SRI LANKAN
GEOTECHNICAL CONFERENCE
1992

Theme

Ground Development Techniques

Organised by the

SRI LANKAN GEOTECHNICAL SOCIETY

DATE : February 8, 1992

Venue : Auditorium, Sri Lanka Association
for Advancement of Science
Vidya Manderaya
Vidya Mawatha, Colombo 7.

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Sri Lankan Geotechnical Conference 1992
Programme

Registration

8.15 – 8.55 Registration of Participants

Inauguration

09.00 – 09.10 Inauguration address by Prof. A.Thurairajah,
President Sri Lankan Geotechnical Society

09.10 – 09.50 Introduction to Ground Improvement Techniques – Mr.K.S.Senanayake

09.50 – 10.00 Tea

10.00 – 10.50 Mechanical Soil Stabilisation – Dr.T.Sivapatham
10.50 – 11.30 Soil Stabilisation for Road Works – Mr.D.P.Mallawarachchi
11.30 – 12.10 Discussions

12.10 – 13.00 Lunch

13.00 – 13.40 Geotextiles – 1
13.40 – 14.20 Geotextiles – 2
14.20 – 15.00 Foundation Techniques in Marshy Areas
Dr.J.J.P Ameratunga & Mr.K.S.Senanayake & Mr. H. D. J. P. Samaranayake
15.00 – 15.50 Discussions *
15.50 – 15.55 Closing Session

16.00 – Third General Meeting of the Sri Lankan Geotechnical Society (Members only)

Sri Lankan Geotechnical Conference 1992
CONTENTS

INTRODUCTION TO SOME GROUND IMPROVEMENT TECHNIQUES

Kirthi Sri Senanayake,

Geotechnical Consultant,

National Building Research Organisation

1.0 INTRODUCTION

Worldwide, economic and technological development has advanced at a rapid rate in the recent decades resulting in an increased demand for large scale and accelerated construction activities. However, due to scarecity of easily buildable land space in many parts of the world, especially in the congested urban areas and highly populated suburbs where land prices are exhorbitantly high, today the planners are often left with no better choice than to utilise lands which were hitherto considered as not suitable for construction purposes owing to inherent poor ground conditions and the related financial & technological constraints. Therefore, the engineer today has to face the challanges of developing and constructing on difficult terrain with steep slopes which are perhaps prone to landslides; in valleys and coastal areas with alluvial or lacustrine deposits of weak and soft clays or loose sands; low lying marshy lands with thick deposits of highly compressible peat and other organic soils; and on ground consisting of expansive soils or other problematic soils. However, on many an instance, the engineer is capable and confident to take up such challange, thanks to the modern technologies developed in the field of geotechnical engineering especialy during the past few decades which have given versatile solutions to many problems associated with poor ground conditions.

The geotechnical engineer, in tackling difficult ground conditions, has the options of adopting foundation treatment or ground treatment or a combination of the two. Where these measures alone cannot help, he may consider reviewing the design to make possible alterations in the structure to be constructed so as to suit the prevailing ground criteria.

In this paper some conventional and recently developed ground improvement techniques which are being widely used today are presented. Some techniques used under special circumstances are also mentioned for general information. Among them, there are methods which are considered suitable for local conditions in Sri Lanka, perhaps with some modifications, for example in applying to the development of low lying marshy areas, road works and slope stabilisation.

TABLE 1. CLASSIFICATION OF GROUND IMPROVEMENT TECHNIQUES

OBJECT OF IMPROVEMENT & METHOD OF INSTALLATIOI CONSTRUCTION	NAME OF THE METHOD (Note 1)
A. ADJUSTMENT IN STRUCTURE OR FOUNDATION	
Counter-berms (for stability of embankments)	Counter berm method
Reduction of Load (use of lighter materails)	Expanded polysterene (EPS) method
Strengthening of Structure/Foundation	
Sheet Piling (against seepage,earth pressures)	
Change of Foundation Type	
B. IMPROVEMENT OF FOUNDATION SOIL/GROUND	
1. REPLACEMENT OF POOR SOILS	
Total replcement by excavation/dredging	Rolled and compacted fill method
Partial replacement by excavation/dredging	Hydraulic fill method
Partial replacement by water jetting	Replacement in suspension method
Extrusion by earth banking	Earth banking method
Extrusion/compaction by Blasting	Blasting method
2.COMPACTION	
Compaction by Blasting	Blasting method
Compaction Piles/Columns	
In-situ Compaction by Vibro-probes	Vibro-compaction method
	Direct power compaction method
	Terraprobe method
	Vibro-rod method
Compaction by Vibroflot	Vibroflotation method
Compaction by vibration and compressed air	Vibro Composer method
Vibro-displacement (water jetting)	Stone column method
Vibro-replacement	Gravel column method
	Slag pile method
	Sand-cement pile method
Compaction at Ground Surface	
Compaction by falling weight	Heavy tamping method
Compaction by vibration	Heavy vibro-rolling method
	Mammoth vibro tamper method
3.PRECOMPRESSION	
Preloading	
Surcharging with fill	Embanking method
Lowering of groundwater table	Well point method
	Deep well method
Applying atmospheric pressure	Atmospheric consolidation method
Accelerated Drainage	
Vertical drains	Conventional sand drain method
	Fabri-packed (sand) drain method
	Flexible band-shaped drains
	Paper drain method
	Cardboard drain method
	Plastic drain method
	Filter drain method
Chemical dehydration	Lime column method
Surface drainage	Drainage pit method
	Drainage trench method
Drainage by electro-osmosis	Electro-osmosis method

TABLE 1. CLASSIFICATION OF GROUND IMPROVEMENT TECHNIQUES (CONTD.)

OBJECT OF IMPROVEMENT & METHOD OF INSTALLATIOI CONSTRUCTION	NAME OF THE METHOD (Note 1)
4. STABILISATION BY ADMIXTURES	
Stabilisation of deep layers	
Admixture mixing/churning	Clay mixing consolidation method
	Deep lime mixing method
	Lime column method
Jet mixing	Dry jet mixing method
	Chemical churning pile method
	jet grout mixing method
Grouting	Cement/admixture grouting methods
Stabilisation of surface layers	
Cement/Lime stabilisation	Soil-cement stabilisation method
	Soil-lime stabilisation method
Sludge/Mud fixing	Mudfix method
	Conseal method
	Minimax method
5. STRENGTHENING OF SURFACE LAYERS	
Surface reinforcement	Fagot method
Sheet/Net laying	Sheet laying method
	Net laying method
	Rope-net fill method
	Bamboo frame method
	Pile net method
Earth filling	Compacted fill method
	Sand mat method
Natural drying and consolidation	
Promoting surface drainage	Drainage pits/pumping method
	Trenches/pumping method
	Deep well method
6. THERMAL STABILISATION	
Sun drying (with drainage) of surface layers	Natural desication method
Burning with gases	Gas burning method
Freezing by liquid nitrogen/carbon dioxide	Freezing method
7. EARTH REINFORCEMENT	
Reinforced earth walls	Terre Armee method
	Free basket method
Slope stabilisation	Soil nailing method
	Reticulated root piles method

Note 1. Same method may be known by different names in different countries. Some names used here are trade names. This table does not provide an exhaustive list of all the current ground improvement methods.

2. 0 PROBLEMS DUE TO POOR GROUND CONDITIONS AND WHY GROUND IMPROVEMENT IS NECESSARY

2.1 Problems in Poor Ground Conditions

Under certain ground conditions, there are situations where the stability of a structure to be constructed can not be achieved due to problems of inadequate strength or excessive deformation of the ground. Problems of poor ground may be related to bearing capacity and settlement in foundations, stability of slopes and embankments, earth pressures acting on retaining walls, liquifaction of loose sands, groundwater seepage etc., to mention some. Shear and deformation characteristics of the ground are of primary importance and the structure need to be designed to accomodate inherent properties of the ground so that the intended functions and performance of the structure will not be adversely affected. But this is not always possible and could often be very costly. Avoiding the use of poor ground altogether may appear to be the ideal solution, but finding alternate land is practically not possible in most cases and the geotechnical engineer is generally compelled to look for other solutions. Problems due to poor ground conditions could be tackled in different ways; by re-designing to adjust scale, load, layout, stiffness etc. of the structure itself or by adopting suitable foundations or otherwise by improving the ground to desired levels.

2.2 Why Ground Improvement is Necessity?

When ground conditions are not favourable for construction activities or for the functional performances of the structure, ground improvement work is undertaken. If intolerable deformation or failure is anticipated under the stresses imposed by construction activities, the foundation ground could be refered to as "poor ground". However, it is difficult to define poor ground only by its engineering properties. Because, a 'poor' ground for a heavy and important structure may at the same time be 'fairly good' ground for supporting lightly loaded structures. Again, sandy ground which may be considered somewhat good ground under static loading becomes poor ground when liquefied under seismic conditions if in a loosely compacted condition. Ground improvement is sometimes undertaken even for so-called good ground, e.g. when it is necesary to carry out construction work in dry condition below the groundwater table. Therefore necessity of ground improvement depends on many factors other than engineering properties of the ground.

It is a general characteristic that poor grounds which require improvement of shear and deformation properties have a loose soil structure with a high void ratio irrespective of whether they consist of cohesive soils or cohesionless soils. Here, the basic principle of improvement is to reduce the porosity of the soil and this could be achieved by replacing it partly or fully with denser material, compacting or consolidating to rearrange the soil skeleton to a denser state, infilling the voids or by reinforcing the soil structure. Again, reduction of the permeability of grounu can be achieved by infilling the voids on a permanent basis by grouting or by freezing pore water in the soil as a temporary measure. On the contrary, improvement of ground may mean increasing of soil permeability as in the case of gravel columns applied in sandy ground to cope with liquefaction or vertical drains employed to expedite consolidation in clays. In some instances, ground improvement may mean treatment for altering certain properties of the soil.

2.3 Situations where Ground Improvement is Required

Ground improvement works become necessary in the follwoing situations;

a) when alterations cannot be made to the scale of the structure or when alternative site with good ground conditions is not available and if ground improvement appears to be the inevitable solution. In this case, sometimes there is a tendancy for the economical aspects to be disregarded and the choice of improvement technique may be restricted.

b) When it is economically advantageous to use the site after ground improvement. In this case there would be flexibility in choosing the most appropriate ground improvement technique.

c) When speedy construction of structures, as in the case of relief measures, is required or when improvement of the ground is necessary as emergency or remedial measures. Only the techniques which show rapid effects are appropriate.

d) When it is necessary to ensure safety and convenience of construction or to maintain stability of structures and ground within the site or in adjacent lands temporarily during construction. In this case, ground improvement techniques which have temporary effect would be adequate.

e) When it is necessary to control distress in existing foundations or earth structures or to strengthen such structures.

2.4. Judgement on the Necessity of Ground Improvement

From the discussion above it follows that the necessity of ground improvement is judged neither by the scale and importance of the structure alone nor by the ground conditions alone. Careful consideration of many factors including, cost, time and construction convenience etc. is also necessary before planning to apply ground improvement works at site. Following procedure may be adopted to arrive at the judgement on the necessity of ground improvement.

a) Consider the importance of the structure to be constructed, type, scale and functions of the structure, allowable or tolerable settlement, safety factor against failure, allowable construction time determined by the scheduled time of completion.

b) Carry out field and laboratory investigations to establish nature, extent and depth of poor soils, stratigrahy of the ground, depth to bearing stratum and the engineering characteristics of each soil stratum.

c) Estimate the stability and settlement.

d) Study the existing structures near by.

e) Examine possibilities of changing the type of structure and type of foundation, eg. from shallow foundation to a deep foundation.

f) Make judgement on the necessity of ground improvement.

2.5 Selection of Appropriate Ground Improvement Technique

In selecting the appropriate ground improvement technique a proper decision may be arrived by considering the following;

a) Purpose of improvement: for example, to increase strength, to accelearte or to control settlement, to improve or stop seepage, to control liquifaction, to change physical or chemical properties of soil, etc.

b) Extent of improvement; the area, depth and total volume of soil to be treated.

c) Suitability of the technique for the soil type/s at site.

d) Adequacy of the technique for the purpose of improvement; sensitivity of design, construction efficiency, realiability and convenience in estabilishing effectiveness.

e) Availability of the necessary materials and space.

f) Availability of plant and equipment and technical background, knowhow and skills for the technique.

g) Cost of execution of the technique.

h) Time necessary to achieve desired effects of ground improvement.

i) Environmental conditions that may restrict working conditions, e.g. noise and pollution, damage to adjoining structures, interruptions to public services, etc.

j) Necessity of temporary works and measures.

Therefore to reach a proper decision, it is necessary to have a thorough knowledge of the mechanism and limitations of ground improvement technique, and the materials and workmanship etc., required for each method.

3.0 TYPES OF GROUND IMPROVEMENT TECHNIQUES AND PRINCIPLES OF IMPROVEMENT

3.1 Classification of Ground Improvement Techniques

Large cities of the past built up in difficult terrains suggest that ground improvement techniques had been practiced by man for a considerable time. It is interesting to note that some of these techniques had been practically adopted quite successfully even before their theoretical background or the mechanism of performance was clearly understood. With the advancement of geotechnical engineering knowledge, new concepts of ground improvement techniques have been introduced and adopted for general use after many years of improvement following research and development supported with theory and experience in practice.

It is also interesting to note that the diverse geotechnical problems associated with poor ground conditions have lead to the development of over 50 different types of ground improvement techniques which are currently available for application in the industry. These techniques vary depending on the principle or mechanism of improvement, purpose of application, materials used, method of installation or construction, type of ground applicable etc.

A broad classification of ground improvement techniques according to the basic principle of improvement is given in Table 1. The list of methods given therein is not exhaustive.

3.2 Outline of Some Ground Improvement Techniques

3.2.1 Replacement of Poor Soils

Replacement of poor ground partially or totally with suitable material is one of the simplest ground improvement methods which has a long history. Poor soils within an economical depth, usually within 3m, may be dredged or excavated for replacement with material rolled and compacted or hydraulically filled. Where lighter loadings on the underlying layers are intended, light weight materials such as expanded polysterene, timber bark etc. are used. In soft ground the weak soil is sometimes replaced by allowing the ground to fail gradually under the fill load. In the Blasting method, detonation of buried explosives can provide a rapid, low cost means of replacing poor soils with better material although this method can not be recommended for use in the built up areas. Loose fine material in poor ground can be replaced with coarser soil particles by allowing to settle in a soil-water suspension formed by the jetting of water.

3.2.2 Deep Compaction

Deposits of loose cohesionless soils may need improvement in order to overcome excessive total and differential settlements in the structures supported over them or to minimise possibility of liquifaction under dynamic loading. Improvement can be achieved suitably by in-situ densification of the loose soil. Deep compaction of cohesionless soils involve dynamic methods where displacement of soil is done by the insertion of a probe or a casing accompanied by in-situ construction of sand or gravel columns. Forced replacement of poor soil can be achieved by the introduction of e.g. 'Composer Piles' or 'Sand Compaction Piles' which are essentially columns of coarser material pressed in to the ground. The ability of these methods to accomplish the required improvement in properties depend on several factors such as; soil type, gradation and fines content, degree of saturation and ground water table, initial relative density and in-situ stresses, intial soil structure, effects of age and cementation etc., and the special characteristics of the method used. Whether it is Blasting, Heavy Tamping or Vibrocompaction, densification of cohesionless soil is caused by breaking down the initial soil structure and allowing the soil particles to move to new packing arrangements. In saturated soils, this is readily achieved by inducing liquifaction with sudden build-up of pore water pressure which greatly reduces the shear strength and eventual failure by successive shear waves. In partially saturated soils, including those containing fines and some waste fills, densification is caused by collapse of the soil structure and escape of gas (and some moisture) from the voids as happens in the laboratory compaction of soils by impact of drop rammer.

3.2.3 Precompression

Precompression and strengthening of weak and compressible soils by preloading prior to construction is one of the oldest and widely used ground improvement methods. Precompression or preconsolidation is well suited for soils which are highly compressible and gain strength under sustained static loads, if sufficient time is available for the required compression to occur. The time anticipated for the required precompression, when excessive and likely to affect construction programme, may be shortened by improving drainage for faster dissipation of built-up pore pressures in the soil by using vertical drains or well points. Forced lowering of the ground water table using well points serves both the functions of loading and drainage. Earthfill, ballast or stocks of construction material etc. may be used as the surcharge load for preloading. Precompression may also be achieved by applying atmospheric pressure as the surcharge load, by electro-osmosis or by absorption of pore water using lime columns installed in the ground.

3.2.4 Soil Reinforcement

Bamboo frames, fagots, timber stakes etc. have been used since olden days to reinforce soil. Today a large variety of materials including metal products, petroleum products and agro products are widely used for this purpose. Geotextiles used for strengthening surface layers or for reinforcing earth walls, the steel bars used in soil nailing, timber piles and metal mesh used for reinforcing weak ground, polymer yarn in Tex Sol, and the root piles used for stabilization of slopes are some examples of soil reinforcement.

3.3 Brief Introduction to Some methods of Ground Improvement

3.3.1 Blasting

Densification of deep layers by blasting can be achieved as a rapid and economical method of ground improvement suitable for deep layers. In this procedure, a charge of dynamite, TNT or ammonite is placed at the required depth using a pipe driven into the ground by jetting, vibration or by boring and the hole is backfilled. The charge is then detonated according to a pre-established pattern. Charges, about 1kg to 12kg in size, placed at spacings of 1m to 5m in plan and at 1/4-3/4 depth of the layer to be treated appear to be used in common practice (Mitchel and Katti, 1981). Successive coverages which usually vary from 2-3 in number and each contsisting of individual chrages are seperated by hours or days. Total explosive usage varies from 8-150 gm/m^3 with 10-30gm/m^3.

Saturated clean sands are well suited for densification by blasting. Presence of even small amounts of gas can lead to damping of the shock waves. Hydro-blasting technique, in which water is pumped to flood the ground until water content is ideally increased above the liquid limit has been used in successful compaction of collapsible loess deposits.

3.3.2 Replacement with Light Materials

When road embankments are constructed across deposits of soft clay and peat, stability problems due to both bearing capacity and settlement may be solved by using light material as the fill. Traditionally, wastes from timber industry like sawdust and timber bark have been used as a light fill material in road construction in countries like Sweden and Norway. Wastes from the production of cellular concrete elements with light expanded clay aggregates, coal slag, spongy lava etc., are also used as light fill materials. The weigth reduction obtained by using such materials is often in the order of half the weight of earth, the ordinary fill material.

Expanded polysterene (EPS) has a great advantage due to its very low density of 20kg/m^3 (compared to 2000kg/m^3 of earth), which for design purposes be taken as 100kg/m^3 allowing for increase of water content over its service time, its strength and durability and easy handling. Top of the EPS blocks is protected with a 100mm -150mm thick cover of reinforced concrete to prevent the EPS being damaged by petrol or chemicals when used for road works or in similar environment. According to Frydenlund & Aaboe (1988) in Norway alone over 100 road projects has involved use of EPS amounting to over 200,000m^3 of polystrene fill. EPS now widely used in many other developed countries has its applications also in landslide control. Moimer et al (1985) reports how a landslide affected road was reopened for traffic within six weeks after full repairs using EPS economically for stabilising the slope.

3.3.3 Heavy Tamping

Soil compaction by 'Heavy Tamping' involves repeated dropping of heavy weights on to the ground surface. This technique as developed in the present form pioneered by Menard (1974, 1975) for improvement of large areas to depths of upto about 30m is also termed **Dynamic Compaction, Dynamic Consolidation** or **Pounding.** The pounders or the weights used for heavy tamping may be concrete blocks, steel plates or thick steel shells filled with sand or concrete and may range from a few tons up to 200 tons in weight. The size of pounders which are usually circular or square in shape may be as large as upto a few meters depending on weight required, material to be improved and the dynamic bearing capacity of the ground surface. In some instances drop heights up to 40m have been used.

In partially saturated soils, the densification process is the same as that for laboratory Proctor compaction. In saturated cohesionless soils in which liquifaction can be induced, the densification process is similar to that in blasting or vibro-compaction. Effectiveness of this technique when applied to saturated fine grained soils is uncertain. Waste and rubble fills have been very effectively improved by heavy tamping. It has been possible to bring down the seven day total settlement in a sanitary fill from 292mm to 14mm (Welsh,1983). Succesful application of dynamic consolidation method for improvement of peaty layers in different parts of the world are reported by Narumi (1983), Marine Eng. Co. (1985) and Ramaswamy (1979) among others.

Typical treatment by heavy tamping may involve 2-3 blows/m^2 on the average with repetitive dropping of pounder at points spaced at several meters on a grid pattern. Two to three coverages, seperated by time intervals depending on the rate of dissipation of excess pore pressure, is usually required and the ground is levelled between coverages. The depth of influence of tamping appears to be in the order of 1/2-1 times the square root of the product of the weight of pounder and the drop height, the units being meters and tons.

When heavy tamping is used for improving ground for support of relatively light weight, low-rise structures on shallow foundations, treatment is sometimes made only at the foundation locations as an economical and effective measure to minimise total and differential settlements.

Dynamic consolidation method which requires simple equipment appears to be appropriate for the local conditions in the developing of large extents of reclaimed marshy land.

3.3.4 Vibro Tamper Method

This is a method of compacting top soil with a combination of high-powered vibrator and tamper. Its effect of improvement reaches deeper in the ground while the top soil treatment alone can influence soils to a depth of 3m or more. Use of a crawler crane makes work more easy and economical with mammoth vibro tamper which has a large contact area.

3.3.5 Sand Compaction Piles

This method is originally developed for compacting loose sandy ground, but recently applied to clayey ground as well. It is impossible to compact saturated clayey soils by this method. However, the artificial composite ground composed of compacted sand piles and clayey ground exhibits improved characteristics. When the load is applied on the composite ground, stress concentrates on sand piles to some degree due to the difference of stiffness. Shear ressistance is offered by cohesion of the clay part and friction of the sand piles.

Construction of sand compaction piles is done either by vibration or by hammering to densify the sand introduced through a casing as in the vibro-compaction method or vibro compozer methods.

a) Vibrocompaction

Deep compaction of loose cohesionless soil is characterised by the insertion of a cylindrical or torpido shaped probe into the ground followed by compaction by vibration during withdrawal. A granular backfill is added in the process so that a compacted sand or gravel column is left within the volume of sand compacted by vibration. Water jet at the tip with or without injection of air facilitates penetration of the probe to deeper levels. Compaction piles of sand or gravel can be formed in ground consisting of soft cohesive soils, in which case they function as compression and shear reinforcement.Vibration probes used in vibrocompaction methods may be open tubes, which enables densifying the top 3 m to 4m of ground with marginal effectiveness, or vibro rods with protrusions which are driven deeper using vibratory pile driving hammer.

b) Vibro-Compozer Method

In this method of vibrocompaction, a casing pipe is driven to the required depth by a vibrator at the top and sand is introduced in to the pipe. The pipe is then partly withdrawn while blowing compressed air to hold the sand in place and the pipe is vibrated to compact the sand pile and enlarge its diameter. Process is repeated until the pipe reaches the ground surface (Suematsu et al 1984). Vibro compozer piles have a standard diameter of 700mm, but "Mammoth" compozer piles of 1200mm -2000mm diameter have been used in marine works.

3.3.6 Stone Columns and Gravel Columns

Stone columns or gravel columns are installed in poor ground often using water jetting in the vibro-replacement process. A cylindrical vertical hole is made by a vibrating probe which is penetrated by jetting under its own weight. In some cases a dry process, known as vibro-displacement process, is adopted without using water jets (Barksdale & Bachus 1983). A stone or gravel backfill is dumped in to the hole in increments of 0.4m to 0.8m and compacted by the probe which simultaneously displaces the material radially into the soft ground. The diameter of columns are generally 0.6m to 1.0m but may be larger if the ground is very soft. Stone/Gravel column systems in soft, cmpressible soils are somewhat like pile foundations, but are more compressible. Ranjan (1988) describes the installation of stone columns using simple equipment generally used for augering and compaction.

3.3.7 Direct Power Compaction Method

In this method loose sand layers are densified by compacting the soil in situ using "press panels" which are flap like compaction plates fixed at the lower end of a steel H-section probe. With alternate upward and downward motion and vibration, soil around the flaps are compressed.

3.3.8 Vibroflotation

Vibroflatation method is used for improvement of loose sandy ground and had been developed in Germany half a century ago. Densification of the soil is achieved by vibrative impact of a probe known as Vibroflot advanced in to the ground by water jetting and made to vibrate horizontally. The cavity formed around the vibroflot is continuously filled with coarse sand or gravel during this process for effective vibration and compaction.

The vibroflot is a hollow steel tube containig an eccentric weight mounted on a vertical axis in the lower part so as to give the horizontal vibration. Typical vibroflots have a diameter in the range of 350mm to 450mm and the length is about 5m inclusive of the flexible coupling. It weighs about 2 kN and develops centrifugal forces up to 16 kN and variable vibration amplitudes up to 25mm at usual operating frequencies in the range of 30Hz to 50Hz. Sinking of vibroflots into the ground is facilitated with water pressures up to 0.8MPa and flow rates upto 3,000 l/min. Sinking rates of 1 to 2m/min and withdrawal/compaction rates of about 0.3 m/min are typical. The zone of improvement varies from about 1.5m to 4.0m from the vibrator depending on the soil type and power of the vibroflot.

3.3.9 Precompression by Preloading

Preconsolidation and strengthening of weak and compressible soils by preloading is one of the oldest and widely used ground improvement techniques. This method is well suited for saturated soft clays, compressible silts, organic clays and peats. Excessively large and prolonged settlement in these types of soils due to the loads imposed by construction activities beocmes a serious problem. Therefore it is desirable to have the ground preconsolidated prior to construction of permanent structures so that they will undergo little or no settlement. In preloading method, surcharge loads, i.e. loads in excess of the loads that will be imposed by permanent fill and structures, are applied on the ground before the construction of permanent structures in order to accelerate the consolidation process.

As the surcharge, earthfills are commonly used, but any other system that leads to drainage of pore water and precompression of ground may be suitable. Water in tanks or lined ponds, stocks of construction materials has been used as surcharge. In the atmosperic preloading or vaccum preloading, surcharge loads up to 60Pa to 80kPa can be produced by pumping from beneath an impervious membrane installed over the ground surface. By lowering the groundwater table using well points and pumping, consolidation pressure can be introduced as a result of the increase in unit weight of soil. Electro-osmosis has also been used to induce the driving force necessary for drainage internally by an electric field. Special devices with anchors and jack systems also can be used for preloading of small areas. Applicability of preloading as an appropriate ground improvement technique in the development of low lying areas in and around Colombo is discussed by Ameratunge and Senanayake (1992).

3.3.10 Vertical Drains for Precompression

Often, the time required for surcharging to achieve the desired effect is excessive or the surcharge required for the time available is too large. In such cases, the consolidation process can be further accelerated by providing vertical drains. Placing the drains at closer intervals to shorten the drainage path is desirable since the consolidation time is inversely proportional to the square of the length of drainage path within the soil mass. An advantage of verticle drains is that usually the horizontal permeability of most deposits is several times greater than in the vertical direction.

Conventionally, sand piles typically 200mm to 500mm in diameter were used as vertical drains spaced at 1.5m to 6.0m intervals. But the development of band shaped paper or plastic drains have facilitated efficient installation of more effective vertical drains. Pearmeability of band shaped drains varies, depending on the material used and the structure of the drain, from about 6×10^{-7} cm/sec for paper to 4×10^{-1}cm/sec for synthetic material. With compared to synthetic types, paper drains show poor performance at high lateral pressures (Miura et al 1984).

According to W. Kjellman at Swedish Geotechnical Institute, who invented the prefabricated drain using wood fibre as drain material, the drainage effect of a drain depends to a greater extent upon the circumference of its cross section but very little upon its cross sectional area (Hansbo,1977).

Failure in drainage due to discontinuities or deformations in conventional sand drains have been overcome with the introduction of fabri-packed drains. These are small diameter sand columns which can be machine-installed in flexible and permeable fabric bags at closer intervals, usually four at a time in one machine operation. Small diameter sand drains placed at closer intervals are much effective than large diameter drains which uses the same amount of sand. From Barron's consolidation theory, it has been shown that the volume of sand required for 400mm diameter conventional drains installed at 1.6m intervals is 6.2 times the volume required for 120mm diameter packed drains placed at 1.2m intervals to achieve the same consolidation effect (Taiyo Kiso, 1976).

Vertical drains may not be of any additional advantage in peaty ground where settlement behaviour is dominated by secondary compression.

3.3.11 Lime Columns

By the installation of densely packed or mixed-in-situ quicklime columns, soft clays can be preconsolidated. In the hydration process of quicklime, water is absorbed from the pores and the soil is compressed due to the volume expansion in the lime column. The heat of hydration will further reduce the water in the soil and the gradual difusion of lime into the sorrounding soil leads to cementation and ground strengthening. Holm (1983) reports that gypsum can be added as an admixture to improve the shear strength of lime-stabilised clay by about three times.

3.3.12 Surface Strengthening by Surface Drainage

As a temporary measure for improving trafficability in work yards on soggy ground, surface layers are allowed to dry up and consolidate by promoting surface drainage using trenches and pumping if necessary. After sun-drying, the ground may be covered with a layer of sand or gravel to a desired thickness.

3.3.13 Soil Sabilisation with Admixtures

Soil stabilisation using cement or lime as an admixture is a classical ground improvement method adopted in improving subgrade and base course materials in road works (Dunlop, 1980, Mallawarachchi, 1992). Many other inorganic and organic soil stabilisers e.g. gypsum, silicates, acrylamides, lignins, resins, polyurethenes, polyesters, asphalt etc., are now used to improve or control volume stability (swelling and shrinkage), strength and stress-strain properties, permeability and durability.

a) Stabilisation of Surface Layers

Soft ground or sludge containing a high water content is treated with stabilising agents which are conveyed in slurry form under pressure and then mixed into the ground using agitaters fixed on to pontoon type or self-propelled mobile treatment plant. In very soft layers the depth of treatment can be upto 5m depending on the type of treatment plant.

The hardened surface layer provides a stiff raft which can distribute the imposed loads evenly to the underlying strata. Surface solidification technique is widely used to cover or make inactive the hazardous sludge containing high content of organic matter. As the hydration action of cement is retarded or prevented by the presence of very active organic matter, special high polymer admixtures are also added in such situations.

b) Stabilisation of Deep Layers

Admixtures, usually lime or cement are used to stabilise deep layers by in-situ mixing. Stabiliser is fed into ground through a rotary drill equipped with a special auger bit to both advance to desired depth and to mix the soil and admixture thoroughly during withdrawal. In another method, the admixture is introduced into the ground through a high pressure jet which is rotated around vertical axis to churn the soil-stabiliser mixture thereby forming large size grouted piles or columns of stabilised soil. The stabilised soil columns formed by deep mixing, churning or jet grouting methods can be as large as 1200mm in diameter, and with large equipment it is possible to stabilise ground to depths easily up to 30m.

3.3.14 Strengthening of Surface Soil Layers

a) Fagot Method

In Japan, for several centuries, faggots or bundles of twigs and branches had been used as "matress" laid under river embankments. This practice has been used in early this century as a low cost method for ground improvement in road embankments constructed over peaty soils and furthermore, a design methodology has also been developed (Suzuki, 1981).

The modern fagot method (using geotextile sheets) developed for surface treatment of soft ground is based on the fact that the fagot, raft or other rigid mat-like material laid on a soft ground makes it possible to maintain the stability of a structure supported on such material because of dispersion of the superimposed load and thereby transmitting a reduced and uniformely distributed load to the underlying ground.

When the surface layers are soft and weak, construction activities, to start with earthfilling, can be extremely difficult. In such situations, a permeable sheet is streched over the soft ground and the sheet is then covered with horizontally between layers of earth and the ends of the sheet or net are folded back to retain earth at the wall facing (Kodera 1985).

b) Root Piles

Root piles are small diameter (75-250mm) piles, installed by grouting around steel reinforcement placed in bre-bored holes. In reinforcing slopes with root piles they are used in a group - "Reticulated Root Piles". Although individual root pile in the group structure may be called upon to carry tension, compresion and flexural pressures, the interaction with the included soil is complex, and the soil-pile system is expected to behave as a coherent body.

Reticulated root piles are applied in landslide control works by reinforcing in-situ soil to create a soil-pile gravity retaining wall, without any excavations and without introducing any disturbance of precarious equilibrium of the slope (Lizzi, 1977,1978). The design approach to slope stabilisation using reticulated root piles is described by Lizzi (1978). Root piles introduced to the industry over 30 years ago is widely used also in underpinning works.

c) Soil Nailing

Soil nailing is similar to root piles in many respects and are used primarily for improving stability of slopes and excavtions. The method of soil nailing is based on the principle of reinforcing soil by means of tensioning bars (so-called nails). Soil nailing is being used at present to stabilise natural slopes, cuts or excavation walls in granular soils, stiff clays and soft rocks (Gassler, 1988).

Nails, which are usually 20-30mm size steel rods, are placed in pre-drilled holes and anchored into the natural ground by grouting. The open end of the nails are fixed to wall facings. In cut slopes shotcrete on wire mesh is sometimes used for surface protection of the slope. But concrete frames or cribs are sometimes used along with synthetic nets to promote vegetative growth (JHPC,1987).

d) Tex Sol Method

Tex Sol Technique is a mixing of continuous polymer yarn and soil to form a composite material in which the yarn brings its tensile resistance. In the present practice, the soils used to produce Tex Sol are essentially natural sands for the reasons that; mixing of sand and yarn is easier than with cohesive soils, mechanical performance of sand is very strongly improved by the tensile strength of yarn, natural sands are obtained cheap in many areas. Tex Sol technique developed in France, taking the hint from the roots of plants, is widely used in retaining structures for cuts in natural ground, for retaining fills and, in slope protection. Tex Sol being a deformable material can (sandy/gravelly) soil. Today, woven or unwoven geotextiles are commonly used as the sheet which can be laid in one or several layers. Stability of the ground is maintained by the tension in the streched sheet. Sheet prevents lateral flow and failure of weak ground and caving-in or intrusion of fill material into the soft ground. It also enables construction of a fill with even thickness which is very important in controlling differential settlement due to fill load. Ropes streched in both directions over the ground are used for increased strength of the sheet (Nishibayashi, 1982). Frames of bamboo and vinyl pipes have been used together with synthetic nets in reclamation works in Japan over soft marine clay deposits with more than 40% colloid content (Nishibayashi, 1988).

b) Pile Net Method

In this method timber or precast concrete piles are driven into the weak ground at desired intervals and the pile heads are interconnected with steel bars, which provide a base to stretch a net. It is intended that the loads from the fill are distributed to deeper levels through the pile minimising the direct effect on weak upper soil layers. It is also said to be effective against lateral flow and slip failure of the soft ground and in reducing ground settlement. Application of Pile Net method to a site with 3m of peat underlain by a 4m thick deposit of silt is reported by Kodera (1985).

3.3.15 Slope Stabilisation

Particularly in mountaneous terrain, situations arise where construction involves unstable slopes and stabilisation of natural slopes or the man made slopes becomes necessary. A few methods used for improving the stability of slopes using reinforcing materials are mentioned below.

a) Reinforced Earth Walls.

Reinforced earth is primarily a composite material formed by the association of soil and reinforcement, the latter being, in most cases, made up of metal strips or bars placed nearly horizontally and which can withstand large tensile forces. The essential feature of reinforced earth is the friction between the earth and the reinforcement. Tension built up in the earth mass is transfered from soil to the reinforcemment by means of friction and tension develops in the reinforcement. Soil mass now behaves as though there is cohesion in the direction of reinforcement. It is therefore necessary that non-plastic material which has good internal friction be used in reinforced structures.

There are different methods of earth reinforcing in walls. In the Terre Armee method, precast concrete panels or semi-elliptical metal elements which make the wall facing are held in position by the tension of steel bars or strips placed in the earth in the wall or embankment (Schlosser, 1976). In another method, e.g. in Free Basket method, geotextiles or metal nets are laid absorb considerable shock energy through large deformations (Laflaive, 1988).

3.3.16 Thermal Treatment of Ground

Both heating and freezing can be used for soil improvement. Fine grained soils when dried, gains strength but looses on subsequent rewetting. They can be permanently improved by treating at temperatures above 600^{o}C to decrease moisture sensitivity, compressibility, swelling and to increase strength. Although heat treatment has been found to be more economical for stabilising collapsilble loess in the USSR, its applicability in countries like Sri Lanka is doubtful.

Artificial ground freezing is used under emergency conditions for temporary ground supports and controlling groundwater in excavations and tunneling in soft ground. Liquid nitrogen and carbon dioxide are used in the refrigerant systems which can produce quick results. However, high cost and difficulties in operation make its application restricted.

3.3.17 Porus Column for Liquifaction Control

In seismic regions, loose fine grained soil deposits pose major foundation problem due to liquifaction. Niigata earthquake in Japan in 1964 was a classic example where many buildings and strucures either tilted or collapsed when high pore pressures were suddenly built up in the silty/sandy deposits bringing the ground to a liquified state. Spontaneous development of such high pore pressures can be prevented by providing for faster dissipation of porewater pressure through effective vertical drains. Precast concrete porus pipes, perforated vinyl pipes or gravel columns are generally employed as the vertical drains.

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MECHANICAL SOIL STABILISATION

DR. T. Sivapatham

Irrigation Department

1.0 INTRODUCTION

Almost every building, dam, road, airport etc. must rest upon soil. Many of these structures also employ soil in their construction. The ideal situation is to find a soil at a particular site which is satisfactory for a particular use as it occurs in-situ. But unfortunately this situation rarely exists. The different ways of dealing with unsatisfactory soils are,

(i) to choose another site
(ii) to remove bad soil and replace desirable soil,
(iii) to redesign the strûcture for the conditions at hand.
(iv) to treat the soil to improve properties.

The method mentioned in (iv) above is soil stabilisation. In the broadest sense, soil stabilisation is the alteration of any property of a soil to improve its engineering performance. Examples of soil improvement are : increased strength (as for a pavement subgrade), reduced compressibility (as for the foundation of a structure) and reduced permeability (as for the foundation of a dam).

Soil stabilisation or soil improvement techniques can be classified in various ways: according to the nature of the process involved, the material added, the desired result, etc. On the basis of process we have mechanical stabilisation, chemical stabilisation, thermal stabilisation and electrical stabilisation. Mechanical stabilisation covers two methods of changing soil particles. They are the rearrangement of soil particles and the addition or removal of soil particles. There are several examples of particle rearrangements ie., the blending of the layers of a stratified soil, the remoulding of an undisturbed soil and of most importance, the densification of a soil. Densification of a soil can be achieved by compaction (with mechanical equipment), preloading (by placing a temporary load) and dewatering (removal of pore water and/or reduction of pore pressure).

In this paper, it is intended to cover the various aspects of densification of soil by the method of compaction and the addition or removal of soil particles.

2.0 BASIC PRINCIPLES OF COMPACTION

The single most important task in any earthwork is to ensure good compaction. It is a common fallacy that heavy machinery will effect compaction according to its size and weight, whereas in fact the relevant factor is the bearing pressure exerted at the soil surface. The first principle of compaction is, therefore, to ensure a substantial contact pressure with the soil.

Secondly, it is realised that, a common aid to the compaction of any dry soil is water. Too much water is undesirable, as the soil becomes sticky, or even worse. Too little water is equally undesirable. A certain level of moistness in each soil gives the easier and best compaction. Thus the second principle of compaction is that the moisture content at which the soil is compacted determines the effectiveness of the contact pressure applied. This moisture content is known as the optimum moisture content and for given compactive effort is an unique property of each soil. The relation between moisture content and the dry density of earthen material is shown in Fig. 1. Also shown in this figure is a line known as the zero air voids line, which represents the densities which might theoretically be achieved if no air voids remained in the compacted mass.

It is evident from Fig. 1 that to the right of the optimum moisture content compacted density falls off due to an inability to expel excess water (even the air voids, although small, are not further diminished). In a confined situation, this loss of compacted density, wet-of-optimum, may be attributed to pore pressure in the water phase which cannot dissipate. If only little confining force exists, the material flows away from the compaction zone due to loss of shear strength in the soil.

It is also evident from Fig. 1 that to the left of the optimum moisture content, the compacted density falls off in such a way that air voids increase rapidly. This loss of density may be attributed to either or both the factors of increasing shear resistance to the compacting force, or increasing resistance to grain fracture.

Fig. 2 shows qualitatively the relationship between increased compactive effort and increased density for soils at the same initial density but at different water contents. Curves 1 & 2 in Fig. 2(a), represent water content -

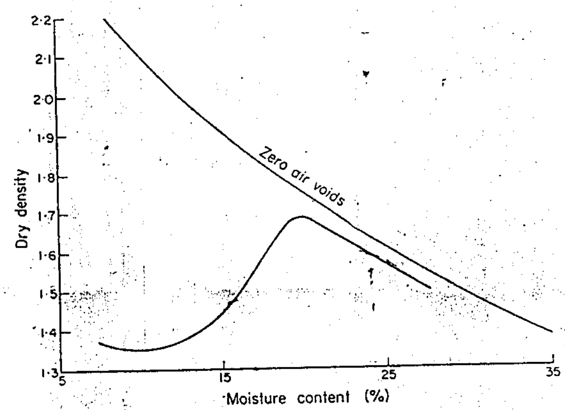

Fig. 1 - Typical relationship between dry density and moisture content for one compactive effort.

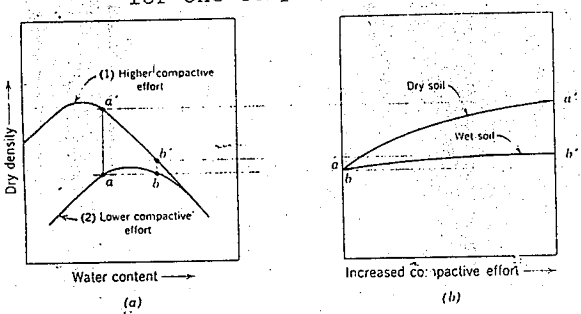

Fig. 2 - Influence of increased compactive effort at the same density and different moisture content.

density curves for the same soil compacted with the same equipment but using two compactive efforts. Points 'a' and 'b', represent two conditions of the soil compacted with the lower compactive effort to the same dry density but at two different water contents (below and above optimum). As the compaction is increased to the higher compactive effort (curve 2), the dry density of the material at 'a' increases more rapidly than at 'b', Fig. 2(b). This difference in the influence of the increased compactive effort on density is due to the fact that the wetter material is soft and the shear stresses imposed on the soil during the compaction process are greater than the shear strength. Hence the compaction energy is dissipated largely in shearing the compacted material without much additional densification. On the dry side of optimum the material is stiffer and more of the compaction energy goes into compressing the soil to a denser state.

In setting requirement for compaction based on moisture content and density, it is always important to keep in mind the purpose of the work. For dams which require low permeability should be constructed slightly wet-of-optimum (with due consideration given to construction pore pressures) whereas stabilised pavements, especially lime stabilised clays, are best constructed slightly dry-of-optimum. It may be mentioned that the moisture content and density referred to above are obtained from a standard laboratory compaction test where the compactive effort is fixed.

The effects of particle crushing due to rolling may be included as part of a more general third principle of compaction. That is to say that highest densities are achieved by mixtures of different particle sizes. This principle was established by Fuller and Thompson (1907) who demonstrated the existence of a series of preferred gradings if the highest densities were to be obtained in mechanical mixtures of gravels. The "Fuller" curves, shown in Fig. 3, express the size gradings for materials with various top sizes which will pack, under a given compactive effort to the highest densities.

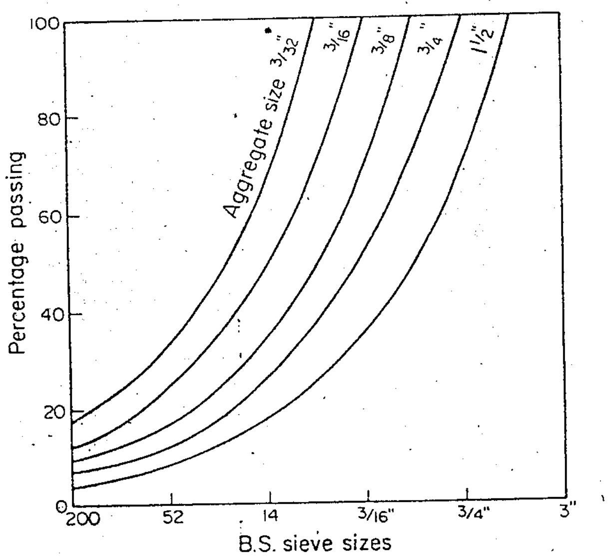

Fig. 3 - Fuller Curves.

All compaction which relies for its density on good grading of the compacted material is essentially achieved by filling the voids between the larger particles with particles of progressively smaller size until even the finest voids have been filled. It may be noted that over working and reuse of soil can create excess fines, drop the shear strength and sometimes destroy natural cohesion such that loss of density occurs by "over-rolling".

The existence of a constraining layer around and below the soil being compacted is a further requirement if any compaction process is to be effective. Hence it could be said that the fourth principle of compaction is that the shear stresses in the soil must be confined if compaction is to be achieved. The confining mass may be the soil itself provided the applied compressive forces dissipate within an acceptable distance from the point of application. But on the other hand if a layer of saturated soil or other soil of low bearing capacity underlies the layer to be compacted then no real compaction can be achieved. The reason being the soft layer will offer no reaction to the load and will deform so as to nullify much of the applied compactive effort. In such circumstances "stage" compaction is adopted by first rolling with light rollers to develop its own strength and then by heavier rollers to complete the compaction.

In addition to the four major principles mentioned above there are other lesser factors such as particle shape, chemical status of the soil water, temperature and mode of application of the compaction force which have an influence on soil compaction.

Natural particles vary both in form and in surface roughness. Particles which are flat and flaky normally pack more densely than those with rounder from. The flat sheets of clay particles and micas possess more surface repulsive forces because of their small size and thus resist compaction. Particles with smoother surface will pack more easily than rough and angular particles due to lower inter-particle friction but are easily displaceable by shear forces.

The compaction aid used, water or any other fluid, also affects the response to a given compactive effort. One such important effect is that of various ionic additives in the soil water. This is due to the reaction of the ions with the primary clay particles to cause greater or lesser shear strength. This is shown in Fig. 4 for clayey soils. Increased temperature yields slightly better compacted densities for the same effort.

The mode of application of the compacting force has influence in two ways. Firstly, it is unidirectional and it will usually produce a state with preferred orientation of the voids. Secondly, a static force is less effective than a dynamic force of the same magnitude since the internal shearing caused under repeated load causes densification faster than that would be obtained by steady creep under a fixed load.

3.0 LABORATORY COMPACTION

3.1 Type of Tests

Laboratory tests are performed by compacting the soil in layers in cylindrical steel moulds. Three general types of tests have been developed :

(i) Dynamic tests in which the soil is compacted in layers by the free fall of a hammer

(ii) Kneading tests in which the soil is compacted in layers with a kneading piston or tamper

(iii) Static tests in which the soil is compacted in layers by static pressure of a piston with area equal to the area of the compaction mould.

Dynamic tests are used almost exclusively for earth embankment. Kneading compaction tests have been performed occasionally, either for research or for the preparation of samples for strength or permeability tests. Static compaction tests have not been employed to any significant degree.

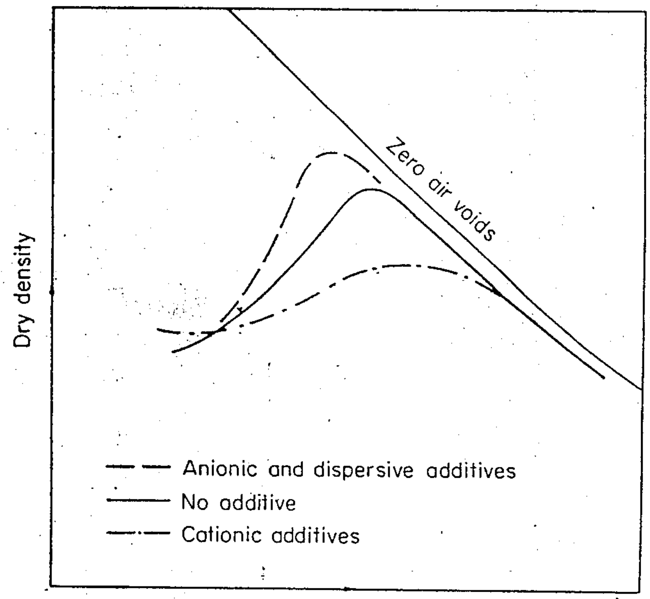

Fig. 4 - Effect of additives on compaction characteristics.

3.2 Dynamic Tests

The details of laboratory dynamic compaction tests in common use are given in Table 1. The test commonly used most is the standard Proctor. Experience shows that the field compactive effort exerted by rollers commonly used for earth embankments is slightly higher than the laboratory standard Proctor test and much lower than the modified Proctor. For this reason an intermediate laboratory test with compactive effort of 19,500 ft lb/ft^3 is sometimes used for construction control.

3.2 Kneading Tests

It has long been observed that the water content-density curves for actual embankments compacted with rollers have maximum densities at higher degrees of saturation than the curves for the same soil compacted in the laboratory with dynamic tests. To simulate the field procedure more closely, kneading tests are made by compacting the soil through the kneading action of a tampting piston or foot. Such tests provide water content - density curves closer to field results.

However, because the differences are not great and because the kneading tampers are more expensive, kneading compaction tests are not widely used.

4.0 FIELD COMPACTION WITH HEAVY EQUIPMENT

An existing deposit of soil can be rolled with compactors to densify it. The compaction of in place soils is usually limited to the top foot or so of a subgrade prior to placement of fill. Sands can sometimes be densified with rollers for a depth of 3 to 6 ft. Most compaction is done on soil freshly placed in layers.

Table 1 *Details of Laboratory Compaction Tests in Common Use*

NAME OF TEST	MOLD SIZE: *Diameter × Height (in.)*	MOLD SIZE: *Volume (ft.³)*	HAMMER: *Diameter (in.)*	HAMMER: *Weight (lb.)*	HAMMER: *Hammer Drop (in.)*	TEST PROCEDURE: *No. of Layers*	TEST PROCEDURE: *No. of Blows Layer*	TEST PROCEDURE: *Maximum Particle Size*	COMPACTIVE EFFORT (FT.-LB/FT.³)
Standard Proctor (ASTM D-698-58T and AASHO T99-57)[2]	4.000 × 4.584	1/30	2	5.5	12	3	25	No. 4[1]	12,375
Modified Proctor (ASTM D-1557-58T and AASHO T180-57)	4.000 × 4.584	1/30	2	10	18	5	25	No. 4[1]	56,250
USBR	4.281 × 6.000	1/20	2	5.5	18	3	25	No. 4	12,375
State of California Department of Water Resources	4.250 × 6.094	1/20	2	10	18	5	13	No. 4	19,500

[1] For coarse soil, 3/4-in. maximum particle size may be used. (See ASTM and AASHO Test Procedures for "Method C".)

[2] In common usage the tests described by the terms "standard Proctor," "standard ASTM" and "standard AASHO" are the same; however, this is not the same laboratory compaction test proposed by Proctor in his original papers. In Proctor's original test, the soil was compacted with a "firmly struck" blow from a height of 12 in. rather than by the free fall of the hammer. Because of the difficulty in duplicating the results, the original test of Proctor has not been widely used and the name "standard Proctor test" has been generally used for the standard test procedures adopted by the ASTM and AASHO.

Table 2 VARIOUS ROLLER TYPES AND THEIR USES

Roller type	*Usual weight (approx.) kg*	*Usual roll (approx.) dia.*	*Usual roll width (approx.) m*	*Contact load kN*	*Remarks*†
Smooth wheel	8100–12 200	1.5 (rear)	1.9	0.35–0.61 per cm width	Better for granular materials. Will operate successfully above O.M.C. (if not too wet).
Grid	6100 unballasted 13 200 ballasted	1.7	1.6	Very high	Should be operated at as fast a speed as possible. Chiefly used for breaking down oversize stones and forcing them below the surface to give a better surface finish.
Vibratory	4050	1.2	1.8	0.21 per cm width	Better for granular materials than smooth wheel rollers; since only half the weight gives equal compaction. Frequency about 2000 Hz cycles/min.
Sheepsfoot	3050–5050 unballasted 5050–7100 ballasted	1.5	1.7	480–1725 per m² (on the feet)	60–120 ft per drum. Best suited for clays, especially in semi-arid zones. The foot pressure may be too high for saturated high moisture clays and too low for very dry clays.‡
Pneumatic tyre	8100–12 200	—	2.3	345–390 per m² (inflation pressure)	4–11 wheels with 9–22 kN per tyre. Best for cohesionless and low cohesion soils, and for surface finishing.

† For roller outputs, see Table 1.2.
‡ A vibratory sheepsfoot is especially good for dry clays.

Compaction is achieved with many types of heavy rollers which include rubber-tyred rollers, sheepsfoot rollers, grid rollers, vibratory rollers, etc. The roller types and their preferred uses are given in Table 2.

The most important features of roller compaction are as follows:-

(i) Repeated passes of the same roller rapidly lose compaction effect at the same moisture content.

(ii) Optimum field moisture content for a particular roller may differ from the value determined in the laboratory and allowance should be made for this fact. In particular the optimum moisture content for the first roller pass may be slightly higher than for the second pass and so on similarly. Therefore some drying time can be advantageous between roller passes.

(iii) The speed of the roller has little effect on the compaction achieved.

(iv) A sheepsfoot roller requires more passes than other rollers because of its limited bearing areas, hence poor coverage.

(v) Different rollers are better suited to different materials. Thus the kneading sheepsfoot or smooth steel rollers are most suitable for clays, but vibratory smooth or grid rollers for sands and gravels. In particular vibratory rollers are found to be effective on sandy-soils as non-vibratory rollers of twice their weight.

(vi) The heaviest roller compatible with (iv) above will give the best compaction in every case. Heaviest roller which does not cause subgrade failures is always recommended.

(vii) Roller pressure should be greater than the tyre pressure of the earth moving plant.

Table 3 shows the output of various rollers. All the rollers are designed to compact soil in layers at the surface and not to the compaction of deep soils in-situ. It has been shown that the effective depth of compaction produced does not usually exceed 25 cm and hence it is usual to restrict the depth of each new layer such that the thickness does not exceed 15 cm to 25 cm after the passage of the roller. Rollers must preferably operate on soils at or below optimum moisture content since they are liable to bog down in loose or saturated weak soils. Roller or rammer compaction is unsuited for weak saturated soils, highly organic (peaty) soils and some very micaceous soils in addition to those deep soft soils which must be compacted in-situ.

5.0 COMPACTION BY CHEMICAL MEANS

Any addition to the soil which reacts with the clay fraction in such a way as to deflocculate or coagulate the clay fraction of the soil will aid the compaction of a soil dry-of-optimum. The addition of anionic additives such as inorganic salts (Sodium Chloride, Sodium Carbonate, Sodium Phosphate etc.) and organic derivatives of sulphonic acid to the soil water will aid the compaction (see Fig.4). The desirable rate of application varies but generally 1% to 2% of the dry weight of soils for inorganic salts and 0.1% to 0.5% for organic derivatives. These are valuable aids to compaction in dry clay soils.

In sandy soils dispersing agents are not effective since there is no cohesion. Agents, capable of either reducing interparticle friction or the residual cohesion forces of moisture, are effective aids to compaction. Wetting agents ensure good fluids distribution and emulsions reduce the residual cohesion in sand. These methods are effective only where mechanical mixing of soil and additive is possible. Quicklime hydration reaction is used where mixing prior to compaction is not possible. If quicklime and water are allowed to react in a confined environment high pressure (temperature also rises) develops. This fact is used in the compaction of deep soft soils by simple drilling or driving holes with a casing, filling the hole with quicklime (for economy lime-sand mixes can be used), withdrawing the casing and sealing the hole. The heat of hydration increases the fluidity of the soil water leading to a rapid extraction of water from the soil and simultaneous lateral consolidation under the expansive forces generated by the lime hydrate formed. This method is suitable for silts and light loams rather than sands or heavy clays.

Another method where saturated clay layers can be consolidated, is to sandwich quicklime layers (spread between thin sheets of drain cardboard) between clay layers. Due to both temperature and the pressure generated in the reaction of the quicklime the drainage of water is speeded and consolidation is accelerated.

6.0 COMPACTION BY SHOCK

Numerous methods for deep soil compaction based on the application of impact or shock waves have been successfully applied in practice. The principle of ramming can be extended to depth and may be accomplished either by direct power compaction or vibrofloation. Direct power compaction consists of driven hollow pile, the end of which is opened or closed as desired, by remote control so that strata at any selected depth can be compacted by alternately slightly raising and then ramming with the closed head. The mode of operation is shown in Fig. 5. This method is suitable for sandy soils.

Table 3 OUTPUT OF VARIOUS ROLLERS (CUBIC METRE PER HOUR) (after Morris and Cochrane, 1964)*

Rollers and speed (m/s)	Mod. max. dry density %	Cohesive materials (clays): Heavy clay	Silty clay	Sandy clay	Non-cohesive materials (granular): Well-graded sand	Gravel-sand-clay
8100 kg steel wheeled roller (self-propelled) 0.9 m/s	85	210	460	280	380	250
	90	90	105	Nil (90)	90	45
	95	Nil (90)	Nil (90)			
	100					
3800 kg vibratory roller (drawn) 0.7 m/s	85	130		170	230	170
	90	25		85	105	105
	95	Nil (90)		Nil (90)		
	100					
3800 kg vibrating tandem roller (self-propelled) 0.7 m/s	85	75		100 (est.)	115 (est.)	115 (est.)
	90	34		50	60	60
	95	Nil (90)		Nil (90)		
	100					
4600 kg taper foot 172 kN/m² sheepsfoot roller (drawn) 0.9 m/s	85	75	165	85		150
	90	Nil (90)	75	Nil (90)		Nil (75)
	95		Nil (90)			
	100					
5050 kg club foot 794 kN/m² sheepsfoot roller (drawn) 0.9 m/s	85	170	150	115		92
	90	100	75	Nil (75)		Nil (74)
	95	Nil (95)	Nil (90)			
	100					
12 200 kg pneumatic tyred 25 kN/m² roller (drawn) 0.9 m/s	85	Nil (85)		305	130	250
	90			Nil (85)	Nil (95)	75
	95					Nil (95)
	100					
20 300 kg pneumatic tyred 55 kN/m² roller (drawn) 0.9 m/s	85	140		460 (est.)	380 (est.)	460 (est.)
	90	Nil (90)		145	Nil (100)	170
	95			Nil (95)		Nil (100)
	100					
13 400 kg grid roller (drawn) 0.6 m/s	85	230		230	150	150
	90	100		70	30	70
	95	Nil (95)		Nil (95)		Nil (75)
	100					

* Figures converted from British units and are rounded off. Figures in brackets are maximum densities after 32 passes (or 16 passes in the case of the 3800 kg vibrating tandem roller).

Vibrofloation like direct power compaction consists of a driven hollow pile and in this case the initial driving can be assisted by water jets. After the required depth is reached, the pile is withdrawn slowly while gravel or crushed stone is supplied down the shaft and a heavy vibrating action applied to the gravel as it is placed. In both direct power compaction and vibrofloatation the driving of the hollow shaft itself causes a compaction of the soil. This method is suitable for sands and silts.

Another such technique is sand piling which is rather similar to vibrofloatation but for economy sand is used instead of gravel and relies in part on the drainage characteristics of the sand piles to produce settlement after placement of the columns. This can be used for silty and clayey soils.

An alternative method of impact compaction for loose, wet sandy soils is by electric shock. This method consists of an impulse generator, high voltage condenser and electric discharge tip which is inserted in the soil with the aid of a water jet and from which discharges are made at 5 - 20 second intervals. Each discharge lasts only about one-millionth of a second and in saturated sandy soils very high pressures can be generated (upto 10,000 kg/cm^2) by the shock wave. Increases in relative density is from about 30% to about 70%. It is known that soils above about 70% relative density are not subject to liquefaction. Hence all the shock treatments which rely on liquefaction phenomena cannot produce relative densities higher than 70%. The radius of effect varies between one and five metres.

Explosive shock waves has been used on many soils to achieve compaction. Consolidation of sandy soils, especially in dams, has been attempted in this way. Also highway embankments across peaty formations has been consolidated by this method. Actually this has been achieved by "bog blasting" which is not intended to compact the very elastic organic soil but rather displace it from under the surcharged fill allowing the fill to settle into the area from which the organic soil has been displaced.

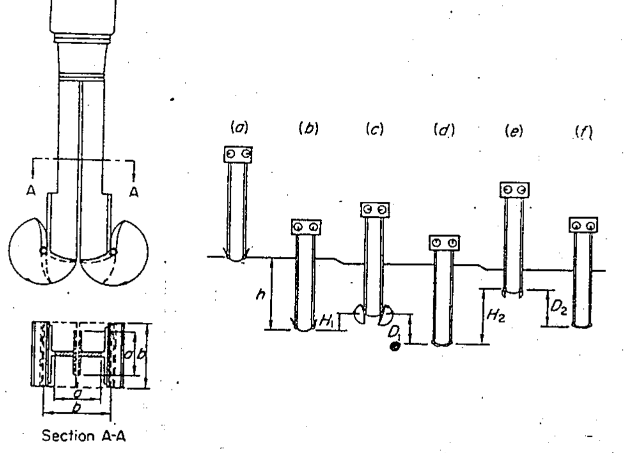

Fig. 5 - Sequence of Operations 'Direct Power Compaction'

7.0 HYDROCOMPACTION

Soils which have accumulated in a loose condition (for example by wind deposition) in arid areas and soils subjected to frost heave with subsequent sublimation of ice can easily self-compact following inundation. Compaction will be aided by surcharge and any other usual means of compaction. The natural collapse following the addition of water to a soil has been termed hydrocompaction. Quick clays of Norway and Canada which become metastable on leaching also belong to this group of materials liable to self compaction.

Soils which have weathered to a mechanically metastable condition (tropical granitic soils) do not settle so readily when water is added due to the existence of mechanical interlock between the soil particles. As this interlock is progressively weakened by weathering, a situation will arise in which a surcharged load which was formerly stable will, when the soil is wetted cause collapse. These types of soils are found in Japan (Masa Soils), North Queensland (granitic soils) and Rhodesia, etc.

8.0 EFFECTS OF COMPACTION ON CLAYEY SOILS

8.1 Effects of compaction on structure

Fig. 6 shows the effects of compaction on clay structure. The orientation patterns shown are simplified. The moisture content at A is so low that the electric repulsive forces between particles are smaller than the attractive forces and the particles therefore tend to flocculate in a disorderly array. As the moisture content is increased, the repulsion between particles increases thereby permitting the particles to disperse and slide into a more orderly array. Beyond B the degree of particle parallelism increases, but the density decreases because water begins to occupy space which could be filled with soil particles.

Increasing the compactive effort at any given moisture content increases the orientation of particles and therefore gives a higher density as illustrated by the point E in Fig. 6.

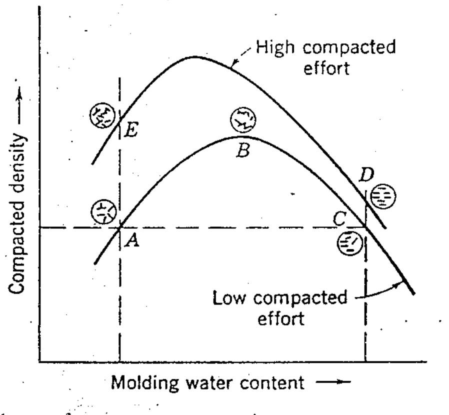

Fig. 6 - Effects of Compaction on Structure

Any clay under a given set of conditions has an equilibrium moisture content. Since the moisture content at A is less than at B, the soil at A is further below the typical equilibrium moisture content and thus has a higher water deficiency than the same soil at B. Whenever the existing moisture content of a clay sample is less than the equilibrium moisture content, the sample tries to absorb water to satisfy this water deficiency. If no water is available pore water tensions develop due to capillary action.

8.2 Effects of compaction on Permeability

Fig. 7 shows the typical marked decrease in permeability with increase in moulding moisture content on the dry side of optimum moisture content. A minimum permeability occurs at or slightly above optimum after which a slightly increase in permeability occurs.

Increasing the compactive effort reduces permeability since it both increases compacted density and the orientation of particles.

8.3 Effects of compaction on compressibility and swellability

Fig. 8 shows the difference in compression characteristics between two saturated samples at the same density, one compacted on the dry side of optimum and one compacted on the wet side. It is seen that at low pressures, the wet side compacted sample is more compressible than the dry-side sample

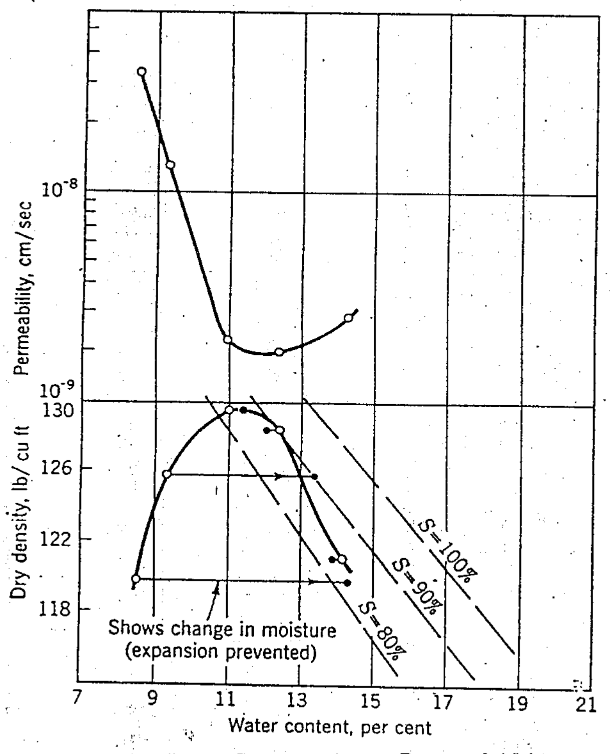

Fig. 7 - Compaction- Permeability Characteristic

since the flocculated particles have more resistance to compression. At high pressures the compression of the dry-side compacted sample is greater than the wet-side sample since the applied pressure becomes great enough to cause particle reorientation. There is also some large pressure at which the densities of the dry-side and wet-side compacted samples are approximately equal.

A sample compacted dry of optimum will swell more (at the same confining pressure) when water is added than a wet-side compacted sample due to its greater moisture deficiency and a lower degree of saturation as well as a more random particle arrangement. Even at equal degrees of saturation, the dry-side compacted specimen tends to swell more than the wet-side sample. Fig. 9 shows the swell characteristics of a sandy clay.

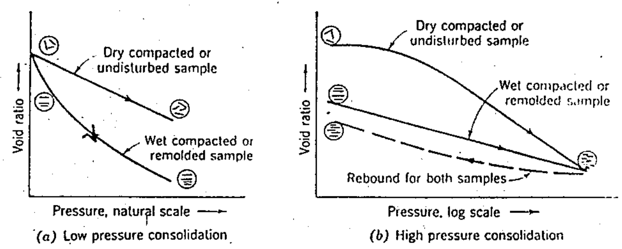

Fig. 8 - Effect of 1-D Compression on Structure

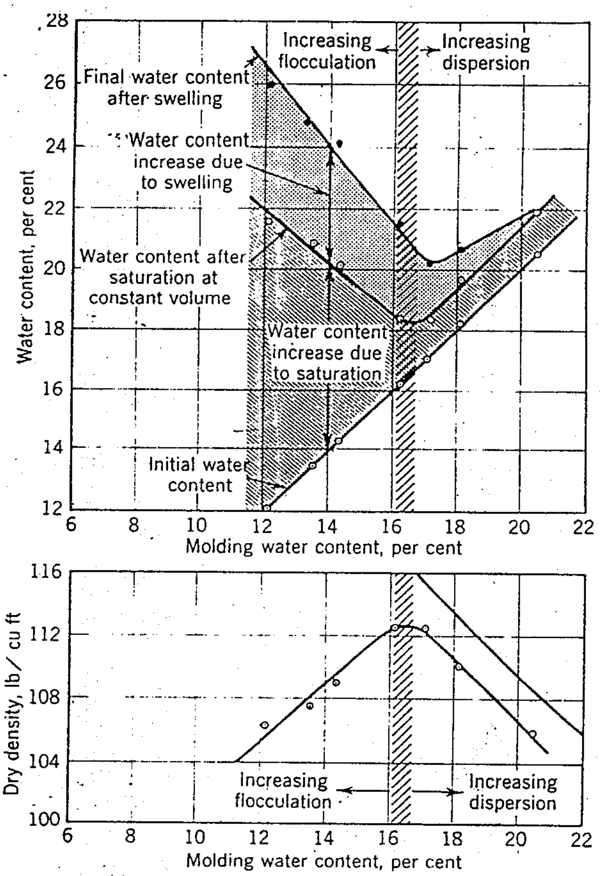

Fig. 9 - Swell characteristics of a sandy clay

A sample compacted wet of optimum will shrink more on drying than a sample at the same density dry of optimum. The more nearly parallel array of particles in the wet-side sample will pack more efficiently as drying occurs.

8.4 Effects of compaction on strength and stress-strain characteristics

The following facts are typical of undrained as-moulded strength properties of a compacted clay:-

(i) Increased compactive effort dry of optimum results in an increase of strength.

(ii) Increased compactive effort wet of optimum can result in a small amount of gain or loss in strength.

(iii) For the same compactive effort, and the same compacted density dry side compaction gives a higher strength than does wet-side compaction.

If two samples, one compacted dry of optimum and the other compacted wet of optimum, are held at constant volume and saturated, the sample compacted on the dry side tends to be stronger than that on the wet side. The strength difference, if any, between the two samples after saturation is, however, much smaller than prior to saturation. If free or even partial swelling is permitted during saturation, the differences in strengths between the dry and wet side compacted samples can be considerably reduced. The undrained shear strength may be about the same. The drained shear strength is about the same for the two samples if swelling is not prohibited during saturation.

Samples compacted dry of optimum (ie. a flocculated structure) tend to give steeper stress-strain curves than samples compacted wet of optimum (ie dispersed structure).

9.0 THE ADDITION OR REMOVAL OF SOIL PARTICLES

The engineering behaviour of soil depends on (among other things) the size distribution and the composition of the particles. This aspect has been described in Section 2 with regard to achieving higher compacted density using specified grading of the soil.

Addition of binder to gravel for road construction works is frequently adopted to improve the performance of roads. The addition of fines to road bases and sub-grades should be done with caution since a free-draining, non-frost-susceptible material may be changed into a poor draining and frost-susceptible soil.

The properties of the materials which occur in clay sizes may vary significantly with the composition of the mineral and with the type of exchangeable ion on the mineral. In general, the properties of a given soil can be altered by adding selected fines. The common application of this technique is the addition of bentonite to reduce the permeability of a soil. A cheaper and superior reduction in permeability can be obtained by adding a good locally available soil. For example a natural floor of a reservoir with pervious material can be mixed with good locally available clay and compacted at optimum moisture content. This will make an effective impervious blanket to reduce seepage under the dam.

The presence of large amount of fines in a gravel make it unsatisfactory for use as pavement base courses and for filter courses. If the excess fines are washed out of the gravel then its use as pavement base courses or filter courses is possible depending on the economy.

10.0 CONSTRUCTION CONTROL OF COMPACTION

10.1 Purpose and methods of control

It should be noted that the most direct measure of the most critical soil property is the accurate method of compaction control. For example in earth dam construction permeability may be more critical than strength whereas in road pavements deformation is the most critical quantity But it is not always possible to measure the most critical parameters by simple field tests. Hence, relationship must be established with more easily measured parameters and then use these parameters for field control of compaction.

Basically construction control is exercised first by visual examination of the uniformity of moisture content and the properties of the soil layer before compaction, the thickness of the compacted layer and the action of the roller and heavy hauling equipment on the construction surface.

To supplement the visual evaluation of the quality of the compaction, tests of the density and moisture content are used as the indirect compaction control parameters. Another useful technique is to keep track of and control the rate of roller production in terms of average values of cubic yards of material compacted per roller per unit of time. This figure will often correlate very well with the average compaction obtained.

It is common for field construction specifications to call for the attainment of some fixed percentage of the laboratory maximum dry density values (Standard or Modified Proctor) usually 95% or more and a range of moisture content usually a little less than OMC to a little more than OMC. Hence the density and moisture content of the compacted section are measured by field density tests and are then compared with the results of laboratory compaction tests on the same sample. If the density and/or moisture content of a section of the compacted layer fall outside the specified limits, the section may be rerolled or broken up, reworked to an acceptable moisture content, and then rerolled.

Two types of field density tests are made, one routine tests and the other on portions suspected to be inadequately compacted. The routine tests are performed at regular intervals (usually for each 2,000 to 3,000 cubic yard of material for big jobs). Results of tests on suspect areas are used to decide whether or not to require reworking and additional compaction. The most common density tests are sand replacement method, core cutter method and rubber balloon method. One of the most frequent causes of error in field density tests is that the compacted material adjacent to the excavated hole tend to squeeze into the hole and reduce its volume. Evidence exists that this causes as much as 5% higher density than the actual value.

The standard method of determining the moisture content of the material taken in the field density test is to dry the specimen in an oven at a temperature of 110°C for atleast 24 hours. This involves a delay of 24 hours in obtaining the result which is not satisfactory for compaction control. Hence several rapid moisture content tests have been developed. Most simple and quicker method is by heating the soil in an open pan over a hotplate or gas flame. Speedy Moisture Meter and Neutron Moisture Meter can be used which also give quick moisture content results.

A number of other rapid methods of moisture content determination have been devised. Proctor introduced a penetration "needle" which tested water content by measuring the resistance to penetration of the soil. Another method developed by Hilf is based on the statistical relationship between OMC and maximum density. This procedure allows the determination of the relationship between the moisture content and dry density of the compacted layer and the laboratory optimum conditions without the necessity for measuring the moisture content.

10.2 Control procedures for materials containing gravel

A typical family of compaction curves for one soil deposit having different uniform index properties for the fines and having different percentages of gravel (particles coarser than No. 4 US sieve) is shown in Fig. 10(b). It is seen that the density of the total material increases and the OMC decreases with increasing percentages of gravel sizes upto a maximum of about 60 to 75%. Above this value the density decreases again (curve 1 in Fig. 10(a). Soils containing more gravels than the value necessary to result in maximum density (60 to 75%) do not have sufficient fines to fill the voids and rapidly becomes pervious material with a small increase in the gravel content.

It has been found from experience that for the material with less than about 30% by weight coarser than the No.4 sieve, the compaction of the fine (finer than No. 4 sieve) fraction is not affected by the presence of the gravels. For soils with

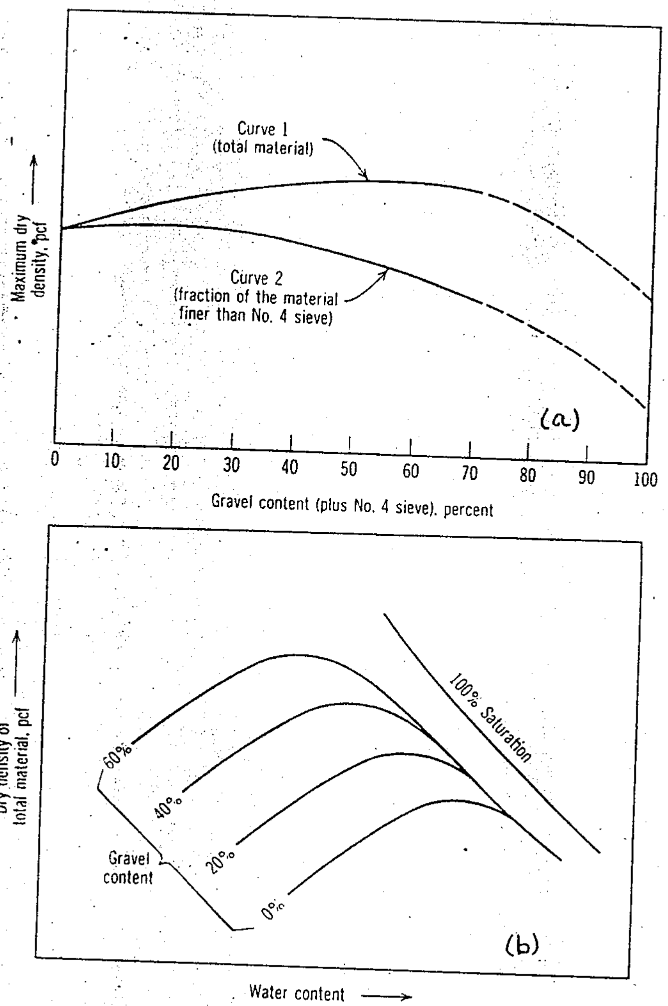

Fig.10 - Relationship between density and gravel content.

gravel content in the range roughly between 30 and 50% the maximum density of the minus No. 4 material is likely to be lower and the OMC higher than they would be if the gravel were not present. This is true for both field compaction with rollers and for laboratory compaction tests on the total material. From laboratory research it has been found that the higher the plasticity of fines, the higher the quantity of coarser material which will be tolerated before the compaction of the fine fraction will be influenced.

For soils containing more than about 50% by weight coarser than the No.4 sieve, the gravel content has a large influence on the moisture content and the density of the minus No. 4 material. For these coarse soils the moisture content of the minus No. 4 fraction necessary to achieve maximum density for the total material is higher than the normal OMC.

For soils with a gravel content upto 25 or 30% the most widely used and satisfactory control method is to compare the density of the minus No. 4 (or 3/4 in) fraction of the compacted layer with the results of the laboratory test performed on the same fraction. This is done by computing the density of the fine fraction of the embankment material by measuring and subtracing the weight and volume of the gravel particles encountered in the field density test. In soils where only a few pieces of gravel are encountered in the field density test, the influence of the existence of the gravels on the computations can be obviated by replacing the gravel particles in the density test hole before pouring in the calibrated sand to measure the volume of the hole. For some soils this method can be used up to a gravel content of 50%.

There is no widely accepted "best" method of compaction control for the coarse soils. The straight forward method is to use a laboratory test with a compaction cylinder which is large enough to allow the compaction of the total material including gravels. Normally it is preferred by many engineers to use fewer large scale compaction tests performed on the total material since the tests are only intended to supplement the primary construction control by visual examination.

Another method of density control which has been used widely is to establish a set of standard curves for the soil in each borrow pit such as shown in Fig. 10(b). After these basic laboratory curves are established, the control procedure consists of measuring the density of the total compacted material (field density test) determining the percentage of gravel and comparing the field and laboratory densities. The basic laboratory curves are obtained by compacting representative specimens of the soil with different gravel content in larger sized compaction moulds.

11.00 CONCLUDING REMARKS

Deep soil compaction by the methods of vibrofloation and sand piling are two of several methods which could be adopted for foundations of structures in marshy areas. A study of costs of different types of foundations including the above two methods for marshy areas for choosing suitable and economical foundation types may be a subject for further study.

Since a greater percentage of the gravel size particles in the residual soils found in Sri Lanka are friable, the applicability or otherwise of the methods described in Sub-section 10.2 for compaction control of these residual soils may need further detail studies.

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SOIL STABILIZATION IN ROAD WORKS

Mr. D. P Mallawaratchie

Research & Development Division,

Road Development Authority

1. INTRODUCTION

In ancient times the use of stabilized soil lime mixtures were known as construction materials in various parts of the world. Nearly 2000 year ago Romans used this material in the construction of roads. In Sri Lanka, many ancient monuments have been constructed by using lime soil mortars along with the addition of other naturally occurring cementing materials.

Soil stabilization may be defined as the alteration of soil properties to meet specific engineering requirements. In road works in most cases, this may mean the improvement of the strength of natural soil by the addition of a small percentage of a stabilizing agent which is called admixture stabilization or blending with another soil which is called mechanical stabilization. In construction the purpose is to produce a material which is strong enough to act as a subbase, base or a surfacing and which retains its strength in the presence of water.

Strictly speaking these stabilization methods fall into the category of soil improvements although, in general terms, they can be considered as ground improvements using soil for the passage of vehicles.

The rapid growth in road traffic during the last four decades resulted in the demand for more bitumen surfaced roads. Engineers found that they can construct very good and cheap roads by stabilizing natural materials with cement, lime, bitumen and other admixtures to form road bases and then surfacing them with a chip seal.

In Sri Lanka road engineers are running out of good quality gravels for subbase materials expecially in urban areas. In the case of bases, suitable crushed or broken rocks for the manufacture of such bases are not available in some areas. It has also been found that stabilized material bases are cheaper than aggregate bases, as in the case of the construction of the approach road to the Cement Factory at Puttalam. In the present context, soil stabilization is becoming more and more popular as there is a dearth of crushed rock. Consequently in some parts of the country, it is becoming the only method of constructing road bases.

As there are two papers, one on mechanical stabilization and the other on geotextiles at this Seminar, it is not intended to go into details regarding these aspects but to give only the instances where these methods have been used in Sri Lanka for road works. Only details of admixture stabilization will be given in this paper.

2. ADMIXTURE STABILIZATION

The various types of admixture stabilization have been categorized according to the properties imparted to the soil[1]. Types of admixtures include cementing agents, for increase in strength by cementing action; modifiers to improve plasticity; water proofing agents, such as bitumen; water retaining agents, such as dust palliatives; water retarding agents such as organic polymers and miscellaneous chemicals such as resins, ligning etc, used for special applications.

The cementing materials which are normally used include portland cement, lime and mixtures of lime and fly ash. In addition, the material which is used worldwide is bitumen for bitumen stabilization.

The applicability of various types of stabilization methods for different types of soils, as shown in Ingles and Metcalf (1972)[2] is given in Figure 1.

2.1 Cement Stabilization

Ordinary portland cement has been used successfully to stabilize natural soils in order to use them as base courses and subbases for all types of roads. Occasionally it has been used as a surfacing for unpaved roads. It increases soil strength principally by hydration and by some modification of clay minerals.

The technique of cement stabilization involves the pulverisation of the soil, mixing the cement with the soil and the required quantity of water and compacting in the usual manner. The details of the various alternative methods carrying out these techniques are given in section 3.

Some of the aspects of cement stabilization also applies to other admixture stabilization methods where the admixtures such as lime, bitumen and other chemicals are mixed with the soil.

The procedure of selecting soils for stabilization with cement (or lime) is usually decided on the results of laboratory CBR tests. The tests are normally conducted on samples of the soil mixed with different amounts of stabilizing agent and compacted to density (normally 100% standard) expected to be attained on the construction job and then cured moist for three days and immersed in water for four days prior to testing. The recommended criteria are; CBR of not less than 30% for subbases and CBR of not less than 80% for bases. 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.

In projects where small stabilizer contents are being contemplated, the lower limits are decided by effectiveness with which the stabilizer can be mixed and therefore it is necessary to stipulate the minimum values for field mixing. The suggested stabilizer percentages in TRRL, UK Road Note 31(1977)[3] are given below.

Gravel	(i) Mix in place *	3%
	(ii) Plant mix and travelling plant *	2%
Sand & light clays	(i) Mix in place *	4%
	(ii) Plant mix and travelling plant *	3%

* These methods of stabilization are described in section 3 of this paper.

Soil cement base courses where high percentage of cement is used behaves as a semi-rigid slab, and normally develops shrinkage cracks. These cracks are not considered detrimental but should be sealed in order to prevent infiltration of water that could cause decrease in durability.

2.2 Lime Stabilization

The next category of cementing agent, which is often used in tropical countries is lime. The primary mechanics of stabilization is pozzolanic action, which is the formation of cementitious silicates and aluminates. The other mechanics of stabilization include alteration of the water film surrounding the clay particles and flocculation of clay particles. As lime soil mixtures are susceptible to freezing and thawing action, their utilization is limited to regions where such climates are not present.

Stabilization of soils with hydrated lime is broadly similar to cement stabilization in that similar criteria of testing and construction techniques are used. It differs, however, in several aspects. Firstly, it is applicable to far heavier clayey soils, and is less suitable for granular soils (see Figure 1). Secondly, the pavement obtained is a flexible pavement when compared to a semi rigid pavement for cement stabilization. Thirdly, the method of obtaining strength as described earlier is different and fourthly, there is no rigid time period between adding water and compacting as for cement stabilization. However, studies with respect to the latter aspect has shown that there is a loss of strength if there is a delay in compaction and a period not less than 3 hours is stipulated by the Indian Roads Congress(IRC).[4]

In the lime stabilization process, the soil should have clay minerals for the stabilization reaction to take place. For lime stabilization TRRL, UK, Road Note 31[3] stipulates that the soil should have at least 15 per cent particles finer than 425 μm and for the plasticity index to be at least 10 per cent. It has been found out that organic matter in excess of 1 to 2 per cent is likely to be deleterious to strength and the IRC recommends a maximum limit of 2 per cent organic for use in lime stabilization.

Both quick lime and hydrated lime are most commonly used due to their convenience in working. Lime prepared by calcining dolomitic limestone from the up country has been successfully used for stabilization.

Unlike cement, which is of a known quality, the quality of lime obtained from suppliers is of wide variety and it is necessary to ensure the requisite quality of lime before work is started. A rapid method of estimating the available lime (calcium oxide) content called the "active lime content" has been worked out by the R&D Division and has been used to determine the quality of lime.

Since recarbonation of burnt lime takes place with time giving a poorer quality lime, it is suggested that freshly burnt lime is used for lime stabilization work especially for large projects.

2.3 Lime fly ash stabilization

The addition of fly ash which is produced by burning coal and which contains silica and aluminia, to lime stabilized soil speeds up the process of pozzollanic action. As the amount of fly ash which has to be added in this work is high, its use is limited to areas where fly ash is produced as waste, such as the thermal power stations in India.[5]

As this method does not have any applications in Sri Lanka it is not described further in this paper and the reader is referred to reference 5.

2.4 Modifiers

In many projects, the use of a cementing material is cost prohibitive and therefore in some of these projects in other countries, low quantities of the material (as little as 2%) may be mixed to the soil merely to modify it. Frequently, in these countries, extremely wet soil conditions are encountered and it is impossible to properly dry the materials. Under these conditions modifiers which could be used include cement, lime and bitumen. Cement and lime will modify the clay mineral and will decrease its plasticity. Small percentages of bituminous materials can be used in soil aggregate mixtures having impurities of clay, where the bituminous material retards the adsorption of moisture to the clay particles.

These materials can be generally adopted for use in the case of borderline base course materials[1].

2.5 Bitumen Stabilization

The next type of stabilization refers to the water proofing materials, such as bituminous materials which coat soil or aggregate particles. Bitumen stabilization adds cohesive strength and, with cohesive materials, it "water proofs" the soil thus reducing loss of strength with increase in moisture content.

This type of stabilization is best suited for granular soils using cutback bitumens and bituminous emulsions, but its use for stabilizing plastic soils is limited. Well graded non plastic angular sands with particles passing the 75 µm sieve not exceeding 10 per cent have given the best results. In the tropical countries cutback with upper limits of viscosity represented by a MC 800 grade can be used. In general, the stability of the mix will be higher with a more viscous cutback. Best results can be obtained if penetration grade bitumens are used. However, this requires heating the binder as well as the sand. Practically this is not possible unless expensive premixing plants are available in the close proximity of the construction work.

The requirements of binder will generally vary from 3 to 6 per cent by weight of the dry sand, the higher percentages being required for fine sand. Addition of 1 to 2 per cent lime to non plastic sands (not more than 3 per cent clay) may promote adhesion between bitumen and sand and improve the strength of bitumen stabilized materials.[6]

TRRL, UK Road Note 31 (1977)[3] has given criteria for determining the amount of bitumen required, based on either the Hubbard Field or the Marshall test. These are given in Table 1 for sand bitumen base materials for tropical roads carrying medium to light traffic.

Table 1
CRITERIA FOR SAND BITUMEN ROAD BASE MATERIALS

	Light traffic	Medium traffic
Hubbard Field stability at 60°C (min)	300 kg	430 kg
Marshall stability at 60°C (min)	100 kg	150 kg
Marshall flow value at 60°C (max)	2.5 mm	2 mm

Note: Light and medium traffic are defined as less than 150 and 300 commercial vehicles per day respectively. They have an average damaging power per vehicle of 0.5 standard 8200 kg(18000 lb) axles or less.

2.6 Membrane Stabilization

Stopping or retarding moisture movements into soil such as for a subbase can also be achieved by enveloping the soil in an asphaltic membrane (see Figure 2).

2.7 Stabilization with calcium chloride and sodium chloride

Some chemicals, when mixed with soil or when applied on the surface of a soil layer, will increase the water absorption from the atmosphere keeping the soil moist. They include calcium and sodium chloride, which have been used as dust palliatives on unpaved road surfaces for more than 50 years. The reaction of soil and the chloride is brought about mainly by changes in the soil water itself, thereby decreasing the rate of evaporation of soil water. Thus they have also been used as a method of assisting construction work to retard evaporation of the soil during compaction[1].

2.8 Stabilization with water retarding agents

Several other chemicals can be used for stabilization. Some of these chemicals will make the soil hydrophobic. They will decrease the rate of water absorption to some extent. Their general use is limited, as they have drawbacks, including their expense[7].

3. CONSTRUCTION TECHNIQUES

3.1 Stabilization method using plant, machinery and equipment

The procedure of construction will mainly depend upon the type of plant and machinery available at site. There are normally four methods in use :-

(1) Mix in place
(2) Travelling plant
(3) Stationery plant at a central yard
(4) Mixing with front end loaders at a central yard

In the case of the mix in place method either a series of different machines is worked over the soil to be processed or a single pass stabilizer is used. In the first method, different machines such as rippers, scarifiers, rotary tillers, stabilizer spreaders, rotary mixers and compacting equipment pass over the insitu soil one after the other in carrying out the different operations such as scarifying, pulverizing the soil, spreading the stabilizer, watering, mixing and compaction. In the single pass stabilizers these operations are done with just one pass.

In the case of the travelling plant method, the pulverized soil and stabilizer are hoisted into a hopper from where they are fed into the mixer along with the required amount of water. The mixture is then discharged onto the road and spread with a motor grader and compacted.

In the case of the stationery plant method, the soil and the stabilizer are mixed with the required quantity of water in a plant at a central yard. This method is normally restricted to soils with low combined silt and clay content as other soils can not be satisfactorily. After mixing, the material is transported to site and spread by hand, a grader or a bituminous paver and compacted.

Mixing with front end loaders at a central yard has been successfully worked out locally in the dry zone even with a gravelly soil having medium plasticity. The method is described in section 5 of this paper.

In Sri Lanka where manual labour is cheap the stabilization work can be done manually for small jobs.

In this section, a general description of the method of stabilization is given. Important aspects are described in the ensuing paragraphs.

3.2 Pulverization of soils

The pulverization of soil can be carried out by various methods, depending upon the type of soil being processed, the type of equipment available and other factors such as climate, weather etc.
On small labour intensive jobs manual labour can be successfully used. The lumps can be pulverized with mammoties, pick axes, shovels or rammers. Passes of an agricultural plough driven by a bullock have also been tried out in other Asian countries and have proved effective sometimes. If plenty of water is available, lumps of clayey soil can also be slowly broken by alternatively sprinkling and allowing the soil to dry. Another method where only steel wheeled rollers are available and the soil is dry is to break the lumps of soil by passing the roller a number of times on them.

The following agricultural machinery can be used to pulverize clayey soils :-

(i) Ploughs
(ii) Disc harrows
(iii) Rotorvators

Specialized machinery such as rotorvators and pulvi-mixers are available for doing the dual job of pulverization and mixing. In the single pass stabilizer the operation of pulverizing is done by the plant itself. The speed of the plant controls the degree of pulverization.

3.3 Spreading the binder

Bags of hydrated lime or cement are placed at equal distances calculated from the percentage of stabilizer to be added. Small piles are then made using the contents of each bag and the lime spread uniformly with showels and rakes giving an even continuous layer over the soil to be stabilized.

3.4 Water for stabilization

Normally there is no precise test for the quality of water required, it being generally regarded that potable water is satisfactory and acidic (orgqnic) water should be avoided.

3.5 Compaction

Various types of equipment can be used for compaction. The choice depends on the type of stabilized soil to be compacted. Initial rolling can usually be done with pneumatic rollers or sheet foot rollers or steel wheeled rollers. For gravelly soils, the main compaction may be done with pneumatic or steel wheeled rollers. Normally the specifications for stabilized soil mixtures require that the compaction should be carried out to 100 per cent of the standard density.

3.6 Final shaping

The surface is finally brought to the required levels and the crossfall as shown in the plans, correcting any irregularities that may have been caused.

3.7 Quality Control Measures

Stabilized construction requires a good degree of quality control if it has to be successful. In the case of cement and lime stabilized subbase and bases, the standard specifications for construction and maintenance of roads and bridges, detail the measures that are needed, test methods and their desirable frequencies[8].

4. APPLICATIONS OF SOIL STABILIZATION IN SRI LANKA

4.1 Lime Stabilization

Since the establishment of the R&D Division in the then Public Works Department, Highways Department and presently the Road Development Authority, vast number of natural soils from various parts of the country have been tested for their reactivity with lime. It has been found that most soils in Sri Lanka, other than coastal beach and river sands and organic soils such as peaty soils and topsoils etc., when mixed in laboratory with 2 to 5 per cent lime by weight give adequate strengths for use as subbase and bases in road pavement.

A typical 4 day soaked CBR curve with increase in lime content is given in Figure 3.

Based on this, several roads have been constructed on an experimental basis using the techniques of lime stabilization for the past 25 years in Sri Lanka.

As the method of soil stabilization was expected to be a promising one, it had been anticipated that lime would have to be produced in large quantities. Hence studies have been carried out pertaining to the availability and the production of lime from coral stone and sea shells and from dolomitic limestone.[9,10]

An example of lime stabilization is documented where a road of length 2 km, close to Negombo was constructed along the sea beach[11]. The insitu sand was first mixed with imported gravelly soil and then this soil was mixed with lime.

In 1981, a surfacing of a unpaved road of length 100 m was constructed on Yaggapitiya-Uyandara Road, close to Kurunegala. On the same road a section of 300 m was surfaced with a chip seal. The soil used for stabilization was a gravelly soil which had 4 day soaked CBR values of 17-23 per cent when compacted to 100% standard density and which when treated with 4 per cent lime achieved CBR values of over 70 per cent. In the preceeding examples, rotorvators were used for pulverization and mixing processes.

A case study of stabilized soil construction on the Palavi-Kalladi Road which is the approach road to the Puttalam Cement Corporation is described in section 5 of this paper.

Recently soil lime was used in the construction of a shoulder surfacing on the Colombo-Horana Road from Kohuwela to Pepiliyana.

4.2 Cement Stabilization

Although cement stabilization tests have been carried out in the laboratory on various soils, it was only recently that a length of more than 1 km of road on the approaches to the Cement Factory at Puttalam was stabilized with cement. The details of this work is described in this paper in section 5.

Recently the drain bottom linings on the Colombo-Horana Road from Kohuwela to Pepiliyana were constructed with insitu gravelly soil stabilized with 3 per cent of lime and 8 per cent of cement. As the performance of this mixture was not satisfactory, the drain bottom linings were constructed with 3 per cent lime and 12 per cent cement stabilized soil and were observed to be performing more satisfactorily.

4.3 Bitumen Stabilization

Base construction by bitumen stabilization has been tried out on a short section of a road at Angulana in 1970. The method tried out involved the mixing of insitu sand with cutback bitumen using a rotorvator. This procedure proved to be a very difficult one in practice and was abandoned. With the commencement of a bituminous emulsion plant at Angulana in 1985, it is thought that suitable procedures could be worked out for bitumen stabilization of beach sands which would have applications in the coastal belt where good aggregates are not available.

In order to work out the details of this method, it is proposed to carry out an experimental stretch during the construction of a new road along the sea beach.

4.4 Other admixture stabilization

Various other chemical admixtures have been tried out in the laboratory with varying results.

Soil on two sections of road, one on Colombo-Galle Road at Katubedde and the other at Torrington Square, Colombo were stabilized with a chemical admixture in 1978, but proved to be failures. Laboratory testing of the stabilized soils on a later date also confirmed the results.

Another stretch of road stabilized in 1985, with 75% cement along with 0.6% chemical admixture has behaved satisfactorily with a few cracks appearing on the surface of this unpaved road of low traffic intensity. However the high cost of this imported chemical admixture made this method about twice as costly as the construction of this road with aggregate or lime stabilized bases.

4.5 Mechanical Stabilization

The proportions of soils for mechanical stabilization have been successfully worked out for many projects for road work in Sri Lanka. However these projects did not get off the ground as cheaper alternative gravels were found for these purposes.

One project which was carried out in this regard in 1975, was the gravel surfacing of the ceremonial pathway at Kataragama Devale where a gravelly soil was blended with another soil to give the required properties.

Some work which was recently done at the new road to Panadura through Egoda-Uyana, include the construction of a subbase by mechanical stabilization of 80% transported soil mixed with 20% insitu soil in order to reduce the plastic properties of transported soil.

5. Case study-Experimental sections of lime and cement stabilization construction on Palavi-Kalladi Road

5.1 Introduction

In 1988 it was decided to convert the improvements to a section of the approach road to Puttalam Cement Factory from 1.5 to 2.5 km, to an experimental project, where various mixes of lime and cement stabilization for base construction and other types of work could be constructed.

In this project several sections of lengths varying from 100 to 200m were constructed with stabilized soil containing 3,4 and 5 per cent lime and 3,5 and 7 per cent cement.

The surfacings of bitumen primed double bituminous surface treatments (chip seals) were constructed on two stabilized sections but in the other sections they were constructed on aggregate bases laid on the stabilized bases. Later the work on cement stabilization base construction was extended to cover the section from 0 to 1.250 km of this road with a few intervening cement concrete sections. In these sections, chip seals were laid directly on the stabilized bases.

The traffic on this road included heavily loaded cement trucks resulting in a design traffic of 2.3 x 10^6 axles for a design life of 15 years. By the time the experimental project was commenced a subbase of gravelly soil having a 4 day soaked CBR of 25 had been constructed on the prepared existing road surface.

5.2 Materials and initial testing

With an objective of stabilizing gravelly soil from Cementigama quarry (situated about 4 - 6 km from the experimental section) for base construction, a series of laboratory tests were carried out to determine the suitable admixture proportions of lime and cement. The test conditions adopted for CBR determination were 4 day soaked CBR for soils without any stabilizer and 7 day cured and 4 day soaked CBR for stabilized soils.

An abstract of laboratory test results are given in Table II.

The lime used was from Digana, Kandy with an active lime content of 40%. The cement used was ordinary portland cement from the Puttalam Factory.

5.3 Construction

5.3.1 Construction machinery, equipment and tools

Construction machinery, equipment and tools available for the stabilization work are given below:

1. Agricultural Plough
2. Disc harrow
3. Motorgrader
4. Front End loader
5. Lime/cement spreader
6. Water distributor bowser
7. Compactor - 5÷20 tonne vibratory roller
8. Compactor - 25 tonne heavy pneumatic tyred roller
9. Tippers
10. Camber boards
11. Rakes, mammoties and showels

5.3.2 Pulverization and mixing of gravel with stabilizer

5.3.2.1 Lime Stabilization

For lime stabilization work, the gravelly soil was transported to site and levelled by the motor grader.

As this work was done in the dry zone during the dry season, the gravelly soil when excavated by dozers and spread by motor graders did not have many lumps. The remaining lumps were pulverized manually and what could not be broken up were hand picked and discarded.

The lime was supplied as a hydrated powder in 50 kg bags. After checking their weight, the bags were transported to site by tractor trailer or by a combination of lorry and manual transport to yield a spread of the required quantity at the laboratory value of dry weight. The bags were then opened and the lime spread uniformly with showels and rakes onto small areas of about 30 m^2 in one operation in order to minimise the lossses due to wind. Lumps of unburnt limestone were hand picked and discarded.

The application of stabilizer to gravel was also carried out by using the spreader. In this case the lime was sieved to remove the unburnt limestone and a capacity of 40 to 50 bags were placed in the spreader and depending on the rate of spread of lime, the speed of the spreader was altered to obtain a uniform but required spread of lime. For higher lime contents it was necessary to spread the required lime in two operations. This method proved to be not very satisfactory with respect to the lime dust prevalent in the atmosphere during the construction work, making it somewhat injurious to the labour working on this project.

Next the lime was mixed with the soil by running the agricultural plough or the disc harrow and then further mixing with the motor grader until a uniform mix, as judged visually, was obtained.

The mixture was then watered to the required amount by means of a water distributor bowser. Next the lime soil mixture was further mixed by repeating the mixing process till such time the water was uniformly distributed in the lime soil mixture. The lime soil mixture was shaped to the correct profile by means of a motor grader and compacted to the required density of 100 per cent standard by initial rolling with two passes of a heavy compactor without vibration and final rolling with the same roller with vibration with 10 passes.

The stabilized soil bases were cured by keeping the surface continuously moist by sprinkling water at frequent intervals.

5.3.2.2 Cement Stabilization

In addition to the methods given above for lime stabilization, central yard mixing using a front end loader was carried out in the case of cement stabilization.

In this process, dry gravel was transported from the quarry to the central mixing yard within a distance of less than 1.5 km from the section of the road to be stabilized. The gravel was then heaped up with a top flat area where the required quantity of cement was spread uniformly. The front end loader worked in a series of forward and backward movements in a way that the gravel was mixed gradually but uniformly with cement. It was found that it takes about 4 hours to mix 100 cubic metres of gravel. After mixing work was completed, the mixed material was transported and spread on the surface of the subbase which had been previously wetted. Then the watering and compaction (as given in section 5.3.2.1) were repeated to form a stabilized base.

5.4 Quality Control Tests

A field laboratory was set up at this site to control the quality of the material and the work.

Using the facilities available in this laboratory, the following tests were carried out at intervals of about 50 m along the stabilized sections of this road :-

1. Field moisture content
2. Field density
3. Standard compaction test.

In addition to these tests, 7 day cured 4 day soaked CBR tests at 100 per cent standard density were carried out on samples taken from the site after field mixing, and these test results are given in Table III.

The results of the tests indicated that, generally, the control of quality of the material and the work has been carried out satisfactorily at this site.

6. CONCLUDING REMARKS

Soil stabilization has been used successfully to construct subbase and bases in other countries for the last four decades. In Sri Lanka the time has come when the methods of stabilization, especially lime and cement, should be exploited to the maximum for reasons given in this paper.

For this purpose, the construction industry should be developed by the production of cheaper stabilizers with respect to cement and better quality stabilizers with respect to lime along with the purchase of pulvertization equipment such as rotorvators etc., working out locally suitable stabilization procedures and training of personnel in such procedures.

The importance of the determination of the suitability of the stabilizer by testing is emphasized. The approach of "Let's try and see what happens", is seldom successful. In selecting a stabilizer, the admixture which produces the laboratory soil property changes required to obtain the desired effects for the least cost, from the stand point of initial cost and future maintenance cost, should be selected.

When carrying out stabilized soil construction, it is important that sound construction practices should be followed. Also it must be remembered that basic principles of road engineering such as provision of drainage at all stages of work should be followed.

ACKNOWLEDGEMENTS

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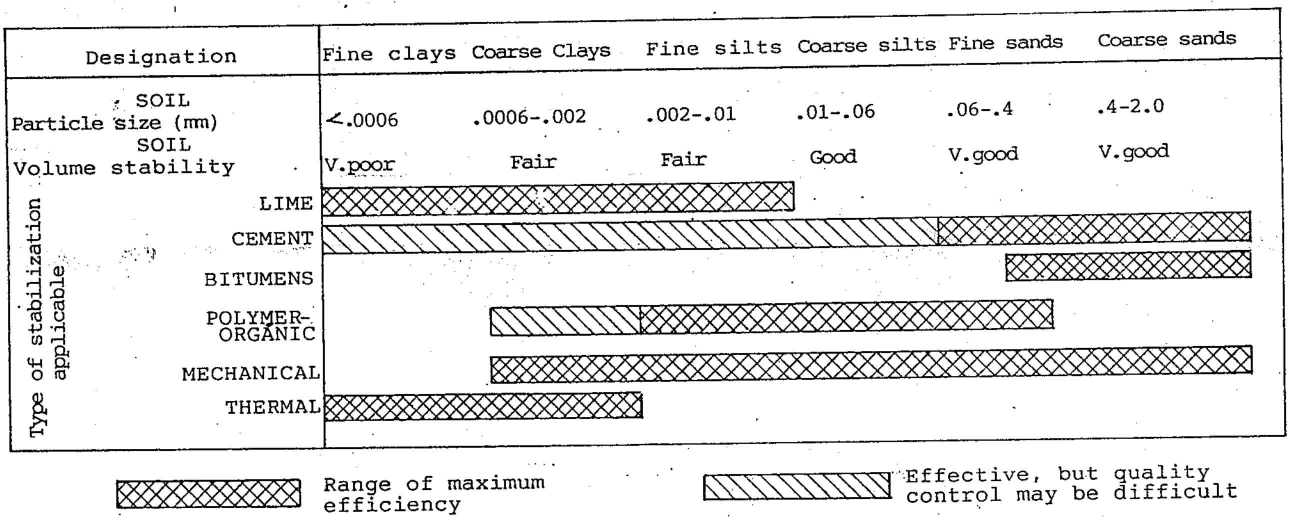
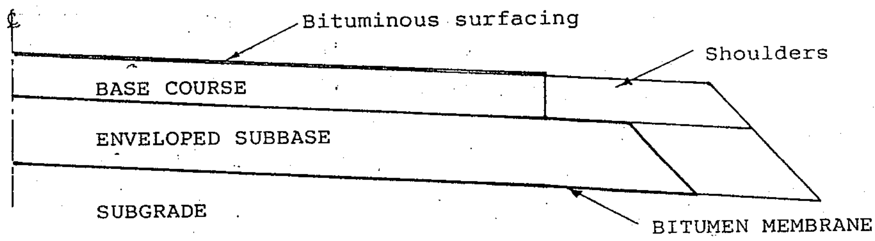


APPLICABILITY OF STABILIZATION METHODS, AFTER INGLES & METCALF (1972)[2]

FIGURE 1



MEMBRANE STABILIZATION OF A SUBBASE

FIGURE 2

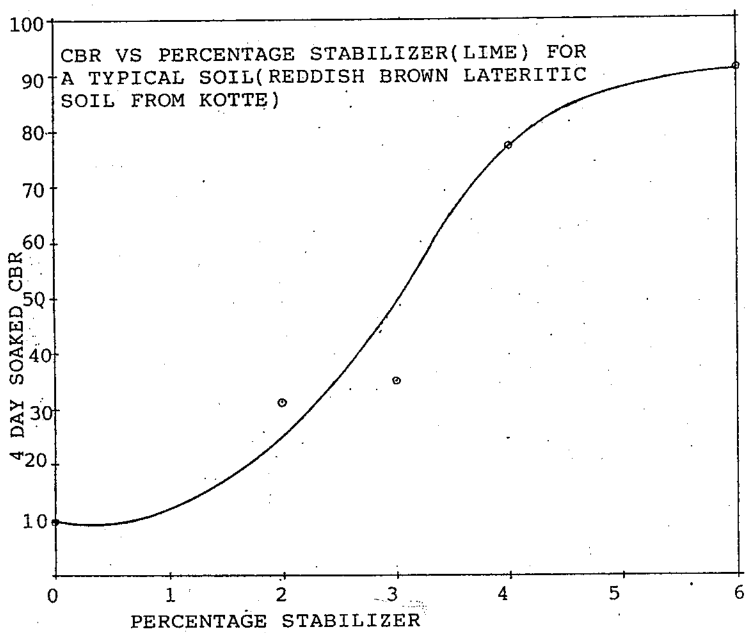


FIGURE 3

TABLE II - ABSTRACT OF LABORATORY TEST RESULTS FOR PALAVI - KALLADI ROAD

Sample Description	Sieve Analysis Percentage Passing — Sieve Sizes U.S.													Liquid Limit Percent	Plastic Limit Percent	Plasticity Index Percent	Standard Compaction		C.B.R. Percent
	2"	1½"	1"	3/4"	½"	3/8"	No. 4	8	16	30	50	100	200				Optimum Moisture Content Percent	Maximum Dry Density 10^3 kg/m³	
Yellowish brown gravel from Cementigama Kalladi		100	97	90	67	54	46	41	37	34	30	23	19	35	24	11	8.2	2.22	23
with 2% Lime														44	29	15	9.4	2.16	150
With 3% Lime														-	-	-	9.8	2.20	160
with 4% Lime														40	27	13	9.9	2.16	170
Yellowish brown gravel from Cementigama Kalladi	100	99	91	79	57	46	32	28	24	21	19	14	11	35	23	12	6.6	2.24	30
with 2% cement														44	28	16	7.4	2.21	170
with 3% cement														-	-	-	8.2	2.19	130
with 4% cement														43	27	16	7.6	2.20	130

TABLE III - CBR TEST RESULTS OF SAMPLES TAKEN AFTER FIELD MIXING — PALAVI - KALLADI ROAD

First layer of cement stabilized soil of thickness 100 mm

Location (km)	C.B.R. per cent
0 + 080	150
0 + 150	160
0 + 200	140
0 + 250	135
0 + 312	160
0 + 346	160
0 + 398	175
0 + 438	160
0 + 505	150
0 + 535	160
0 + 623	180
0 + 683	> 200
0 + 735	180
0 + 795	190
0 + 944	135
0 + 995	150
1 + 052	160
1 + 205	115
1 + 230	150

Second layer of cement stabilized soil of thickness 100 mm

Location (km)	C.B.R. per cent
0 + 575	130
0 + 623	160
0 + 638	140
0 + 640	100
0 + 682	65
0 + 702	95
0 + 780	135
0 + 830	180
0 + 890	180
0 + 940	195
1 + 020	145

One layer of lime stabilized soil of thicknesses varying from 100 to 200 mm

Section (km)	C.B.R. per cent
1 + 500 - 1 + 600	75
1 + 600 - 1 + 700	70
1 + 700 - 1 + 775	85
1 + 800 - 1 + 900	110
1 + 900 - 2 + 000	100
2 + 000 - 2 + 100	140
2 + 100 - 2 + 150	160
2 + 150 - 2 + 200	150
2 + 200 - 2 + 300	175
2 + 300 - 2 + 350	160
2 + 350 - 2 + 400	130
2 + 400 - 2 + 450	145
2 + 450 - 2 + 500	50

One layer of cement stabilized soil of thicknesses varying from 100 to 200 mm

Section (km)	C.B.R. per cent
1 + 900 - 2 + 000	150
2 + 000 - 2 + 100	175
2 + 100 - 2 + 200	190
2 + 200 - 2 + 300	200
2 + 300 - 2 + 400	200
2 + 400 - 2 + 450	185
2 + 450 - 2 + 500	> 200

FOUNDATION TECHNIQUES FOR BUILDING IN MARSHY AREAS

J. J. P. Ameratunga[1], K. S. Senanayake[2] & H. D. J. P Samaranayake[3]

1.0 INTRODUCTION

The city of Colombo has developed over the years in the North-South direction along the coastal belt, conveniently avoiding the vast expanse of low lying marshy lands that existed towards the west. These low lying lands were treated as unsuitable for building purposes due the high cost and technical difficulties involved with reclamation and subsequent construction. Besides, functioning as ideal detention basins, they protected the city at times of flood distress. However, the accelerated development programmes embarked on during the last two decades necessitated the developers to focus their attention on these marshy lands. By the mid seventies, 90% of the city area had been developed, rather in a haphazard manner, and the major part of the balance area available for future development were situated within the low lying areas.

In the early eighties, accelerated construction saw the development of low lying areas for housing, office & commercial buildings, warehouses etc. Nevertheless such development did not necessarily take place using adequate engineering expertise. Reasons for this is the lack of knowledge of the behaviour of subsoils in the marshy areas and the failure on the part of developers and the industry not giving due recognition to the importance of geotechnical engineering. Many buildings were designed in a very conservative manner making the cost of construction very high. Some others were constructed with inadequate foundations and ended up with major distress requiring very costly treatment for their repairs and restoration. A balance between the two extremes have to be found with appropriate foundations which interact with the ground and superstructure to give the best performance at the lowest cost.

2.0 CONDITIONS PREVAILING IN THE LOW LYING AREAS

In finding foundation techniques suitable for low lying areas, the significant features of the land and the characteristics of the subsoil must be well understood. It is then only, these techniques could be exploited to the maximum advantage.

The significant features of the low lying areas in and around Colombo city are;

1) They generally exist below 2.0 m MSL contour line but most parts lie below 0.5m contour and perennially remain as submerged or water-logged lands. Even in the other parts ground water table is generally high.

2) Storm drainage network interlinked by the streams and canals pass through these lands which serve as detention basins during flood times. High flood levels in the order of 2.0m to 2.5m MSL can be anticipated depending on the location even after the rehabilitation of the drainage network.

3) Ground consists of cumulative deposits which are very weak and highly compressible.

Extensive investigations carried out by the National Building Research Organisation (NBRO) in the last decade have revealed the existence of organic subsoils in the low lying areas.

Some important characteristics of the subsoils are;

1) Organic soils are predominantly peat and are distributed over large extents and to considerable depths often exceeding 10m to 15m.

2) Organic soils occur as a single layer or as pockets or layers sandwiched among inorganic soil strata.

3) Peaty subsoils, by virtue of its formation and decomposition over the years, exhibit highly variable properties and possess very high moisture content, high void ratio, high compressibility and very low shear strength. Fundamental geotechnical properties of the organic soils in low lying areas in and around colombo are given in Table 1. The least decomposed material exist close to the surface as fibrous peat and has a high coefficient of compressibility compared to the more decomposed amorphous peat or the organic clays.

4) Peaty soils show rapid rate of consolidation in the primary stage, but the secondary consolidation is also significant and ground settlement owing to this factor cannot be ignored.

3.0 SOME PROBLEMS ASSOCIATED WITH THE DEVELOPMENT OF LOW LYING AREAS

From what has been discussed above, it is therefore evident that for any development to take place, it is necessary to raise the existing ground surface to maintain a Planned Ultimate Formation Level (PUFL) well above the ground water table and, as well, above the highest flood level anticipated in each area. Inevitably, in most areas, a fill a few meters high would be required to achieve the PUFL. On the other hand, the earth fill so constructed should provide a strong base to support the foundations. Therefore the fill should be well compacted and must have adequate thickness.

When a fill is placed over the marshy land and a building is constructed with its foundations supported on the fill, the building begins to settle. The important constituents of this settlement are;

1) Settlement of the subsoils due to building load (S_{sb})
2) Settlement of the subsoils due to fill load (S_{sf})
3) Settlement of the subsoils due to self weight (S_{ss})
4) Settlement within the fill due to building load (S_{fb})
5) Settlement within the fill due to self weight (S_{ff})

In the case of a well compacted gravelly fill placed over a naturally consolidated ground, the effect of components S_{ss} and S_{ff} could be negligible.

Settlement of a building can become a serious problem, particularly when it is excessive or when the settlement is not even throughout the building. On the other hand, significant differential settlement may lead to distress that could be aesthetically unpleasant and even pose danger or create psychological discomfort to the occupants.

The component S_{sf} or the settlement in the underlying soils due to the load of fill could be considerably large when the ground is highly compressible and/or when the fill height is large. Sometimes the fill material intrudes and sink into the natural ground particularly when the ground is very soft. Normally it is neither possible to lay the fill in uniform thickness throughout the reclaimed area nor to expect the subsoils to be reasonably uniform in characteristics and stratification as well. Therefore, it is reasonable to expect high differential settlement owing to S_{sf}.

Influence of the building load on differential settlement could be large particularly when uneven loads are transmitted through large foundations which can affect subsoils at deeper elevations.

4.0 COMMON FOUNDATION TECHNIQUES

4.1 Ground Improvement

Although the basic concepts of ground improvement were developed many thousand years ago, it is only in the last few decades they came into limelight. During the past 50 years or so many new methods of ground improvement have been developed (Senanayake,1992). There are different methods applicable to almost all kinds of soil; sands silts, clays and organic soils such as peat. Generally the action of these techniques is to either replace, densify, harden or reinforce the weak ground. Some of the methods that would be appropriate for the low lying areas in the Colombo area are discussed here.

4.1.1. Preloading

Preloading, as the name suggests, is the technique of loading the ground to pre-introduce settlements that would be expected under building loads. This method is suitable for the peaty soils which are highly compressible and predominant in the low lying areas. Vertical drains are generally used in combination with preloading to accelerate the consolidation process.

4.1.2. Surface Stabilisation

The shear strength of the subsoils can be increased by stabilising using lime or cement based hardening agents. Although this method has been applied for organic soils as well, effectiveness of stabilising peaty soils still remains doubtful.

4.1.3. Dynamic Consolidation

Dynamic consolidation or heavy tamping is a technique first developed for densification of loose sandy ground by tamping with a heavy hammer dropped through a considerable height. This method has become popular in the recent years for the improvement of organic soils as well, including peat and sanitary fills.

4.1.4. Partial or Full Replacement of Soil

If the weak soil layer is not very thick and exists within the first two to three meters of depth, one method for construction is the replacement of weak soil with well compacted selected material. However, in most low lying areas where ground water table is high and excavation becomes difficult, this method may not be suitable unless pumping facilities, dredging equipment and material suitable for hydraulic fill are conveniently available.

Another appropriate technique is the replacement of weak soils found beneath foundation locations of the building. The depth of replacement below the foundation level should be at least 2.0 to 2.5 times the width of the footing. It should be also ensured that the extent of replacement is wider than the footing at least by an amount equivalent to the above specified depth of replacement. This is because, in such an arrangement the foundation stress will be contained within the zone of replacement and moreover, at such depths, the influence of stress bulb of the foundation loads will be insignificant hardly affecting the underlying weak soils. Granular earth is desirable to be used in the fill which should be well compacted in layers to achieve a dry density of at least 95% of the Proctor dry density.

4.2 Foundations

4.2.1. Shallow Foundations

Considering economics and ease of building construction, shallow foundations appear to be the most appealing for low rise buildings in marshy areas. They are broadly classified in to;

i) Strip foundations
ii) Pad foundations and
iii) Raft foundations

and, are found in practice in many variations and combinations. The shallow foundations which are considered most appropriate for use in marshy areas will be discussed in section 7.0.

4.2.2. Deep Foundations

Owing to the unfavourable subsoil conditions found in most low lying areas in and around Colombo, deep foundations transmitting heavy loads should be founded on weathered or fresh rock and designed as end bearing piles. Deep foundations with end bearing piles therefore cannot be justified for low rise buildings, but are appropriate for high rise buildings which transmit heavy loads to the foundations.

In some low lying areas, moderately strong and sufficiently thick soil layers which are suitable as bearing stratum are found within reasonable reach, i.e. about 3m to 7m from ground surface. For moderately loaded structures, when shallow foundations are found to be inadequate, under-reamed piles, cylinders or stone/gravel columns can be used to transmit the loads to such bearing strata found at intermediate depths.

5.0 CONSTRUCTION OF FILL

5.1 Importance of Fill in Building Construction in Marshy Areas

As mentioned earlier, the marshy lands that are to be developed must be reclaimed by at least a meter or two, to raise the ground level well above the expected high flood level. On the other hand, because of weak subsoils which do not possess adequate bearing strength, it is very important to provide a stronger base which can be used to support the structures or to facilitate construction activities. To achieve this, earth filling must be carried out in a controlled manner. The importance of a well controlled fill and some measures that could be taken to minimise settlement and distress in buildings constructed on fill are discussed below.

5.2. Settlement of Subsoils due to Fill Load and Building Load

When building load is transmitted to the ground through the foundation, the extent to which the stress bulb extend depends on the width of foundation. Say for a strip footing of width B, the increase in ground stress at a depth of 2B below footing level is estimated to be only about 10% of the bearing pressure. Therefore it could be seen that the stress bulb of a narrow footing would affect only the uppermost layers, and perhaps may not affect the subsoils appreciably if the filled layer is sufficiently thick. On the other hand, stress bulb of a wide foundation, such as a raft foundation can affect subsoils in deeper elevations. Similarly, since earth fills on marshy land are spread over a wide extent of ground surface, fill load will be fully experienced by subsoils throughout the overburden. As such, settlement in marshy ground due to fill load could be significantly high. Ameratunga and Senanayake (1989), for example, have shown with test data that for a fill of 2m height, the settlement of the subsoils could be as high as 1m when the thickness of the weak subsoil strata was only 4m. However, this is not an exaggeration when considering the large volumes of earth that sink into the swampy ground at the initial stages of filling operations, unless precautions are taken to separate the fill material from the subsoil using geotextile sheets etc. Anyway, such order of large settlements cautions us on the need of additional filling that would be required to maintain the planned ultimate formation level.

The more concerning aspect of this settlement is associated with the building construction itself. If the building is constructed before major part of the settlement due to fill is over, the effects on the building could be highly disturbing. If the settlement due to fill load, which is often the largest component of settlement, is rather even throughout, there would be no appreciable 'differential settlement' contributed by fill load although the building will be subjected to 'total settlement'. However, if the subsoil strata are heterogeneous, which is often the case, settlement will be uneven and the building will undergo distress depending on the order of differential settlement.

Senanayake (1986) showed that the settlement expected in underlying soil due to the building load in most cases is in fact one order lower than that due to the fill load. Therefore if the building construction is delayed after the fill is replaced, the residual settlements can be brought within tolerable limits.

An important consideration when designing warehouse type structures in marshy areas is the emphasis on live loads over the building loads. Live loads in such structures are distributed over a wide extent of ground surface and the effect will be similar to that due to a fill load. A classical example of a warehouse building that had undergone distress due to the intensity of live load is described by Sivakugan et al (1989).

5.3 Settlement within the Fill due to its Own Weight and Building Loads

When a fill is placed without properly compacted, a loose state of compaction prevails over the extent of the fill. depending on the intensity of building loads and the heterogeneity of the fill, large differential settlements could take place within the fill which may cause distress in the building. Loose fill consolidates under its own weight and also could settle as a result of subsurface seepage and erosion. Further, a loose fill will not be a good bearing stratum and may cause bearing failures. In fact a well compacted fill of selected material could provide a stronger bearing stratum than most naturally existing residual soil strata found in the region.

5.4. Construction of the Fill

5.4.1. Quality of Fill

Herath (1989) explains in detail the selection of borrow material, the recommended compaction procedures and associated testing for quality control. In general, well graded soils with fine content not exceeding 30% and the standard Proctor density not less than 1.8 g/cm^3 can be taken as a good fill material provided it is free from deleterious material. Selected fill material should be placed in layers not more than 30cm thick in loose state and must be compacted to at least 95% of the standard Proctor density in order to achieve a satisfactory earth fill suitable for general building purposes.

It is essential to carry out quality control tests for selection of fill material and the field compaction as well. Herath (1989) lists the following factors that are of paramount importance and need to be checked to achieve a good quality fill.

a) quality of fill material
b) optimum moisture range
c) optimum lift thickness of fill layer
d) appropriate compaction plant.

5.4.2. Time Lag between End of Filling and Beginning of Construction

The time period between end of filling and beginning of building construction (t_c) becomes crucial in most cases as far as the tolerable settlement is concerned. It would be possible to obtain a very crude value for t_c by using known theoretical equations with parameters like the thickness of compressible soil strata and the coefficient of consolidation (c_v). The value of c_v is conveniently predicted from the oedometer tests, but it is generally accepted that the field c_v value could be several orders higher than the laboratory value. This seems to be true even for organic soils, especially peat, which is apparently due to the macro-structure of subsoils, numerous drainage paths owing to sandwiched sand/silt layers and many other non-homogeneities. It is considered that a more reasonable value of c_v could be obtained by using the coefficient of volume compressibility (m_v) predicted in a laboratory test and the coefficient of permeability determined from a field test in the following equation:

$$(c_v)_{field} = (k)_{field})/ \gamma_w.(m_v)_{field}$$

However, the best method of assessing the rate of consolidation may be by means of a trial embankment loading. Observations of the settlement of trial embankment when plotted against log-time, will enable the $(c_v)_{field}$ of the strata to be estimated. Realistic ranges of coefficient of consolidation obtained for organic soils found in and around Colombo through field trials, if conducted, will be immensely helpful to the practicing engineer.

As an alternative to a trial embankment, the settlement of the constructed fill can be monitored over a period of time before deciding on the appropriate time to begin construction. A method for establishing the coefficient of consolidation and hence the total settlement, where consolidation is practically one-dimensional, is given by Asaoka and Suzuki and described by McAnally and Boyce (1980). If settlement readings are taken at equal time intervals, and plotted with the total observed settlement at the end of the preceding time interval, a straight line plot will result. The

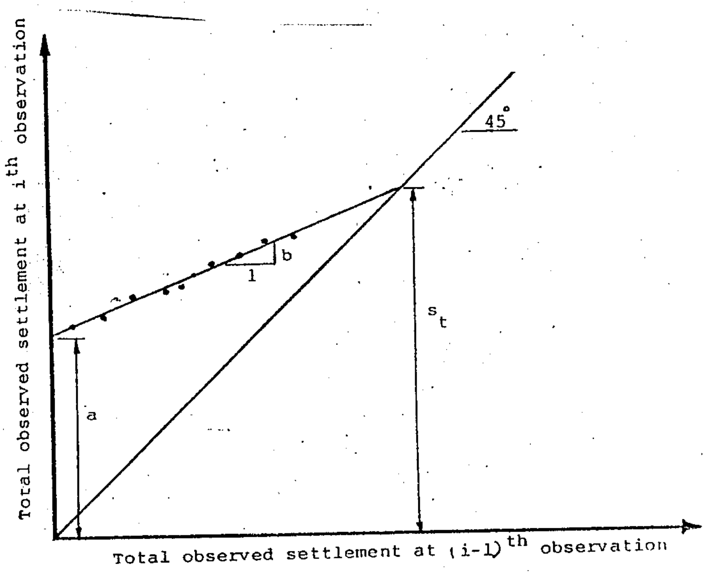


FIGURE 1

total settlement from the time of commencement of observations will then be given by (Fig. 1)

$$s_t = a/(1-b).$$

In order to establish the straight line relationship as early as possible, grouping of the settlement observations can be made. If the time interval between observations are too small, then the line will be close to 45_o gradient, but the convergence with the 45_o gradient will not be well established. As an example for such grouping, observations may be made on a weekly basis. Then the observations at 10 weeks, 20 weeks, 30 weeks etc., will form one group while observations at 9 weeks, 19 weeks, 29 weeks etc., will form a second group giving more points on the plot in a relatively short duration of monitoring. Once the linear relationship is established, c_v can be estimated from the following equations;

$b=\exp(-6c_v.\hat{}t/H^2$ for permeable top and bottom boundaries
$b=(-2c_v.\hat{}t/H^2)$ for one impermeable boundary.
where H is the full depth of the consolidating stratum.

It is claimed that this method will allow the c_v value and the total settlement to be estimated within the first 10% to 20% of the consolidation process.

Now, using this c_v value which has been estimated, the ultimate settlement due to fill load and building load etc. it will be possible to determine when to commence building construction so that residual settlement will be contained within tolerable limits.

6.0 PRELOADING AS A GROUND IMPROVEMENT TECHNIQUE

6.1. Adaptability of Preloading for Low Lying Areas

Preloading is called by different names by different groups viz. precompression, surcharge etc. but is considered as one of the most economical methods of foundation techniques appropriate for marshy areas. According to Stamatopoulos and Kotzias (1985), conscious reliance on preloading to improve soil began in the twelfth century, although the concept itself may be much older. In Sri Lanka, only a few instances where preloading has been used are documented.

Apart from its economy, one of the major advantages under Sri Lankan conditions, is the fact that this technique needs only equipment associated with earthworks. By measuring ground movement and pore pressures in the ground, it is possible to arrive at realistic designs. One disadvantage of the technique is that it needs space beyond the perimeter of the planned building which becomes a problem in small sites particularly in built-up areas and if there are adjoining structures liable to settlement. A large volume of earth, a few times the volume of earth necessary for the fill itself, would be required for the preloading operation. This becomes another disadvantage if borrow material could not be found easily or if the disposal of excess material after preloading becomes a problem. Where a large site is to be developed, available borrow material for reclamation could be used as a preload and the ground improvement can be progressively carried out across the site with careful planning of the construction sequence.

Time becomes a crucial factor in preloading technique because the building construction gets delayed by that much as the duration needed for application and removal of preload. Delays due to waiting time of consolidation can be well compensated for by proper planning and timely decision making which otherwise take more time in many instances due to logistics. If the need for ground improvement is identified at the planning stage and if preloading work is planned and executed while the detail designs are in progress, the time factor will not be critical.

A favorable aspect in the low lying areas is that peaty soils, unlike inorganic clays, have a relatively higher rate of settlement and therefore shorter duration is required for preloading. Consolidation process under preloading can be further accelerated by improving drainage within the subsoil using vertical drains. Soil profiles in the Colombo low lying areas often show presence of lenses and layers of high permeable soils, such as silts and sands, sandwiched in between the low permeable strata and therefore vertical drains may not prove to be always necessary.

6.2 Design of Preload

Mitchell and Katti in their State-of-the-Art Report on Soil Improvement gives a detail account of the work carried out by Aldrich (1965), Johnson (1970) and many others.

Settlement at any time (s_t) can be expressed as;

$$s_t = s_i + U.s_{cons} + s_{sec}$$

where,

s_i = immediate settlement

U = average degree of consolidation
s_{cons} = final consolidation settlement
s_{sec} = secondary settlement.

The objective of the design may be either

1) to determine the magnitude of surcharge pressure (p_s) required to ensure that the total (or desired part of) settlement expected under the final pressure (p_f) will be complete in a given length of time, or

2) to determine the length of time required to achieve a given amount of settlement under a given surcharge load.

Consolidation settlement estimates for permanent fill and structure loads are made in the usual way, by dividing the stratum to a series of sublayers, with the use of Terzaghi's equation. Now if a surcharge load which applies an additional stress p_s were also used and left in place indefinitely, then a time-settlement curve could be plotted in the same way as shown in Fig. 2. If the surcharge were left in place for time t_{SR}, and the layer settles by an amount equal to that to be expected under the permanent fill alone, i.e. $s_R=s_f$, then the average degree of consolidation U_{SR} reached would be,

$$U_{SR} = s_f/s_{f+s}$$

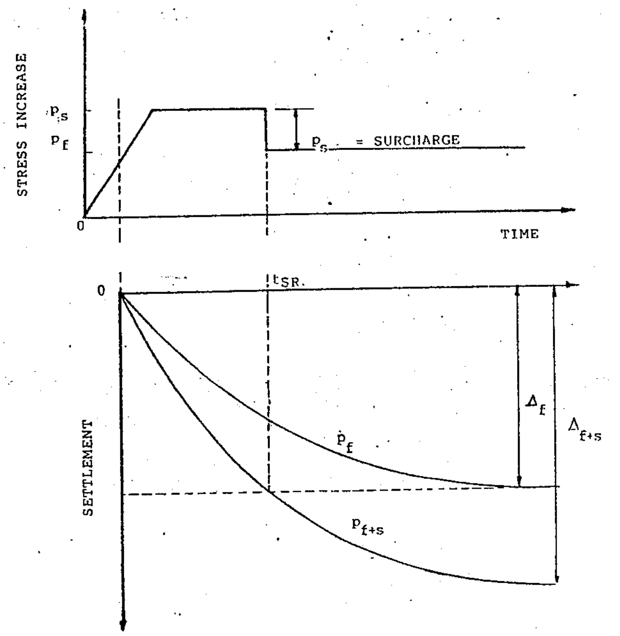


FIGURE 2

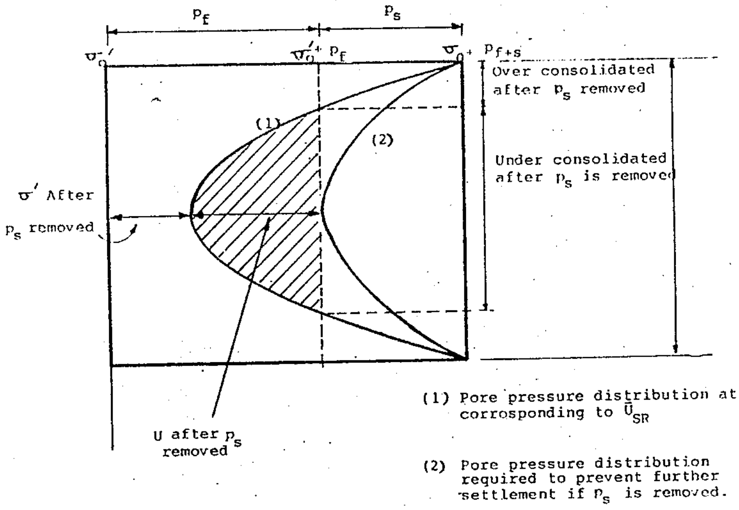


FIGURE 3

As pointed out by Aldrich (1965) and Johnson(1970), the distribution of effective and excess pore water pressures before and after surcharge removal will be as shown in Fig. 3 for a clay layer drained at both boundaries. Therefore, a substantial portion of the layer may undergo further consolidation after removal of the surcharge load, whereas the remainder will be unloaded. As the heave may not be very high, the additional consolidation in the central part may be important. To eliminate this, the surcharge should be left in place until pore pressure at the most critical point, i.e. the point that is last to consolidate, has itself reached a consolidation ratio U_z given by,

$$(U_z)_{f+s} = p_f/(p_f+p_s) \ldots\ldots. (1)$$

Times corresponding to U_{SR} and $(U_z)_{f+s}$ are found by using the Terzaghi's theory and the coefficient of consolidation c_v according to,

$$t = T.H^2/c_v$$

where, T is the appropriate non-dimensional time factor.

Secondary compression (s_{sec}) represents a very significant portion of the total compression of organic soils, specially peats. Preloading may be effective for minimising the effects of subsequent secondary compression under permanent loads.

$$s_{sec} = C_a.H_p \log(t/t_p)$$

where,

$t>t_p$

C_a = vertical strain per log cycle increase in time subsequent to the end of primary consolidation at t_p, and

H_p = layer thickness at time t_p.

The corresponding equation to (1) for the critical point for this case is

$$U_z)_{f+s} = \{s_f + C_a.H_p \log(t/t_b)\}/s_{f+s} \ldots\ldots(2)$$

The nature of secondary compression is such that some time after surcharge p_s is removed, secondary compression will reappear under p_f, but this effect is small and can usually be neglected.

7.0 FOUNDATION TYPES USED IN THE LOW LYING AREAS

7.1 Type of Foundations

Foundation types used in low lying areas can be categorised as follows;

1) Masonry strip footings
2) RC strip with rubble foundations
3) Inverted T footings
4) Vierendeel girder type foundations
5) Raft foundations
6) Pad footings

For single storeyed buildings, the most common footing type used in the low lying areas is the strip footing with or without a rubble base. For two storeyed structures stiffened foundations such as the inverted T and vierendeel girder type are increasingly becoming popular. RC pad footings for columns and RC strip foundation for walls are in common usage , with or without plinth beams, in many of the low-rise framed structures.

7.2 Appropriateness of Foundations

Based on the observations made on the performance of a large number of buildings constructed in the low lying areas, (Sivakugan et al 1989, Tennekoon, 1989, Tennekoon et al 1989) appropriateness of the different types of foundations can be summarised as follows.

7.3 Strip Foundations

Many of the lightly loaded buildings monitored in the low lying areas have been founded on rubble/brick masonry strip foundations where as in some others RC strips have been employed beneath the masonry. It had been observed that masonry foundations undergo total settlements as much as RC foundations. Although an RC strip is expected to reduce settlement, observation of building performance in the low lying areas (Tennekoon et al 1989) have shown that it has no rationale. It is postulated that masonry foundations respond with arching action whereas RC foundations respond with bending action in carrying the load. Therefore, in situations when strip foundations are found to be feasible, masonry foundations are found to be suitable for lightly loaded houses built on compressible ground.

7.4 Stiffened Foundations

Often independent pad footings are used for columns in framed structures whereas the walls are supported on strip foundations. But in general in the low lying areas, RC framed buildings having independent footings performed less satisfactorily than others. This can be expected if the footing level is close to underlying weak soil layer and differential settlements are high due to different column loads and due to variable soil conditions. The relative stiffness between the structure and the ground, and the stiffness of the structure as a whole are very important factors that govern building response to foundation settlement. By introducing connections between columns i.e. by stiffening, differential settlements could be reduced in some cases. Framed structures tend to be less stiff than load bearing wall structures because of the large openings that were provided in the walls for doors and windows. As a result the contribution of the frame to the overall stiffness is small compared to the contribution of the infilling (Tennekoon et al 1989). Provision of a stiffened foundation such as Inverted-T or a vierendeel girder could help to even out differential settlements. Thirteen 2-storeyed buildings constructed in a single site on a 2m thick fill over a 5m thick peat layer using inverted-T foundations performed fairly satisfactorily with only minor distress shown. Vierendeel foundation type was found to be a good method of providing stiffness to framed structures.

7.5 Raft Foundations

Raft foundations are used when the bearing pressures should be drastically reduced owing to low bearing strength of the supporting soil or when it is necessary to reduce differential settlements by bridging over variations in subsoils. With the increased size of foundation it is generally possible to increase the bearing capacity of the ground by using a raft. However, this is counter effective in low lying areas as the weak soils below the fill layer of limited thickness is affected by the foundation load to a greater depth. Therefore, in adopting raft foundations on fills over weak subsoils, it is important to depend mainly on the strength and stiffness of the fill which jointly functions as a raft.

7.6 Pad Footings

In developing a low lying area for building purposes, a fill with a thickness of at least 1m - 2m would be necessary. If the fill is well compacted and offers adequate strength to support the foundation, pad footings could be selected so that the stress influence zone does not extend deep into the underlying weak ground. The stress increment due to foundation load at a depth of 2B below foundation level, B being the width of footing, is about 10% of the foundation load. It is possible to spread the stress distribution horizontally rather than vertically by placing smaller pad footings at smaller intervals. This way the strength of the compacted fill can be utilised to the maximum while the the underlying layers are least affected by foundation load. However, as discussed under strip footings, it is desirable to connect all the columns by a grid of beams at plinth level in consideration of stiffness required to withstand differential settlements.

7.7 Deep Foundations

Although deep foundations such as piles supported on rock or other suitable deeper stratum appear to be a solution to problems of settlement and bearing capacity posed by weak subsoils in low lying areas, they are generally expensive and beyond the reach of average developer, not to mention the low-income house builder particularly when the bearing strata are found only at depths in the order of 10m to 25m in Colombo region.

Under-reamed piles (Thayalan et al 1989) can be employed as a low cost foundation alternative for lightly to moderately loaded buildings in areas where moderately strong inorganic soil strata are encountered.

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