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GOVERNMENT OF INDIA
MINISTRY OF RAILWAYS

GUIDELINES ON SOFT SOILS-STAGE CONSTRUCTION METHOD

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Geo-technical Engineering Directorate,
Research Designs and Standards Organisation
Manak Nagar, Lucknow - 11

PREFACE

Soft soils are characterized by low shear strength, high compressibility and low bearing capacity. Due to these factors, construction of railway embankment on these soils poses formidable challenge. In general, there is little appreciation of the problems associated with construction on soft soil, as a result, a number of projects have run into rough weather.

Given the recent impetus to port connectivity and other infrastructure development projects, construction on soft soils may witness an increase. Keeping this in view, need was felt to bring about awareness among field engineers about the various problems associated with construction on soft soils.

The guidelines deals with concept of soft soil and various available options for construction of embankment on soft soils. In particular, it emphasizes on stage construction method, which happens to be the most simple and economical method. Detailed sample calculations have been illustrated to help field engineers in making decision at their end.

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(Nand Kishore)
Executive Director/Geo-technical

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SOFT SOILS - OCCURRENCE & CHARACTERISTICS

1. OCCURRENCE

Soft clays are generally recent sediments laid down by rivers, sea or lakes. These deposits are characterized by bedding and laminations, sometimes intercalated with sand or silt seams and are usually subject to repeated desiccation and wetting near the surface. Soft soils exist in following environments-

- In low land areas near sea coasts where marine sediments are often found.
- In the vicinity of rivers, especially those which have been subjected to meandering.
- In local depression where the runoff is restricted and the soil contains appreciable amount of organic matter.

2. GEO-TECHNICAL CHARACTERISTICS

Soft clays are highly plastic fine grained soils with moderate to high clay fraction. They are characterized by high compressibility and low shear strength (generally less than 25 Kpa).They have following typical characteristics-

i) Predominantely fined grained i.e more than 50% of soil passing through 75μ IS sieve
ii) High liquid limit (W_L) & plastic limit (Wp) values
iii) High natural water content (NMC) and even higher than the liquid limit
iv) Low material permeability but the overall permeability can be more
v) Low shear strength which usually varies with depth. Based on the values of undrained strength, soft soil are classified into two categories-
   - Undrained strength less than 12 kPa represents the very soft soil
   - Undrained strength less than 25 kPa represents the soft soil

vi) Highly compressible , organic content increases the compressibility.

3. SHEAR STRENGTH CHARACTERISTICS

The undrained shear strength of normally consolidated clays increase almost linearly with depth. The ratio of undrained shear strength to the effective overburden pressure is related to plasticity index as proposed by Skempton by the following expression:

$$\frac{Su}{Po} = 0.11 + 0.0037\,Ip.$$

where

Su	=	Undrained shear strength of soil
Po	=	Effective overburden pressure
Ip	=	Plasticity Index

The above relationship is applicable to soft marine clay adjacent to the coastline where primary consolidation has just been completed. Cox reported a relationship for South East Asian clays, away from the coastline, which is approximately represented by the following equation-

$$\frac{Su}{Po} = 0.18 + 0.0058\ Ip.$$

The Su/Po values reported by Cox are higher than that reported by Skempton.

4. CONSOLIDATION CHARACTERISTICS

normally consolidated clays, a number of relationships as given below have For been proposed between the compression ratio $Cc/(1+e_o)$ corresponding to the virgin consolidation line and natural water content-

i) Lambe & Whitman

$$\frac{Cc}{1 + e_o} = 0.358 \log W - 0.448 \qquad \text{(Valid for water content } W > 34 \text{ per cent)}$$

ii) Cox

$$\frac{Cc}{1 + e_o} = 0.0045\ W$$

The Indian soft clays are generally lightly over consolidated with natural water contents in the majority of causes greater than 34 percent. The relationship proposed by Lambe & Whitman fits well for Indian soft marine soils & the error is about 15 per cent. The relationship proposed by Cox over-predicts the compression ratio considerably.

5. SOFT SOILS IN INDIA

In India, the major proportion of soft clays are marine and river delta deposits and are of pleistocene to recent in origin. They cover vast areas of the Gulf of Kutch, river delta areas, shores of the Gulf of Cambay and in general, the entire Eastern and Western coastal belts. The colour of these deposits is somewhat blue black, blackish or blue grey. Lab test reports of some of soft soils used in railway projects are given in Annexure-I.

6. TESTING FOR DESIGN OF EMBANKMENT ON SOFT SOIL

The investigation and preparation of soil samples including that of soft soils can be carried out as per the detailed instructions and procedures contained in following codes:-

- Classification as per IS 1498 -1970
- Determination of water content as per IS 2720 (Part 2)-1973
- Determination of Liquid and Plastic Limit as per IS 2720 (Part 5)-1985
- Determination of shrinkage factors as per IS 2720(Part 6) - 1972
- Determination of consolidation properties as per IS 2720(Part 15)-1986
- Determination of shear strength parameter as per IS 2720(Part 12) – 1981 & IS 2720 (Part 11) -1971
- Field Vane Shear test as per IS 4434 -1978

BASIC CONCEPTS

1.0 The design and construction of embankment pose little problem when the underlying subsoil is good bearing stratum. Soft soil conditions, however create several complexities for the designer and field engineer. When faced with the situation of constructing an embankment on a soft subsoil, the following problems are encountered:

- Low shear strength
- Stability of embankment
- High compressibility and settlement of embankment

The design of embankments is based on bearing capacity, settlement and stability considerations. All the conditions have to be satisfied with an adequate margin of safety for each of the factors.

2. SHEAR STRENGTH

2.1 The shear strength of a soil in any direction is the maximum shear stress that can be applied to the soil mass in that direction. When this maximum stress value is reached, soil is regarded as having failed, i.e., the strength of soil is fully mobilized. The soil gains its shear strength from two sources, viz., internal friction and cohesion between soil particles. A relationship between normal stress and shear strength was given by Coulomb as follows:

$$\tau_f = C + \sigma \tan\phi$$

τ_f = Shear strength of soil
C = Cohesion intercept
σ = Normal stress acting on the plane of the failure
ϕ = Angle of internal friction

The importance of pore pressures and drainage in controlling the shear strength of soils was studied by Terzaghi and the above equation was modified as follows:

$$\tau_f = C' + (\sigma - u)\tan\phi'$$

τ_f = Shear strength of soil
C' = Effective cohesion intercept
σ = Normal stress acting on the plane of the failure
u = Excess pore water pressure
ϕ' = Effective angle of friction

2.2 DRAINED AND UNDRAINED SHEAR STRENGTH

When a fine grained saturated soil is rapidly loaded, excess pore pressure is generated in the soil mass. The excess pore pressure generated by the sudden application of load gets dissipated by drainage over a period of time which in the case of clay deposits, may extend for many years.

The term sudden application of load is relative and a load application over several months during a construction period may be relatively a sudden application of load. The subsequent consolidation, under the influence of the increased load, gives rise to increased strength, and leads to increased stability. The lowest shear strength and, therefore, the most critical stability condition are generally at the end of construction when loading is completed. The critical shear strength at the end of construction, is termed as undrained shear strength, since no significant drainage has taken place and the stress state has not altered. When the rate of loading is very slow in the case of fine gained soils, it will result in enough time for the dissipation of excess pore water pressure. The soil structure in the shear zone is able to undergo volume change due to dissipation of pore water pressure. The shear strength under this condition is defined as drained shear strength. The undrained shear strength is different from drained shear strength and is generally considerably less in case of soft clays.

2.3 METHODS OF MEASUREMENTS OF SHEAR STRENGTH

2.3.1 DIRECT SHEAR TEST

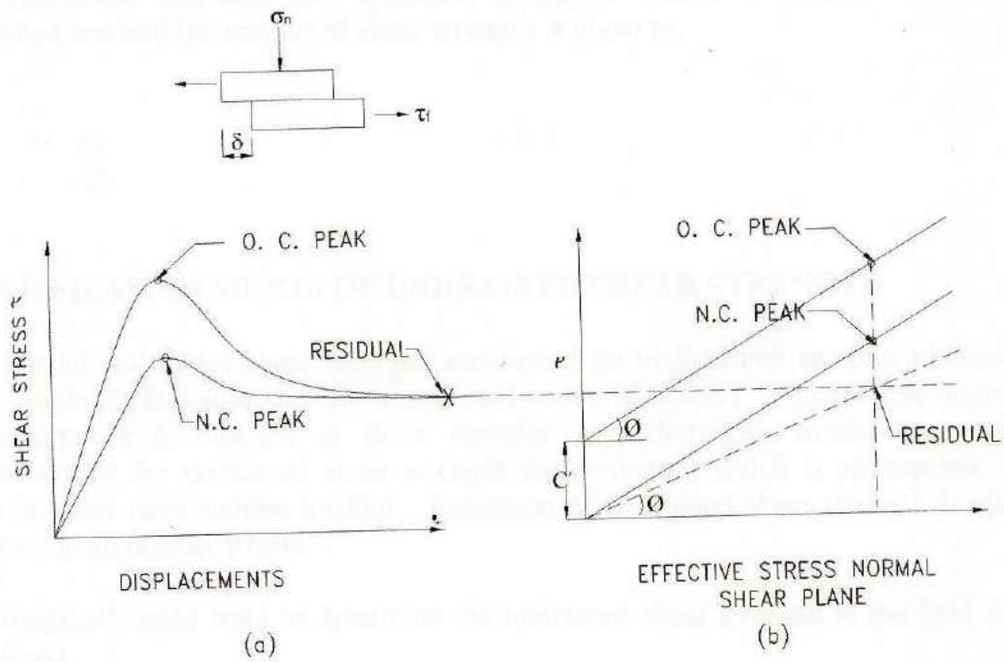
This test is a simple and an inexpensive one. The normal load is varied and for each normal load, shear stress upto and beyond peak value is recorded and plotted. This generally approximates to a straight line which is called the failure envelope. When test is conducted at a high shear strain rate, undrained shear strength is measured. When the test is conducted at a very slow shear strain rate, the excess pore pressure is almost zero, the resulting shear strength measured is drained shear strength. After exceeding the peak shear strength as shown in figure below, clays soften to a residual value of strength. The residual shear strength is given by the following equation:

$$S_r = C_{ri} + \sigma \tan \phi_{ri}$$

where Sr = Residual shear strength

C_{ri} = Residual cohesion intercept

ϕ_{ri} = Residual angle of internal friction



RELATION OF PEAK SHEAR STRENGTHS TO RESIDUAL SHEAR STRENGTHS AS MEASURED IN THE DIRECT SHEAR BOX FOR OVER CONSOLIDATED (O–C) AND NORMALLY CONSOLIDATED (N–C) CLAYS

Most over consolidated clays show a marked decrease in strength from peak to residual. These effects increase with clay content and degree of over consolidation.

2.3.2 TRIAXIAL COMPRESSION TEST

The greatest advantage of the triaxial cell over the shear box is the control of drainage through the base pore water ducts. It is possible to measure the total volume change of specimen in a drained case or excess pore water pressure in an undrained case. The shear strength of the soil is determined by Mohr Coulomb's failure criteria. The triaxial test gives undrained shear strength from Mohr's circle construction in unconsolidated undrained (UU) test. The drained shear strength is determined from consolidated undrained ($C\bar{U}$) test with pore pressure measurement and consolidated drained (CD) tests.

2.3.3 UNCONFINED COMPRESSION TEST

It is axial compression test. The cylindrical specimen of saturated clay is not sheathed in a rubber membrane and is not confined by a cell pressure, i.e., $\sigma_3=0$.

The cylindrical saturated clay specimen is rapidly loaded to failure. It is an undrained test and the undrained shear strength is given by,

$$Su = \frac{\sigma_1}{2}$$

2.4 FIELD MEASUREMENTS OF UNDRAINED SHEAR STRENGTH

The triaxial and direct shear tests are conducted on undisturbed samples obtained from boreholes and subsequently transported to the laboratory. Appreciable degree of disturbance is inherent in these samples and, therefore, 'in-situ' tests are recommended for undrained shear strength determination which is appropriate in clays in short term sudden loading. Indication of undrained shear strength is also obtained from plasticity tests.

The frequently used tests to determine the undrained shear strength in the field are as follows:

- 'In-situ' vane shear test
- Static cone penetration test
- Pressuremeter test
- Standard penetration test
- Dynamic cone penetration test
- Large diameter plate load test

In soft clay deposits, undrained shear strength can be assessed realistically by conducting 'in-situ' vane shear test.

3. STABILITY ANALYSIS

3.1 The objective of stability analysis in the design of embankments built on soft clay is to ensure that there is no risk of shear failure in soft clay, causing a disastrous collapse. A number of factors are known to affect soil behaviour & the stability of the embankment. The following critical conditions are normally considered:

i) **End of construction**:

This relates to the condition of stability developed during construction.

ii) **Long term condition:**

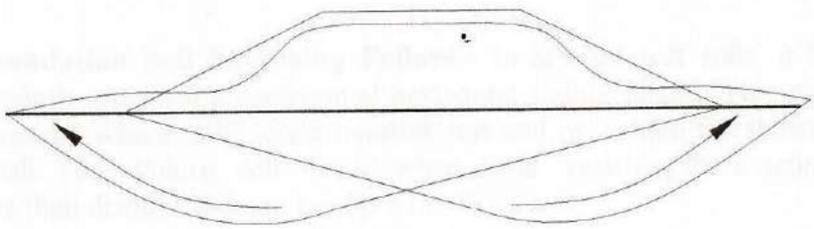
This relates to the condition when the excess pore pressure developed during construction are fully dissipated.

In recent years, the advantages of an observational approach have been demonstrated and it is usual to monitor the performance of embankments during and after construction. The construction of test sections of embankments, is particularly useful for large projects.

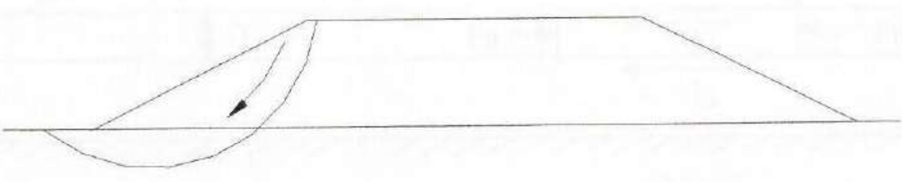
3.2 MODES OF FAILURE OF EMBANKMENT ON SOFT SOIL

Embankments generally fail by one of the following mechanisms:

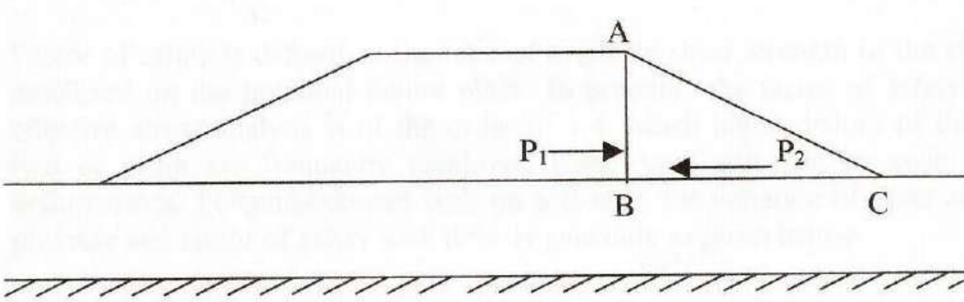
3.2.1. Bearing Capacity Failure –The collapse height of embankment H_{Max} is determined from consideration of bearing capacity failure.



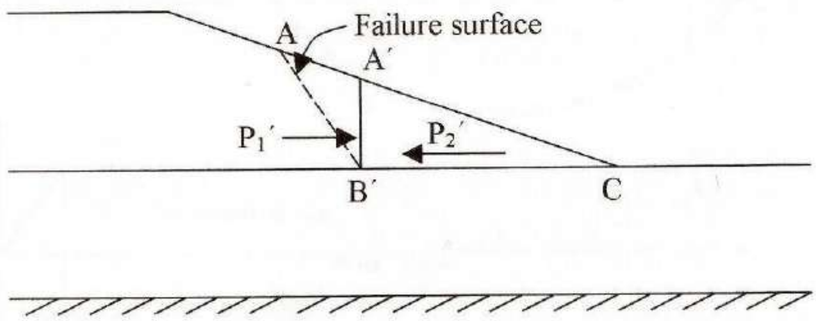
3.2.2. Rotational Failure – Rotational failure occurs when embankment height is less than or equal to H_{Max}. Failure is along circular arc passing through the foundation soil and the embankment.



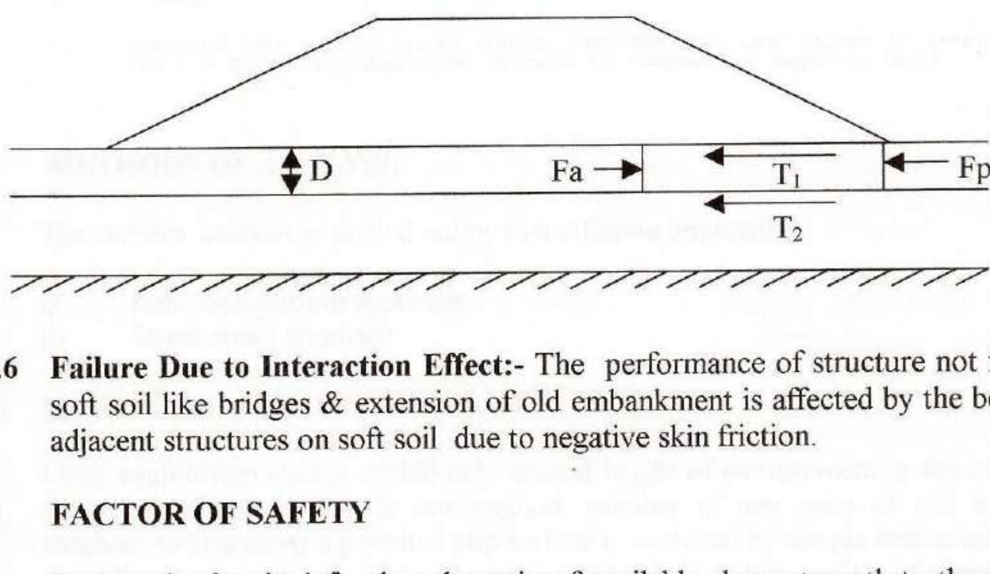
3.2.3 Sliding Failure- In the sliding failure, slope portion ABC slides laterally as a rigid body due to active pressure acting on it. Failure occurs when $P_1 > P_2$



3.2.4 Spreading Failure:- In this failure, soil wedge A B′C slides along B′C due to the active pressure P_1' acting on the face A′B′. Failure occur when $P_1' > P_2'$



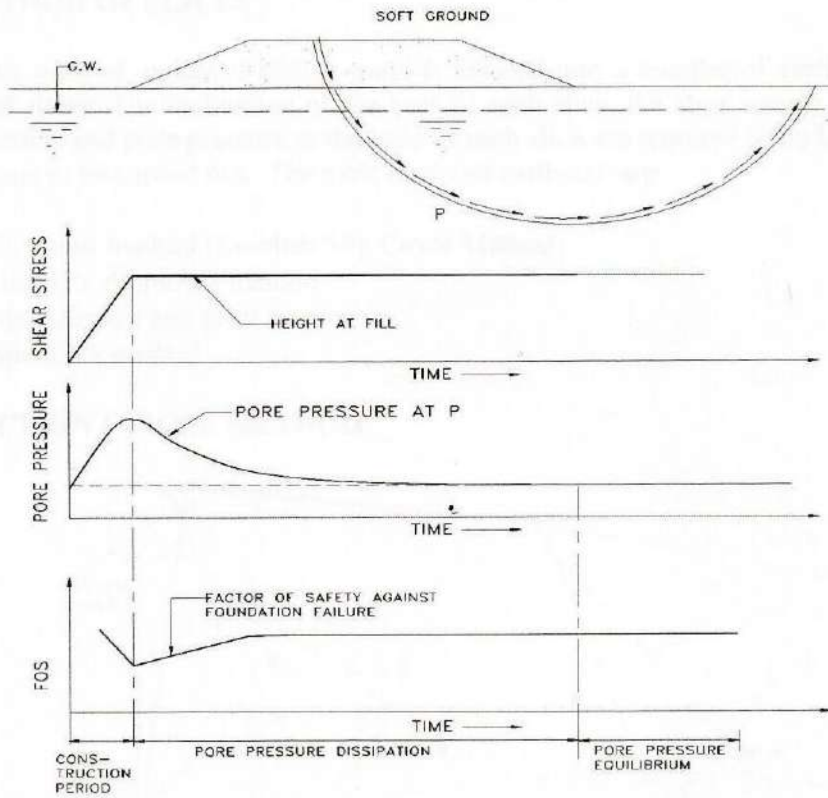
3.2.5 Foundation Soil Squeezing Failure:- In layered soft soils, a layer with a lower strength can form a preferential horizontal sliding plane. This mechanism may be favoured where stiff crest overlies soft soil or where the thickness of soft soil is small. The failure will occur when total resisting force acting on the block is less than disturbing force i.e. $F_p + T_1 + T_2 < F_a$.



3.2.6 Failure Due to Interaction Effect:- The performance of structure not founded on soft soil like bridges & extension of old embankment is affected by the behaviour of adjacent structures on soft soil due to negative skin friction.

3.3 FACTOR OF SAFETY

Factor of safety is defined as the ratio of available shear strength to the shear stress mobilized on the potential failure plane. In practice, the factor of safety taken for effective stress analysis is of the order of 1.4. Much higher values of the order of two or more are frequently employed with very soft soil to avoid excessive deformations. For embankment built on soft clay, the variation of shear stress, pore pressure and factor of safety with time is generally as given below-



VARIATION WITH TIME OF SHEAR STRESS, PORE-PRESSURE AND FACTOR OF SAFETY FOR THE SOFT CLAY FOUNDATION BENEATH AN EMBANKMENT (Bjerrum, 1972)

3.4 METHODS OF ANALYSIS

The stability analysis is carried out by two different approaches:

i) Limit equilibrium approach
ii) Stress strain approach

3.4.1 LIMIT EQUILIBRIUM APPROACH

Limit equilibrium studies enable only critical height of embankment or the minimum factor of safety of slope In this method, stability of any mass of soil assuming incipient failure along a potential slip surface is analyzed by simple statics with some simplifying assumptions. The soil mass bounded by a failure surface of simple shape is considered to be a free body. The resisting and disturbing forces at the assumed failure surface are estimated, and force and moment equilibrium of the potential sliding mass are evaluated. The calculations are repeated for a number of trial slip surfaces to find the potential slip surface which would be the trial slip surface giving the least factor of safety. Embankments built on soft clays have been observed to fail along circular slip surfaces. Therefore, in soft clays the slip surface is assumed circular. The following methods are widely used:

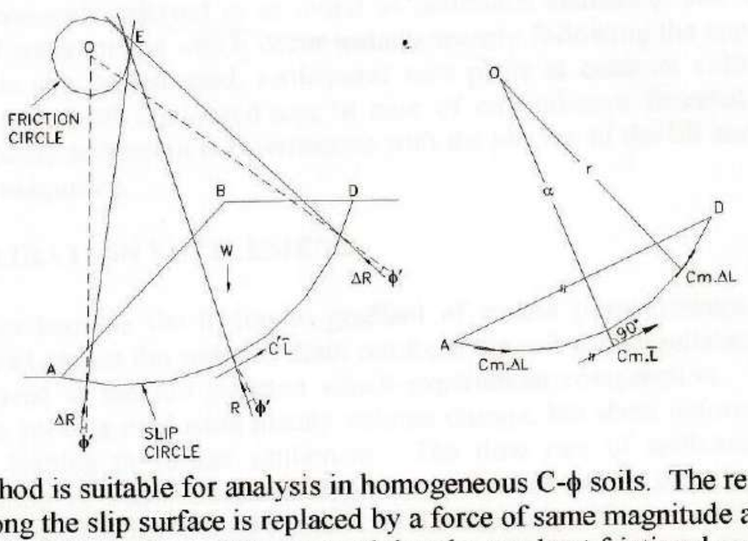
i) Method of slices
ii) Friction circle method

3.4.1.1 METHOD OF SLICES

In this method, potential sliding mass is divided into a number of vertical elements called slices. The inclination of the base of each slice, the slice weight, the material properties and pore pressure at the base of each slice are required to be known for the analysis to be carried out. The most common methods are:

- Fellenius method (Swedish Slip Circle Method)
- Bishop's simplified method
- Morgenstern and price method
- Spencer's method

3.4.1.2 FRICTION CIRCLE METHOD



The method is suitable for analysis in homogeneous C-ϕ soils. The resultant cohesive force along the slip surface is replaced by a force of same magnitude acting parallel to the chord of slip surface. It is assumed that the resultant frictional reaction 'N' passes through the intersection of resultant cohesive force along slip surface and weight of sliding mass. The resultant frictional reaction 'N' is assumed to be tangent to the circle of radius R=R sin ϕ_m. The error resulting from this assumption is on the safe side. Force equilibrium is considered analytically or graphically by means of force polygons. The magnitude for forces 'N' and 'C' are obtained from force polygons. The factor of safety Fe is defined as the ratio of available unit cohesion to the value of cohesion required for equilibrium. The calculations are repeated for a number of trial slip surfaces. In order to reduce the time in repetitions, Taylor, produced stability charts in terms of the stability factor Ns. The value of Ns varies with slope angle and friction angle.

3.4.2 STRESS STRAIN APPROACH

This method allows determination of stresses and strain within the potential sliding mass. The knowledge of state of stress and deformation in the soil is essential for accurate analysis of slope behaviour as zones of excessive deformations lead to progressive failure. . These deformations can be simulated by use of stress-strain approach. The stress strain approach allows a continuous assessment of soil behaviour up-to the point of failure. Generally, it is carried out by finite element method.

4. SETTLEMENT

4.1 Soils experience volume change when subjected to changes in applied stresses. The magnitude and rate of deformation depends on type of soil and on nature of applied loads. Terzaghi's theory of consolidation has provided the mechanism of the response of soil to applied loads. The soil parameters required for settlement prediction may be derived from consolidation test on undisturbed samples. When an embankment is constructed on soft clay, the following three components of settlements take place:

4.2 IMMEDIATE SETTLEMENT

This is commonly referred to as initial or undrained settlement and takes place on account of shear strains which occur instantaneously following the application of the load. If the clay is saturated, settlements take place at constant volume caused by shear strains beneath the loaded area. In case of embankment founded on soft clay, the immediate settlement is coterminous with the placing of the fill and as such of no further consequence.

4.3 CONSOLIDATION SETTLEMENT

This arises because the hydraulic gradient of excess pore pressure set-up by the applied load causes the water to drain out from the soil and simultaneously the stress is transferred to the soil skeleton which experiences compression. This is a time dependant process producing mainly volume change, but shear deformations are also involved leading to further settlement. The time rate of settlement for primary consolidation in one dimension may be determined using the classical Terzaghi theory. Method of computing consolidation settlement and other allied aspects are as detailed in stage construction method.

4.4 SECONDARY COMPRESSION

This is commonly referred as drained creep and the main part takes place essentially after complete dissipation of excess pore water pressures and at practically constant effective stress. There is at present no general agreement on how to separate consolidation into its primary and secondary components. It is convenient to consider primary consolidation and secondary compression in separate phases, even though both types of volume change may be in progress concurrently. An estimate of the secondary settlements requires a knowledge of the stress strain time relationship of the clay.

VARIOUS TECHNIQUES

1.0 A few of the techniques to improve the engineering properties of the soft subsoil have been discussed in brief below-

- Preloading
- Vertical drain
- Stone column
- Geosynthetics
- Dynamic consolidation

2.0 PRELOADING

The preloading technique is a simple one and is an economical method for accelerating consolidation as compared with other methods of improving ground support. However, adequate instrumentations for monitoring the settlements and the development and dissipation of pore water pressures is essential for the success of this technique. Preloading is particularly economical technique in the construction of railway fills on soft clays, since, the material can stay in place and need not be relocated. Preloading is especially attractive when fill material is subsequently used on the same project for site preparation. Another advantage of preloading is that is allows an immediate and direct assessment of its effects. By measuring the ground settlements and pore pressure, it is possible to assess quantitatively the extent of ground improvement in terms of increase in the shear strength and predict its future behaviour.

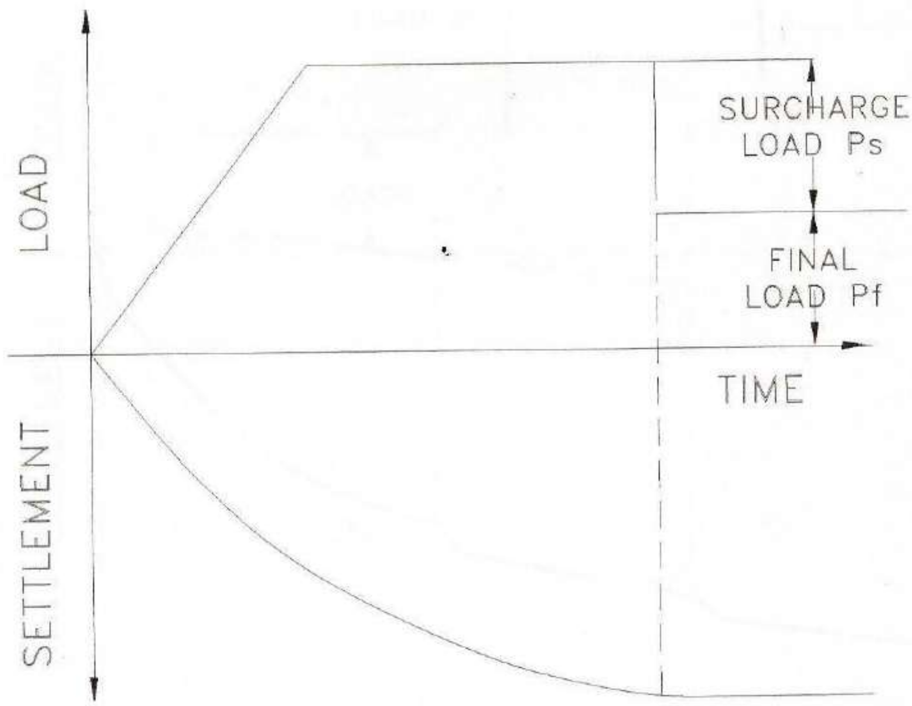
The duration of preloading from the beginning of embankment placement to the end of removal of load depends on the ground response. The pre-loading technique is likely to be inefficient when used alone because of very long periods of time required for obtaining significant consolidation settlements and subsequent appreciable strength gain of the soft clay to support the embankment loads. The preload time can be drastically reduced by the installation of vertical drains as they shorten the drainage path under which the clay will consolidate.

The pre-loading technique takes two forms:

i) Overloading
ii) Stage construction

2.1 OVERLOADING

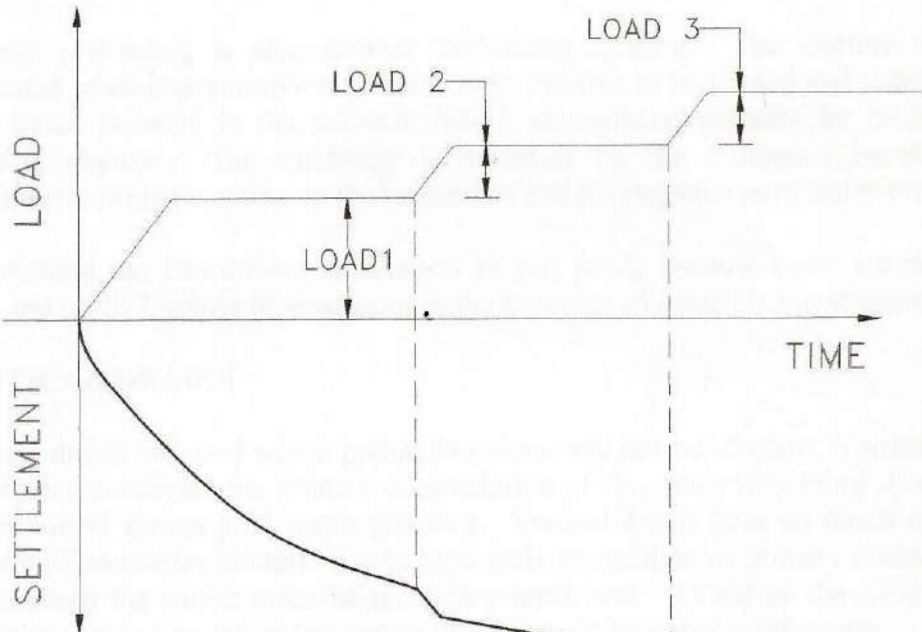
In the overloading process, a surcharge (overload) is placed temporarily on the ground and after a pre-determined time lapse, the intended structure can be built with occurrence of little or no additional settlement. The ratio of surcharge load to design load is known as overload coefficient. The charge is normally a uniformly distributed surface load which is placed prior to the construction of the intended structure. A part or all of the surcharge may be removed before the construction commences, depending on the requirements. The magnitude of the surcharge load and its duration of application are determined by the conventional settlement calculations. The settlement which occur under overloading results in an increase in the undrained shear strength of the clay. The principle of overloading method is shown in figure below-



2.2 STAGE CONSTRUCTION METHOD

Since the undrained shear strength of base soil is low initially, this restricts the construction of high embankments over soft clays in single stage or at a fast rate construction. Construction of high embankment over soft clays can be done in single stage safely only having very flat slopes or with wider sub-banks which is highly uneconomical with respect to land and earthwork. These problems may be overcome by the method of stage construction with observational method.

Stage construction is employed mainly as a means of gradually increasing the shear strength of a soft clay which would otherwise be inadequate to carry the intended embankment load without failure.. In stage construction, advantage of increase in shear strength of sub-soil strata due to consolidation by surcharge of embankment loading is taken into account. The gain in shear strength is a function of angle of shearing resistance improved in terms of effective stress parameters and degree of consolidation. The principle of stage construction method is shown in figure below-



Theoretical basis of design using stage construction method, solved practical examples and instrumentation scheme for monitoring the behaviour of embankment on soft soil are covered in detail in later chapters.

2.3 PRELOADING METHODS

2.3.1 The most common method of preloading is by heaping fill material over the intended site. The site must be cleared of vegetation and covered by base layer of free draining material. Surface vegetation must be removed to prevent future settlement due to long term decay of wood leaves and so forth and also to facilitate the placement of the base layer. The base layer must have a thickness of about 0.60 m and should consist of a mixture of gravel and sand and be free from clayey admixtures. It functions as a drainage face during the consolidation process, and also serves as working platform for construction equipment.

2.3.2 Another method of preloading is by lowering the water table. This is achieved by well points, trenches or vacuum pumping in deep wells. As the water table drops, the soil looses its buoyancy and its unit weight increases by about 10 kN/m^3. Each metre of drop in the water level produces about the same loading as half a metre of fill. Preloading by lowering the water table is generally avoided.

2.3.3 Jacking has been another method of preloading. This method has been applied mostly to individual footings of either new buildings or buildings to which extra storeys are to be added.

2.3.4 Vacuum preloading is also another preloading method. The method involves placement of an impermeable membrane over the area to be treated and reducing the pore water pressure in the sub-soil, below atmospheric pressure by means of a pumping shyster. The surcharge is achieved by the difference between the atmospheric pressure acting on the membrane and the negative pore water pressure.

The method has had limited application as yet, partly because costs are relatively high, and partly because of limitations in performance of available membranes.

3.0 VERTICAL DRAINS

Vertical drains are used where preloading alone will not be efficient. Vertical drains in soft clay accelerate the primary consolidation of clay since they bring about rapid dissipation of excess pore water pressure. Vertical drains have no direct effect on the rate of secondary compression but the early completion of primary consolidation brings about the earlier onset of secondary settlement. Therefore the structures or embankments can be put to use earlier than it would be possible otherwise.

The accelerated rate of gain in shear strength of clay enables the loads to be applied more rapidly than would otherwise be possible. Steep side slopes and avoidance of berms in case of embankments may be possible when sand drains are used.

The effectiveness of vertical drains depends mainly on the engineering properties of soils, namely, soil permeability and coefficient of consolidation and their variations in space and time. Vertical drains can be successful in accelerating the rate of consolidation of soft fine-grained soils. They are, however, ineffective in organic soils and highly stratified soils.

3.1 TYPE OF VERTICAL DRAINS AND INSTALLATION TECHNIQUES

Large diameter drains cause considerable smear problem around the drains. The alternative to large diameter sand drains was the much smaller band shaped drains. Generally such band drains consist of a central core whose function is to act as free draining water channel surrounded by a thin filter jacket which prevents the surrounding soil from entering the core. These band drains could be synthetic manufactured from polyethylene, PC m polypropylene and polyester, etc or natural geotextile. There are four types of vertical drain used in engineering practice and are as follows-

i) Sand drains
ii) Sand wicks
iii) Cardboard drains
iv) Synthetic drains

3.1.1 SAND DRAINS

Sand drains have been in use for the last fifty years. They have been installed by a great variety of procedures as shown in table below. The most common are closed mandrel and open mandrel methods.

Common methods of Installation

Group Description	Particular methods	Remarks
Displacement methods	• Driving vibration • Pull down (static force) • Washing • Combination of above	A mandrel with or without a disposable shoe is used in each case
Drilling methods	• Rotary drill, with or without a casing • Rotary auger, including continuous standard and hollow flight augers • Percussive methods with or without casing • Hand auger	————
Washing methods	• Rotary wash jet • Washed open ended casing • Weighted wash jet head on flexible hose	Methods in which sand is washed in via the jet pipe, are not suitable for prefabricated drains

Generally the drains are installed by any of these methods depending upon the site conditions and availability of equipment.

Closed mandrels consist of steel tubes closed at the lower end by a loose cap. They are driven down by hammering and vibration. It is a displacement method. Its major drawback is that lot of disturbance in soil takes place during installation. This results in decreased shear strength, permeability and increased settlement of the soil.

In open mandrel method, soil is remoulded by augering or percussion method and steel tube is inserted. Sand charge is placed in the hole and steel tube is recovered. Significantly less disturbance results from the open mandrel installation process than with the closed mandrel method. The diameter of sand drains range from 150 mm to 500 mm. Large diameter sand drains also act as granular piles in soft soil and modify the settlement behaviour of the structure.

3.1.2 SANDWICKS

Sand wicks are ready made small diameter sand drains pre-packed in filter stocking. In early days woven jute canvas was used as filter stocking, but presently polypropylene woven and melt bonded fabrics are used.

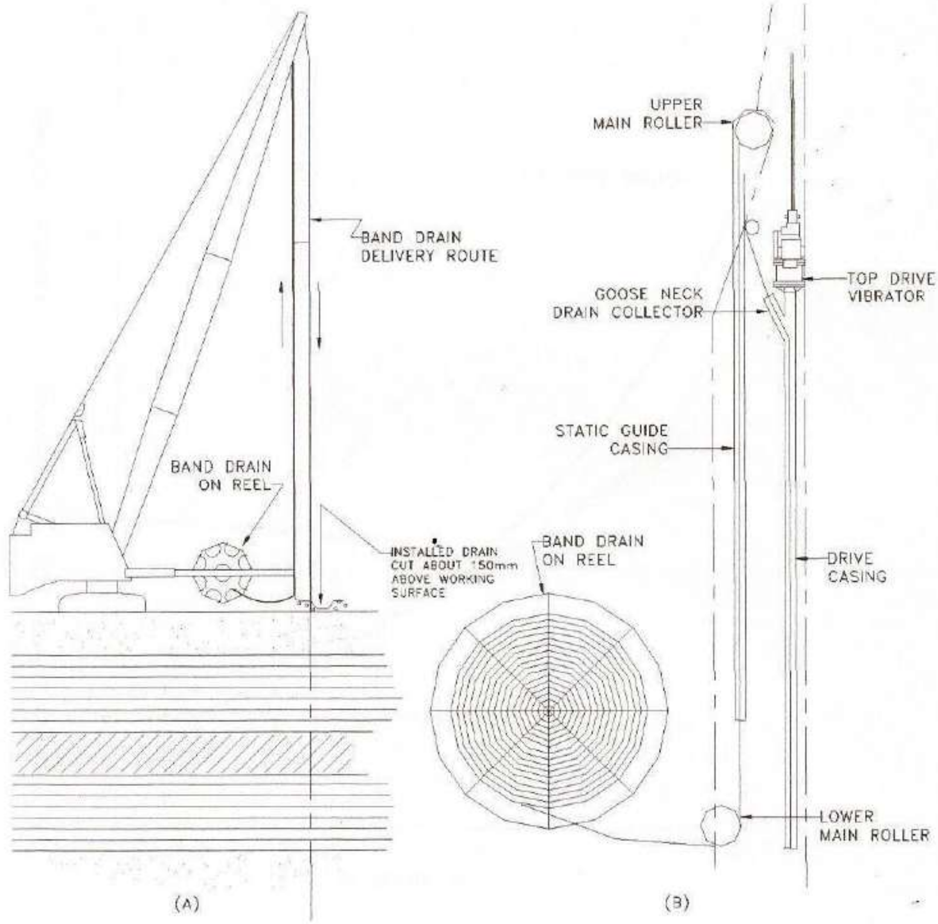
3.1.3 CARDBOARD DRAINS

They are inserted into the ground by means of a mandrel which is then removed. Channels in the cardboard facilitate the removal of water. They are easy to install and cause little soil disturbance. The specially processed cardboard has a long life and are quite durable.

3.1.4 SYNTHETIC DRAINS

Synthetic drains have replaced cardboard drains and they are widely in use. Synthetic fabric drains have shown wide acceptance. They are less bulky and easier to handle at site. However, they require sophisticated installation techniques. Synthetic fabric drains have not yet been widely used in India. The installation of plastic drains cause little disturbance to the neighbouring soil and have proven efficient over the sand drains. Large settlements of substrata do not destroy the drain continuity. These drains have been dealt in R.D.S.O. report no. GE-R-68 titled as "Prefabricated vertical PVC drainage system for construction of embankment on compressible soft soil".

The plastic drains are usually installed by displacement method. Auger and washing methods are not usually suitable. The mandrels used for plastic drains are hollow and rectangular or trapezoidal in cross section. The plastic drains are introduced at the top crane hoist by rollers and are placed over the mandrel by way of a goose neck' as shown in figure below-

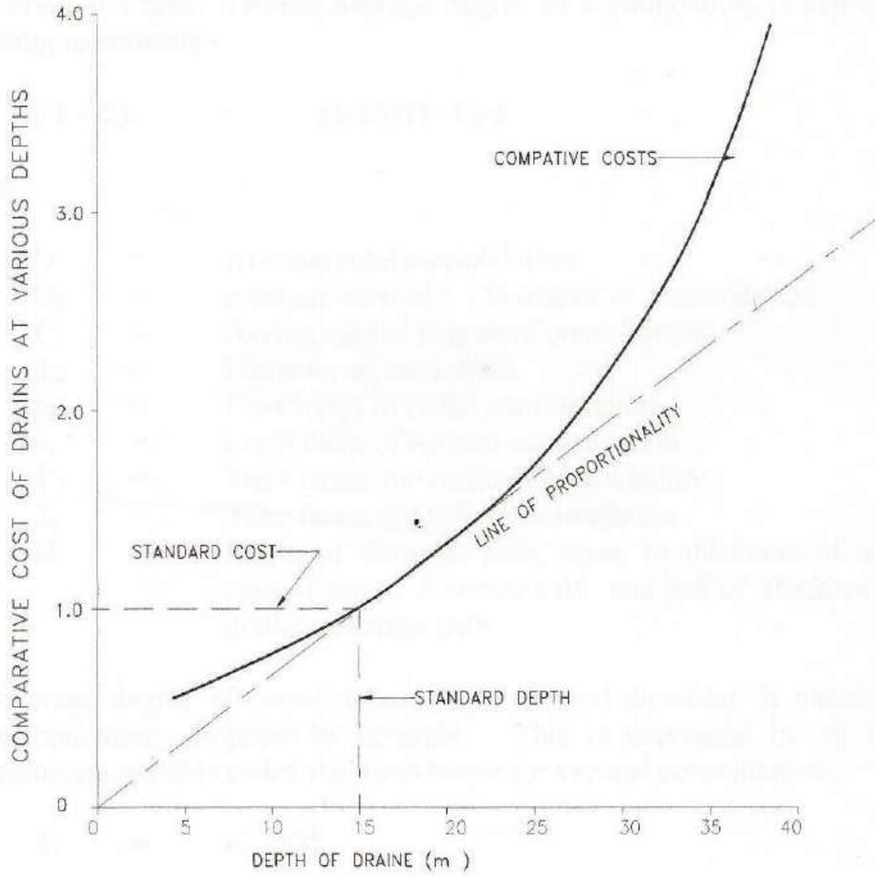


SCHEMATIC SKETCH OF A BAND DRAIN INSTALLATION RIG: (A) GENERAL ARRANGEMENT OF INSTALLATION; RIG: (B) BAND DRAIN DELIVERY ARRANGEMENT

3.2 SPACING, DEPTH & COST EFFECTIVENESS:

The design of any vertical drain project involves the determination of drain spacing which will give the required degree of consolidation in a particular period of time for a known type of drain.

The vertical drains are usually installed in triangular or square grid pattern with spacing ranging from 1 m to 4 m. The spacing is generally fixed depending upon the loading pattern of embankment and soft subsoil characteristics. The depth of treatment is often taken as the full depth of soft clay. For depth of 5-20 m of soft clay, full depth vertical drains prove to be economical. Beyond 20 m depth, the installation cost rise markedly. The relationship between cost and depth of plastic drains is illustrated in figure given below-



RELATIONSHIP BETWEEN COSTS AND DEPTH OF SYNTHETIC DRAINS TAKING THE COST FOR 15m DEPTH AS THE UNIT COST

Analysis of cost effectiveness of vertical drain technique is essential before opting for the same. The size of the job has a significant influence on the overall economics as it forms a significant part of the overall cost. For a small job, the cost of the machinery required will form a large proportion of the overall cost. Hence, it may not be attractive. The % cost benefit of closer spacing at shallow depth against increased depth of treatment should be a prime design consideration.

In case of sand drains, cost of sand is a major component. In sand wicks, the labour cost of prefabricating the drain is relatively high compared to the cost of sand. The synthetic drains have found wide acceptance in Europe & Western countries. Based on these countries experience they are cheapest in terms of basic material cost. However, such data is not available for Indian conditions.

3.3 DEGREE OF CONSOLIDATION

Consolidation of cylindrical body of soil around a vertical drain is three dimensional and is dependent on following parameters, namely, coefficient of radial consolidation, coefficient of vertical consolidation, pore pressure, radial distance from drain and time. Overall average degree of consolidation, is expressed by the following relationship-

$$(1 - U) = (1-U_r)(1-U_z)$$

where

U	=	Average total consolidation
U_z	=	Average vertical 1 - D degree of consolidation
U_r	=	Average radial degree of consolidation
d_w	=	Diameter of sand drain
c_r	=	Coefficient of radial consolidation
c_v	=	Coefficient of vertical consolidation
T_v	=	Time factor for vertical consolidation
T_r	=	Time factor for radial consolidation
H	=	length of drainage path, equal to thickness of clay layer in case of single drainage path and half of thickness in case of double drainage path

The average degree of consolidation in the vertical direction is based on the one dimensional theory proposed by Terzaghi. This is expressed by an independent dimensionless variable called the time factor for vertical consolidation.

$$T_v = c_v \, .t/H^2$$

The average radial consolidation is given as follows-

$$U_r = 1 - \exp\{-8\, T_r/F(n_r)\}$$

Where $T_r = C_r \, . \, t/d_e^2 \quad n_r = d_e / d_w$

$$F(n_r) = \left(\frac{n_r^2}{n_r^2-1}\right) \ln . nr - \left(\frac{3\, n_r^2-1}{4\, n_r^2}\right)$$

The diameter of the equivalent cylinder of soil surrounding each drain 'd_e' is calculated on the basis of equivalent cross sectional areas of drain, grid pattern and spacing of drains. The vertical drains are generally installed in square and triangular patterns.

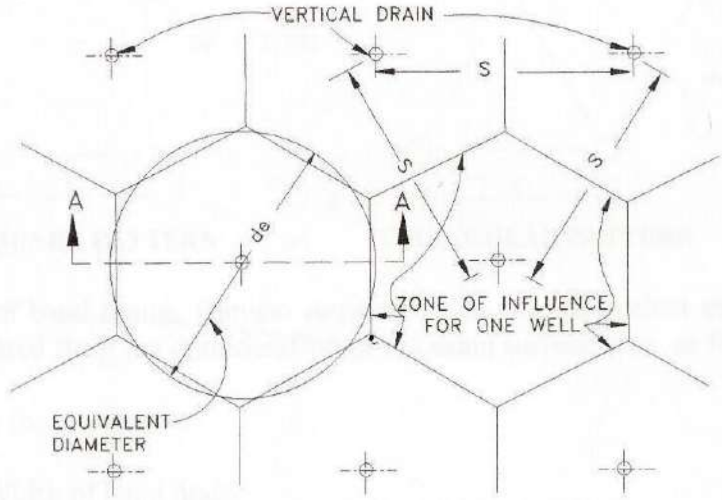
In a square grid pattern

$$d_e = 1.13\ S$$

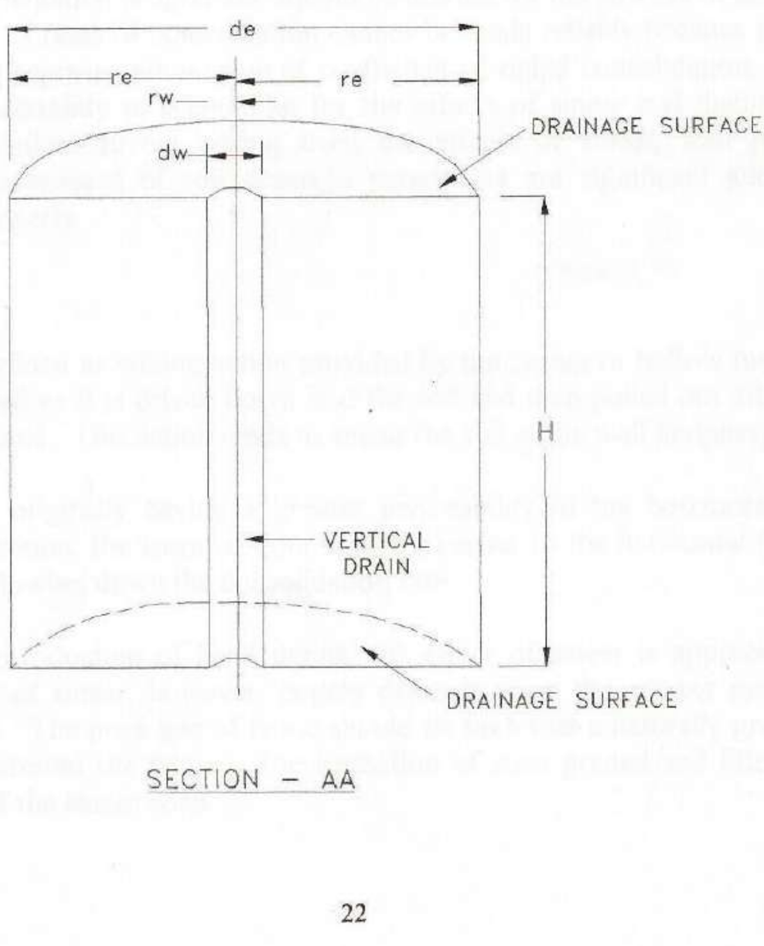
In a triangular grid pattern

$d_e = 1.05$ S

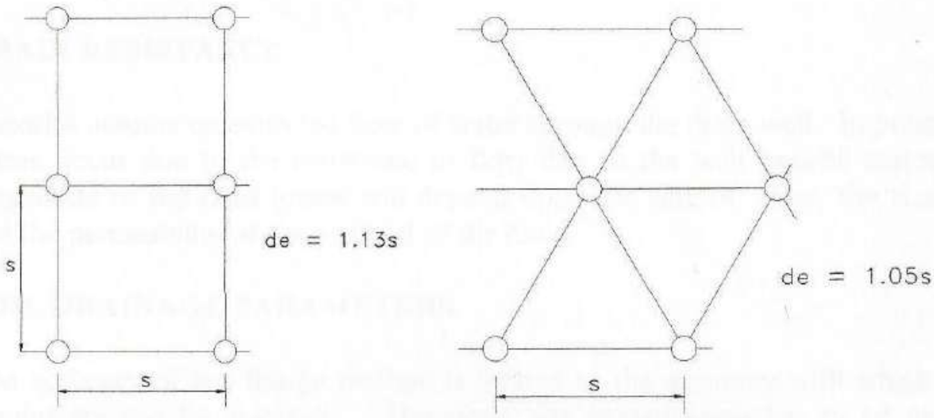
where S = spacing of vertical drains



PLAN OF VERTICAL DRAIN PATTERN



SECTION - AA



SQUARE PATTERN **TRIANGULAR PATTERN**

In case of band drains, Hansbo suggested that the equivalent diameter ' deq' could be estimated from the consideration of the drain surface area as follows-

Deq $= 2 (b_b + t_b))/\text{pie}$

b_b = Width of band drain
t_b = Thickness of brand drain

In most field situation, instrumentation, such as, piezometers, settlement gauges and inclinometers will be used to monitor performance of drains. Therefore, a careful field instrumentation programme should be drafted for the success of this technique. Predictions of rates of consolidation cannot be made reliably because of difficulty in determining representative value of coefficient of radial consolidation, Cr. There is inherent uncertainty in accounting for the effects of smear and disturbance during drain installation. Errors arising from the effects of smear, well resistance and incorrect assessment of soil drainage parameters are significant and need to be assessed properly.

3.3.1 SMEAR

Smear is defined as wiping action provided by the casing or hollow mandrel used to form the well as it is driven down into the soil and then pulled out after it has been filled with sand. This action tends to smear the soil at the well periphery.

For a soil originally having a greater permeability in the horizontal than in the vertical direction, the smeared zone forms a barrier to the horizontal flow of water, therefore, slowing down the consolidation rate.

With the introduction of band drains, the effect of smear is appreciably reduced. The extent of smear, however, largely depends upon the correct pore size of the drain fabric. The pore size of fabric should be such that a naturally graded soil filter is formed around the fabric. The formation of such graded soil filter reduces the thickness of the smear zone.

3.3.2 DRAIN RESISTANCE

Theories assume unrestricted flow of water through the drain well. In practice, head losses occur due to the resistance to flow due to the well backfill material. The magnitude of the head losses will depend upon the rate of flow, the size of drain and the permeability of the material of the drain.

3.3.3 SOIL DRAINAGE PARAMETERS

The accuracy of any design method is limited to the accuracy with which drainage parameters can be assessed. Therefore, site investigation has to be carried out under strict supervision so as to enable the evaluation of the required parameter realistically.

4.0 STONE COLUMN

4.1 Strengthening of soft sub-soils through stone column technique is a recent development. Stone column consists of granular material compacted in-situ in long cylindrical bore-holes. Well-graded granular material having high angle of internal friction, chemically inert, hard and preferably angular in shape is used. Stone columns are usually constructed in an equilateral pattern. The diameter of the stone column depends upon the undrained shear strength of the soil, gradation of the stone aggregate and construction method. A detailed field and laboratory testing is required for successful design and construction of stone columns.

When applied loads are transmitted to the cohesive soils reinforced with stone columns, a large portion of the total load is initially resisted by the relatively strong stone columns which are far more rigid compared to the surrounding cohesive soil. The remainder of the load is carried by soft cohesive soil. Variations in the sharing of the total applied load between the stone columns and the soils takes place over a period of time until strains in both the materials achieve compatibility and equilibrium conditions are attained.

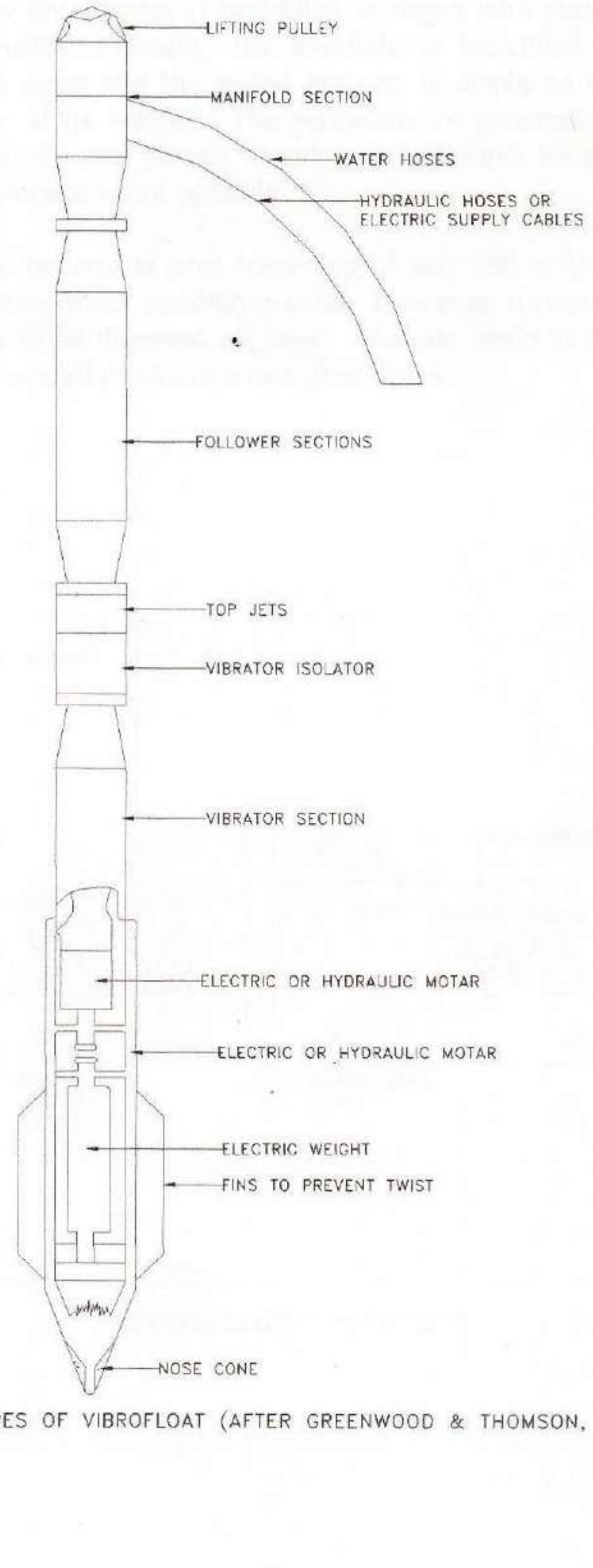
Stone columns interact with soil so as to increase the average shear strength and reduce the overall compressibility of the treated soil. At the same time, they act as drains and consolidation settlements are accelerated and post construction settlement are minimized. It provides increase in load carrying capacity and a significant reduction in total and differential settlements.

4.2 Installation of stone columns is carried out by following methods:

- Vibro-flotation
- Ramming techniques
- Sand compaction piles

4.2.1 VIBRO-FLOTATION PROCESS

Vibro-replacement (wet process) and vibro displacement (dry process) are two techniques of this process.

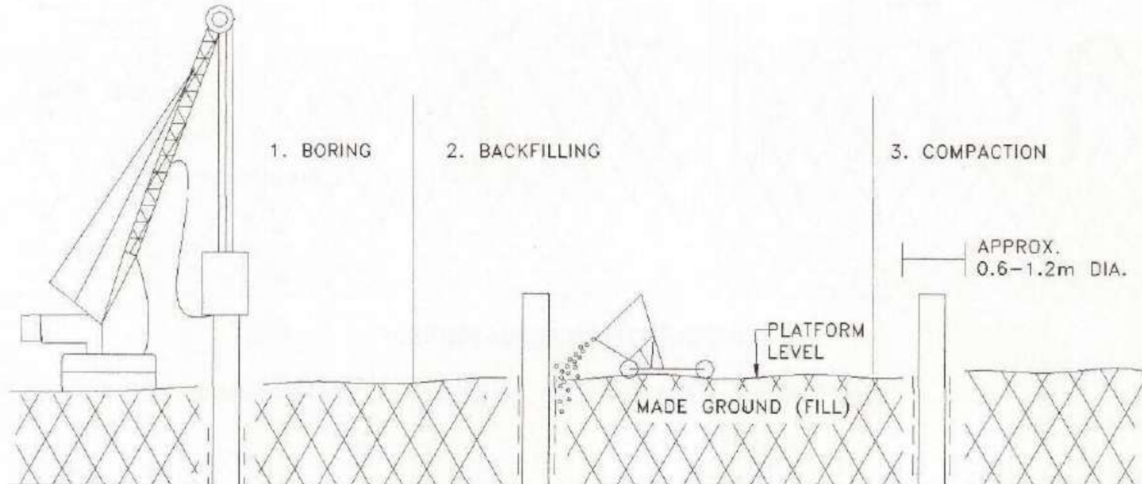


ESSENTIAL FEATURES OF VIBROFLOAT (AFTER GREENWOOD & THOMSON, 1984)

4.2.1.1 VIBRO-REPLACEMENT INSTALLATION (WET PROCESS)

The vibrator after being switched on sinks rapidly under its own weight assisted by vibration and jetting action of the water until it reaches its predetermined depth. The hole formed by the vibrator is backfilled in stages with granular materials with specified characteristics. Usually, the borehole is backfilled in 1m lifts. The vibrator is lowered again and the added material is displaced into the parent soil under the influence of its weight. The procedure of penetration and retention is repeated again until the soil cannot absorb any additional backfill or the further penetration of the vibrator is not possible.

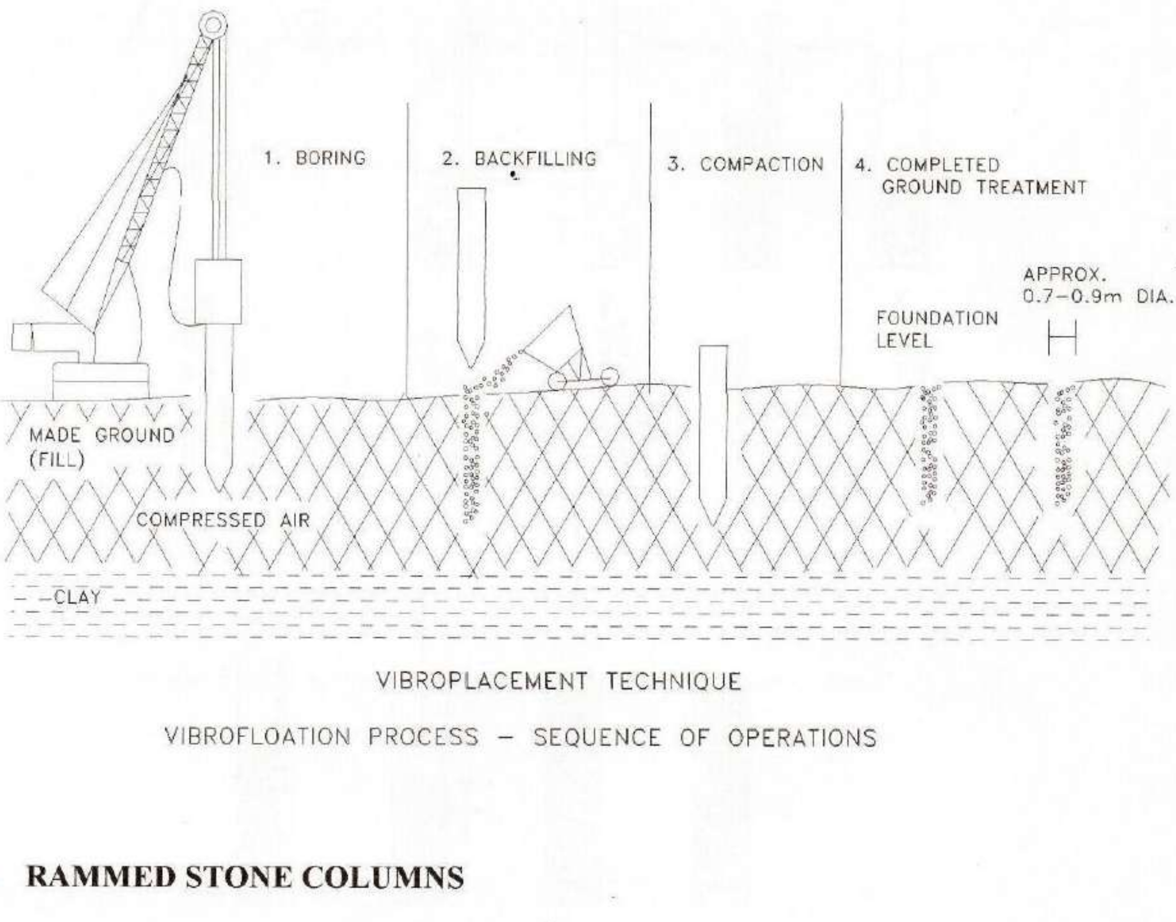
This method should be used at sites consisting of very soft soils, unsupported holes and where high ground water conditions exist. However, it requires large quantities of water which has to be disposed off later. The site tends to become slushy with stagnant water and overall production rate goes down.



VIBROPLACEMENT TECHNIQUE

4.2.1.2 VIBRO-DISPLACEMENT INSTALLATIONS (DRY PROCESS)

The vibro displacement is a dry process. The main difference between vibro-replacement and vibro displacement is the absence of jetting water during the formation of hole. Instead, compressed air is used. The compressed air generates a back pressure in the hole, thus, improving the stability of the hole.



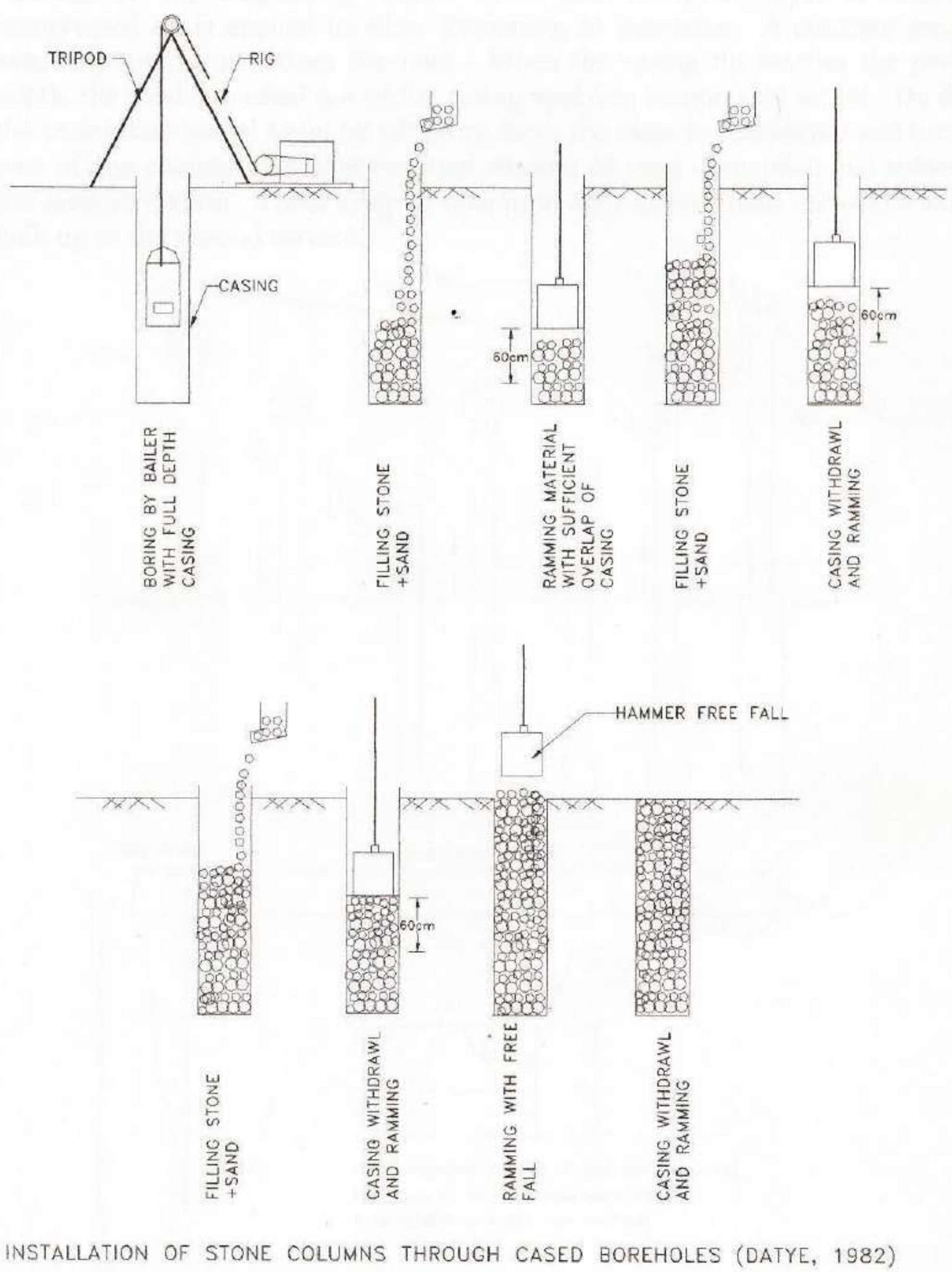
VIBROPLACEMENT TECHNIQUE

VIBROFLOATION PROCESS – SEQUENCE OF OPERATIONS

4.2.2 RAMMED STONE COLUMNS

Borehole is made with the use of bailer or auger. Driving of casing and bailer or auger boring is done alternatively to progress the borehole. The borehole with casing is done to full depth. Casing is provided with flat door windows generally at 1.0m interval. These doors are in closed positions at all times except when granular fill is to be provided , at that time that particular window is opened.

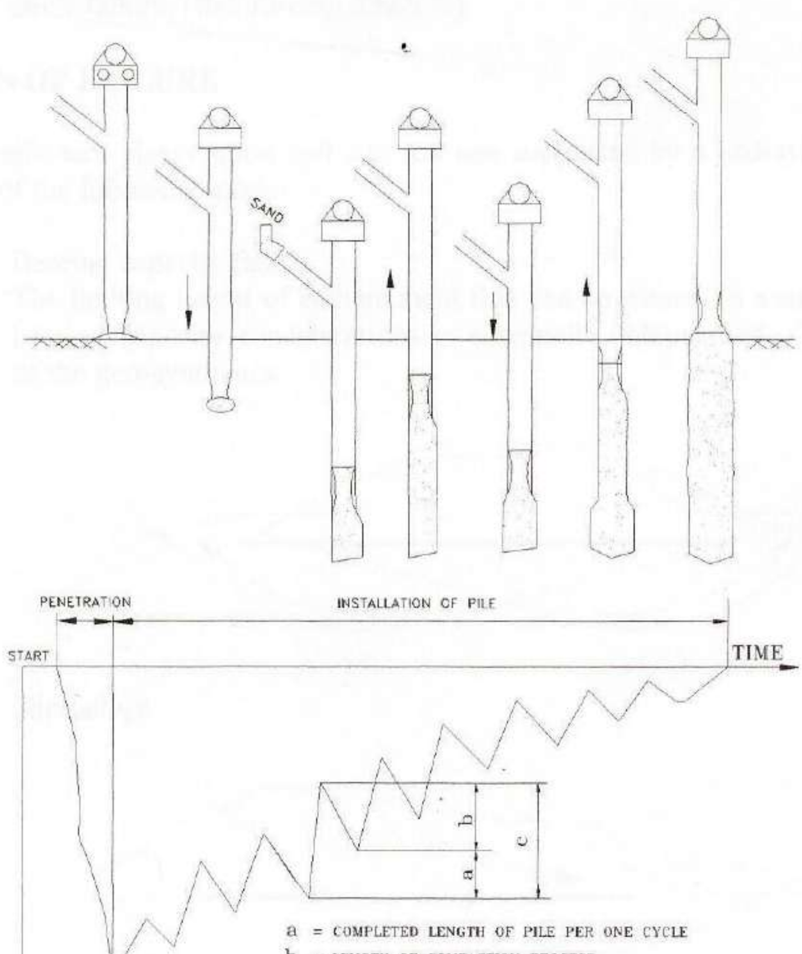
Graded coarse backfill is placed in the borehole through the window close to ground level by chute. The amount of backfill is restricted such that a column of not more than 2m height is formed at a time. After the backfill is placed, the casing is withdrawn by not more than 1.25 m. Thereafter, a hammer used for compaction falling through a height of 1.4 - 2.0 m. Next charge is placed and the casing is withdrawn further and compaction is carried out till stone column upto ground level is formed.



INSTALLATION OF STONE COLUMNS THROUGH CASED BOREHOLES (DATYE, 1982)

4.2.3 SAND COMPACTION PILES

These piles are installed by means of crane, vibrator, installation pipe and accessories, such as, sand supply, suspending wire and compressed airline. The casing is driven downward by the operation of the vibrator and a simultaneous relaxing of the suspending wire. If a thin resistance layer is encountered compressed air is applied to allow the casing to penetrate. A constant amount of sand is poured down from the inlet. When the casing tip reaches the prescribed depth, the sand is pushed out of the casing applying compressed air jet. By drawing the casing downward again by vibratory force the same is compacted and becomes a part of one column. Another constant amount of sand is supplied and subjected to the same operation. These cycle of operation are repeated until the whole column is built up to the ground surface.



METHOD OF INSTALLATION OF SAND COMPACTION PILES

5.0 GEOSYNTHETICS

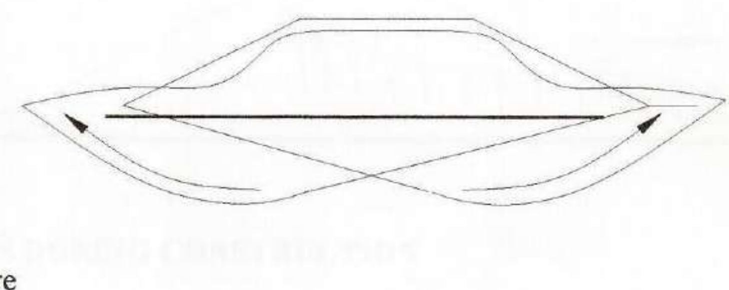
5.1 The main functions of geo-synthetics are reinforcement, filtration, drainage and separation. For construction of embankments on soft soil, problem arises due to the inadequate strength of the foundation soil on which the fill rests though embankment itself is basically stable. Geo-synthetics are employed as a horizontal reinforcement at the base of the embankment for improving the load carrying capacity of soft soil. The main functions intended are:

- To prevent mixing of the borrow fill and soft sub-soil resulting from local bearing capacity failure.(Separation function)
- To prevent excessive vertical and horizontal deformations.(Reinforcing function)
- To compensate for low shear strength of the soil that might lead to a slip circle failure. (Reinforcing function).

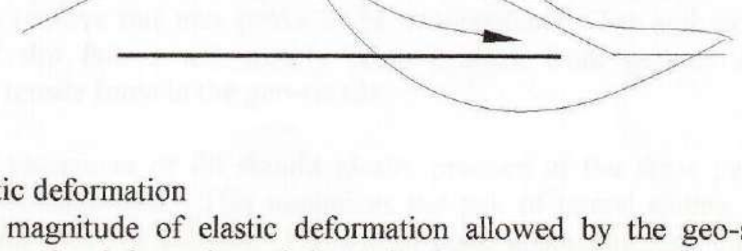
5.2 TYPES OF FAILURE

A embankment placed upon soft sub-soil and supported by a geo-synthetic may fail in any of the following modes:

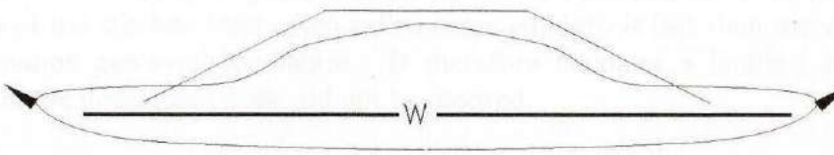
i) Bearing capacity failure
The limiting height of embankment that can be placed on a sub-soil based on bearing capacity considerations is essentially independent of the properties of the geo-synthetics.



ii) Slip failure

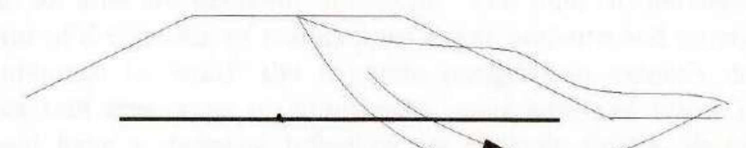


iii) Elastic deformation
The magnitude of elastic deformation allowed by the geo-synthetics will govern the deformation of the embankment. Large deformation will cause embankment racking and loss of overall stability.



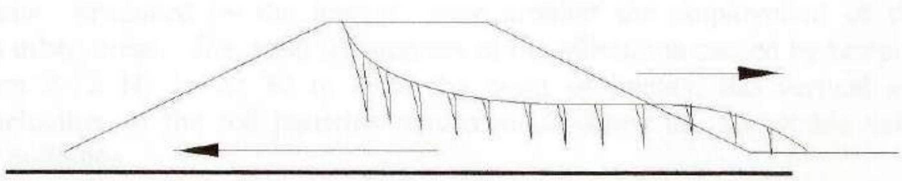
iv) Pullout failure

Sufficient length of geo-textile on each side of the failure surface should be provided to ensure a safe bond between the geo-textile and the soil.



v) Lateral spreading

During lateral spreading, tension cracks are generally observed on the surface of embankment. The tendency to lateral spreading becomes severe for steep slope angles and very smooth geo-synthetic surfaces.



5.3 PRECAUTIONS DURING CONSTRUCTION

During the construction it is essential that the geo-textile is put gradually in tension to obtain the full benefit of its strength. In case where the geo-textile remains slack or is folded beneath the embankment after construction, then the movement necessary to remove this may prove to be unacceptable. The soil movement at the initiation of slip failure will merely remove slack from geo-textile rather than mobilize the tensile force in the geo-textile.

The rate of placement of fill should ideally proceed at the same pace along each edge of the embankment. This minimizes the risk of lateral sliding of the fill and the geo-textile over the sub-soil. The use of plant producing high contact pressure should also be avoided particularly during early stages of the embankment construction.

During the construction, geo-synthetics are placed in rolls and sewn in the field. The strength of the stitched joint often called seam strength is less than the strength of the continuous geo-synthetic fabric. It therefore becomes a limiting strength requirement in the design and it should not be ignored.

6.0 DYNAMIC CONSOLIDATION

Dynamic consolidation involves repeated dropping of 10-40 tons weights on the ground surface from 10 to 40 m height following a well defined pattern as regards time and space appropriate to the site. The weights used for heavy tamping may be concrete blocks, steel plates or thick steel sheets filled with concrete or sand. Two to three coverage of an area are generally sufficient. The time lag between passes is dependent on ratio of dissipation of excess pore water pressure and strength regain. The depth of influence to which the dynamic compaction extends depends on impact energy per unit area, type of equipments, soils type and water conditions. Soft clay and peat have a damping influence on dynamic forces, therefore, they cannot be strengthened to the same level as coarser materials.

The soft soil, which is to be consolidated, is overlain by working platform of granular soil to support the weight of heavy tamping machines (60-200t). During dynamic consolidation some water rises to the surface, therefore, a preliminary requirement is to provide horizontal drains generally constructed by trenching and back filling with sand and gravel.

The vibrations produced by the impact may prohibit the employment of this technique in urban areas. The usual frequencies of the vibrations caused by tamping vary between 2-12 Hz. At 30 m from the point of impact, the vertical and horizontal velocities of the soil particles remain much below the acceptable value for ordinary buildings.

Application of this technique results in a significant reduction in void ratio of sub-soil leading to an increase in its strength and bearing capacity. The post treatment settlements are considerably reduced. Dynamic consolidation is an effective ground improvement technique for loose sands, soft clays and peats. Unless a very large site is required for treatment it would probably be not economical to use this technique. A number of theories have been proposed by various authors for this technique, however the mechanism is not yet clearly understood.

DESIGN WITH STAGE CONSTRUCTION METHOD

1.0 TERMINOLOGY

1.1 Terzaghi's theory of consolidation

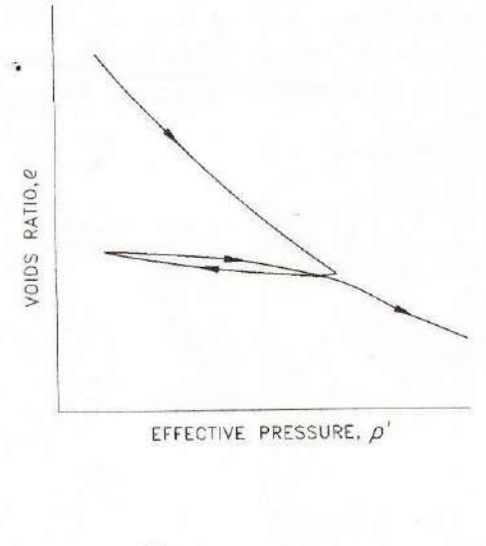
Terzaghi theory of one dimensional consolidation is used to calculate both the magnitude and rate of consolidation settlement. The main assumptions on which the theory is based are:

- The soil is saturated.
- The water and the clay particles are incompressible.
- D'Arcy's law is valid.
- For a change in void ratio corresponding to a given increment in effective stress, the permeability, k , and the coefficient of volume change, m_v, remain constant.
- The time taken for the clay to consolidate depends entirely on the permeability of the clay.
- The clay is laterally confined.
- The flow of water is one dimensional.
- Effective and total stresses are uniformly distributed over any horizontal section.

These assumptions correspond to the odometer test in the laboratory, and to a clay layer in the field subjected to uniform global loading, that is, a uniformly distributed loading applied over an infinite area. Based on above assumptions, the relationship involving excess pore water pressure, position and time is given.

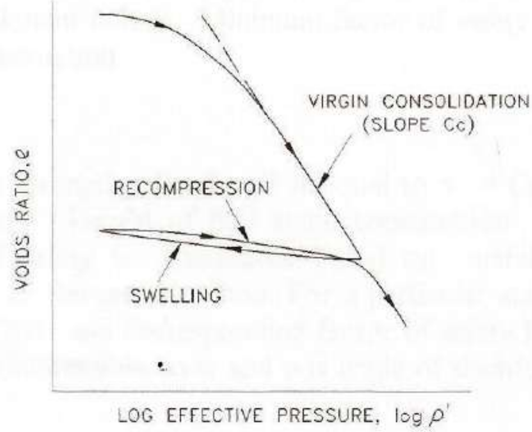
1.2 Relationship between void ratio & effective pressure

Typical relationship between void ratio and effective pressure is shown below:



1.3 Relationship between void ratio & the logarithm of effective pressure

Typical relationship between void ratio and logarithm of effective pressure is shown below:



1.4 Compression index Cc

Compression index Cc is the slope of e – log *p'* line as shown above.

1.5 Coefficient of consolidation

$$C_v = k/ m_v\gamma_w$$

Where

m_v = Coefficient of volume change, defined as the volumetric strain per unit increase in effective stress

k = Permeability of soil

γ_w = Unit weight of water = 10 kN /m^3

1.6 Pre-consolidation pressure

This is the greatest effective pressure that the soil has carried in the past. If pre-consolidation pressure is more than the over burden pressure, the clay is over consolidated and if the pre-consolidation pressure is equal to the over burden pressure, the clay is normally consolidated.

1.7 Stability number

Stability number is defined as $C_U/ \gamma H$ where C_U is un-drained shear strength of soft soil, γ is the saturated density of fill soil and H is the height of construction considered.

2.0 Design Procedure

The factor of safety against slope failure adopting normal slope 2:1 to 2.5:1 is worked out using effective shear strength parameters and pore water pressure for both sub-soil and embankment failure. Minimum factor of safety of at least 1.2 is ensured at the end of construction.

Step-1

Initially, undrained shear strength of soft soil is equal to $\tau = C_U$ where C_U is un-drained cohesion intercept. Height of first stage construction is assumed and corresponding factor of safety is calculated based on stability chart by Guy Sangler, Gelbert Olibery & Bervard Combon. For a particular stability number, this chart is a plot between D/H and corresponding factor of safety for different values of ø. D is the depth of compressible layer and ø is angle of shearing resistance of fill material.

Step-2

As a result of first stage construction, there is a increase in un-drained shear strength which is calculated as follows-

Obtain effective stress increment ($\Delta \sigma_1$) due to load of first stage of construction,

$$\Delta \sigma_1 = H_1 \times \gamma_b \times U\%$$

Where

H_1 = Height of first stage of construction

γ_b = Unit weight of fill or bank soil

U% = Percent consolidation of sub-soil due to first stage of construction

Stress implement is assumed uniform over entire depth of clay layer. Therefore, increased shear strength at the end of waiting period-

$\tau = C_U + \Delta \sigma_1 \tan ø'$

where ø' is the angle of effective shearing resistance of soft soil.

Step-3

From enhanced value of un-drained shear strength, safe height of construction can again be worked out as given in step-1. Let this height be H_2. Therefore, embankment height that can be constructed during 2nd stage is $H_2 - H_1$.

Step-4

Time required for different percentages of primary consolidation has to be worked out depending on time schedule of project. Generally, degree of consolidation is taken not less than 30%.

Waiting period between 1st and 2nd stage construction can be calculated from Terzaghi theory of consolidation-

$$t = \frac{T_v H^2}{C_v}$$

Where

t = time elapsed after load application

Tv = dimensionless time factor depending upon degree of consolidation

Cv = coeff. of consolidation as determined from laboratory test

H = length of drainage path, equal to thickness of clay layer in case of single drainage path and half of thickness in case of double drainage path

It has been confirmed from experience based on various case histories that settlement in field occurs at much faster rate nearly 8 to 10 times than the rate predicted from Terzaghi's one dimensional consolidation theory. This factor should be accounted for.

Step-5

If waiting period as calculated from step 4 does not fit in with overall time schedule of project, step 2 to 4 can be repeated taking different value of U% for calculating revised waiting period.

Step-6

The above procedure can be repeated for working out height of subsequent stages.

6.0 Settlement Computation

Procedure of computing total settlement for the embankment is as follows.

Step-1

Plot complete bore log indicating type of soils at various depths with classification, index properties of each stratum, natural moisture content, consolidation test data (duly corrected), UCC and other test results for clays. Results of other field tests done, such as un-drained modulus, over consolidated ratio, Skempton's pore pressure parameters A and B etc. should be tabulated.

Step-2

Decide the size of proposed foundation and loads coming on it. Work out increment of stresses beneath the foundation in each layer, at its centre, using new mark's chart/ Osterberg's chart or any other suitable means.

Step-3

Work out elastic settlement from :

$$Se = p \cdot b \, \frac{1-\mu^2}{E} \, I$$

Where,

Se = elastic settlement

p = net foundation pressure

b = width of loaded area

μ = poissons ratio (0.5 for undrained loading)

E = Young's Modulus

I = influence value which depends on shape of loaded area and depth of deposit.

I, the influence value for embankment loading can be found from the tables given by Jean Piere Giround and Andre Rabatel in their paper on " Settlement of Embankment on Layer of Soil". Practical determination of settlement W under symmetrical trapezoidal embankment has been worked in this paper as –

$$W = \frac{pa^2}{E(a - a')} \left[\gamma_H - (a'/a)^2 \, \gamma'_H\right]$$

Where

p = max. normal contact stress exerted by embankment on ground surface

E = Young's Modulus of the soil bearing embankment

a = half width of the lower part of embankment

a' = half width of the upper part of embankment

γ_H = Dimensionless coefficient related to H /a, H being thickness of soil layer.

γ'_H= Dimensionless coefficient related to H/a'.

Elastic settlement is very small. For all practical purposes, this can be neglected.

Step-4

Calculate primary consolidation settlement from Oedometer test results

$$S_{oed} = \frac{Cc}{1+eo} \, H \log_{10} \frac{p_0 + \Delta p}{p_0}$$

The value of Cc is taken from the laboratory tests.

Step-5

Apply correction to S_{oed} by multiplying by λ as given by Skempton and Bjerrum reproduced in IS 8009 – 1976(Pt. I) " Code of Practice for Calculation of Settlement of Foundations (Pt.I- Shallow foundations)".
Consolidation settlement $= \lambda \times S_{oed}$

Step-6

Calculate secondary consolidation as below:

(A) For over consolidated clays

Case 1 $\Delta P > (Pc - Po)$

$$Ss = \frac{Cc}{1+e_o} H \log_{10} \frac{Pc}{P_1}$$

Where Pc/P_1 = Pc/Po of normally consolidated clays

$P_1 = Pc/(Pc/Po)$ of normally consolidated clays

Case-2 $\Delta P < (Pc - Po)$

$$Ss = \frac{Cc}{1+e_o} H \log_{10} \frac{Po + \Delta P}{P_1}$$

(B) For normally consolidated clays:

Case-I $\Delta P > (Pc - Po)$

$$Ss = \frac{Cc}{1+e_o} H \log_{10} \frac{Pc}{Po}$$

Case-2 $\Delta P < (Pc - Po)$

$$Ss = \frac{Cc}{1+e_o} H \log_{10} \frac{Po + \Delta P}{Po}$$

Ss = Secondary settlement
ΔP = Increase in pressure due to embankment loading
Pc = Pre-consolidation pressure
Po = Effective over burden pressure

Cc = Compression index
H = Thickness of layer.

Step-8

Total expected settlement = $Si + \lambda\ S_{oed} + Ss$

3.0 Soil Data Required

Following data is needed for the scientific designing of embankment in such area :

- Sub soil profile along the proposed alignment upto a depth of about 2 to 3 times the height of the bank.
- Shear strength parameter for slope stability analysis i.e. C', Ǿ, Skempton' s pore water pressure parameters 'B' & 'A'.
- Consolidation properties viz. compression index Cc for determining of amount of settlements and coefficient of consolidation Cv for predicting the rate of occurrence of these settlements.

4.0 Soil Exploration

For ascertaining above design parameters, test boring at every 100 metre along the center line of the alignment is necessary.

Samples in any case, should be collected at every 1.5m depth and note should be made at every change of strata to enable exact logging for every bore hole.

This must be done before construction of embankment commences. It is assessed that only minor alternations in the allowances for settlement would be required after revisions on the basis of above exploration. This can be taken care of before construction commences.

5.0 Stage Construction control chart

5.1 Computation of Average Pore Pressure Ratio ($\bar{B}$)

Following procedure is adopted for computing the average pore pressure ratio ($\bar{B}$)-

i) Compute excess pore water pressure Δu at each piezometer tip due to embankment loading at different depths. Static water pressure should be deducted from the observed pore water pressure.

ii) Draw contours of excess pore water pressure of sub soil in the zones of critical slip circle assuming pore pressure is zero at ground level due to presence of sand layer.

iii) The wedge above the critical slip circle of the constructed bank profile is divided into a number of strips. The number of strips are kept about fifteen for accuracy. The width and average height of each soil strip above ground level is measured and recorded.

iv) The increase in major principal stress ($\Delta\sigma'_1$) due to embankment loading of each strip is calculated. For practical purpose, (σ'_1) is taken as weight of fill material above ground level which is equal to Υ. h for unit thickness of the section, where Υ is the bulk unit weight of fill material and h is the average height of strip.

v) Excess pore pressure or excess head of water (Δu) on the slip plane at the point of action of $\Delta\sigma'_1$ is calculated for each strip at its centre by interpolation from pressure contours.

vi) Pore pressure ratio ($\bar{B}$) for each strip is calculated by dividing the excess pore water pressure (Δu) by $\Upsilon.h$ ($= \Delta\sigma'$) for each strip.

vii) The average pore pressure ratio ($\bar{B}$) of each strip is found by multiplying the $\bar{B}$ with the unit width of the strip.

viii) The weighted average of pore pressure ratio of sub-soil is then calculated by dividing the total average pore pressure ratio along the slip circle by length of slip circle. The length of slip circle is taken as the total length of all the strip (ΣL).

5.2 For quick evaluation at any stage of construction, a control chart of average pore pressure ratio ($\bar{B}$) of sub soil vs factor of safety for different heights is prepared before starting the construction. For preparing control chart, the factor of safety for a particular height of construction with different values of average pore pressure ratio ($\bar{B}$) for the sub soil is calculated and plotted. A typical calculation chart for computing $\bar{B}$, showing design profile of bank, various stages of construction with critical slip circle is attached as Annexure-V.

For evaluating the factor of safety at any stage in the field, the pore pressure ratio ($\bar{B}$) is computed from the observed pore pressure data and factor of safety corresponding to that ($\bar{B}$) and height of construction is immediately read off from the control chart. For intermediate heights not given in the chart, a linear interpolation is done as the relation between ($\bar{B}$) and factor of safety is linear.

SOLVED EXAMPLE

Project:-Construction of Railway formation on soft marine clay in Seawood (Belapur –Uran) MTP Railway Mumbai

1.0 PROBLEM STATEMENT

CE / MTP, Mumbai vide letter no. MT/W/254 dated 3.4.1998 had requested RDSO to suggest ways and means by which the formation for B.G.double line can be constructed between Seawood (Belapur)-Uran over soft marine clay. The proposed alignment is passing through low lying marshy area mostly affected by sea water during high tide. The marine deposit in Mumbai area is mainly confined to narrow belt of tidal flats all along the coast. The deposits have formed from the deposition of sediments brought down by the rain water streams flowing down the Ghats. Soft marine deposits thickness varies upto 16 m.

Chainage	Depth of clay GL	Maximum height of bank from GL
0595 to 02720	4 m	22.21 m
4005 to 05500	2 m	8.03 m
11473 to 13500	4 m	11.63 m
13500 to 21760	7 to 16 m	4.65 m

Ground water table varies from 0.45 m to 2 m depth below GL. GE Lab./RDSO had carried out testing on six U/D samples collected from alignment and 2 from borrow soil .

2.0 RESULTS OF LAB TESTING

The soil is CH, MH type which was highly compressible one. Natural moisture content varied between 46% to 70% and NDD was about 1.0 gm/cc and liquid limit varied between 51% to 72%.Coefficient of consolidation Cv was determined from consolidation test as 0.03 cm^2/min.The results signified that soil was soft compressible in nature. Significant settlement was expected during the construction of embankment. This condition warranted special precautions to be taken to avoid failure during construction. RDSO had suggested stage construction method for construction of embankment. For ch. 13500 to 21760, sample calculation for stage construction are given below.

3.0 CALCULATIONS

3.1 SETTLEMENT TIME FOR VARIOUS DEGREES OF CONSOLIDATION

Average depth of compressible clay layer = 9.5 m

Single drainage has been considered as the bottom layer consists of gravel & weathered rock.

$t \quad = \quad T_v.H^2/C_v$

Where

T_v = Dimensionless time factor depending upon degree of consolidation (values taken from standard table given as Annexure-II)

$H \quad = \quad$ Drainage path
$\quad = \quad$ 9.5 m (for single drainage)
$\quad = \quad$ 950 cm

$C_v \quad = \quad$ Coefficient of consolidation
$\quad = \quad$ 0.03 cm²/min (as per consolidation test data)

(i) Time required for 20% consolidation.

$T_{20} \quad = \quad 0.031$

$t \quad = \quad$ 0.031x 950x 950/0.03 minute

$\quad = \quad$ 0.031x 950x 950/0.03x60x24x365 year

$\quad = \quad$ 1.77 year

Considering actual rate of settlement 5 times faster than theoretical from experience

Actual time required = t/5

= 0.354 year

= 4.25 month

Considering actual rate of settlement 8 times faster than theoretical from experience

Actual time required = t/8

= 1.77/8 year

= 2.65 month

(ii) Time required for 30% consolidation

$T_{30} \quad = \quad 0.071$

$t \quad = \quad$ 0.071x 950x 950/0.03 minute

$t \quad = \quad$ .071x950x950 / 0.03x60x24x365 year

$\quad = \quad$ 4.06 year

Considering 5 times faster rate of settlement

Actual time required

= t/5

= 4.06/5

= 0.81 year

= 9.75 month

Considering 8 times faster rate, actual time required

t = 4.06/8 year

= 6.09 month

(iii) Time required for 40% consolidation

T_{40} = 0.1261

t = 0.126x 950x 950/0.03 minute

t = 0.126x950x950 / 0.03x60x24x365 year

= 7.21 year

Considering 5 times faster rate of settlement

Actual time required

= t/5

= 7.21/5

= 1.44 year

Considering 8 times faster rate, actual time required

t = 7.21/8 year

= 10.81 month

(iv) Time required for 60% consolidation

T_{60} = 0.287

t = 0.287x 950x 950/0.03 minute

t = 0.287x950x950 / 0.03x60x24x365 year

= 16.43 year

Considering 5 times faster rate of settlement

Actual time required

= t/5

= 16.43/5

= 3.28 year

Considering 8 times faster rate, actual time required

t = 16.43/8 year

= 24.64 month

(v) Time required for 90% consolidation

T_{90} = 0.848

t = 0..848x 950x 950/0.03 minute

t = .0848x950x950 / 0.03x60x24x365 year

= 48.53 year

Considering 5 times faster rate of settlement .

Actual time required

= t/5

= 48.53/5

= 9.706 year

Considering 8 times faster rate, actual time required

t = 48.53/8 year

= 72.79 month

3.2 COMPUTATION OF DEGREE OF CONSOLIDATION

i) Consider waiting period of 20 month
Considering 8 times faster rate consolidation

i.e. 1.67 year

t = 1.67 year

t = $T_v H^2/C_v$

T_v = t x C_v/H^2

= 8x1.67x60x24x365x0.03/950x950

= 0,029 x 8

= 0.232

Consolidation corresponding to this T_v = 54.27%

ii) Consider waiting period of 12 month i.e. 1 year
Considering 8 times faster rate consolidation

t = $T_v H^2/C_v$

T_v = t x C_v/H^2

= 8x1x60x24x365x0.03/950x950

= 0.14

Consolidation corresponding to this T_v = 42.12 %

iii) Consider waiting period of 6 month i.e. 0.5 year
Considering 8 times faster rate consolidation

t = $T_v H^2/C_v$

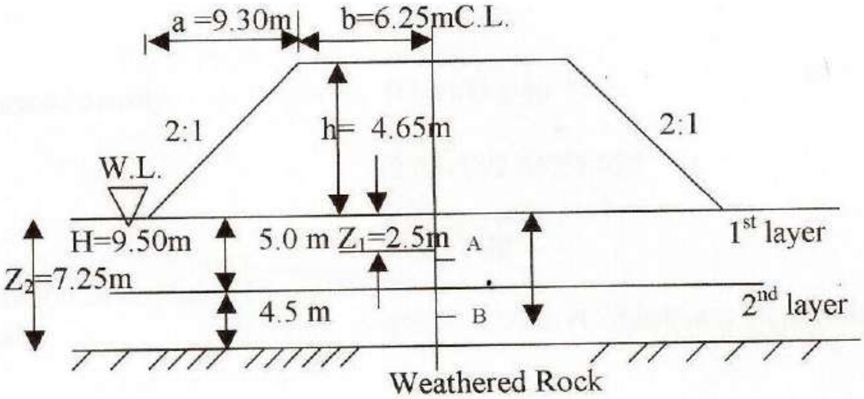
T_v = t x C_v/H^2

= 8x0.5x60x24x365x0.03/950x950

= 0.07

Consolidation corresponding to this T_v = 29.77 %
= 30% (say)

3.3 COMPUTATION OF SETTLEMENT



Symbol used:

h	=	Height of bank
H	=	Depth of compressible clay layer
a	=	Horizontal distance between toe of the bank and end point of the berm / cess.
b	=	Horizontal distance between center of bank and end point of the berm/cess
Z_1	=	Vertical distance between bottom of embankment and mid point of first compressible layer
Z_2	=	Vertical distance between bottom of embankment and mid point of second compressible layer.
I_1	=	Influence value of half embankment loading due to first compressible layer (from Osterberg chart)
I_2	=	Influence value of half embankment loading due to second compressible layer (from Osterberg chart)
I	=	Influence value of full embankment loading
G	=	Specific gravity of soil
γ_d	=	Dry density of soil
e_0	=	Initial void ratio
C_c	=	Compression Index
P_c	=	Pre-consolidation pressure

Bank profile

h	=	4.65 m
Top formation width	=	12.50 m
Slope	=	2 H : 1 V
Bank Soil- MDD	=	1.658 gm/cc
		98% of MDD
	=	1.625 gm/cc
Saturated density	=	$(G-1)/G \times \gamma_d + 1$
	=	$(2.65-1)/2.65 \times 1.625 + 1$
	=	2.00 gm/cc
	=	2.00 t / m^3
Ch 20500 Bore Hole –1 Depth	=	4.50 – 5.00 m (Consultancy Report of MTP, Mumbai)

Base Soil

γ_d	=	1.00 gm/cc,
G	=	2.63
e_0	=	1.63
Cc	=	0.86
Pc	=	0.90 Kg/Cm^2
	=	9.0 t/m^2
γ_{sub}	=	$(G-1)x\gamma_d$ /G
	=	1.63x1.00/2.63
	=	0.61 gm/cc
	=	0.6 t/m^3

Effective over burden pressure at A (Depth = 2.5 m below GL)

$\overline{P}_0$ at A	=	250 x 0.61
	=	152.5 gm/cm^2
	=	0.1525 kg/cm^2
	=	1.525 t/m^2

Effective over burden pressure at B (Depth = 7.25 m below GL)

$\overline{P}_0$ at B	=	725 x 0.61
	=	442.25 gm/cm^2
	=	0.442 kg/cm^2
	=	4.42 t/m^2

To find influence of embankment loading at point A,
(Centre point of 1st layer of clay)
(Ref: Osterbergs influence chart Annexure-III)

a	=	9.30 m
b	=	6.25 m
Z	=	2.50 m
a / Z	=	9.30/2.50
	=	3.72
b / Z	=	6.25/2.50
	=	2.50
I_1	=	0.495

Due to symmetrical embankment loading

I	=	2 x I_1
	=	2x 0.495
	=	0.99

Increment of over burden pressure due to embankment loading at point 'A'

Δ P at A	=	0.99 x 4.65 x 2.00
	=	9.207 t/m²

To find influence of embankment loading at point-B
(Centre point of 2nd layer of clay)

a	=	9.30 m
b	=	6.25 m
Z	=	7.25 m
a / Z	=	9.30/7.25
	=	1.28
b / Z	=	6.25/7.25
	=	0.86
I_1	=	0.445

Due to symmetrical embankment loading

I	=	2 x I_1
	=	2x 0.445
	=	0.89

Increment of over burden pressure due to embankment loading at point 'B'

Δ P at B	=	0.89 x 4.65 x 2.00
	=	8.28 t/m²

Computation of primary settlement 1st layer P_0 at

A	=	1.525 t/m²
Δ P	=	9.207 t/m²
$P_0 + \Delta p$	=	.525 + 9.207
	=	10.732 t/m²
P_C	=	9.00 t/m²

Settlement would be due to loading upto Pc and there after from Pc to ($P_0 + \Delta p$)

Settlement upto P_C

i) $S_1 = \dfrac{\Delta e}{1+e_0} \times H$

$= \dfrac{1.63-1.45}{1 + 1.63} \times 5.00$

$= 0.342$ m

ii) Settlement from P_C to ($P_0 + \Delta p$)

Settlement $S_2 = \dfrac{Cc}{1+e_0} \times H \times \log_{10}\dfrac{P_0 + \Delta p}{Pc}$

$= \dfrac{0.86}{1 + 1.63} \times 5.0 \times \log_{10}\dfrac{10.732}{9.00}$

$= 0.125$ m

Total settlement of 1st layer

$S_{oed} = S_1 + S_2$

$= (0.342 + 0.125) = 0.467$ m

Total settlement of 2nd layer

P_0 at B $= 4.42$ t/m²

$\Delta p = 8.280$ t/m²

$(P_0 + \Delta p) = 4.42 + 8.28$

$= 12.70$ t/m²

Settlement upto P_C

$S_1 = \dfrac{\Delta e}{1+e_0} \times H = \dfrac{1.55-1.450}{1 + 1.63} \times 4.5 = 0.171$ m

Settlement from P_C to ($P_0 + \Delta p$)

Settlement $S_2 = \dfrac{0.86}{1+1.63} \times 4.5 \times \log_{10}\dfrac{12.70}{9.00} = 0.22$ m

Total settlement of 2nd layer

S_{oed}	=	$S_1 + S_2$
	=	$(0.171 + 0.220)$
	=	0.391 m

Total primary settlement	=	0.467 + 0.391
For full depth of clay	=	0.858 m
Corrected primary consolidated		
Settlement Sc	=	$S_{oed} \times \lambda$
	=	0.858 x 0.85
	=	0.729 m
	=	73 cm

($\lambda = 0.85$ considered as the soil is normally consolidated)

3.4 ANALYSIS OF EMBANKMENT STABILITY DURING VARIOUS PHASES

(Ref: Guy Sanglert etc. Graph, Annexure-IV)

3.4.1

Height of bank h	=	4.65 m
Depth of compressible clay layer (H)	=	9.50 m
Bank soil γ_{sat}	=	2.00 t/m³
Bank soil Ø	=	33°
Bank soil C	=	0

Sub soil γ_d	=	1.00t/m³
Effective angle of shearing resistance Ø	=	24°
Undrained shear strength, C_u	=	0.13 kg/cm²
	=	1.30 t/m³

3.4.2 Stage I

1st Phase height of construction considered = 3.0 m

D/H	=	9.50/3
	=	3.16
N	=	$C/\gamma.H$
	=	0.3 / 2.0 x 3.0
	=	0.217

From stability chart ;

For N	=	0.2
Ø	=	33°
D/H	=	3.16
FOS	=	1.075

For N	=	0.3
Ø	=	33°
D/H	=	3.16
FOS	=	1.590

Interpolating for N = 0.217

$$F = 1.075 + \frac{1.59 - 1.075}{0.1} \times 0.17$$

$$= 1.16 < 1.20 \text{ (which is less than 1.20)}$$

Hence, constructing 3 m height during stage I is not safe. Therefore, height of stage I is to be reduced.

Height of construction during 2nd trial phase considered = 2.5 m

$$D/H = 9.50/2.50$$
$$= 3.80$$
$$N = C/\gamma.H$$
$$= 1.3 / 2.0 \times 2.5$$
$$= 0.26$$

From stability chart

For N	=	0.2
Ø	=	33°
D/H	=	3.80
FOS	=	1.075
For N	=	0.3
Ø	=	33°
D/H	=	3.80
FOS	=	1.590

Interpolating for N = 0.26

$$F = 1.075 + \frac{1.59 - 1.075}{0.1} \times 0.06$$

$$= 1.384 > 1.20 \text{ (which is greater than 1.20)}$$

Hence during stage I, 2.5 m height can be constructed.

Stage-II

Allowing 1st waiting period of about 6 month, there would be an average consolidation of 30% in compressible clay layer due to 1st stage of construction

(2.5 m height).

The effective stress increment due to the load of 1st stage of construction. (2.5m height)

$\Delta\sigma_1'$ = 2.50 x 2.00 x 0.3
= 1.5 t/m²

Assuming that the stress increment is uniform over the entire clay layer including where failure circle could occur.

The increase in un-drained cohesion would be,

ΔC_u = $\Delta\sigma_1'$ Tan 24°
= 1.5 x Tan 24°
= 1.5 x 0.4452
= 0.67 t/m²

At the end of 6 month, we may consider

C_u = 1.30 + 0.67
= 1.97 t/m²

2nd Phase height of construction considered = 1.50 m

Now total height H = 2.50 + 1.50 m
= 4.00 m

D/H = 9.5/4.0
= 2.375

N = $C/\gamma H$

$$= \frac{1.97}{2.0 \times 4.0}$$

= 0.246

From stability chart

For N = 0.2
Ø = 33°
D/H = 2.375
FOS = 1.075

For N = 0.3
Ø = 33°
D/H = 2.375
FOS = 1.590

Interpolating for N $=$ 0.246

$$F = 1.075 + \frac{1.59 - 1.075}{0.1} \times 0.046$$

$= 1.31 > 1.20$ (which is greater than 1.20)

FOS $=$ 1.20 may be considered in short term stability during construction.

Stage III

After allowing 2nd waiting period of 6 month, 1st stage of construction (2.5 m height) will under go 42.12% consolidation and for 2nd lift (1.50 m), the subsoil will under go 30% consolidation.

i) Stress increment due to 1st lift (2.50 m)

$\Delta\sigma_1' = 2.50 \times 2.00 \times 0.421$

$= 2.10\ t/m^2$

Increase in undrained share strength due to 1st lift $=$ 2.10 Tan 24°

$= 0.93\ t/m^2$

ii) Stress increment due to 2nd lift (1.50 m)

$\Delta\sigma_2' = 1.50 \times 2.00 \times 0.3$

$= 0.9\ t/m^2$

Increase in undrained shear strength due to 2nd lift $=$ 0.9 Tan 24°

$= 0.40\ t/m^2$

At the end of 2nd waiting period we may consider,

Cu $= 1.30 + 0.93 + 0.40$

$= 2.63\ t/m^2$

Height of embankment considered during 3rd stage of construction

$= 0.65m$

Total height H $= 4.65$ m

D/H $= 9.5/4.65$

$= 2.04$

$$N = C/\gamma H = \frac{2.63}{2.0 \times 4.65}$$

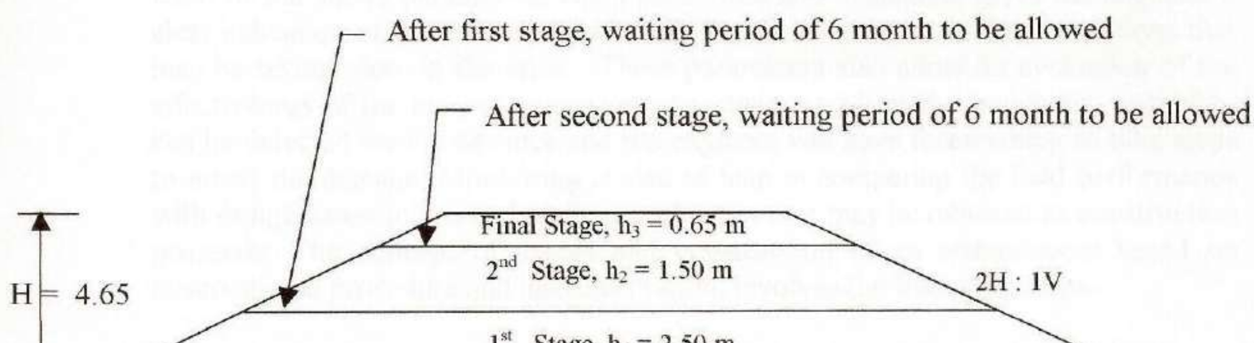
$= 0.283$

From stability chart

For N $=$ 0.2

Ø	=	33°
D/H	=	2.04
FOS	=	1.075
For N	=	0.3
Ø	=	33°
D/H	=	2.04
FOS	=	1.590
Interpolating for N	=	0.283
F	=	$1.075 + \dfrac{1.59 - 1.075}{0.1} \times 0.083$
	=	$1.50 > 1.20$ (which is greater than 1.20),safe

In conclusion, planning for embankment construction for 4.65 m height can be shown as follows-



INSTRUMENTATION AND MONITORING

1.0 Embankments on soft clay deposits are built with a low initial factor of safety for reasons of economy and also because the long term factor of safety is anticipated to be higher than the initial factor of safety, due to the gain in strength of the soft clay on account of consolidation under the embankment load. To ensure that the actual ground response closely corresponds to the one assumed, the following parameters have to be closely monitored in the field:

- Build up and dissipation of pore pressure
- Rate and magnitude of the vertical settlements of the sub-soil under the applied embankment loads
- Horizontal spreading, if any, of the sub-soil under the applied load

Each of the above parameters, when monitored and evaluated, gives the engineer a clear indication of the state of stability of the embankment and of any variations that may be taking place in the same. These parameters also allow an evaluation of the effectiveness of the ground improvement techniques adopted. Impending instability can be detected well in advance and the engineer will have forewarning to take steps to arrest the damage. Monitoring is also of help in comparing the field performance with design assumptions and make any changes that may be required as construction proceeds. The concept of design and construction of an embankment based on observational procedure and instrumentation, involves the following steps-

- Design of the embankment with the help of available field and laboratory data concerning soil parameters
- Prediction of the response of the embankment and sub-soil, based on the design assumptions and soil parameters. The response is normally monitored through the monitoring of critical parameters, such as, pore water pressures, settlements, etc.
- The predicted and actual performance are compared and evaluated regularly during the construction. This enables the designer to take any corrective action that may be required, well in time, before any failure is likely to occur. The rate of placement of fill can be regulated accordingly.

The observational procedure is an indispensable phase in the design and construction of embankment. It comes after conducting the site investigation and numerically predicting the various critical parameters at various intervals of construction on the basis of collected field and laboratory data. The monitoring of construction is done by using instrumentation to measure the critical parameter, namely; pore pressure generation and dissipation, vertical and lateral displacement.

2. Instrumentation

Monitoring the rate of settlement, the dissipation of pore pressure and the progress of consolidation can be achieved by the use of following instruments:

- Settlement gauge
- Piezometer
- Inclinometer

3. Settlement gauge

Changes in the height of embankment and heaving at the areas close to the toe are normally measured with surveying instruments. For this purpose, settlement gauges are installed in the foundation soil. Settlement gauges consist of a steel base plate and string of rods at rightly angles to the plate. The plate is set horizontally on the natural soil surface after removing the top soil and back filling with properly tamped sand. Initially, only one rod is attached to the plate, but as the preload fill is built-up, more rods are added. The plate allows determination of the total settlement under it by simple levelling. Various types of settlement gauges commercially available are as follows:

- Platform type settlement gauge
- Liquid level gauge
- Magnetic heave/settlement gauge

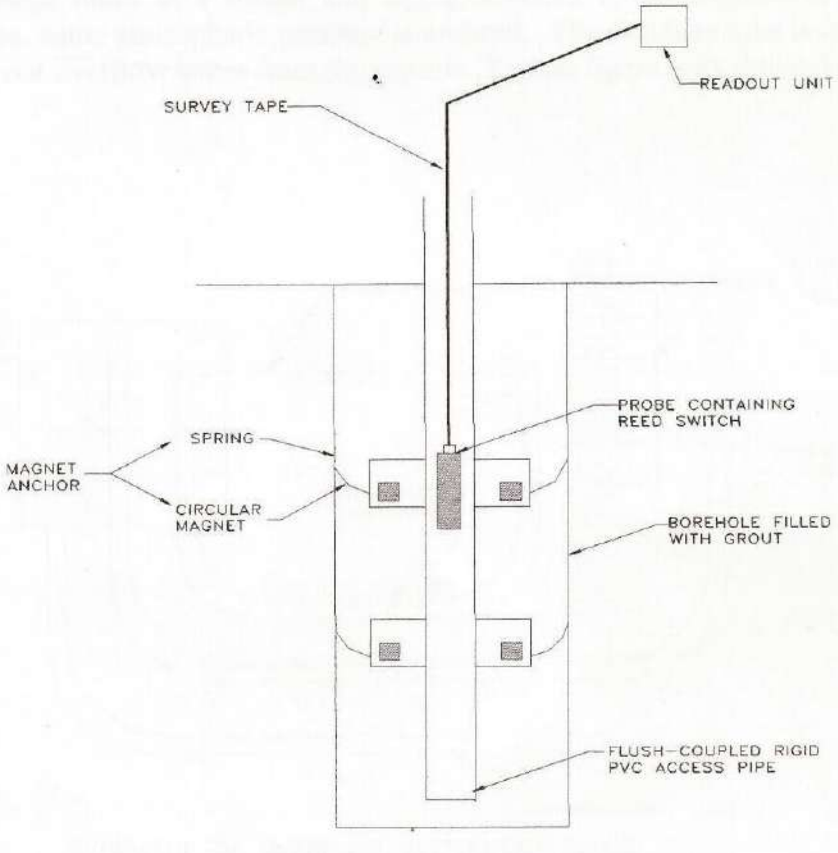
3.1 Platform type settlement gauge

A platform type settlement gauge is relatively simple device for monitoring settlements by taking levels at the top of the platform at suitable interval of time. Their use is justified in that their fabrication is easy, economical and their monitoring is relatively simple. These are installed at the ground level. In order to overcome the effect of skin friction, the stem above the platform level is encases in a freely moving casing pipe. A typical sketch of settlement gauge is shown in Annexure-VI.

3.2 Magnetic settlement gauge

Magnetic settlement gauge works on the principle that a sensor gets activated when it enters a magnetic field axially and can be made to emit a signal at the ground level. The magnetic ring consists of four arc shaped magnets fixed to four sides of a Perspex ring. Each magnetic ring has three upward leaf springs mounted at intervals of 120° around the ring. The probe is a brass cylindrical cone with a tapered end and houses a reed switch. The reed switch consists of two ferrous electrodes typical hermetically sealed in a glass tube with a small separation between them. The leads of the reed switch are connected with the help of a graduated wire to a control box housing an electronic circuit. When the circuit is completed a sound signal is generated.

The installation of magnetic settlement gauge is done by first making a borehole in the sub-soil. The borehole is usually cased and is filled with clay grout. A non-ferrous access pipe generally of PVC is greased and inserted. The casing is raised to a level where magnetic ring has to be installed. The magnetic ring is pushed downward over the access pipe until the leaf springs snap out of the casing bottom and bite into the surrounding in-situ soil. The procedure is repeated until all the planned magnetic rings are installed and the casing is then withdrawn. A schematic sketch of magnetic rings and reed switch installed in the borehole is shown in figure below-

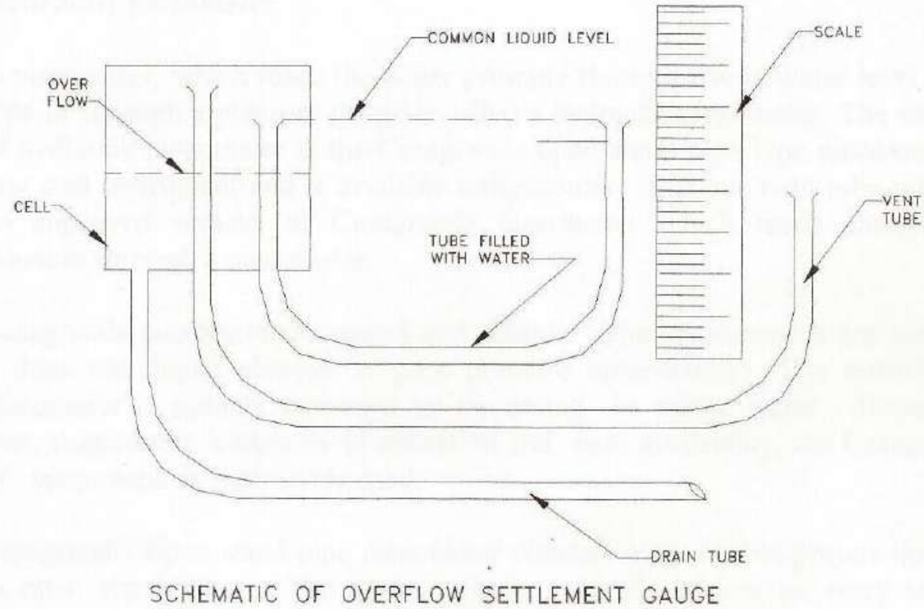


SCHEMATIC OF PROBE EXTENSOMETER WITH MAGNET/REED SWITCH TRANSDUCER IN A BOREHOLE

While the installation is relatively simple, there is a possibility of the magnetic rings getting grouted with the access pipe, thereby inhibiting the utility of the magnetic rings. The method, therefore, may be used only for monitoring small vertical displacements.

3.3 Water over flow settlement gauge/liquid level gauge

To achieve a better degree of accuracy, the possible alternative to settlement gauge is water overflow type settlement gauge. The instrument is available indigenously. This gauge is based upon the principle that any two inter-connected, water filled chambers will always exhibit the same water level in them, as long as, the pressure on the water columns in the two chambers remain same. In the water overflow type settlement gauge, the two interconnected chambers assume the form of a large U tube, one limb of which is constituted by water level stand pipe in the settlement capsule and the other by measuring stand pipe in the gauge box. The connection between the two limbs is provided by a water level tube. By the provision of an air tube emanating from the tip of capsule chamber, laid along with the water level and the drainage tubes in a trench and finally exposed to atmosphere in the gauge box/house, same atmospheric pressure is ensured. The drainage tube is intended for draining out overflow water from the capsule. Typical figure is as shown below-



SCHEMATIC OF OVERFLOW SETTLEMENT GAUGE

4. Piezometer

A piezometer is an instrument for measuring pressure in the pore water. Piezometers are installed at critical or representative locations, within the subsurface in order to follow & monitor, first the development and later the dissipation of pore water pressure. Knowledge of the magnitude of pore water pressure helps to determine the progress of consolidation and also the possibility of a base failure.

The most common type of piezometer is simply an open stand pipe that is installed inside a bore. The space between the stand pipe and the soil is backfilled with impervious material, such as, clay and a short section (about 1 m in length) is filled with a pervious medium such as coarse sand. If both clay and sand layers are present, the measured value may indicate the water pressure in the sand rather than in the clay. The sand, because of its greater permeability, is likely to follow more readily the changes of head in the standpipe.

An ideal piezometer is one which is reliable, sensitive, rugged and easy to operate. A large variety of piezometers are commercially available and are classified as follows:

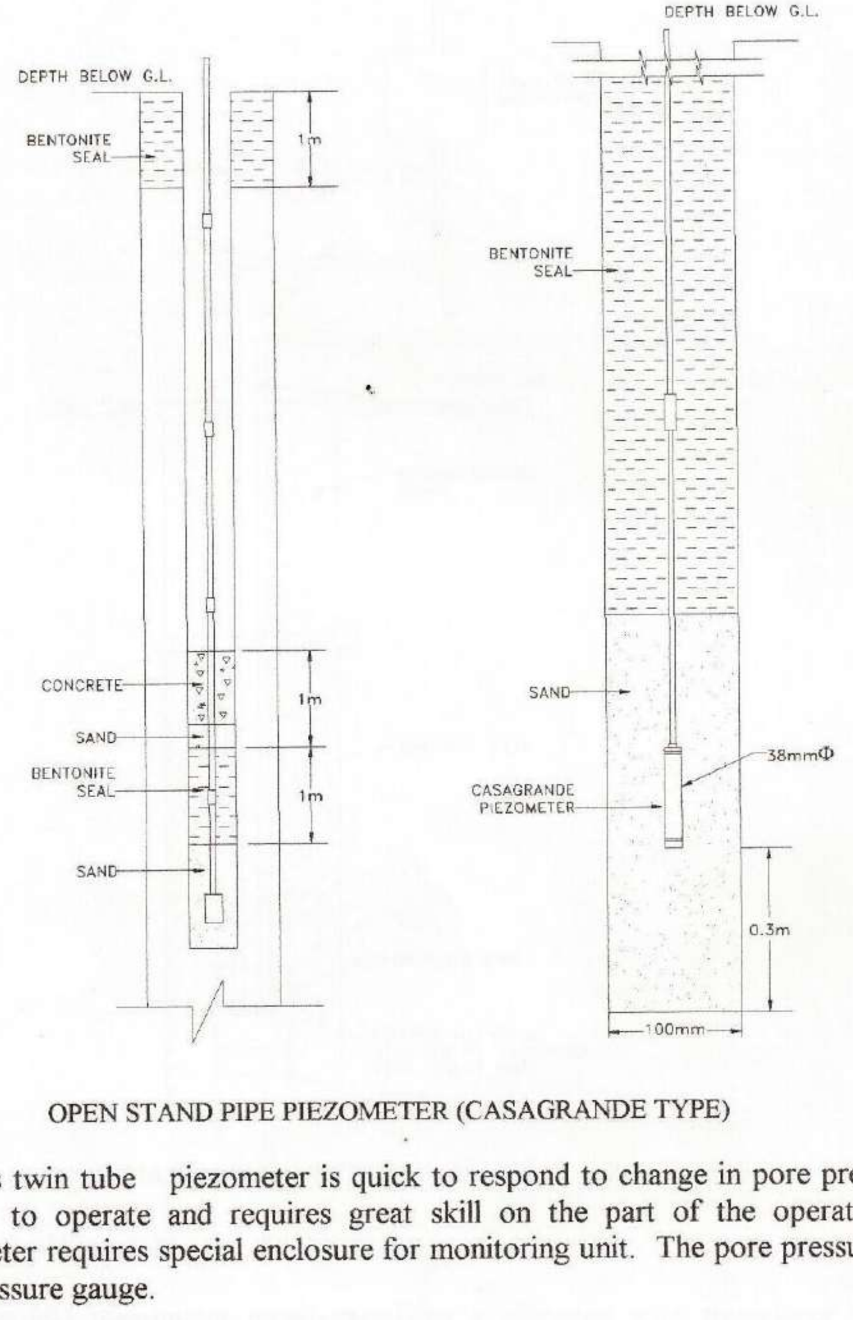
- Hydraulic piezometer
- Pneumatic piezometer
- Electrical piezometer

4.1 Hydraulic piezometer

A piezometer, which reads the water pressure through rise in water level in its stand pipe or through a pressure gauge is called a hydraulic piezometer. The simplest type of hydraulic piezometer is the Casagrande open stand pipe type piezometer. It is a low cost instrument and is available indigenously. Bishops twin tube piezometer is an improved version of Casagrande piezometer which reads changes in pore pressure through a manometer.

Casagrande piezometer is rugged and reliable. The instrument is not sensitive, i.e., it does not depict changes in pore pressure immediately. The sensitivity of the piezometer is suitably increased by increasing its intake factor. Because of low cost, ruggedness, simplicity in operation and easy availability, the Casagrande type of piezometer is extensively used.

Casagrande open stand pipe piezometer consists of a ceramic porous tip connected to open stand pipes. The ceramic tip is generally of low air entry value which exhibits very high water permeability. The piezometer is installed in a cased borehole and shrouded with sand. Depending upon the pore water pressure existing at the porous tip water would rise in the standpipe until the hydrostatic head of the column of water in the stand pipe is equal to the pore water pressure. The height of water in the stand pipe may be measured with an electronic water sensor. This consists of a probe connected with a graduated cable and either connected to a sound signalling device or to an ohm meter which gets activated as the probe touches the water. Increase in pore pressure would result in rise in water level in stand pipe. In order to record a given incremental pore water pressure within the ambient soil, large volume of water is required to flow into piezometer unit. The time required for equalization of pore pressure is called time lag. It is dependent primarily on the type and dimensions of the piezometer and the permeability of the ground. Typical figure is as shown below-

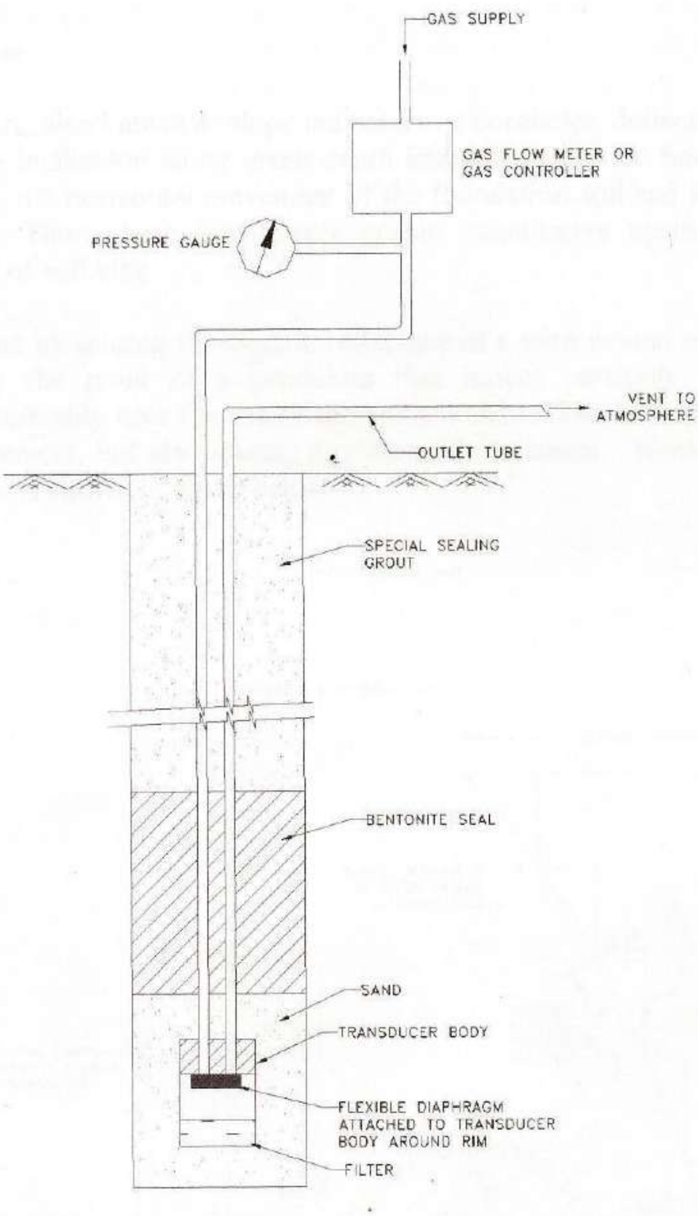


OPEN STAND PIPE PIEZOMETER (CASAGRANDE TYPE)

Bishop's twin tube piezometer is quick to respond to change in pore pressure but difficult to operate and requires great skill on the part of the operator. This piezometer requires special enclosure for monitoring unit. The pore pressure is read by a pressure gauge.

4.2 Pneumatic piezometer

The pneumatic piezometers are difficult to operate and have many disadvantages, such as, regulating the gas pressure at par with pore pressure in sub-soil and detecting the outflow of gas for pore pressure indication. Figure below gives details of a pneumatic piezometer-



PNEUMATIC PIEZOMETER INSTALLED IN BOREHOLE

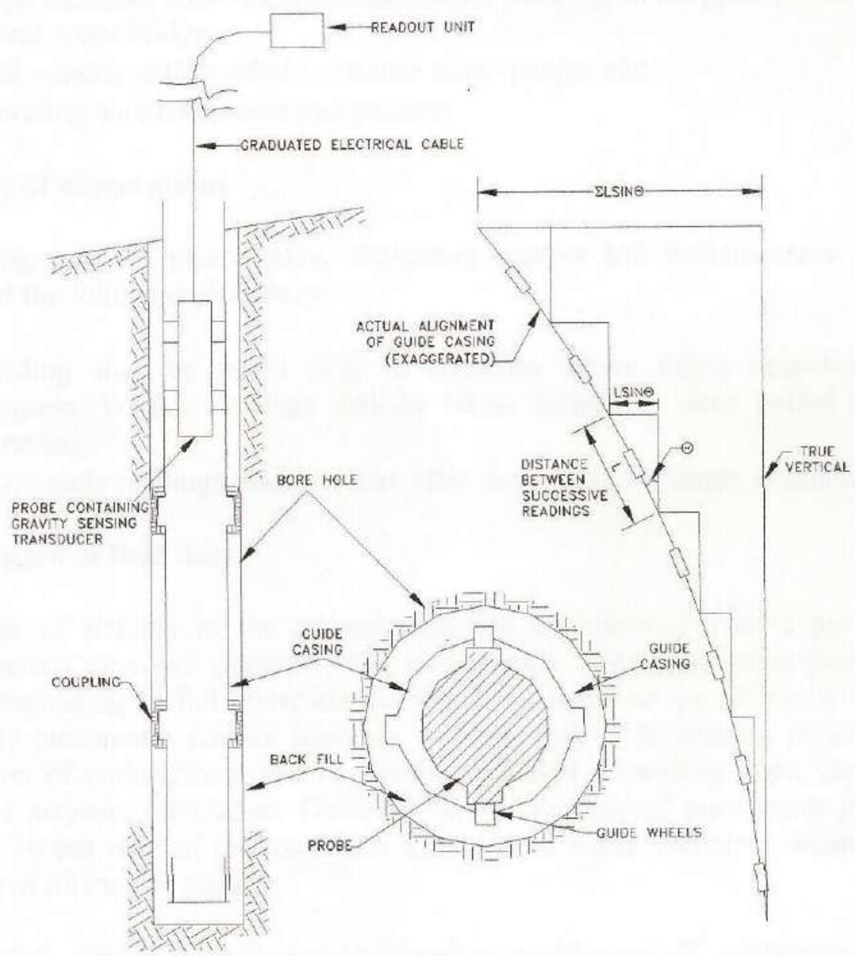
4.3 Electrical piezometer

The electrical piezometer mostly contains a vibrating wire transducer on which water pressure acts through a ceramic tip. Its easy installation and quick response to pore pressure is a critical factor to monitor. Whereas hydraulic piezometer, especially the Casagrande type is installed in a borehole, at a particular depth and the piezometer tip is sealed to prevent surface water interfering with the pore water, electrical piezometer can be pushed in the soft soil with special equipment to the desired depth. The electrical piezometer requires to be calibrated before installation as they instruments cannot be calibrated after being installed.

5.0 Inclinometer

Inclinometers, also known as slope indicators or boreholes deflectometers, measure the average inclination along given depth intervals inside the boreholes. From the inclinations, the horizontal movement of the foundation soil and its time rate can be determined. Thus, these instruments enable quantitative measurement of lateral movements of soft clay.

They operate by sensing the electric resistance of a wire wound arc, whose length is affected by the point of a pendulum that moves vertically. Inclinometers are installed, preferably near the toe of the embankment. The instrument not only gives lateral movement, but also reveals direction of movement. Working principle of an inclinometer is shown in figure below-



SCHEMATIC SKETCH OF INCLINOMETER

Most of the inclinometers have a probe containing gravity sensing transducers which measure inclination of the pipe with vertical. Horizontal movement of the pipe with respect to its vertical alignment is computed from the data, provided by the inclinometer. Special pipe which is grooved in the longitudinal directions is placed in near vertical position in the borehole and the probe travels vertically downwards. Measurements are taken at regular intervals of depth with a readout unit. Though, a variety of inclinometers are available in the market, essentially all of them have four similar features:

- A permanent guide casing installed in a near vertical alignment
- A portable probe containing a gravity sensing transducer
- Portable readout unit for power supply and indication of probe inclination
- Graduate electrical cable linking the probe to the readout unit

Depending upon the type of transducer used in a probe, types of inclinometers available in the market are:

- Inclinometer with force balance accelerometer
- Slope indicator with suspended pendulum working on the principle of wheat stone bridge.
- Inclinometer with bonded resistance strain gauge, and
- Vibrating wire transducer inclinometer

6.0 Frequency of observations

The readings on the piezometers, settlement gauges and inclinometers may be recorded at the following frequency:

- Reading shall be taken daily in stretches where filling operation is in progress. Weekly readings shall be taken during the same period in other stretches.
- Fortnightly readings shall be taken after the desired fill height is achieved.

7.0 Interpretation of field data

The course of stability of the embankment can be followed from a pore water pressure versus time plot obtained from piezometers. The equilibrium piezometric level corresponding to full consolidation would be equal to the reference value of the dummy piezometer remote from the embankment. The waiting period in the construction of embankment can be suitably adjusted depending upon the rate of pore water pressure dissipation. Generally, when the residual pore water pressures are about 10 per cent of the maximum excess pore water pressures recorded, the next stage of fill may be placed.

Observational data collected would acquire significance if settlement, lateral deformations and piezometric observations are interlinked with each other. Such a step would facilitate realistic and rational interpretation of the observational data so that the state of safety of the embankment is always known. Proper installation and maintenance of measuring units and adoption of correct measurement procedures is essential for ensuring satisfactory functioning of the instruments. Therefore, for proper installation of instruments, meaningful interpretation and maintenance of measuring units, services of an experienced geo-technical engineer would be essential.

The observational data would have to be recorded for a long period to monitor the long term performance of the embankment on soft ground. It is, therefore, essential that the equipments are not tampered and stolen. Suitable precautions should be taken in this regard.

8.0 Selection of instruments

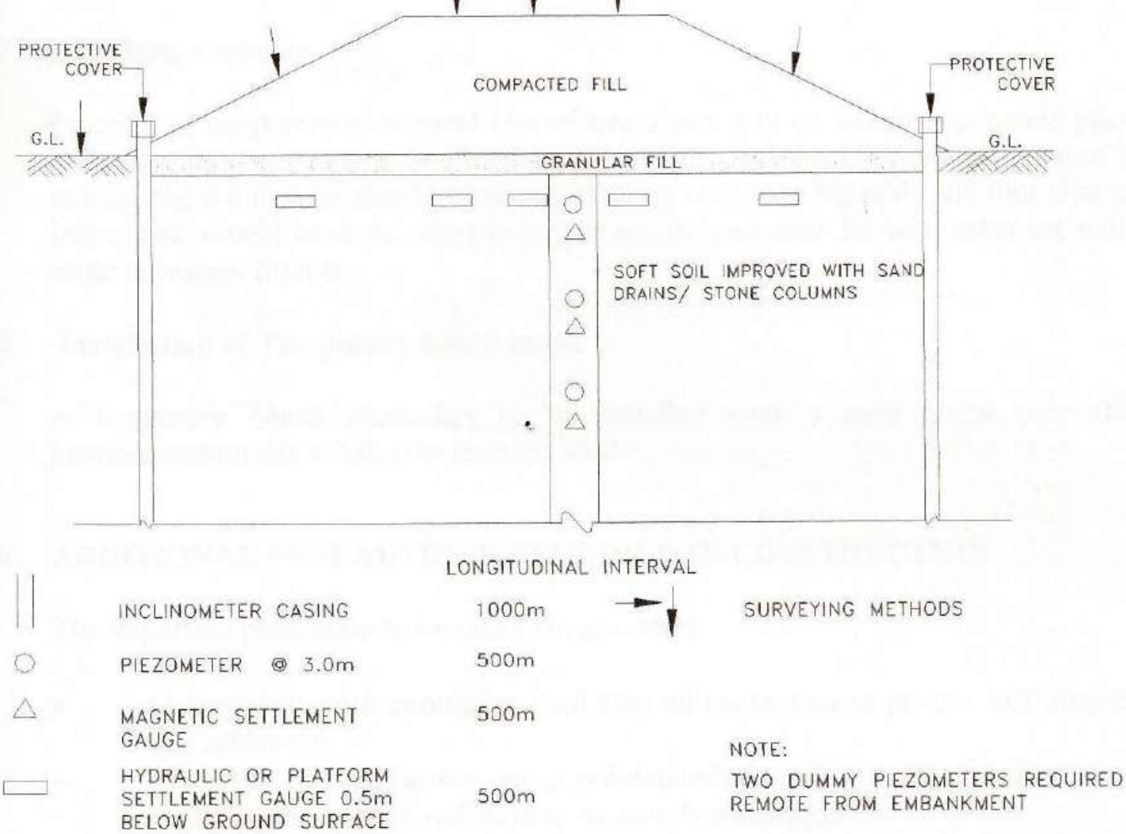
While selecting the instrument, the most desirable feature is reliability, repeatability and ruggedness. The simplicity decreases and reliability increases with the following order of principle.

- Optical
- Mechanical
- Hydraulic
- Pneumatic
- Electrical

The instruments installed for embankment are exposed to earth moving machinery and weather. Therefore, the instrument should be rugged and reliable. The instruments should be easy to install and operate. Sophisticated instruments are not only costly but are difficult to install and require a great expertise of the operator.

9.0 Layout of instruments

A general pattern of layout of instrumentation for monitoring behaviour of soft soil under the embankments is shown below. Surveying methods are used to check the formation level of the embankment and the heaving is monitored with a theodolite. Lateral flow of the soft soil is monitored with a inclinometer installed at the toe of the embankment. Settlement gauges are placed slightly below the ground level before placing the fill. Piezometers are installed near the middle of the cross section of the embankment to monitor the changes in the pore water pressure. A few piezometers are installed away from the embankment to serve as dummies and also to record seasonal variation in pore pressure.



LAYOUT OF INSTRUMENTATION BETWEEN EMBANKMENT

12. INSTALLATION OF EQUIPMENTS

12.1 Frequency of installation

The settlement gauges may be installed along the alignment of the embankment at longitudinal interval of 500 m. The platform settlement gauges may be installed along the center line of the embankment and on either side of center line in a staggered fashion at 0.5 m below the ground surface. The magnetic settlement gauges may be installed along the center line of the embankment at 3.0 m depth intervals in the soft clay.

Piezometers are generally installed along the center line of the embankment at longitudinal interval of 500 m. Two dummy piezometers may be installed remote from the embankment to monitor any variations in the ground water pressure that may result from other causes.

Inclinometers may be installed at 1000 m longitudinal intervals along the length of the embankment near both toes of the embankment in a staggered arrangement. The casing unit shall be installed to the full depth of soft clay.

12.2 Installation scheme

Erection of temporary instrument Hut of size 3 m x 4 m on wooden /concrete piles in the marine clay strata on non settling foundation. Inside room , one platform of 3 m long and 0.6 m wide should be provided along with one big table and four chairs. Instrument should have AC electric power supply/generator set with other set with other necessary fittings.

12.3 Installation of Temporary bench mark

A temporary bench mark has to be installed over a hard strata near the instrumentation site to take the reduced level.

13.0 ADDITIONAL PRECAUTIONS REQUIRED IN CONSTRUCTION

The important precaution to be taken are as under:

- During earthwork contractor shall take all precaution to protect instruments and cables etc.
- The work shall be carried out in pre-determined stages along with necessary waiting period which will have to be closely monitored.
- The construction of each soil layer should start from edges and finished at the center.
- Compaction by rollers on each layer near the instrument installed should be avoided and small machine or hand ramming should be resorted to for compaction.
- Service road, truck/bullock cart plying etc. on both side of instrumented section should be prohibited.

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Annexure-I

DIVA-BASSEIN ROAD RAIL LINK PROJECT ,CENTRAL RAILWAY

SERIAL NO.	LOCATION	TYPE OF SAMPLE	DEPTH BELOW G.L	PARTICLE SIZE DISTRIBUTION				CONSISTENCY LIMIT		IS CLASSIFICATION	NMC	NDD	TRI. SHEAR TEST $\overline{CU}$		OMC	MDD	COMPRESSION INDEX Cc	PRECON.PRESS.Pc (KG/CM^2)	COEFF. OF CONSOLIDATIONCv	INITIAL VOID RATIO e_0
				GRAVEL	SAND	SILT	CLAY	LIQUID LIM.	P.I				c'	Φ'						
1	9.55	UD	.53	3	23	61	13	49	28	CI	18.5	1.48	.02	15°6'	-	-	-	-	-	-
Borrow PITS SOIL																				
1	9.5	D	.60	-	11	68	21	56	27	CH	-	-	..11	12°5'	25.4	1.55	-	-	-	
2	9.7	UD	1.28	2	25	49	24	64	36	CI	36.6	1.27	.01	5°30'	-	-	.51	.27	.028	1.65
3	16.10	UD	.53	-	-	-	-	-	-	-	45	1.09	.09	.13° 23'	-	-	-	-	-	-
4	do-	UD	1.45	-	4	96	-	75	43	CH	87	0.78	-	-	-	-	.99	.58	.01	2.31
5	do-	UD	2.36	-	3	81	16	68	37	CH	64	0.97	-	-	-	--	.56	1.11	.03	1.46
6	19.20	UD	.91	-	5	51	44	97	54	CH	69.7	0.90	.21	6°31'	-	-	-	-	-	-
7	do	UD	1.60	-	5	73	22	79	51	CH	83.5	.80	-	-	-	-	.51	.81	.O16	1.46
8	do	UD	2.93	-	2	62	36	87	62	CH	81	.84	-	-	-	-	.64	.69	.021	1.69
9	do	UD	4.28	-	2	55	43	85	61	CH	81	.82	-	-	-	-	.73	.50	.016	1.96
10	21.40	UD	.83	-	1	55	44	64	34	CH	79.8	.85	-	-	-	-	1.14	.65	.022	1.44
11	do	UD	1.45	-	1	66	33	70	35	MH	89.6	.80					.58	.83	.023	2.15

MRTS-II MADRAS

SERIAL NO.	LOCATION	TYPE OF SAMPLE	DEPTH BELOW G.L	PARTICLE SIZE DISTRIBUTION				CONSISTENCY LIMIT		IS CLASSIFICATION	NMC	NDD	TRI. SHEAR TEST $\overline{CU}$		COMPRESSION INDEX Cc	PRECON.PRESS.Pc (KG/CM^2)	COEFF. OF CONSOLIDATIO. (CM^2/ MIN)			VOID RATIO	SP.GRAVITY
				GRAVEL	SAND	SILT	CLAY	LIQUID LIM.	P.L				C′	Φ′			At. 8	At.2.0	At 4.0		
1	18495	UD	-	-	2	66	32	72	36	MH	40	1.254	.1088	23°11′	0.426	0.75	.048	.018	.104	0.89	2.49

Annexure-I

TAMLUK-DIGHA RAIL LINK, S.E. RAILWAY

LOCATION	BORE HOLE NO.	TYPE OF SAMPLE	DEPTH BELOW G.L	PARTICLE SIZE DISTRIBUTION				CONSISTENCY LIMIT		IS CLASSIFICATION	BULK DENSITY (GM/CC)	NMC (%)	NDD	TRI. SHEAR TEST CU		SOIL COHESION(T/M^2)	ANGLE OF INTER. FRICTION Φ'	COMPRESSION INDEX Cc	VOID RATIO	COEFF. OF CONSOLIDATIO. (CM^2/MIN)
				GRAVEL	SAND	SILT	CLAY	LIQUID LIMIT	P.L					C' (KG/CM^2)	Φ'					
SOURIPUR KHAL	BH 1	UD	21	0	33	46	21	33	16	CL		36	1.48	0.20	13	2.0	13	0.25	0.85	0.25
	BH 2	UD	15	0	19	56	25	37	19	CI		21	1.46	0.22	13	2.2	13	0.23	0.66	0.23
BAGDA KHAL	BH 1	UD	15	4	10	42	44	61	31	CH	1.8	29	1.46	0.30	8	3.0	8	0.27	0.91	0.27
CONTAI CANAL	BH 1	UD	15	0	20	68	12	32	25	ML	1.9	27	1.53	0.33	8	3.3	8	0.3	0.84	0.24
	BH 1	SPT		0	93	7	-	NP	-	SW-SM	-	-	-	-	-	-	-	-	-	-
	BH 1	SPT		0	90	1	0	-	NP	SW-SM	-	-	-	-	-	-	-	-	-	-
	BH 1	SPT		6	18	54	22	49	23	CL	-	-	-	-	-	-	-	-	-	

ALLEPPEY-KAYAKULAM RAIL LINK PROJECT, SOUTHERN RAILWAY

SERIAL NO.	LOCATION	TYPE OF SAMPLE	DEPTH BELOW G.L	PARTICLE SIZE DISTRIBUTION				CONSISTENCY LIMIT		IS CLASSIFICATION	NMC	NDD	TRI. SHEAR TEST $\overline{CU}$		COMPRESSION INDEX Cc	PRECON.PRESS.Pc (KG/CM^2)	COEFF. OF CONSOLIDATIO (CM^2/ MIN)					SP. GRAVITY
				GRAVEL	SAND	SILT	CLAY	LIQUID LIM.	P.L				C′	Φ′			At .25	At.50	At.1.0	At.2.0	At 4.0	
1	20900 (CL)	UD	1.0-1.48	-	79	5	16	NP	NP	SM	14	1.75	0.16	35°16’	.0698	0.80						2.64
2	20900 (T)	UD	do	-	-	-	-	-	-	-	197	0.42	.005	29°11′	-	-	-	-	-	-	-	-
3	20750 (CL)	UD	1.2-1.68	-	91	9	-	-	-	SP-SM	8	1.85	C = 0.08	Φ= 29°′	-	-	-	-	-	-	-	-
4	20750 (T)	UD	do	-	-	46	54	190	40	CH	158	0.50	-	-	1.11	0.18	.018	.002	.005	.004	.003	2.56

VALUES OF TIME FACTOR

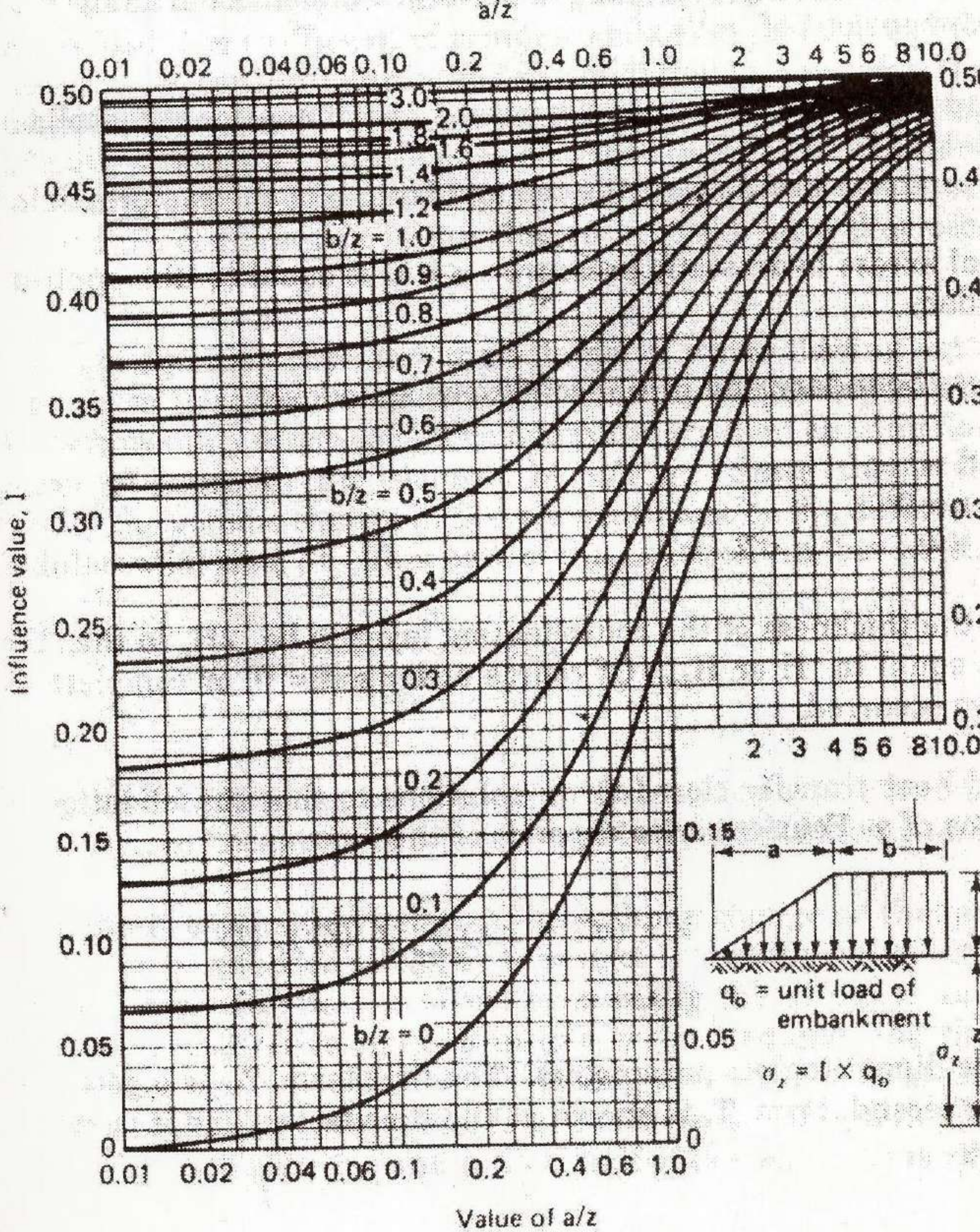
U (%)	T_V	U (%)	T_V
5	0.002	55	0.238
10	0.008	60	0.287
15	0.018	65	0.342
20	0.031	70	0.403
25	0.049	75	0.477
30	0.071	80	0.567
35	0.096	85	0.684
40	0.126	90	0.848
45	0.159	95	1.129
50	0.197	100	∞

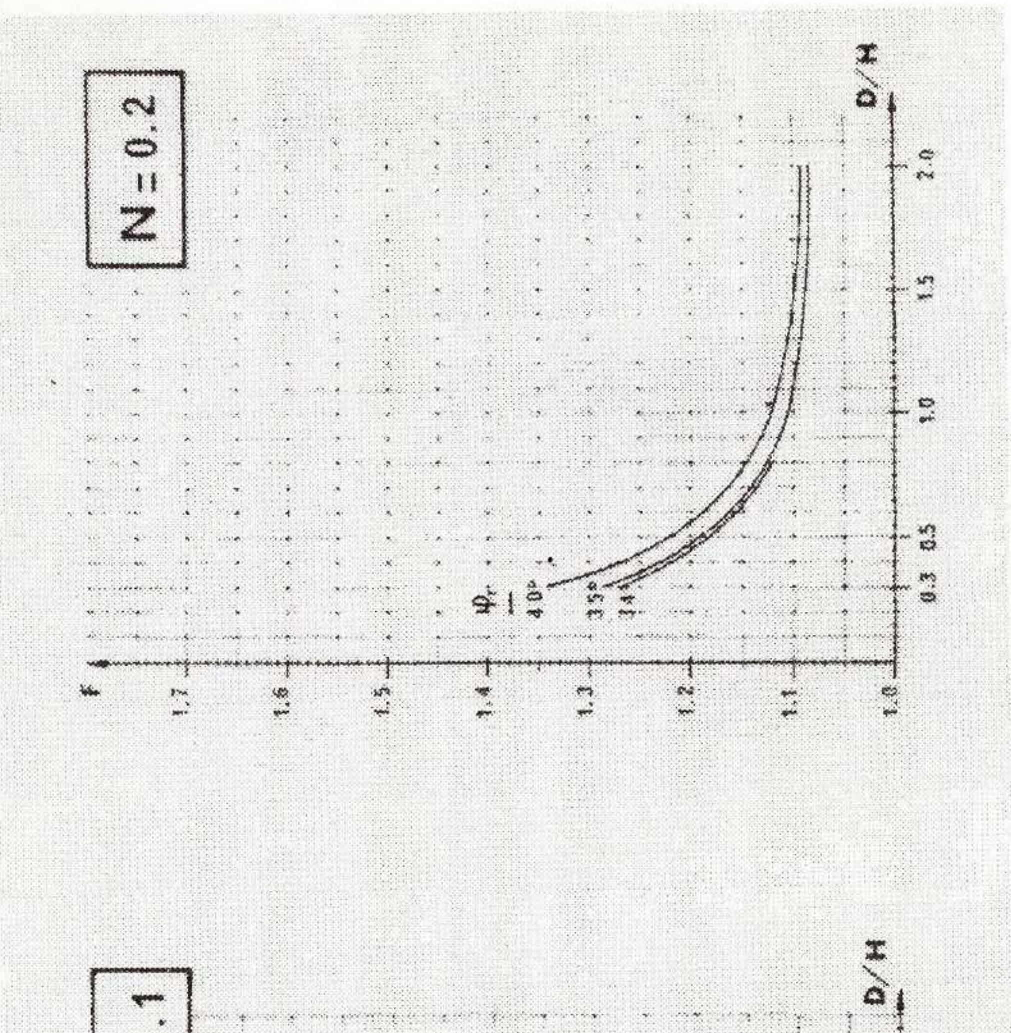
Note: The values given in above table are valid for following boundary conditions:

(i) Double drainage with all linear distribution of consolidation pressure

Or

(ii) Single drainage with uniform distribution of consolidation pressure


a/z
0.01 0.02 0.04 0.06 0.10 0.2 0.4 0.6 1.0 2 3 4 5 6 8 10.0
3.0
2.0
1.8
1.6
1.4
1.2
b/z = 1.0
0.9
0.8
0.7
0.6
b/z = 0.5
0.4
0.3
0.2
0.1
b/z = 0
Influence value, I
0.50 0.45 0.40 0.35 0.30 0.25 0.20 0.15 0.10 0.05 0
2 3 4 5 6 8 10.0
a
b
q_o
q_o = unit load of embankment
z
σ_z
$\sigma_z = I \times q_o$
0.01 0.02 0.04 0.06 0.1 0.2 0.4 0.6 1.0
Value of a/z


N = 0.2
F
1.7
1.6
1.5
1.4
1.3
1.2
1.1
1.0
0.3
0.5
1.0
1.5
2.0
D/H
N = 0.1
F
1.2
1.1
1.0
0.9
0.8
0.7
0.6
0.5
0.3
0.5
1.0
1.5
2.0
D/H

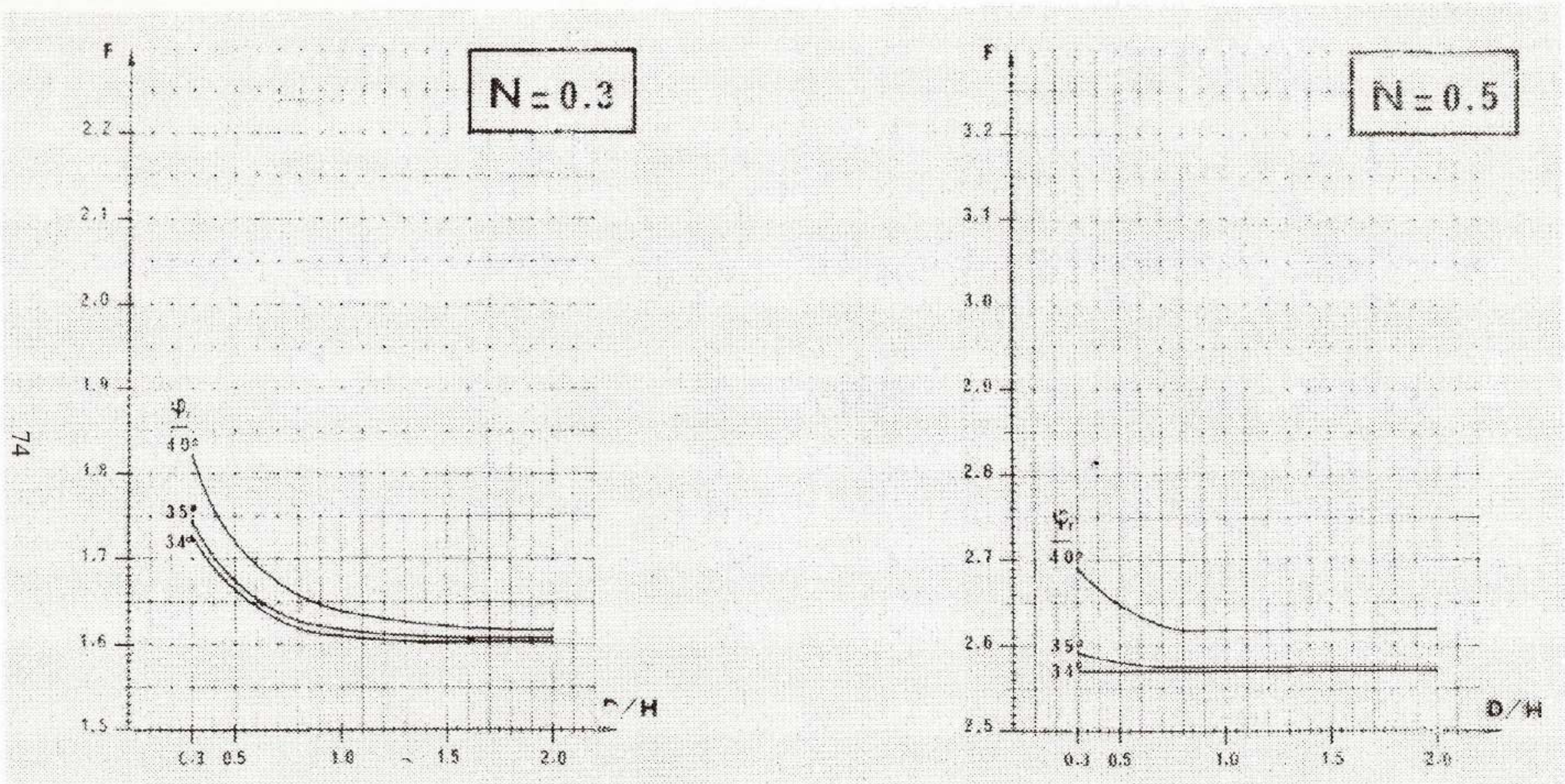


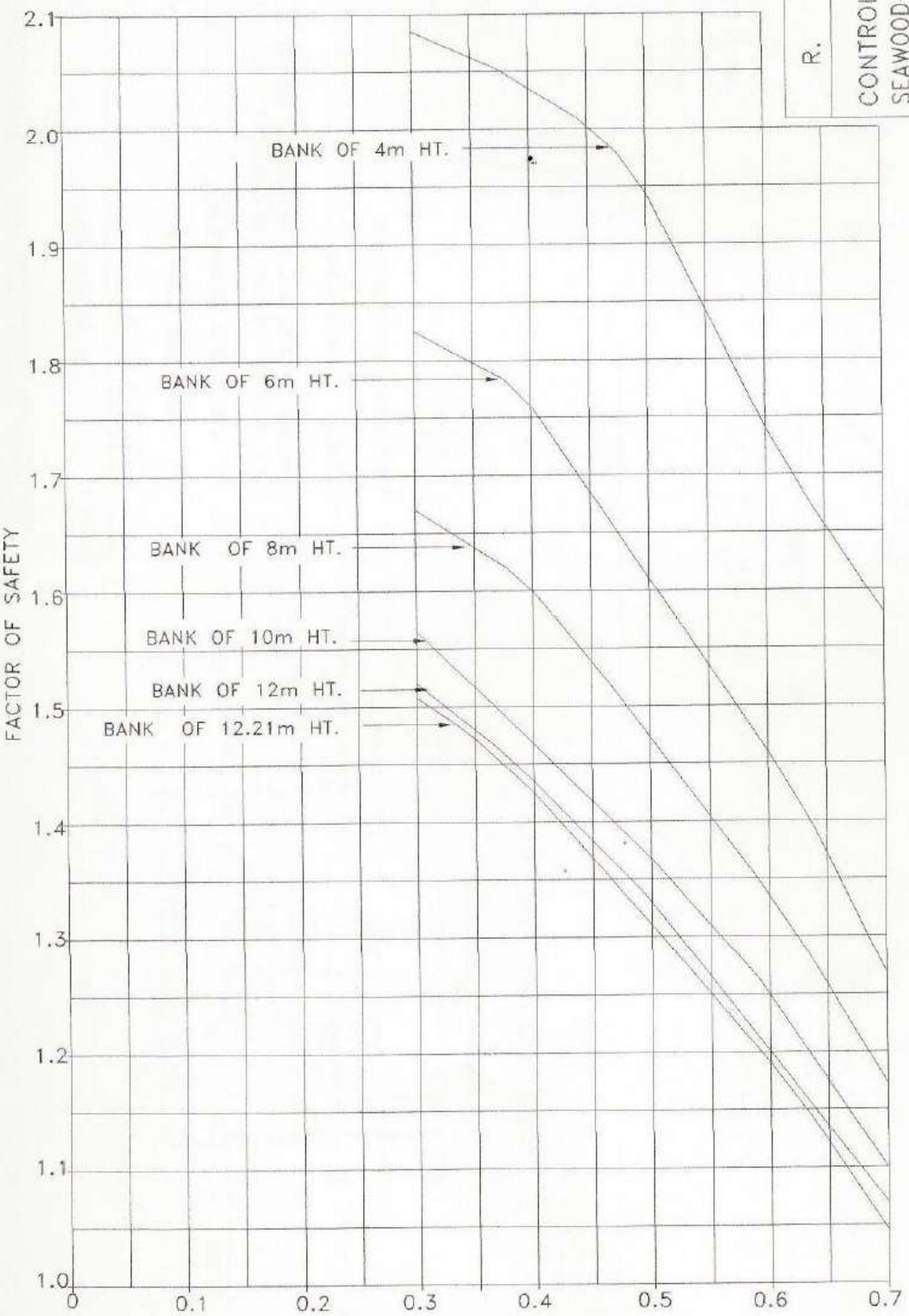
Fig. 12.28. After Pilot and Moreau. Slope 3/2. $N = c_u/\gamma H$. (c_u: undrained shear strength of clay; φ_r: friction angle of the fill).

Annexure-V

SOIL PARAMETERS ADOPTED ARE:

	$c' - t/m^2$	ϕ'	r_u	r_{sat} t/m^3
BANK	0.5	33^0	0.23	2.00
BASE	1.0	24^0	VARYING	1.60

$\bar{B}$ = EXCESS WATER PRESSURE/r_{sat} . HT. OF BANK



R. D. S. O.

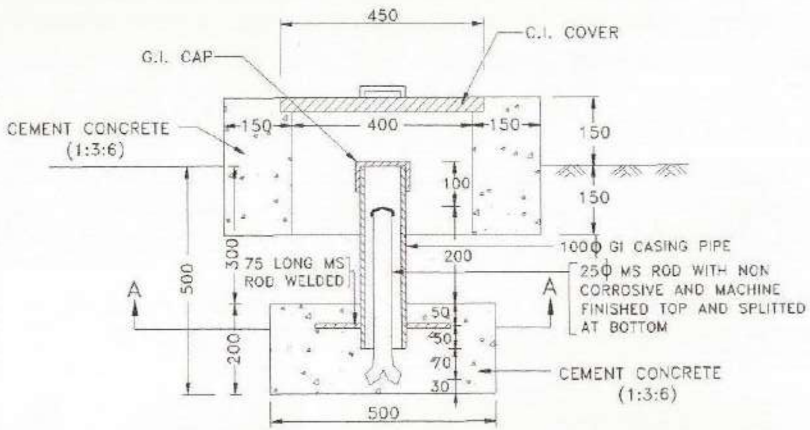
CONTROL CHART FOR SEAWOOD-URAN RAILWAT BANK CONSTRUCTION MTP(RLY) MUMBAI

DRG.NO: GT/SK/GL/0160/Rev.0/2001

NOT TO SCALE

APPROVED BY	
DIR/GE	ED/GE

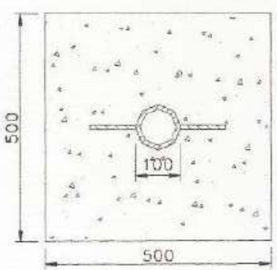
PLOTED	COMPARED
DRAWN	CHECKED



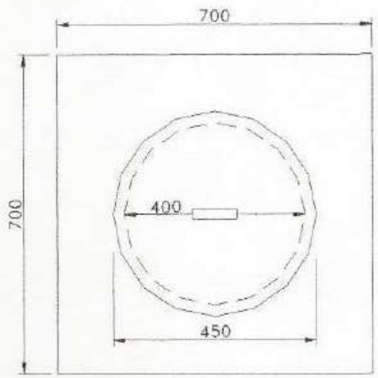
CROSS SECTIONAL ELEVATION

MATERIALS REQUIRED FOR SURFACE SETTLEMENT PLATEFORM

S.No.	DESCRIPTION	SPECIFIED SIZE	NO. REQUIRED
1.	CAST IRON COVER	450mm φ	1 No.
2.	G.I. CASING PIPE 100mm φ	450mm LONG	1 No.
3.	G.I. CAP TO SUIT 100mm φ PIPE	–	1 No.
4.	M.S. ROD 25mm φ WITH NON CORROSIVE AND MACHINE FINISHED TOP & SPLITTED AT BOTTOM.	370mm LONG	1 No.
5.	CEMENT CONCRETE 1:3:6	–	$0.160m^3$
6.	M.S. ROD 10mm φ 75 LONG	–	2 No.



CROSS SECTIONAL PLAN AT AA



PLAN AT THE TOP

NOTE:-

INSTEAD OF C.I. COVER SUITABLE RCC COVER RECTANGULAR OR OTHERWISE WHICH CAN BE LIFTED BY TWO MEN, CAN BE USED.

	R. D. S. O.
	DETAILS OF SURFACE SETTLEMENT PLATFORM
SCALE	
1 : 10	GE/SK/GL/0321/REV.0/2003

ALL DIMENSIONS ARE IN MILLIMETER

ACKNOWLEDGEMENT

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