PROCEEDINGS OF THE SRI LANKAN GEOTECHNICAL SOCIETY SEMINAR ON

EXPERIENCES IN TECHNIQUES OF FOUNDATION TREATMENT AND GROUND IMPROVEMENT

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2nd Floor Lucky Plaza, Colombo 3

SEMINAR ON

Experiences in Techniques of Foundation Treatment and Ground Improvement

VENUE : Auditorium of the Institute for Construction Training Development (ICTAD)

DATE : 12th September 1995 at 9 AM - 5 PM

AGENDA

8.30 - 9.00	Registration
8.30 - 9.00	Inauguration & lighting of lamp
9.05 - 9.15	Welcome Address by President SLGS
9.15 - 9.40	**Ground Improvement Techniques Applicable to Sri Lanka** Dr. S.A.S. Kulathilake
9.40 - 10.00	**Experiences in Heavy Tamping at Madiwela** Mr. W.M. Jayawardhane
10.00 - 10.30	Discussion
10.30 - 11.00	Tea
11.00 - 11.20	**Innovative Ideas and Techniques of Ground Improvement through Geosynthetics** Mr. A.A. Virajh Dias
11.20 - 11.40	**Stage 1 Remedial Measures for Stabilizing Landslide at Beragala on Beragala Haliela Road** Mr. D.P. Mallawaratchie
11.40 - 12.00	**Treatment of Dam Foundations for Stability and Water Tightness** Mr. K.W. Perera
12.00 - 12.20	**Geotechnical Investigations for Yan-Oya - Padaviya Agricultural Extension Project and Proposed Ground Improvements for Dam and Spillway Foundation** Mr. N.M.S.I. Arambepola

12.20 - 13.15	Discussion
13.15 - 14.30	Lunch
14.30 - 15.00	**Construction of Upper Sub-Base with Cement Stabilized Gravelly Soil on Colombo-Galle Road Deviation from Moratuwa to Panadura via Egodauyana** Mr. D.P. Mallawaratchie
15.00 - 15.15	**Foundation in Weak Ground for the Ambatale - Jubilee Water Conveyance System** Mr. K.S.K. Ranasinghe
15.15 - 15.45	Discussion
15.45 - 15.55	Summing up - Dr. Sunil de Silva
15.55 - 16.00	Vote of Thanks - Mr. K.L.S.Selvarajah
16.00 - 16.30	Tea
16.30 - 17.00	Special General Meeting of SLGS

CONTENTS

Ground Improvement Techniques Applicable to Sri Lanka

Dr. S. A S Kulathilaka – University of Moratuwa

1. Introduction

Increasingly civil engineering structures and infrastructure are to be built in areas where existing ground conditions are not very satisfactory. For example in Sri Lanka in and around Colombo most of the good grounds have been already used and new structures are to be erected on weak-soft ground.

In these situations Geotechnical Engineer can either

- accept the limitations imposed by the in situ soil properties and design the foundation accordingly, or
- Can improve the properties of the existing ground to the desired level

If the latter alternative is more economical, naturally that is what should be adopted.

In this process of "ground improvement ", desirable properties of the existing ground such as strength and stiffness, permeability, durability and volume stability are improved. In general, the improvement is brought about by controlling the void ratio of the soil (densification), by introducing a cementing or water proofing agent, or by injecting a substance to fill the pore volume (solidification).

The most appropriate ground improvement technique for a site can be selected giving due consideration to factors such as; site condition, significance of the structure and applied loading and period of construction.

Methods of ground improvement techniques are classified here as;

1. those applicable to cohesive soils,
2. those applicable to cohesionless soils, and
3. other methods

As we have to deal mainly with soft organic clays and Peats in Sri Lanka this paper will concentrate on methods applicable to such soils.

2 Methods Applicable to Cohesive Soils

Methods that can be applied to improve soft - weak cohesive soils are;

1. Replacement,
2. Preloading; methods based on consolidation of the clay by loading;
   - preloading without any special drains,
   - preloading with vertical drains,
3. Methods based on consolidation with no external load ;
   - vacuum consolidation,
   - quick lime pile,
4. Methods based on chemical reactions ;
   - deep mixing method,
   - heat treatment,
   - electro chemical grouting,
5. Methods of mechanical stabilisation forming composite ground;
   - sand compaction pile,
   - stone columns,
6. Dynamic consolidation - Heavy Tamping of the ground

Methods in categories 1, 2, 3 and 6 can be applied in Sri Lankan situations without making big investments for specialised and expensive machinery.

3. Replacement Method

In this method unsuitable soft cohesive soil excavated and replaced with good sand. This can be used only for small scale jobs when soft layer thickness is small. For structures with increasing scale amount of soil to be removed is massive and this method may not be practical. Furthermore, soil to be removed may be highly polluted and there can be drastic environmental effects due to their removal and exposure to the atmosphere. Removal can also have unfavourable influence on existing adjacent structures.

4. Preloading Methods : Consolidation by External Loading

4.1 General

In these methods a load equivalent to or exceeding the design load on the soil (due to the structure) is applied before the construction and left until the required consolidation of the soft clay is achieved. The time period over which the load should be kept is termed "preloading period ". During this time soil will consolidate and will increase it's shear strength. With the removal of the preload soil will become an overconsolidated soil. When soil is loaded again due to the weight of the structure it will behave as an overconsolidated soil and residual settlements due to the structure will be small.

It is usual practice to provide the preload by an earth fill. (Any other construction material can also be used). This filling process is done gradually, or sometimes in stages. Thus the soft soil below will consolidate under the preload gradually and gain strength. Soil underneath should be able to bear the weight of the fill without causing any shear failure. If the preloading fill is not sand or gravel, a drainage layer is necessary at the under surface of the fill. For this purpose a sand or gravel drain or geotextiles may be used.

Preloading results in one or more of the following three effects;

1. Primary consolidation settlement,
2. Secondary consolidation settlement,
3. Increased undrained strength of the soil

Preload may be equal to greater than the design load of the structure. When the preload applied is greater than the design load the excess is termed as " surcharge ". Required " preloading time " can be estimated using conventional consolidation theories. *Settlement* Vs *Time*, curves for the backfill for, preload equal to design load and preload with two different surcharges are depicted in Figure 1. Preload should be kept till it causes a settlement equal to that would be caused by the design load. Let p_f be the design load intensity due to the structure and the surcharge at two instances be p_{s1} and p_{s2}. ($p_{s2} > p_{s1}$). Under the design load of p_f, 100% (or 90%) consolidation will take a time t_f. If a preload equal to the design load is applied on the soil it should be maintained for a time period of t_f. (curve (1)).

If the preload is greater than the design load say $p_f + p_{s1}$, the Time - Settlement behaviour under the preload can be presented by Curve 2. There the final settlement that would occur under the preload would occur only after a time t_{s1}. Thus the preload can be removed after that time and the construction of the structure may be started. With a further high preload p_{s2}, Time Vs Settlement behaviour is given by curve 3 and preload may be removed after time t_{s2}.

During this time void ratio of the underlying soil will decrease from point A to B in the e Vs $log\sigma_{v'}$ plot. When the preload is removed soil will unload and rebounce and the void ratio will increase from point B to point C. With the reloading due to the construction of the building soil will reload from point C to B. It is very clear that the reduction in void ratio from C to B is much smaller than that from A to B. Therefore the settlement due to the construction of the structure had been considerably reduced. Other advantage is that the shear strength of the soil has also been increased.

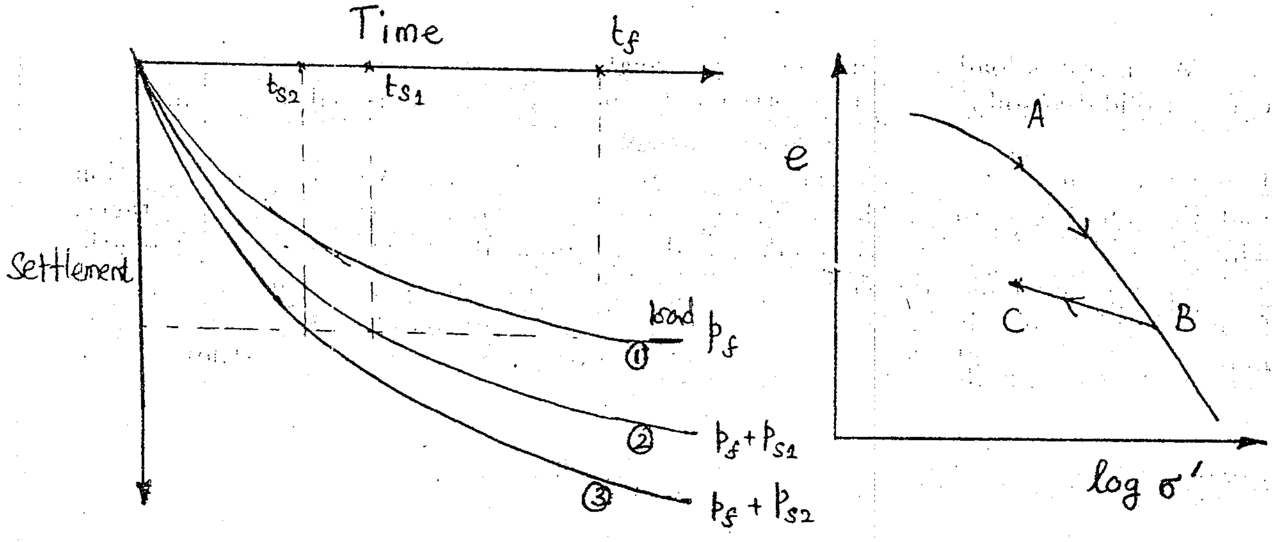


Figure .1: Settlement Vs Time — and e Vs Log p Relationships

Computation of Preloading Time

Let the effective vertical stress in the center of the soft clay layer be σ'_{vo}, the design load of the structure p_f and the surcharge be p_s; The expected primary consolidation settlement due to the design load will be;

$$\delta_f = \frac{H}{1+e_o} C_c \, log\left(\frac{\sigma'_{vo} + p_f}{\sigma'_{vo}}\right)$$

where;
H = Thickness of the soft clay layer
e_o = Natural void ratio of the soil (under σ'_{vo})

The primary consolidation settlement that would occur under the preload with a surcharge is given by ;

$$\delta_{f+s} = \frac{H}{1+e_o} C_c \, log\left(\frac{\sigma'_{vo} + p_f + p_s}{\sigma'_{vo}}\right)$$

If U_{f+s} denotes the degree of consolidation of the soft clay required under the preload of intensity $p_f + p_s$, to produce the 100% consolidation settlement expected under the design load, it can be written that;

$$\begin{aligned} \delta_f &= U_{f+s}\delta_{f+s} \\ U_{f+s} &= \frac{log\left(1 + \frac{p_f}{\sigma'_{vo}}\right)}{log\left[1 + \left(\frac{p_f}{\sigma'_{vo}}\right)\left(1 + \frac{p_s}{p_f}\right)\right]} \end{aligned}$$

Note - Above equations are valid only if the preloading is taking soil along the virgin compression line. (that is when the preloading is most effective).

Having obtained the "degree of consolidation U " required first " Time factor - T" can be found. Then the actual time required can be estimated having considered the drainage conditions.

Monitoring of field behaviour

At the design stage preloading time required under the applied preload can be estimated theoretically based on the laboratory determined consolidation properties of the soil. Nevertheless consolidation behaviour in the field can be quite different. Thus the monitoring of field consolidation behaviour is essential. Settlement gauges can be placed in the fill and peizometers shall be placed in the clay layer to monitor the pore water pressure dissipation. Decision to remove the preload shall be made only after confirmation of the field consolidation behaviour.

Secondary compression caused by preloading

Generally, the primary consolidation settlement predominates in preloading and for many preloading projects it is the only one considered in design.

Secondary consolidation settlement takes place at the same time with the much larger primary settlement. The secondary settlement can be gauged from Figure 2, presented by Bjerrum (1972), for the relationship between void ratio and effective stress.

When the load $p_f + p_s$ is applied to the clay surface, the path " abd " will be followed. Primary consolidation will take it along path "abc" and secondary consolidation along "dc". If the preload consisted only of p_f, path "bf" will be followed during primary consolidation and path fe will be followed during secondary consolidation, and the secondary compression will take a very long time.

This figure illustrates that both the magnitude and the rate of secondary compression under the design load are reduced by the prior application of a preload.

For this to be of practical significance preload must remain in position until well after the excess pore water pressures in the layer have dissipated.

Secondary compression at time t_s at a rate δ_s is given by;

$$\delta_s \quad = \quad C_\alpha H_p log(t_s/t_p)$$

where
H_p = Thickness of the layer at time t_p
C_α = Coefficient of secondary compression

The settlement behaviour of a clay under the loads p_f and $p_f + p_s$ is illustrated in Figure 3. If no primary or secondary compressions are to occur under the design load p_f, the preload $p_f + p_s$ must be sustained for a period t_{SR} such that the total settlement, δ_{SR} is

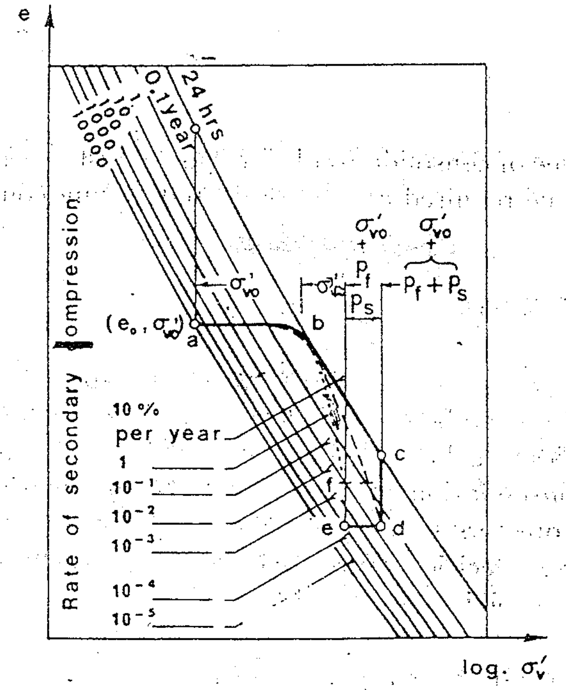


Figure 2: Secondary compression caused by preloading - Bjerrum (1972)

the sum of the primary and secondary settlement that would have occurred under the load p_f alone. i. e.

$$\begin{aligned}
\delta_{SR} &= \delta_f + \delta_{SC} \\
\delta_f &= \frac{2H}{1+e_o} C_c log \frac{\sigma'_{vo} + p_f}{\sigma'_{vo}} \\
\delta_{SC} &= (H - \delta_f) C_\alpha log \frac{t_s}{t_p}
\end{aligned}$$

The desired settlement δ_{SR} can then be related to the primary settlement under the load $p_f + p_s$ by;

$$\delta_{SR} = U_{f+s}\delta_{f+s}$$

substituting we get;

$$U_{f+s} = \frac{\left(1 - C_\alpha log\frac{t_s}{t_p}\right) log\left(1 + \frac{p_f}{\sigma'_{vo}}\right) + \frac{C_\alpha}{C_c}(1+e_o) log\frac{t_s}{t_p}}{log\left[1 + \left(\frac{p_f}{\sigma'_{vo}}\right)\left(1 + \frac{p_s}{p_f}\right)\right]}$$

The preload time can then be readily calculated from U_{f+s}. The secondary consolidation time t_s is selected in relation to the life of the structure or as the time required to give a desired magnitude of secondary compression compensation.

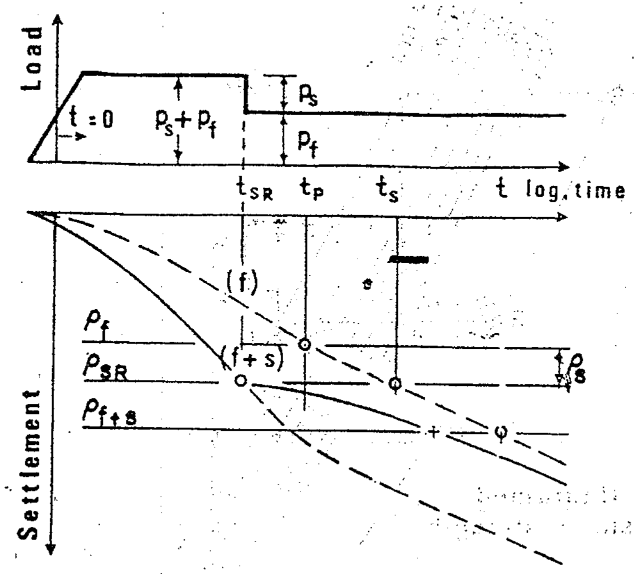


Figure 3: Determination of preload time necessary to eliminate secondary compression under design load

Undrained strength gain by preloading

The compression which occur under preloading results in increase in increase in the undrained shear strength of the clay. The manner in which primary and secondary consolidation contribute to this strength gain is illustrated in Figure 4. It shows e Vs $log\sigma'_v$ relationship for a clay together with the relationship between undrained shear strength, s_u, and the effective stress, σ'_v. The effect of loading the surface of the clay can be summarised with reference to this figure, thus;

1. If the loading p_f (or $p_f + p_s$) does not exceed the pre-consolidation pressure p_c (i. e. If path ab is followed) no strength gain is expected. This is because the void ratio has not changed appreciably.
2. Primary consolidation along path bc results in strength gain from B to C.
3. Secondary consolidation along path cd results in a strength increase from C to D.

Strength gain due to primary consolidation is generally larger than that due to secondary consolidation.

The prediction of strength increases resulting from consolidation are generally based on field established relationships between " Plasticity Index PI " and the ratio s_u/σ'_{vo} or on undrained strength envelops obtained from consolidated undrained triaxial tests. A general indication as obtained by triaxial tests is given by $\Delta s_u = 0.2\Delta\sigma'_v$.

Some experiences in strength gains are;

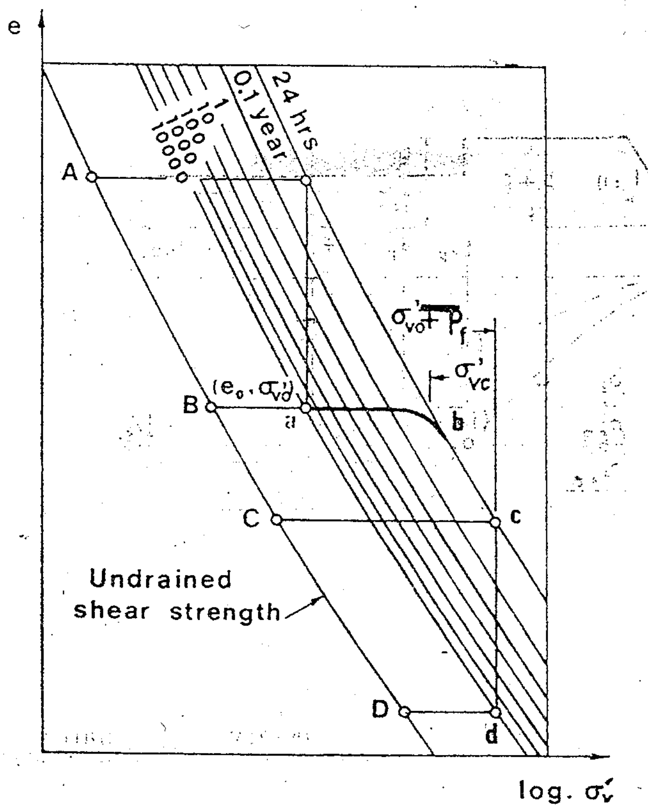


Figure .4: Gain in undrained shear strength due to preloading (Bjerrum 1972)

- Rico et al (1969) - A shear strength s_u increase from 15 - 23 kPa, over 30 months.
- Lambe (1972) - A 100% shear strength increase of a 7 m thick soft clay due to a 230 kPa load,
- Bouges et al (1973) - A shear strength increase from 30 -50 kPa, on a 24m thick soft silty clay in Palavas France, after increasing the stress from from 75 - 130 kPa, over 26 months.

4. 2 Preloading with Vertical Drains

4.2.1 General

The major problem associated with the preloading projects is the "Preloading time". Provision of an additional surcharge to reduce the preload time can be costly and also can cause instability. In such instances vertical drains may be used to reduce the preloading time required and possible surcharge needed. Considering various consolidation theories it can be said that;

1. Time of one dimensional consolidation is a function of the square of the thickness of the compressible layer "H".
2. Time of radial consolidation is a function of the square of the drain spacing "s "

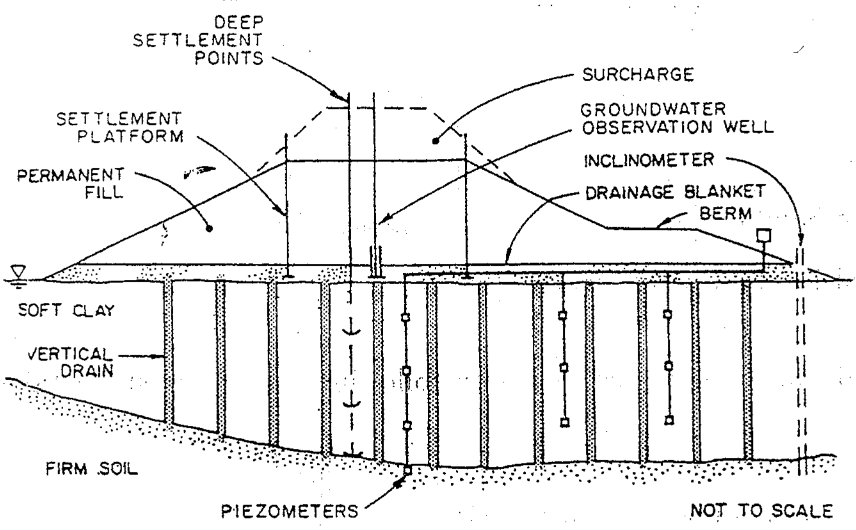


Figure 5: Vertical drain application

3. Degree of consolidation U_{rz} of a combined process of 1 D vertical consolidation and radial consolidation is given by;

$$U_{rz} = 1 - (1 - U_z)(1 - U_r)$$

where U_z and U_r are the respective degrees of vertical and radial consolidation at a time.

Note -

$$U_z = 1 - \frac{2}{M^2} e^{-M^2 T_v}$$

$$U_r = 1 - e^{\frac{-8T_r}{F(n)}}$$

Use of vertical drains reduces the drainage path and also mobilizes horizontal permeability which is usually much grater than vertical permeability. Vertical drains can also be used in conjunction with dynamic consolidation.

Vertical drains do not change the increase in undrained shear strength due to consolidation. But as the rate of consolidation is increased strength gain will also be quicker. Also vertical drains have no direct effect on the rate of secondary compression. But due to the early completion of the primary compression, significant amount of secondary compression can occur early.

Although the concept of vertical drains was developed more than a century ago, the method was not applied until 1930's. Since that time however, the method had been widely used and great advances were made in the way in which the concept was applied to solve practical problems. The abundance of literature on vertical drains is a reflection of the widespread use of the technique.

Different types of vertical drains are;

1. Sand drains,
2. Prefabricated Sand drains,
3. Prefabricated Plastic (or cardboard) drains,

In selecting a drain for a particular situation following factors must be taken in to account;

1. Comparative costs of the drain installation,
2. Smear and distortion of the drain walls - this will reduce the drain permeability,
3. Disturbance and lateral deformation of the soft ground due to the drain installation - this will reduce the permeability, coefficient of consolidation and undrained strength of the soil.

4.2.2 Sand Drains

Sand drains have been widely used since around 1925. In this method vertical holes are made through the thickness of the soft clay layer and hole is filled with clean filter sand. The range of diameter is from 200 - 500 mm and have been installed to large depth using different procedures. The most common installation methods are "closed mandrel" and "open mandrel" methods.

Close mandrels consists of steel tubes closed at the lower end by a loose cap. They were driven by percussion or vibration or some other suitable means and then filled with sand. Thereafter the tube is extracted. This is a simple and cheap method and is very popular. However its major drawback is the disturbance caused in the in situ soft soil by the displacement in both vertical and horizontal directions during the installation.

Open mandrels (whose bottom end is open) are also driven into the soil by a similar procedure. The soil inside the tube is later removed by jetting or by auguring. Thereafter sand is placed in the tube and the tube is withdrawn. Disturbance caused during the installation process is significantly less than in the closed mandrel method. But the problem of smear around the circumference still exists.

Some extensive investigations have taken place about the performance of sand drains. The installation method to be adopted largely depend on the local conditions and experience.

In the site drains are located in a square grid or triangular grid as illustrated in Figure 6. Area serviced by a sand drain can be closely represented by a circle of an area equal to that contained by the hexagon as shown in Figure 6 (b).

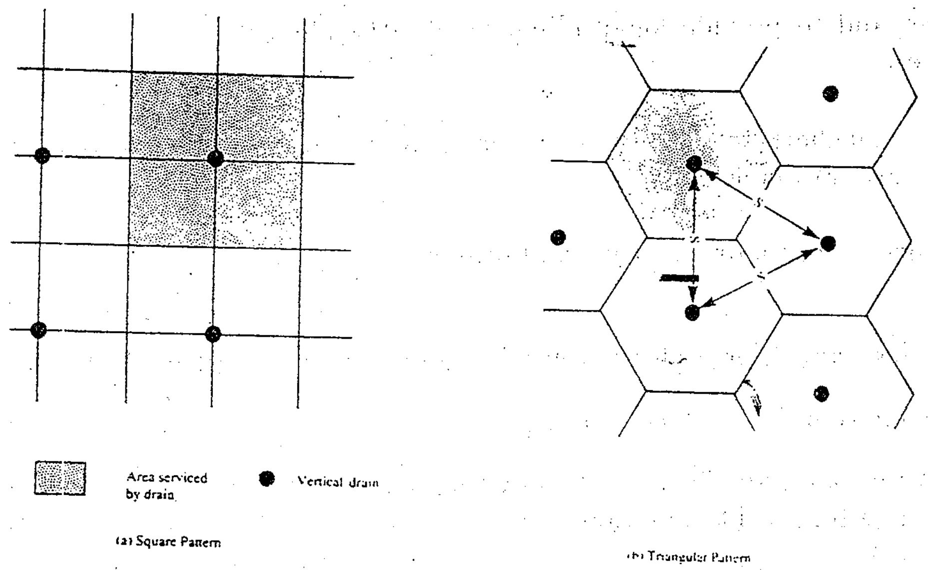


Figure 6: Layout of Vertical drains

4.2.3 Prefabricated Sand Wick Drains

Prefabricated sand drains were first used by Dasidar et al (1969) in India. They are made up of fabric stockings pneumatically filled with sand. They are usually installed by the closed mandrel technique of sand drain installation.

Two main brands were known, namely Sand wick drains and Fabridrains.

4.2.4 Prefabricated Plastic drains

Sand drains are now been superseded by prefabricated plastic drains due to many advantages of the latter type.

Oldest prefabricated drain was due to Kjellman(1939) and was made up of cardboard. At present over 50 different brands of plastic prefabricated drains are used in practice.

A prefabricated vertical drain can be defined as any prefabricated material or product consisting of synthetic, filter jacket surrounding a plastic core having following characteristics.

1. Ability to permit porewater in the soil to seep into the drain
2. A means by which the collected porewater can be transmitted along the length of the drain.

The jacket material consists of non-woven polyester or polypropylene geotextiles or synthetic paper that functions as physical barrier separating the flow channel from the surrounding soft soil and acts as a filter to limit the passage of fine particles into the core to prevent clogging. The plastic core serves two vital functions, namely; to support the filter jacket and to provide longitudinal flow paths along the drain even at large lateral pressures.

Figure 7 gives some characteristics of nine types of plastic drains out of a total of about 50 brands currently available.

Main advantages of plastic drains over sand drains are;

- They are less expensive, lighter in weight and more consistent in quality.
- Their installation does not require water or sand
- They have no adverse effect on the environment and the drainage blanket stays relatively free from contamination.
- they have a long working life period and can sustain large deformations. They will not shear or breakdown during precompression.
- They can be used to depths greater than 50m.

Drain Installation

The normal sequence of operations is as follows.

- Top soil is removed and a free draining blanket of 0.6-1.2m thickness is placed. This will also acts as a working platform.
- Mobilise the right type of equipment.
  Conventional cranes with high booms can be used. However specific equipment, sometimes known as stitches, may be more efficient. Choose suitable driving equipment and mandrel.
- Place drain rolls on reels and insert the plastic band into the mandrel- fix a disposable end shoe at the toe of the plastic band.
- Drive each drain vertically to the required depth, following the design pattern. Static pressure or vibration auguring may be required in compact fill.
- Pull up the mandrel and cut off prefabricated drain about 150mm above working platform elevation

Typical production rate is about 100 - 600 linear meters of plastic drains per hour. With sand drains rate is about 10-30m/hr.

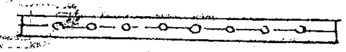
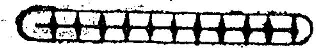
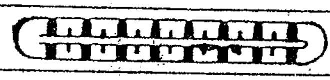
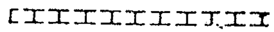
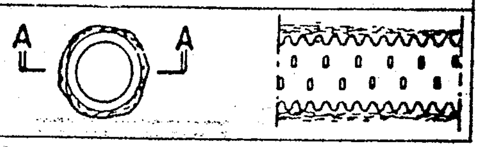
BRAND	DIMENSION (mm)	MATERIAL		CROSS SECTION
		CORE	FILTER	
BAND SHAPED DRAINS				
KJELLMAN	100 x 4	CARDBOARD		
GEODRAIN	100 x 4	polyethylene LD	fiber cellulose or polyester	
ALIDRAIN	100 x 6	plastic	non woven polyester	
COLBOND	Var. x 4	polyester fibers	non woven polyester	
ROPLAST	100 x 3	celluloid	non woven plastic fibers	
MEBRA-DRAIN	100 x 3	polypropylene	typar	
DESOL	95 x 3	perforated polyolefine		
P.V.C.	100 x 1.5	microporous P.V.C.		
PIPE DRAINS				
SOIL DRAIN	50-200 O.D	polyester	felt	

Figure 7: Characteristics of Some Plastic Drains

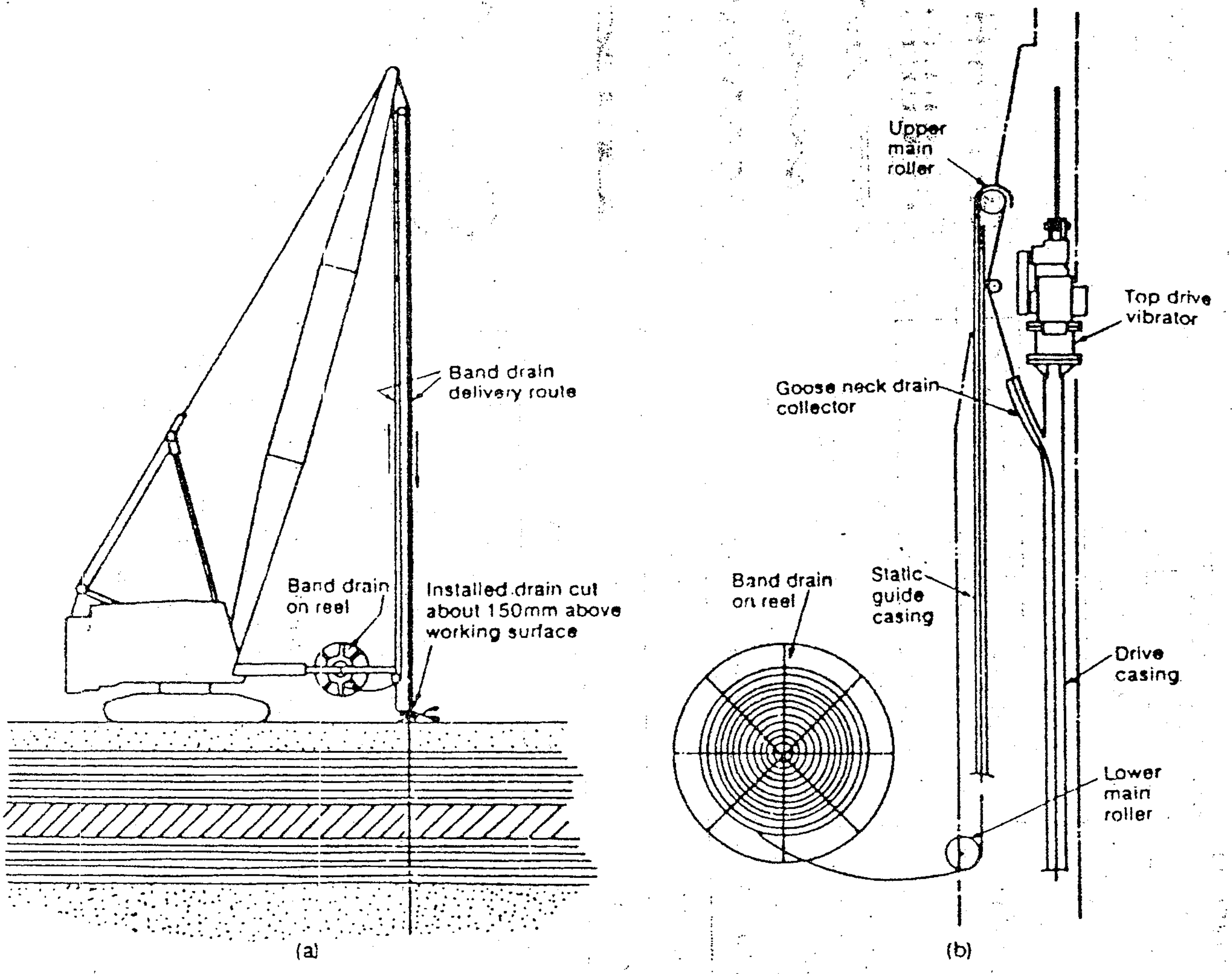


Figure 8: General View of Plastic Drain Installation (McGown and Huges)

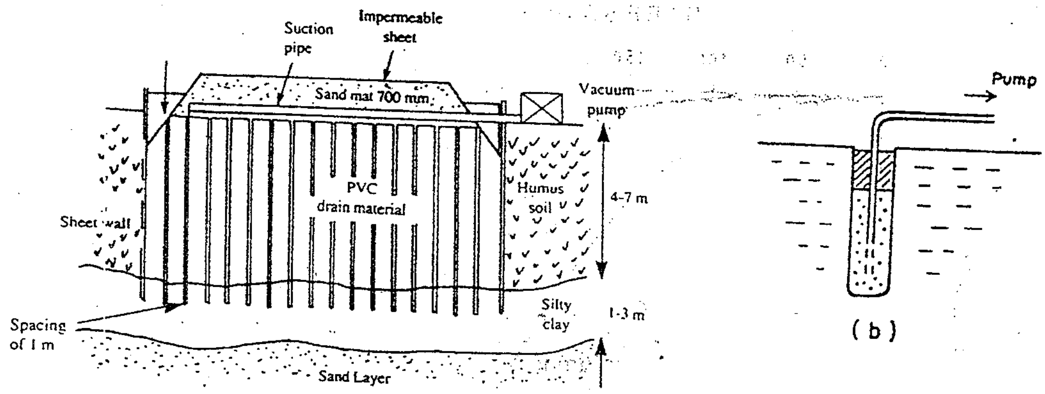


Figure 9: Vacuum consolidation

5. Preloading Methods : Consolidation Without external Loading

5.1 Vacuum Preloading

This is also known as vacuum dewatering and vacuum consolidation. Principle of improvement is consolidation of the soft clay as in the preloading. This technique can be used either with or without vertical drains.

In the case of vacuum preloading an external preload is not applied. Soft clay layer is covered with an impermeable membrane over the ground surface. Membrane is properly sealed along the perimeter of the area to be treated. Some times a sand layer of about 100mm thickness may be spread over the ground surface before covering it with the membrane. Thereafter pressure between the ground surface and the membrane is lowered to near vacuum level. With this hydrostatic pore water pressure is reduced by the same amount as the atmospheric pressure in the long run. As the total stress is unchanged this will result in an increase in the effective stress.

Vacuum preloading is often used in combination with with vertical drains to accelerate the consolidation. A typical layout for vacuum consolidation is shown in Figure 9.

Advantages of the method are;

1. It does not require fill material - So it is ideal when fill material cannot be found economically.
2. Installation and the removal of the preload is easy.
3. It will not cause shear failures in the soft soil. Otherwise when loaded with a fill one has to make sure that, the preload due to the fill is low enough not to cause slope failures or plastic flow in the soil.

The major difficulty with the method was due to the problems of membrane strength strength and durability. Most of these problems are now overcomed.

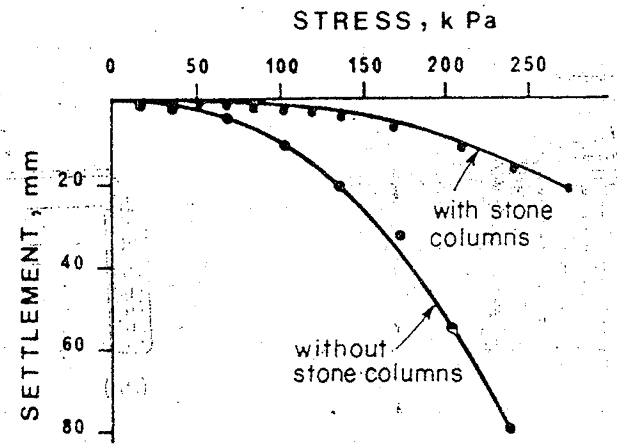


Figure 10: Effectiveness of Stone Columns - Engelhardt et al 1974

An alternative to use of a surface membrane is the provision of a series of wells to which the vacuum is applied. (Figure 9 (b)). The vacuum well method can be carried out concurrently with pumping from deep wells that penetrate any available, more permeable layers. In spite of obvious advantages of vacuum preloading it is not frequently used till recently. One well known case is - in Japan Australian Trade pavilion at Osaka at Hazawa railway station, 4 - 7m layer of humas and silty clay was stabilised using this method. PVC plastic wicks at 1m spacing were used as drains. There was strength gain between 50 - 100 %.

6.0 Methods of Mechanical Stabilisation Forming Composite Ground

6.1 Stone columns

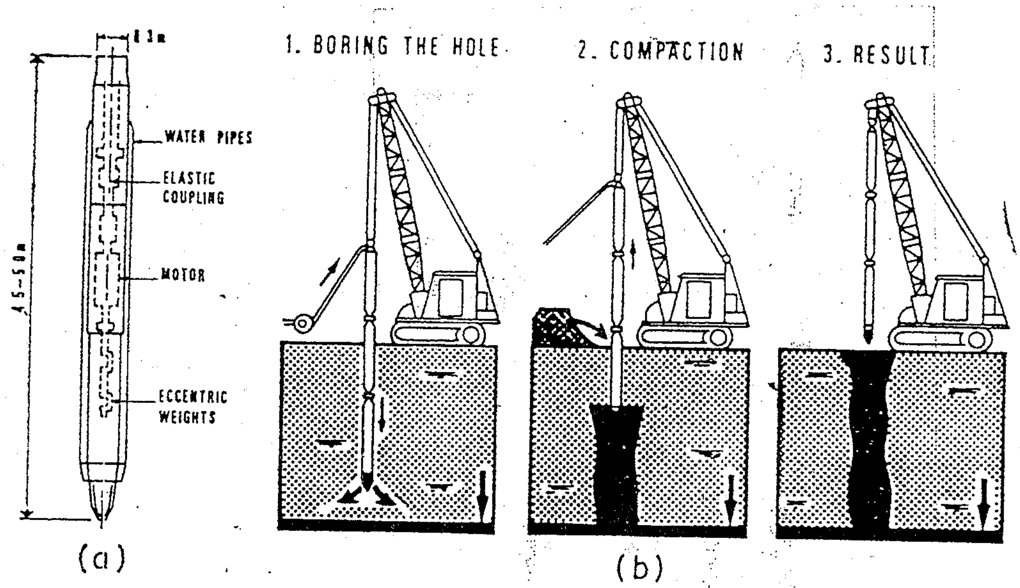
This is a recent method developed in 1960's. This method consists of forming vertical holes in the ground and filling them with crushed rock to form columns or piles confined by the soil. These columns fulfil two functions

1. Provide strength reinforcement to the soil,
2. Act as vertical drains enabling consolidation to occur under surface loading.

Because of the relatively high modulus of the columns, a large proportion of the vertical load applied to the ground is taken up by the columns. Beneficial effects of stone columns have been demonstrated by Engerlhardt et al (1974) (Figure 10)

Construction of the stone columns is carried out in two stages;

1. Borehole is drilled to the required depth using a vibratory drive element - Vibroflot or a torpedo, (Consists of a pointed cylinder 2 - 5m long and 0.3 - 0.4 m diameter)
2. The vibratory element is removed and the hole is filled with crushed rock.



Installation equipment and procedure for stone column construction: (a) torpedo for hole formation, (b) column construction.

Figure 11: Installation of Stone Columns

Process is illustrated in Figure 11.

Factors governing stone column behaviour are;

1. Undrained shear strength of the soil,
2. In situ lateral stress of the soil,
3. Radial stress - strain characteristics of the soil,
4. Initial column dimensions, and
5. Stress - strain characteristics and angle of internal friction ϕ' of the column material

It can be seen that the stone columns are generally only applied in thick deposits of soils of very low shear strength. Columns themselves do not penetrate the layer of soft soil. Column diameters vary between only 0.7 -0.9 m and the density of installation vary between one per 1.1 m^2 to one per 5.7 m^2.

6.2 Cased Borehole Method

In this method piles are constructed by ramming granular material in the prebored holes in stages using a heavy falling weight. (Usually 15-20kN) from a height of 1.0-2.0 m (Dalye and Nagaraja 1975, Bergado et al 1984). This method is useful in developing countries as it does not require any special equipment. However disturbance and subsequent remoulding by the ramming operations may limit its applicability to sensitive clays. Installation process is illustrated in Figure 12.

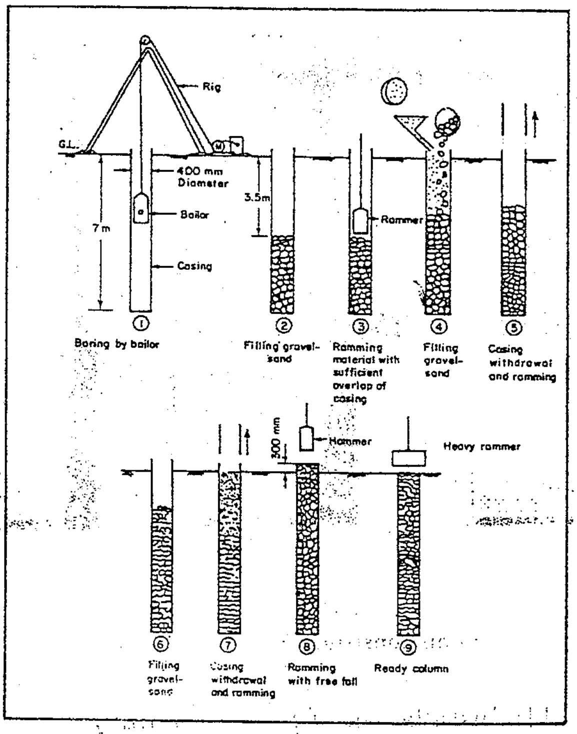


Figure .12: Cased Borehole method - Datye and Nagaraju 1985

7. Ground Improvement by Heavy Tamping

In this method ground is improved by the application of very high blows to the ground surface by dropping of a heavy rammer. This method had initially being applied only to granular soils, but it has been used recently on silts soft clays and on peat. Like other methods of ground improvement it increases the shear strength of the soil and reduces the settlement under the structures.

First a layer of granular material is spread over the ground surface for a thickness of about 1- 2 m. The rammer (weight - pounder) is then lifted and dropped by a crane. Several blows are applied at one spot. A regular pattern of indentation is achieved. On completion of the initial compaction pattern, the surface of the fill is levelled, and second stage of compaction is applied. This second stage is applied after waiting for a sufficient period for the dissipation of the excess pore pressures generated by the first stage. If necessary further stages are carried out. Rammers used generally vary in weight from 120 kN to 200 kN and drops of about 20 m are common.

Menard and Broise (1975) suggested the technique is effective due to following reasons.

1. Many soft clays are not fully saturated, and the small percentage of gas in the voids is dissolved in the pore water under the hammer impact, thus reducing the void volume.

2. Soft clay often liqify under impact.

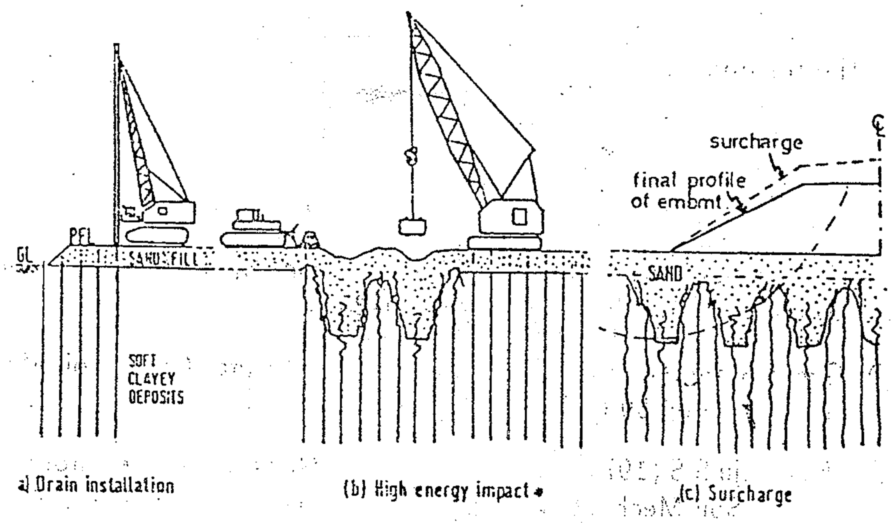


Figure 13: Dynamic Compaction

3. The permeability is increased during compaction because of liqefaction, fissuring and shearing and therefore pore pressure will dissipate rapidly.

4. Thixotropic strength gain follows the shear strength decrease caused by the compaction.

Sand columns can be formed in soft clay by systematically punching sand into clay by dynamic replacement(DR). When increased energy was applied on the sand column sand jets eject from them into the peaty clay(Dynamic Replacement and Mixing -DRM). This is suppose to disrupt the insitu soil fabric nullifing its secondary compression characteristics. Lo et al(1989) discussed an experience in Singapore.

With standard high energy impact technique one physical limitation is depth. Specially when the soil is saturated pore water pressure dissipation is slow. There vertical drains can be used in conjunction with the high energy impact method. Fibredrain, manufactured in Indonesia from natural jute and coconut fibre had been used in several projects in Asia(Lee et al 1988)

8. Concluding Comments

In this paper a large number of ground improvement techniques that can be employed to improve soft cohesive soils were discussed. The technique to be used for a project depends on;

1. Its suitability to the existing weak soil,

2. Machinery and other equipment and trained personal available,

3. Economy of the technique,

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Construction of Upper Sub-Base with Cement Stabilized Gravelly Soil on Colombo-Galle Road Deviation from Moratuwa to Panadura via Egodauyana

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1.0 INTRODUCTION

A new deviation from Moratuwa to Panadura Via Egodauyana on Colombo Galle Road was proposed in 1980 by the then Highways Department to ease congestion of traffic in the Moratuwa Urban Council areas and the road from Old Moratuwa bridge to the Panadura Town as there were no other practically possible alternative. This proposed deviation commences on the 19th km at cross junction in Moratuwa and ends at Walana on the 26th km.

Out of its 7.0km length, 1.5 km length commencing from cross junction at Moratuwa had been constructed upto the base level and nominally surfaced with 1st and 2nd coat applications by the then Highways Department. The balance portion of the deviation of about 5.5km was constructed under this project by the Road Development Authority which was the successor to the Highways Department. The total width of right of way of road was 27.2m, considering a dual carriageway of width 13.6m with two lanes in each direction and including a centre median, shoulders and side drains.

Based on the traffic data and subgrade strength, the designed pavement structure of the road consisted of a lower subbase of soil having a 4 day soaked CBR of not less than 8, an upper subbase of gravelly soil having a 4 day soaked CBR of not less than 20 and a dense graded aggregate base and bitumen bound base cum surfacing.

In the area where this road was situated there was a dearth of suitable gravelly soil for the upper subbase, coupled with a poor supply of suitable dense graded aggregate material for the base. Considering these problems, it was decided to use the available gravelly soil by improving the quality by cement stabilizing same to have an amended upper subbase having a 4 day soaked CBR not less than 50 along with a base cum surfacing of a bitumen bound base material.

In ancient times stabilized soil with cementitious material such as lime were known as construction materials. In Sri Lanka, many ancient monuments have been constructed by using cementitious materials. Cement stabilization may be defined as alteration of soil properties especially the strength to meet specific engineering requirements. In road works, the purpose is to produce a material which is strong enough to act as a subbase or a base and which retains its strength in the presence of water. Strictly considering, the cement stabilization methods fall into the category of soil improvements, although, in general terms, they can be considered as ground improvements using soil for passage of vehicles.

With the experience we have gained in the past, it has been found out that we can construct very strong sub bases and bases economically by stabilizing gravelly soils with cement. Using this method, this section of road was constructed by the Road Construction and Development Company (Pvt) Ltd., (RC&DC) and supervised by the Research & Development (R&D) Division and the Chief Engineer's (Colombo) Office of the Road Development Authority (RDA). The intention of this project was to construct this section of the road to the best of our ability with the available equipment and staff so that it could be considered as a test track for monitoring its behaviour under a known traffic condition.

In this project, a road pavement of a lower subbase of soil of thickness 150mm having a 4 day soaked CBR of not less than 8 compacted to 100 percent standard density, an upper subbase of cement stabilized soil of thickness 150 mm having the required strength and a base cum surfacing of thickness 100mm was placed on a compacted subgrade as per Standard Specification for Construction and Maintenance of roads and bridges (SSCM). The ensuing sections describe the construction of the upper subbase of cement stabilized soil. The construction work was commenced in November 1990 and completed in November 1992.

2.0 MATERIALS AND INITIAL TESTING

2.1 Selection of gravelly soil

Gravelly soil quarries available around the project area were surveyed by the laboratory staff of the R&D Division, RDA. Samples from test pits were collected and tested at the Central Laboratory of the same division.

All gravelly soils that were to be used for cement stabilization, were found to be having high plasticity characteristics with excessive clay contents. Therefore in order to reduce the plasticity characteristics of such soils, sands were blended with gravelly soil prior to stabilization. Such sands were obtained when wide and deep open soakage pits were excavated for drainage purposes for the road.

The most suitable proportions of blending were decided after carrying out trials in the Central Laboratory.

The selection of gravelly soil for stabilization was based on a 3 day moisture cured and 4 day soaked CBR of not less than 50 as given in SSCM and a stabilized soil liquid limit (LL) and plasticity index (PI) of not more than 40 and 15 respectively. Table 1 tabulates the material properties of the gravelly soil before blending with sand and after addition of sand and 2 or 3% cement by weight of combined soil for one or two strata in the quarries at Dampe, Egodawatte, Pinwala, Pinwala - Sepalika and Nampamunuwa. The material characteristics include the LL, PI, optimum moisture contents and maximum dry densities for standard conditions of compaction and the relevant cured and soaked CBR values at 100 percent standard compaction. As the period of 3 day curing and 4 day soaking for cement stabilized soil was considered too long a delay in construction activities, 1 day curing and 3 day soaking, was used in this construction project. The CBR values that were obtained in the latter method of testing was considered conservative when compared with the former as specified in the SSCM.

The test results indicate that 15 to 40% sand by total weight of soil has to be added to the cement stabilized soil to reduce the LL & PI to the stipulated values and a 2 percent cement by total weight of soil has to be added to obtain the required CBR value. As the efficiency of mixing in the field is less than that in the laboratory, it is the normal practice to specify an additional 1% of stabilizer above the laboratory requirements for field work and in this case 3% cement by weight of soil was used.

2.2 Cement

"Sanstha Thammanna" ordinary portland cement (OPC) was used for the cement stabilization work in this project. Acceptance tests were carried out according to the SLS 107 to ensure that the strength characteristics of the cement were obtained before the construction work. Random tests were also carried out during the construction.

2.3 Water

Potable water was used for the cement stabilization work in the project.

3.0 CONSTRUCTION

3.1 Equipment and Machinery

Construction equipment and machinery used for the stabilization work are given below.

1. Front end loaders
2. Rotovator driven by tractor
3. Motor grader
4. Water distributor bowser
5. 8-20 tonne vibratory roller.

3.2 **Preparation of gravelly soil for cement stabilization**

Selected gravelly soil and the above mentioned sand were brought close to the location of laying site for stabilized material and stock piled. The roots, vegetable materials and big lumps were removed manually from the gravelly soil before use. Then the gravelly soils and sand were blended to the correct proportions as given in Table 1. The prepared materials were stock piled at regular intervals to enable easy mixing with machinery.

3.3 **Stockpile mixing**

Measured stockpiles of the dry gravelly soil - sand mix and the correct amount of cement (3% by weight of soil) were mixed thoroughly with front end loaders until they formed a homogeneous mix. This method was found to be the best practicable method, out of a number of procedures in mixing cement and gravelly soil tried out including insitu mixing with a rotovator or a grader.

In batch mixing, the quantity of soil was measured using the front end loader bucket. This was far more accurate than taking the stock pile volume. Quantity of cement required was calculated and added in bags of 50 kg. weight.

Uniformity of the mixture was excellent when gravel was dry and the mixing machine operator was skilled. A quantity of 120 cu.m. could be mixed per 8 hour shift.

3.4 **Spreading of Material**

On the compacted lower sub-base after watering the surface, cement gravelly soil mixes were placed up to level peg marks, that were previously placed according to the cross fall of the surface. Front end loaders were used to transport the material to the required areas and also for initial levelling.

3.5 Watering the material

For uniform watering of the materials, water bowsers with distribution bars were used. Whenever the sprinkler bar was not working, watering by manual means was adopted after training the workmen.

3.6 Mixing after watering

A tractor driven rotovator was available at the site for mixing after watering. Re-watering was carried out where the material was partially wetted. This was followed by rotovator mixing. Whenever the rotovator broke down, the grader was used for mixing. This process was continued until the cement soil mix was 1% - 2% moisture content above the optimum value, as there was evaporation during mixing, spreading and rolling. However the field moisture contents given in Table 2 indicate that they are lower than the O.M.C.. This is due to the hydration of cement that takes place during the period when water is mixed with the cement stabilized soil.

3.7 Levelling

After the completion of watering and mixing, motor graders were used for levelling the surface. Level pegs were taken as reference points during the process of levelling. In the final levelling stage, a string was drawn across the road in a manner which showed the humps or depressions in the initially levelled surface. These sections were corrected by cutting or filling and the final touches were carried out using the grader blade. As care should be taken to complete the rolling as given in 3.8 within 1 1/2 hours after addition of water for mixing, steps 3.5 to 3.8 should be expedited. In view of this and as step 3.7 was time consuming, special care should be taken to complete such work without delay.

3.8 Compaction

Vibratory rollers were sent over the finally levelled surface for 8-10 passes. Initially two passes were carried out without vibration and the balance passes were done with maximum vibration. This procedure gave the required density. When there was insufficient water for compaction it was difficult to compact and cracks appeared on the surface. When there was too much of water for compaction, the material got adhered to the roller and the failure of the layer took place by heaving. This indicated that the watering should be carried out uniformly and up to the optimum moisture content (OMC). As drying could take place, a maximum of 2% over the OMC was allowed.

3.9 Curing

Cement stabilized upper sub-base layers were cured for a period of 7 days by continuous wetting of the surface.

3.10 Priming of the surface

When the stabilized layers were completely cured a bituminous prime coat was applied. Cationic slow setting (CSS-1) emulsion diluted to 1:1 with water was used for priming the stabilized layers. The surface was sprinkled lightly with water and cleaned using a power broom. Then the prime coat was applied at a rate of 1.0 l/sq.m. using a small bitumen distributor. Whenever the prime coat was damaged before setting, such areas were reprimed.

3.11 Quality control tests

All possible measures were taken to achieve good quality in the construction of this project, during the selection and acceptance of all materials and during all steps of construction. A field laboratory was set up at the site in addition to the facilities available at the Central Laboratory that was only 10km away from the site.

Using these facilities, the tests such as maximum dry density and optimum moisture content at 100 percent standard compaction, laboratory 1 day cured 3 day soaked CBR values at 100 percent standard conditions of compaction, field dry density, field moisture content by oven drying and also by using a speedy moisture content apparatus, degree of compaction and thickness of upper subbase were carried out at various locations of the upper subbase construction work at intervals of 15 - 30m and are given in Table 2. Field CBR values at intervals of about 50 to 100m were also carried out and are also given in Table2.

After completion of mixing of cement and gravelly soil - sand blend, samples were collected for laboratory testing as per a method given in Annex 1. The cement content in cement stabilized soils were determined by means of a titration method. The results of these tests at various locations are given in Table 2. As the acid used in the titration was neutralised by both cement and alkalis in the soil and as these soils have varying percentages of alkalis, this test can be considered as only a guidance to see whether uniform mixing is done. As the cement contents determined by the titration test as given in Table 3 varies between 2.6 and 4.8 it may be considered that the mixing is satisfactory.

From the above results, it could be seen that the degree of compaction, laboratory CBR and field CBR values and the thickness of the layer were satisfactory in most cases but from section to section the values varied with some of them below the values for acceptance. A few sections where the laboratory CBR was low, were removed and redone with fresh materials.

4.0 STABILIZATION WITH HARDENED CEMENT

A request was made by an organization to make use of about 20 tonnes of hardened cement that was to be disposed of. Action was taken to explore the possibility of making use of the material for stabilization at this site.

Laboratory tests were carried out by the Central Laboratory to determine the content of hardened cement to be used if possible for the stabilization. The material used for stabilization in the laboratory was taken after powdering in the field as given in the next paragraph. According to the laboratory trials, it was required to use 12% of hardened cement by weight of soil to get the specified CBR value.

A section of about 72m of this deviation was stabilized with hardened cement on an experimental basis. Initially, hardened cement was brought to powder form as much as practicable by means of a steel wheeled vibratory roller sent over same. The powdered material was found to be about 50% passing the 0.150mm sieve. The construction work was carried out in the same manner as described for OPC. Results of random tests similar to those given in table 2, that were carried out during construction including cement contents determined by the titration tests (as described in Annex-1) are given in Table 3.

5.0 PROBLEMS ENCOUNTERED AND COMMENTS ON THE METHODS TRIED OUT

5.1 Selection of gravelly soil

The properties of materials from the various gravelly soil quarries varied considerably throughout them. In some areas of such quarries, materials inferior to those tested were present. In order to prevent such inferior materials being brought to the laying site and thereafter being rejected, a trained officer, was stationed at the quarry, with samples of the earlier accepted materials. By this method only selected gravelly soil was brought to the mixing and laying site for stabilization.

5.2 **Mixing gravelly soil with cement**

The easiest method of mixing gravelly soil with cement is by using a front end loader. However, as it was difficult to obtain dry soil during the wet season, it was solved as given below.

During this season, when rain is eminent, the soil was heaped up into high conical piles reducing the surface area of pile and adequate drainage around the pile was provided in order to reduce the ingress of water into the area. Whenever there was hot sunshine, the material in the piles were spread into thin layers in order to dry same and to obtain suitable dry soil for mixing.

5.3 **Watering, levelling and compaction**

The whole exercise of cement stabilization depends on the steps of watering, levelling and compaction. The important requirement of compacted density, thereby the requirements of strength, cannot be achieved if the following are not carried out properly:-

1. Uniform watering and mixing the dry cement soil mix with water to keep the moisture content within the tolerances.

2. Completion of all the above procedures within the stipulated time period.

It has to be emphasized that if the required densities are not achieved in a particular area, then the cement stabilized material in this area has to be removed and redone and such an area cannot be recompacted later as in the case of lime stabilized soil. This is because the cement stabilized material would have hardened without achieving the required densities.

It is essential that the required equipment should be obtained if we are to undertake cement stabilization work. In this respect water bowsers with distributors, rotovators and heavy rollers in adequate numbers in working order should be brought to site before the work commences.

6.0 CONCLUDING REMARKS

The upper subbase of a road of length 5 1/2 km and total width of 13.6m have been successfully cement stabilized using a combination of road construction and agricultural plant, machinery and tools. The techniques adopted at site include the mixing of selected dry gravelly soil with cement by using front end loaders and mixing the cement soil mix with water by using a rotovator.

In future with the running out of suitable gravelly soils for subbases having CBR values of more than 20 and restrictions in the procuring of aggregate base materials, this method will have a lot of applications both as upper subbases and bases. In this work it is recommended that adequate plant and machinery in working order should be available at site if such projects are to be undertaken.

With respect to costs, cement stabilized bases have been found to be generally cheaper than dense graded aggregate bases even at 1995 costs.

7.0 ACKNOWLEDGEMENTS

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ANNEX 1

DETERMINATION OF CEMENT CONTENT IN CEMENT STABILIZED SOILS BY MEANS OF A TITRATION METHOD

The cement soil sample is reduced by quartering. Fifty gms. of the sample is ground by using a mortar and pestle and sieved through a 425 um sieve. The sample is then oven dried at a temperature of 110^{0}C for three hours. Five grammes of this sample is taken and transferred to a 500ml capacity volumetric flask. 100ml of decinormal Hydrochloric acid (N/10HCl) is added to the flask. The solution is made up to 500ml with distilled water and shaken thoroughly. The prepared solution is kept overnight for the reaction to be completed.

50ml of this clear solution is transferred to a titration flask and a few drops of Methyl Orange is added to the solution as an indicator. The solution is titrated against decinormal Sodium Carbonate (N/10 Na_2Co_3) in the Burette, when the colour changes from pink to straw colour.

The volume of unreacted acid is determined by the above titration and let this volume be V. Then the volume of decinormal Hydrochloric Acid (0.1N Hcl) reacted with cement soil sample is (100-V)ml.

The percentage of cement in the cement soil sample is read directly from the graph obtained as given below.

METHOD OF OBTAINING THE GRAPH FOR THE DETERMINATION OF CEMENT CONTENT

Cement soils samples are prepared with varying percentages of cement. Namely 1% to 5% of cement.These cement soil samples and a blank soil sample are ground separately and sieved through a 425 um sieve as previously done. The samples are oven dried at a temperature of 110^{0}C for three hours. Five grammes of each samples is taken and treated separately with varying volumes of decinormal Hydrochloric acid and the solutions made up to 500ml with distilled water described briefly as follows.

1. **BLANK SOIL SAMPLE**

   Five grammes of the soil sample (without cement) is treated with 30ml of decinormal Hydrochloric acid instead of 100ml of acid as given above and the volume of decinormal Hydrochloric acid reacted with five grammes of soil sample is determined as given above.

   Similarly cement soil samples containing 1%, 2%, 3%, 4% and 5% cement are prepared and reacted with 50,70,90,110 and 130 ml respectively of N/10 HCL and the volume of decinormal hydrochloric acid reacted with each sample is found out.

A graph is plotted with volume of decinormal Hydrochloric acid reacted with the sample on the Y axis and the percentage of cement in the sample on the X axis.

A typical plot of a such a graph for a particular soil is given below.

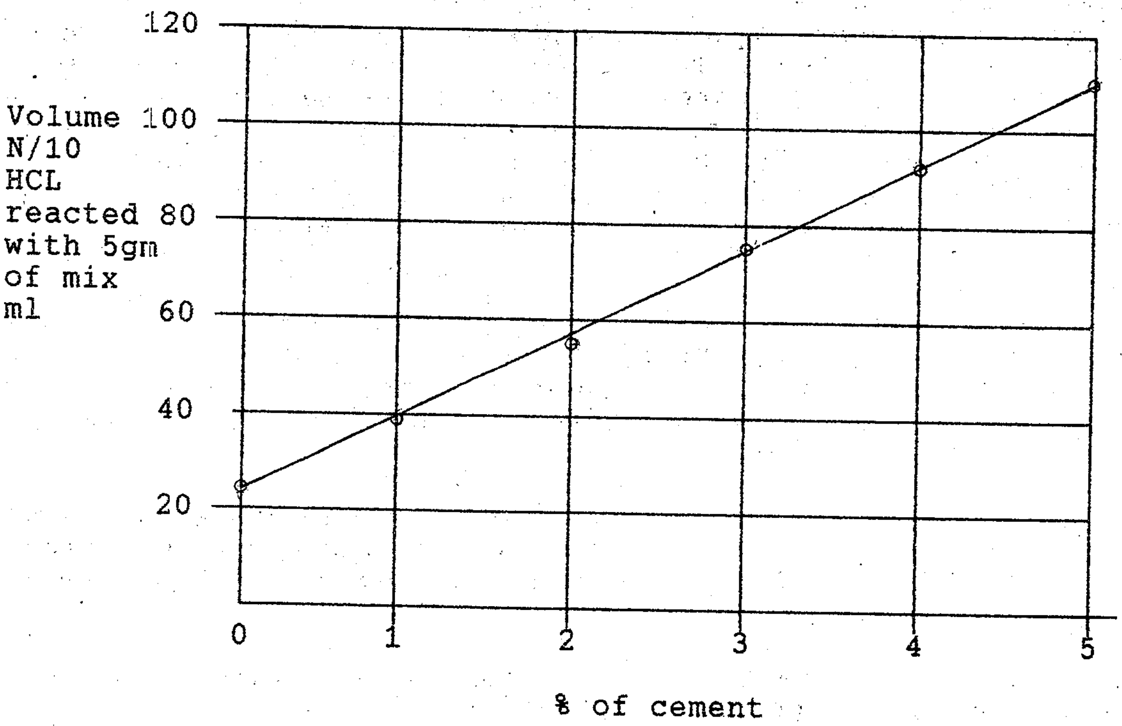

Table 1: Properties of gravelly soil from one or two strata of various quarries before blending with sand and after addition of sand and 2 or 3 percent cement by weight of combined soil

Name of Gravelly soil Quarry	Dampe						Egodawatta						Pinnawala						Pinnawala Sepalika					Nampamunuwa		
Layer depth	0–1m			0.5–1.8m			0–1.1m			0.0–2.5m			0.0–0.8m			0–1.1m			0–1.0m					0.15–1.3m		
Sample No.	S/2706			S/2708			S/2800			S/2801			S/2849			S/2850			S/2902					S/2953		
% of sand	0%	40%		0%	40%		0%	20%		0%	30%		0%	20%		0%	30%		0%	20%		30%		0%	15%	
% of cement	0%	2%	3%	0%	2%	3%	0%	2%	3%	0%	2%	3%	0%	2%	3%	0%	2%	3%	0%	2%	3%	2%	3%	0%	2%	3%
Liquid Limit (%)	47	28	31	63	32	32	32	28	32	55	29	27	39	28	27	48	28	28	34	29	31	24	25	44	30	30
Plasticity Index (%)	13	07	07	27	06	06	13	05	06	25	08	05	15	06	08	19	05	07	14	07	09	05	04	16	07	07
OMC (%) Standard	15.0	10.8	11.1	15.4	12.9	13.0	10.0	8.3	8.2	13.6	9.7	9.6	13.5	12.2	12.3	13.1	11.9	12.6	11.4	10.3	10.2	10.7	10.5	11.2	10.1	8.8
OMC (%) Modified	–	9.5	9.2	–	10.2	10.6	–	7.0	6.9	–	8.9	8.9	–	10.1	10.0	–	9.4	9.6	–	8.5	9.0	8.4	8.2	–	8.2	8.6
MDD 10 kg/m³ Standard	1.90	1.97	1.95	1.78	1.88	1.94	2.20	2.25	2.26	198	2.12	2.14	1.90	1.97	1.99	1.88	2.00	1.98	1.98	2.04	2.01	2.02	2.04	2.15	2.19	2.19
MDD 10 kg/m³ Modified	–	2.08	2.11	–	2.07	2.06	–	2.36	2.35	–	2.22	2.21	–	2.09	2.12	–	2.14	2.14	–	2.10	2.13	2.13	2.13	–	2.30	2.30
CBR Standard	13	53	80	33	97	115	35	108	133	13	110	144	26	72	94	34	96	114	27	108	130	91	95	37	109	91
CBR Modified	–	127	130	–	115	138	–	121	148	–	125	155	–	130	151	–	162	151	–	148	214	161	148	–	207	198

Note: OMC – Optimum Moisture Content
MDD – Maximum Dry Density
CBR – Californi Bearing Ratio

TABLE 2 – RESULTS OF QUALITY CONTROL TESTS

Date	Section Chainage in m	Test point Chainage m	Field CBR %	M.D.D. 10^3 kg/m³	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m³	F.M.C. %	Degree of Compac–tion	Speedy M.C. %	Thickn–ness of layer	Cement content %	Remarks
17.12.1990	1+220 – 1+270	1+229	60	2.11	11.2	151	1.98	8.4	94	6.5	170		
		1 + 246.5		2.11	11.2	151	2.04	9.5	97	7.6	–		
		1+254		2.11	11.2	151	2.09	7.9	99	6.5	120		
18.12.1990	1+270 – 1+320	1+279		2.11	11.2	146	1.94	8.4	92	7.6	130		
		1+302		2.11	11.2	146	2.05	9.1	97	7.8	125		
19.12.1990	1+320 – 1+370	1+345		2.15	10.7	147	2.22	8.7	103	9.8	200		
		1+355		2.15	10.7	147	2.13	10.0	99	9.8	150		
20.12.1990	1+370 – 1+420	1+386.5		2.04	12.4	195	1.95	9.5	96	9.8	160		
		1+410		2.04	12.4	195	2.02	11.4	99	9.8	170		
21.12.1990	1+420 – 1+510	1+430		2.06	11.0	198	2.08	8.8	101	11.7	200		
		1+460.5					2.08	9.2	101	11.7	200		
		1+480					2.12	9.9	103	12.8	200		
22.12.1990	1+510 – 1+550	1+513.5		2.06	11.0	126	1.93	12.0	94	11.7	190		
		1+540.5					2.00	11.4	97	11.7	190		
24.12.1990	1+550 – 1+600	1+565		2.01	12.3	111	1.98	10.0	99	11.5	130	2.8	
		1+593					1.90	11.3	95		190		
26.12.1990	1+600 – 1+630	1+615		1.97	13.5	53	1.94	10.2	98	11.0	100	2.75	
27.12.1990	1+630 – 1+710	1+640		1.91	13.8	104	1.82	11.8	95	13.6	170		
		1+665					1.89	11.7	99		120	2.70	
		1+690					1.95	11.2	102		125		
28.12.1990	1+710 – 1+760	1+727.5		1.99	13.2	47	2.00	11.6	100	12.3	150	3.5	
		1+741.5					1.95	12.0	98		160		
29.12.1990	1+760 – 1+810	1+777		2.01	12.1	74	1.96	9.3	98	9.2	70	3.4	
		1+799					1.96	9.0	98		85		
01.01.1991	1+810 – 1+865	1+817.6		1.97	13.4	85	2.00	9.1	102	8.7	110	3.3	
		1+835					1.98	9.5	100		100		
02.01.1991	1+865 – 2+000	1+865		2.02	12.3	48	1.91	12.0	95	11.0	150	3.2	
		1+889					1.90	10.8	94		120		

M.D.D. – Maximum Dry Density
O.M.C. – Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus
Cement content – As determined by titration (See Annex–1)

TABLE 2 – RESULTS OF QUALITY CONTROL TESTS

Date	Section Chainage in m	Test point Chainage m	Field CBR %	M.D.D. 10^3 kg/m³	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m³	F.M.C. %	Degree of Compac–tion	Speedy M.C. %	Thickn–ness of layer	Cement content %	Remarks
02.01.1991	1+865 + 2+000	1+902		2.02	12.3	48	2.02	10.8	100	11.0	150	3.2	
		1+956					1.83	8.1	91		30		
		1+943		1.97	13.5	53	1.91	12.6	95		130		
		1+982		1.91	13.8	104	2.03	19.2	100		110		
03.01.1991	2+000 – 2+055	2+006		2.02	12.2	62	1.97	9.7	98	10.0	150	2.90	
		2+031					2.04	9.0	101		90		
04.01.1991	2+055 – 2+160	2+062		2.00	12.2	103	1.89	9.4	95	11.0	50	2.8	
		2+090					1.80	13.0	90		100		
		2+121					1.89	10.2	95	9.2	140		
		2+144					1.93	10.4	97		150		
05.01.1991	2+160 – 2+240	2+161		2.00	13.5	75	1.93	11.4	97	8.7	140	2.6	
		2+180					1.91	10.6	96		160		
		2+203.6					1.82	10.6	91		140		
07.01.1991	2+240 – 2+350	2+247		2.00	11.6	129	1.87	9.4	94	8.7	120	2.8	
07.01.1991	2+240 – 2+350	2+250	90										Before mixing
		2+273		1.28	13.5	53	1.86	10.2	93	11.0	160		
		2+279	100		13.8	104							
		2+285	90										
		2+294		2.02	12.2	62	1.82	10.6	91		140	2.90	
		2+310	80										
		2+315					1.75	11.4	87		125		
		2+234					1.92	11.2	96		150		
08.01.1991	2+350 – 2+450	2+351		1.95	14.4	121	1.88	12.0	96	9.8	140	3.4	
		2+377					1.91	11.5	98		240		
		2+403					1.86	12.3	95		200		
		2+440					1.82	10.2	93		100		

M.D.D. – Maximum Dry Density
O.M.C. – Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus
Cement content – As determined by titration (See Annex–1)

Table 2: **RESULTS OF QUALITY CONTROL TESTS**

Date	Section Chainage in m	Test point Chainage m	Field CBR	M.D.D. 10^3 kg/m³	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m³	F.M.C. %	Degree of Compac–tion	Speedy M.C. %	Thickn–ness of layer	Cement content %	Remarks
09.01.1991	2+450 – 2+540	2+469.5		1.98	13.2	145	1.75	11.9	88	9.3	160	3.4	
		2+498.5					1.92	15.2	97		175		
		2+523.5					1.94	14.2	98		120		
12.01.1991	2+540 – 2+650	2+547	100										
12.01.1991	2+540 – 2+650	2+556		1.99	12.2	42	1.79	9.5	90	7.6	350	3.3	
		2+582				110	1.78	9.4	89		234		
		2+583.5	150										
12.01.1991	2+540 – 2+650	2+610		1.99	12.2	42	1.77	9.1	89	7.6	200	3.3	
		2+618.5	150										
		2+634.5				110	1.77	7.5	89		170		
15.01.1991	2+650 – 2+730	2+655		1.90	15.3	172	1.82	12.9	96	12.3	130	3.4	
		2+690					1.75	15.3	92		200		
		2+704					1.74	15.1	92		160		
17.01.1991	2+730 – 2+780	2+741.5		1.92	15.7	40	1.87	10.6	97	11.3	200	3.3	
		2+769					1.76	11.1	92		160		
18.01.1991	2+780 – 2+840	2+799		1.84	15.7	60	1.78	12.9	97	12.6	160	2.8	
		2+829					1.76	11.6	96		200		
19.01.1991	2+840 – 2+900	2+859.5		1.83	15.2	76	1.72	12.7	94	15.9	180	3.6	
		2+883					1.72	12.7	94		190		
21.01.1991	2+900 – 2+980	2+915		1.81	15.0	22	1.86	13.0	103	13.0	210	3.8	
		2+938					1.61	10.0	89		210		
		2+960					1.80	13.1	100				
23.01.1991	2+980 – 3+060	2+987		1.84	15.7	90	1.81	12.9	98	13.0	100	3.6	
		3+032					1.68	11.2	90		200		

M.D.D. – Maximum Dry Density
O.M.C. – Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus
Cement content – As determined by titration (See Annex–1)

TABLE 2 – RESULTS OF QUALITY CONTROL TESTS

Date	Section Chainage in m	Test point Chainage m	Field CBR %	M.D.D. 10^3 kg/m^3	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m^3	F.M.C. %	Degree of Compac– tion	Speedy M.C. %	Thickn– ness of layer	Cement content %	Remarks
		3+053					1.79	11.9	99		200		
24.01.1991	3+060 – 2+120	3+073		1.81	16.4	50	1.70	10.2	94		200	3.1	
		3+115					1.88	10.5	104		180		
25.01.1991	4+220 – 4+290	4+232		1.97	12.6	24	1.97	9.9	100	13.0	150	3.6	
		4+236	310										
		4+264					2.05	10.2	104		180		
		4+280					1.97	10.8	100		190		
26.01.1991	4+120 – 4 + 220	4+140		1.92	13.6	141	1.92	8.0	100	12.0	225	3.4	
26.01.1991	4+120 – 4 + 220	4+159					1.96	11.4	102		180		
		4+190					1.94	10.3	101		180		
		4+195					1.90	8.7	99		200		
28.01.1991	4+120 – 3+995	4+105		2.00	12.6	105	2.00	8.4	100	11.0	205	3.6	
		4+084					2.03	8.7	102		220		
28.01 1991	4+120 – 3+995	4+063		2.00	12.6	105	1.98	7.9	99	11.0	210	3.6	
		4+040					1.88	7.3	94		205		
		4+011					2.00	8.3	100		190		
30.01.1991	3+995 – 3+855	3+983	150										
		3+982		2.00	12.5	26	1.94	11.3	97	11.0	100	2.8	
		3+960					1.99	10.3	100		200		
		3+933					1.91	10.8	96		200		
		3+920	170										
		3+915					1.91	9.7	96		150		
		3+877					1.96	10.4	98		225		
01 .02 1991	3+810 – 3+710	3+725		1.93	13.4	39	1.88	9.7	97	9.8	200	3.5	
		3+755					1.93	9.2	100		200		
		3+761	140										
		3+775					1.93	9.4	100		190		
		3+790	170										

M.D.D. – Maximum Dry Density
O.M.C. – Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus

TABLE 2 – RESULTS OF QUALITY CONTROL TESTS

Date	Section Chainage in m	Test point Chainage m	Field CBR %	M.D.D. 10^3 kg/m^3	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m^3	F.M.C. %	Degree of Compac-tion	Speedy M.C. %	Thickn-ness of layer	Cement content %	Remarks
		3+794					1.90	9.4	98		150		
05.02.1991	3+710 – 3+660	3+701		1.89	14.4	28	1.91	14.3	101	11.7	200	2.7	
		3+655					1.88	14.8	99		265		
06.02.91	3+660 – 3+600	3+605		1.92	12.3	18	1.96	12.6	103	11.7	200	2.7	
		3+629					1.88	13.4	98		220		
		3+631	120										
		3+651					1.81	14.2	94		230		
07.02.91	0+980 – 1+050	0+985	220										
		0+995		1.94	13.2	20	1.93		99	14.4	230	2.7	
		1+019					1.98	13.0	102		160		
08.02.91	1+050 – 1+130	1+053		1.91	13.7	27	1.88	12.0	100	10.0	200	3.8	
		1+063					1.93	11.5	98		250		
		1+107					1.95	12.8	101		210		
09.02.91	1+130–1+180	1+139		1.98	12.9	51	1.95	9.2	99	7.6	130	3.4	
		1+161					1.93	11.1	97		190		
11.02.91	1+180+1+220	1+192		2.00	13.9	119	1.98	12.7	99	8.7	180	3.8	
		1+215					2.03	9.8	102		90		
12.02.91	3+590 – 3+540	3+581		1.96	12.8	49	1.91	14.0	97	7.6	190	3.6	
		3+571	270										
		3+573	60										
		3+559					1.85	11.6	94		240		
		3+560	140										
12.02.1991	01+765–01+790	01+778		2.13	11.5	155	2.10	13.3	99	9.8	180	3.5	
13.02.1991	3+540 – 3+490	3+520		1.96	12.8	170	1.92	12.1	98		225	3.3	
		3+500					2.01	12.3	103		250		
13.02.1991	3+130 – 3+160	3+142		1.80	15.0	55	1.76	12.8	98	16.6	200	3.4	
14.02.1991	3+420 – 3+490	3+434		2.00	12.4	29	1.96	11.3	98	12.1	210	2.8	
		3+456					1.86	10.2	95		200		
		3+459	210										

Note:– Field CBR values and test locations are indicated in red.

M.D.D. – Maximum Dry Density
O.M.C. – Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus
Cement content – As determined by titration (See Annex–1)

TABLE 2 – RESULTS OF QUALITY CONTROL TESTS

Date	Section Chainage in m	Test point Chainage m	Field CBR %	M.D.D. 10^3 kg/m³	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m³	F.M.C. %	Degree of Compac–tion	Speedy M.C. %	Thickn–ness of layer	Cement content %	Remarks
		3+487					1.97	10.4	99		160		
14.02.1991	3+160 – 3+220	3+170		1.86	15.7	63	1.78	13.6	96	16.0	200	3.1	
		3+182				32	1.79	15.4	96		200		
		3+194	210										
		3+215					1.77	14.3	95		220		
16.02.1991	1+940 – 1+970	1+952		1.91	14.0	75	1.94	10.5	102	13.0		3.1	
	2+055 – 2+075	2+065		1.96	13.4	96	1.97	10.3	101	12.1		3.5	
18.02.191	3+360 – 3+420	3+364		1.97	14.0	17.4	1.86	9.4	94	14.2	175	3.3	
		3+364	170										
		3+397					1.93	10.0	98		180		
19.02.1991	3+220 – 3+360	3+242		1.88	14.4	140	1.87	13.4	99	13.7	200	3.4	
		3+265		1.88	14.4	140	1.85	13.2	98		200		
		3+285		1.88	14.4	140	1.87	11.2	99		220		
		3+314		1.95	13.6	140	1.96	10.3	101		180		
		3+342		1.95	13.6	140	1.92	9.9	98		200		
20.02.1991	4+395 – 4+420	4+407		1.90	14.6	194	1.86	18.1	98	14.7	160	3.1	
21.02.1991	4+420 – 4+490	4+434		1.92	14.2	158	1.98	11.7	103	13.1	200	2.8	
		4+460					1.88	13.0	98	13.9	190		
		4+476					1.91	9.7	99		150		
22.02.1991	4+490 – 4+610	4+583		1.95	14.0	118	1.90	9.9	98	15.1	150	3.4	
		4+555					1.89	10.5	97	14.7	200		
		4+532					1.82	14.4	93	13.1	210		
		4+506		1.92	14.2		1.89	9.8	98		190	3.1	
23.02.1991	4+610 – 4+710	4+615		1.95	14.0	51	1.96	10.6	101		150	3.5	
		4+632		1.92	13.8		1.83	9.9	95		220		
		4+657		1.92	13.8		1.88	8.3	98		160		

M.D.D. – Maximum Dry Density
O.M.C. – Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus
Cement content – As determined by titration (See Annex– 1)

TABLE 2 – RESULTS OF QUALITY CONTROL TESTS

Date	Section Chainage in m	Test point Chainage m	Field CBR %	M.D.D. 10^3 kg/m^3	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m^3	F.M.C. %	Degree of Compaction	Speedy M.C. %	Thickness of layer	Cement content %	Remarks
		4+680		1.98	11.1		2.01	8.7	102		190		
25.02.1991	4+710 – 4+760			1.99	12.2	145				10.1		3.3	
		4+769		1.94	13.1	116	1.87	11.2	96	12.7	200	3.6	
		4+793					1.90	13.6	98		225		
		4+818					1.89	11.7	98		200		
26.02.1991	4+820 – 4+955	4+877	130										
		4+880		1.87	16.0	62	1.78	9.5	95		170	2.8	
		4+900					1.76	12.6	94		200		
		4+902	140										
		4+929					1.83	10.3	98		190		
		4+945					1.91	7.3	102		150		
	4+955 – 5+020	4+968		1.92	14.5	46	1.94	7.6	101		170	3.1	
		4+988					1.94	9.4	96		165		
		5+005	90										
27.02.1991	5+020 – 5+140	5+024		1.95	12.3	59	1.92	9.6	98		175	3.4	
		5+047					1.94	7.4	99		200		
		5+068	190										
		5+070		1.92	13.5		1.81	8.3	94		155	3.5	
		5+090					1.87	9.1	97		160		
		5+120					1.93	9.5	101		160		
01.03.1991	5+140 – 5+220	5+158	210										
		5+158		1.93	13.7	74	1.91	8.6	99		150	3.1	
		5+185		1.93	13.7		1.95	10.3	101		170		
		5+190	190										
		5+213		2.01	12.8		1.98	9.3	99		200		
02.03.1991	05+220 – 5+340	5+242	200										
		5+267		1.96	12.8	82	1.90	9.6	97		190	3.3	
		5+283		1.99	13.0		1.95	9.4	98		210		

M.D.D. – Maximum Dry Density
O.M.C. –Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus
Cement content – As determined by titration (See Annex–1)

TABLE 2 – RESULTS OF QUALITY CONTROL TESTS

Date	Section Chainage in m	Test point Chainage m	Field CBR	M.D.D. 10^3 kg/m^3	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m^3	F.M.C. %	Degree of Compac– tion	Speedy M.C. %	Thickn– ness layer	Cement content %	Remarks
		5+303	140										
		5+307		1.98	12.5		1.97	9.5	100		150		
13.03.1991	3+810 – 3+860	3+822		2.02	11.4	78	2.02	11.2	100	10.6	170	3.1	
		3+846					2.03	11.6	100		200		
14.03.1991	5+320 – 5+390	5+333		1.95	13.5	134	1.91	10.9	98	11.0	155	3.4	
		5+334	120										
		5+338	75										
		5+356		1.98	14.9		1.76	9.8	94	11.0	140		
		5+358	140										
	5+390–5+440	5+390		1.91	14.7	63	1.89	10.8	99	11.0		3.5	
		5+410					1.87	10.8	98	10.7			
		5+422	230										
		5+432					1.88	12.0	99	9.6			
15.03.1991	5+440 – 5+560	5+460		1.85	16.0	52	1.81	13.1	98	8.7	200	3.1	
		5+468	100										
		5+483		1.85	14.0		1.84	12.5	99		200		
		5+510		1.91	14.7		1.87	12.9	98	8.0	180		
		5+525	220										
		5+548		1.96	12.9		1.92	12.0	98		170		
16.03.1991	5+560 – 5+630	5+590		1.96	12.9	37	1.89	8.2	96		150	2.8	
		5+591	160										
		5+620					1.92	8.5	98		170		
18.03.1991	5+630 – 5+690	5+633		1.98	12.2	93	1.92	11.6	97		135	3.1	
		5+658					1.94	12.6	98		145		
		5+683					1.96	13.3	99		190		
20.03.1991	5+690–5+750	5+707		1.86	15.1	109	1.86	12.4	100	14.9	180	3.5	
		5+730					1.83	13.3	98	16.3	170		

M.D.D. – Maximum Dry Density
O.M.C. – Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus
Cement content – As determined by titration (See Annex–1)

TABLE 2 – RESULTS OF QUALITY CONTROL TESTS

Date	Section Chainage in m	Test point Chainage m	Field CBR %	M.D.D. 10^3 kg/m³	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m³	F.M.C. %	Degree of Compaction	Speedy M.C. %	Thickness of layer	Cement content %	Remarks
	5+750 – 5+810	5+755					1.83	12.7	98	16.9	160	4.4	
		5+775					1.88	11.1	101	16.9	200		
		5+795					1.77	11.6	95	19.0	190		
		5+800	260										
21.03.1991	5+810–5+890	5+820		1.89	13.8	45	1.84	13.1	97	20.4	180	3.5	
		5+845	160										
		5+845					1.84	11.6	97	16.9	175		
		5+851	260										
		5+870					1.80	12.5	95	14.2	130		
	5+890 – 5+950	5+887		1.89	13.8	163	1.84	8.8	98	19.0	150		
		5+915		1.89	13.8		1.90	8.9	101		150		
		5+940		1.85	14.6		1.76	9.8	95		185		
22.03.1991	5+950–6+070	5+965		1.85	14.6	122	1.85	9.6				3.3	
		5+992					1.82	8.3					
		6+015					1.87	8.5					
22.03.1991	5+960–6+070	6+040		1.88	14.5	122	1.85	8.5	98	17.6	200	3.3	
		6+065					1.80	10.9	95		200		
25.03.1991	6+070–6+240	6+075	100										
		6+090		1.85	15.1	72	1.80	13.1	97	12.5	110	3.1	
		6+115				80	1.84	12.4	100		145		
		6+140	180										
		6+140					1.80	12.1	97		190		
		6+162					1.81	12.0	98		200	3.4	
		6+187		1.81	15.6	75	1.81	10.0	100		180		
		6+210					1.77	11.4	98		140		
		6+235	180										
		6+235					1.73	10.6	96		130		

M.D.D. – Maximum Dry Density
O.M.C. – Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus
Cement content – As determined by titration (See Annex–1)

TABLE 2 – RESULTS OF QUALITY CONTROL TESTS

Date	Section Chainage in m	Test point Chainage m	Field CBR %	M.D.D. 10^3 kg/m^3	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m^3	F.M.C. %	Degree of Compac–tion	Speedy M.C. %	Thickn–ness of layer	Cement content %	Remarks
26.03.1991	6+240–6+360	6+262		1.81	15.6	75	1.83	8.8	101	12.5	160	3.3	
		6+289					1.85	10.0	102	15.6	130		
		6+311		1.89	15.1	30	1.91	10.9	101	16.9	170	3.5	
		6+339	220										
		6+339					1.83	11.2	97		200		
		6+355	190										
27.03.1991	6+360–6+420	6+364		1.97	13.1	55	1.95	11.2	99		200	3.1	
		6+388		1.89	15.1		1.90	12.8	101		200		
		6+400	150										
		6+413					1.88	12.0	99		170		
25.04.1991	6+033–6+157	6+060		1.74	17.0	71	1.72	13.7	99			3.2	
		6+127					1.69	14.6	97				
29.04.1991	6+160–6+260	6+171		1.92	12.8	55	1.92	10.3	100			3.8	
		6+236					1.86	11.0	97				
30.04.1991	6+260–6+300			1.85	14.4	57						3.1	
02.05.1991	5+965–6+060					42						3.4	
05.05.1991	0+060–6+110	6+093		2.04	12.9	81	2.00	10.7	98			3.4	
06.05.1991	6+110–6+255	6+164		1.97	13.6	94	1.89	8.8	96			3.1	
		6+197		1.99	12.7	12.7	2.08	7.8	104				
		6+226		1.96	13.7	13.7	1.98	8.7	101				
07.05.1991	6+255–6+375					60						3.2	
08.05.1991	6+327–6+375					26						3.8	Mergin section
09.05.1991	1+077–1+122					30						3.1	Mergin section
13.05.1991	0+978–1+077											2.7	Mergin section

M.D.D. – Maximum Dry Density
O.M.C. – Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus
Cement content – As determined by titration (See Annex–1)

TABLE 3 – RESULTS OF QUALITY CONTROL TESTS
SECTION OF STABILIZATION WITH HARDENED CEMENT

Date	Section Chainage in m (RHS)	Test point Chainage m	Field CBR %	M.D.D. 10^3 kg/m^3	O.M.C. %	Lab. CBR %	F.D.D. 10^3 kg/m^3	F.M.C. %	Degree of Compac-tion	Speedy M.C. %	Thickn-ness of layer	Cement content %	Remarks
26.03.92	3+555 – 3+565	3+561.3	–	1.90	14.0	145	1.87	10.9	98	–	–	4.80	
27.03.92	3+565 – 3+581	3+573.1	–	1,94	12.3	247	1.87	14.2	96	–	–	4.80	
28.03.92	3+581 – 3+592	3+588.8	–	1.83	14.1	258	1.77	15.5	97	–	–	4.50	
04.04 92	3+775 – 3+795	3+787.3	–	1.78	15.8	93	1.75	17.1	98	–	–	4.80	
07.04.92	3+795 – 3+810	3+803.3	–	1.72	16.7	284	1.82	16.7	106	–	–	–	

M.D.D. – Maximum Dry Density
O.M.C. – Optimum Moisture Content
C.B.R. – California Bearing Ratio
F.D.D. – Field Dry Density
F.M.C. – Field Moisture Content
Speedy M.C. – Moisture content (M.C.) determined by the speedy M.C. apparatus
Cement content – As determined by titration (See Annex–1)

Experiences in Heavy Tamping at Madiwela

W.M. Jayawardhane
D.S. Jayasinghe

Link (Engineering) Ltd

Synopsis:

A 5.5 ha low lying land at Madiwela, close to the new parliament at Kotte was to be developed for a housing scheme. The land was owned by the Urban Development Authority, and Link (Engineering) Ltd. was awarded the job through competitive tender.
Investigations revealed the existence of large amounts of peat which would undergo compression on filling as well as due to loads during construction and afterwards. Heavy Tamping was selected as the best method of ground improvement under the given constraints. This paper describes the problems faced by the developer in using this technique.

Introduction:

The Urban Development Authority decided to develop a low lying land at Madiwela close to the new parliament for a housing scheme comprising 340 units of two storey houses. The scheme was comprehensive in the sense it had its own sewerage disposal scheme, overhead tank, play ground etc.A joint venture by the name of Link Developments Ltd. was formed with equity participation from Development Finance Corporation of Ceylon and National Development Bank and Link.It was the first such scheme to be undertaken as a joint venture between the UDA and a private developer.

Site Conditions:

The land was previously used as paddy fields and went under water during the rainy seasons. The average level of the land was 1.58 m above mean sea level.Although there was a canal on one boundary the drainage was poor due to overgrowth and siltation.

To bring the land above flood levels it was necessary to fill by an average of 60 cm. However the unprecedented floods of 1992, which occurred while we were at the site, made it prudent to raise the fill by another 30 cm.

Soil investigations revealed
i.a ground water table about 0.15 m below ground level
ii.the site was covered with a very weak and highly compressible peat deposit of thickness about 4.0 m.
iii.for most of the site the peat was followed by grey plastic soft clay.

Loading as per design of houses: wall foundations 45 kn/m
Bearing pressure with 500mm wide footings 90kn/m2

Recommended construction methods:

Recommendations by specialists for the site were to select one of the following methods.

i.fill the site by 1.5 m and allow to settle for 3 months after which a settlement of about 0.39 m. could be expected.Then use shallow foundations.

ii.construct foundations on timber piles driven to sand at 6 m depth.

iii.remove 1.5 m of peat and replace with sand and use shallow foundations.

Selection of method of construction:

Alternative (i) was the common method used in this country. However we had a buyer for the first 120 housing units who wanted them ready for occupation in one year. Hence a waiting time of 3 months was not acceptable.

Alternative (ii) would improve only under the foundations. As we had sewer lines, man holes, roads etc. they would be affected by settlement with the passage of time.

Alternative (iii) had the immediate problem of disposing such a large quantity of peat. Further with the very shallow water table, removing peat would have been a problem.

It was when we were debating with this idea that we met our friend Dr. G.P.Karunaratne who was on a visit to Sri Lanaka. He was confident that Heavy Tamping would solve the problem, but we were not sure. However after debating for a while we finally decided to accept Heavy Tamping for this site and engaged the services of Dr. Karunaratne who was working in Singapore as a Consultant.

Heavy Tamping:

Heavy Tamping as the words suggest is to drop a weight from a height and filling the crater thus created with granular material. The tamping is continued until a pre-determined enforced settlement was achieved. To ensure uniform settlement,tamping was done in a pattern so that in the second round the weight was dropped at the centre of the 4 print marks of the first round.

The enforced settlement was dependent on the design loads on the area. eg. The sewage treatment plant area which was subjected to a higher load was tamped to a higher degree,to obtain a higher enforced settlement.

Equipment for Heavy Tamping:

i. a 15 ton tamper
ii. a crane capable of lifting and dropping a 15 ton tamper.
iii. a hydraulically operated clamp.

i.Tamper: The usual construction of the tamper is by assembling a series of steel plates by bolting them together to get the necessary weight which in this case is 15 tons. However the price of steel plates in this country was prohibitive and we had to look for an alternative.

With Dr. Karunaratne's consent, we settled for a tamper with a shell of steel plates 50 m thick reinforced on the inside with H-sections, and filled with lead. The shell weighed about 8 tons and when filled with lead and some sand came up to 15 tons.

ii.Crane: The crane had to work with the boom at an angle of 50° to the horizontal for several reasons.

a.to reduce rocking of the crane when the weight is released at the required height.

b.to increase the distance of the point of impact from the base of the crane.

Further as we were to work on soggy land the crane had to be a crawler crane and not wheel mounted.

A 120 ton crawler crane was available at the State Engineering Corporation and they were willing to hire it to us, even though the application was not lifting, as is usually done by cranes.

We were very happy that this crane could handle 15 tons at a boom angle of $\underline{50^{\circ}}$ and so we organised the trial tamping. The crane lifted the weight and released the brakes at the required height, but weight only touched the ground very softly. No indentation was made. The crane was not lifting the weight with a single cable, but with a pulley arrangement with several falls. Even when the brakes were released the speed at which the weight descended was too slow to give the full impact.
We had to find a crane which could lift 15 tons with a single cable. Even if we found one the operation required great accuracy. On the downward run with brakes off, the tamper gathers speed and the application of the brakes had to be to split second accuracy. Too early application of the brakes would reduce the impact and a too late application would cause damage to the hook and pulley which would crash on to the tamper.

iii. Clamp - **Search for an alternative:**

We were forced to look for an alternative method of dropping the weight or abandon Heavy Tamping.
The crane could lift the 15 ton weight without any problem. If we could find a mechanism to release the tamper at the required height, the problem could be solved.

Several alternatives were considered and finally we settled on a hydraulically operated clamp. At the required height the crane operator could release the weight by activating a spool valve mounted near the operator. This required a hydraulic pump, a set of high pressure hoses of length 30m each. Although in theory the system could work we were not confident until we saw it working.

This system, in a way was superior to a single cable crane operation because it used 100% g whereas with a crane, part of the acceleration would be lost due to the inertia of the system.

Shortage of fill material:

The heavy rains of 1992 and the restrictions on winning of sand in Kelani river made supply of sand for the operation very difficult. Earlier it was planned to use an average of 1 m fill of sand for the phase I work which worked out to 17000 cu.m. This was a large quantity to be collected in a short time.

Supply of lateritic soil was very much easier, but we were not sure whether it would mix with the peat layer as it would have been with sand. Further we had doubts whether the soil layer would produce the same effect as the sand layer, during tamping,as the latter was a better medium to transfer the tamping force directly to the underlying peat layer.

As a compromise we filled an area with lateritic soil and filled 30cm on top with sand. Tamping on this proved successful thus reducing the requirement of sand by 60%.

Procedure and readings:

Every time the tamper was dropped, the depth and diameter of the crateywas measured (assuming the crateyis circular). This is to measure the volume of fill in order to assess the enforced settlement.
Care was taken to see that no persons remained in vicinity of the print on which the tamper was dropped.

Problems with the neighbours:

The site bounded on two sides by roads and on the other two sides there were houses at a minimum distance of about 50 m.from the boundary.

When tamping for the sewage treatment plant was in progress the owner of a two storey house 50m away complained of severe disturbance to his family and danger of damage to his house. We anticipated such complaints and wanted to assess the actual disturbance at his house.
With his permission we installed strain gauge points across some cracks that existed previously. One day we went into one of his rooms where he complained the vibrations were most severe.
A wine glass filled to the brim was placed on the ground and the weight was dropped 50 m. away.

There was not even a ripple in the water in the glass, but all of us felt a vibration soon after the weight hit the ground. This was due to the fact that human perception was very much sensitive than the method of observation. The strain gauges did not indicate any increase in crack widths.Yet all efforts to convince him that such vibrations would not damage the structure failed and he threatened legal action.

We looked for some one who could actually measure the vibrations and the CISIR undertook the job.

Measurements showed that the peak particle velocity near the house when tamping was done 50 m away was only in the order of 2.5 mm/sec compared to a very conservative upper velocity limit of 25 mm/sec.

Further tests indicated that a trench of depth 2 m. and length 40 m. reduces the peak particle velocity by 50-60%. The height of the drop also had some relationship to the level of vibration. However the first drop always caused a higher vibration than the subsequent drops for the same drop height.

Whenever tamping was done near the boundary, we took all precautions to minimize the vibrations.

Considerable interruption to work was cause by frequent complaints to the police and on one occasion we had to go to courts to face charges of public nuisance. The complainant did not come to court for the hearing and we were discharged.

Discussion:

The construction work in phase 1 is complete and the houses are now in occupation for over 1 year. No serious cracks or other signs of stress are visible as yet.

Construction of 24 houses in phase 2 is complete and some are in occupation. The sewerage treatment plant area which is subject to the highest load conditions do not show any signs of settlement.

These observations would prove that the method of improving the ground conditions is successful.

Cost of improving ground using HT is about Rs 4.8 million. per ha. or Rs.12000 per perch.

Limitations:
The innovative approach in solving the technical problems faced in this exercise is very satisfying. However it must be mentioned that no discipline in engineering can develop in isolation. In civil engineering and building construction operations, one has to depend so much on equipment, a developed mechanical engineering back up is essential. This back up should include experienced and innovative mechanical engineers, modern work shop facilities, and a market place where material and equipment for innovative work are readily available.

The time lost due to the break down of the crane over the tamping period was 15%. The other equipment on hire were idle when the tamping was not in progress but hire charges had to be paid, thus increasing the cost.

It must be mentioned that for making the hydraulic clamp, all the parts were bought in junk yards and when a replacement was needed it was impossible to get one.

With only a few more years to celebrate the millennium, and talk of NIC status for our country by then, state policy should encourage investment in all fields of engineering relevant to our country, for innovation to be possible.

Acknowledgements:
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Foundation in Weak Ground for the Ambatale–Jubilee Water Conveyance System

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1.0 INTRODUCTION

Large diameter pipes have to be laid to transmit water from distant places in major water supply schemes. When difficult ground conditions such as ground having soft materials or highly variable materials are met, water works engineers are faced with problems to lay pipes through it. Finding an economic and durable solution to laying of pipes through such weak soils is a challengable task. This may involve improving the ground conditions or designing suitable types of foundations for the pipeline or both so that distress is not caused to the pipeline.

This paper critically evaluates the foundations in weak ground that had been carried out in the construction of Ambatale-Jubilee Water Conveyance System.

2.0 PROJECT DETAILS

Improving the drinking water supply to Greater Colombo region and providing drinking water facilities to Maharagama town and suburbs were a long standing problem for the National Water Supply and Drainage Board (NWSDB). Various studies had been done to ensure that the rapidly expanding population of Greater Colombo including Maharagama area were catered with sufficient drinking water but the project was delayed due to number of reasons of which the main one was the non-availability of funds.

A feasibility study for the Rehabilitation/Reinforcement of the Greater Colombo Transmission/Distribution system was carried out by the NWSDB in consultation with W.S. Atkins International of the UK in 1985. The recommendations relating to the Ambatale - Jubilee water conveyance system were approved by the NWSDB and the International Development Association (IDA), and included in the Sri Lanka Water Supply & Sanitation Rehabilitation Project for funding by the World Bank. The consultants Howard Humphreys and Partners commenced work on the detailed designs on 15th January 1987 and submitted the final documents and design report in January 1988.

Since the Maharagama Water Supply Scheme interacts with Ambatale - Jubilee Water Conveyance System considerably, both schemes were combined to form a single contract. The contract was initially estimated for Rs. 1300 million and awarded to M/s Josef Riepl of Germany in June 1991. The contract period was for 03 years and the project was successfully completed in June 1994. Figure 01 gives the works included in the project.

2.1 1100MM DIAMETER DUCTILE IRON TRANSMISSION MAIN

Out of the project components illustrated in Figure 01, this paper focuses attention only on the laying of the water transmission main from Ambatale Water Treatment Plant to the Jubilee Reservoir.

The NWSDB required the design for the Ambatale - Jubilee Water Conveyance System to be 1100mm in diameter and the main to be laid to cross the lake that surrounds Parliament and pass through Beddagana. The reasons for this proposal were that the lake to the south east of Parliament building was to be extended, making the originally proposed route non available, and to extend the project's design horizon beyond the year 2000. Meanwhile the increase in the amount of water that could be pumped through the main to Jubilee would provide an alternative means of supplying parts of Colombo in the event of a failure elsewhere in the city's bulk water supply system.

This route has a length of 8km and original estimate for the laying of transmission main along this was Rs. 455.8 million. A sketch of the route is shown in Figure 02.

3.0 SOIL INVESTIGATIONS FOR THE PROPOSED ROUTE

It had been decided to undertake the drilling of boreholes (BH) along the proposed pipeline route from Ambatale Treatment Plant to Jubilee reservoir in order to carry out soil investigations.

3.1 Field Investigations

During the first stage of the investigation 12 boreholes numbered BH1 to BH12 were advanced along the proposed trace for 1100mm diameter transmission main, as shown in Figure 02. The trace followed the new road constructed leading upto the Parliament. Drilling for boreholes BH1 to BH10 was carried out from the top of the road whilst the drilling for boreholes BH11 and BH12 was carried out from the Parliament lake.

Standard Penetration Tests (SPT's) were carried out at 1.5m intervals. Undisturbed samples were collected (when possible) from the clayey and peaty soils. Disturbed samples from the SPT tubes were also collected for laboratory investigation.

These investigations showed the presence of very weak peaty soils whose location and thickness varied considerably. The extent of this peaty soils would be a determining factor in the foundations for the pipeline, and therefore some futher investigations were done at intermediate boreholes numbered BH-3A, BH-6A, BH-7A, BH-7B, BH-10A, BH-11A and BH-12A. (Boreholes 10A, 11A and 12A were carried out in the Parliament lake)

3.2 Soil Profiles

The sections showing the soil profiles of the ground across BH1 to BH5; BH5 to BH7B and BH7B to BH12A are given in Figures 3A, 3B and 3C respectively. The results revealed that the road has been filled with a lateritic fill, the largest thickness being around BH7A where the fill is 3.0m thick.

The sub soil of certain sections of the route was found to be very compressible and having time dependent settlement characteristics. Those were the peats and soft clays. The peats are the most compressible and the thickness of these layers at the borehole locations are given in Table - 01.

It was noted that the thickness of the soft clay or peat found at very shallow depths varies from 0.5m to 6.05m. The proposed invert levels of the pipeline also lie in these soft layers of soil.

3.3 Laboratory Investigations

Detailed laboratory investigations had been done at the soil mechanics laboratory of the University of Moratuwa. Consolidation tests and triaxial tests had been done on undisturbed samples taken from the peat and soft clays which could be sampled. Other tests done on the disturbed samples of peat and clays were : Atterberg Limits, Particle size distribution, natural moisture content, specific gravity and organic content.

4.0 SETTLEMENT OF FOUNDATIONS OF THE PIPELINE

When foundations for the pipeline in compressible ground were considered, one of the most important characteristics considered was the settlements that may occure during the lifetime of the pipeline. It was observed that filling of the new road to Parliament was still continuing. This would cause further settlements which could be categorised as;

i) Primary consolidation settlement which takes place with the dissipation of pore water pressure.

ii) Secondary consolidation settlements which is due to creep.

iii) Settlements due to the lateral movement of the peat due to filling.

The settlements due to the filling of the road could be much larger than the settlements due to either the traffic loads or the loads from the pipeline. As the NWSDB was not in a position to restrict the on going filling there would be no control on the actual settlement. This was a major consideration to be taken into account in the design.

Considering the high foundation costs involved for a length of more than 3.5 km, NWSDB decided to abandon the proposed route and to select an alternative route.

5.0 NEW ROUTE

Deciding to select a new route did not solely depend on the weak soil properties observed along the previous route. Avoiding the pipe laying across the lake around the Parliament and keeping the pipeline away from the Parliament security zone etc: were also taken into consideration before deciding to change the route of the pipeline so that it would be convenient to NWSDB to maintain the pipeline without much hindrance.

5.1 Investigations for the New Route

NWSDB proposed to lay the transmission main along the same route of the existing 1000mm diameter cast iron pipeline. This pipeline had been laid 30 years back and was still standing solidly without giving much difficulties to maintain. This is the major gravity main that carries water from Ambatale Treatment Plant to the Dehiwela Reservoir from which the Colombo South area is being fed.

When the soil investigations were carried out for this route it was found that similar type of weak soils were present along sections of the route. The total length of the route was 9.3 km and the weak sections were identified. But considering the various factors like easy access for maintenance, avoiding cross country situations, moving away from the Parliament security area and avoiding the lake crossing etc, NWSDB decided to go-ahead with laying of transmission main along this route even though there were sections which contained weak sub-soils.

5.2 Alternatives for Pipe Laying Across the weak sections of the new route.

Following alternative methods were selected after overall evaluation by giving top priority to economic, safety and durability conditions

5.2.1 Surface Treatment

For ensuring the mobility of heavy construction equipment and also to protect local failures of sub surface soil a combination of coarse aggregate in different sizes on geotextile sheets were adopted prior to the laying of pipes. This method was suitable enough to improve the moderately weak ground conditions. Dewatering was carried out to lower the water table to facilitate construction. Settlements were monitored by installation of 75mm diameter PVC pipes with settlement plates placed on the DI transmission main that have been laid. This was done continuously while carrying out the backfilling and compaction for the trenches.

The Geotextile used was a permeable synthetic membrane which has a breaking load of 12 KN/m and 35 l/m^2S of permeability. It can be used in the processes of separation, drainage and erosion control. Geotextile material is normally available in standard rolls. What was used in improving the sub soil conditions in the pipe line trench had a width of 4.5 m, length of 100 m and the weight of a roll was 95 kg. According to the specifications provided, when widths or lengths greater than those supplied on one roll are required jointing is normally effected by simple over lapping. However, depending upon application, sub grade conditions, material loading, convenience and cost, alternative methods such as pegging, sewing, stapling or gluing may be used.

The used Geotextile material contained 70% of polypropylene and 30% polyethylene. It is alkali resistant to all naturally occuring soil alkalic and acid resistant to all naturally occuring soil acids. It is unaffected by bacteria, fungi etc. Since it is not a source of nourishment rats and termites will not eat the geotextile as food. The tensile strength of geotextile decreases with increase in temperature, but recovers fully when the fabric is returned to normal ambient temperature.

The locations at which the geotextile was used along the pipe line are illustrated in figure 04.

5.2.2 Self Anchoring Joints

Pipe laying is an important civil engineering construction which involves lot of restrictions and limitations. It is the duty of the Engineers to engage in this excercise whilst giving least inconvenience to the general public and without damaging the existing utilities.

When pipes were to be laid along steep slopes which has very soft sub soil characteristics and the access to the location is difficult, alternative methods have to be adopted. In Ambatale- Jubilee water conveyance system NWSDB came across this type of situation at several locations. At chainage 1979 to 2036 and chainage 2728 to 2801, 1100mm diameter pipeline had to be laid in a very steep slopy area. Relatively soft soils were also present along these chainages.

At these locations,the existing 1000mm diameter cast iron pipeline was supported on piers just 3m away from the proposed pipeline route. Therefore using of piles to support the pipes was ruled out in order to protect the existing pipeline as well as to prevent any damages that could occur to the houses built nearby. It was noted that access to these locations was also very difficult.

Therefore, self anchoring pipes were the obvious choice for pipe laying in these areas.

"STANTON" self anchoring pipes were used to overcome these problems. Joints of these pipes were designed to achieve the capability to eliminate the usage of concrete blocks to anchor the pipes. These pipes were manufactured using standard production tyton or stantyte ductile iron spun pipes with the addition of one loose socket flange and one welded spigot flange. Anchorage is provided by bolting between the two tie flanges after making the joint in the usual way. 1100mm diameter ductile iron self anchoring pipes designed for a working pressure upto 25 bars and which can withstand a deflection of upto four (04) degrees were used.

Figure 05 illustrats the sections along which the self anchoring joints were laid in 1100mm diameter transmission main.

5.3 Cost Comparison Between Two Ground Improvement Methods

1800 square meters of Geotextile material have been used along 400 m of length for the pipeline and the supplying cost of Geotextile material per square meter was Rupees Forty Eight and Twenty Eight Cents (48.28) and the laying rate for same was Rupees Ninety and Sixty Cents (90.60), according to the contract.

The gross supplying cost of a self anchoring joint was Rupees Sixty Six Thousand Five Hundred and Eleven (66,511/=), but the laying cost for self anchoring joints varies from place to place due to influences such as weak ground, difficult access and the type of fittings for the joints to be used from joint to joint. All the costs involved in overcoming the difficulties to prepare the background to lay a self anchoring joint had to be considered as the expenditure to fix a self anchoring joint to the pipeline. 69 Nos. of self anchoring joints have used in the pipeline at shown locations in Fig.05.

6. Piling

It was noticed from the beginning that highly compressible peat or organic clay would be met in the low lying marshy region east of the city.

Since most sections of the pipeline in this area was to lie along major roads and the pipeline should last in the ground for generations, NWSDB was very particular in preventing the pipeline from any subsequent settlements after laying. Even maintaining the pipeline would have been difficult if differential settlements occured in the pipeline subsequently. It was also observed that the pipes had settled considerably at certain locations of the pipeline along the Kaduwela Road. The settlement that had taken place at Koswatte Junction was observed as 300 mm. after 10 months. Settlement of the pipeline at Thalangama bus deopt was reported as high as 350 mm after a year. Therefore, the NWSDB proposed to lay the pipes along selected sections of the road on piles, and pre-cast concrete piles were selected.

When driving the precast concrete piles for the laying in these areas, it was found that 10 m length pipes were not sufficient sometimes. In the Baddegana valley even the 20m length piles had gone it's full length into the ground. This also had proved the ground improvement could not be done for those sections easily and economically. Even when improving the ground one cannot predict about the occurrence of subsequent sub soil failures if the conditions are not fully controlled when improving the ground.

Previously used soil improvement methods like introducing the geotextile material under the compacted aggregate fill or using self anchoring joints were not identified as reliable alternatives to the piles.

After the piles were installed, a few selected pipes were subject to load tests. Appendix I illstrates the results of load tests.

7. Conclusions

It is evident from above that most convenient and cost effective method of ground improvement is the using Geotextile material under a compacted fill. As the laying of a large diameter pipeline has special significances of it's durability and operation and maintenance works, other alternatives such as introducing self anchoring joints and piling were to be adopted appropriately. Although the pipeline construction involves a relatively lesser area across the roads, restrictions are much more considerable. Allowing the traffic flow with giving less inconvenience to the public, not damaging the existing utilities, preventing the road failures and safe guarding the pipeline are imparative constraints that should be given priority consideration. Eventhough it was relatively expensive, introducing self anchoring joints and laying pipes on the piles had to be adopted due to unavoidable conditions which could give rise to sub soil failures.

Ambatale-Jubilee water conveyance system designed to cater the millions of consumers living in the Greater Colombo Area including Maharagama. Once the water is provided through the system it should be reliable through out it's design period. As the empowered drinking water Authority of Sri Lanka, it is the responsibility of NWSDB to adopt most suitable and durable methods when laying such transmission mains ensuring this reliability to the people. If a failure occures in a large diameter pipe line such as Ambatale-Jubilee water conveyance system which spans over 9.3km through out the important cross sections of the city, damage which could be created be much severe. Having foreseen such an unpredictable situations, measures had to be adopted to prevent the uncertenities in order to minimise the failures of the pipeline and ensure the reliability of supplying the safe drinking water to the people.

Although the using geotextile material would be a low cost method to develop weak ground these constraints also should be given more consideration when laying of large diameter pipes in weak ground under the main roads keeping minimum cover as only 01m over the pipeline upto the surface level of the road. Therefore, certain sections of the pipeline had to be rested on precast concrete piles driven to the hard bed strata.

8. Acknowledgements

The Author wishes to thank Mr. K.M.N.S. Fernando, Deputy General Manager (IDA) of NWSDB for the encouragement he has given while making this paper. The Author gratefully acknowledge the guidance and comments given by Prof. B.L. Tennekoon of University of Moratuwa.
The association with Mr. U. Ratnapala, Deputy Resident Engineer of the Project has given the Author strength and courage to present this paper.

Results of Load Tests

Three test piles were driven at Parliament Junction (TP-1) at Koswatte Hospital (TP-2) and at the Baddegana Marsh (TP-3). Each of this precast concrete piles had a cross section of 300 mm x 300 mm and was of length 10 m. The piles had been designed for a working load of 30 Tons based on concrete strength. The final set measured for the 03 piles in cm per blow were 0.5, 0.24 and 0.60 respectively. It should be noted that two of above three locations situated adjacent to the flood retention basins maintained by Sri Lanka Land Reclamation and Development Co-operation. Very thick peat layers were observed in the sub soil at the Parliament Junction and Baddegana Valley. The Zero SPT values achieved at shallow depths when boring was done in these area.

Following results were observed from the load tests.

Pile TP-1

When load	=	30 Tons;
Maximum Settlement	=	4.485 mm
Residual Settlement	=	1.23 mm

Hence pile can carry this load safety

When load	=	40 Tons;
Maximum Settlement	=	19.2 mm
Residual Settlement	=	15.5 mm

From consideration of the residual settlement, the pile is inadequate to carry this load.

Pile TP-2

When load	=	40 Tons;
Maximum Settlement	=	6.32 mm
Residual Settlement	=	2.51 mm

Hence pile can carry this load safety

Pile Tp-3

When load	=	30 Tons;
Maximum Settlement	=	8.1 mm
Residual Settlement	=	7.6 mm

From consideration of residual settlement, the pile is inadequate to carry this load.

When, Load = 39 Tons, very large total settlement taken place.

As the tests reported, TP-1 and TP-3 had failed. Test pile TP-1 was loaded to 40 tons, whilst TP-3 had an ultimate load carrying capacity of 39 tons. All the test piles had a design working load of 30 tons based on concrete strength. Therefore it was concluded that the piles which failed have most probably failed as a result of ground failure. Results of monitered load tests are shown on figure 06.

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(i) Contract 3/1 : Ambatale - Jubilee Water Conveyance System Design Report; Prepared by Howard Humphreys and Partners - U.K. - 1988.

(ii) Geotechnical Practices in Difficult Ground conditions; Paper presented by Prof. A. Thurairajah on 07th April 1993 to a seminar organised by the Sri Lanka Geotechnical Society (SLGS).

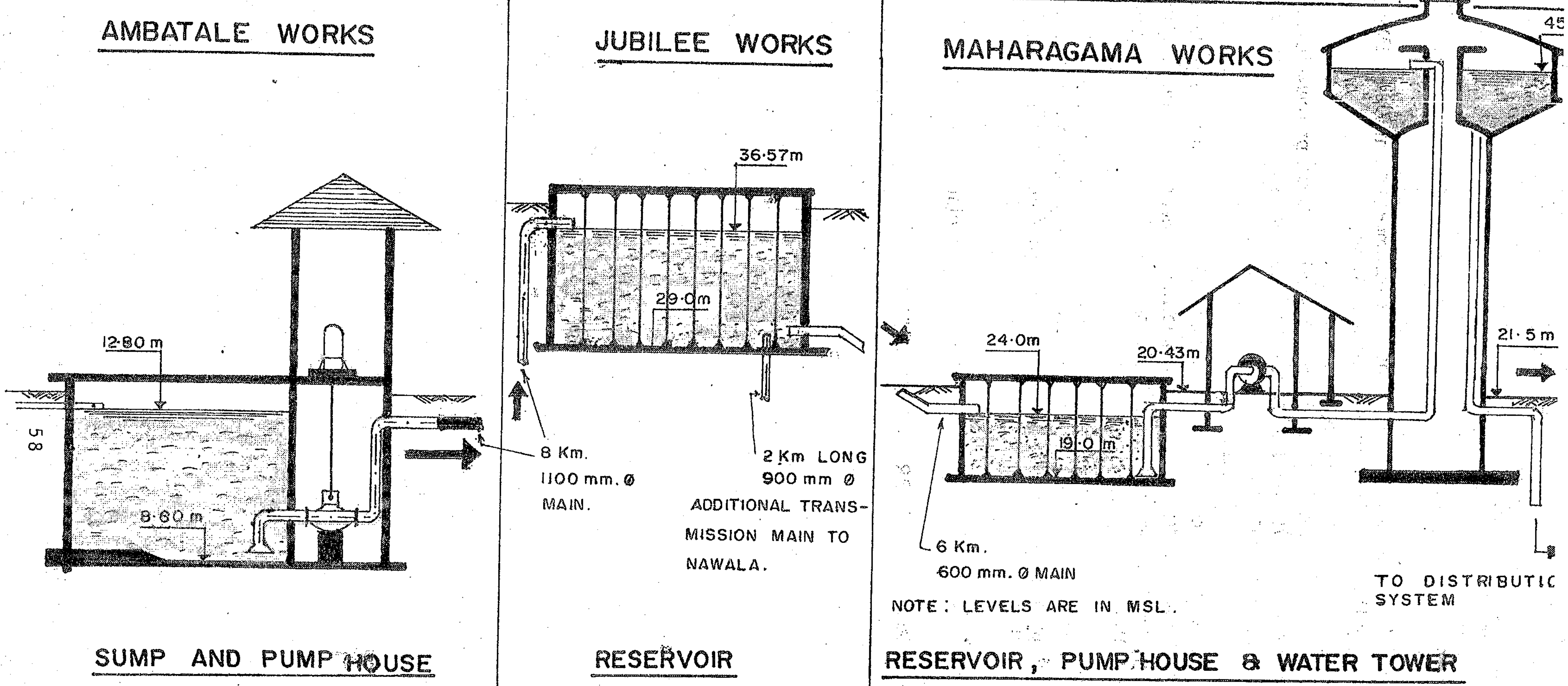
(iii) Site Investigation for the Foundations of the pipe line for the Ambatale - Jubilee water conveyance system from the Koswatte Road Junction upto the Parliament lake along the new road to Parliament; Report of Prof.B.L.Tennakoon prepared for NWSDB on 12th November 1991.

(iv) Evaluation of load carrying capacity of Driven Pre-cast concrete piles; paper presented by Prof. B.L. Tennakoon to a seminar organised by SLGS on 13th December 1994.

(v) STANTON Catalog; for Ductile Iron Self anchoring joints.

(vi) REHAU Catalog : On TERRAM PRODUCT 1500.

AMBATALE - JUBILEE WATER CONVEYANCE SYSTEM & MAHARAGAMA WATER SUPPLY SCHEME - CONTRACT 3/1



PROPOSED EXTENSION WORKS AT AMBATALE	8 Km LONG 1100mm. DIA. D.I. TRANSMISSION MAIN.	PROPOSED 9000 m. R.C.C. RESERVOIR	6km. LONG. 600 mm. DIA. D.I. GRAVITY TRANSMISSION MAIN.	PROPOSED $3600\,m^3$ GROUND RESERVOIR BOOSTER PUMP HOUSE & GENERATOR ROOM AND $954\,m^3$. WATER TOWER. 45 Km. DISTRIBUTION NET WORK.
COST - RS. 89·57 mill.	RS. 455·8 mill. (ORIGINAL PROPOSAL)	RS. 134·8 mill.		RS. 378 mill.

FIG. O

LOCATION OF BOREHOLES

NOT TO SCALE

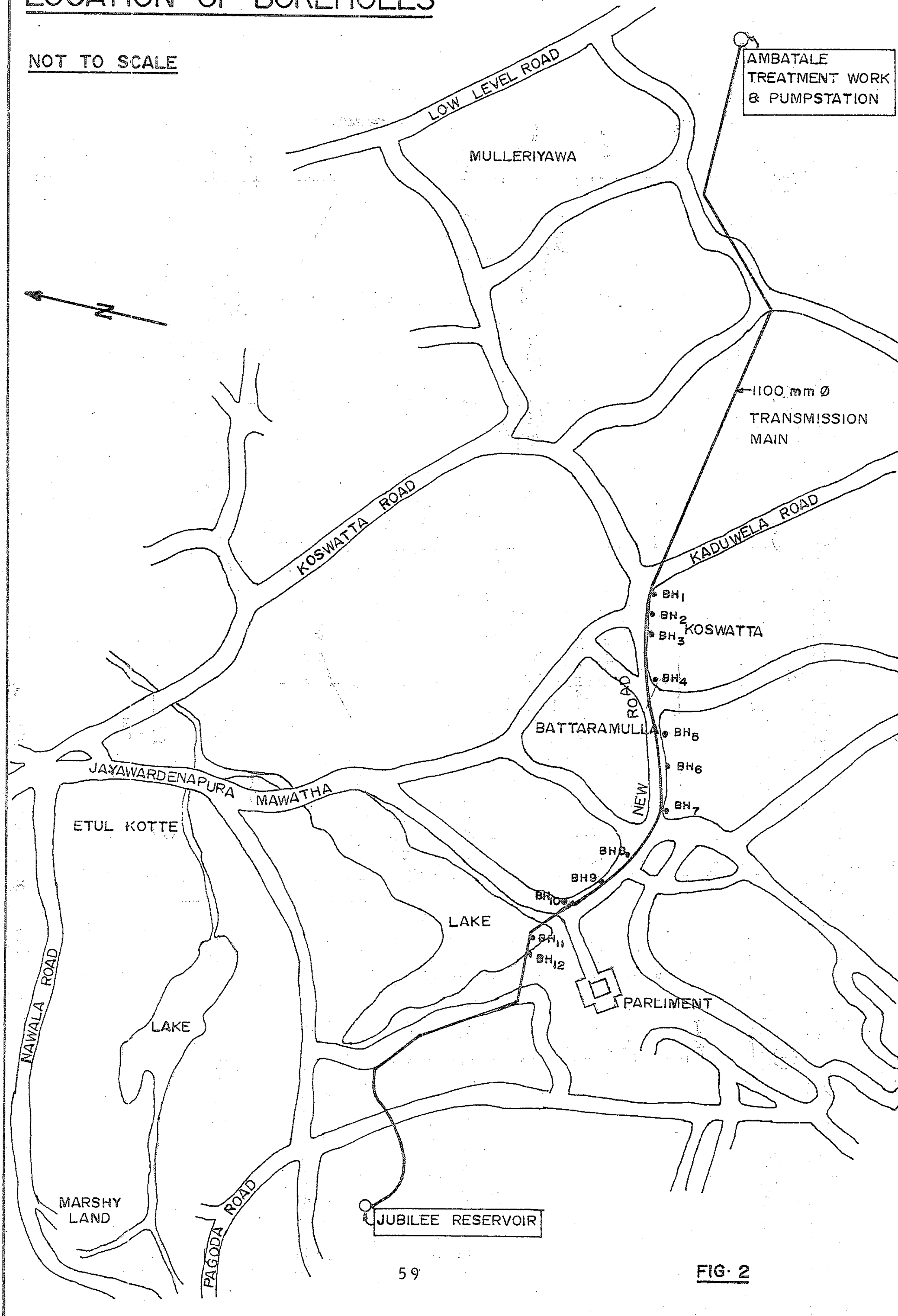


FIG. 2

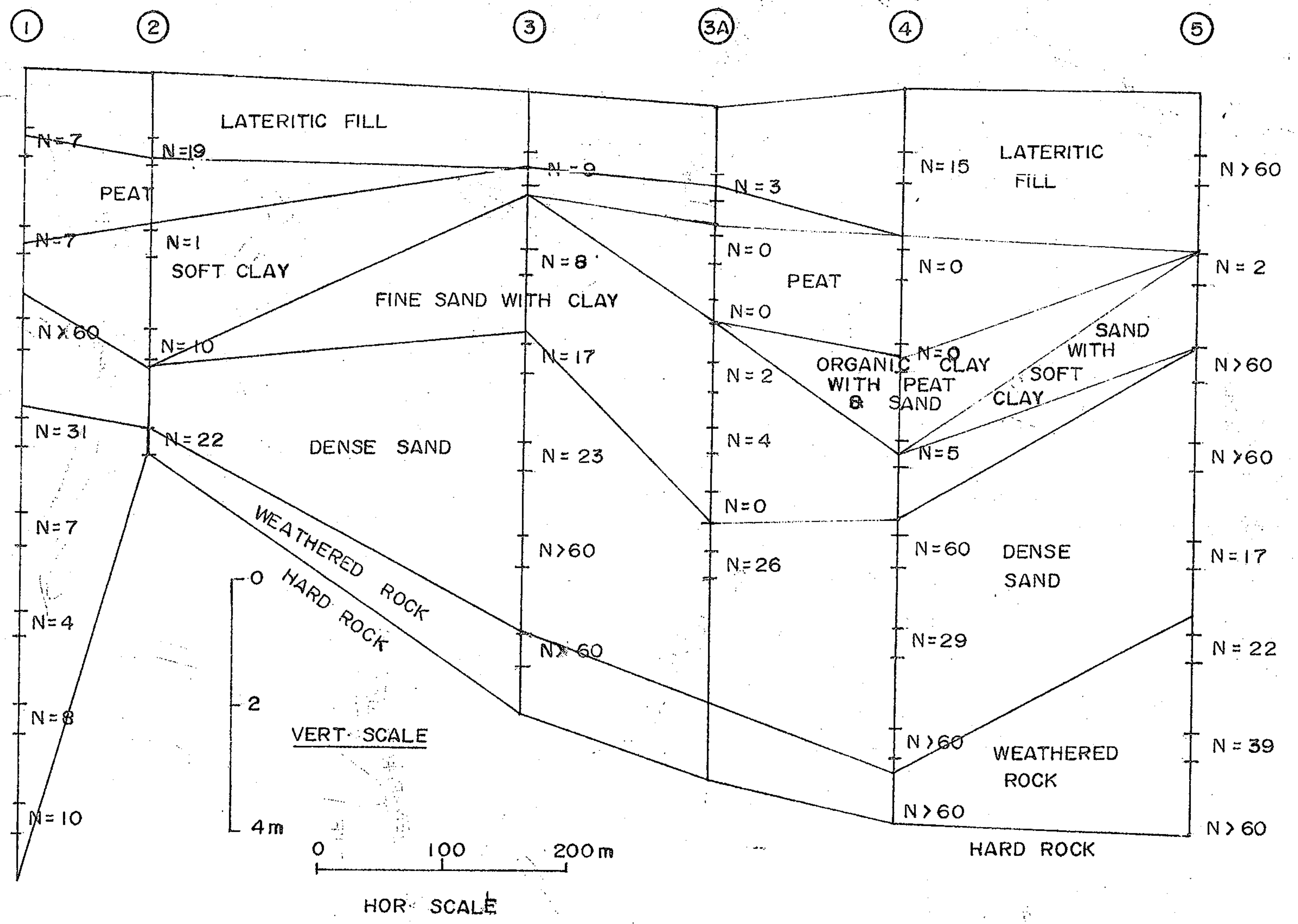
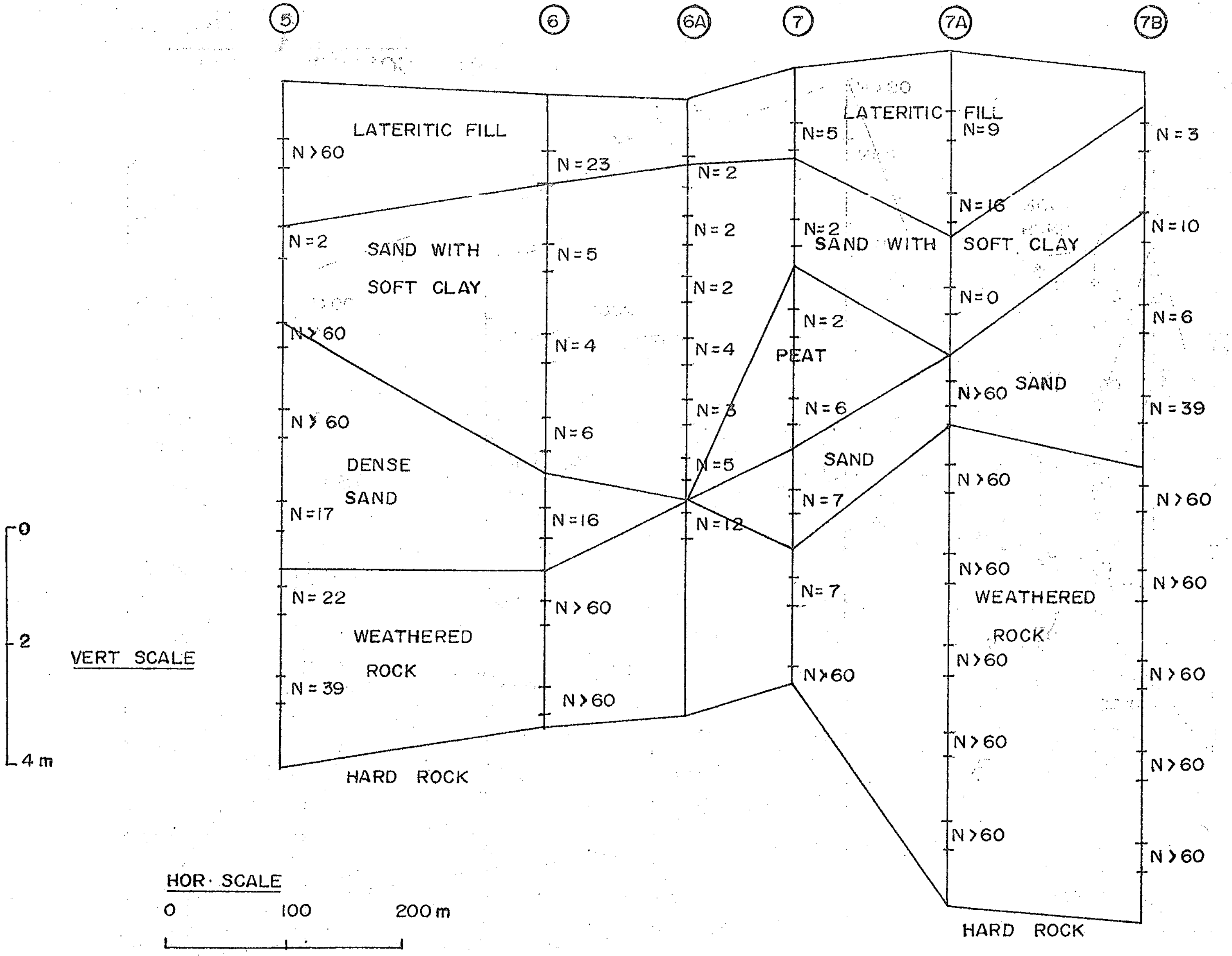


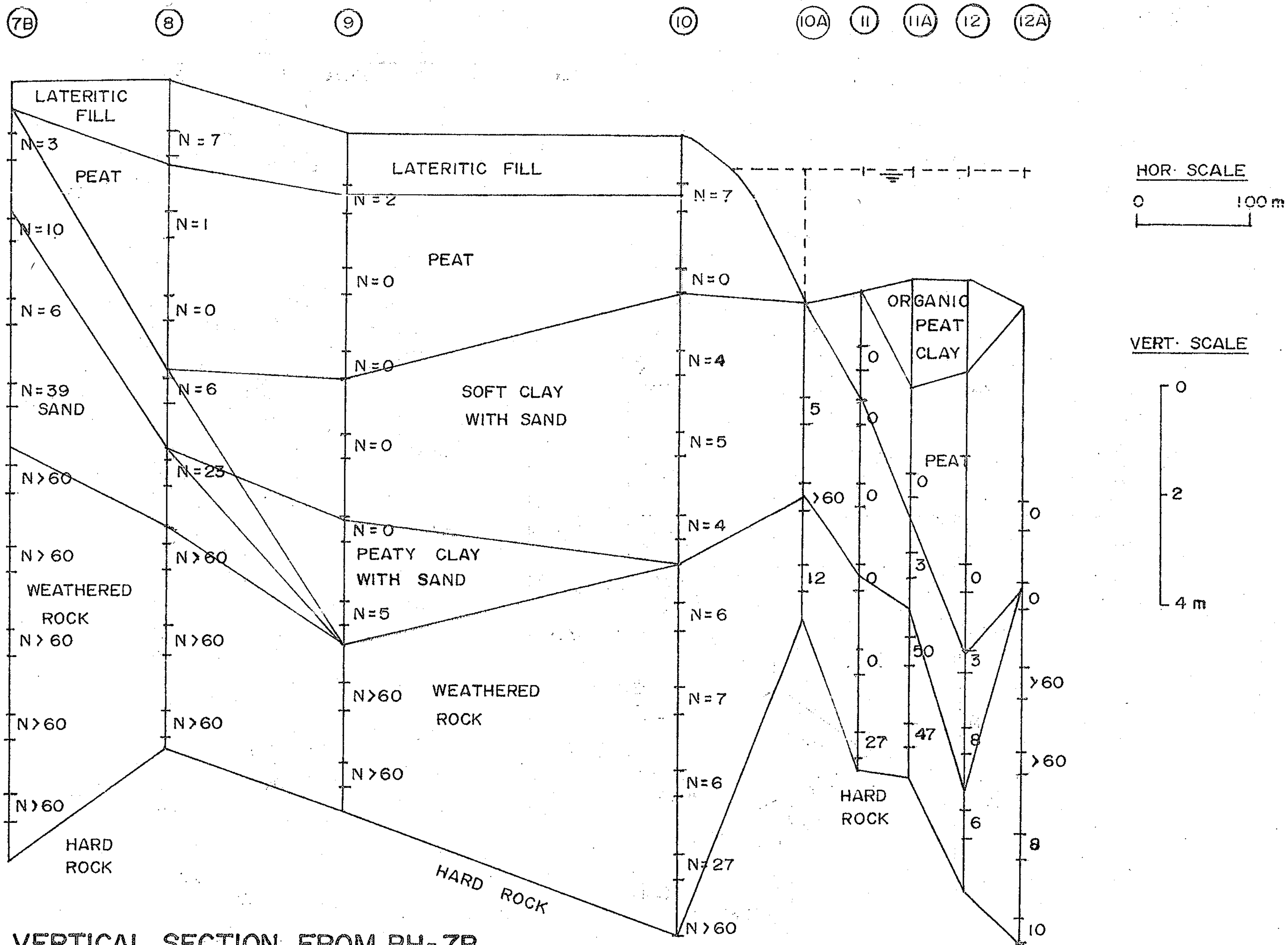
FIG 3A

VERTICAL SECTION FROM BH-1 TO BH-5



VERTICAL SECTION FROM BH-5 TO BH-7B

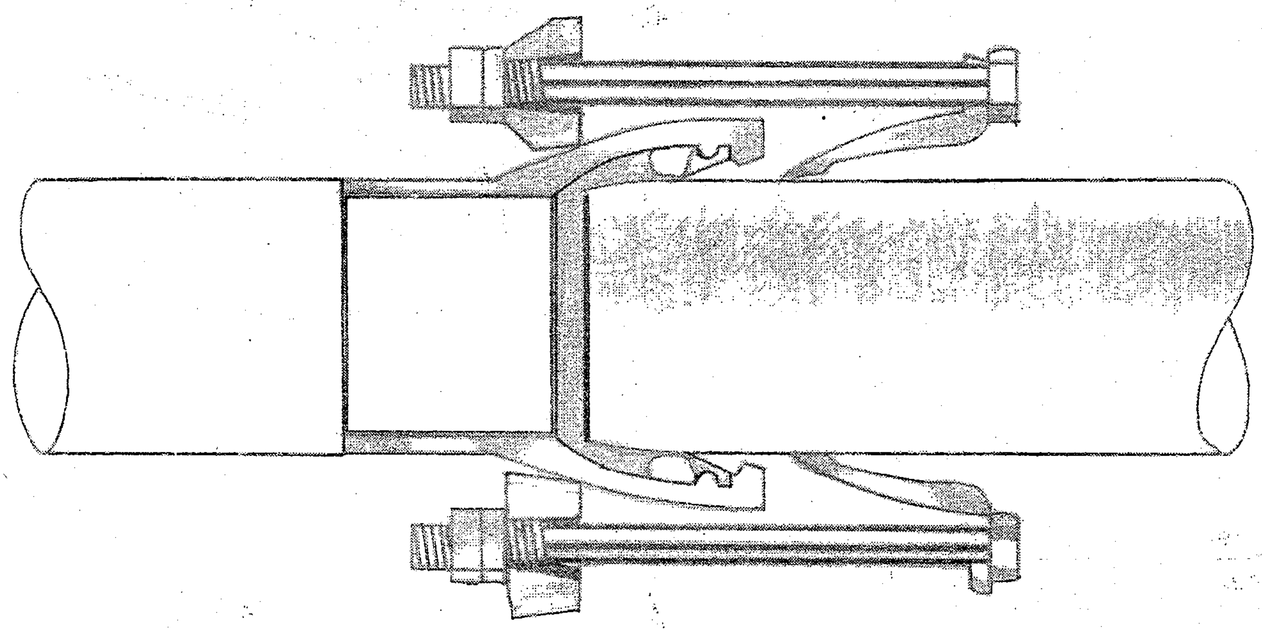
FIG 3B



VERTICAL SECTION FROM BH-7B TO BH-12A

FIG· 3C

SELF ANCHORING JOINT

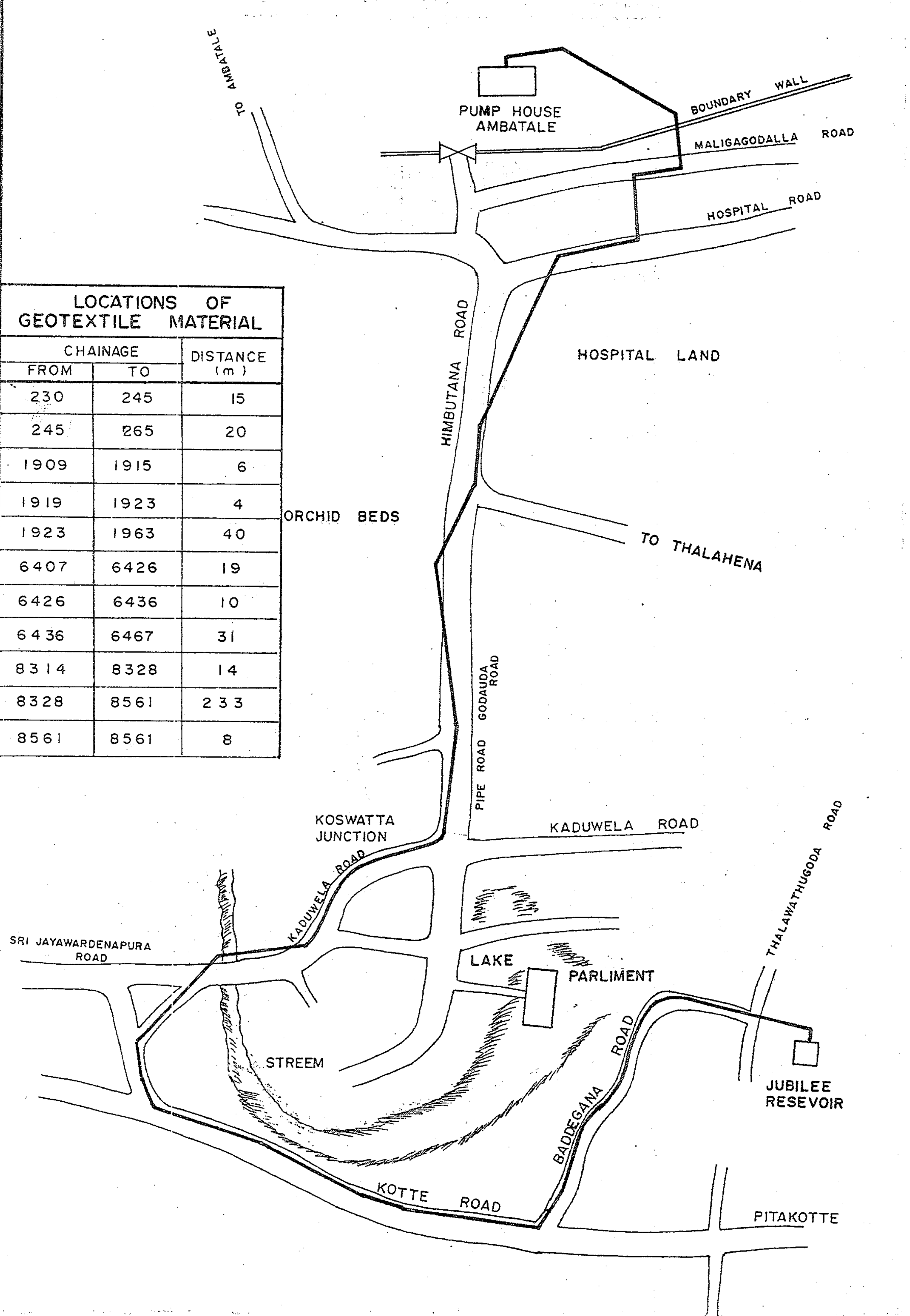


BH No.	Thickness of peat layer (m)
BH - 1	1·65
BH - 2	1·10
BH - 3	-
BH - 3A	1·55
BH - 4	3·65
BH - 5	-
BH - 6	-
BH - 6A	-
BH - 7	3·10

BH No.	Thickness of peat layer (m)
BH - 7A	-
BH - 7B	-
BH - 8	3·70
BH - 9	5·65
BH - 10	1·85
BH - 10A	-
BH - 11	2·05
BH - 11A	4·50
BH - 12	6·65
BH - 12A	1·65

TABLE 01

NEW ROUTE OF 1100 m.m DIA. TRANSMISSION MAIN



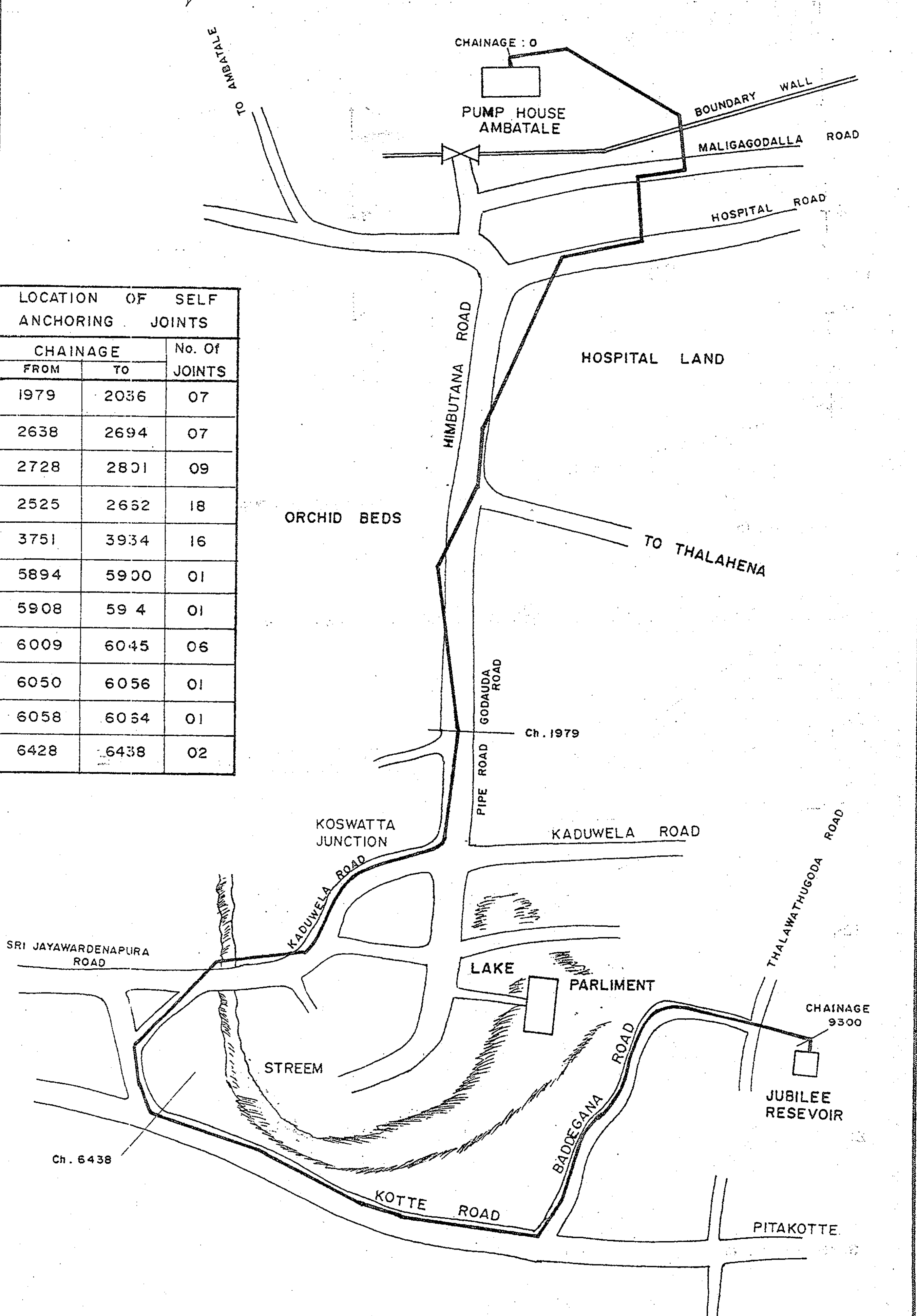
LOCATIONS OF GEOTEXTILE MATERIAL

CHAINAGE		DISTANCE (m)
FROM	TO	
230	245	15
245	265	20
1909	1915	6
1919	1923	4
1923	1963	40
6407	6426	19
6426	6436	10
6436	6467	31
8314	8328	14
8328	8561	233
8561	8561	8

SCALE :- NOT TO SCALE

Figure 04

NEW ROUTE OF 1100 m.m DIA. TRANSMISSION MAIN



LOCATION OF SELF ANCHORING JOINTS		
CHAINAGE		No. Of JOINTS
FROM	TO	
1979	2036	07
2638	2694	07
2728	2801	09
2525	2662	18
3751	3934	16
5894	5900	01
5908	59 4	01
6009	6045	06
6050	6056	01
6058	6064	01
6428	6438	02

SCALE :- NOT TO SCALE

Figure · 05

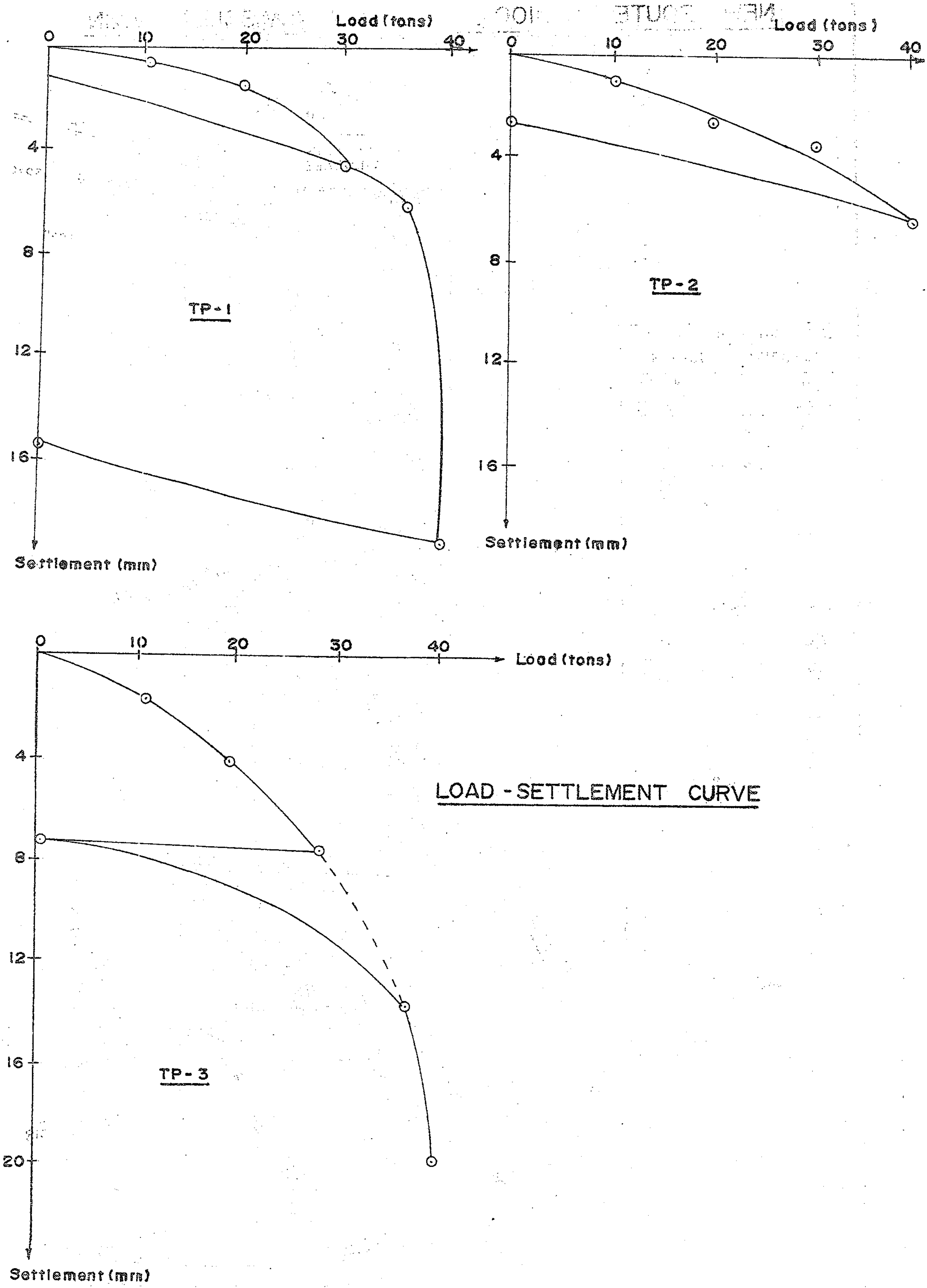

Load (tons)
0
10
20
30
40
4
8
12
16
TP-1
Settlement (mm)
Load (tons)
0
10
20
30
40
4
8
12
16
TP-2
Settlement (mm)
0
10
20
30
40
Load (tons)
4
8
12
16
20
TP-3
Settlement (mm)
LOAD - SETTLEMENT CURVE

Figure. 06

SECTION OF PIPELINE WITH GEOTEXTILE

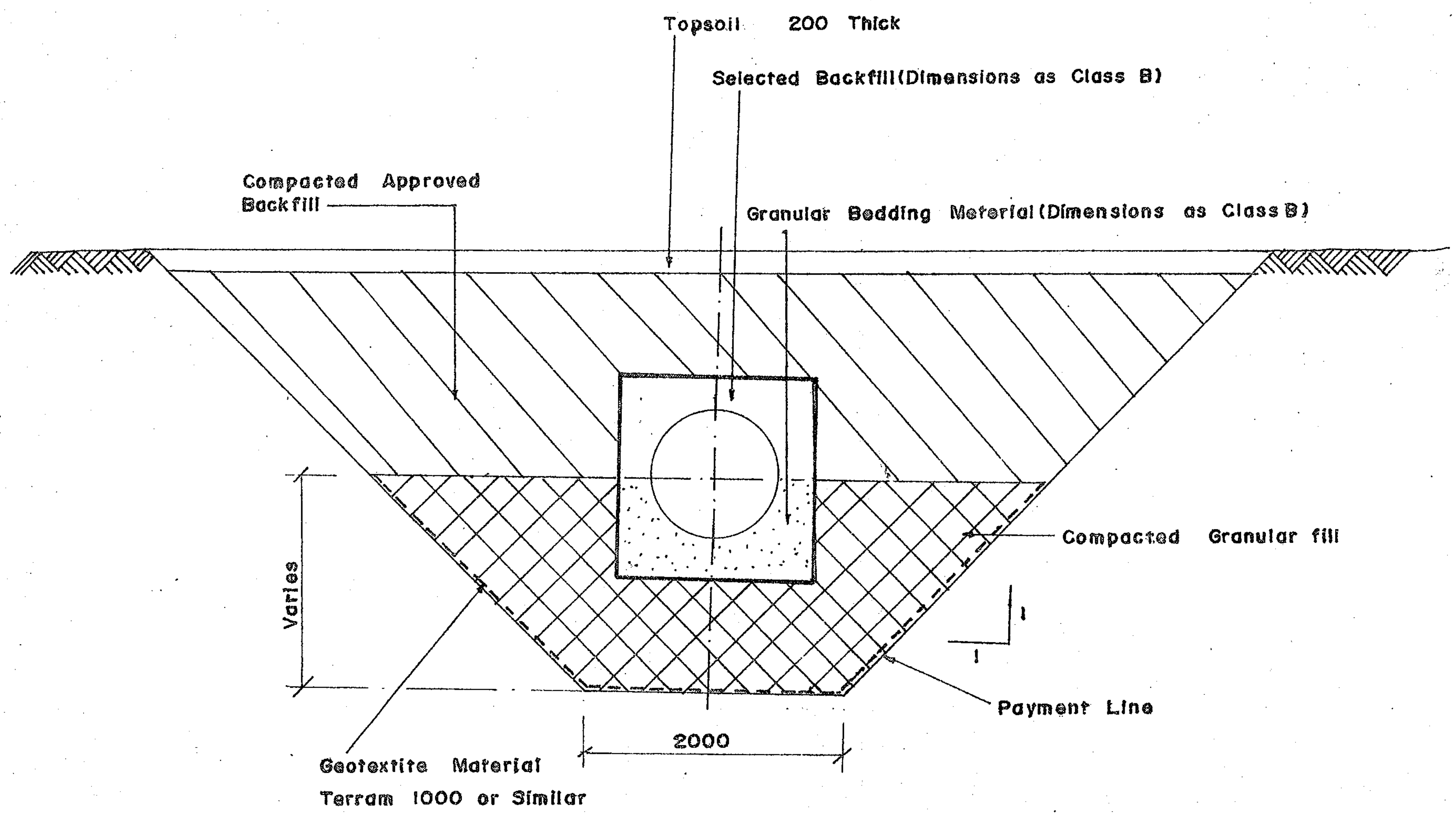


Figure 07

Geotechnical Investigations for Yan Oya – Padaviya Agricultural Extension Project and Proposed Ground Improvements for Dam and Spillway Foundation

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Preamble

The present development area under Padaviya Irrigation Project has been experiencing continues hardships due to shortage of water which has lowered the cropping intensity considerably.

The Yan Oya - Padaviya Agricultural Extension Project will generate direct benefits to the people of the area by increased agricultural production, inland fishery and opportunities of employment.

The feasibility studies of the Yan Oya Agricultural Extension Project was undertaken in order to explore the possibilities of upgrading the Irrigation in the Padaviya command area to redress the problem by extending the irrigation facilities to adjoining basin. The feasibility studies of the project was undertaken by Central Engineering Consultancy Bureau in 1992 April and project report was submitted to Ministry of Lands Irrigation and Mahaweli Development in 1994 using local resources and expertise. This paper deals with the results of investigations and suggested ground improvements for the main dam foundation and spillway.

Location

The Yan Oya catchment is located in the North-East of Sri Lanka between the river basins of Ma Oya (where Padaviya reservoir is located) and Kanchikubukkan Aru. The Yan Oya originates from Sigiriya area and drains about an area of approx. 1500 km2 during its 150 km long passage to Indian Ocean, South of Pulmodai (North Eastern sea cost). The proposed dam across Yan Oya is located near village Pangurugaswewa within the irrigation commend area of Wahalkeda Tank in the North-East Provincial Council. The main access from Colombo is through Kurunagala, Dambulla, Mihinthale, Medavachchiya, Kebithigollawa and approximate distance is about 250 km.

Project features

The proposed earthfill dam across Yan Oya will be a homogeneous earthfill dam which is 2350 m long and 20 m high at the highest point (above the deepest river bed level). The dam top level is 42.25 m.a.s.l. with a top width of 8 m. There will be two earthfill saddle dams on right bank with a total length of about 3200 m and maximum height is approximately 10 m. An ungated ogee type spillway will be constructed at the right bank 1st saddle with a total water way of 544 m and crest level will be 38 m.a.s.l. Two bottom outlets will be provided under the spillway to regulate the reservoir level.

The conveyance canal from the reservoir up to Jayanthiwewa has a maximum design capacity of 12 cumecs at the head end which will be reduced to 6 cumecs at the tailend. The main canal intake is located at left abutment. The canal will be concrete lined throughout and provisions will be made for seven off takes along the canal for irrigating the existing areas and proposed new development areas under the project.

(b) Metamorphic Rock of Highland Complex

The bed rock in the area is consist of metamorphic rocks of Highland complex. The dominant rock type within the project area is charnockite/charnockitic gneiss. Alternating without a clear boundary is amphibolite and cuartz biotite gneiss which is more often found within the charnokitic gneiss layer. Another common rock type in the area is quartzite or quart-feldspar gneiss and granulites, well banded light coloured sometimes gneissic but more often equigranular which are made up chiefly of quartz and feldspar with varying amount of mica and garnets. The quartzites are coarse grained or medium grained rocks, whitish in colour and glassy in appearance and highly jointed or fissured. Even in the aerial photographs parallel bands of quartzites can be traced for miles due to its colour which frequently form prominent ridges. Marbles or metamorphosed crystalline limestones are very rare in the project area.

Discontinuities

An overturned fold was observed during the mapping in which the axis lies towards North-East direction. The above is dissected by highly jointed closely spaced fracture zones with the orientation of North-West or North. Along most of these fracture systems rock is highly techtonized. On slickensided joint surfaces, indications of traces of movements are visible. These technoized fracture zones are mainly connected with a lineament which can be observed in aerial photographs over a length of about 4-5 km, along the river course.

Orientation of foliation gradually changes from left bank to right bank. This change is quite regular and dip angle changes from 70° - 80° to 55° - 65° while dip direction changes form North-West to East.

Other than the foliation two joint sets were observed at the dam site area. Joint set 1 is sub-parallel to the dam axis and dips at steep angles of
80° - 85° towards downstream. This joint set is parallel to above mentioned system of fracture zones. This joint set is well pronounced in the river section about 50m below the dam axis. AT various locations of the project area a second joint set was observed which is oblique to the fold pattern (foliation). This joint set appears to be somewhat less pronounced in the left bank but is more prominent on right bank.

Geotechnical Assessment

Geotechnical investigations were carried out along two dam axes and the upstream axis was preferred against the downstream axis because the foundation conditions were more favourable. The weathering thickness along the downstream axis which is tied down to a weathered quartz feldspar granulite ridge is considerably high and more over water pressure tests results indicate that the permeable rock exist to a depth of 30-40 m below the foundation which will require additional ground improvements to be implemented to such depths.

Therefore more attention was paid to U/S axis which is tied down to a rock spur consist of charnokite. During the course of investigations it became obvious that the most important factor would be the thickness of overburden, mainly the alluvial deposits and its composition. This was proved to be a vital factor in cost benefit calculations because the material found to be unsuitable for the dam foundation. As the cost involved in removal and replacement with suitable material will be a considerable percentage of construction cost of the dam.

Based on the composition, and physical parameters the alluvial material can be grouped into two units. The upper layer which consists of black or dark brown highly plastic organic clay with bands and lenses of clayey silt/clayey sand. The lower layer consists of bands and lenses of undifferentiated light gray or light brown clayey silt or clayey sand or very fine sandy silt. Occasional interbanded layers of gravel also can be observed within this unit. While formation of the first layer may have taken place under conditions of stagnant water within the flood plain or by sedimentation of suspended organic particles the second formation is influenced by velocity of the flowing water. In comparison to first layer, the second layer consists of fever quantity of organic matter. The upper layer is plastic and impermeable but the lower layer is mostly permeable and none plastic or with very low plasticity.

Most notable feature of this material is the variation in quality and composition along the vertical and horizontal directions. Within the alluvial deposits constant head permeability tests conducted in drill holes show permeability values varying from 10-4 cm/sec to 10-5 cm/sec. (Please see Table - 01 - Permeability Test results).

Hill slopes, adjacent areas of both abutment and spillway area are covered with talus material consist of hillside waste and debris up to a depth of 5m.

Results of drill holes indicate that the alluvium or colluvium lies directly on the residual soil layer or completely weathered rock layer and generally the weathering thickness is more towards the abutments. The maximum thickness of overburden including the residual soil layer and completely weathered rock layer is within the main river section and its in the region of 12 m. Bed rock seems to be more or less impermeable but individual zones with high water intakes were evident from water pressure tests conducted in drill holes. The water table draws down to 6-7 m during the dry periods and during the rainey periods rises upto 1m from surface.

Suggested methods for ground improvement of dam foundation and spillway foundation.

On the basis of information obtained during the feasibility studies the following ground improvement methods can be suggested:

(1) Existence of a thick alluvial layer consist of material with relatively inconsistent properties specially the compressibility and permeability was observed as a result of investigations. In order to provide favourable foundation conditions for the homogeneous earthfill dam, construction of a cut off trench up to the hard rock layer will be one of the solutions. But removal of unsuitable material involves deeper excavations (for example; river section) Therefore underground seepage into construction area may pose problems which will be time consuming and lead to extra costs. However bottom of the cutoff trench can be raised suitably towards the abutments. In order to minimize the underground seepage after removal of first layer of alluvium which mainly consists of unconsolidated organic matter a vertical concrete cut off wall with a thickness of 01M, can be constructed within the deepest river section. It will be important to remove all material having organic matter to ensure against undue long term settlements of the dam.

(2) Under the spillway foundation all overburden material will be removed in order to expose the moderately weathered rock on which the spillway structure will be founded.

(3) From the bottom of the cut off wall at the river section a single line graut curtain will be executed. The graut section will have to be water pressure tested and grauted in 3 m sections upto a depth where two consecutive sections are found to be impermeable.

Geotechnical investigation

The data provided for the design is based on the information obtained through Engineering Geological mapping, Aerial photo interpretations, core drilling, bore hole testing (S.P.T. constant head permeability tests, water pressure tests) bore hole instrumentation and test pits. (Please see Fig. 01 locations of drill holes and test pits)

Geological formation in the project area

Site investigations have revealed that the major lithological formations in the region consist of

(a) Quaternary deposits
(b) Metamorphic rocks of highland series

(a) Quaternary deposits

The quaternary deposits are mainly represented by alluvial deposits, colluvium at the vicinity of rock ridges and lake deposits found in the abandoned reservoirs.

Yan Oya area experiences draughts almost every year except during the North-East monsoon period. During the rainy period, Yan Oya overflows and floods the flat valley but during the major part of the year Yan Oya flows sluggishly. Various elements of micro relief is observed within the Yan Oya flood plain due to this pattern of events. (Fig. 03 - U/S Axis and D/S Axis - Geomorphological features).

Very often these elements composed of several different units exist with an amplitude of elevations of few meters very often. It is evident that over the years Yan Oya continued to flow within the flood plain over its own alluvium extensively, developing bends or meanders of varying shape and size. In the course of their development these meanders form loops. During rainy seasons when power of high water increase channels straightens leaving cutoff oxbow lakes. These lakes are often overgrown, silted up and sometimes turned into swamps. The farmers of the area have named the silted up cut off meanders as "EBA". (Fig. 03 - U/S Axis and D/S Axis - Geomorphological features).

Different to above described unit another type of microrelief observed in the flood plain is levee which develops at the vicinity of the channel part as an embankment structure along the banks. The formation of the levee is due to the changes of velocity of the water flow, at the point of transition from the low water channel to the flood plain. Every time the river overflows its banks, stream velocity at the edge of the channel is retarded and as a result the coarser particles drops and develops into a embankment like ridge bordering the river channel.

Other microrelief observed in the flood plain is remnants of earlier structures consist of bed rock or residual soil surrounded or at times fully covered with alluvium. The composition of above units varies from pervious layers, (such as, gravelly sand, silty sand and silt) semipervious to impervious bands (such as clay, silty clay or sandy silty clay and organic clay deposits). At places presence of organic clay, peat like material, loams or sandy loams of organic origin, also is observed. These organic soils take a form of lenses or bands.
(Please see Geological section of Main Dam - Fig. 2)

Lake deposits are found in the abandoned ancient reservoirs and colluvium is observed in the down slope of rock ridges or insulburge structures.

Conclusions

The following conclusions can be drawn from the investigations carried out at Yan Oya site.

(a) Within the dam axis, a thick overburden cover mainly consist of alluvial deposits containing highly compressible organic soils is recorded in drill holes and test pits executed during the studies. The constant head permeability test conducted in drill holes and test pits show a permeability variation ranging from 10^{-2} cm/sec to 10^{-5} cm/sec. This layer will have to be replaced with suitable impervious material of permeability less than 10^{-6} cm/sec. A cut off trench and a cut off wall is proposed to be executed below the dam foundation. However problems of underground seepage water may pose difficult and time consuming problems leading to extra costs.

(b) The other main geological feature which necessitated ground improvements within the dam foundation and spillway foundation is extensive jointing. Major fracture zones have been recorded within the river section and open joints have been observed up to a depth of 30 m below the dam foundation.

(c) Cement grouting will have to be carried out in order to minimize the seepage below the dam foundation and spillway foundation. Method of execution of grauting and other parameters of grauting works (number of drill holes, inclination and graut consumption) will depend on the characteristics of individual joints and fracture systems encountered.

Acknowledgement

The authors wishes to express their gratitude to the management of Central Engineering Consultancy Bureau for granting permission to publish this material. Grateful acknowledgement is made by the authors for the assistance provided by the staff of Central Engineering Consultancy Bureau and National Building Research Organization for preparation of illustrations and Miss. Ramani Kodagaoda for typing the manuscript.

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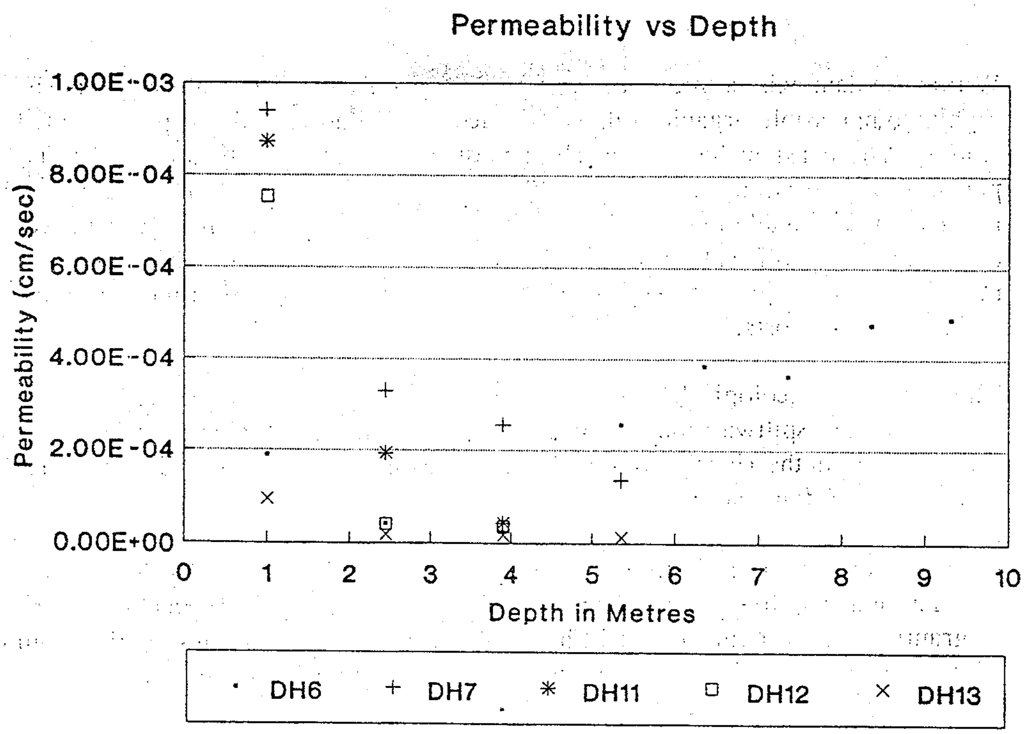
Yan Oya Padaviya Agricultural Extension Project

Permeability Test Results - Downstream Axis



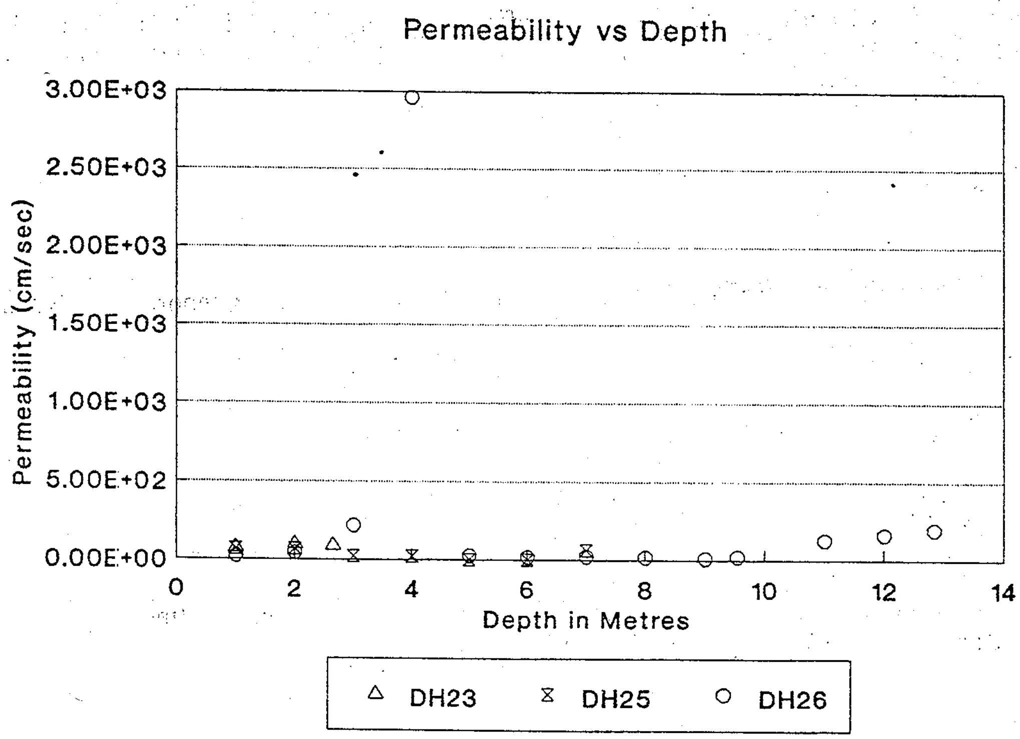


TABLE -01

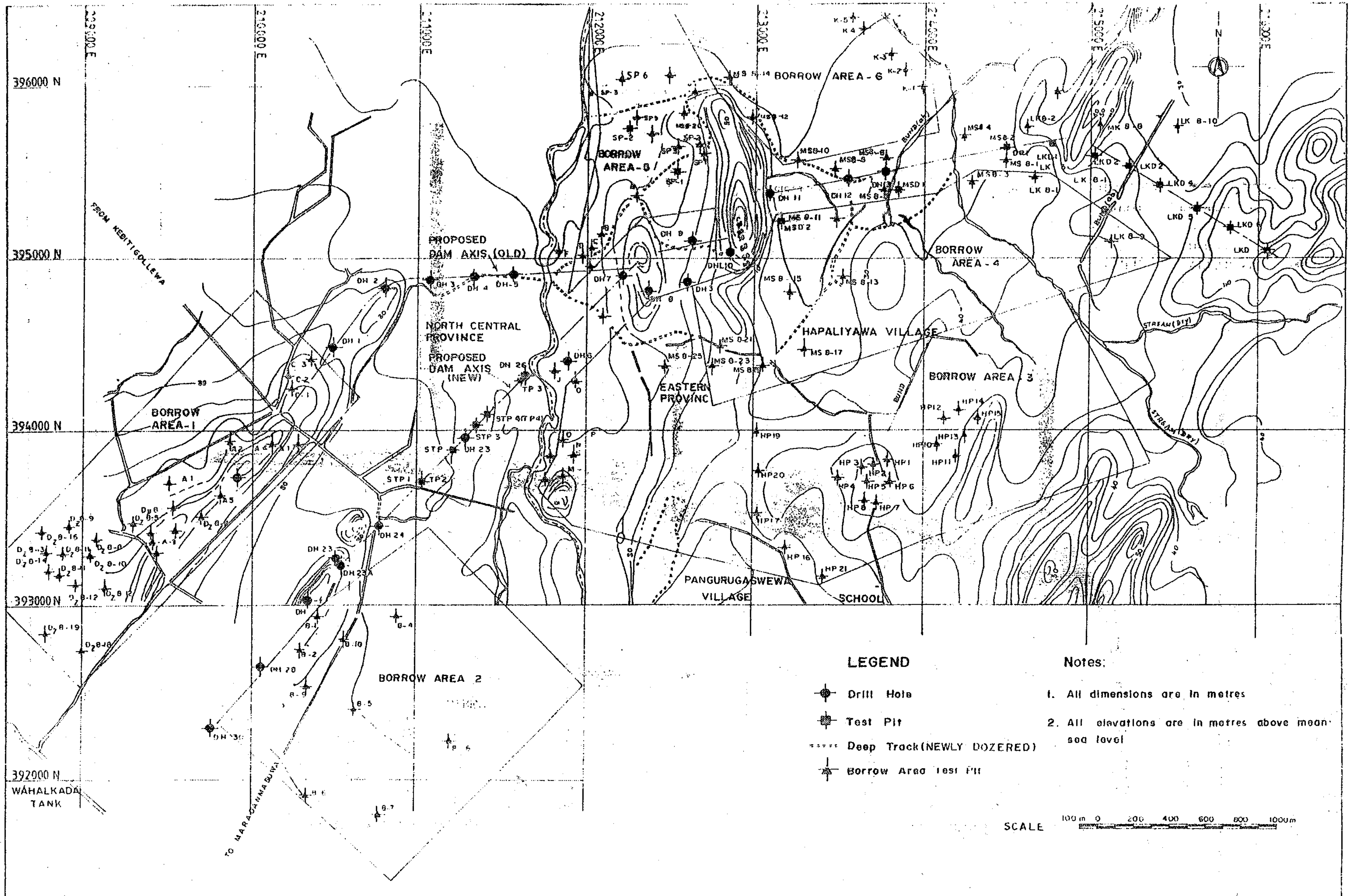
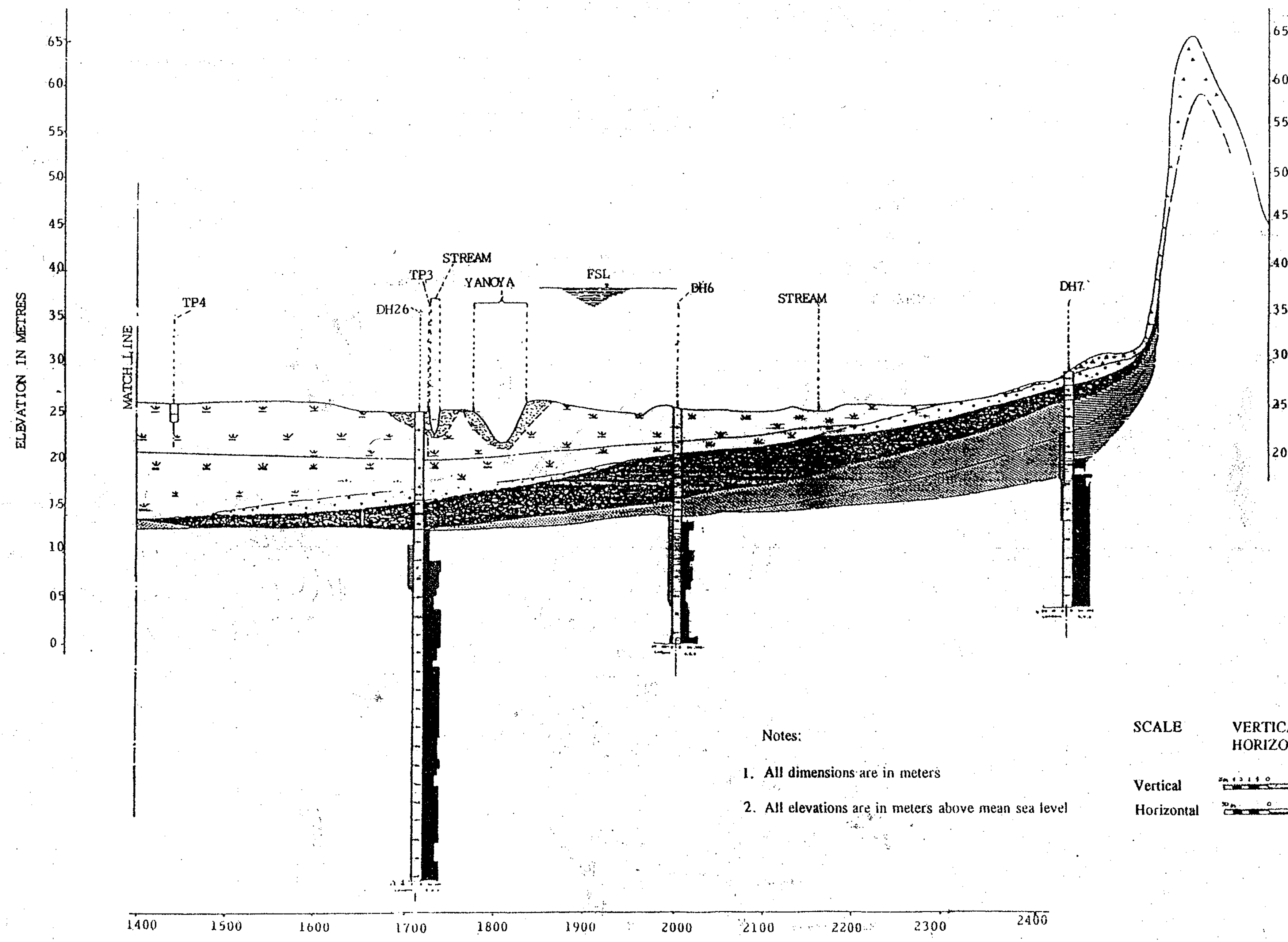


Fig-01 Locations of Drill Holes and Test Pits



Notes:

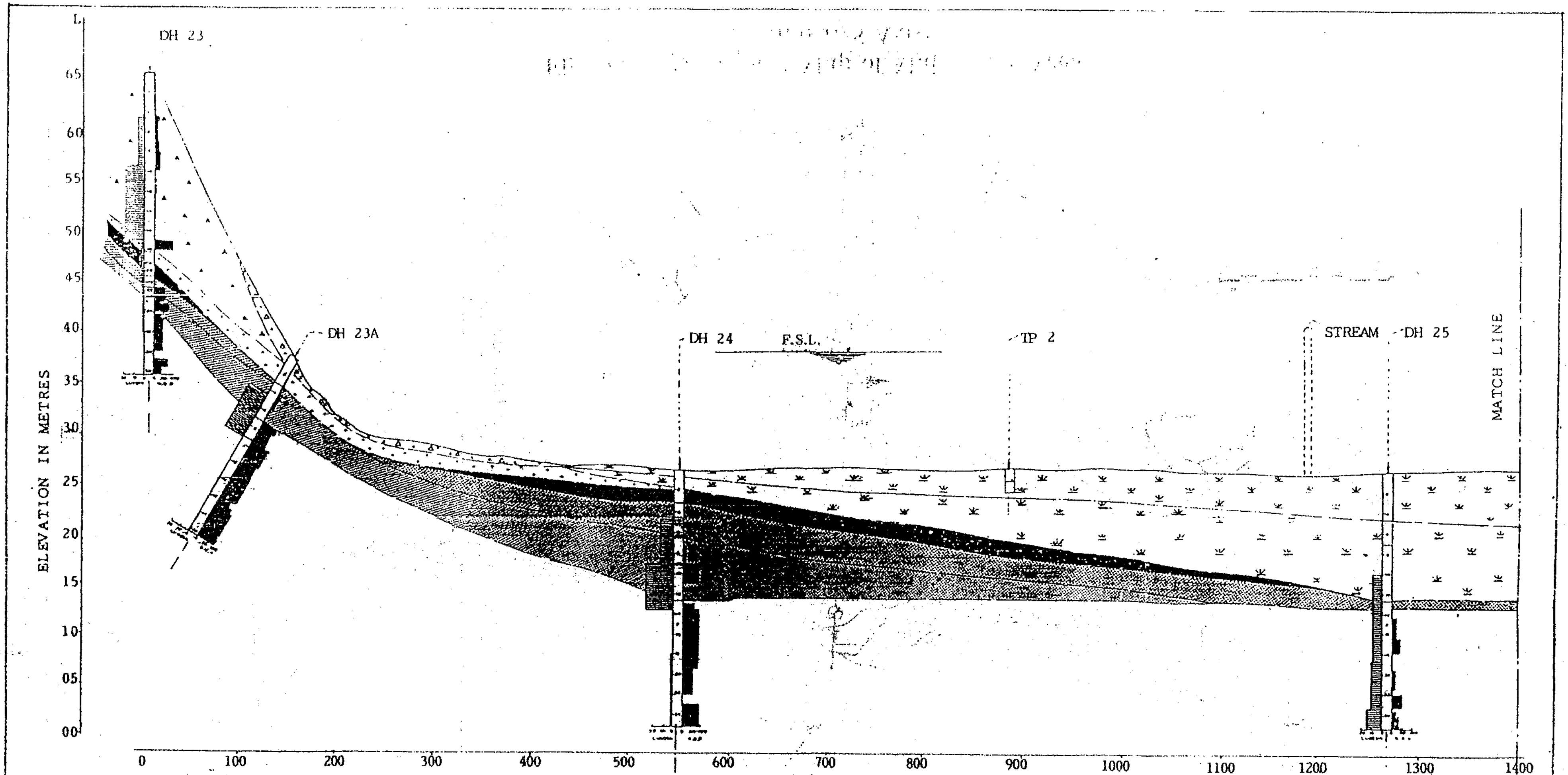
1. All dimensions are in meters
2. All elevations are in meters above mean sea level

SCALE VERTICAL 1 : 200
HORIZONTAL 1 : 2000

Vertical

Horizontal

Fig. 2 (02 of 02)



LEGEND

Abandoned or existing ancient earth dams made of predominantly clayey silty sand or gravelly clayey sand.

Lake bed deposits in abandoned or existing ancient reservoir beds. Dark brown to black clayey silt or black organic clay with shells or fresh water molluscs.

Alluvium-layered flood plain deposits, black to dark brown highly plastic organic clay with bands and lenses of clayey silt/sand. Organic matter is present mainly in dispersed colloidal form. At places decomposed fiberous vegetable matter remains within organic clay up to 5%. Light grey concretion of silica up to 2.5cm occurs within clay.

Alluvium-layered flood plain deposits bands and lenses of undifferentiated light grey/dark brown clayey silt. Clayey sand, sand and gravel.

a.1 Talus - Accumulation of boulders hillside waste and debris on hill slopes size of boulders varies from 1m. to 5m.

b.1 Accumulation of detached rock blocks on exposed bed rock.

Residual soil, reddish brown, yellowish brown or dark brown clayey sand or gravelly clayey sand or silty clayey sand or clayey silt. (Where paddy fields are developed on residual soil thin cover up to 0.5m of black organic clay is formed over this layer of residual soil.

Completely weathered rock yellowish brown or reddish brown clayey silt or silty clayey sand with traces of foliation.

Highly weathered rock

Moderately weathered rock

Slightly weathered rock / Fresh rock

Interpreted Geological Boundary

Assumed Geological Boundary

Notes:

1. All dimensions are in meters
2. All elevations are in meters above mean sea level

SCALE VERTICAL 1 : 200
HORIZONTAL 1 : 2000

Vertical

Horizontal

DH.26 Drill Hole No.

TP 2 - Test Pit No.

Result of Water Pressure Test

LUGEON R.Q.D

Fig. 02 (01 of 02) Geological Section of Main Dam (U/S Axis)

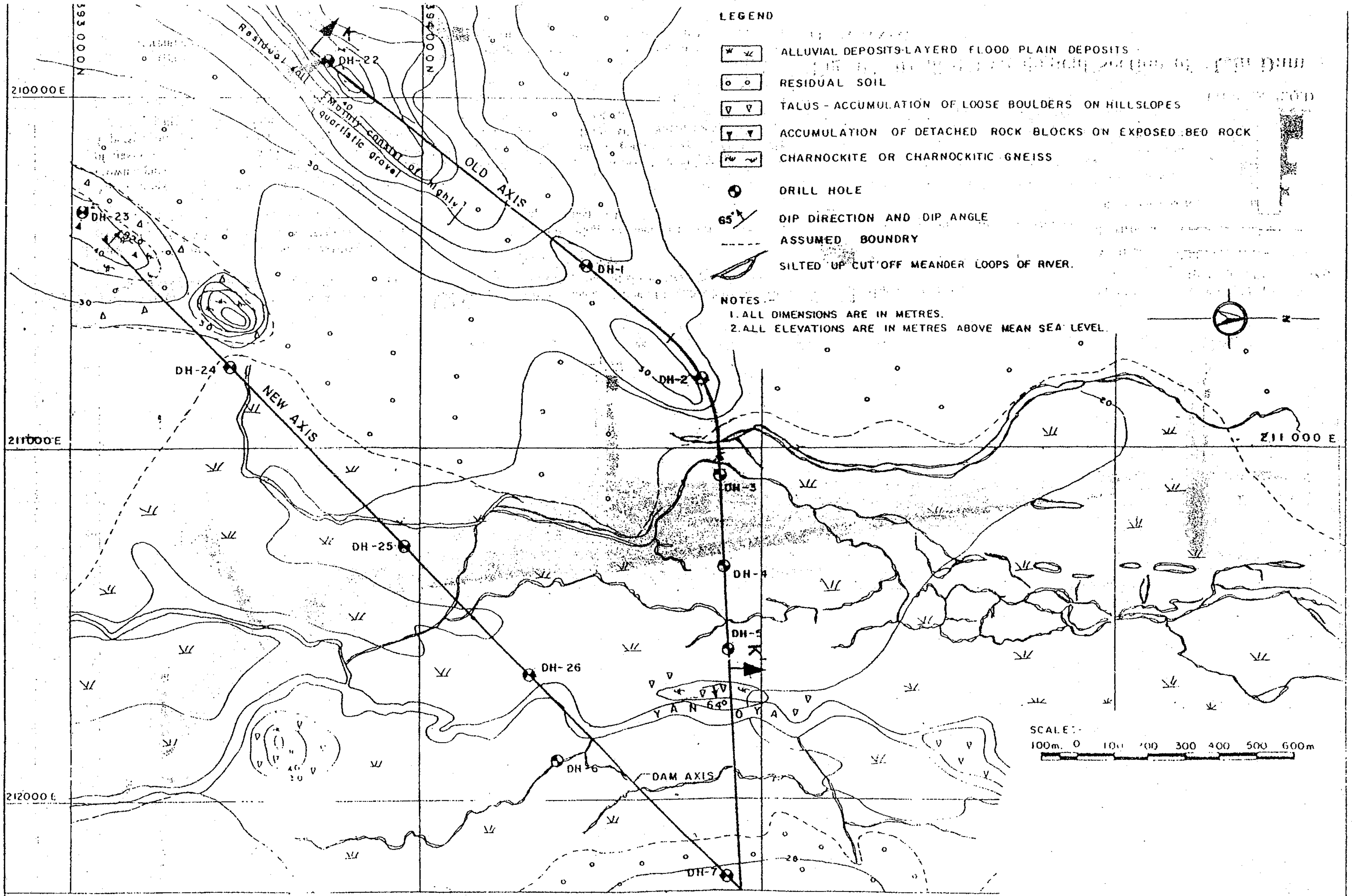


Fig. 03 - Geological Map of Main Dam Area
(U/S Axis and D/S Axis)

Treatment of Dam Foundations for Stability and Water Tightness

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INTRODUCTION

"No structure grips the ground so closely as a dam. It holds on at its base and at its flanks. In other words a dam consists of two parts, the artificial dam, man made, and the natural dam which continues it, surrounds it, and on which it is founded. The more important of the two is the latter which is unnoticed."(3)

The different characteristics of the ground to be considered in the design of dams may be classified as;

- Water tightness
- Stability
- deformation

The influence of the above characteristics is a function of the prevailing geological condition and the selected type of structure. Because of large number of dams already constructed, sites suitable for dams have become scarce and now dams have to be built on less favourable geological conditions. Therefore this necessacitates the treatment of dam foundation to achieve the above characteristics of the ground.

This paper deals with the foundation problems encountered in Kirindi Oya dam and the methods adopted for treating the foundation to achieve the stability and water tightness of the dam. The treatments adopted depend on the particular geological condition at each zone of the foundation.

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KIRINDI OYA

The Kirindi Oya reservoir which is located in the dry zone on the south quarter of Sri Lanka has a gross capacity of 184,000 acre feet of water at full supply level of 191 ft. above MSL. The dam is constructed across Kirindi Oya at Lunugamvehera village. This is the longest earthen dam in Sri Lanka. The length of the dam is 3.25 miles and the maximum height of the dam is 80 ft. at the river section. The 328 ft. long radial gated spillway is located on the right flank from chain 140+26 ft. to chain 143+54 ft. and has six radial gates of 48 ft. by 25 ft. The general lay out of Kirindi Oya dam axis is given in Fig.1.

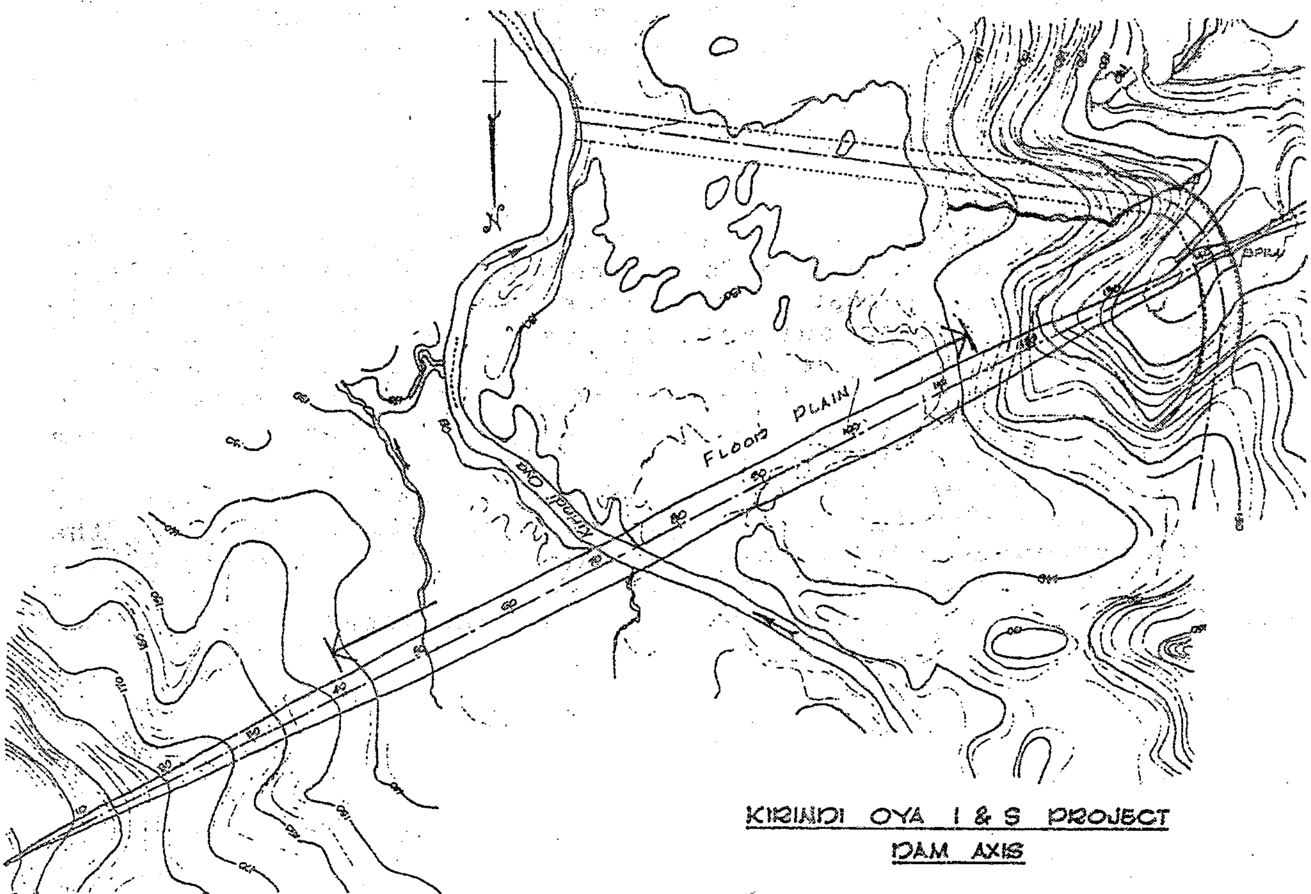
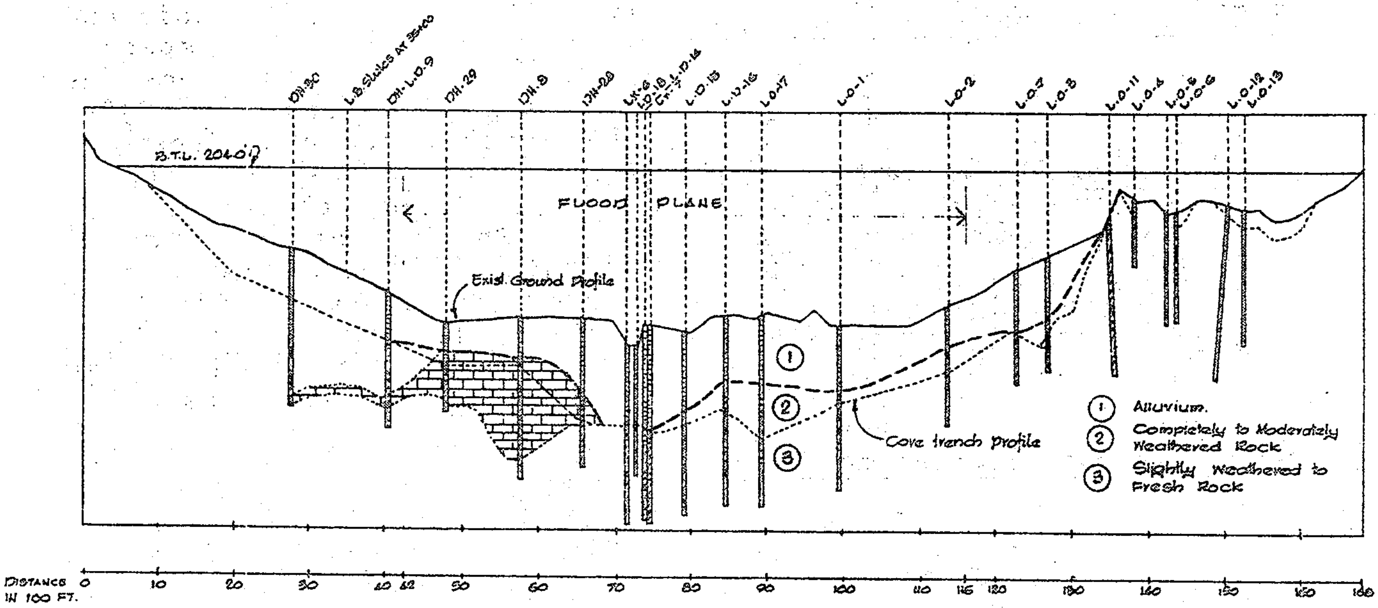


Fig.1 - General lay out of Kirindi Oya Dam Axis

GENERAL GEOLOGY AT SITE

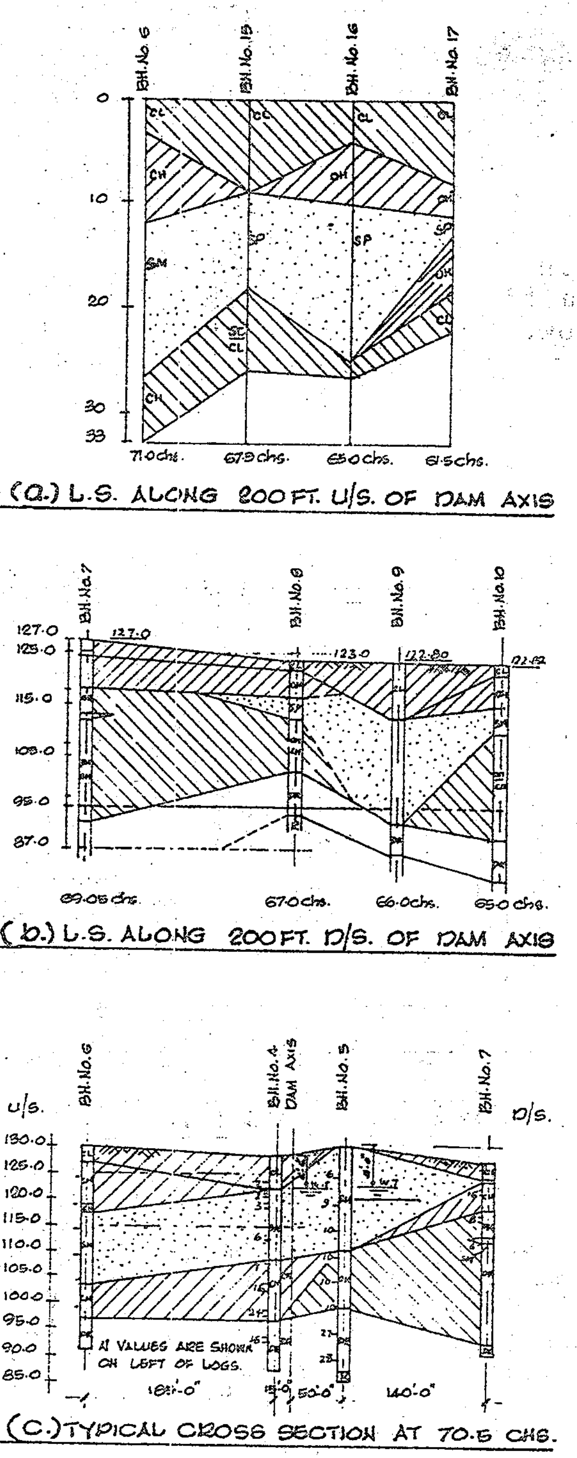
Geologically the reservoir area is located in the boundary of two geological units viz. the Vijayan series and Kataragama complex which differ in lithological and structural attitudes. The left bank falls within Kataragama complex which resembles rocks of highland series consisting of predominantly crystalline limestone, metasediments and charnokites. The right bank falls within the Vijayan series which consists essentially of gneissic type rocks. The river section forms the transition zone.On the left bank the calcareous rocks are generally affected by weathering which assumes the form of solution channels along joints and tectonized zone of highly soluble material. The resulting openings are infilled with clayey silty sandy soils. However the degree of karstification is low and the depth to which the karstification has occurred is shallow. (Approximately about 20 ft. from core trench bottom). (See Fig.2)



GEOTECHNICAL SECTION OF FOUNDATION ALONG THE DAM AXIS

Fig 2

The subsurface of the dam base in flood plain could be classified into top alluvial soil or residual soil passing downward to regoliths (which are the insitu weathering products of parent rock) and underlain by parent rock. These regoliths on slight disturbance disintegrates into materials of sand, silt and clay often with gravel sized nonweathered quartz and feldspar. The degree of weathering decreases with depth and fresh rock is encountered at depths 30 ft. to 50 ft. in main valley and 10 to 20 ft. in the flanks.



3 (a.) L.S. ALONG 200 FT. U/S. OF DAM AXIS

3 (b.) L.S. ALONG 200 FT. D/S. OF DAM AXIS

3 (C.) TYPICAL CROSS SECTION AT 70.5 CHS.

Fig. 3

Foundation conditions under the earth dam could be grouped under two categories. (See Fig.2)

i. Foundation on left bank flank (0ch - 42 chs) and right bank flank (116 ch to end)

ii. Foundation in the flood plain of the river. The main river is from 69 to 76 chains.

The foundation materials in the two flanks is residual soil and do not pose any special problems. In these reaches the dam height is less than 60ft. The central reach of the dam (42 to 116 chains) is located in the flood plain of the river. The foundation materials in this reach are transported soils and contain layers of sand, silt and organic and inorganic clays of high plasticity.The thickness and sequence of these layers vary along the reach. Fig. 3 a,b and c shows some typical sections through foundation material between 60 to 80 chains (based on bore hole data). In the left bank of flood plain the overburden is underlain by calcareous rocks which are affected by weathering forming solution channels. Hence the presence of fat clays in overburden and karstification of rock strata in left bank resulted in most of the foundation problems under earth dam that were encountered in the flood plain.

Problems under the earth dam could be grouped into two categories namely;

1. Water tightness of the foundation.

2. The stability of embankment slopes in reaches where foundation material consists of thick layers of highly plastic clays.

FOUNDATION TREATMENTS UNDER THE EARTH DAM

Foundation treatment under the earth dam could be group into three headings viz;

1. Provision of core trench to prevent seepage through the overburden.

2. Provision of a cement grout curtain in selected reaches of left bank below the bottom level of core trench to control seepage through the rock strata.

3. Stabilization measures in the flood plain reach where fat clays were encountered under the dam base to ensure the stability of the embankment.

Core trench to prevent seepage through overburden.

The insitu permeability tests carried out in the overburden along the dam axis have indicated that the coefficient of permeability is in the range of 400 to 1000 ft. per year for the residual soils. In the flood plain the weathered rock layer is overlain by alluvial deposits of sand, silt and clay of varying sequence and varying thickness. The coefficient of permeability in the flood plain zone is unrealistic due to the presence of sand layers.

In order to prevent seepage through the overburden a core trench has been provided under the entire length of dam which has been excavated up to the bottom elevation of moderately weathered rock strata to cut off seepage through the weathered layers also. Fig. 2 shows the depth of core trench under the dam. The depth of core trench within the flood plain (42 to 116 chains) varies from about 20 ft. to 43 ft.

Grout curtain to prevent seepage through the rock strata.

The water pressure tests carried out in rock strata has indicated that in the river section and in right bank the permeability of the sound bedrock encountered at depths varying from 30 to 50 ft. as less than 5 lugeons and in the fracture zones as between 10 to 20 lugeons. On the left bank, in the reaches where the calcareous rocks are affected by weathering, the permeability varies from 30 to 60 lugeons in moderately weathered rock strata and 15 to 20 lugeons in slightly weathered rock strata. These values very clearly demonstrated the irregularity of the permeability due to varying degree of karstification and weathering condition of rock. In these reaches of karst openings infilled with loose soil materials, piping could be possible under seepage gradient after impounding the reservoir.

In order to seal the solution channels in the left bank of the river section, a grout curtain in selected reaches below the bottom elevation of core trench has been provided. Fig.4 shows the reaches where grouting was done.

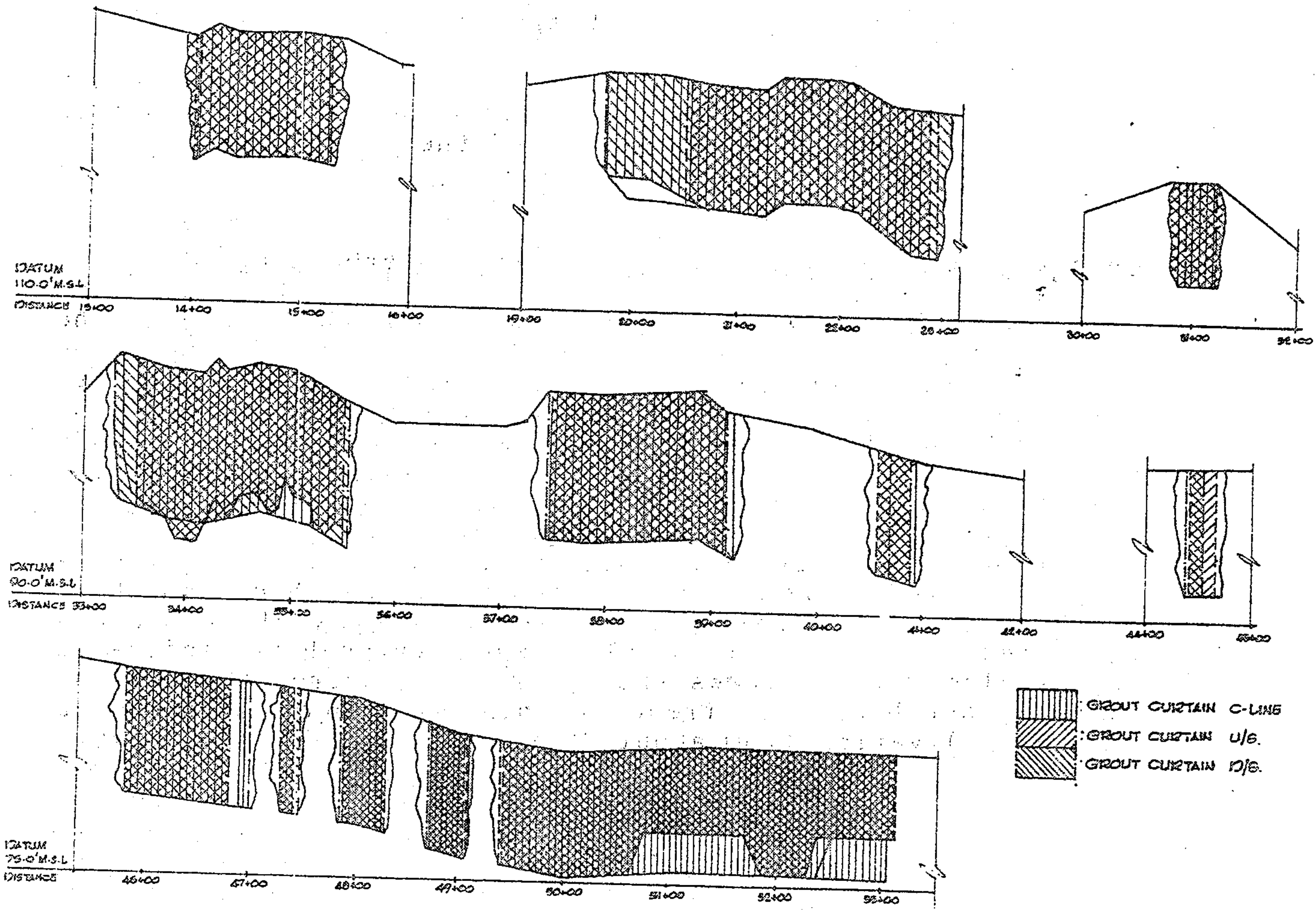


Fig.4 - L.B Core Trench Grout Curtain

Three rows of grout holes, 10 ft. apart were drilled (in a 20 ft. wide core trench) at a spacing of 10 ft. The upstream and downstream holes were drilled at approximately 15^0 and the center holes were drilled vertically.

The depths of holes varies from 20 to 35 ft. Grouting was done in two stages from top to bottom.

First stage - 15ft approximately, grouting pressure 20psi
Second stage - 15 to 20ft., grouting pressure 40psi
Table 1 gives the details of grouting carried out.

TABLE 1 - Details of Grout Curtain

S.No	Reach	Depth of section	No of rows	Hole spaceing Ft.	Total No of holes	Approx. Depth meters	Total depth of drilling meters	Total grout consumpt. Kg.	Specific grout intake Kg/m.
1.	14+00' to 15+25'	125	3	10	42	9.20	381	16,370	43.0
2.	19+75' to 23+00'	325	3	10	88	9.20	812	29,896	36.8
3.	30+85' to 31+25'	40	3	10	13	9.20	119	10,180	85.5
4.	33+25' to 35+50'	225	3	10	64	10.70	684	25,980	38.0
5.	37+40' to 39+20'	180	3	10	51	10.70	542.5	17,760	32.7
6.	40+55' to 40+95'	40	3	10	11	9.20	100.6	3,995	39.7
7.	44+40' to 44+70'	27	3	10	9	9.20	82.2	3,411	41.5
8.	45+80' to 47+00'	120	3	10	37	9.20	338.3	24,173	71.5
9.	47+30' to 47+55'	25	3	10	7	9.20	64	2,124	33.2
10.	47+90' to 48+30'	40	3	10	15	8.30	123.4	4,910	39.8
11.	48+70' to 49+10'	40	3	10	16	8.30	146.3	5,448	37.2
12.	49+30' to 53+10'	380	3	10	109	6.1to9.2	869	34,600	39.8
13.	57+50' to 60+50'	290	3	10	83	6.1to9.2	648.4	31,580	48.7
								210,427	

The average grout intake on left bank side was about 40 to 50 kg. per meter and total grout intake per hole varied from 1000 to 2000 kg. except in one hole where the grout intake was 3327.5 kg. per meter. In order to determine the effectiveness of grout curtain 30 check holes were drilled and water pressure tested. In all the tested holes the lugeon values were closer to nil and always less than 10 lugeons indicating the effectiveness of the curtains.

Stabilization to ensure the stability of embankment in reaches of the flood plane where fat clays are encountered.

In the flood plain reach (42 to 116 chains) thick deposits of highly plastic clays and organic clays with variable strength were encountered during construction. Sometimes lower values were obtained at deeper depths. During excavation of core trench presence of softer clay layers with slickenslides in bands were observed in different reaches within the flood plain. Remedial measures involving a combination of relief wells, partial or complete removal of fat clay layers and placement of fills were adopted depending on their thickness and their depth of occurrence.

Chain 48 to 60 and chain 91.5 to 106

The depth of clay layer was less than 20ft. CH and OH soils in these reaches were removed by excavation and the embankment now rests on sandy materials or weathered rock. The downstream horizontal blanket has been laid at ground level after backfilling the excavated deep portion with SC material.

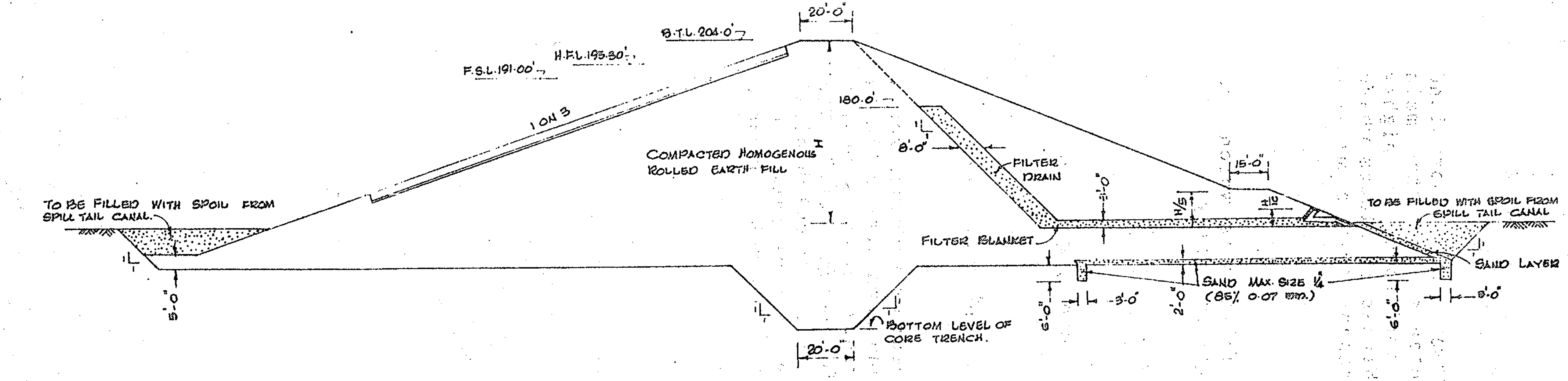
Between 91.5 to 106 chains two longitudinal sand drains 3ft. wide, 6ft. deep were constructed into weathered rock and were connected by sand blanket laid over the weathered rock surface. (See Fig.5)

Chain 60 to 64.5

In this stretch CL material with very low shear strength were encountered. In this reach 15ft. deep and 40ft. bed width key trench was cut under up stream and down stream toes of the embankment and back filled with SC material. These shear keys were taken up to poorly graded sand (SP materials) in the foundation. (See Fig.6)

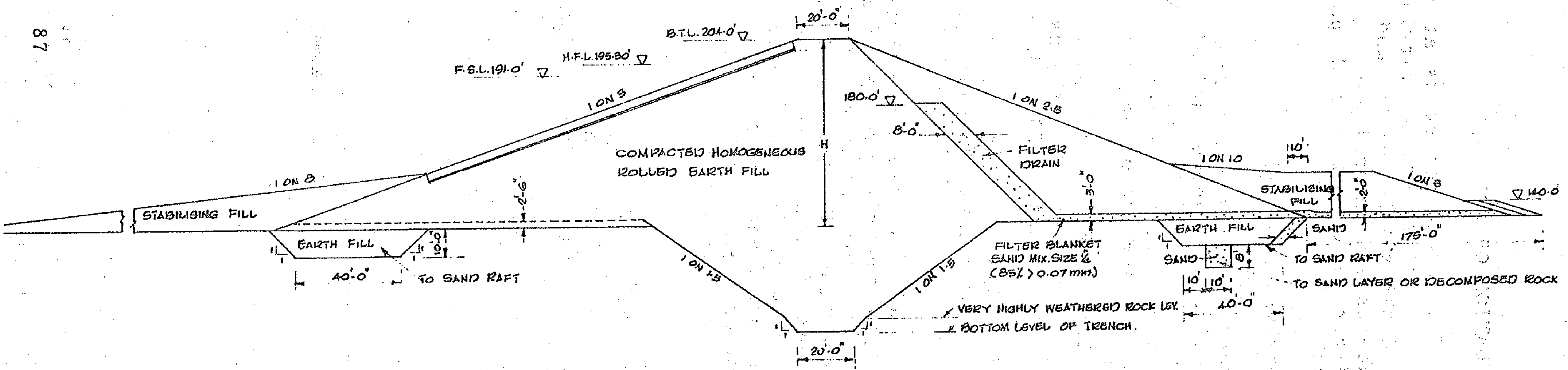
Chain 64.5 to 91.5

In this reach erratic and heterogeneous soil formation with thick deposits and lenses of plastic and organic clays interbeded with sands except for the top few feet of lean sandy and silty clays was encountered. Here remedial measures involving provision of stabilizing fills both up stream and down stream in addition to shear keys and relief trenches were adopted. (See Fig.6)



TYPICAL SECTION BETWEEN 48.0 chs. To 60.0 chs AND 91.5 chs. To 106.0 chs.

Fig. 5



TYPICAL SECTION BETWEEN 60.0 chs. To 91.5 chs.

Fig. 6

FOUNDATION OF SPILLWAY

The foundation rocks at the spillway site is hornblend biotite gneiss with varying percentages of the main constituents. In bands and lenses, the rock is highly micaceous and intensively foliated whereas in places where feldspar or quartz - rich portions present form a massive rock.

Structural Geology

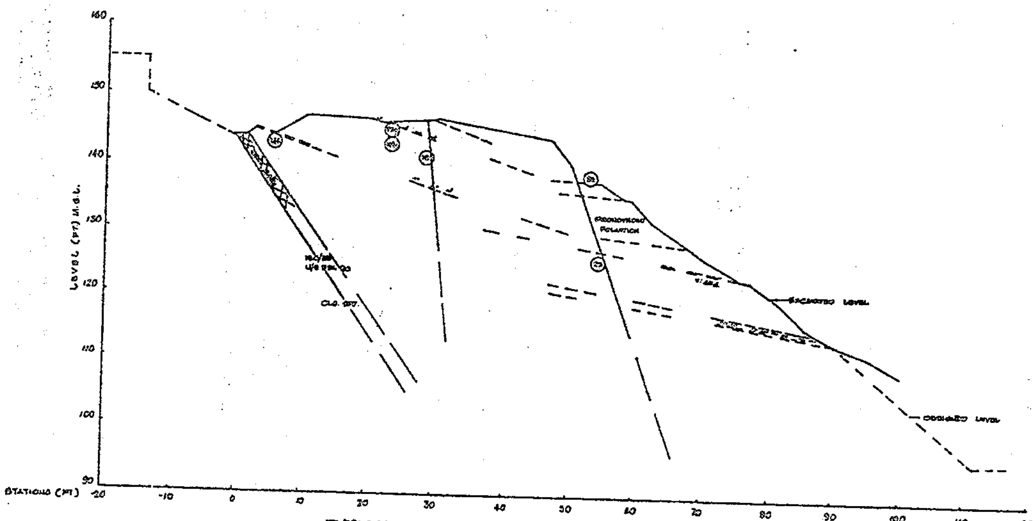
Structurally there are three dominant joint sets which are effective both geomechanically and geohydraulically.

1. Foliation plane :
   Average orientation 170 - 200^{0} / 10 - 20^{0}

2. Joint set 1 (Js-1) :
   Orientation ranging from 055 - 070^{0} / 85 - 90^{0} to 235 - 250^{0} / 85 - 90^{0}

3. Joint set 2 (Js-2) :
   Orientation ranging from 160 - 180^{0} / 85 -90^{0} to 340 - 360^{0} / 85 - 90^{0}

In addition there are three major shear or fracture zones two of them nearly forming the upstream and down stream limits of the control structure and the third shear zone is nearly parallel to foliation and is called "foliation shear". The upstream fracture zone is of particular geomechanical importance as it has allowed weathering to penetrate along foliation planes and joint set 1 & 2 towards downward direction. (See Fig.7)



TYPICAL GEOLOGICAL SECTION ALONG THE SPILL AXIS

Fig. - 7

The fracture orientation exhibits little variability across the site. The foliation plains are continuous over the full width of ogee. The joints are less continuous. In general the rocks become less fractured with depth and there are insufficient continuous fracture planes below EL. 125ft. to form potentially unstable wedges.

Many of the joint surfaces are weathered and some of the foliation planes contain weathered light brown colored mica. The thickness of weathered material varies from 1/10 inch to 1 inch. In general the degree of weathering decreases away from the fracture zone and there is little weathering on the glacies face. Many of the unweathered joints are infilled with chlorite and calcite and majority of foliation planes have more infilling and are unweathered.

<u>Permeability of rock mass and uplift conditions</u>

The water pressure tests were carried out in 30 investigation holes.In vertical holes which generally intercept only foliation planes, permeability was found to be very low (lugeon < 1 corresponding K value is approximately $1*10^{-5}$ cm per second). Inclined holes also showed low to very low permeability values except where weathered joints were intercepted the permeability shot up to 10 lugeons ($1*10^{-6}$ cms. per second).

Summerising the above test results it could be stated that the permeability of rock mass is generally low to very low which is due to the tightness of foliation planes and tight and/or healed joint planes. Only open weathered joints represent potential seepage paths.

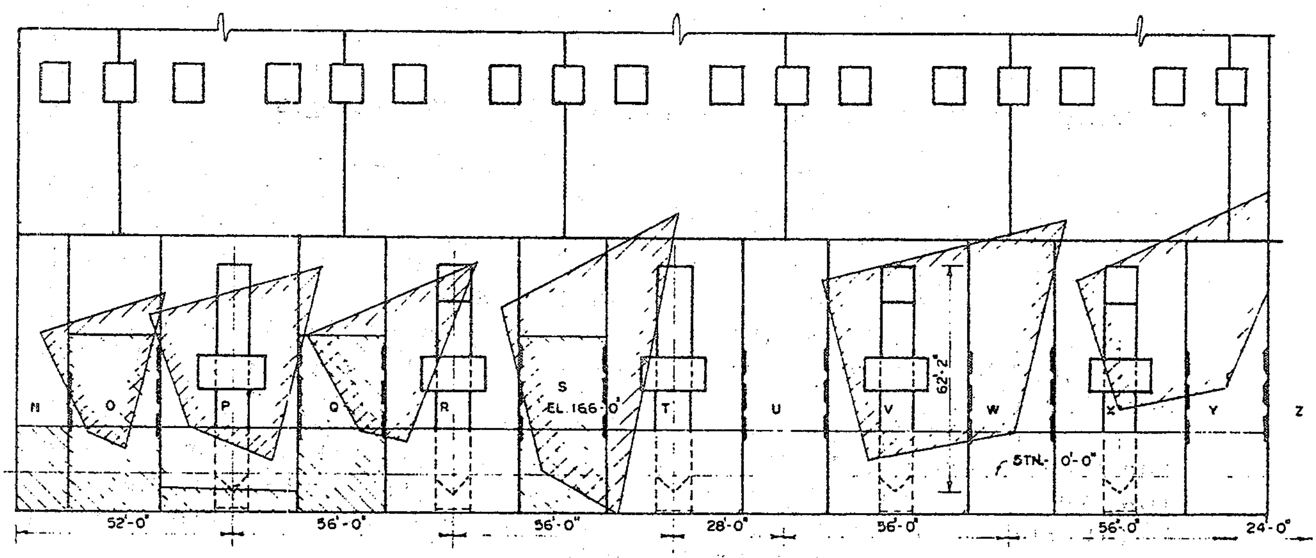
There are only few of them and occurrence is generally limited to the vicinity of upstream fracture zone and there are only two or three major weathered joints which extend more than 40ft. down stream of fracture zone.

STABILITY PROBLEMS

1. Stability of wedges formed by interception of foliation planes, joints and fractures.

The foundation rock under spillway block is strong, generally unweathered biotite gneiss which is uniform across the site except for variation in the biotite content. The rock strength is considerably greater than the stresses induced by the spillway structure and therefore, the stability will be governed by the characteristics of the natural fracture zones.

Investigations and geological mapping carried out during excavation of foundation has revealed that there are wedges formed by the interception of foliation plane, joint set 1 & 2 and up stream fracture zone which are potentially unstable. There are six wedges which have been identified as unstable. Fig.8 shows the shape and locations of the six wedges. The stability of these wedges depend, part on the shear stresses developed along the fracture surface. The factor of safety against sliding ranges from 0.25 to 0.75 for the top foliation plane starting from upstream level at EL. 150ft.



LOCATION OF ROCK WEDGES IN SPILLWAY FOUNDATION

Scale - 1 Inch to 24 Feet

LEGEND-

APPROXIMATE SURFACE OUT LINE OF MAJOR WEDGES
OTHER WEDGES MAY FORM ALONG PARALLEL GEOLOGICAL STRUCTURE

Fig. 8

2. Drainage and Uplift pressures

Due to tightness of foundation rock the seepage through the foundation is low to very low resulting in poor drainage and increase in uplift pressures.

FOUNDATION TREATMENT

1. Stabilization of unstable wedges

In order to ensure the overall stability of spill foundation it was found necessary to stabilize the potential unstable wedges by anchoring using post tensioned rock bolts. The factor of safety achieved against sliding by providing such anchors is given in table 2.

Table-2 - Support forces required for spillway structure using anchor bolts

Structure Type	Block No.	Width Ft.	Rock bolts only Bolt tension/block*		
			FOS=1.0	FOS=1.5	FOS=2.0
End pier	Y+Z	22	0.6	1.0	1.4
Pier	W2+X+Y1 U2+V+W1	56	0.9	2.9	3.9
Pier	T,R,P,Y V	34	0.9	1.8	2.4
Abut.Pier	N,Z	11	0.4	1.5	1.7
Ogee	U	11	0.3	0.5	0.8
Ogee	S,Q,O Y,X,U	22	0.3	0.7	0.9

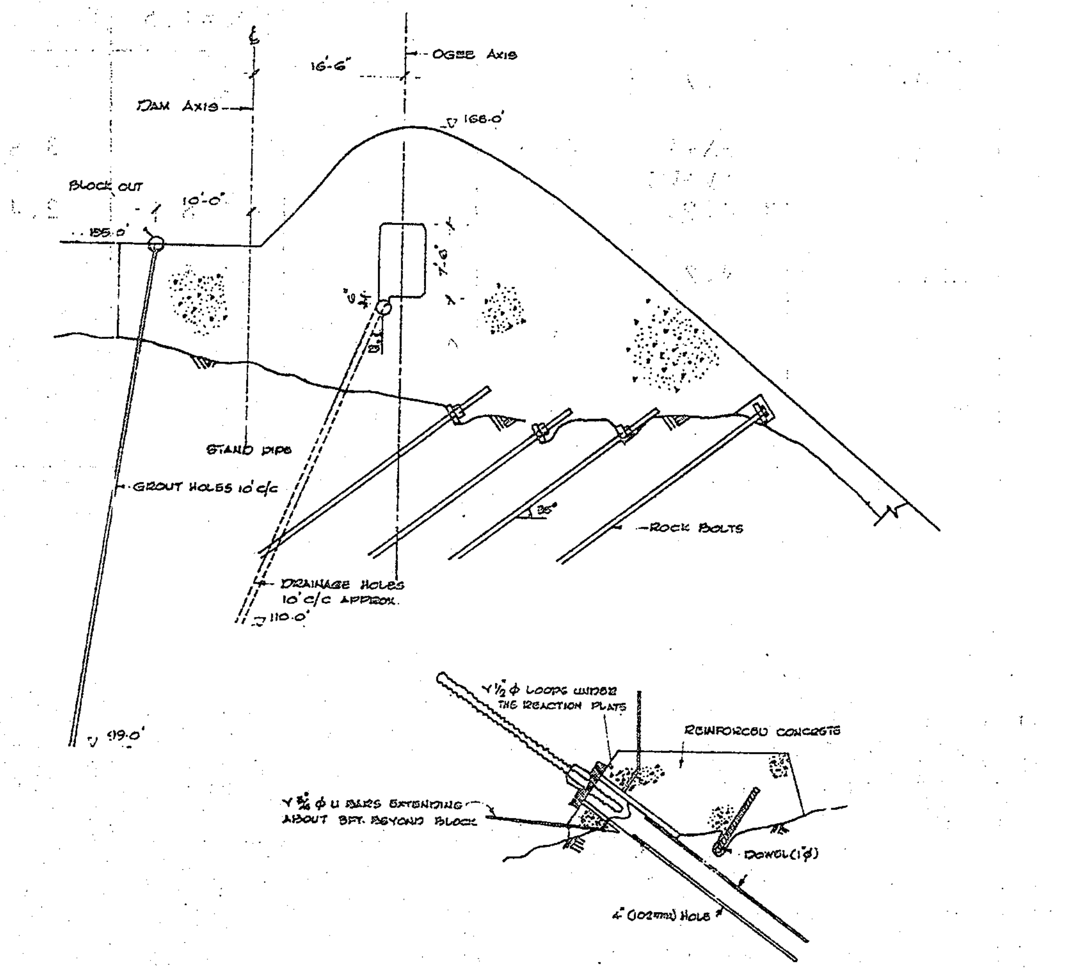
* All forces are in millions of lbs.

ROCK BOLTING

The general arrangement of rock bolting on spillway structure is given in Fig.9. Except in block Q & S where part concrete has already been done, all rock bolts were installed from the rock surface. Concrete reaction pads mostly without reinforcement were casted for tensioning the bolts.

The rock bolting was done using DYWIDAG type rib tor steel rock anchors of ST 855/1030 steel grade with double corrosion protection arrangement. The rock bolt with a diameter of 36 mm and length 19 ft. has a minimum yield load of 74,000 lbf.

For extension purposes 5 ft. long threaded bars of the same material were used. Since the length of the rock bolt required were 38 ft. and 24 ft. lengths, couplers capable of developing 100% of minimum yield load were used for joining the short lengths. In order to prevent corrosion, the rock bolts were covered its entire length with a 1 mm thick straight ribbed sheath of PVC 100 having an inside diameter of 55 mm. To anchor the wedges 200 no. 4 inch diameter holes at an angle of 35° to vertical were drilled with percussion drilling equipment. The holes were tested for watertightness by water pressure test and those having high water intake were regrouted and redrilled. After cleaning the holes rock anchors were placed in position and installed with reaction block and mild steel plate after primary grouting. The rock anchors were tensioned, tested and secondary grouted after the performance and proof tests were completed.



TYPICAL CROSS SECTION OF ROCK BOLTING, GROUT CURTAIN & DRAINAGE WORKS

Fig 9

2. Drainage and uplift pressure

(a) To prevent water flowing along the strike of the upstream shear zone which would feed water into the rock under the structure, two connected grout curtains were provided viz.

i. Grout curtain upstream of spillway structure over a length of 100 ft.

ii. Grout curtain under the upstream end of the ogee apron to the full length of spillway.

(b) Provision of drainage holes to reduce uplift pressures.

GROUTING

Two connected grout curtains were formed. First grout curtain was formed by grouting of shear zone upstream of the spillway structure over a length of 50 to 100ft. The purpose of this grouting is to prevent water flowing along the strike of the shear zone which would bring water into the rock under the structure.

The second grout curtain was installed under the up stream end of the ogee apron which extend over the full length of the spillway. The depth of each grout holes is 50 ft. at a maximum spacing of 10 ft. depending on water pressure tests results. The inclination of grout holes varies from angle 60^0 to 80^0 depending on the geological conditions. Grouting was carried out in 3 stages each 15 ft. deep. The permeability criteria used for a successfully grouted hole or tight rock in ungrouted hole was one lugeon. (See Fig.9)

DRAINAGE

All drainage holes were provided from the 6 ft. by 8 ft. drainage gallery which is located in the ogee structure along the ogee axis with the floor level at EL. 147 ft. MSL on a 1% grade. The gallery had to be restricted to block R to Z with access provided through block Z. Extension of gallery to blocks O, P and Q were prohibited as it entailed rock excavation by blasting. The draining the foundation of blocks O, P and Q were made through a system of pipes laid within the concrete on the upstream slope of ogee. All drainage holes were positioned at locations of near vertical joints which are usually brown stained indicating that they carry seepage water. The holes were close to vertical and designed to intersect both the foliation planes and the joints.

The drainage holes are located depending on the spacing of joints which are between 5 & 7 ft. apart. The diameter of drainage holes is 3 inches. The drainage holes extend to a depth of 40ft. from the floor of the gallery which is at EL 147ft. above MSL. (See Fig.9)

MONITORING

1. To monitor the effectiveness of drainage under the spill foundation a total of 10 piezometer were installed, five of them along the dam axis and the other five, 15 ft. down stream. All were installed in holes drilled through the five piers to EL127ft. above MSL.

2. To monitor the behavior of rock anchors load cells (Huggenberger load cells) have been installed at 20 selected locations to represent the entire foundation. These load cells have been connected to the read out station on top of the piers and then to the computer installed in the spillway.

ACKNOWLEDGEMENT

We would like to thank the Director of Irrigation for granting permission to publish this paper and all those who have helped us in the preparation of this paper.

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Stage 1 Remedial Measures for Stabilizing Landslide at Beragala on Beragala Haliela Road

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ABSTRACT

Landslides are a major natural hazard that disrupts facilities, lives, properties and affect the country's economy. Most of the landslides in Sri Lanka have been triggered off just after heavy rains.

Stability of the soil and rock slopes are mostly dependent on the ground water conditions. As such drainage of unusual build up of pore water pressures during the time of heavy rains is an important remedial measure to stabilize such slopes.

This paper presents the design and construction aspects of stage 1 remedial measures for stabilizing a land slide at Beragala on Beragala-Haliela Road (A16). This landslide also affects the Colombo-Ratnapura-Wellawaya-Batticaloa road (A4) at Beragala. These two roads run through hilly terrain and are gateways to the Uva Province and North-Eastern (East) Provinces.

The major cause of this landslide was the water logging within the unconsolidated colluvium. In order to arrest the landslide, remedial measures were envisaged in two stages, that is stage 1 and stage 2. This paper describes the stage 1 remedial measures.

In stage I, a diversion drain with appropriate drop structures and surface drains were constructed to lead away the surface run off beyond the landslide area within a shortest possible time. Further a trench drain (sub surface drain) of depth ranging from 3 to 4.5m and horizontal drains were provided by using locally available materials to release the ground pore water pressures. The techniques of construction of trench drains and horizontal drains have to be developed in Sri Lanka. The pioneering efforts made on trench drains at Pussellawa on Kandy-Nuwaraeliya road and on horizontal drains at Watawala Railway landslide have encouraged the Road Development Authority to carry out this work described in this paper.

The future remedial measures for stage II such as additional horizontal drains, crack sealing and diversion of stream paths and lining of natural streams are also discussed in this paper.

1.0 INTRODUCTION

Beragala landslide area is located between culverts Nos. 2/1 and 2/3 on Beragala-Haliela Road (A16) and between culverts Nos. 184/15 & 185/7 on Colombo-Ratnapura-Wellawaya-Batticaloa Road. (See Figure 1)

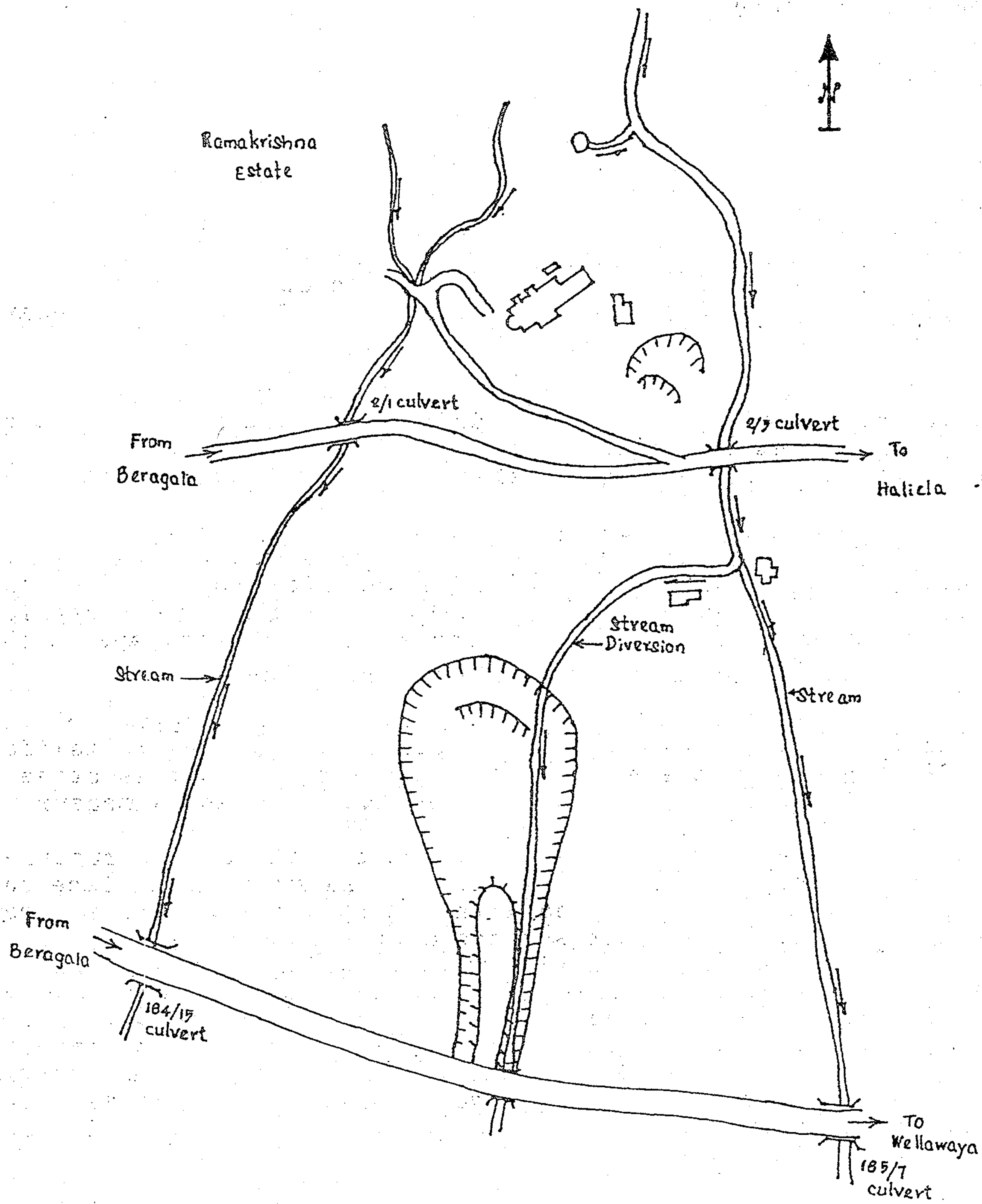


Figure 1: Sketch showing the Beragala Landslide

The soil cover of this area consists of colluvium deposits and rock debris with large size boulders deposited on the chanockite bed rock. The average slope of the ground formation is about 20° to 36° (RDA (1989), Mallawaratchie (1994)). The sketch showing the cross section of Beragala landslide along the axis of the slide is given in Figure 2.

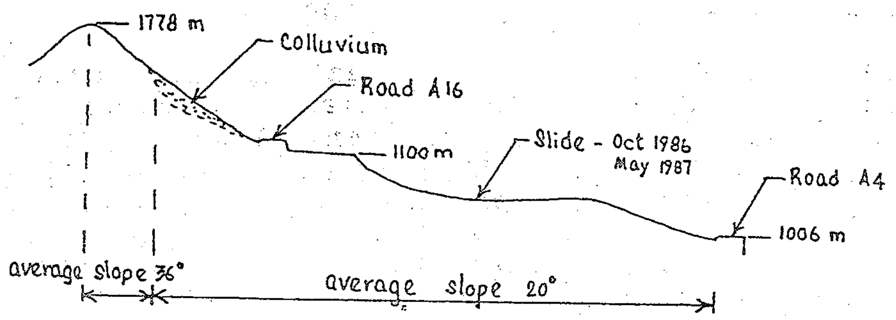


Figure 2: Sketch showing the cross section of the area affected by Beragala landslide

Tea small holdings which were in this area of landslide had been badly maintained and the land on the upper slopes had been replanted with vegetables after clearing of land. Uncontrolled watering of vegetable plots, collecting water in unlined collection pits, unsatisfactory maintenance of earlier contour ridging with dry rubble retaining walls and the allowing of earlier rubble paved drainage system into dilapidated disuse appear to have contributed to the instability of the steep mountain slopes. The water infiltration causing saturation and increase of pore water pressures with appearance of springs and formation of marshy land in this area had aggravated the situation.

The stream flowing past culvert No. 2/3, has a diversion towards Beragala, that had been made several years ago to the lower part of sliding area (See Figure 1). As a result of this diversion, the ground water level had been raised in this area. Surface water in the stream had penetrated into the sub soil through cracks on the surface of the ground causing high pore water pressures at locations down slope.

In some areas the ground water level had been raised to the ground level and had appeared as seepage water in the lower part of the slide area.

Due to the above reasons, slides between the two roads had taken place in October 1986 and also in May 1987, during periods of heavy rain.

As the water transported in the existing streams had been blocked by these slides at some locations, uncontrolled transport of the surface material and erosion of soil had taken place. This had caused the covering of road (A4) with large quantities of debris in October 1986 and May 1987.

In May 1987 cracks had also appeared to a length of about 100m on the road (A16) between culverts Nos.2/1 & 2/3 and the road had sunk to a depth of more than 1.5m. Commencing from this period, this section of road had been sinking by a few centimeters every rainy season.

In 1988, geotechnical investigations were carried out at this landslide site under Asian Development Bank (ADB) Funded Second Road Improvement Project for the Road Development Authority (RDA) in order to rehabilitate this section of road. Based on these investigations, consultants for this project submitted their report (RDA (1989)) to the RDA in March 1989. However this work was not taken up under the ADB Project due to lack of funds.

This matter was again taken up in late 1991 and early 1992 when the site was further inspected and surveyed by a team of Civil Engineers from the R&D Division of the RDA. These inspections and surveys were carried out after studying the ADB consultants report (RDA (1989)) on the above landslide. In these surveys, the possibilities of diverting streams and lining of streams as given in the above report were also studied and some of these recommendations were incorporated in the proposals for the initial stage (stage 1) of control measures. (RDA (1991 to 1995))

2.0 Remedial Measures

Based on the above it was understood that the stability of the soil slope mostly depended upon the ground water regime. Therefore the following stage-1 remedial measures were proposed to improve the stability of the landslide area to a safer level:-

2.1 Construction of a diversion drain to a length of about 500m from culverts Nos. 2/3 to 1/13 to divert surface run off and stream water flowing through culverts Nos. 2/3 & 2/1. Construction of a bell mouth inlet structure at culvert No. 2/3 with a gate arrangement to close the diversion drain in case of effecting any repairs to it caused by cracks occurring after subsidence.

2.2 Till such time step 2.1 was carried out, the diversion of the stream below culvert 2/3 which had been made several years ago to irrigate the lower part of the site, to be demolished and the stream rediverted to its original path (See Figure 1). In case of repairs to the diversion drain mentioned in step 2.1, step 2.2 will be useful in diverting water away from the landslide area.

2.3 Construction of a trench (subsurface) drain in between culvert 2/2 and 2/1 on road A16 to lower the ground water table by 3 to 4.5m.

2.4 It was observed that boulders on the steep side slopes close to and below the road A16 between culverts 2/3 and 2/2 that were keeping them stable, were being blasted and sledged for use on the road in 1991. This was immediately banned.

2.5 Construction of horizontal drains below the culvert No. 2/2 (that was closed when the diversion drain was constructed) on road A16.

3.0 Design and construction aspects, including problems encountered during construction

3.1 Design and construction of diversion drain

The diversion drain is about 500m long and located from culverts Nos. 2/3 to 13/1 on Beragala-Haliela road (A16). The purpose of construction of this diversion drain is to divert all water collected at culverts Nos. 2/3 & 2/1 to a safer location without allowing same to flow into the landslide area below.

The catchment area for design purposes was worked out as 12.5 acres. Based on the rainfall data given in page 34 of the book on Design of Irrigation head works for small catchment by Mr. A.J.P. Ponrajah (1984) for zone 5 of the country, the rain fall intensity for the 10 years return period was read off as 106.7mm per hour (4.2 inch per hour). Assuming the coefficient of run off as 0.5, the maximum run off was worked out 4.3 cumec (152 cusec).

Based on the above design parameters and Manning formula, a 1.2m (4 ft.) diameter concrete spun pipe drain was designed for the diversion of the design run off. In addition to the above, following factors were also considered during the selection of the type of drain.

a. As it is easy to repair pipe joints rather than repairing cracks in a random rubble masonry or concrete open channel in case of subsidence of diversion drain, a concrete spun pipe drain was selected for unstable sections.

b. Since the frequent maintenance of the diversion drain in the unstable sections is essential, a person should be able to go into the concrete spun pipe for inspecting the serviceability and effect the repairs, if any.

Therefore it was decided to have two types of construction for the diversion drain.

The area where movement is evident, 1.2m diameter concrete spun pipes were used. In the balance stable sections, open channels were constructed. In both types of constructions, suitable drop structures were built to suit the site conditions and to dissipate the kinetic energy of the water flow. A typical drop structure is shown in Figure 3.

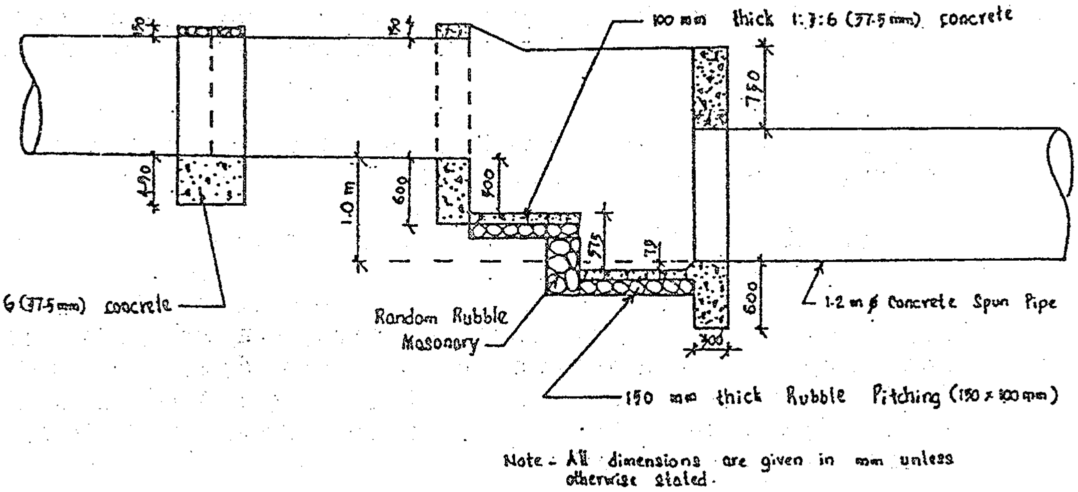


Figure 3: Sketch showing a typical drop structure of the diversion drain.

The construction of the diversion drain was carried out by the Road Construction and Development Company (PVT) Ltd. (RC&DC) and supervised by the Provincial Director's Office, Uva Province, RDA and the R&D Division, RDA.

In the construction work, not many difficulties arose except in the handling of 1.2m diameter concrete spun pipes, where a backhoe loader was used for lifting and laying the pipes.

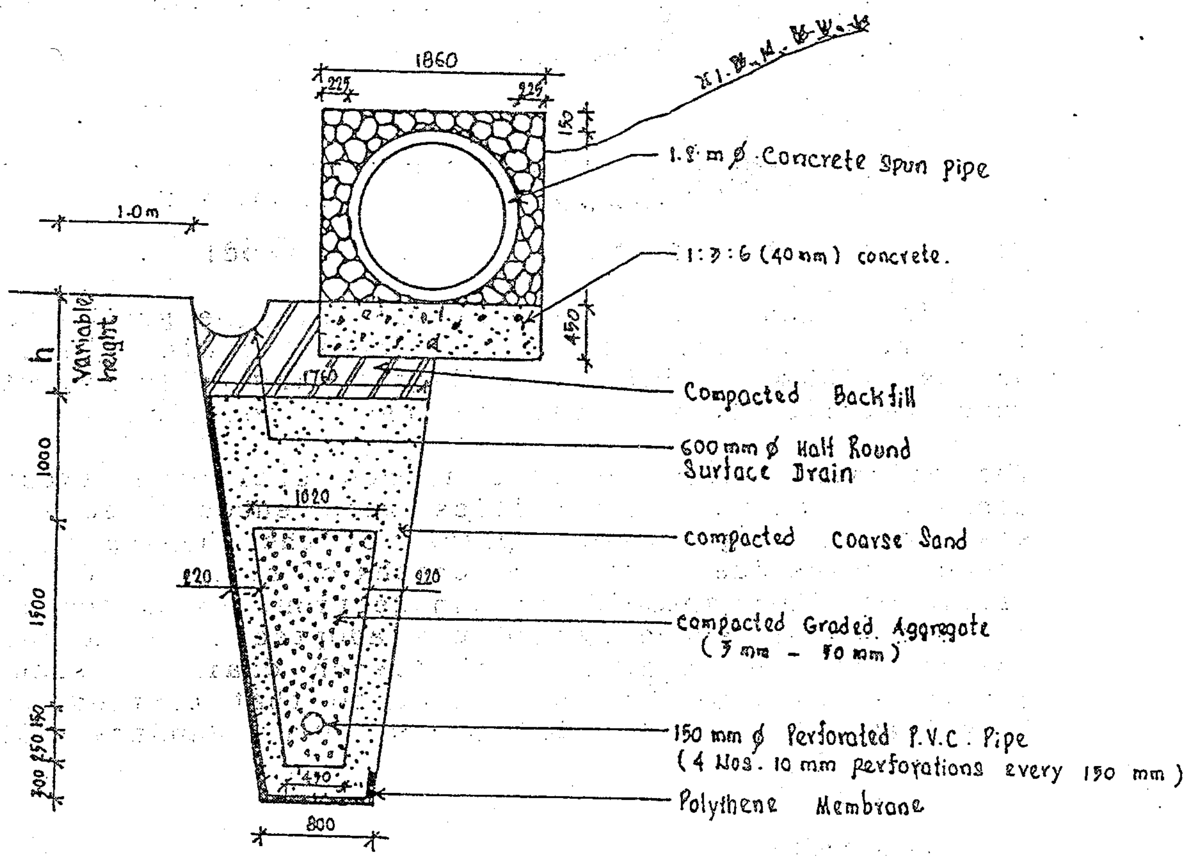
However, at few locations, seepage of water was observed under the diversion drain pipes. A filter was constructed using sand, aggregate and rubble (100mm) to avoid clay particles being washed off with the seepage water. The side drain of the road was constructed with precast half round concrete sections in between the diversion drain and road shoulders to collect the above mentioned seepage water and the surface run off from the road platform during rains.

3.2 Trench Drain (sub surface drain)

In addition to the diversion drain, a trench drain of about 50m in length was constructed between culvert No. 2/1 to

2/2 . The purpose of this drain is to reduce the ground water table to a depth of about 3 to 4.5m below the ground level. The cross section of trench drain is shown in Figure 4.

The work was carried out and supervised in the same way as for the diversion drain.



Note : All dimensions are given in mm unless otherwise stated.

Figure 4: Cross section of the trench drain

A manual method was used at the beginning of excavation and the depth of excavation was limited to about 1.5m at the first stage. Shorings were installed in order to protect the side slopes from collapsing. The construction of shorings were carried out as depicted in Figure 5.

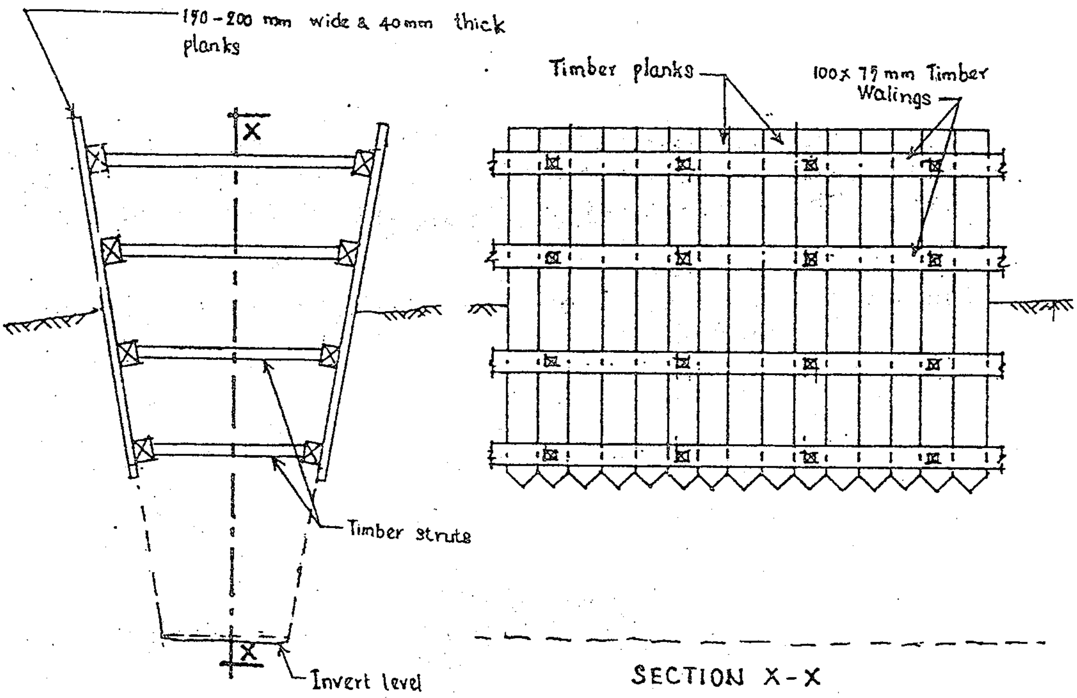


Figure 5: Sketch of shorings (first method)

In this method, planks were driven into the ground to fit into the outer periphery of the trench drain. The timber walings (100mm x 75mm) were fixed horizontally and struts were installed to keep the shorings in position.

The balance section of the trench drain was excavated manually. However difficulties were encountered at the stage of driving sloping planks of the shoring into the trench drains due to varying lengths of struts at different levels of the trapezoidal drain section. To overcome this difficulty, a second method was adopted using a backhoe loader for the excavation of trench drain as shown in Figure 6. This method is similar to that carried out at Pussellawa (Mallawaratchie, Rajapakse and Bandara (1994))

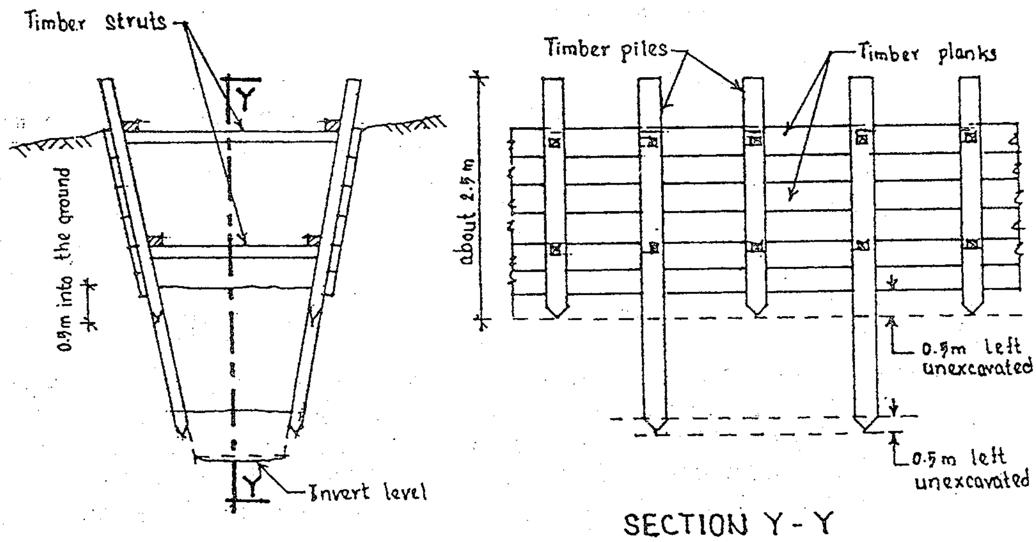


Figure 6: Sketch of shorings (second method)

About 25m length was excavated with the backhoe loader almost upto the final depth. However on the following day it was observed that the trench had been filled with mud and debris. The top 1.5m height was again excavated and timber piles (100mm x 75mm) were driven into the soil and then planks were fixed laterally at the outer periphery of the trench drain. Then the balance excavation was done manually upto the required depth. At the bottom level, shoring frames were positioned and braced exactly to the design section. Over excavated sections or cavities behind the shoring were back filled with sandy materials.

In some areas, boulders were encountered at certain levels. Breaking of rock was carried out by heating the rock boulders to a very high temperature and cooling it suddenly by spraying water on to the heated boulder. In this method, cracks were formed in the boulder and it was removed part by part by sledging manually. Explosive materials were not used generally for blasting of boulders since the vibration due to blasting could affect the stability of the landslide. However at a few places where the rock could not be removed by the former method, very light charges of explosives were used to break the boulders. In this case, sand bags were packed on the boulders before blasting to reduce the throw of blasted rock and the damages to the side slopes of the excavation.

After the final excavation and the construction of the shorings, a polytene membrane was placed on the bottom of the excavation and the down stream side of the trench drain as shown in Figure 4. The gauge of polytene used for this work was 0.75mm. The overlapping length of the polytene layers were maintained to a minimum of 750mm. To avoid the damages to the polytene sheet the entire sheet was not spread at once. The folded polytene sheet was gradually unrolled when the construction of filter drain was in progress.

The filter materials used for this trench drains were selected to satisfy the specifications given in Annex 1.

The shape of the trench drain was obtained by using formwork, but after filling and compaction of the filter material, the removal of formwork became a difficult task. To overcome this situation, the trench drain was constructed in steps of about 500mm height and the planks were removed after completion of each filling. Vibrating rammer plates were used for compaction of filter layers.

To separate the graded aggregate and sand in the core of the filter, at each filling, timber planks were used and removed after the filling was completed.

The water getting collected inside the trench drain was drained through perforated PVC pipes to the next drop structure by gravity as shown in Figure 7.

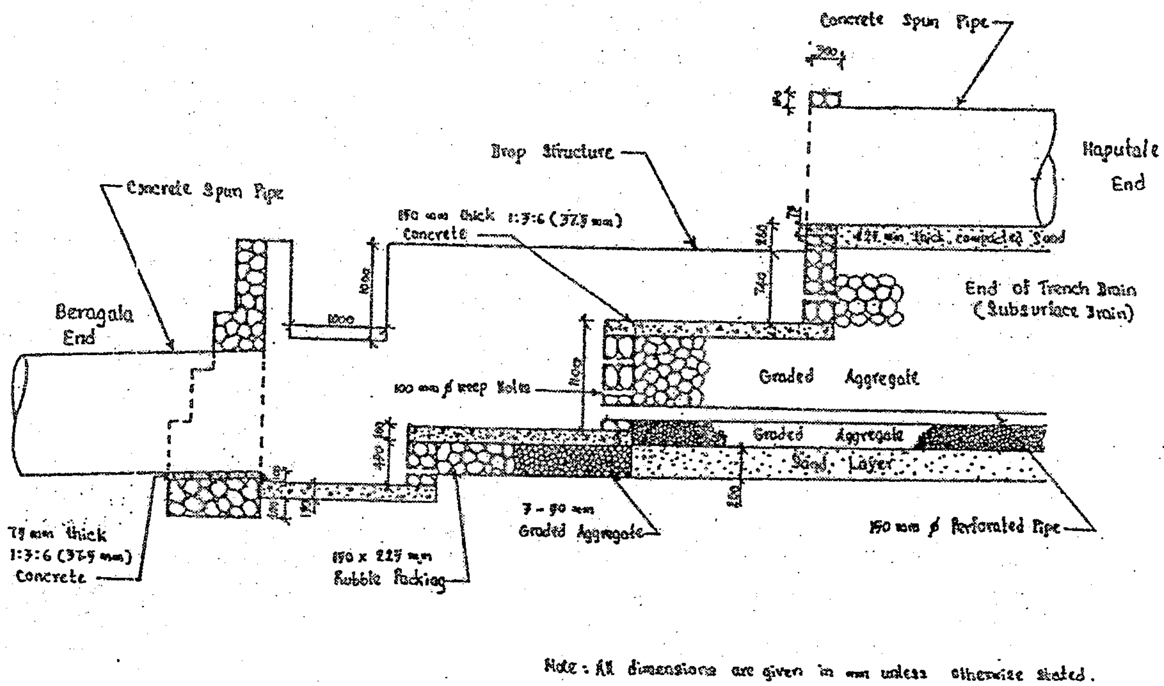


Figure 7: Sketch showing a drainage of trench drain to drop structure

The gradient of about 5^0 was kept for laying of the perforated PVC pipes to avoid any siltation inside the same and to suit the site conditions.

3.3 Construction of horizontal drains

Horizontal drains are small diameter pipe drains that are installed into hill sideslopes at a rising gradient of about 2^0 to 20^0 to the horizontal. These pipes are perforated at intervals in order to collect subsurface water through the perforations and transport same out of the hill sideslopes by gravity. (Perera (1994))

At Beragala landslide, it was observed that the ground water table rises up to the road level during heavy rains and remain at the same level for a long duration even after the rains have ceased. To over come this unfavourable condition, 3 Nos. horizontal boreholes were driven at 5^0 to the horizontal into the road embankment sideslopes at two elevations. The method adopted was similar to that given by Craig & Gray (1985). These boreholes were protected with suitable casings while boring. Subsequently 50mm dia. PVC perforated pipes were installed inside the bore hole and the casings were removed. Figure 8 shows the completed horizontal drain. This work was carried out by the drilling unit staff of the R&D division, RDA.

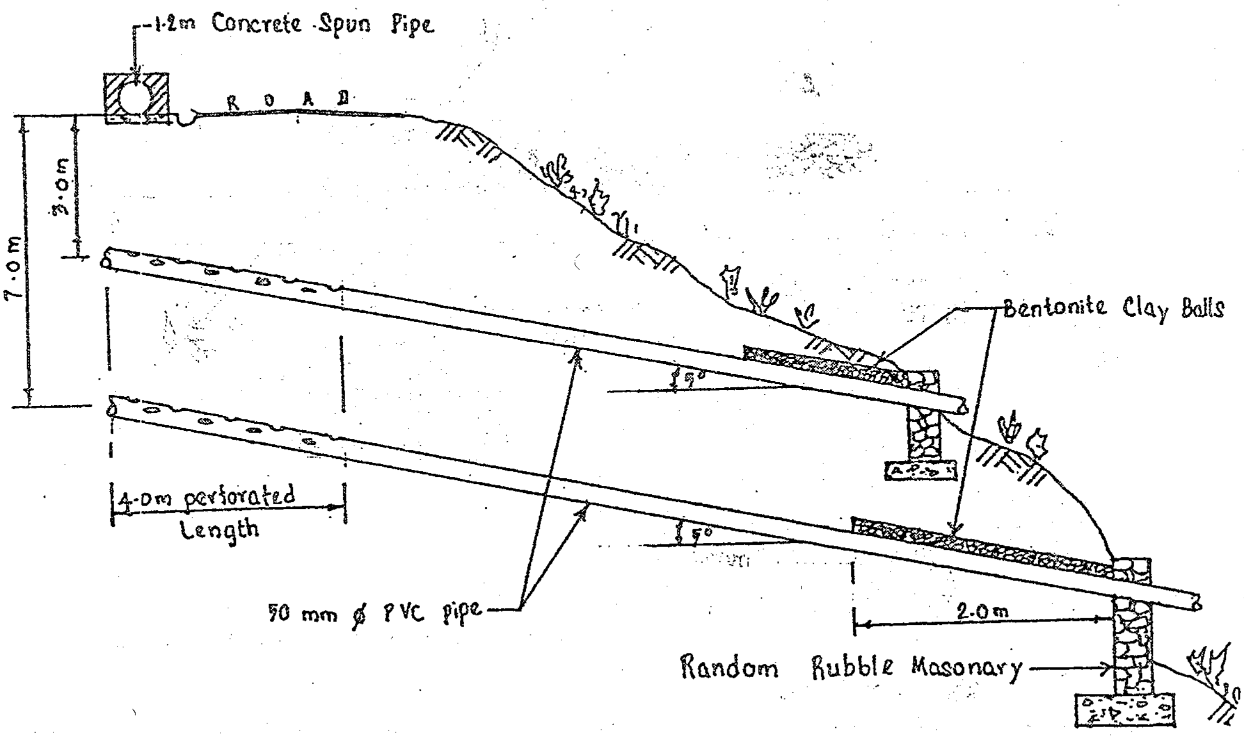


Figure 8: Sketch showing horizontal drain

Construction difficulties were encountered during the drilling operation due to boulders. To overcome this problem, 3 sizes of casings were used at different lengths of drilling, but the total drilling length was curtailed due to this problem. However at this location, it was possible to drill to lengths of about 40m. of horizontal boreholes ensuring that these drains pass under the entire road formation.

A highly jointed rock layer was encountered at the extreme ends of the boreholes. The perforated 50mm PVC pipes wrapped around twice with plastic meshes having hole sizes of 2mm were installed within that rock layer and subsequently connected to additional pipes without perforations as shown in Figure 8.

The space between the PVC pipe and the soil to a length of about 1.5-2.0m at the outlet of the horizontal drains were filled with bentonite clay balls in order to seal the water flowing outside the PVC pipe. A concrete end wall was constructed embedding the PVC pipe to protect it from soil erosion. The bentonite clay plug and the endwall at the outlet of a horizontal drain is shown in Figure 9.

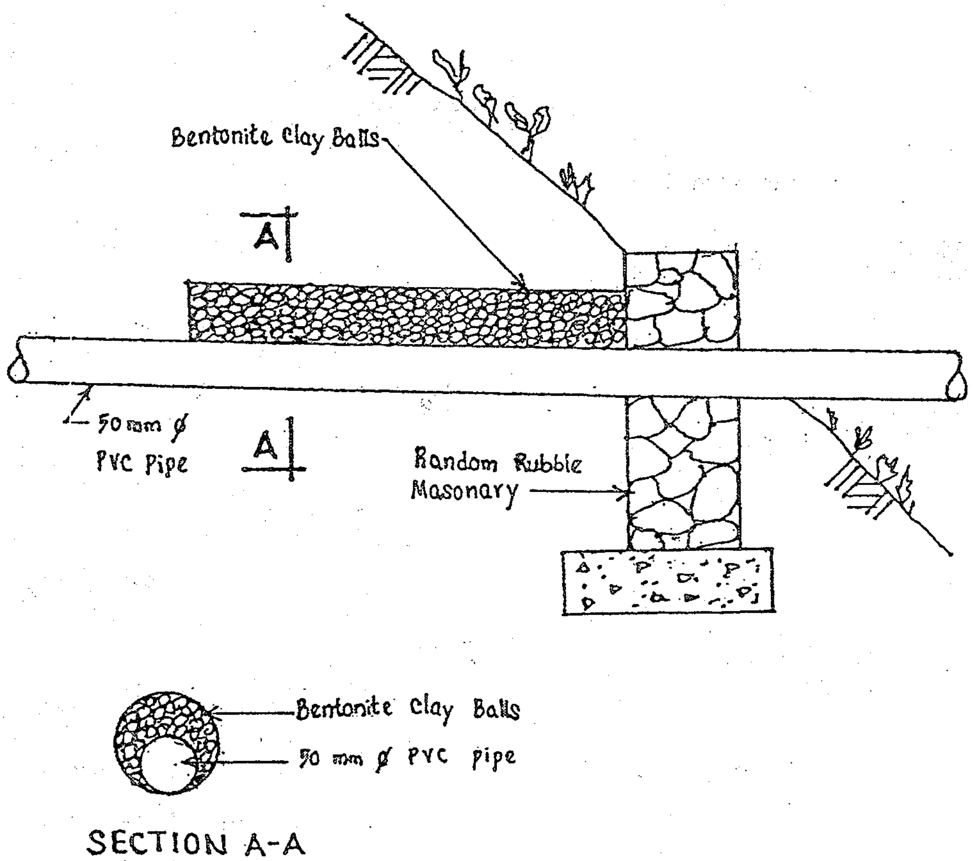


Figure 9: Sketch showing the bentonite clay plug and the endwall at the outlet of horizontal drain.

All water discharged from the horizontal drains will be diverted away from the landslide area to avoid the same infiltrating again into the soil at the outlet. As a result of this construction, the ground water table was reduced sufficiently to stabilize the road embankment slopes.

4.0 Remedial measures proposed in Stage II construction

Further mitigatory action proposed by Research & Development Division, RDA in order to control the landslide are as follows:

4.1 The water pockets located below the road A4 formation level to be back filled with a filter arrangement of coarse sand at the bottom followed by 40mm graded filter material and 100mm rubble and all stagnated water to be drained off from the landslide area.

4.2 A few more horizontal drains to be constructed to drain off the subsurface water above road (A16).

4.3 Sealing of the streams above road (A16) to be carried out without disturbing the original shape and the grade of the streams. Before sealing, all loose material of the bed of the stream to be removed and small boulders and pebbles to be packed in the bed. Then the bed to be sealed with an impermeable material such as bituminous material or cement mortar after temporary diversion of streams. When there is a drop less than 0.5m in existing stream, a 75mm concrete lining on a rubble packed bed to be constructed. If the drop is more than 0.5m, a special drop structure to be designed and constructed.

4.4 Tension crack covers to be provided by using bituminous jute fabric or polytene as shown in Figure 10.

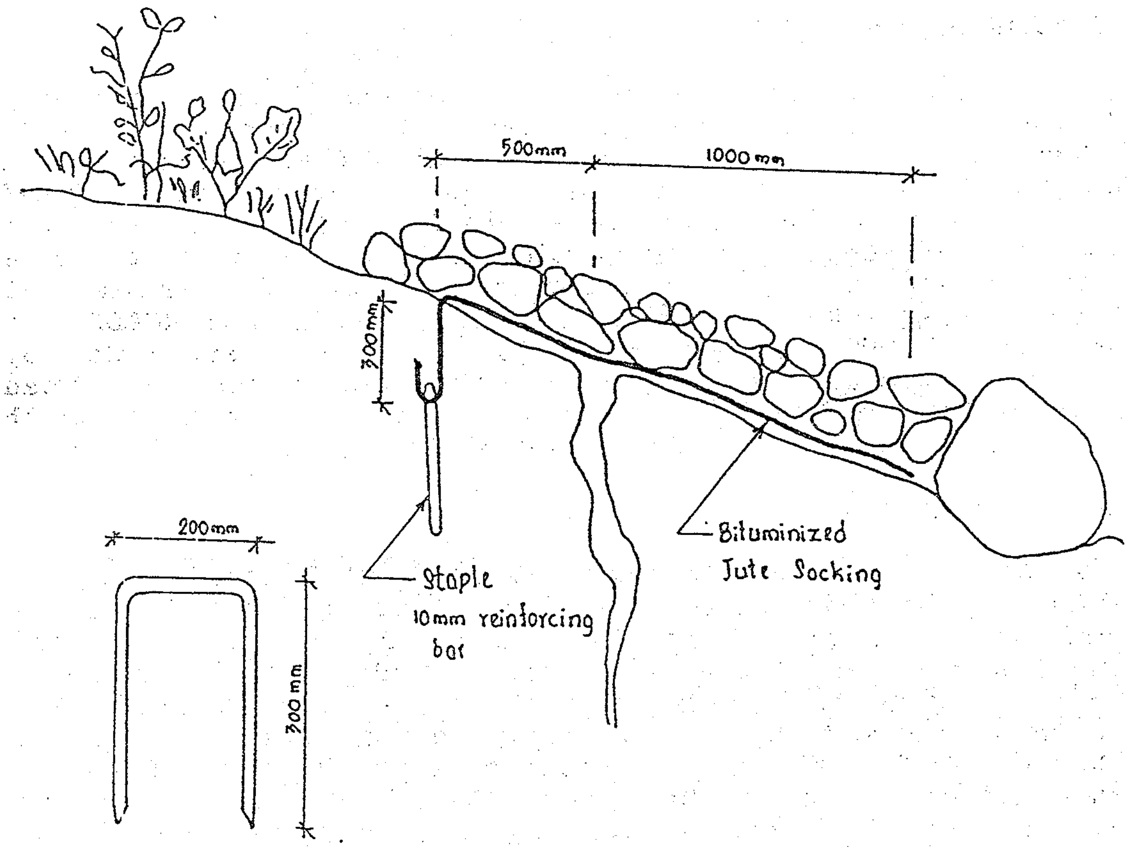


Figure 10: Sketch showing a tension crack cover

All surface cracks to be located and all woody plants growing within 1 meter of the crack to be cut off at ground level. Small light grasses to be left in place. About 500mm up slope of the crack, a shallow trench to be formed and suitable lengths of woven jute sacking secured with mild steel staples as shown in Figure 10.

The jute to be sprayed on both sides with an appropriate cut back bitumen. To protect the material from damage, two layers of stones to be laid on top supported by larger boulders laid in a depression in the ground, on the lower side of the crack.

Although jute sacking is available locally, it's long term performance as a water proofing seal is unknown. It may be more cost effective to use synthetic materials such as polytene or other impervious products.

4.5 A network of trench drains to be constructed below the road A16. the subsurface water from trench drains and the surface run off to be leadaway from the landslide area on a lined drain.

5.0 Concluding remarks

In this work a large quantity of surface water entering a landslide area has been diverted by constructing a 500m diversion drain using mainly 1.2m diameter concrete spun pipes. This is the first time, the construction work of this nature has been undertaken in Sri Lanka as a remedial measure to control a landslide. Pioneering efforts have been made on the construction of trench drains at Pusellawa on Kandy-Nuwaraeliya road by RDA & RC&DC and horizontal drains at Watawala railway landslide by Irrigation Department. These have encouraged the Road Development Authority to go ahead with this work with confidence and further develop these methods in Sri Lanka.

Acknowledgement

The authors wish to thank the Chairman and General Manager, Road Development Authority for granting permission to present this paper. They also wishes to thank the staff of the RDA offices of the R&D Division, Provincial Director, Chief Engineer, Bandarawela and Executive Engineer, Bandarawela and the Road Construction and Development Company (PVT) Ltd. Special mention has to be made of Mr. S. Nagodavithana, Senior Engineer, RDA for studying the area of the landslide and suggesting the construction of the diversion drain, Mr. Emil Fernando, Engineer, RDA for the design of diversion drain and Mr. N. Madusudhanan, consultant for the advice given in the design of the bell mouth of the diversion drain. They also wish to thank all those who have helped in the preparation of this paper.

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Annex 1

Specification of filter materials:

The grading curve of the filter should have roughly the same shape as the grading of the protected soil, and also should satisfy the following limitations.

a) $$\frac{D_{15} \text{ of the filter}}{D_{15} \text{ of the base material}} = 5 \text{ to } 40$$

Provided that the filter does not contain more than 5% of the material finer than 0.074 (No. 200) sieve.

b) $$\frac{D_{15} \text{ of the filter}}{D_{85} \text{ of base material}} = 5 \text{ or less}$$

D_{15} is the size at which 15 percent of the total soil particles are smaller. The percentage is by weight as determined by mechanical analysis.

D_{85} is the size at which 85 percent of the total soil particles are smaller.

Innovative Ideas and Techniques of Ground Improvement Through Geosynthetics.

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Abstract:

This paper presents state-of-the-art examples to introduce the development of innovative products and applications of geosynthetics in the world today. The variety in the basic products reflect the very wide range of applications that has been and are being developed. Geosynthetic have pervaded geotechnical engineer to the point where it is no longer possible to compete with the innovations without focusing the geosynthtics materials. The variety of applications and variety of functions performed by geosynthetics intimately linked with easy of handling, cost effectiveness and environmental benefits. Increasingly, the principal virtues described are in the areas of reinforcement applications, hydraulic applications, environmental applications and related research. Lessons learned from the earthslide case records are outlined and an approach to monitor the efficacy of membranes each suggested for situations similar to the one encountered. Hopefully, in future, geosynthetic regardless of the country or origin should meet novel techniques and standards acceptable to most operations and strongly global concerned for the environmental protection.

1. Introduction

Cities the world over are facing tremendous pressures in coping with the unparalleled demographic changes of this century. Just about every city is forced to house many millions of people more than the city's designers ever foresaw. The result is that the urban limits are forever moving outward and sites once considered unbuildable - swamps, marshes and other low-lying areas - are being rapidly reclaimed and labelled as 'prime sites'.

While reclaiming these low-lying areas by landfill is not difficult, confronting the subsequent settlements of the new fill is far more challenging. Many methods have been adopted to speed up the process of consolidation, so that, by the time the site is built upon, most of the settlement is over.

Depending on the various type of civil engineering problems, and the requirements it is possible to select from the varied range of geotextiles and geomembranes to obtain the optimum technical and economical solutions without compromise, regardless of whether the problems involves separations, protection, drainage, sealing, filtration and contaminations with the environment.

Geosynthetics offers a wide market oriented range of synthetics; woven and non-woven, geogrids and geocomposites as well as knitted fabrics for civil engineering applications. Raw materials such as polyethylene, polyporpylene, polyamide, polyester or armid are processed, using modern and efficient production facilities all around the world today.

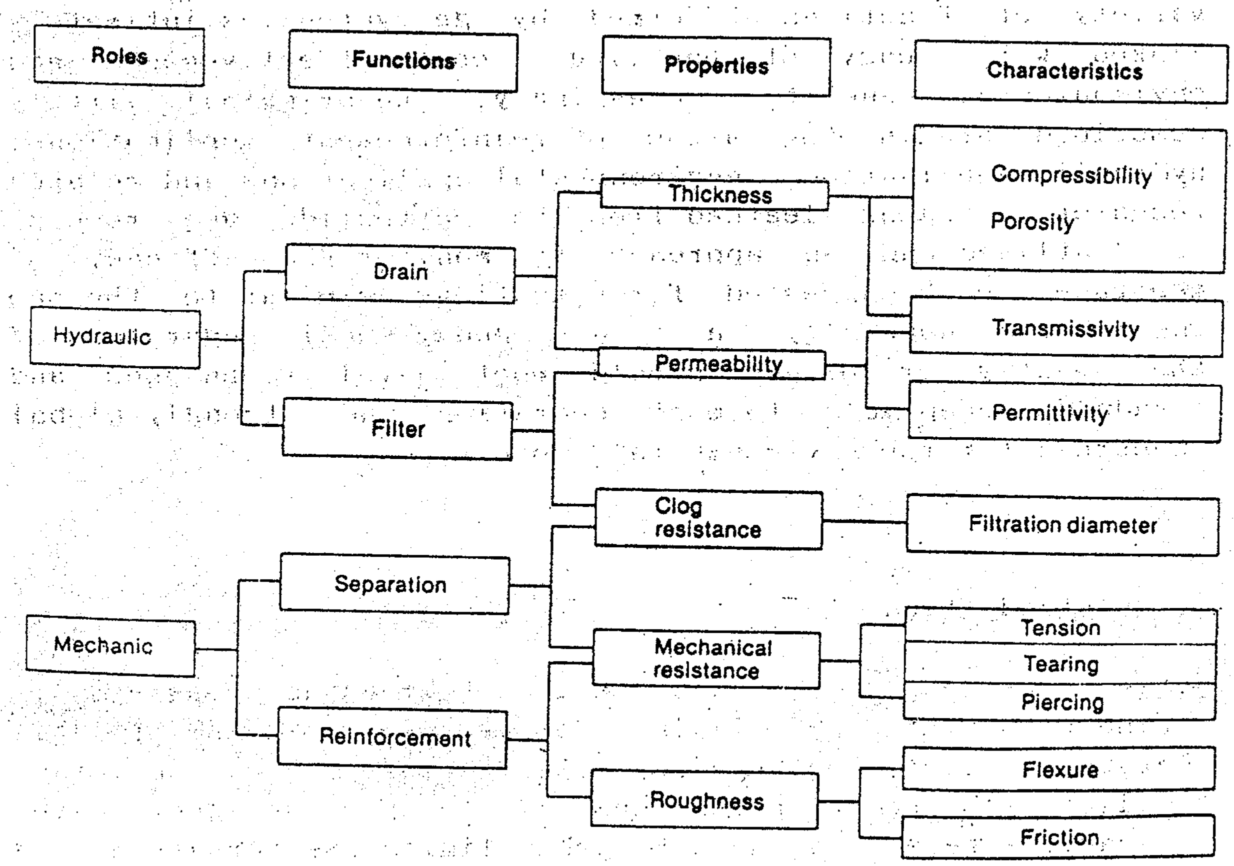
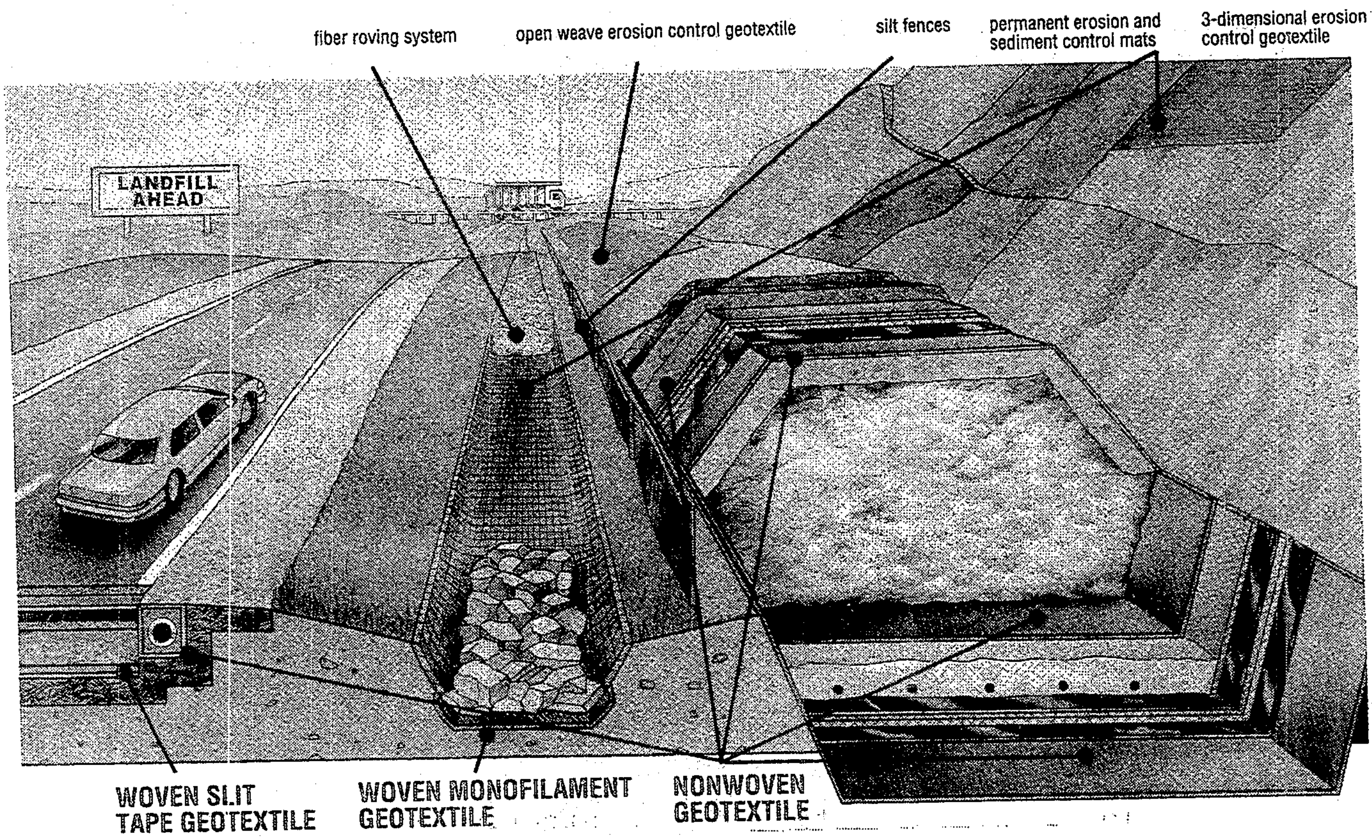


Fig.1 : Role of properties/characteristics as related tofunction (after Gicot and Perfetti, 1982)

2.Principal Functions of Geosynthetics.

Before entering into details of the engineering applications and the environmental aspects to be dealt with, it seams also convenient to recall the main functions performed by geosynthetics and related natural products (See Fig-1, Fig-2 and Table-2). The main functions could be briefly defined as follows:

(A) **Absorption**: the process of fluid being assimilated or incorporated into a geotextile (Frobel, 1987) or a bioproduct (Ranganathan, 1994). This function even if not usually mentioned, has to be emphasized essentially for two specific environmental aspects; water absorption in erosion control applications (typical of bioproducts) and also recover of floating oil from surface waters in occasion of ecological disasters (typical of geotextiles).



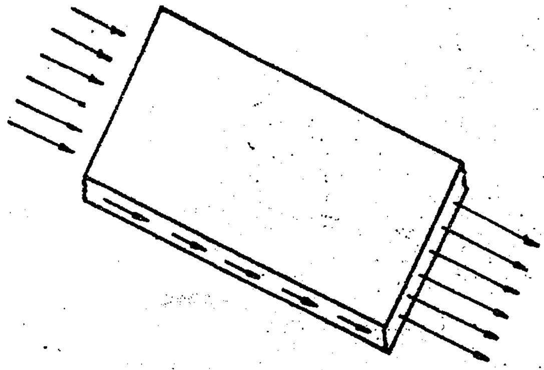
Looking towards an applications of Geosynthetics

Table-1. list of currently used geosynthetic and biodegradable natural products with related abbreviations (modified after IGS, 1993 and Rimoldi et al., 1993).

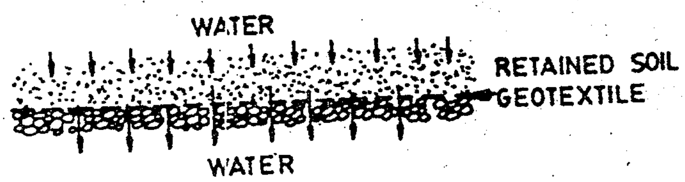
Abbreviation	Geosynthetic/Bioproduct
GT	Geotextile (generic)
GTW	Woven geotextile
GTN	Nonwoven geotextile
GTK	Knitted geotextile
GG	Geogrid (generic)
GGE	Extruded geogrid
GGB	Bonded geogrid
GGW	Woven geogrid
GN	Geonet
GS	Geospacer
GA	Geomat
GL	Geocell
GM	Geomembrane (generic)
GMP	Plastomeric geomembrane
GME	Elastomeric geomembrane
GMB	Bituminous geomembrane
GC	Geocomposite (generic)
GCD	Geocomposite drain
GCR	Geocomposite reinforcement
GCL	Geocomposite clay liner
GCM	Geocomposite membrane liner
BT	Biodegradable natural textile ("biotextile")
BA	Biodegradable natural mat (biomat")
BL	Biodegradable natural cell ("biocell")
BC	Biodegradable natural composite (generic)
BCD	Biodegradable natural composite drain ("biocomposite drain")

(B) **Barrier (to fluid)**: the ability of a geosynthetic to prevent migration of fluids (both liquids and gases).

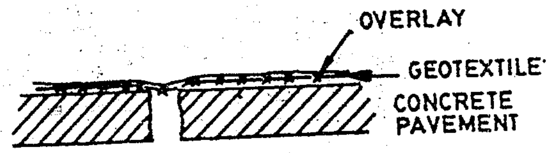
(C) **Cushion**: the ability of geosynthetic to control and eventually to damp dynamic mechanical actions (Jappelli and Cazzuffi, 1991). This function, even it is not usually mentioned, has to be emphasized particularly for the applications in canal revetments and is shore



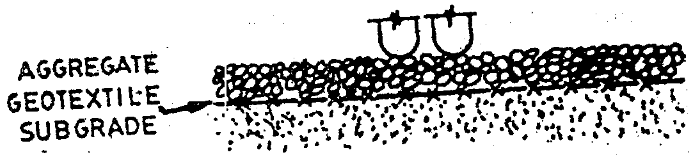
Fluid Transmission in a Geotextile



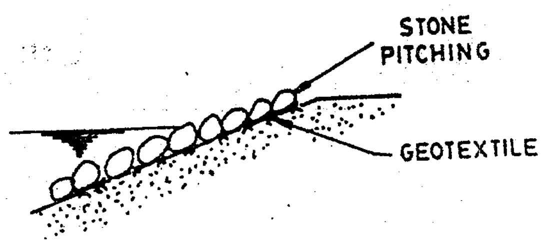
Filteration in Geotextile



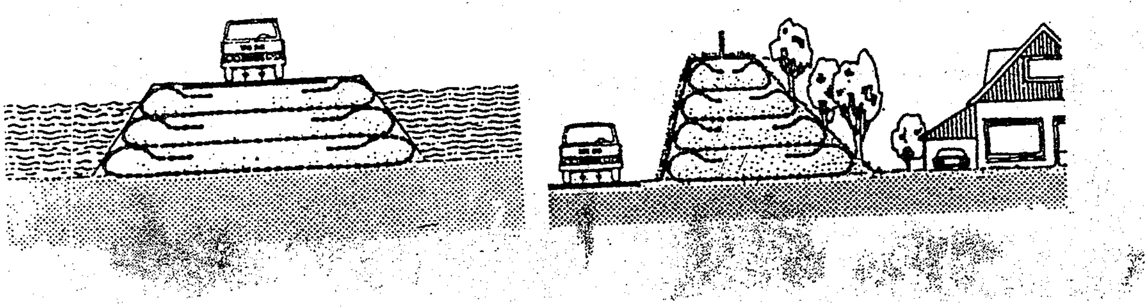
Interface Protection



Seperation with Geotextile

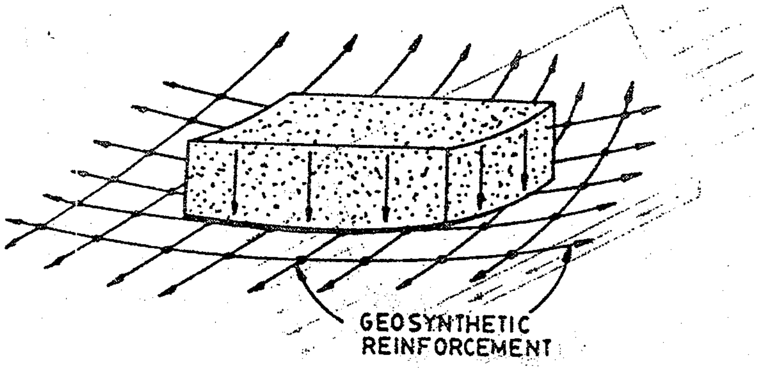


Surface Protection

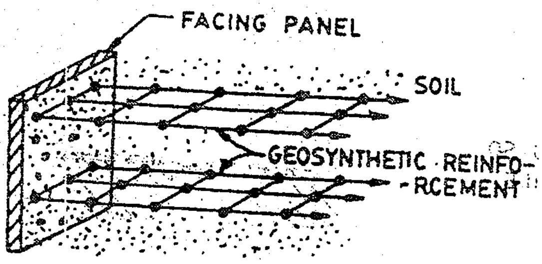


Geotext ile Reinforced steep Slopes

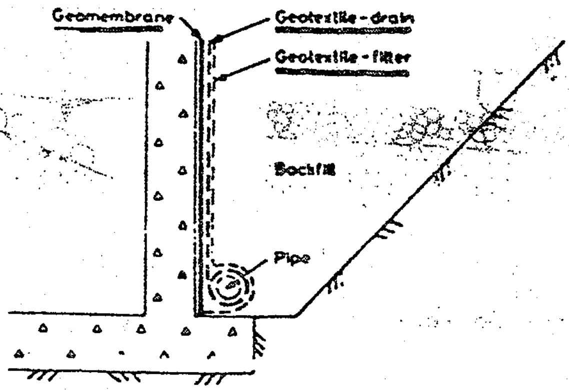
Fig.-2 : Principal Funtions of Geosynthetics (cont..)



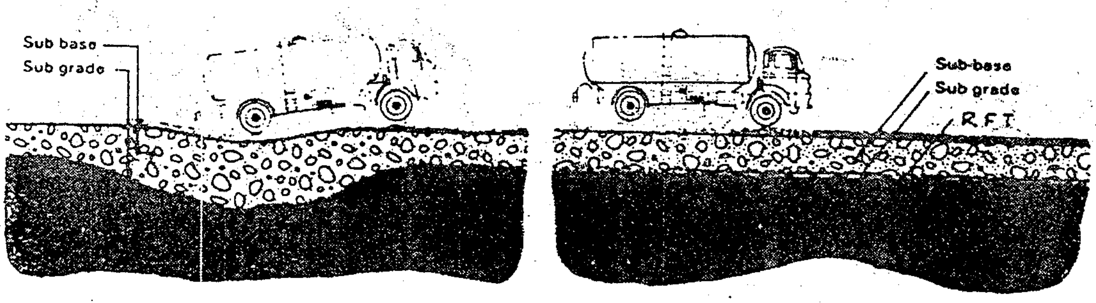
Tensioned Member Concept of Reinforcement



Geosynthetics as Tensile Member



Geocomposite for Protection of a Basement wall from Ground Water



Road surface deformation caused by penetration Stabilised road construction

Geosynthetics in Road Pavement Construction

Fig -2: Principal Fuctions of Geosynthetics protection incorporating geotextiles (dynamic loads induced by waves) and also for the applications in geosynthetic strip layer as seismic base isolation of earth structures (dynamic loads induced by earthquakes), as illustrated by Kavazanjian et al. 1991)

Table-2. Main functions with related abbreviations (modified after IGS, 1993) and typical geosynthetic and biodegradable natural products.

Main function and abbreviation		Typical products (see also Table 1)
A	Absorption	GT, BT, BA, BL
B	Barrier (to fluid)	GM, GCL, GCM
C	Cushion	GT, GN
D	Drainage	GT, GN, GS, GCD, BCD
E	Surficial Erosion Control	GT, GA, GL, BT, BA, BL
F	Filtration	GT
I	Interlayer	GT
P	Protection	GT, GN, GS
R	Reinforcement	GT, CG, GCR
S	Separation	GT, BT

(D) **Drainage**: the ability to collect and carry of fluids (water, leachate, gas) within a geosynthetic or, eventually, within a biodegradable natural composite drain (Lee et al 1994).

(E) **Surficial erosion control**: the complex function carried out by a geosynthetic or by bioproduct to prevent ground surface soil particles from detachment and transport.

(F) **Filtration**: the ability of a geotextile to retain soil particles while being crossed by flowing water or, eventually, leachate.

(P) **Protection (of geomembranes)**: the ability of a geosynthetic to prevent local damage to a geomembrane due to concentrated mechanical stresses.

(R) **Reinforcement**: the result of stress transfer from soil to a geosynthetic.

(S) **Separation**: the ability of a geosynthetic or, eventually, a bioproduct to prevent intermixing of adjacent soils and/or fill materials

In drainage applications for controlled extraction of water from soil, filters are used to prevent in-situ soil from being washed into natural or man-made drains. Such washed-in soils cause clogging of the drains and potential surface instability of a land adjacent to the drains.

To prevent reliable drainage system from silting up or clogging, geotextile is applied as filter material (^Fig-3). The structure of the non-woven geotextile ensures excellent porosity while at the same time retains soil particals. After eroding of very fine particles, a natural filter is created against cloth, preventing further erosion of the soil. If the pores of the filter fabric are far bigger than the particle sizes to be protected, then a continual piping will occur. If the pore sizes of the membrane are too small, then a small diameter bridging network will build up, resulting in a tight filter cake with low permeability.

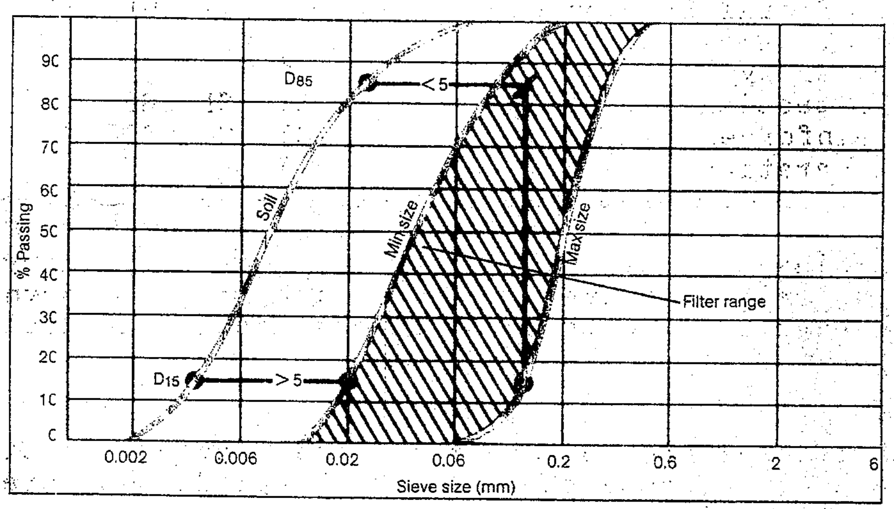


Fig.3: Design of filter operning size asa funtion of partical size distribution of soil

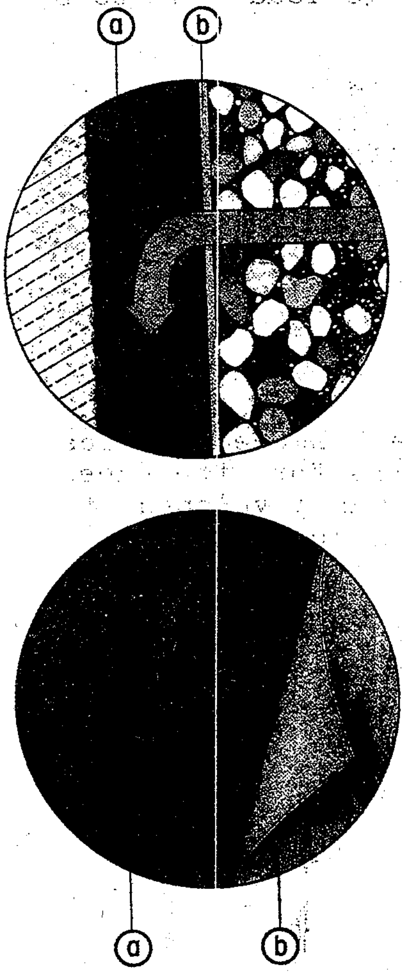
Therefore, low permeability geotextile is highly suitable for drainage trenches, as a filter blanket for rip rap protection on dillies, banks and erosion control of slope etc.

Special care should be taken when using Geotextilies as a filter membrane for very uniform soils, where such a natural graded filter cannot be built up.

Before finding any correlation to the soil particle size distribution it is necessary to determine the pore size of the fabric and particularly the maximum pore size (O_{max} which defines the size of the largest soil particles that can pass through the fabric. O_{max} is generally equivalent to O_{95}, O_{95} being the size of the pore for which 95% of all the pores are smaller

Drainage in Buildings and Underground Construction

Structural parts of buildings in contact with the earth can be damaged by moisture from the outside, by penetrating water due to seepage layers and also due to development of hydrostatic pressure. See Fig.-4 . In projects such as underground railway system, basement floors for underground garages and tunnels , a huge amount of water has to be coped with during and after construction. The solution to the problem is a perfect and long lasting seal, which quickly remove the water that collect and prevent the build up of hydrostatic pressure throught geosynthetics. Seepage layers ensure that water is taken away at low pressure without obstructing construction load due to



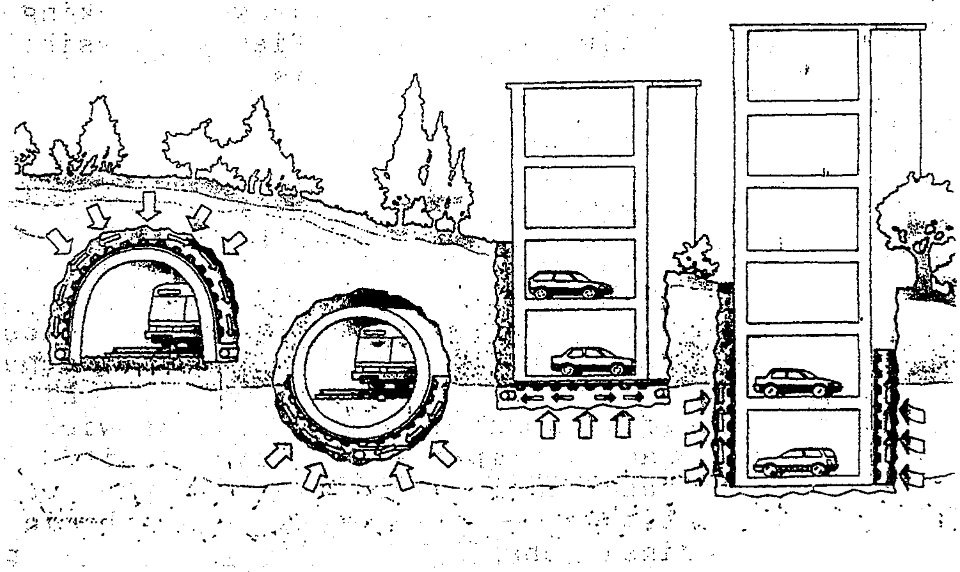


Fig. 4: Drainage in building in..udder ground constructions

Fig.5: Dimpled geomembrane sheet
a. dimpled b. geotextile filter

pressurized water and, has to be uniformed on all surfaces in contact with earth. This will provide rerouting of water condensing strata and finally the ground water must be taken out through suitable channel or piping system.

A dimpled geomembrane sheeting is an innovative application to cope with the above problem as shown in Fig.5 . Different stud heights make sure that water can drain away properly in horizontal or vertical seepage layers. A fabric filter prevents ground particals from floating in to drainage layer. In some cases thermic-welded plastic mesh layer allows dimpled sheeting to be used as the base for shotcrete or mortar. These application are very common in various civil engineering projects, such as and underground construction walls, retaining wall, bridge abutments, tunnels and roof slab covered with earth etc.

Soil Settlements through Vertical Drains

The original concepts of "vertical drains" was developed in the 1920s by creating sand columns in the ground. The sand columns acted as drains since they were more permeable than the surrounding clay/silt. Sand drains are essentially a series of vertical sand columns laied in a close proximity grid. Although the sand drains worked well in ideal conditions, there were however many problems.

* The sand filled columns had no shear strength at all and thus shear failure while placing the surcharge load was very common. the surcharge load had to be placed slowly and carefully.

* During the installation of the drains, when the mandrel is withdrawn, 'necking' of the hole was always an unverifiable possibility; and too many drains would be discontinued without being know.

* Boring of the hole required heavy machinery.

* Sand was not always available in the required quality and quantity.

In order to accelerate this process, closely spaced vertical drains are installed to shorten the flow paths for the excess pore water. In the 1940s Mr. Walter Kjellman developed the first prefabricated vertical drain ('wick') which consisted of a few channels imprinted into a stiff cardboard core. The concept was further developed in the early 1970s with the introduction of drains using a synthetic drainage core with longitudinal 'channels' enveloped in a paper or non-woven

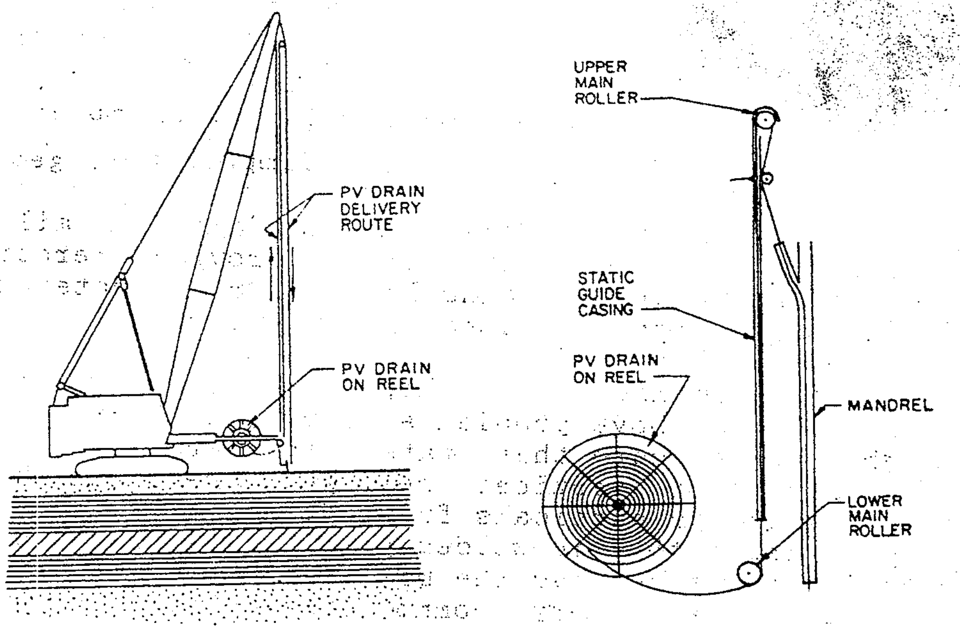


Fig.6:Installation of prefabricated vertical drains

fabric. The composite of geotextile and geomembrane products which includes the grooved drains ensure that the adequate discharge capacity which is maintained at all the times, even under the most sever conditions. The large number of commercial products are available in a choice of filter fabrics to suit various soil conditions and engineered practices. Example of such fabric and the mechanism of the activity as shown in the Fig.-6.

The pressurised pore water flows horizontally towards the nearest drain, where it can then escape freely. This drainage takes place by means of synthetic "wick drains" composed of a rigid core with high discharge capacity jacketed with a filter fabric, of Geomembrance.

By this method the settlement period is tremendously reduced, thus permitting the consolidation process to be completed during the construction period.

The filter jacket is the key element of the prefabricated drain. it must fulfill several very important requirements:

* great permeability even under high compression so that the excess pore water can escape freely;

* at the same time excellent filtration properties which reliably prevent fine soil from entering the draining core even under high hydraulic gradients;

* high tear resistance to withstand damage during installation;

* dimensional stability to resist penetration into the core corrugations under high soil pressure.

Recently in order to speed up the consolidation of soft clay layer geotextiles covered geomembrane approved drains were installed at part of the road access project of Port of Colombo Extension Project, Sri Lanka.

Soil Reinforcement on Road Construction

As a result of varying wheel load, the foundation layer of a road construction can mix with the unstable sub soil, which greatly decrease the bearing capacity of the foundation. Application of separate layer is common for foundation for motorways, airport, railroads, water disposal sites etc.

As a protection layer on the top and bottom of watertight geomembranes, geotextile prevent vulnerable liner, from being damaged by sharp stones as shown in Fig.12 . A thick and strong non-woven geotextile not only protects the liner, but also forms a drainage layer which drains off percolation water or change the direction of excess pore pressure. This application replaces the traditional layer of sand, enabling steeper slopes which result in a considerable increase in the water capacity.

When placed between subgrade and aggregate base, Geotextilies fulfils 3 important functions.

Separation

Geotextilies prevents the fine-grained subgrade from mixing with the aggregate base. Bearing capacity will therefore remain unaffected.

Geotextilies prevents the loss of aggregate into the soft subgrade. Therefore a higher degree of compaction is obtained, with subsequent better bearing capacity. Construction can be undertaken in adverse weather conditions and therefore time can be saved.

Basically, in any situation where the fine-grained subgrade can be affected by subsurface water, the separation function of Geotextilies will definitely increase the road service life.

Filtration

The range of opening sizes of Geotextilies allows free passage of subsurface water while preventing migration of fine-grained subgrade towards the aggregate base.

Aggregate and traffic loads will squeeze out excessive pore water from the subgrade that will lead to an increased shear strength and greater resistance to rutting of the subgrade (consolidation).This clearwater can be readily drained away by sloping the subgrade to one side of the road and installing longitudinal drains.

The thickness and the water permeability of the aggregate base being important compared to the thickness and in-place permeability of the fabric, the "drainage" function of the fabric is irrelevant in pavement structures.

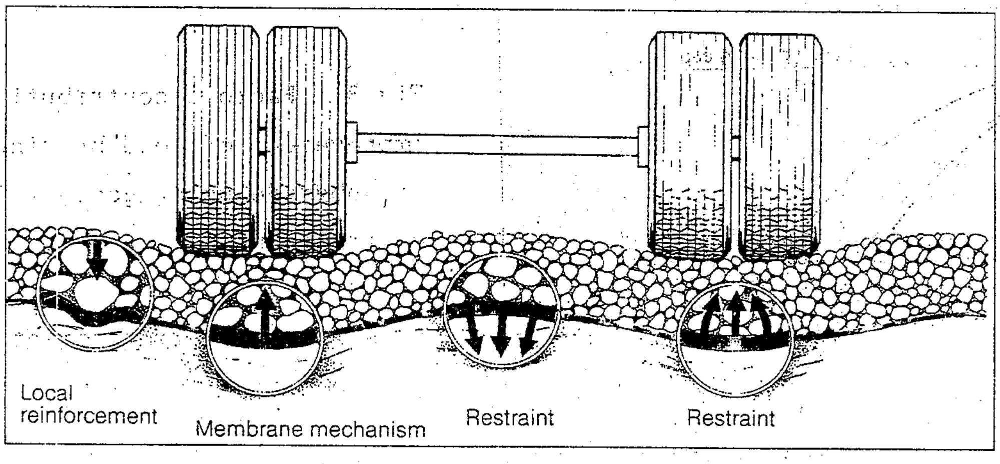


Fig 7 : Three distinct mechanisms by which geotextile reinforces an aggregate-fabric-subgrade system

Reinforcement

There are three distinct mechanisms by which Geotextiles reinforces an Aggregate-Fabric-Subgrade (AFS) system and improves its resistance to permanent deformation under repetitive loading (See Fig -7):

i. **Restraint**

In this mechanism there are two types of restraint.

One is related to the reverse curvature of Geotextilies outside the wheel path where a downward pressure is created which has the effect of a sucharge load. The result is an increase of subgrade bearing capacity underneath the wheel path.

The other is the restraint Geotextilies provides when aggregate particles adjacent to the fabric attempt to move away from under the loaded area. This confinement of the aggregate increases it s strength and moddles, which in turn decreases the compressive stress ont he subgrade underneath the wheel load.

ii. **Membrane Mechanism**

As the AFS system undergoes deformation due to the applied loading, Geotextilies develops in-plane tensile stress which induces a component of stress perpendicular to the plane of the fabric.

Rutting and the resulting spread of the load by the geotextile continues until an equilibrium is reached, that is, until the subgrade can bear the distributed load without further plasitc deformation.

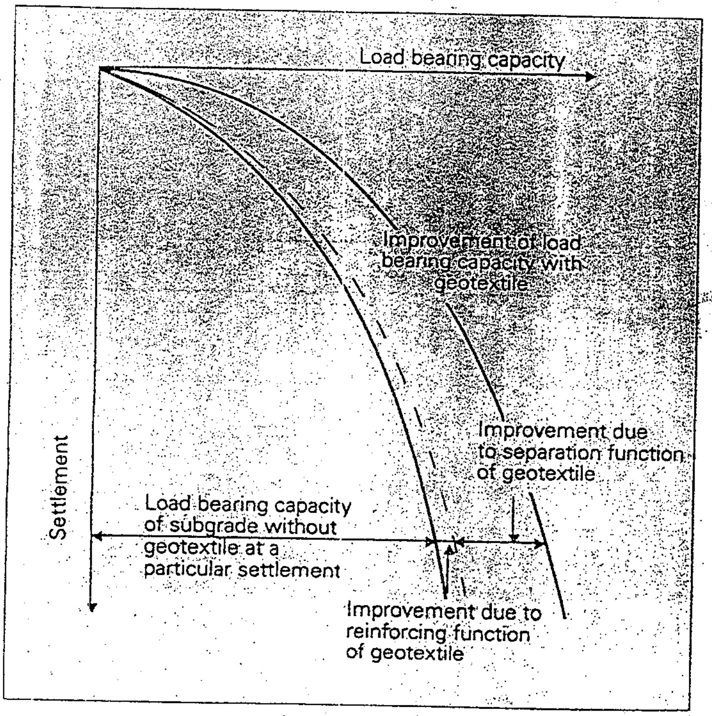


Fig 8: Factors contributing to the improvement of load braring capacity of subgrade using geotextile for road costruction:

i. significant improvement due to seperation function of geotextile.

ii.neglegible improvement due to reinforcing function of geotextile.

iii. Local reinforcement

Loads on individual stones can cause spot failures in the subgrade. Geotextilies distributes the load, reduces the stress and provides resistance to displacement.

Since the relative contribution of separation, filtration and reinforcement functions vary in importance with different aggregates, subgrade and other local conditions, it is necessary for the fabric to possess a good balance of properties(Fig. 8).

Geomembrane to Control Movements of Rain Induced Landslides

The landslides are belong to a part of natural slope stability in geotechnical engineering. It has been well known that the causes of landslides or natural slope instability is closely related with water ingress into the ground. Surface drainage was the simplest application to reduce water ingress into subsurfaces. Also trench drains and subsurface horizontal drains are constructed to collect surface water and lead it to outside the sliding area. But a relatively large amount of water percolate into the ground before the water reaches the surface drainage system is in common in high rainfall intensity areas. Therefore, certain added considerations must be taken into design and placement of membrane systems when aiming to prevent water ingress through slide shear boundaries or through slope tension cracks of a landslide.

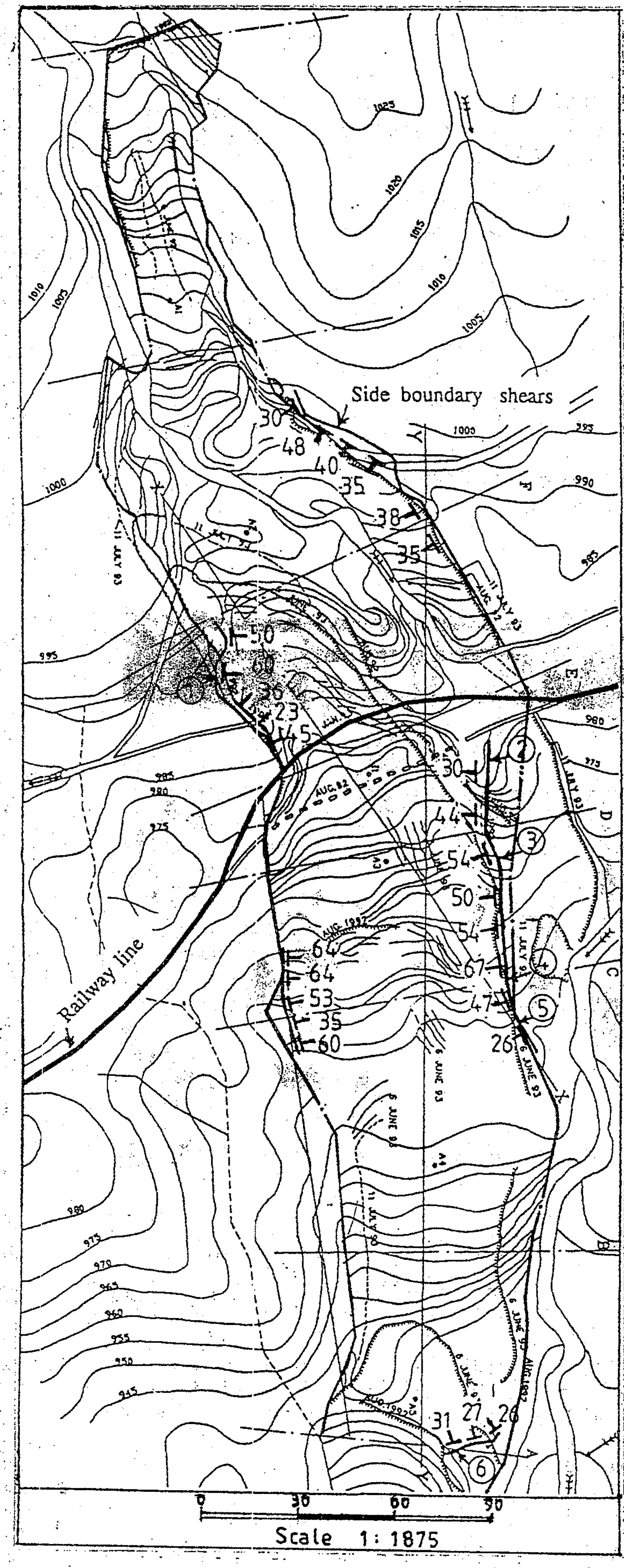
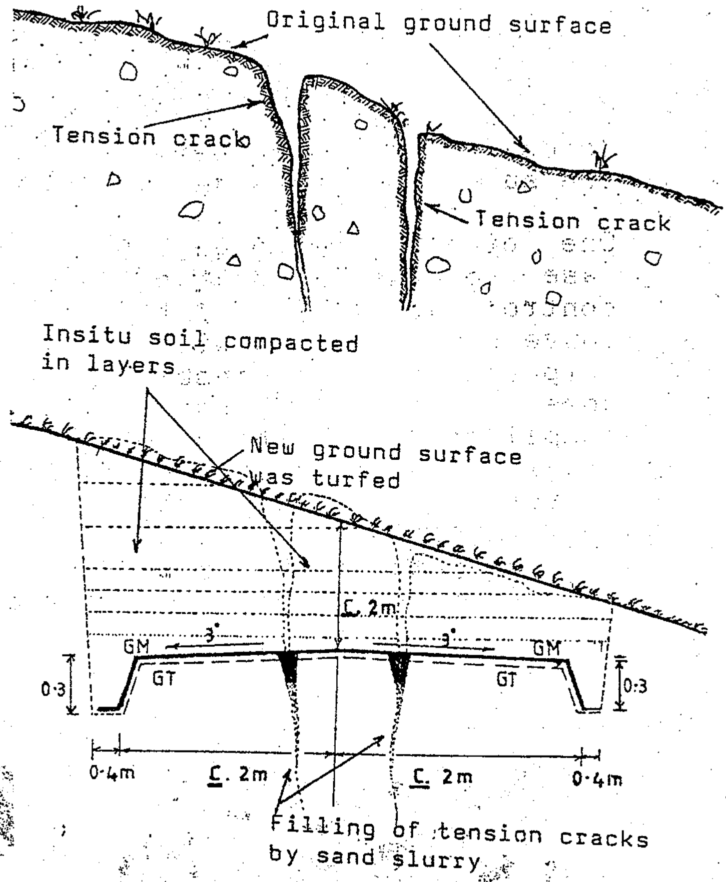
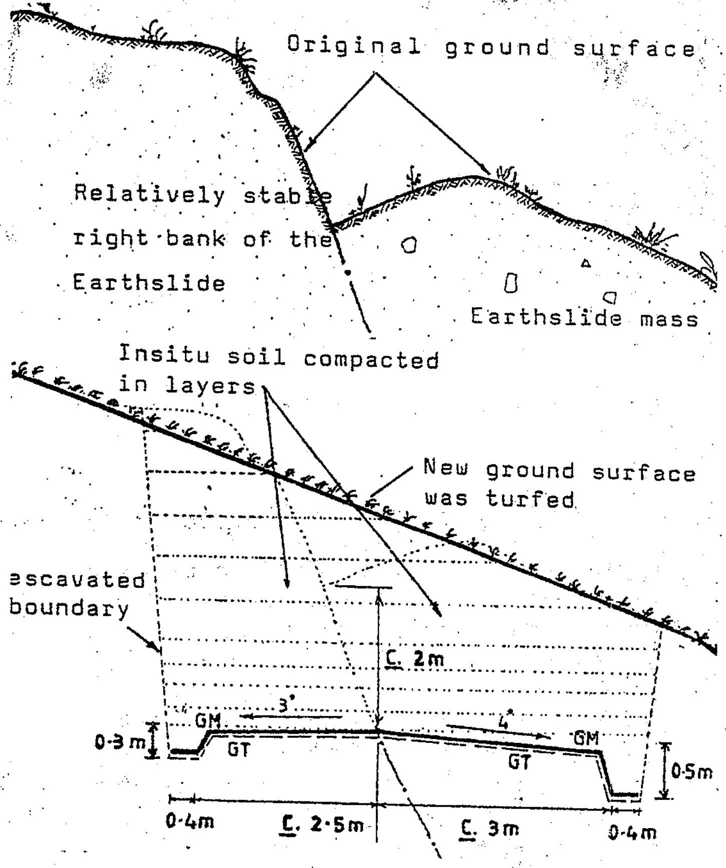


Fig 9:Paln of Watawala Earthslide



Sealing of tention cracks



Sealing of shear boundary

The earthslide at Watawala in Sri Lanka and Jitsukiyama landslide in Japan were taken as case records to convey that the success of sealing by membranes(Fig - 9).

One of the most significant lessons learned from the above case records is that techniques of membranes for landslide control are very different from those of their using conventional or routine type of projects, although the basic purpose of water insulation may be more or less the same. However, some of the rain induced landslides are very complicated in their mechanism, it is very difficult to predict the location of sliding surfaces and the amount of pattern of rainfall which causes the failure. This was the obvious identification of Jitsukiyama landslide in Japan and surface sealing using geomembrane was extended to cover wide area from the assumed boundary of the earthslide.

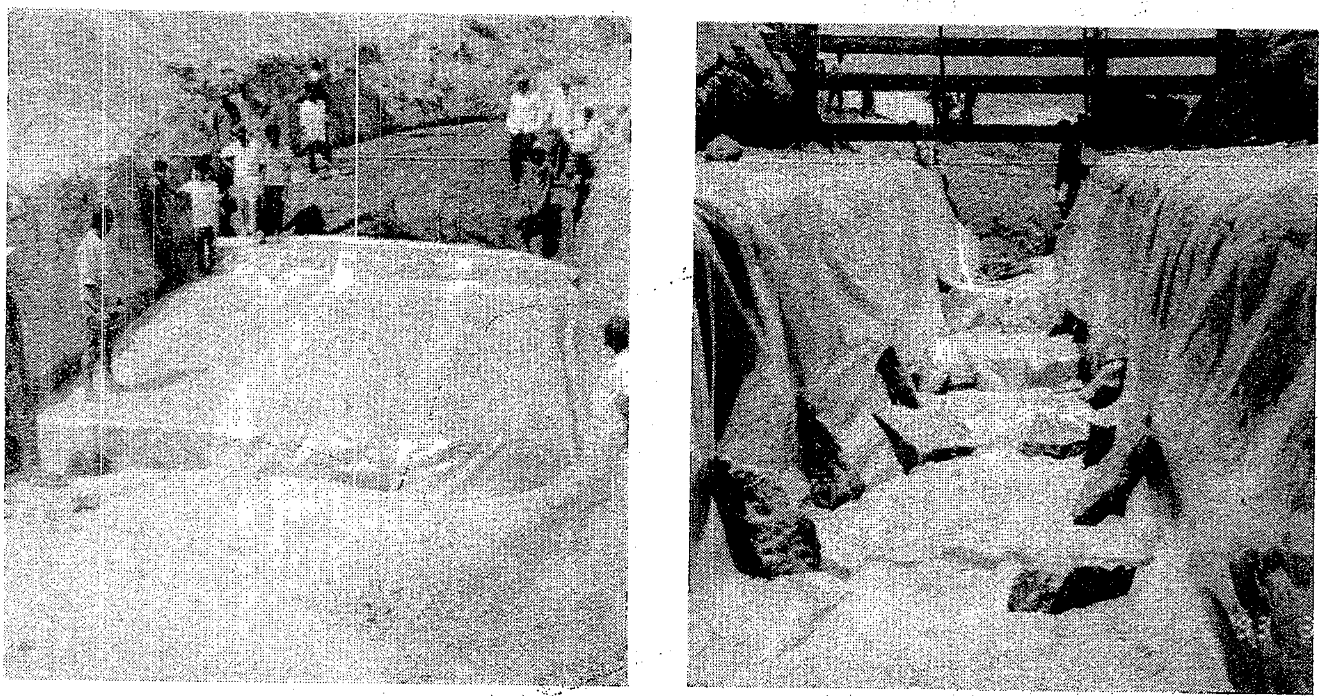
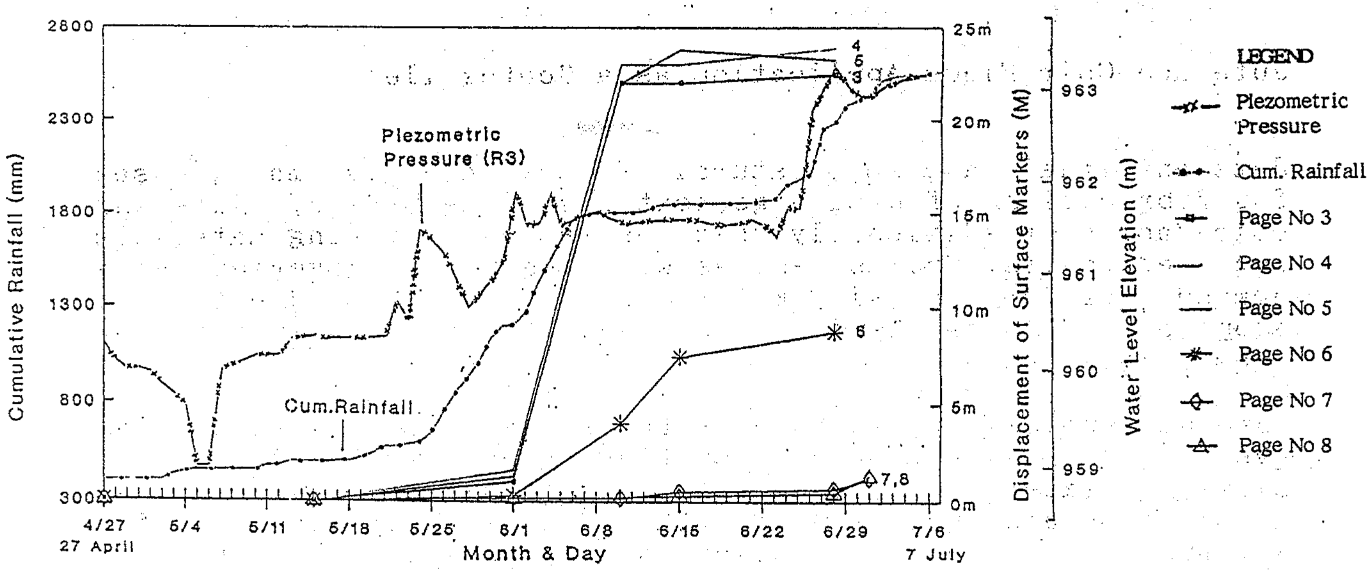
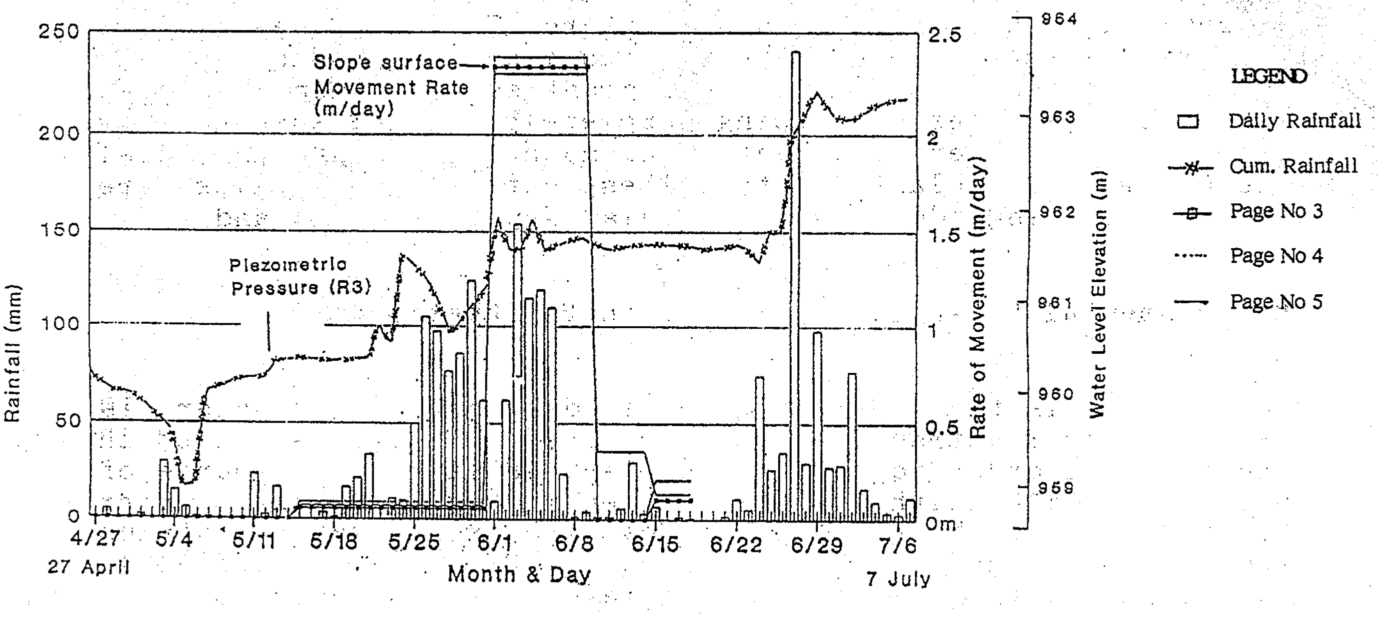


Fig 10 : Sealing of boundary shear surface and construction of trench drain using geosynthetics as an interim control measures.

In earthslide control works a membrane system is just one of the many elements of an overall remediation package and wholly depends for its efficacy of the collective affect of all the remedial measures. Rigours of membrane specifications for a given job or care to detail of the procedure of its placement are not enough in themselves to achieve end results. The considerations of membrane capacity to absorb large strains as also their frictional characteristics on slope to inhibit slippage at the interface are quite important in the case of landslides.



(a)



(b)

Fig 11: Correlation between Rainfall, Piezometric Pressure , Displcement of surface markers andSslope Surface Movement Rates.

Jute and Coir Finds Application as a Geotextile

Jute and kenaf are soft natural fibers which is mainly used for fabrication of woven geotextiles. Time before, jute and coir fabric traditionally used as various packing materials for vegetables, fruits and as well as floor covering. As a demand for jute packing materials as declined, due to the adoption of bulk handling and due to the competition of geosynthetics.

Jute geotextile a woven mesh of thick jute yarn, when applied over soil, provide thousands of tiny check dams to impede the flow of water and help soil stabilisation and allowing germination and successful growth of vegetation. The material absorbs five times its weight of water and this water is slowly released to the soil for proper growth of vegetation. As the material is made of natural fibers it biodegrade gradually over a period of two years providing nutrients to the soil.

The coir geotextile have a very high tensile strength enabling them to hold the soil together until plants and trees are grown. Coir is strong cellulose fibre with a high lignin content (58 percent which is higher than that in teak wood. It is resistant to rot, mould and moisture. The life-span of coir geotextiles ranges from five to ten years depending on the environment and this allows full plant establishment and land stabilisation. Besides being completely biogegredable they also absorb water and act on much providing an ideal microclimate of plant growth. These natural geotextiles are produced in Bangladesh, China, India, Nepal and Thailand.

Environmental Protection Measures in Reclamation

Reclamation is the work in which a defined area of water is enclosed by revetment and soil or sand or waste is filled in the area in order to develop a piece of land. One source of water pollution with the reclamation work is the extra water flowing out of the reclaimed area. The extra water contains suspended, organic and hazardous materials and oily substances. Generally, hazardous materials are considered to be removable and are attached to the suspended materials.

The free-standing Silt Protector (see Fig-11) will support the curtain from the sea bottom utilizing a sand discharge pipe or the like.

This protector is suited to large-scale dredging work conducted at great depths where of the sludge is expected to diffuse over a wide range. In case of movement, air is sent

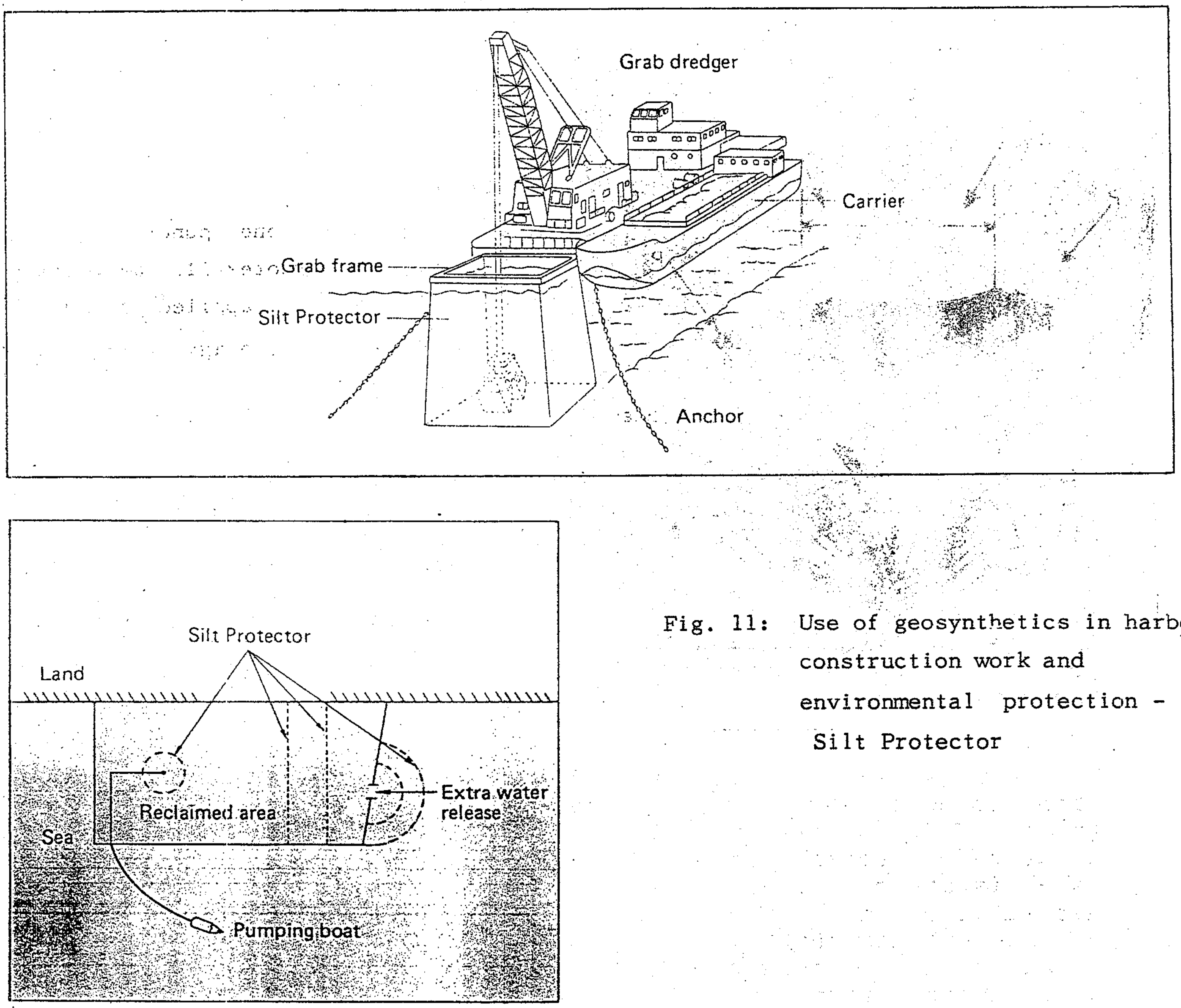


Fig. 11: Use of geosynthetics in harbour construction work and environmental protection - Silt Protector

into the sand discharge pipe to raise it to surface, and the protector is dragged by barge.

Resistance to Installation and Construction Damage

For effective separation performance, the geotextile must not be punctured or damaged during construction. During fill placement, especially when large size, sharp angular fill materials are present or when insufficient fill thickness is adopted, the geotextile is highly susceptible to puncture and damage. The latter requires that a minimum design thicknesses maintained throughout the construction process.

Construction damage due to stone aggregate puncturing through the geotextile is the most critical form of damage likely to occur. Thus, the geotextile must fulfill minimum puncture resistance criteria. Geotextile performance or test criteria that presumes damage and does not ensure prevention (i.e. tear resistance) is not relevant and should not be used as the

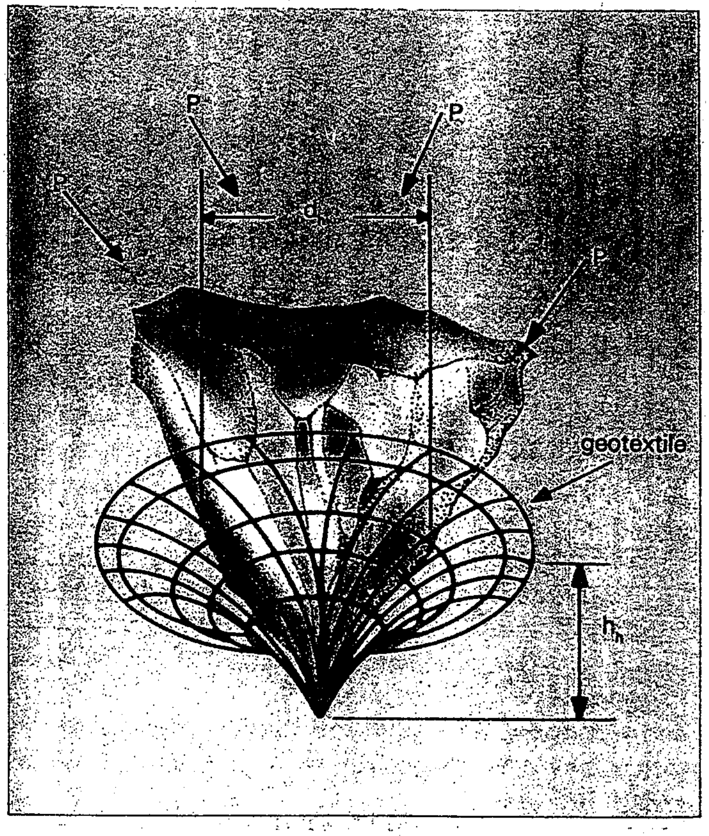


Fig. 12 : Stone puncturing a g geotextile as presure is applied on the stone aggregate.

basis for design. It is more rational to design to avoid puncture by specifying minimum puncture resistance requirements than to allow damage and attempt to limit subsequent tear propagation.

To prevent the geotextile from puncturing during construction, the following influencing parameters must be evaluated to determine the anticipated puncture force:

* Initial thickness of fill above the geotextile which is a function of subgrade CBR,

* Presence of stones in the fill especially in laterite soil (i.e. 50mm mean diameter),

* Type of construction vehicle, wheel load and contact area and thus the pressure exerted at the elevation of the geotextile.

Conclusion

The last decades have seen impressive progress in geosynthetics technologies and applications and substantial developments of design methods as conveyed by the many international conferences, symposia and technical publications dedicated to the paper highlights. The recent evolution of the state of the art shows a growing interest of researchers and users towards the development of the products contributing to effective environmental protection.

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