer in order to identify the outward movement of the anchor tyres. Observations made in the top most layer was ignored due to the high level of disturbance due to surface settlements. The outward movement (maximum of the anchor tyres were 3 mm at level 2, 2 mm at level 3, 1 mm at level 4, (Figure 3) thereafter no measurable outward movements were noted. Hence the somewhat large outward movement of the facing can be attributed to the extension of the nylon wires. The use of smaller diameter wires in the second series lead to a increased outward movement.

For the next series of tests the prepared model wall was saturated by immersing it in a shallow container and flooding. Degree of saturation was found to be close to 100% through subsequent measurements. During the application of the vertical load vertical settlements of the order of 40 mm were observed and settlement was much more uniform. Hence a loading intensity of 325 kN/m^2 could be achieved. The maximum outward movements of the wall was around 40 mm.



Displacement (mm)

Figure 2 - Typical Deflected form of the Facing (Lateretic Fill)

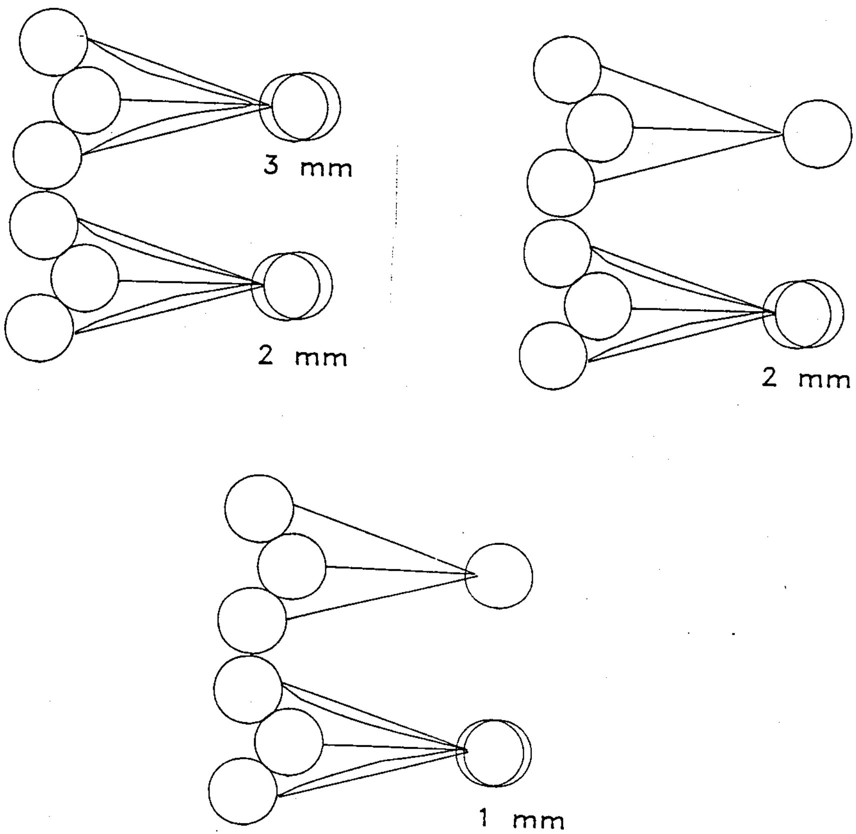


Figure 3 - Outward Movement of Anchor Tyres – Lateretic Fill

Model tests with lateritic fill indicated that

(a). Model wall could be loaded to very high loading intencities without causing plastic failure.
(b). The outward movement of the facing was mainly due to the extension of the wires and deformations could be reduced by using stiffer (or layer diameter) wires.
(c). Saturation increased deformation, but did not cause plastic failure of the structure.

4.2 Tests with a Sandy Fill

Outward movement of facing tyres were measured over the full height and deflected shapes were established. Increase in wall deflection with the loading is depicted in Figure 4. Maximum outward deflections along vertical alignments were between 20 – 25 mm at a final loading intensity of 175 kN/m^2 . Further loading was not possible due to slipping of the vertical load setup.

The failure of the structure would be achieved either by pullout of anchors or due to tensile failure of the connecting wires. Measurements of strains and hence computations of stresses (or tensile forces) in the nylon wires by the use of strain gauges was not possible. Also it had not been possible to provide vertical loads high enough to induce failure by either of the mechanisms. This was partly due to the increase of the pullout resistance caused by the vertical loading.

Therefore in order to induce failure another series of tests were carried out where the connection of facing tyres to the anchor tyres was done with threads used in sewing work. Facing tyres were connected to each other by the nylon wires as before. When the loading intensity reached 50 kN/m^2, the structure failed showing large outward movements and bulging of the facing (Plate 2).

The failed model was excavated carefully layer by layer and the breaking point of the wires (threads) were located. All the wires were initially numbered to prevent any confusion in the process. The measurements made in the breakage points in the two units of the structure were transferred to a sectional view to yield the failure profiles presented in Figure 5. This agrees well with the profile of maximum tensile stresses obtained for reinforced earth, anchored earth and soil nailing reported in literature.

5. Pullout Resistance of Anchor Types

5.1 Studies at TRRL – UK With Bent Triangular Anchors

In their study with bent triangular anchor elements Murray (1983) proposed the collapse mechanism shown in the Figure 6 and developed an expression for pullout resistance as

$$P = \frac{k_P \sigma_V^{/} \ \omega t}{\cos \ \alpha_1} \left[\exp 2(\pi - \alpha_1) \ \tan \phi'\right]$$

Pullout tests were carried out in the laboratory in the apparatus shown in the Figure 7 using model anchors formed from 2.3 mm diameter mild steel rods. The proposed expression underestimated the measured values.

Jones et al (1985) carried out an alternative simpler analysis for the anchor pullout. He considered two possible mechanisms of soil flow around an embedded anchor as shown in the Figure 8. They are

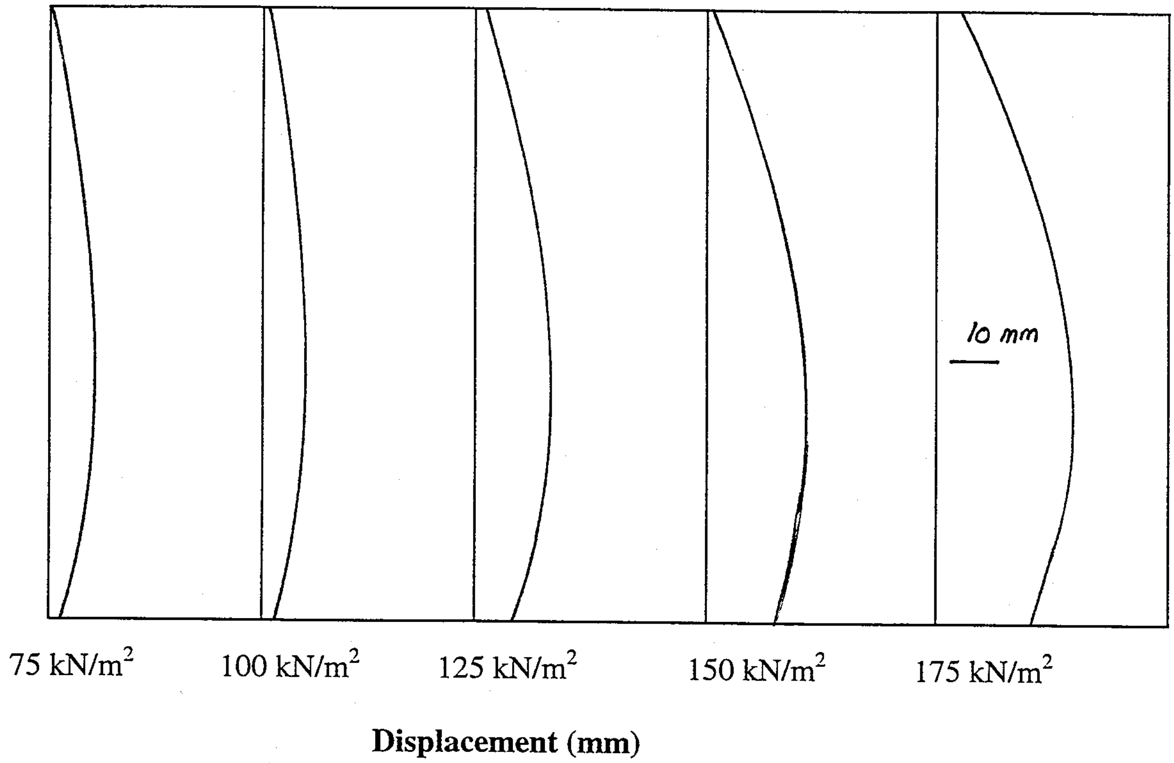


Figure 4 - Facing Movement with Loading (Sandy Fill)

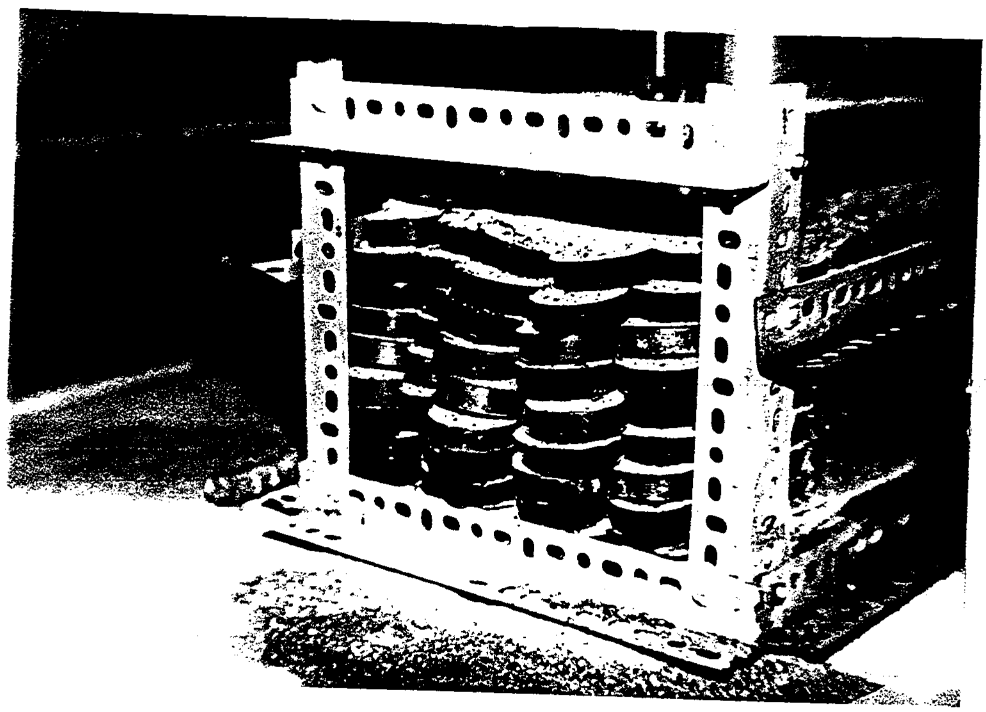


Plate 2 - Collapse of the Wall When threads are used for tying

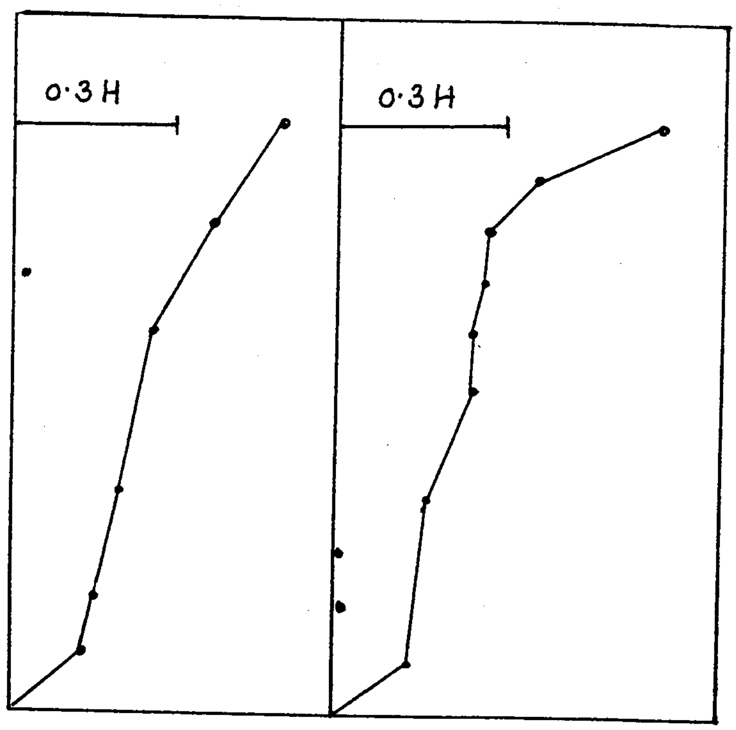


Figure 5 – Profile of Break Points in Threads

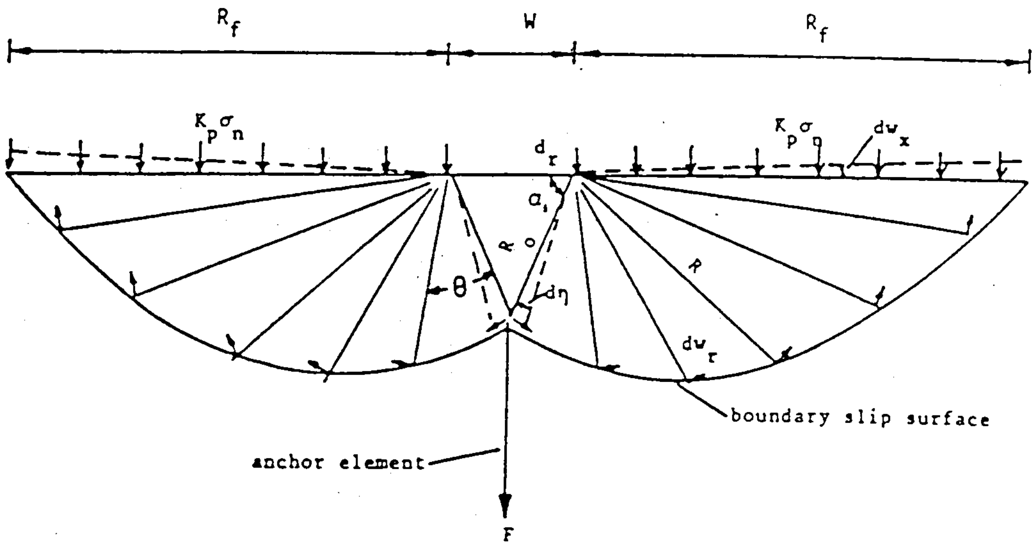


Figure 6 – Collapse Mechanism for Anchor Pullout (Murray 1983)

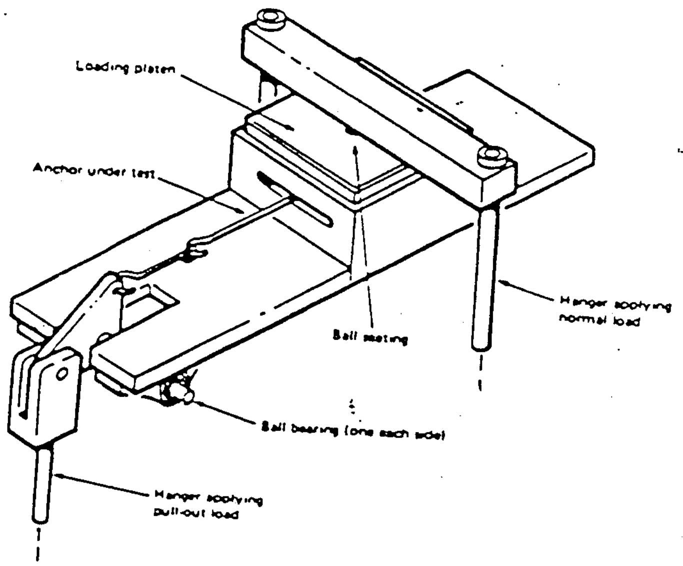


Figure 7 – Apparatus used for Laboratory Pullout Tests (Murray and Irwin)

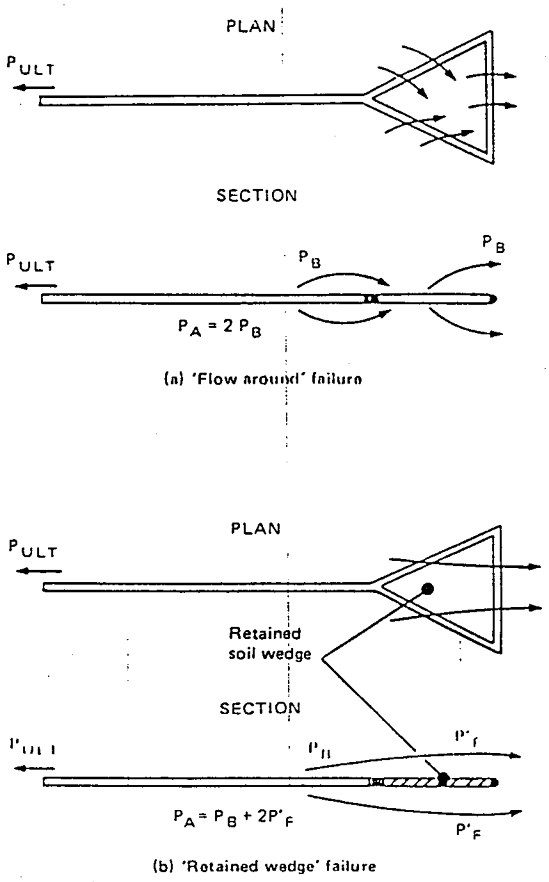


Fig. 2 Pull out mechanisms for a triangular anchor

Figure 8 – Pullout Mechanism for a Triangular Anchor (Jones et al)

(a). Soil bearing pressure on the projected area of bars
(b). Soil bearing pressure on front bars and frictional resistance of soil enclosed by the triangle

In mechanism (a) pullout force is mobilized by the soil bearing pressure on the anchor bar. The pullout force is computed in terms of the surface area A_b of the bar on which a bearing pressure will develop. The pullout resistance P is given by,

$$\therefore \; P = \sigma_b \; A_b$$

where $A_b = 2\,\omega t$
t being the thickness of the anchor element.

Jones et al (1985) suggested that

$$\sigma_b = 4 \; k_P \sigma_V'$$

$$\therefore P = 8\omega t \; k_P \sigma_V' \qquad \text{\_\_\_\_\_\_\_\_\_ (A)}$$

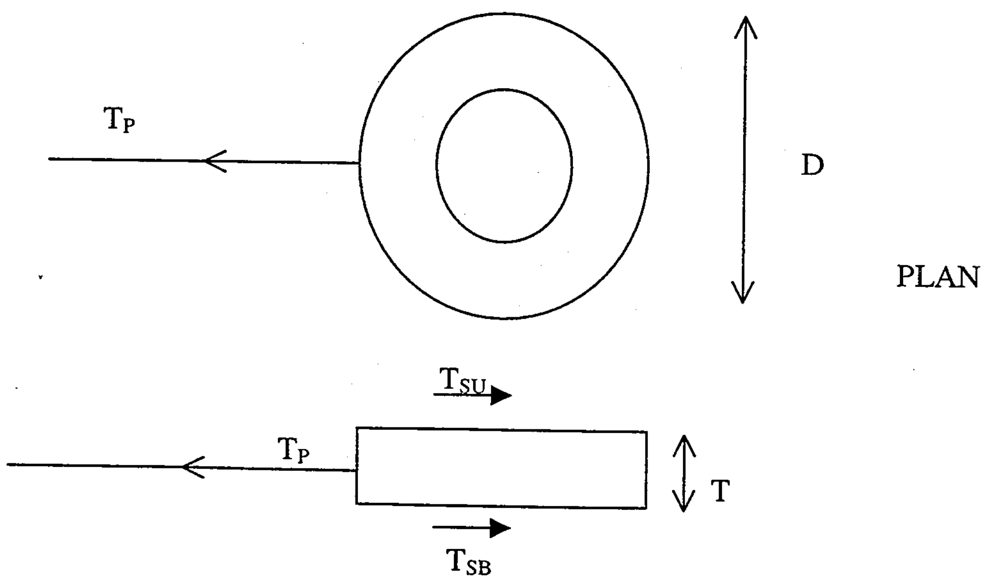
Mechanism (b), considers the soil contained by the triangular anchor to be fixed. Thus the surface area of the anchor for bearing is reduced, and an additional frictional component of pullout resistance develops. The pullout resistance may be expressed as

$$P = \sigma_b \omega t \; + \; \sigma_V \tan\phi \; \times 2 \times A_{TR} \qquad \text{\_\_\_\_\_\_\_\_\_ (B)}$$

The lower value of the pullout resistance (A) and (B) was selected

5.2 <u>Pullout Resistance of Anchored Tyres</u>

The pullout resistance of an anchored tyre depend on the vertical overburden stress, the shear strength parameters of the soil and the dimensions of the anchor. Pullout resistance is derived from the passive resistance in front of tyre and the interface friction between the tyre and the fill at top and bottom surfaces.



Pullout Mechanism for Anchored Tyre

T_P - Total pullout resistance
T_{PP} - Pullout resistance due to passive pressure development
T_{SU} and T_{SB} - Pullout resistance due to interface friction at top and bottom surfaces

$$T_P = T_{PP} + T_{SU} + T_{SB}$$

An expression for T_{PP} was proposed based on passive resistance in front of laterally loaded piles as;

$$T_{PP} = 3\ k_p \sigma_V' \ DT$$

$$T_{SU} = T_{SB} = A(\sigma_V' \tan\phi' + c')$$

The available pullout resistance at each anchor level is estimated based on the above formulation and programmed through a EXCEL worksheet.

In a separate series of experiments tyres were buried in the fill and a nylon wire was taken out and connected to the pullout loading setup. The loading mechanism is still being developed and the present system consist of four spring balances placed in parallel. Attempts were made to pullout the tyre by applying a tensile load through the system after applying a vertical surcharge load at the top of the fill. The vertical surcharge was necessary due to small height of the wall and the resulting low overburden stress. Pullout force is applied through a car jack. When the applied tensile force was increased, a slight outward movement of the anchors (1-2 mm) could be deduced. The tensile force vs movement plots were still showing signs of increase when the maximum possible force was applied through the system. The force that could be applied through the system was limited due to its flexible nature.

However the available data shows that the proposed expression for the pullout resistance under estimates the measured maximum values. Hence the proposed expression for pullout resistance can be considered as providing a conservative or "lower bound" value.

6. Proposed Design Method

6.1 Introduction

There are two aspects of stability in an internally stabilized earth retention systems. The wall unit constructed will resist the overturning and sliding forces exerted by the fill behind. This external stability aspect should be evaluated as for a conventional gravity retaining wall. (Figure 10).

In the internal stability considerations the possibility of internal failure through pullout or tensile failure of the anchor wires was investigated. A wedge stability analysis is proposed here for this purpose. Number of trial wedges are considered at different elevations.

The forces acting on the free body of the wedge are shown in the Figure10, where;

W – Self weight Q – Surcharge
N – Normal reaction S - Shear resistance
and T is the Total anchor force required to keep the system in equilibrium.

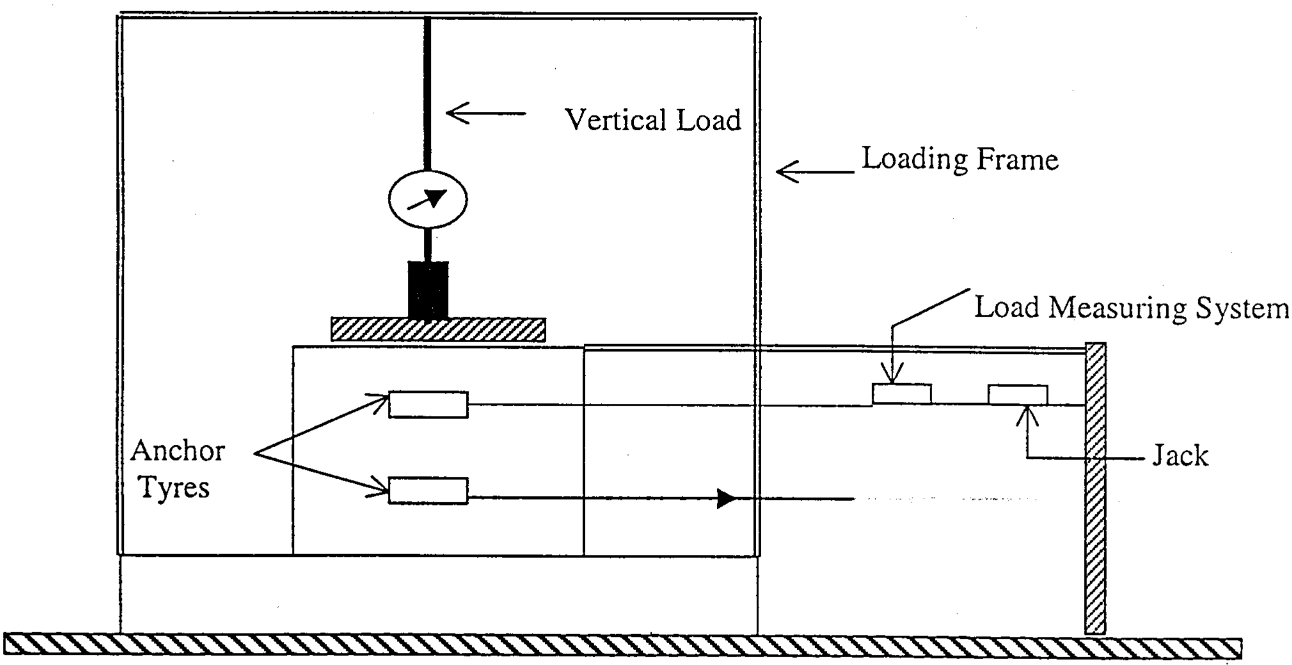
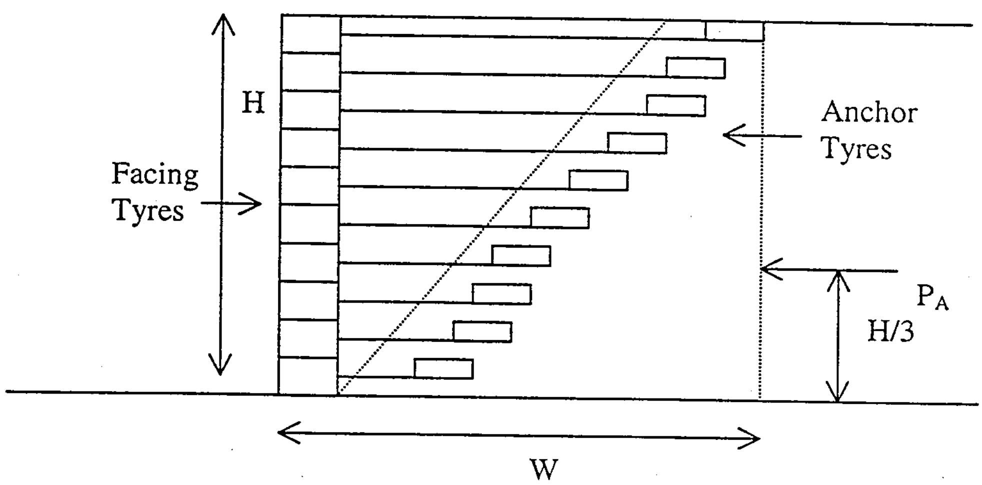


Figure 9 – Pullout Loading Setup



Overturning Moment = P_A H/3

Sliding Force = P_A

Weight of the Structure = W H γ

Figure 10 – External Stability

The equilibrium of the failure mass is considered and an expression is derived for the $T_{(required)}$. The shear strength of the soil is assumed to have fully mobilized over the failure surface. The critical θ giving the maximum value of $T_{(required)}$ can be obtained by differentiation of the general expression.

T available in the structures is the lower of $\sum T_{pullout}$ and $\sum T_{tensile}$. The expression derived for the pullout resistance of an anchor tyre and their tensile strength are determined. The pullout resistances or tensile strengths of the anchors intercepted by the considered failure wedge is summed up and the lower value is taken.

i.e. $\sum T_{available}$ = lower of $\sum T_{pullout}$ and $\sum T_{tensile}$

The factor of safety on internal stability is then defined as

$$FOS = \frac{\sum T_{available}}{T_{required}}$$

Different failure wedges going through various levels of the wall are considered and the factor of safety is computed. A work sheet is developed for this purpose through EXCEL and the results of the analysis of a 8 m high anchored tyre retaining wall is presented in Table 1. It illustrates that the factor of safety values on internal stability are very high. This is typical of the anchored tyre earth retaining structures. These high factor of safety are due to the high pullout resistance of the anchor tyres. Anchor tyre can develop a high pullout resistance due to its large size.

Application of this method to the failed wall with sand gave a factor of safety of 1.05 for the critical wedge going through the bottom of the wall. In that case $\sum T_{available}$ was computed by summing up of the tensile strengths of the threads.

7. Concluding Comments

Some results of model tests on anchored tyre retaining structures are presented. A design procedure for the anchored tyre earth retaining systems is also proposed. The findings of this research indicate that the anchored tyre earth retaining structures are a very economical and efficient system for retaining earth if the necessary width is available. These high safety margins are achieved without high cost.

8. Acknowledgement

The experimental work reported in this research project was carried out during two undergraduate projects by Ms. E N D De Silva, Ms. Paranawithana, Mr. U P Nawagamuwa, Mr. A P Vandabona and Mr. W P K P Perera. The continued interest and cooperation extended by Mr. I H D Sumanarathna – General Manager, RC & DC is gratefully acknowledged.

Design of Anchored Tyre Earth Retaining Structures

Height (m)= 8
Tyre Dia(m)= 0.6
Tyre Thk (m)= 0.2
No of levels = 40

c kN/m^2 = 10
ϕ= 32
γ kN/m^3= 20
kp= 3.254

Unit width(m) = 1.600
tanϕ = 0.625
tan(45+ϕ/2)= 1.804
Area (m^2)= 0.283

Anchor Level	σ_v kN/m^2	T_{p1} kN	T_{p2} kN	T_p kN	T_p/m kN	ΣT_p/m kN	T_{γ}/m kN	FOS Pull out
1								
2	8	9.37	2.83	12.20	7.62	0.00	0.89	0.00
3	12	14.06	4.24	18.30	11.44	7.62	2.00	3.82
4	16	18.74	5.65	24.40	15.25	19.06	3.55	5.37
5	20	23.43	7.07	30.50	19.06	34.31	5.54	6.19
6	24	28.11	8.48	36.60	22.87	53.37	7.98	6.69
7	28	32.80	9.90	42.70	26.69	76.24	10.86	7.02
8	32	37.49	11.31	48.80	30.50	102.93	14.19	7.25
9	36	42.17	12.72	54.90	34.31	133.43	17.96	7.43
10	40	46.86	14.14	60.99	38.12	167.74	22.17	7.56
11	44	51.54	15.55	67.09	41.93	205.86	26.83	7.67
12	48	56.23	16.96	73.19	45.75	247.79	31.93	7.76
13	52	60.91	18.38	79.29	49.56	293.54	37.47	7.83
14	56	65.60	19.79	85.39	53.37	343.10	43.46	7.89
15	60	70.29	21.21	91.49	57.18	396.47	49.89	7.95
16	64	74.97	22.62	97.59	60.99	453.65	56.76	7.99
17	68	79.66	24.03	103.69	64.81	514.64	64.08	8.03
18	72	84.34	25.45	109.79	68.62	579.45	71.84	8.07
19	76	89.03	26.86	115.89	72.43	648.07	80.04	8.10
20	80	93.72	28.27	121.99	76.24	720.50	88.69	8.12
21	84	98.40	29.69	128.09	80.06	796.74	97.78	8.15
22	88	103.09	31.10	134.19	83.87	876.80	107.32	8.17
23	92	107.77	32.52	140.29	87.68	960.67	117.29	8.19
24	96	112.46	33.93	146.39	91.49	1048.35	127.72	8.21
25	100	117.14	35.34	152.49	95.30	1139.84	138.58	8.23
26	104	121.83	36.76	158.59	99.12	1235.14	149.89	8.24
27	108	126.52	38.17	164.69	102.93	1334.26	161.64	8.25
28	112	131.20	39.58	170.79	106.74	1437.19	173.84	8.27
29	116	135.89	41.00	176.88	110.55	1543.93	186.47	8.28
30	120	140.57	42.41	182.98	114.37	1654.48	199.56	8.29
31	124	145.26	43.83	189.08	118.18	1768.85	213.08	8.30
32	128	149.94	45.24	195.18	121.99	1887.03	227.05	8.31
33	132	154.63	46.65	201.28	125.80	2009.02	241.46	8.32
34	136	159.32	48.07	207.38	129.61	2134.82	256.32	8.33
35	140	164.00	49.48	213.48	133.43	2264.43	271.62	8.34
36	144	168.69	50.89	219.58	137.24	2397.86	287.36	8.34
37	148	173.37	52.31	225.68	141.05	2535.10	303.55	8.35
38	152	178.06	53.72	231.78	144.86	2676.15	320.18	8.36
37	148	173.37	52.31	225.68	141.05	2821.01	303.55	9.29
40	160	187.43	56.55	243.98	152.49	2962.06	354.77	8.35

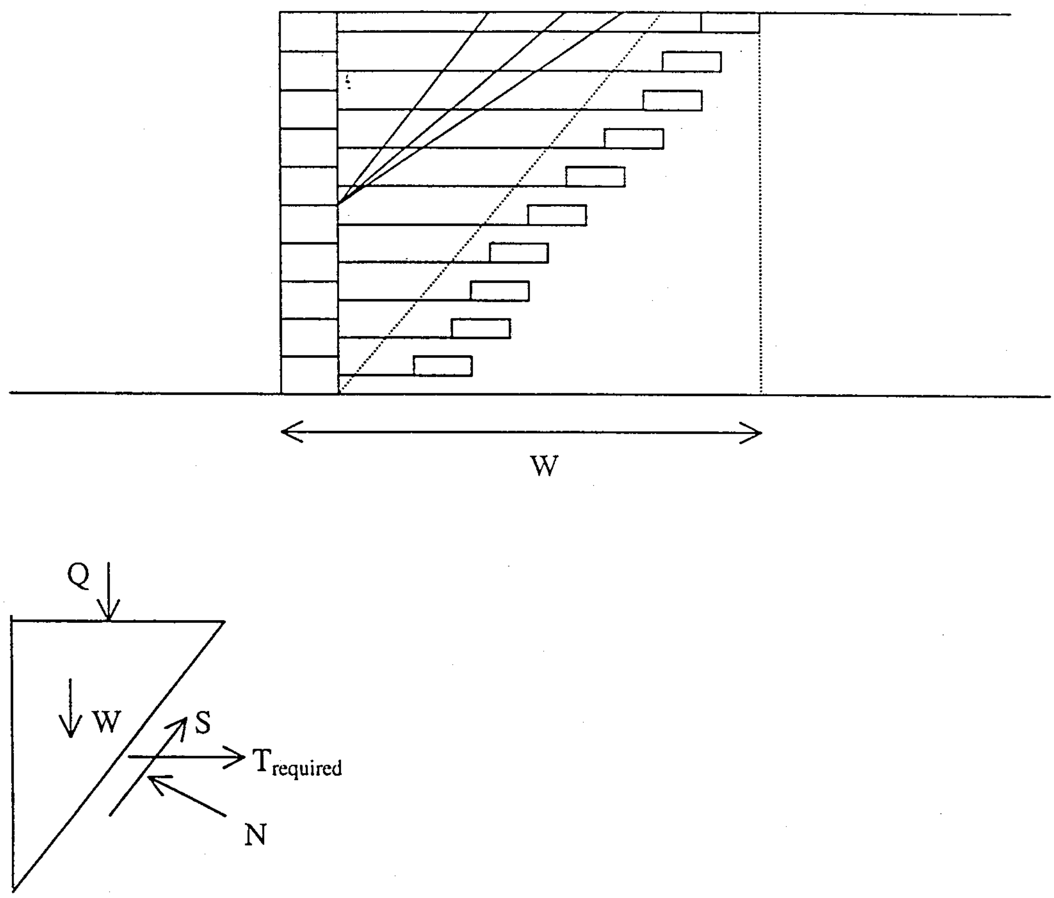


Figure 11 – Internal Stability - Wedge Failure Mechanism

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Jet Grouted Piles As Earth Retaining Structures
A Case Study

S A Karunaratne

Introduction

Jet grouting is a process of eroding the existing structure of a soil by a very high pressure fluid jet accompanied by the injection of cementing fluids to combine with the fluidified soil, to form a mass of grouted soil to produce an impervious barrier or a loadbearing element. The amount of fuidified soil not accommodated in the formation of the grouted mass rises up to the ground surface.

The jet grouting can be broadly classified into three groups, namely "S", "D" and "T" systems. The "S" or the single system involves the injection of a high pressure grout from a single or multiple orifice utilizing single phase rods for drilling and grouting. The grout itself erodes the soil. In the "D" or the double system the grout jet is shrouded by air. The "T" triple system has an independent water jet shrouded by air with grout injection taking place separately beneath the erosion jet. (Figure 1)

The Process

A bore hole of size (approximately 150 mm dia) is drilled to the required depth by water flush. Air-shrouded high pressure water jet rotated in a horizontal direction is operated to cut the soil and form a cylindrical cavity and the eroded soil is flushed to the surface through the annulus around the drill pipe. Cement grout is jetted horizontally either through the water jet nozzle or a separate nozzle. The eroded soil can be wholly removed or partially mixed with the cement grout to form a stabilized soil column say about 0.8 to 2.0 m in diameter. Usually jet grout columns are installed at centre to centre spacing of 0.75 x diameter in a row and the adjacent rows are staggered to form a triple overlap. (Figure 4). The erosion jet is rotated while being withdrawn from the ground to facilitate the formation of cylindrical columns, if not rotated, panels of grouted soil are formed.

The jet grouting process is comparatively costly and has to be monitored carefully to ensure that the required objectives are achieved. When the outflow jetting fluid and the soil becomes throttled or blocked in the annulus around the drill pipe above the cavity, ground surface heaving occurs due to pressure developed in the cavity. Heaving could lead to damage of nearby structures and services. The other problem is the bulky equipment and the disposal of large quantities of water and soil slurry ejected at the surface.

Ceylinco Seylan Developments Limited required the development of its property at number 96, Galle Road, Colombo 03, to accommodate a twin tower structure with two basements. The proposed building spanned from boundary to boundary with access to the premises provided by two ramps located on either sides of the building and abutting the Northern and Southern boundaries.

Anticipated problems with basement construction

The main problems anticipated with the construction of the basement were:

(a) Minimization of damage to adjoining buildings and the main road;
(b) Minimization of ground movements;
(c) Maintenance of existing water table outside the site unchanged

Design of appropriate temporary works had to ensure that ground movements during excavation did not affect adjoining buildings (five storied CMU Head Quarters, Siffani Jewellers building on the North, two storied St. Thomas College preparatory school on the South)and the Galle road on the East. (Fig 2).

Pile driving (sheet piles) was ruled out considering the potential installation problems because of the adjacent buildings founded on shallow pad footings in fine sandy soil. The solution of using a contiguous bored pile wall proposed by the local consultants was found to be expensive. Ove Arup Partners of Singapore were consulted, who proposed the jet grout piling (JGP) system.

Design

To determine the soil parameters 16 bore holes were advanced to an average depth of 14m at this site. Since the soil was sandy, rotary boring instead of percussion boring was used in order to obtain reliable results. Ground water table and the SPT values (N = 6 to 50) at every 1.0m were recorded in all the bore holes. The OASYS - SLOPE computer programme was used to analyse the slope stability and a minimum factor of safety of 1.5 was considered satisfactory due to the temporary nature of the JGP.

The layout of the JGPs is shown in fig. (3). On the North, South and East boundaries 4 rows of interlocking JGPs were used. The diameter of JGPs was 1.5m and a typical arrangement is shown in fig (4). The depth of the pile toe varied from -2.00 to -11.00 m depending on the location. (Refer fig. (5), (6) and (7). At the deepest excavations on the North, South boundaries permanent RSJ's for additional strengthening of the JGP's were embedded in the outer row of JGPs. 28 No. of RSJs were used on each side. JGP's were designed for a minimum cube strength of 5 N/mm2 at 28 days with a minimum mass density of 1800 kN/m3.

Construction

Construction of the JGP's was undertaken by L & M Geotechnic (Pvt) Ltd of Singapore and a total of 741 piles were installed (at an average of 6-7 piles a day) within a period of 4 1/2 months.

Before the installation of working piles; 3 No of preliminary piles were installed within the site and the following parameters were established:

Water cutting pressure	=	400-500 bars
Water content	=	730 Kg/m3
Withdrawal rate	=	8.7 min/m
Grouting pressure	=	150 bars
Consumption of cement	=	0.400 to 0.500 Tonnes/m length of pile
Cement grout mix	=	835 Kg/m3
Rotation per step of 25mm	=	1.5-2.5 revolutions.

Core tests also were carried out which indicated a compressive strength of about 7.5 N/mm2 which is greater than the designed value of 5.0 N/mm2. These piles were exposed and unearthed to inspect the shape and the diameter of the piles.

Technique used at the site.

The system employed at this site consisted of concentric triple tubes with special jet monitors fitted at the bottom. Air and water were jetted .from the upper nozzle at very high pressures (400-500 bars) while cement grout was jetted at slightly lower pressure (150 bars) from the lower nozzle. By rotating the triple tube rods, the high pressure water jet together with air repeatedly impinged on to the soil mass to "erode"the soil. The resulting mass of sludge was removed through the bore hole by air-lift action. The space formed by the water / air jet was filled simultaneously with cement grout from the lower nozzle. The resulting treated soil sludge formed into cylindrical cement mass. By installing JGP on closely spaced grids, a continuous treated layer of improved strength and compressibility characteristic was formed.

Grout Plant

Jet grouting plant is shown in Figure 8. Basically it consist of a main plant (described in section - Jet grouting equipment), high pressure pumps and the rig. High pressure hoses deliver adequate air and grout under pressure from the plant to the grout rigs for the installation of JGP's .

Effluent Disposal.

Drilling effluent and JGP sludge are contained in the working area. This sludge is left to harden, in open areas, and subsequently removed by an excavator and transported by dump trucks for disposal.

Installation Cycle.

The sequence of JGP installation for open area is shown in fig 9. Basically it consists of the following:

a) The JGP rig is set in position for drilling. At each point, the grouting rod and jetting monitor is lowered into the ground by drilling which is effected using air and bottom water discharge through a specially designed triple tube rod system until the designed pile toe level is reached.

b) Upon reaching the required depth, bottom discharge is switched onto a horizonatal direction high pressure water jetting.

c) Injection of cement grout is commenced after the grout nozzle has reached the level of initial of water jetting. While rotating at the predetermined speed, the grout tube is withdrawn at a predetermined rate to form the JGP of required size.

d) Continue withdrawal and rotating of grout tube until the required pile length is complete. During the process, it may be necessary to temporally stop grouting and carry out bore hole flushing above the level of the grout column to "clean" the annular space around the grouting rod and "liquify" the sludge return.

e) The water jet is deactivated upon reaching the cut-off level for the pile. However, the grouting operation is continued and stopped at pile cut off level. The remaining rods are then withdrawn.

f) Upon completion, the grouting rods are removed from the ground.

g) Move to next location and repeat operation.

Setting out

Reference lines were pegged out for the area to be grouted. Two different types of specialist rigs were used in this site.

a) A girder mounted rig: the grids for each line of piles was marked on the running beams and the side of the steel girder itself. The rig was positioned with reference to the grid markings such that the drill rod and jet monitor centred at the intended location of the pile.

b) The crawler mounted rigs: the location of each pile was marked by ground surface peg with reference to the line. The rig was manoeuvred into the location of the pile such that the drill rod and the jet monitor was at the peg location.

Depth

The ground surface level was determined at each sector. Levels were counter checked once every week. From the ground surface level, the required drilling depth was computed. Jet grout rods were in standard lengths of 1 and 2m. The distance of the water jet from the first joint was 200mm. The distance of the grout jet was 750mm below the water jet.

Jet grouting equipment

The main plant include the following :
a) Cement silo complete with grout mixer and agitator
b) Water storage tanks
c) High pressure pumps for Triple Tube System
d) Generators
e) Air compressors
f) JGP rigs : Crawler type/Girder System Type
g) Excavator
h) Mobile crane

Grouting station setup is shown in Fig 8.

Daily reports and Data sheets
During the jet grouting, installation reports and data sheets were submitted, daily. They included material and equipment used, all operational parameters, quality of each material type used, casing, drilling, length of JGP installed, duration and time taken for each installation.

Settlement Points

Temporary Bench Marks were constructed on stable areas near the soil stabilization works. Settlement points were installed on sensitive areas such as, existing structures in the vicinity of the works, boundary of the works, existing pavements, roads and railway.

Inclinometer and Water stand Pipes.

Inclinometers and water stand pipes were installed and monitored during the basement excavations to check whether any movements or rotations of the JGP's have occurred.

Problems encountered during installation

At the Western boundary a sand stone layer was encountered and it was found difficult to penetrate this layer. Therefore, the JGP was constructed in two stages, the first part from the founding level to the bottom of the sand stone layer and the second part commencing from the top of the sand stone layer.

At the design stage, JGP were designed to terminate at 0.5m below the existing ground level. However, due to the sandy conditions JGPs had to terminate at 1.0m below ground level to prevent effluent and sand ejection at very high speed upwards due to the pressure developed in the soil.

4 No. of triple monitors and several lengths of drilling rods were lost in the ground due to wedging in the holes which were drilled, especially in the sand stone layered areas. At these locations the centre was off-set by 200mm and redrilled using higher pressures and the process was successful.

Completion

At selected locations 28 piles were cored, especially at the interlocks and acceptable results; (cube strength and mass density) were witnessed. These cores were tested at the University of Moratuwa for strength characteristics. After completing all JGPs; to ascertain whether there are any ungrouted areas, at every 600mm c/c drilling was done upto 1.0m depth without air, water or grout. One set on the outer row and the other on the inner row of JGPs were performed. Regrouting of 2 No. of piles had to be carried out at the Galle Road boundary where proper stabilization was not achieved.

Conclusions

Even though the total cost of jet grouting was very high, (approximately Rs 96 million), the construction of the basements was executed without damage to the adjacent buildings and to Galle Road which abut the boundaries. Therefore the decision to strengthen the sandy soil by jet grout piling technique, enabled the excavations and construction of the basements to be carried out successfully.

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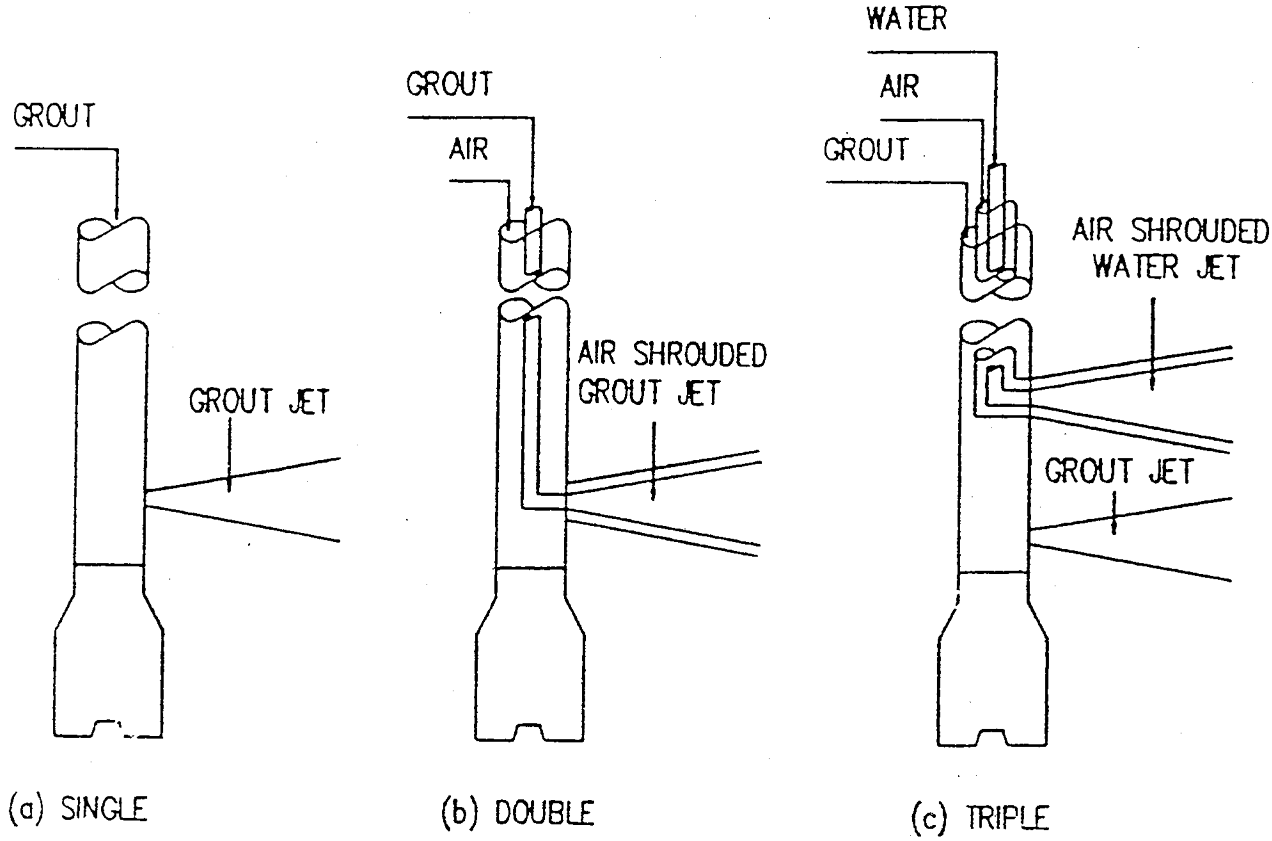
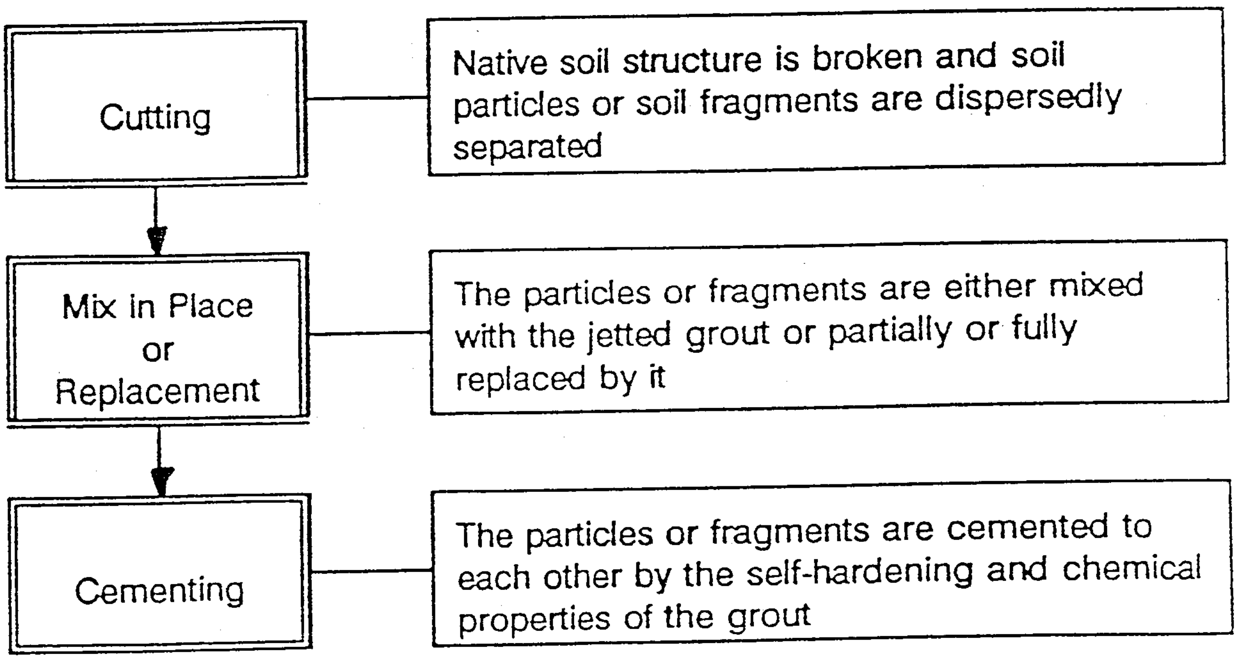
JETSEMII/DN

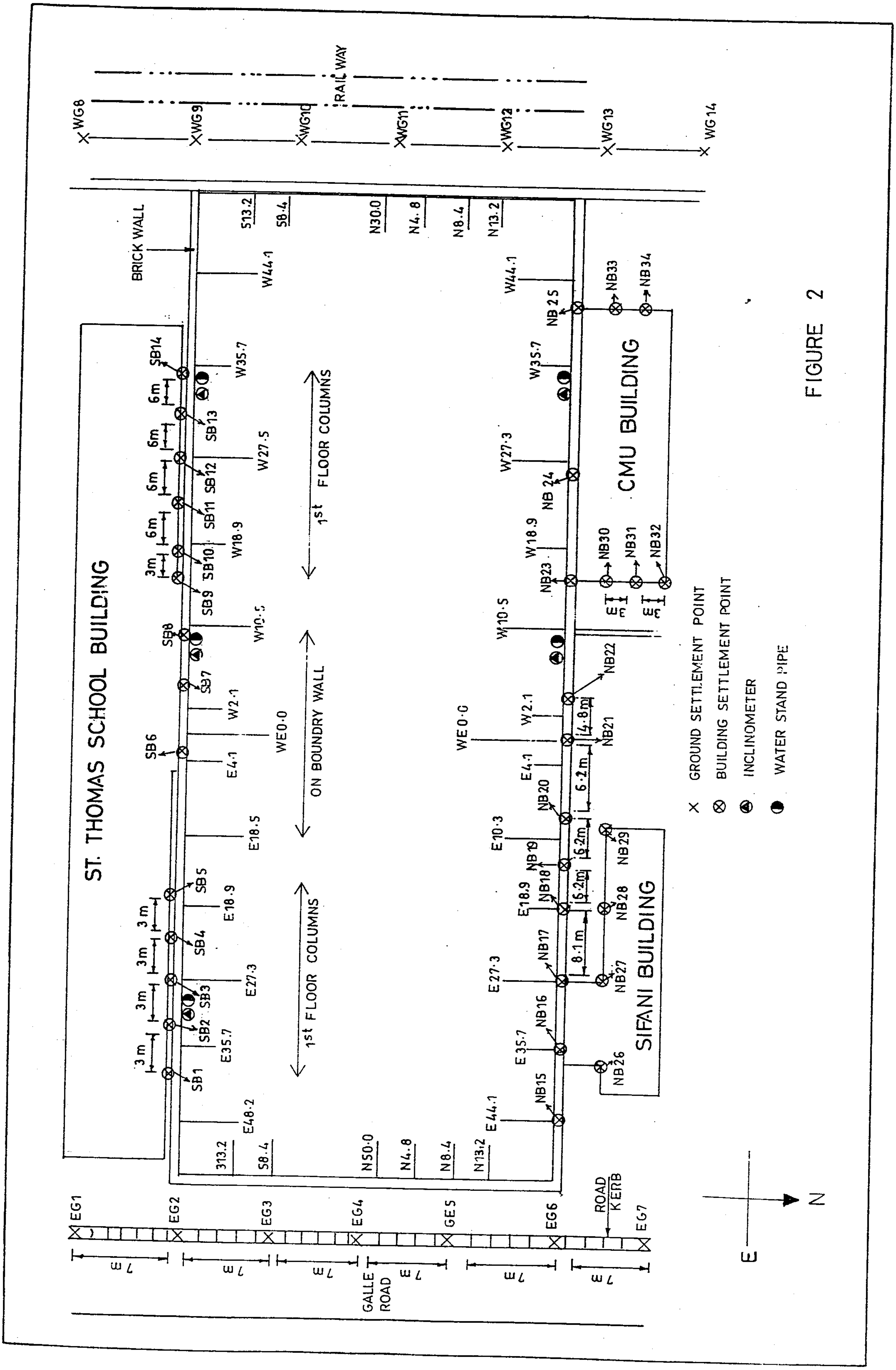


The main variants of the jet grouting processes

FIGURE 01.

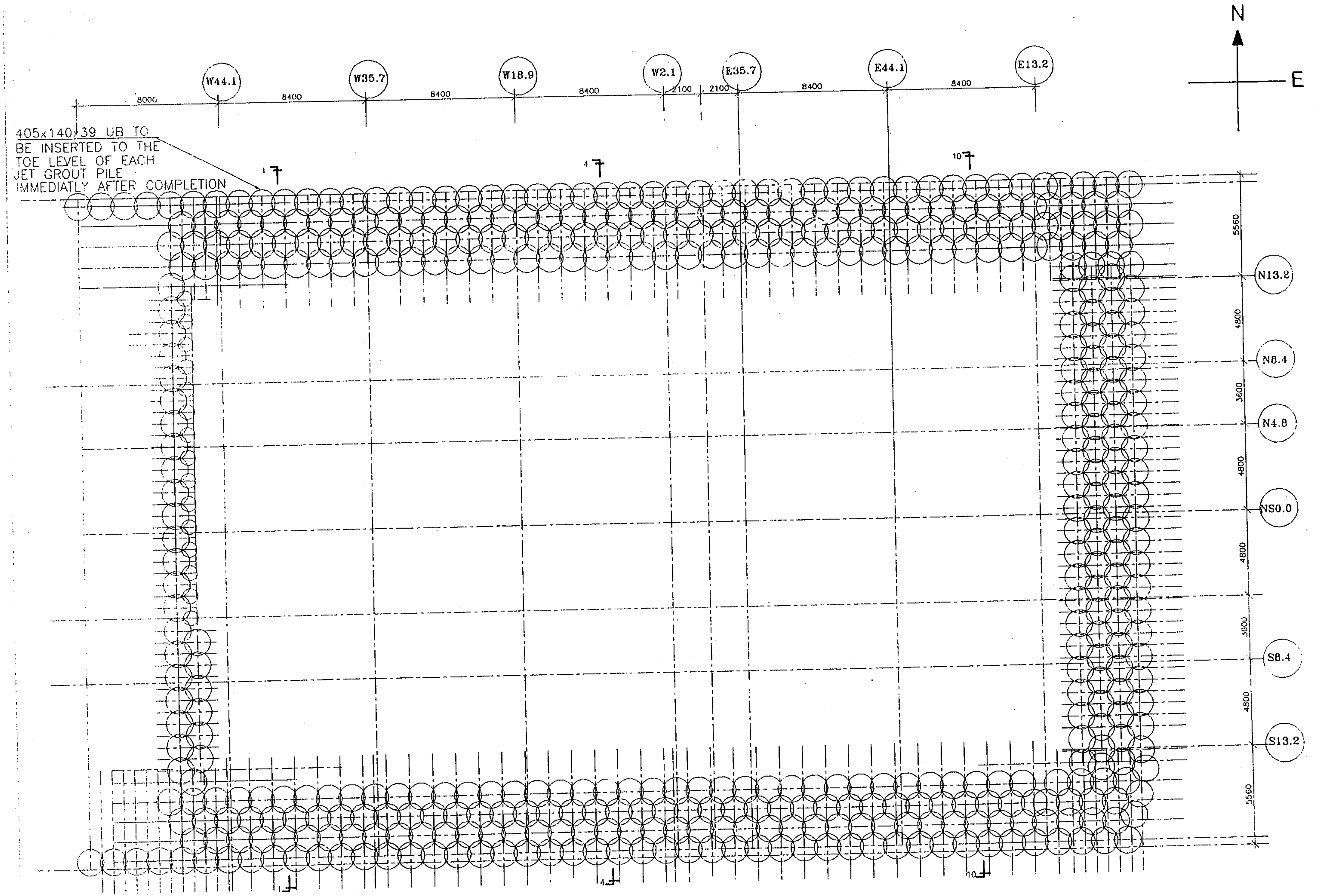


Basic physical principles of jet grouting



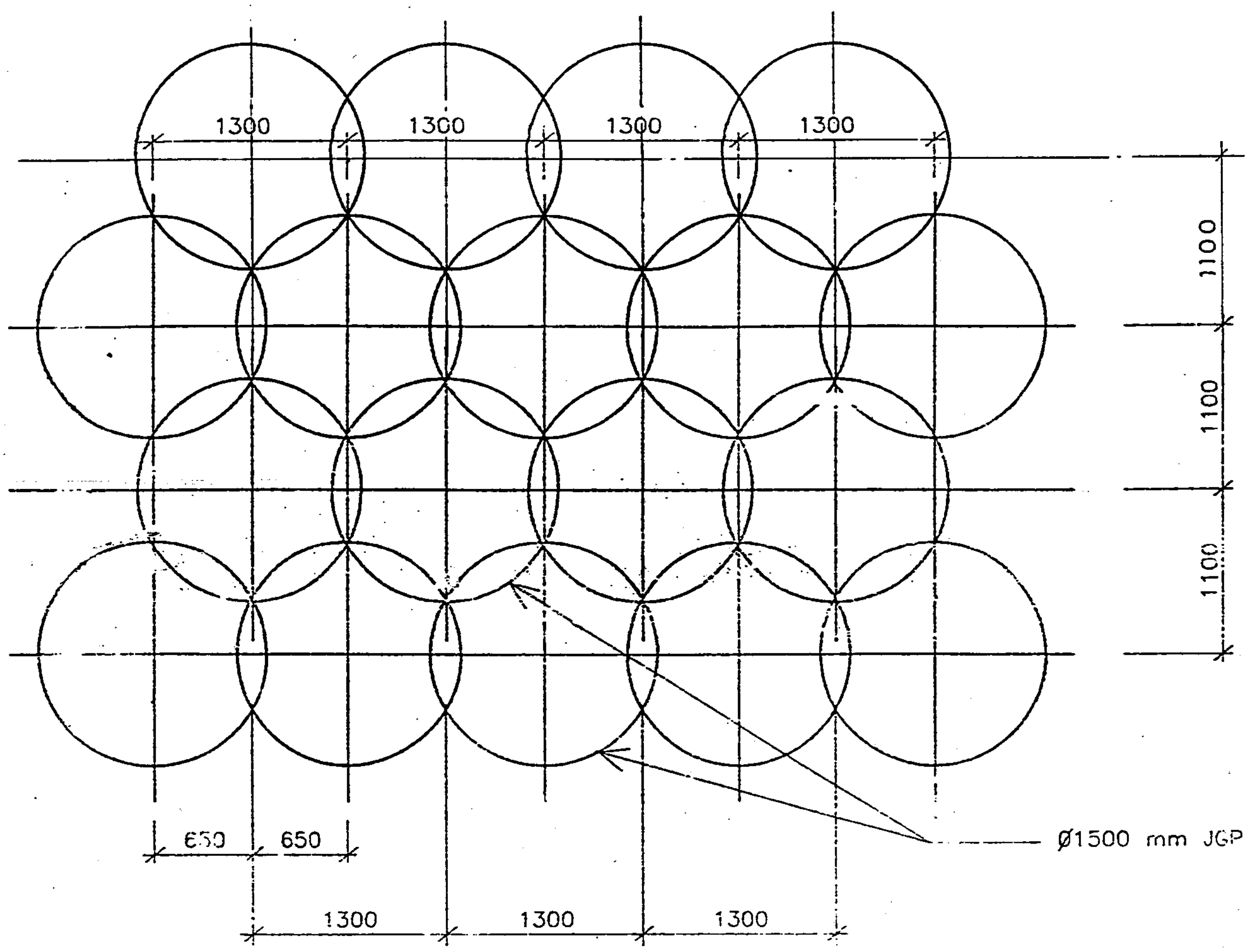
RAIL WAY
WG8
WG9
WG10
WG11
WG12
WG13
WG14
BRICK WALL
ST. THOMAS SCHOOL BUILDING
CMU BUILDING
SIFANI BUILDING
1st FLOOR COLUMNS
ON BOUNDRY WALL
1st FLOOR COLUMNS
SB1
SB2
SB3
SB4
SB5
SB6
SB7
SB8
SB9
SB10
SB11
SB12
SB13
SB14
NB15
NB16
NB17
NB18
NB19
NB20
NB21
NB22
NB23
NB 24
NB 25
NB26
NB27
NB28
NB29
NB30
NB31
NB32
NB33
NB34
3 m
6m
3m
6.2m
6.2 m
8.1 m
4.8m
W44.1
W35.7
W27.5
W27.3
W18.9
W10.5
W2.1
WE0.0
WE 0.0
E4.1
E10.3
E18.5
E18.9
E27.3
E35.7
E44.1
E48.2
S13.2
S8.4
N30.0
N50.0
N4.8
N8.4
N13.2
EG1
EG2
EG3
EG4
GE5
EG6
EG7
ROAD KERB
GALLE ROAD
7 m
N
E
GROUND SETTLEMENT POINT
BUILDING SETTLEMENT POINT
INCLINOMETER
WATER STAND PIPE

FIGURE 2



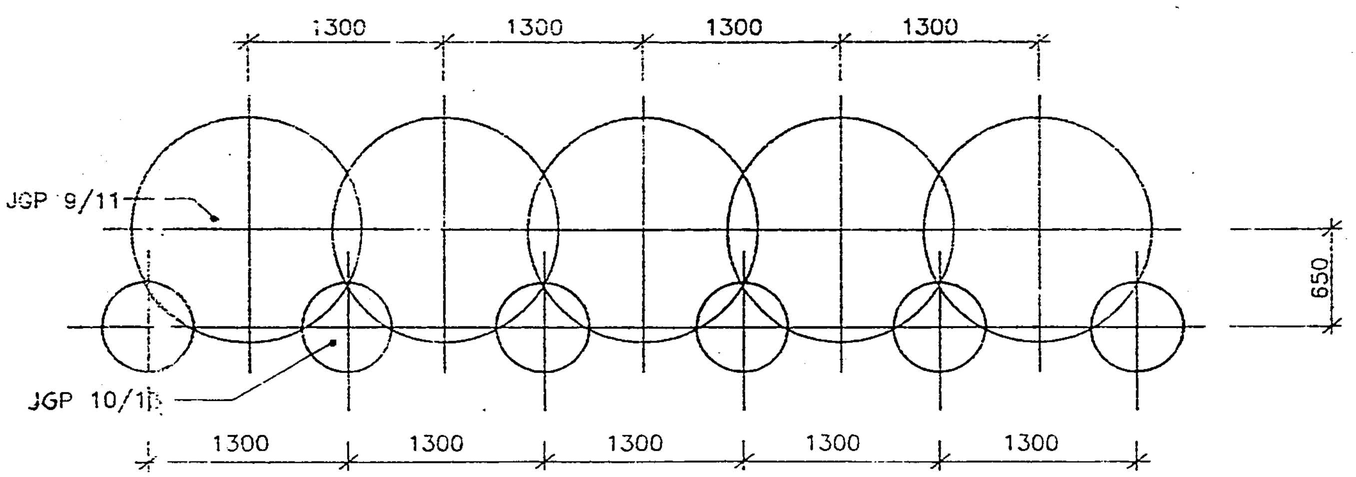
LAOUT OF JET GROUT PILE

FIGURE 3



TYPICAL JGP LAYOUT PLAN

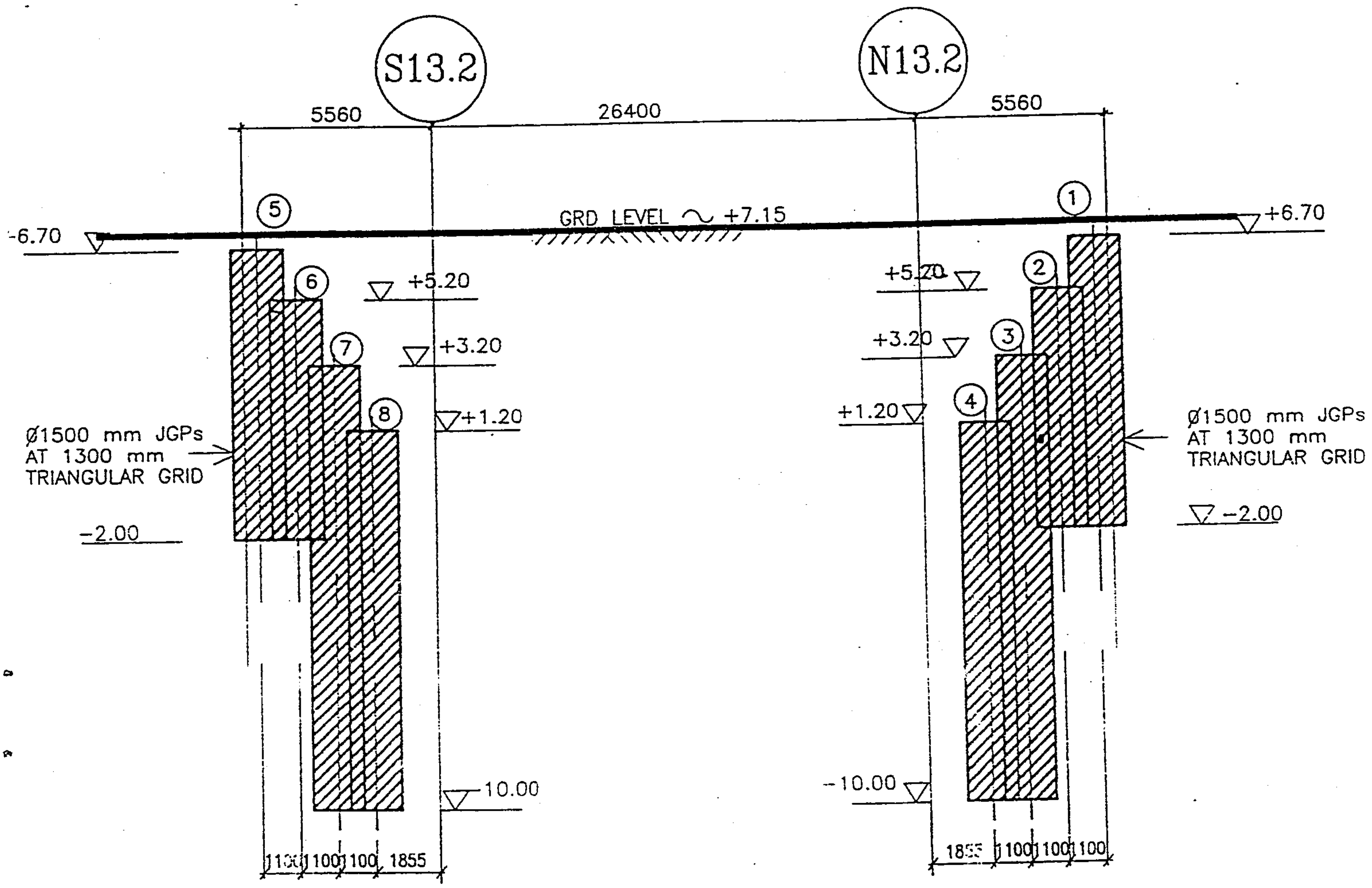
1 : 50



LAYOUT PLAN FOR JGP 9/11 & JGP 10/13

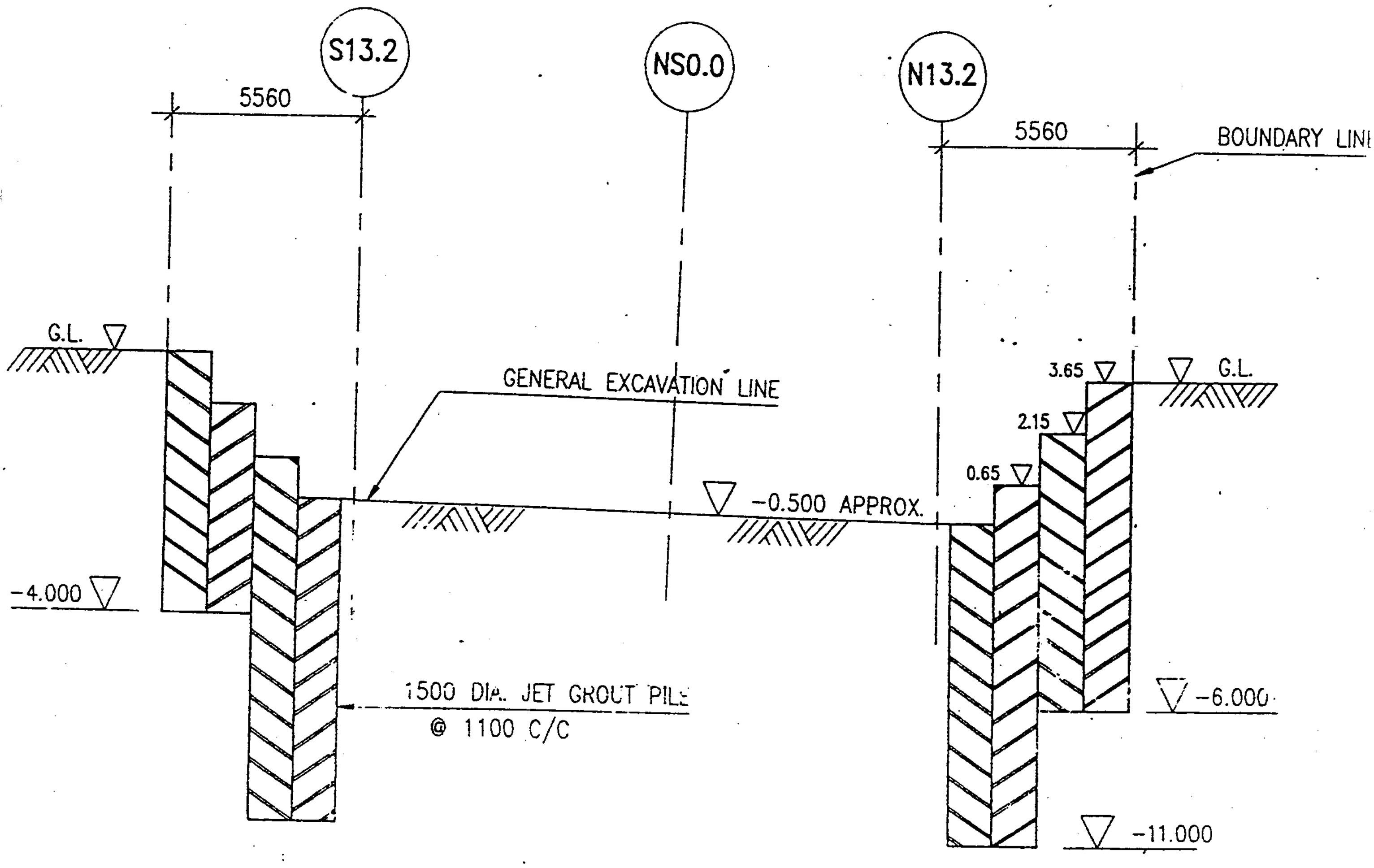
1 : 50

FIGURE 4



SECTION 1 — 1

FIGURE 5

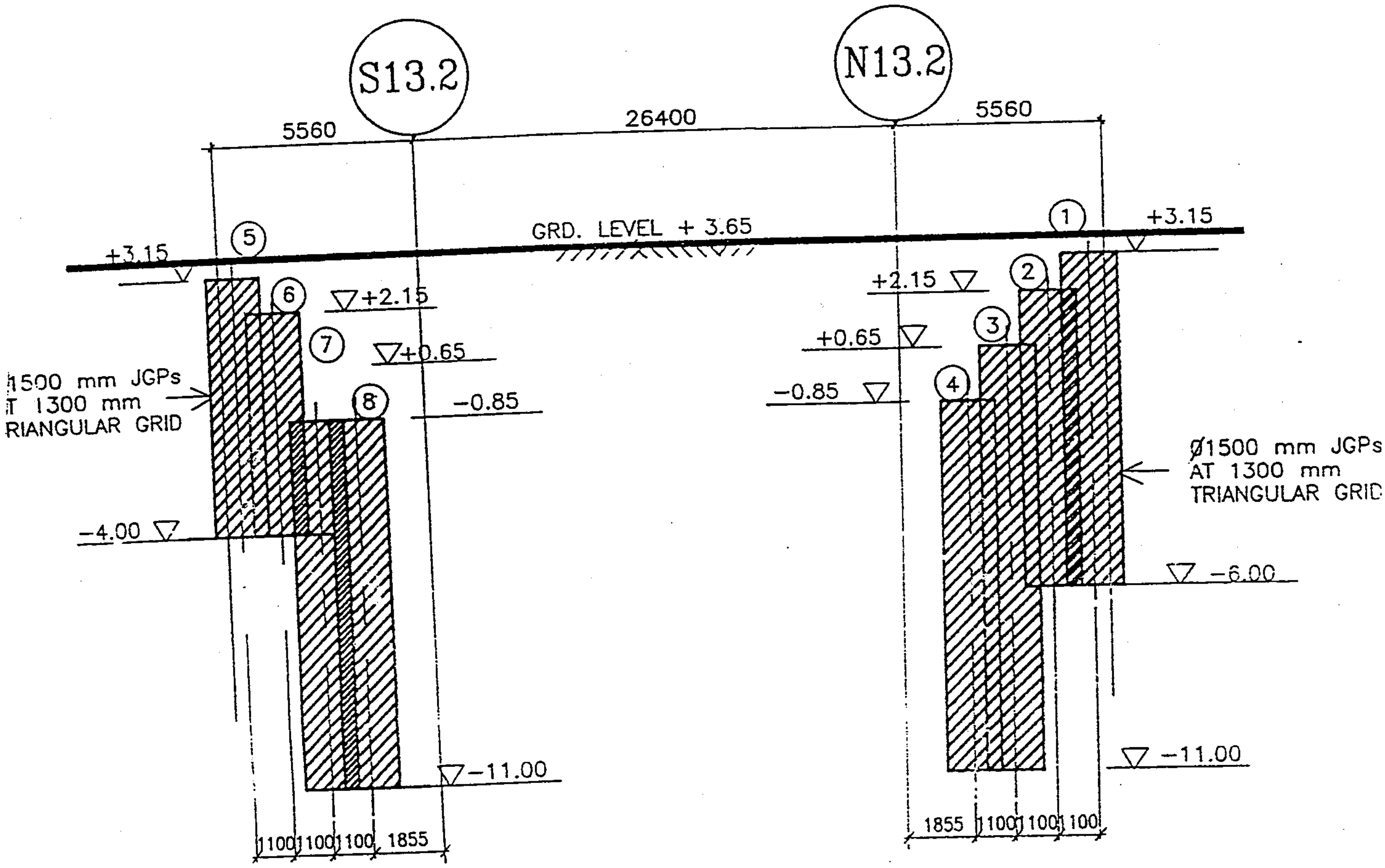


STAGE 1:
JET GROUT PILES TO BE STEPPED

4 – 4

(WITH PILE CAP)

FIGURE 6



SECTION 10 — 10

FIGURE 7

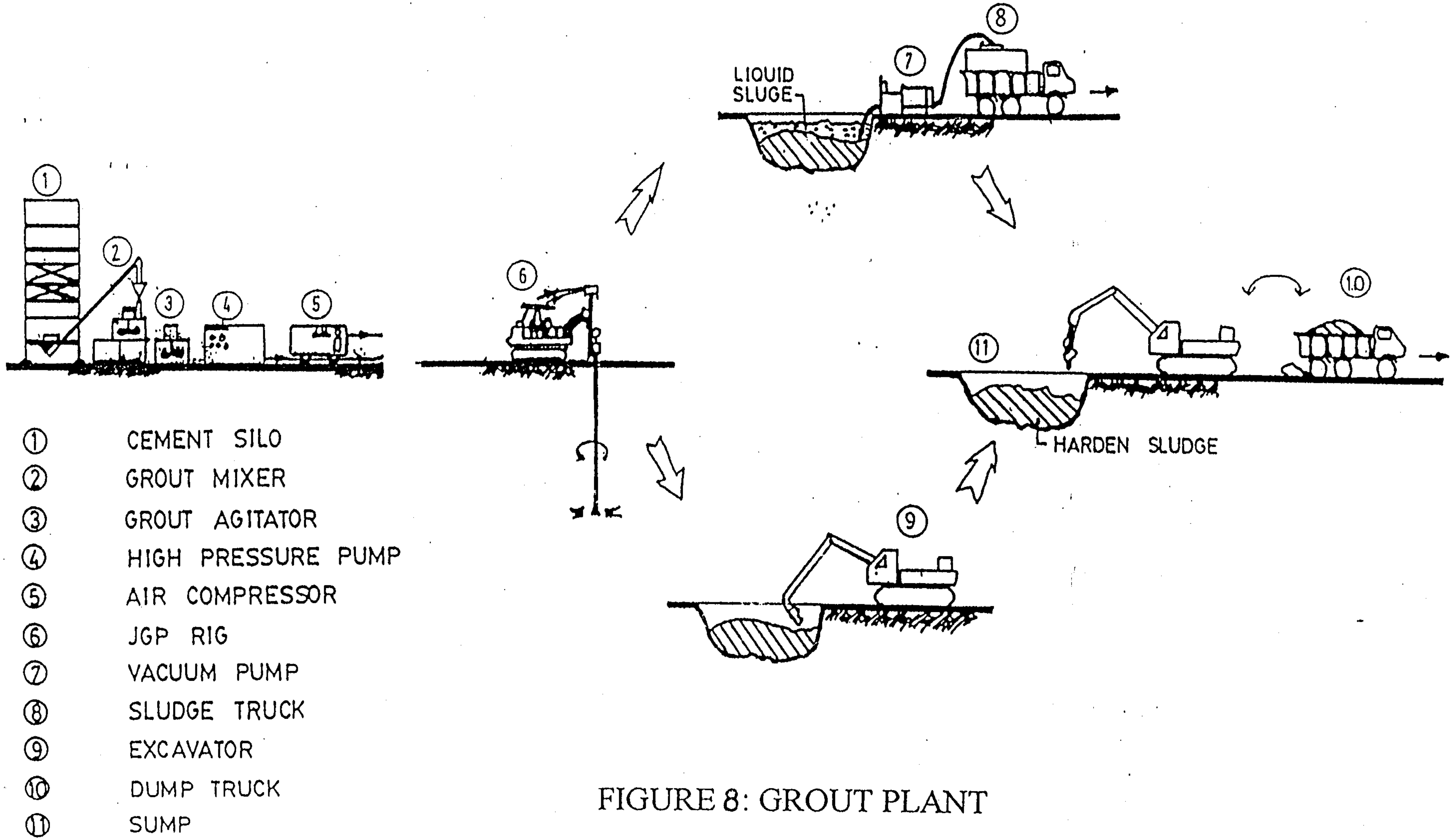
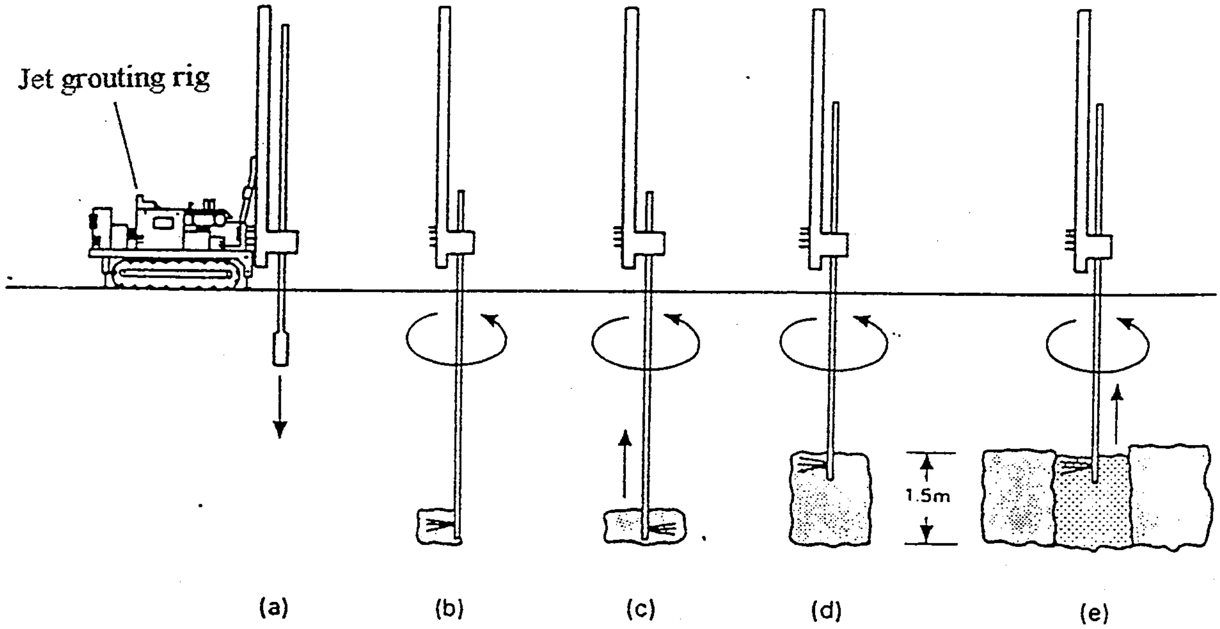


FIGURE 8: GROUT PLANT



Five stages of jet grouting

FIGURE 9

(a) An initial prebore drill hole is formed to the required base depth with the use of rotary water flush technique and a drill bit.

(b) The triple tube system is used and jetting is initiated with the tube rotating at a predetermined rate (approximately 10 rpm) . The air-shrouded water jet (at pressures of 400 bars) is used to cut the soil in situ. The resulting soil slurry is flushed to the ground surface where it is disposed of.

(c) The tubes are then lifted at controlled rates. Simultaneously, grout is injected through the drill bit to replace the removed soil. The advanced triple tube jet grouting process produces in situ cylinders of jet grouted columns.

(d) At the required thickness (1.5 metres) the grouting is terminated and the rods removed.

(e) Once the jet grouted cylinder has formed over the depth range required, the rig is moved to another location. Interlocking cylinders are used to create a retention wall across the site.

EXPERIENCES IN USE OF ANCHORED BORED PILE RETAINING WALL IN CENTRAL BANK BUILDING EXTENSION PROJECT

BY

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1.0 INTRODUCTION

The Central Bank Project envisages the construction of a new building with a floor area of 31,000m^2 (333,500 sq.ft) and overall height of 70m in the adjacent land of the existing Bank Building at an estimated cost of Rs. 2500 million.

The sub-structure of the building consists of a three storey basement of cast-in-situ concrete mainly comprising of vaults, stores and customer parking areas. The superstructure consists of thirteen storeys of floors of in-situ/precast composite concrete slabs with a cast insitu concrete frame work and central shear core comprising stair cases and lift and service shafts.

The entire structure is supported partly directly on the bed rock and the balance on reinforced concrete piles bored into the bedrock.

2.0 THE SITE AND ITS GEOLOGY

The Site which occupies an area of approximately 60m by 50m in the heart of the business centre of the city of Colombo, immediately adjoins and is bordered by the existing Central Bank Head Office Building, Navy Officers' flats, Upper Chatham Street and Janadhipathi Mawatha. (Figure 1). The entire area of the site will be used for construction below existing ground level (EGL) down to the 1st basement level (which is about 6m below the EGL) and approximately 70% thereof for construction down to the third basement level (approximately 15m below the EGL).

Site investigations revealed that the sub-soil of the site mainly consists of lateritic soils of which the soil type varies widely, ranging from stiff clay to loose clayey sand overlying pre-Cambrian metamorphic bedrock. Bedrock levels vary from 8.0m to a depth in excess of 25m below the existing ground level indicating unusual abrupt variation across the site (Figure II). Very steep rock surface is recorded between axis 4 and axis 5 in the East – West direction (Rock level drops about 12m within a distance of 6m) but there are also other local areas where the inclination of the rock is very steep. Floating boulders or hard intermediate rock lenses have been encountered in some places of the site in isolation or in the form of clusters. The thickness of these lies mostly between 1m and 2m, but can also vary above and below these values. The locations of these boulders vary, between ground level and bedrock level.

The average ground water level varies from 4m to 6m below the existing ground level due to yearly seasonal variation.

3.0 CONSTRUCTION OF THE EXCAVATION PIT

The construction of the basements required the excavation of the site down to a depth of 15m from the existing ground level. Since the excavation extended to the boundaries of the site a vertical cut was inevitable for the excavation pit. This vertical cut had to be retained not only against collapse but also against possible lateral movements which would cause settlement and endanger the stability of the existing buildings and roads in the vicinity of the site. Further, the lowering of ground water table outside the site had to be prevented during t dewatering of the excavation pit to avoid settlement of adjoining structures.

For retention of the basement excavation, a tied back overlapped bored pile wall comprising 900mm diameter cast insitu piles socketed into bed rock at its lowest level was constructed. (Figure III)

Drilling of bored piles was carried out using an auger, and the excavation was temporarily retained by casing in the overburden soil, while drilling through rock and boulders was done using core barrel having round shaft bits.

Tied back soil and rock anchors were used to support the overlapped bored pile wall in the deep excavation pit, while bored pile wall at the Navy boundary was supported with inclined steel struts.

In order to resist high loads due to earth and water pressure, upto three/four layers of anchors were installed in the deep excavation area. The first layer of anchors was installed within the overburden soil where the maxium permissible anchor force was limited to 400 kN. Post grout injections were carried out around the anchor bulbs to increase the load carrying capacity of these anchors. The second, third and fourth anchor layers were extended and anchored into the bedrock to achieve a maximum load carrying capacity of the order of 1000 kN.

4.0 BEHAVIOUR OF THE BORED PILE WALL

During the first excavation stage (Figure IV) the pile-wall acts as a cantilevered beam embedded in soil beneath the excavation level.

Before excavation was commenced, earth pressure on both sides of the pile-wall was identical. As the pit was excavated, lateral earth pressure acting on the pile-wall in the part exposed by excavation would disappear. Concurrently, due to reduction of overburden pressure, there would be a reduction of earth pressure in the soil below the excavation on the side of the pit.

Imbalance of pressures on the two sides would cause the wall to move towards the pit. Pressure on the retained side of the pile-wall would be reduced as a result of this movement to a value that lies between the "at rest" pressure and "active" pressure. Movement of the wall would also cause the soil below the excavation to be compressed, resulting in an increase of earth pressure acting on the wall on the pit-side.

Movement of the pile-wall would continue until a position of equilibrium was reached where the earth pressure mobilized below the excavation on the pit-side would balance the pressure on the retained side. Limiting value of pressure that could be mobilized below the excavation was the so called "passive earth pressure" obtained by Rankine Theory.

Maximum depth to which the pit could be excavated with the wall acting as a cantilevered beam was determined by the following:

(a) Strength of the pile wall,
(b) Whether the embedment was deep enough to generate adequate passive resistance to balance forces on the retained side of the wall.

If excavation was to proceed more than this maximum depth, additional lateral support had to be provided for the pile wall. Such support could have been provided either by internal struts, or by tie-back anchors.

Tie-back anchors were steel rods or cables inserted into boreholes drilled into the soil through the pile-wall, backfilled and pressure grouted with cement, prestressed and fastened onto the wall.

These tie-backs had to be anchored through the anchor body which were located outside the failure region of the soil. Prestressing force on these anchors was transferred to the ground by skin-friction along the length of the anchor body.

It should be noted that prestressing of anchors does not reverse the displacement of the pile wall that has already taken place.

For design analysis, the embedded part of the pile wall (i.e. the part below the excavated depth) was assumed to be elastically supported in the lateral direction. For the purpose of mathematical modelling, a series of elastic springs was assumed to act laterally on the pile wall, where the maximum force of each spring was limited to the value of passive earth resistance corresponding to the level of the spring.

In design analysis of the second excavation stage (Figure V), each anchor was assumed to act as a rigid support until the support reaction exceeded the prestressing force. Once the support reaction exceeded the prestressing force, the anchor acted as an elastic spring and further movement of the pile-wall took place.

As well known in soil mechanics, the classical Rankine distribution of active earth pressure is not valid for braced or tied back walls due to the external restraint. When the pit is excavated further after installation of anchors, a redistribution of the earth pressure occurs with an increase of pressure near the supports, and a decrease elsewhere.

After installing the second level of anchors, excavation can be continued to each successive level where a further layer of anchors are required until the final excavation depth is reached (Figure V1 & VII).

On this project, the interaction between wall deflections and subsequent change of the soil behaviour was not analysed by Finite Element Analysis or by Boundary Element Methods. Instead, empirical redistribution diagrams were used for this purpose.

5.0 SUMMARY OF THE DESIGN PROCEDURE

The design procedure of the tiedback bored pile wall could be summarised as follows:

1. The bored pile wall was analysed at each successive construction stage to determine the internal stresses of the wall (i.e. bending and shear), minimum embedment depth of the wall and the anchor force.
2. The loads acting on the wall were assumed to be the lateral earth pressure (deformation related) due to self weight of the soil and the adjacent buildings, and the hydrostatic pressure.
3. It was assumed that the wall was supported partly by prestressed ground anchors and partly by passive earth pressure at the bottom of the excavation.
4. In the analysis, the progressive irreversible wall deformations and safety against the failure of the supporting passive earth pressure at the bottom of the excavation were considered.
5. The length of the anchor was determined by the wedge analysis (slip plane analysis) for the required safety factor (Figure VIII).
6. The required length of the anchor body was determined according to the empirical design chart (Figure IX).
7. The safety against possible slip circle failure of the whole system was also checked (Figure X).

To simplify the analyses, the overburden soil was divided to a few numbers of horizontal soil layers based on the results of the detailed soil investigation. For each layer, soil properties such as internal angle of friction (ϕ), cohesion (C) , in-situ density were determined using undisturbed soil samples collected at the site. Using the water level observed in the boreholes average ground water level was established. Following loadings were considered for the analyses.

(i) Nominal surcharge of 10 kN/m^2 where vehicle movements would be anticipated.
(ii) Actual loads on the foundations of the building within the influence area of the wall.
(iii) Effect of water table.

(iv) Earth pressure distribution (using simplified profiles).
(v) Stepped excavation.

6.0 CONSTRUCTION ASPECTS

6.1 CONSTRUCTION OF GUIDE WALL

After cleaning the area required for the construction of the overlapped bored pile wall, a boring template was constructed in reinforcement concrete along the axis of the wall. This template ensures the correct location of the boring equipment.

6.2 CONSTRUCTION OF OVERLAPPED BORED PILE WALL

Drilling for bored piles was done using the cased boring method. Removal of soil was done using kelly bar and short flight auger driven by Bauer BG 11 and BG 14 drilling rigs.

Core barrels with round shaft bits were used for drilling in rocks. Primary piles constructed in advance were unreinforced. These served as infill piles. Secondary piles were drilled between pairs of primary piles. These secondary piles were reinforced according to statical requirements.

Construction of secondary piles took place 1 to 3 days after completion of primary piles. After the final bore depth was achieved the reinforcement cage was installed (only in secondary piles).

Using a tremmie pipe, piles were concreted. Placing of concrete below the water level was necessary with simultaneous withdrawal of the pile casing.

6.3 TIED BACK ANCHORS

The bored pile wall was tied back using temporary grouted ground anchors designed according to DIN 4125 in order to bear the high loads due to:

(i) earth & water pressure,
(ii) surcharge loads and
(iii) loads due to adjoining buildings.

Two to four rows of anchors were installed depending on the loading requirements. Multi-strand ground anchor system was used. Each anchor consisted of 3 to 9 strands depending on anchor loads. The grout bodies of the 1^{st} anchor layer were situated mostly in cohesive soil and working load of each anchor ranged upto 400 kN approximately. In the case of 2^{nd} and 3^{rd} anchor layers grout bodies were situated in rock and the working loads were 750 kN and 1000 kN respectively.

A typical detail of an anchor is presented in Figure XI. Anchor head transfers the anchor force on the bored pile wall. Anchor length varied from 8m – 30m. Grout bodies which transfer the force from anchor to the ground vary in length between 4.0 to 6.0m.

Core boring of 150mm diameter was carried out using a hydraulically operated core drilling machine through the reinforced pile in order to avoid any vibratory forces on the bored pile wall. Thereafter the anchoring rig was moved to the location and the anchor casing was installed for the full length of the anchor in case of soil anchors and down to rock in case of rock anchors. This was done using a drag bit and with water as a flushing medium. After reaching hard rock, drilling was done using a 'down-the-hole hammer' which was operated by compressed air. When intermediate boulders were encountered these were penetrated by 'down-the-hole hammer' with eccentric bit assembly.

The borehole was cleaned by compressed air and the pre fabricated anchor was installed. Cement grout was pumped into the borehole using the grout tube mounted on the anchor until grout overflowed from the upper end of the borehole. In order to increase the interlock for the load transfer between grout body and surrounding ground the installed anchor was subsequently pressure grouted in the region of the anchor body. In order to prevent this occurring over the whole length of the anchors, a sealing ring was installed between the grout body and the free anchor length. The grout had a water/cement mixing ratio of 0.45 and grout strength at 7 days was expected to be not less than 20 N/mm^2. Once the grout material hardened after one week from grouting, the anchors were tensioned.

7.0 TESTING OF ANCHORS

Two types of anchor testing were carried out

(a) On -site suitability test
(b) Acceptance test

7.1 ON - SITE SUITABILITY TEST

A preliminary, on-site suitability test had to be carried out on three anchors of each type that was installed. This was intended to check whether the particular type of anchors was capable of transmitting the design working load to the surrounding ground under the given local conditions. It also served to verify the bearing capacity of the grout body and the associated displacement.

Test anchors were constructed as described above and tested after the grout body had hardened. (after one week from installation)

The proof load (F_p) was calculated as

$$F_p = N_k . F_w$$

Where

N_k = 1.33 (Safety Factor from DIN 4125, Table 1, Load Case 2)

F_w = Maximum safe working load (i.e. 400 kN for soil anchors and 1000 kN for rock anchors)

In the suitability test, starting from pre-load, Fi, not exceeding 0.2 F_w, the load was increased in stages until proof load F_P was reached. When each of these load levels was reached for the first time the load was kept constant for the minimum observation period and was followed by gradual reduction to Fi to check the anchorage for any plastic displacement. Following this the load was increased to the next load level.

After reading the proof lead, displacements were observed for a longer period (i.e. 1, 2, 5, 10, 15, 20, 25, 30 minutes) and a displacement against time curve was plotted. Using this data creep displacement (k_s) was established.

Following observation of displacements, the load was released to Fi in stages and then the tendons were stressed to working load taking into consideration the slip at the anchor head.

The results of suitability tests were compared with requirements of section 10.4 of DIN 4125.

A sample result sheet of suitability test is presented in Figure XII

7.2 ACCEPTANCE TEST

The acceptance test is intended to verify the bearing capacity of every anchor after installation.

In this case proof load

$$F_p = 1.25 . F_w$$

Starting from pre load Fi the load was increased in stages until proof load was reached. At each step, the displacement of the free end of the anchor and the load were measured.

The acceptance test was considered as successful when the requirements of elastic displacement and creep length (k_s). complied with requirements of section 11.3 of DIN 4125.

A sample acceptance test result sheet is presented in Figure XIII.

8.0 MONITORING OF BORED PILE WALL.

Theoretical deformation of the bored pile wall and anchor forces under the anticipated loading conditions and sequential excavation stages had been obtained from the statical calculations.

Monitoring of the actual deformation of the bored pile wall and actual forces that were induced on the anchors were vital during the construction stage due to the following reasons.

(a) To ensure stability of the bored pile wall.
(b) To take precautionary measures in the event of excessive deformation.
(c) To assess the validity of design criteria assumed for the analyses.

The monitoring programme of the bored pile wall involved the following.(Figure XIV)

(i) Installation of survey monitoring points along the bored pile wall.
(ii) Installation of vertical digital inclinometer leading tubes at 6 locations (P32, P78, P102, P154, P238 & P294).
(iii) Installation of hydraulic load cells at 3 anchor levels at positions P32 and P102.

Survey monitoring points on the bored pile wall were regularly monitored using theodolite and distomat. From these measurements it was confirmed that no precautionary measures were needed since there was no excessive deformation that could have caused instability of the bored pile wall.

More reliable and accurate measurements of the horizontal deformation of the bored pile were obtained by using a vertical digital inclinometer. This equipment consisted of an inclinometer probe, connection cable and read out unit with lap top computer for analysis of results.
Vertical digital inclinometer leading tubes (or access tubes) were installed initially in the bored pile wall at the locations marked in Figure XIV.

The probe was guided through the access tube and measurements were taken at fixed intervals (generally at 0.5 m). At each interval, the inclination with regard to the vertical axis was measured. In order to avoid measuring errors and assure greater measuring accuracy the same procedure was performed with the probe rotated through 180^0.

The output was directly fed into the computer and results obtained. The details of digital vertical inclinometer are given in Figure XV.

The comparison of actual deformation with theoretical deformation of P32 at various excavation stages and 18 weeks after completion of excavation are presented in Figures XVI.

Hydraulic load cells were installed at selected anchors heads for monitoring actual anchor forces that were induced in the anchors. The locations of load cells are presented in Figure XIV. The details of the load cells and arrangement of load cell on the anchor head are shown in Figure XVII.

9.0 CONCLUSIONS

(i) Anchors installed in both soil and rock behaved as expected.
(ii) Actual horizontal deformations of the bored pile wall at some locations were higher than expected but well within the acceptable limits.
(iii) As a whole the performance of the bored pile wall was satisfactory.

10.0 ACKNOWLEDGEMENTS

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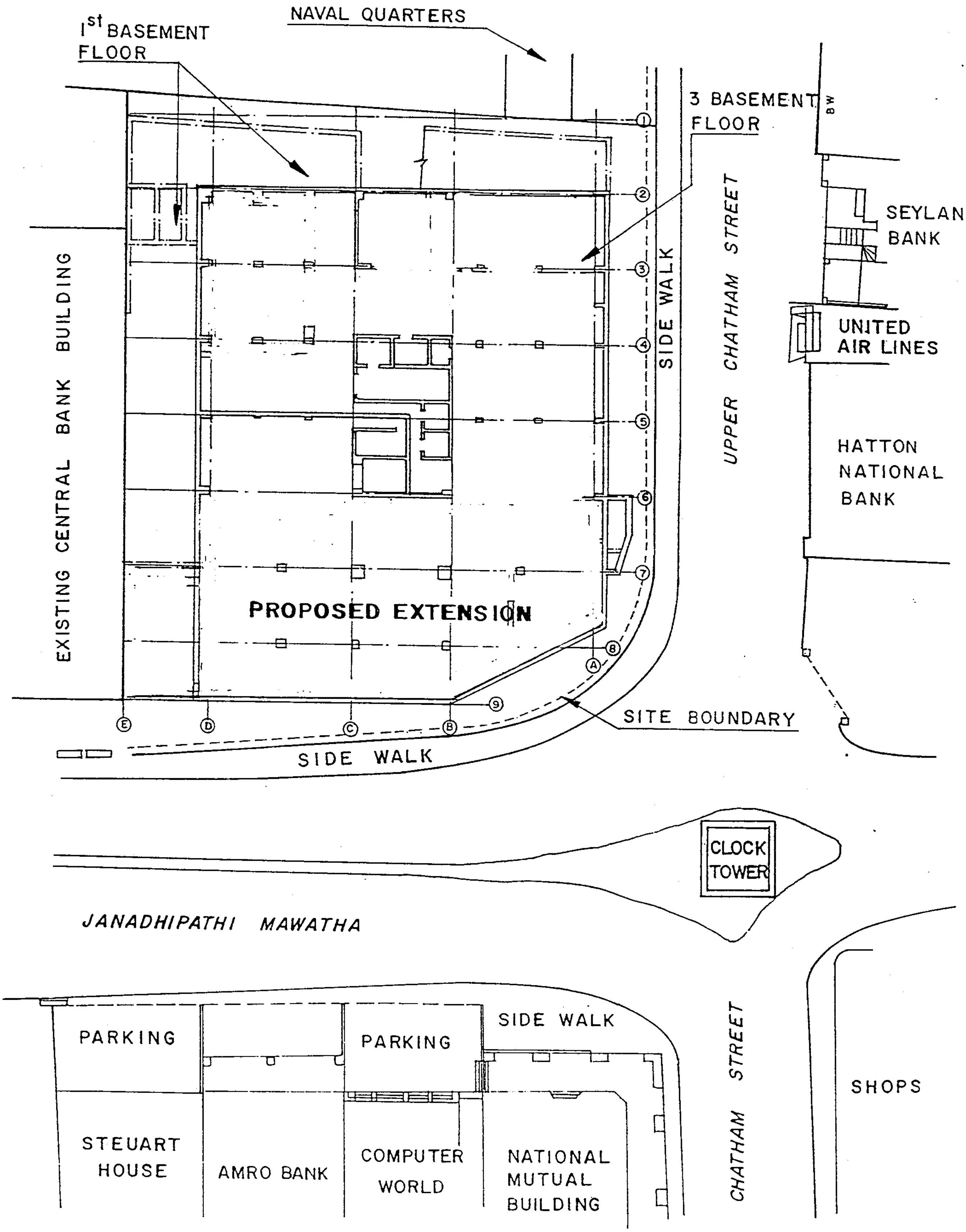
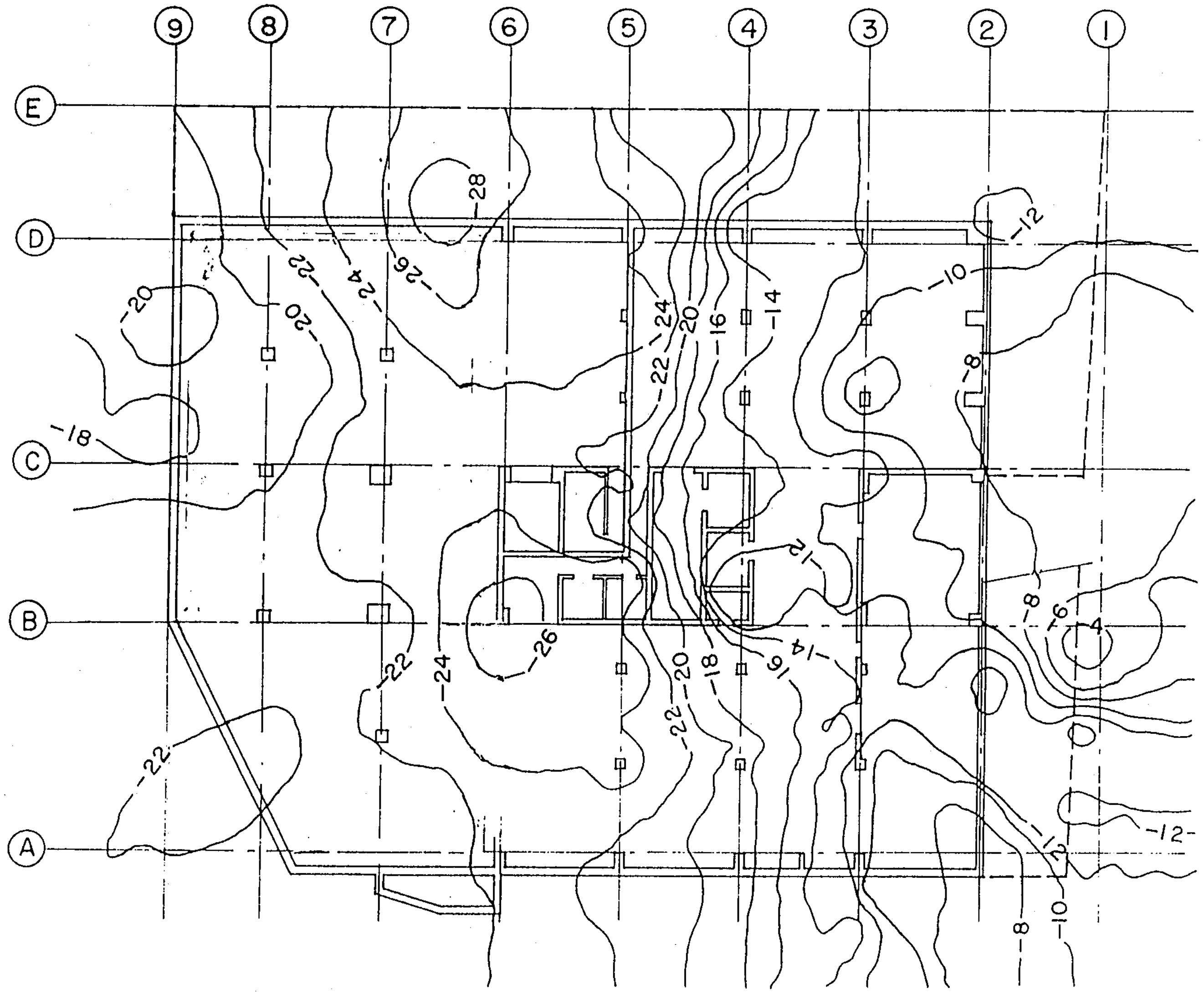
(3) Mr. M.A.C. Perera, Miss Padama Sirimanne, Mrs. Pushpa Karunaratne, Mr. K.H.de S. Talawatte and staff of CECB Central Bank Project for their assistance.

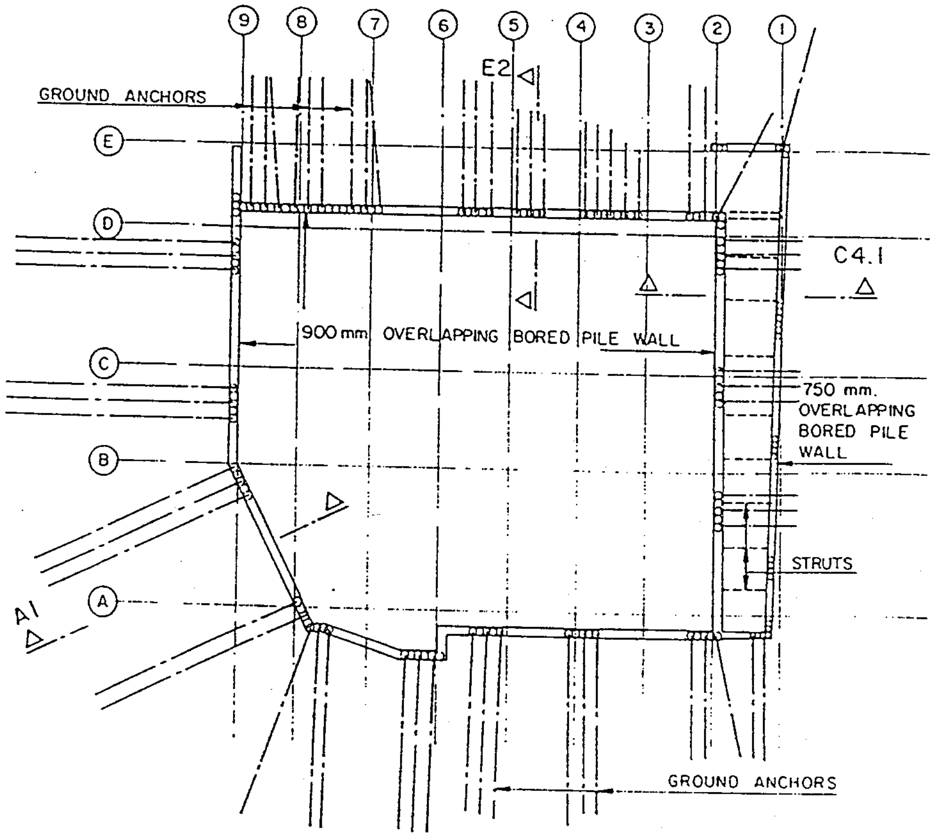


LOCATION PLAN
FIGURE 1

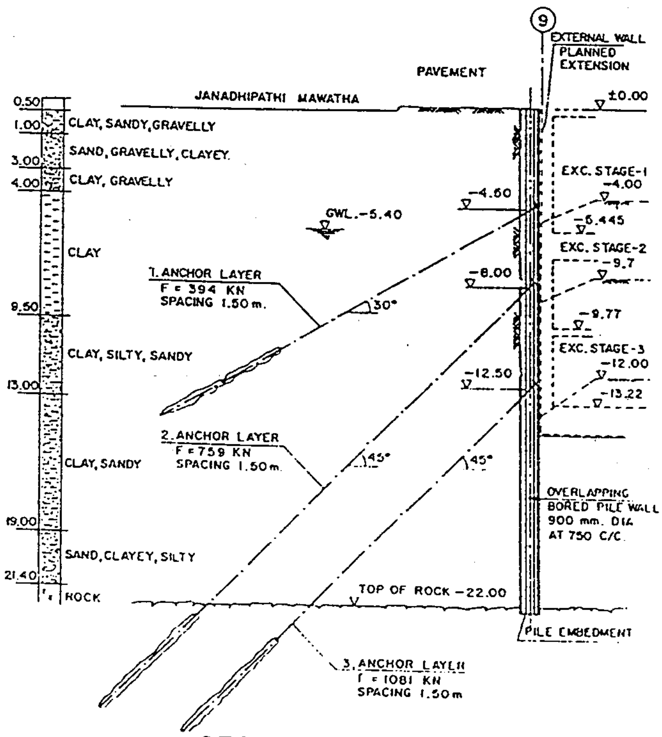


ROCK SURFACE COUNTOUR

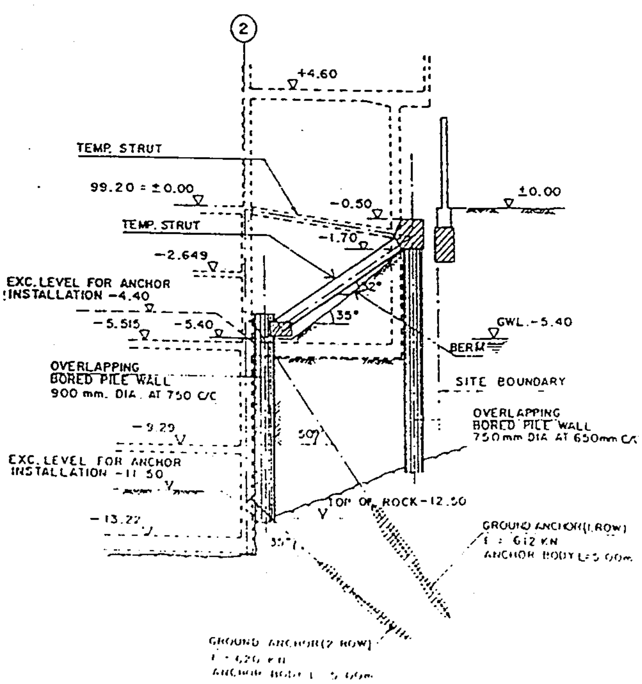
FIGURE II



LAYOUT



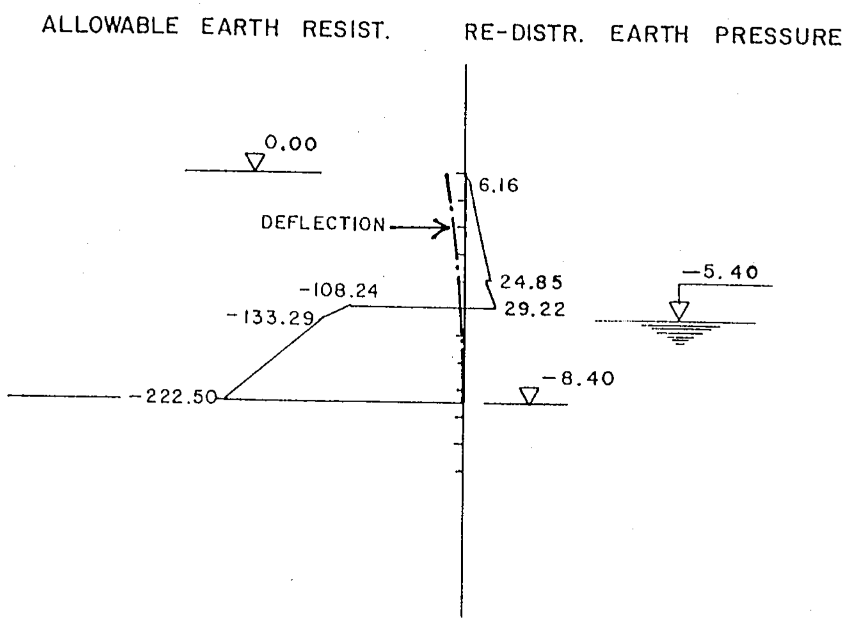
SECTION A1



SECTION C4.1

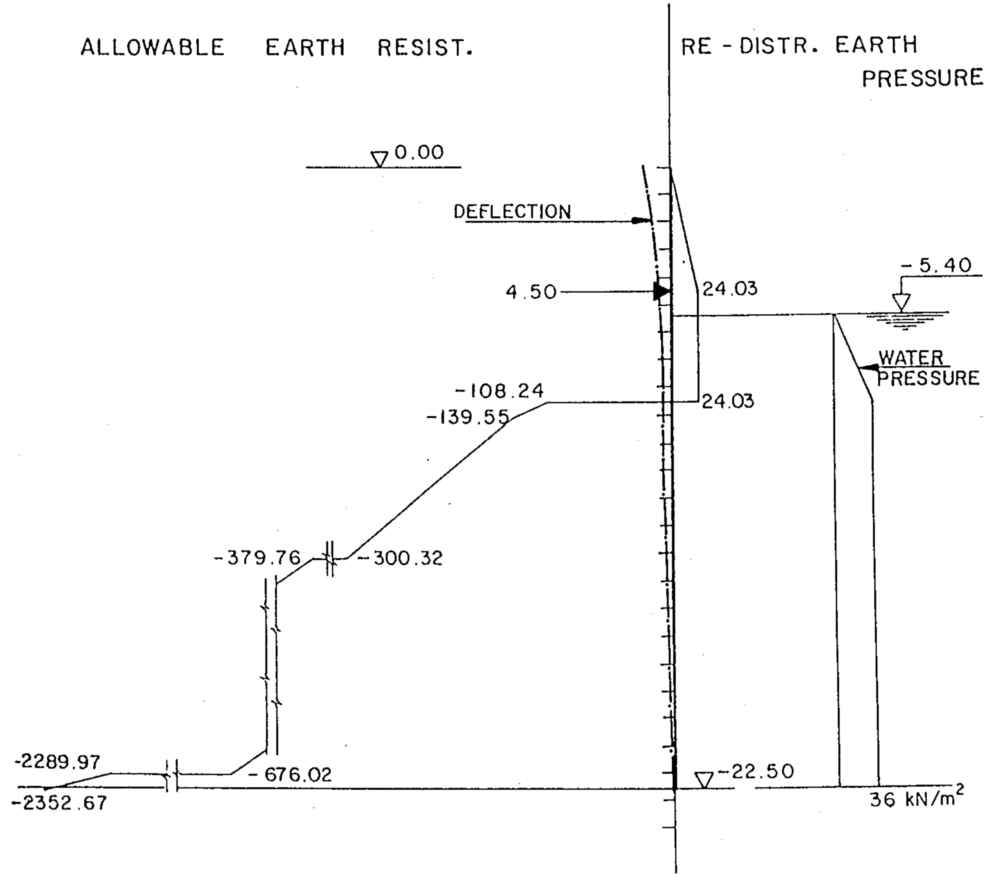
TEMPORARY RETAINING WALL

FIGURE III



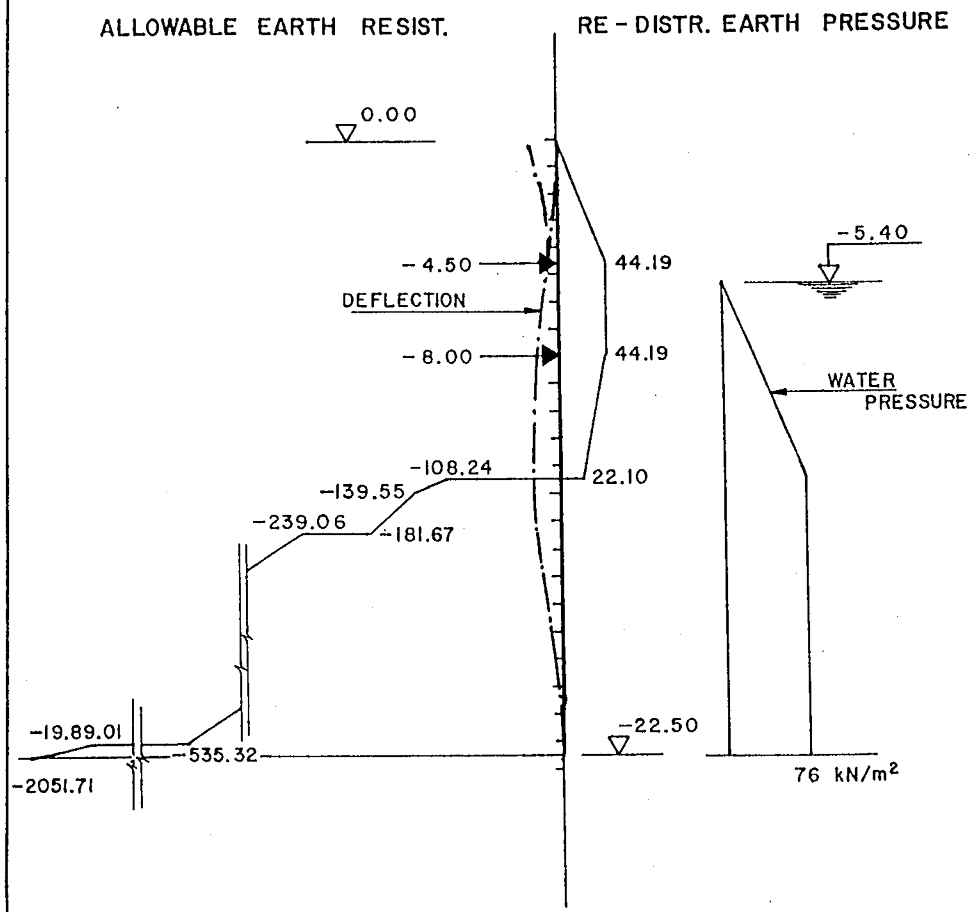
SCALE :– EARTH PRESSURE – 1mm. = 5 kN/m²
DEFLECTION – 1 mm. = 2 mm.

EXCAVATION STAGE - 1
FIGURE IV



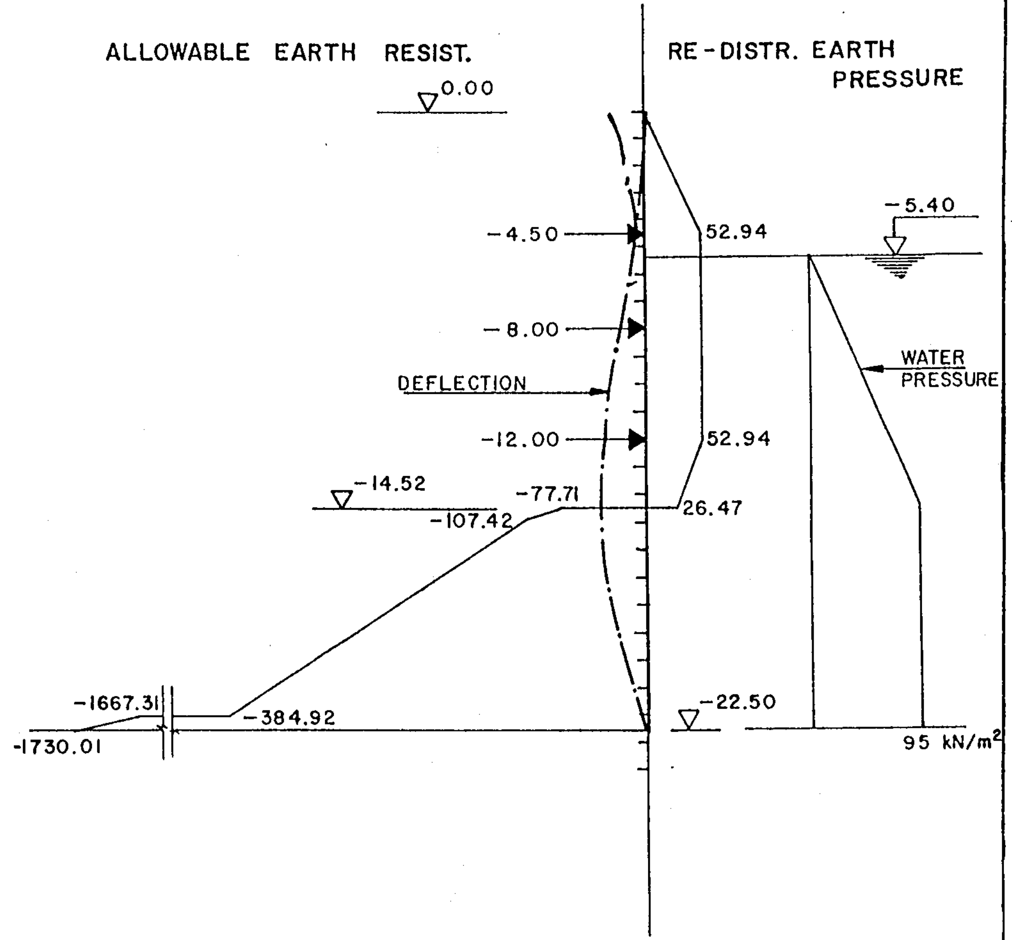
SCALE :- EARTH PRESSURE – 1mm. = 5 kN/m²
DEFLECTION – 1mm. = 2 mm.

EXCAVATION STAGE - 2
FIGURE V



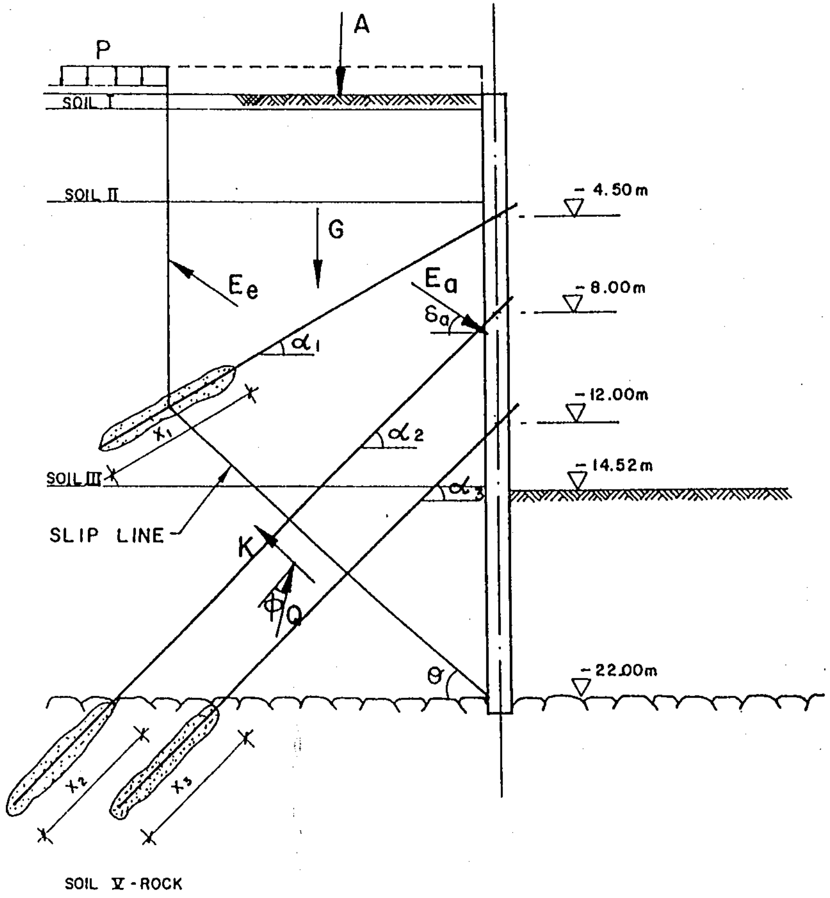
SCALE :— EARTH PRESSURE — 1 mm. = 5 kN/m²
DEFLECTION — 1 mm.= 2 mm.

EXCAVATION STAGE - 3
FIGURE VI



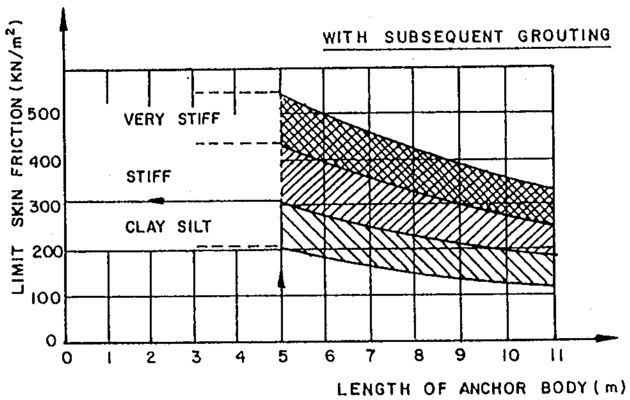
SCALE ;— EARTH PRESSURE —1 mm. = 5 kN/m²
DEFLECTION — 1 mm.= 2 mm.

EXCAVATION STAGE - 4
FIGURE VII



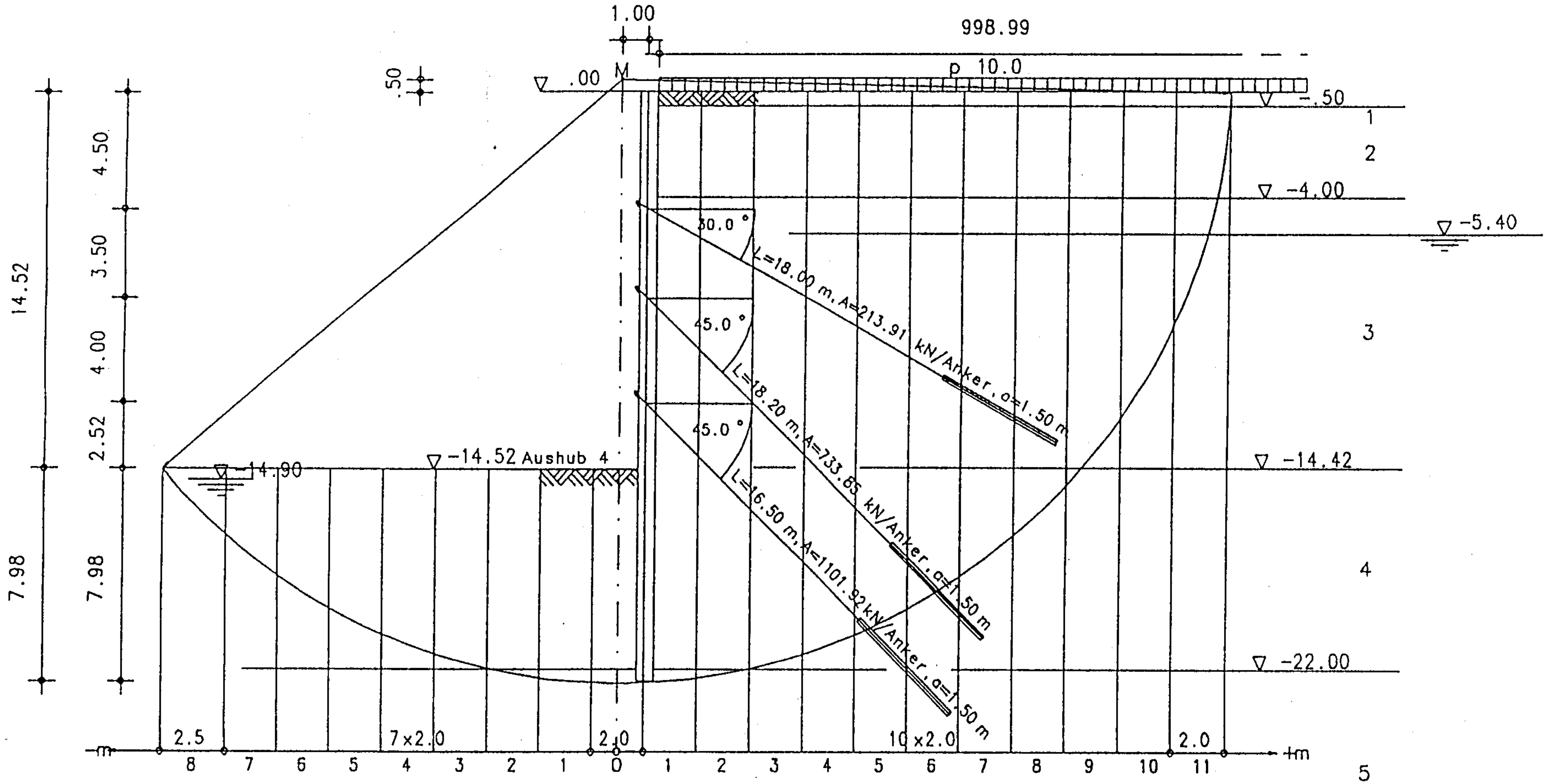
FORCES ACTING ON FAILURE PRISM

FIGURE VIII



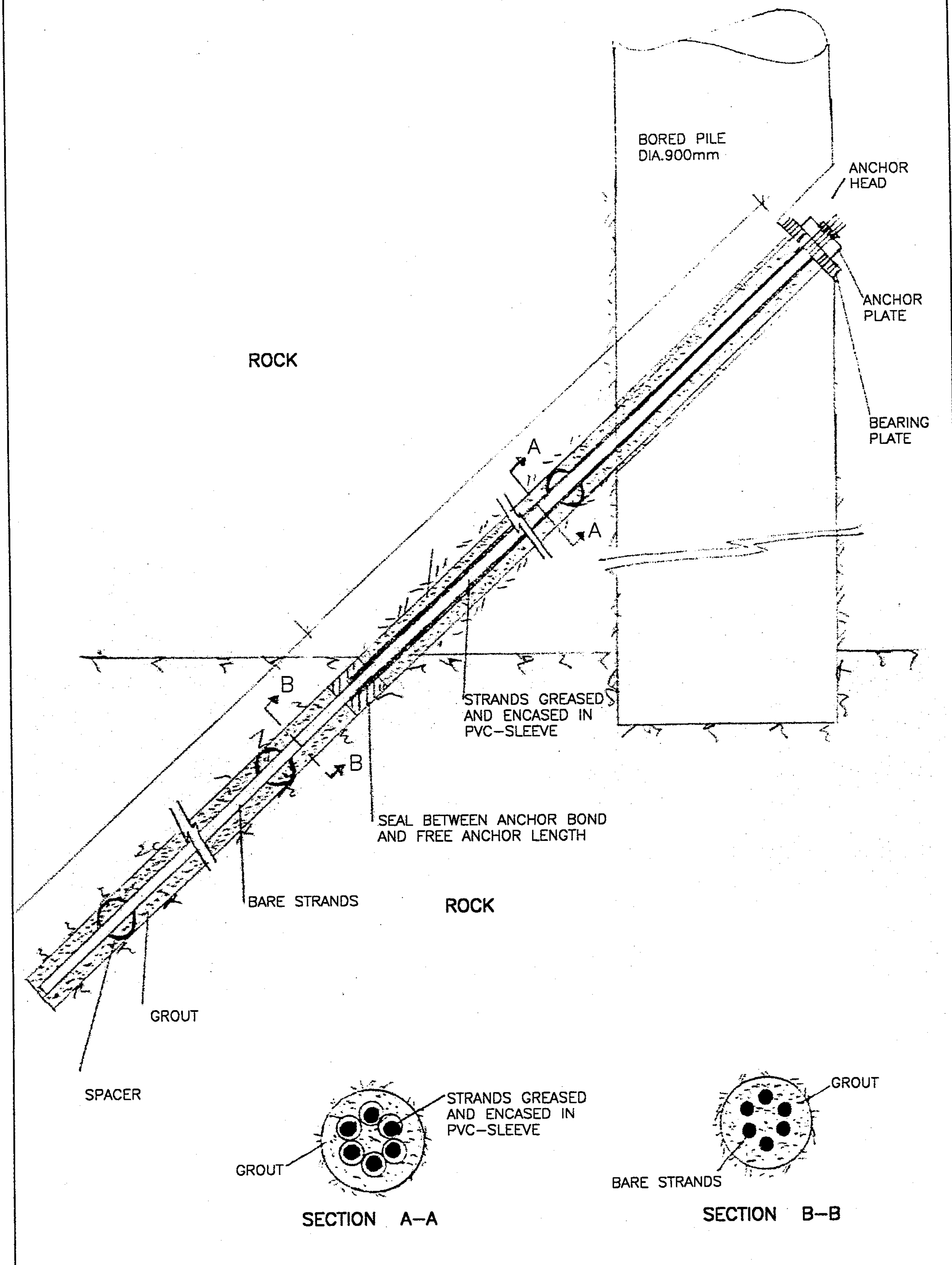
LIMIT LOAD IN COHESIVE SOILS

FIGURE IX



RIB-PROGRAMM BGKOGL 107 BAUGRUBENPLOT
COLOMBO / ROCK - 22m / SECTION A-A / V1-3
Lastfall 1 Aushub 4 M 1:250

FIGURE X



TYPICAL DETAIL OF AN ANCHOR

FIGURE XI

Time: 1530 hrs.

SUITABILITY TEST

anchor strands **temporary anchorage**

drawing no Z/80/ST/PF/1101/f safe working load 396 KN
anchor no 114

Pile no P-29

Fi = 50

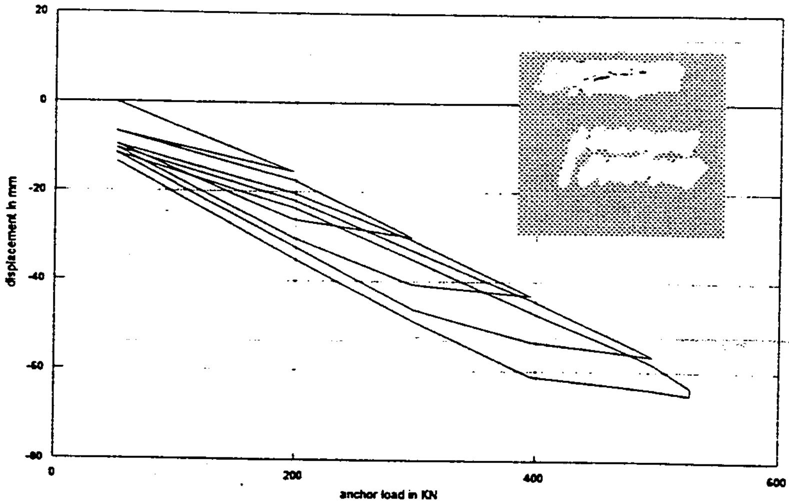
anchor no.			114
anchor type			strands
strand pieces			4
cross section	As	cm2	5,6
steel grade			1570/1770
safe work.loa	Fw	KN	396

safety factor	nk		1,33
proof load	nk*Fw	KN	526,68
anchor length	la	m	18,9
anchor inclination		Grad	30
tendon bond length	lv	m	6

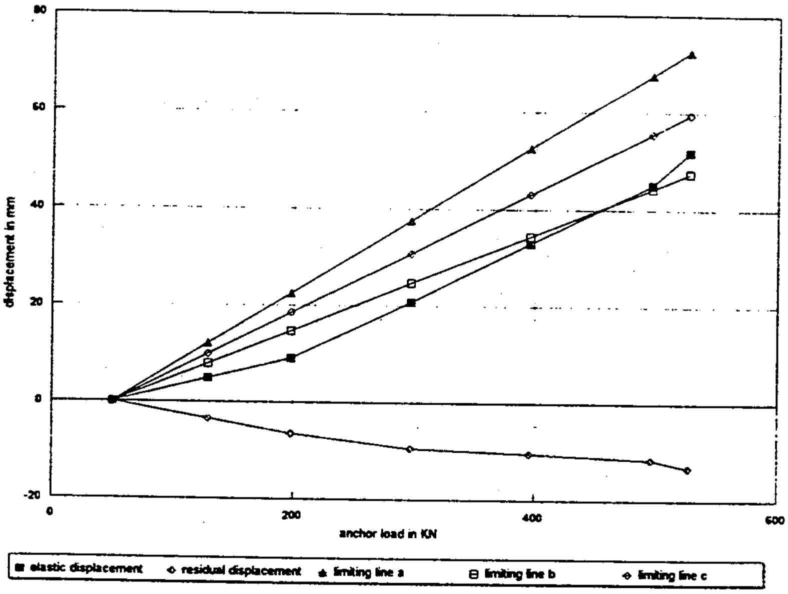
fixed anchor length	lo	m	
compression length	ld	m	
exceeding length	lx	m	0,7
free tendon length	lfs	m	13,6

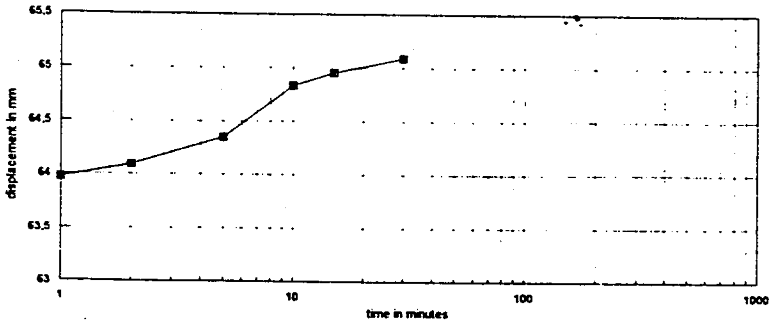
load stage load	load stage exoneratio	force kN	time min	displacement (mm)	creep displ. in mm
Fi		50			
0,5		198		15,5	
0,5		198	1		
0,5		198	2		
0,5		198	5		
	Fi	50		6,55	
0,5		198		17,32	
0,75		297		30,12	
0,75		297	1		
0,75		297	2		
0,75		297	5		
0,75		297	10		
0,75		297	15		
0,75		297	30		
0,75		297	60		
0,75		297	90		
0,75		297	120		
0,75		297	150		
0,75		297	180		
	0,5	198		26,24	
	Fi	50		9,44	
0,5		198		20,27	
0,75		297		30,69	
1		396		43,16	
1		396	1		
1		396	2		
1		396	5		
1		396	10		
1		396	15		
1		396	30		
1		396	60		
1		396	90		
1		396	120		
1		396	150		
1		396	180		
	0,75	297		40,64	
	0,5	198		30,33	
	Fi	50		10,25	
0,5		198		21,84	
0,75		297		33,03	
1		396		43,98	
1,25		495		56,66	
1,25		495	1		
1,25		495	2		
1,25		495	5		
1,25		495	10		
1,25		495	15		
1,25		495	30		
1,25		495	60		
1,25		495	90		
1,25		495	120		
1,25		495	150		
1,25		495	180		
	1	396		53,48	
	0,75	297		46,09	
	0,5	198		32,16	
	Fi	50		11,53	
0,5		198		23,51	
0,75		297		35,1	
1		396		46,72	
1,25		495		58,2	
1,33		526,68		63,34	
1,33		526,68	1	63,97	
1,33		526,68	2	64,09	0,40
1,33		526,68	5	64,35	0,65
1,33		526,68	10	64,83	1,59
1,33		526,68	15	64,95	0,68
1,33		526,68	30	65,09	0,47
1,33		526,68	60		
1,33		526,68	90		
1,33		526,68	120		
1,33		526,68	150		
1,33		526,68	180		
	1,25	495		64	
	1	396		61,11	
	0,75	297		48,72	
	0,5	198		34,96	
	Fi	50		13,27	

Load Displacement Curve



Limiting Lines and Deformation





according to Din 4125 Ausgabe Nov. 1990

Time -displacement curves for determining creep displacement ks at proof load Fp in Suitability Tests.

		granular soil and rock	cohesive soil
short time test	T1 in min	5	5
	T2 in min	15	30
	delta-S=S2-S1	< 0,5	< 0,8
extended time of observation	T1 in min		
	T2 in min	>15	>30
	creep displ. in mm	<2	<2

ACTUAL DISPLACEMENT
T1=5 min
T2=30 min

delta S= 0,74

FIGURE XII.

Record of Anchor Stressing for Acceptance Test

according to DIN 4125 ; Nov.1990

anchor number : 113 a P27

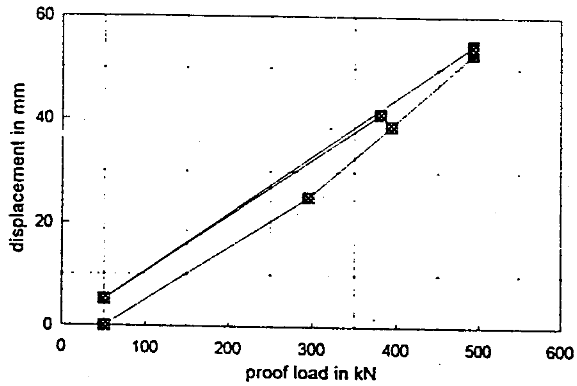
date : 20.07.95

number of strands	4	pieces	anchor length	18,90	[m]	cross section of jack	260	[cm²]
cross section	5,6	[cm²]	exceeding length	0,70	[m]	upper limiting line	67,27	[mm]
steel grade	1570/1770		grout length	6,00	[m]	lower limiting line	44,09	[mm]
E-module	195000	[MN/m²]	free anchor length	13,60	[m]	elastic displacement	49,28	[mm]

safe working load : 394 [kN] prestress load 381 (kN)

anchorage loads / proportion of Fw / minutes	initial load	0,75 Fw	1,0 Fw	proof load 1,25 Fw 1	2	5	10	15	remove-ment	working load
anchor force [kN]	50,0	295,5	394,0	492,5	492,5	492,5	492,5	492,5	50,0	381,0
jack pressure [bar]	19,23	113,65	151,54	189,42	189,42	189,42	189,42	189,42	19,23	146,54
displacement [mm]	0,00	25,05	38,83	52,88	53,85	54,30	54,42	54,45	5,17	41,00

Load - Displacement Diagram

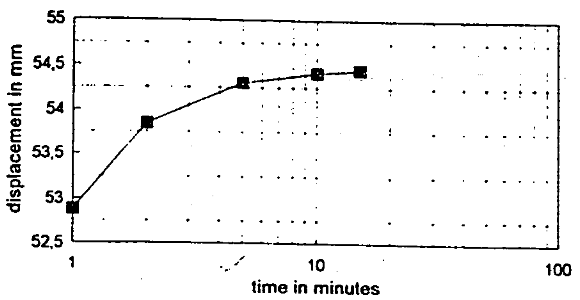


Acceptance Criteria

granular soil / rock			cohesive soil		
S5-S2	criterion	< 0,20 mm	S15-S5	criterion	< 0,25 mm
	result	0,45		result	0,15
	criterion satisfi.			criterion satisfied	
	criterion not satisf.			~~criterion not satisf.~~	
if not satisfied extend the observation period			if not satisfied extend the observation period		
creep displace-ment ks	criterion	< 1,0 mm	creep displace-ment ks	criterion	< 1,0 mm
	result			result	
	criterion satisf.			criterion satisfied	
	criterion not satisf.			criterion not satisf.	
anchor accepted				yes	~~no~~

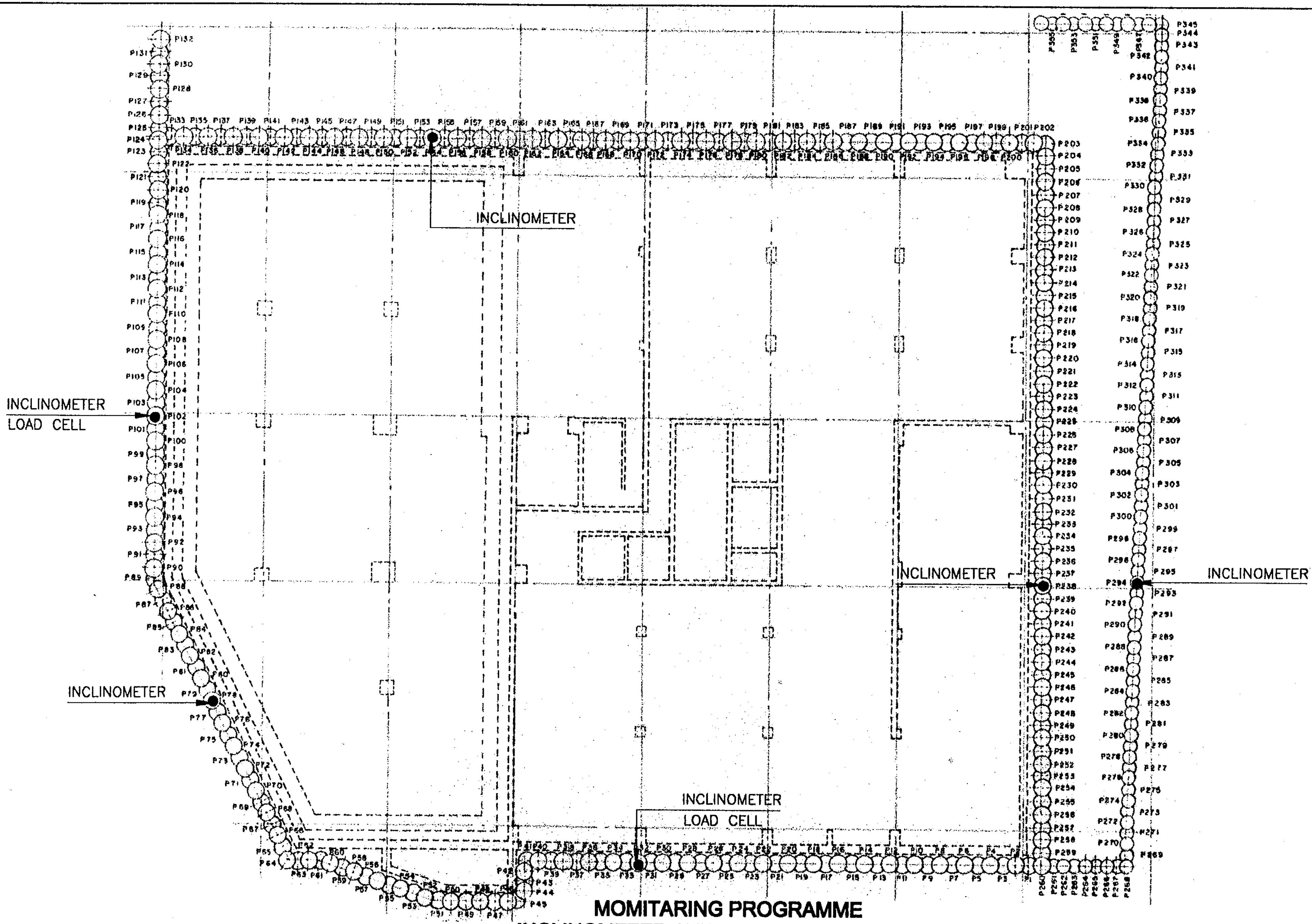
notes :

Time - Displacement Curve for determining creep displacement ks at proof load : ks = (S2-S1) / log (T2/T1)



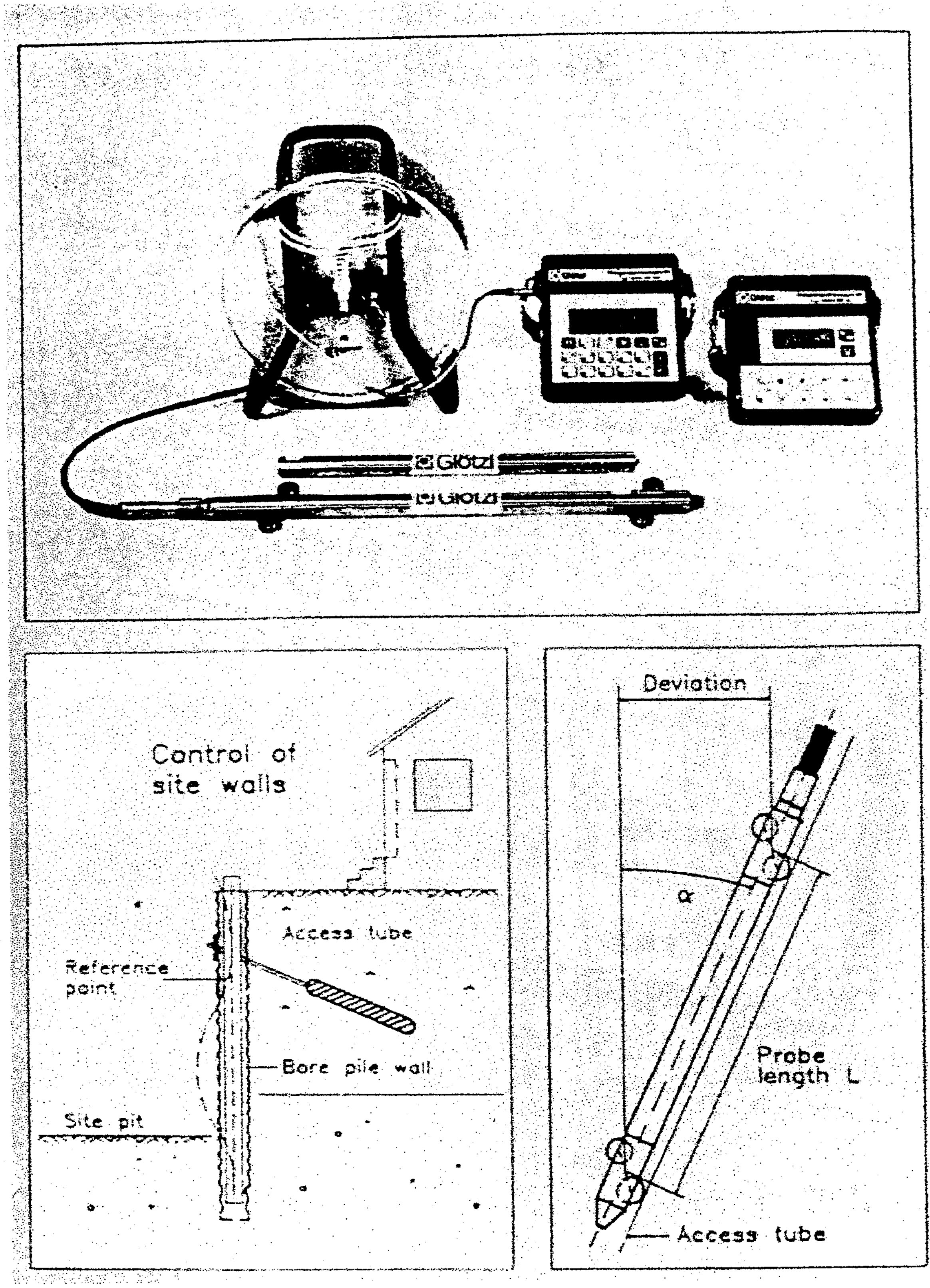
upper limiting line : Sel = [(Fp - Fi) /(E*As)] * [(lfs + lv/2)]
lower limiting line : Sel = [0,8 * [(Fp - Fi) /(E*As)]*lfs]

FIGURE XIII



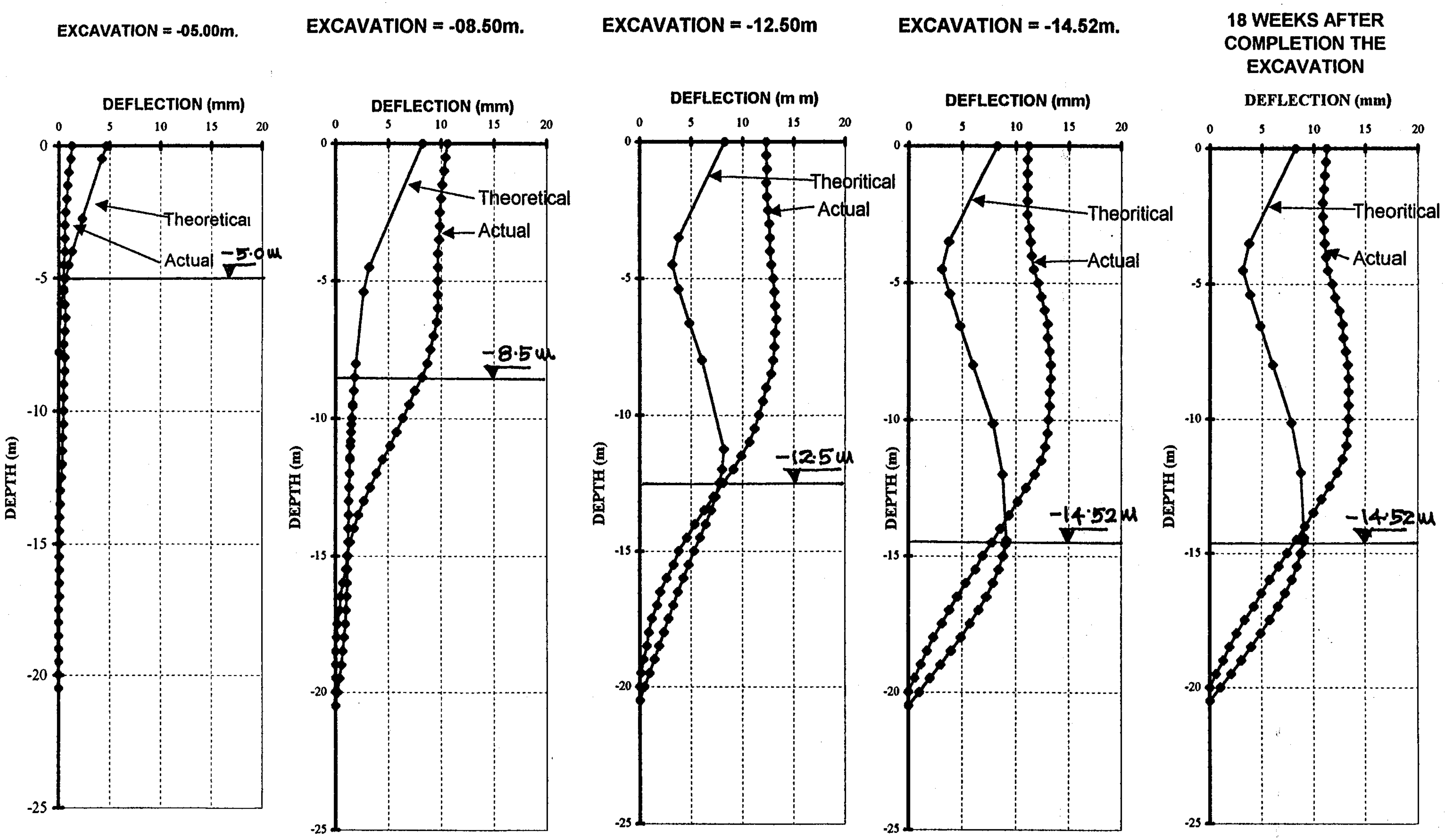
MOMITARING PROGRAMME
INCLINOMETER AND LOAD CELL POSITIONS

FIGURE XIV

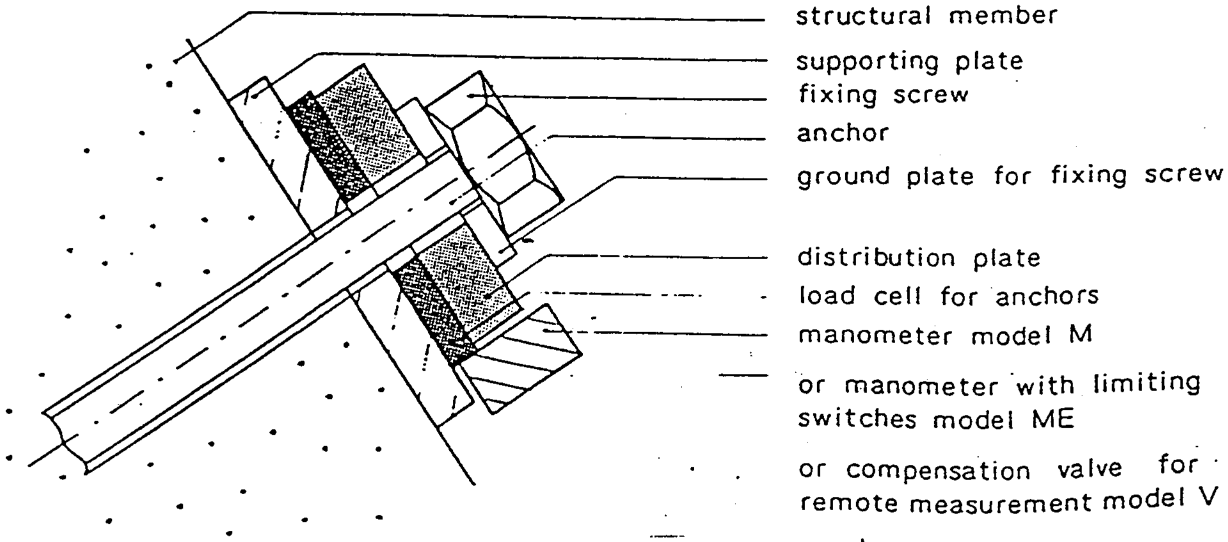


VERTICAL INCLINOMETER

FIGURE XV



COMPARISION OF THEORITICAL AND ACTUAL DEFORMATION
PILE No. P 32
ROCK DEPTH 20m.
FIGURE XVI



INSTALLATION OF THE LOAD CELLS FOR ANCHORS

FIGURE XVII