SPECIAL REPORT 14

State of the Art: High Embankments on Soft Ground - Part B - Ground Improvement

IRC HIGHWAY RESEARCH BOARD
NEW DELHI
1995

SPECIAL REPORT

State of the Art:
High Embankments on Soft Ground - Part B - Ground Improvement

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PREFACE

The State-of-the-Art Report on High Embankments on Soft Ground has been prepared by Central Road Research Institute under a Research Scheme sponsored by Ministry of Surface Transport. This report is in two volumes. Part-B pertains to the studies about "High Embankments on Soft Ground - Stage Construction" and Part-B pertains to "High Embankments on Soft Ground - Ground Improvement".

Part-A of the report consists of the following Chapters :

(1) Shear Strength of Soft Clays
(2) Stability Analysis
(3) Settlements of Embankments on Soft Clay
(4) Engineering Properties of Marine Clays in India
(5) Precompression Technique

It discusses in detail the stage construction technique which is a conventional ground improvement method in use in engineering construction since long. It also provides a general background on principles of soil mechanics which help the engineer in better understanding of different ground improvement techniques which are discussed in detail in Part-B.

Part-B of the report consists of the follwoing Chapters :

(1) Consolidation by Vertical Drains
(2) Stone Columns Technique
(3) Ground Improvement by Lime Stabilisation
(4) Use of Geosynthetics for Ground Improvement of Soft Soil
(5) Dynamic Consolidation
(6) Instrumentation and Monitoring

It discusses in detail wide range of ground improvement techniques employed in engineering practice. It also discusses instrumentation and methods of monitoring the performance of embankments as well as design and construction aspects based on observational procedure.

This report was discussed in the 28th meeting of the Highway Research Board held at Patna on 28th November, 1992 wherein the Board authorised Prof. D.V. Singh, Director, Central Road Research Institute and Shri P.K. Dutta, Chief Engineer, Ministry of Surface Transport to finalise the report based on the comments received for publication through Indian Roads Congress.

It is hoped that it will serve as a useful ready reference book for practising highway engineers and researchers.

M.V. Sastry
Chairman
IRC Highway Research Board &
Director General (Road Development)
Ministry of Surface Transport
(Road Wing)

New Delhi
October, 1994

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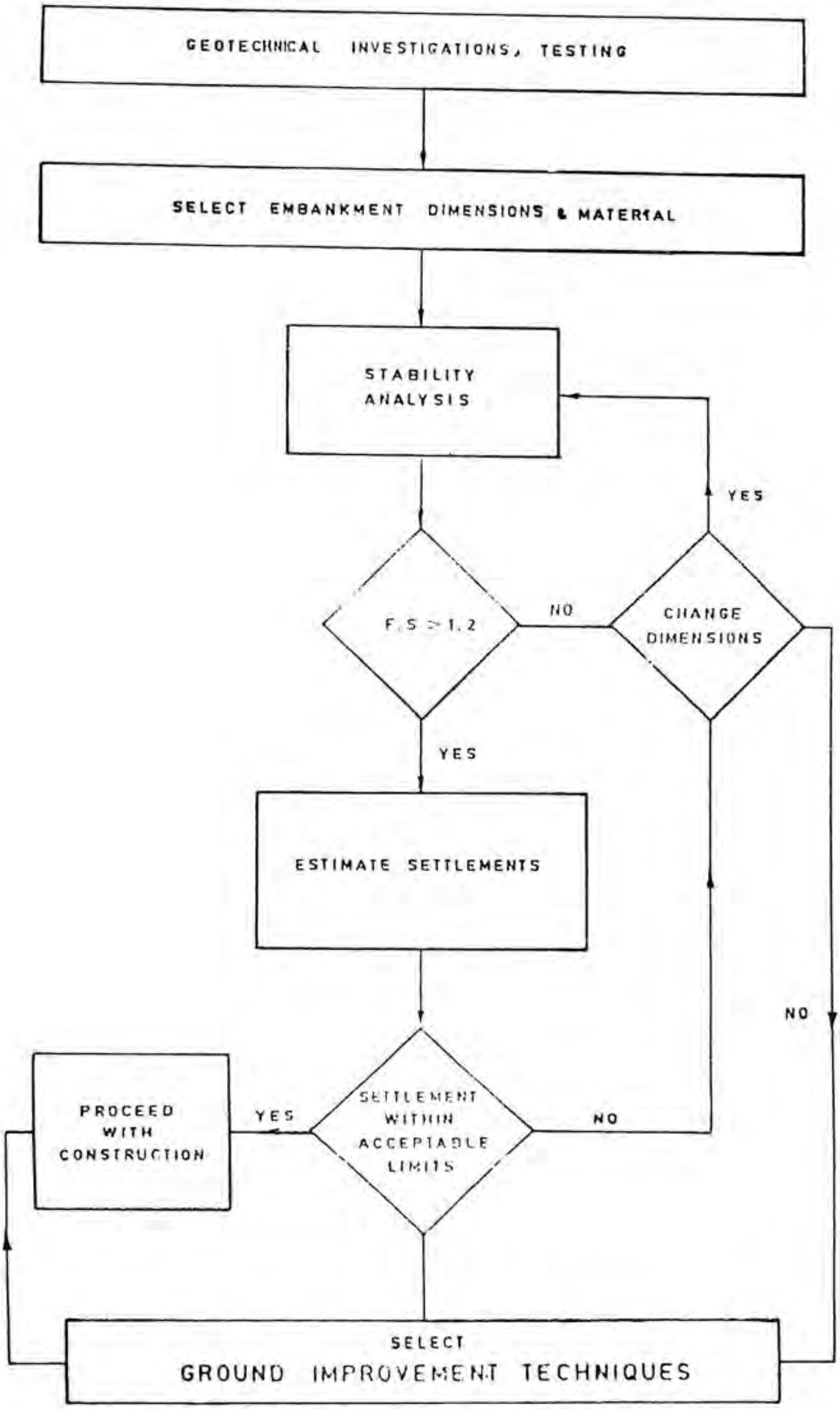
NOTATIONS

A	=	Skempton pore pressure coefficient
Acol	=	Area of lime column
a	=	Henkel's pore pressure parameter
a_s	=	Area replacement ratio
a_v	=	Coefficient of compressibility
B	=	Skempton pore pressure coefficient
B'	=	Width of the lime column block
b	=	Width of the loaded area
b'	=	Width of each slice
b_b	=	Width of the band drain
c	=	Cohesion intercept
C_r	=	Residual cohesion intercept
c'	=	Cohesion intercept based on effective stresses
c_v	=	Coefficient of consolidation in vertical direction
c_r	=	Coefficient of consolidation in horizontal direction
c_v	=	Equivalent coefficient of consolidation
C	=	Resultant cohesive force along slip surface in friction circle method
Cc	=	Compression index N-C soil
Cr	=	Compression index O-C soil
C	=	Coefficient of secondary compression
d	=	Diameter of the column/drain
dm	=	Average particle size
ds	=	Inside diameter of the stand pipe
d_e	=	Equivalent diameter of the unit cell
d_w	=	Diameter of drain well
D_m	=	Maximum depth of influence
Di	=	Diameter of intake filter
D	=	Vane diameter
D15 soil	=	Particle size for which 15% of soil fraction is finer
D85 grout	=	Particle size for which 85% of cement grout is finer
e_o	=	Initial void ratio
Δe	=	Change in void ratio
E	=	Young's modulus of soil
E_s	=	Young's modulus of stone column
E_c	=	Young's modulus of lime column
E_{ef}	=	Efficiency of geosynthetic
E_g	=	Young's modulus of geosynthetic
fs	=	Local friction
F	=	Factor of safety
F'_c, F'_c,	=	Cavity expansion factors
F_R	=	Factor of safety with geosynthetic reinforcement

F_V	=	Factor of safety without geosynthetic reinforcement
FB	=	Frictional bond
$F_{(B)}$	=	Factor of safety against bond failure
$G\beta$	=	Equivalent shear modulus of soil
h	=	Vane height
H	=	Thickness of compressible strata
H_L	=	Length of drainage path in laboratory
H_f	=	Length of drainage path in field
I	=	Influence factor which depends upon the shape of loaded area and depth of clay
I_p	=	Displacement influence factor
I_r	=	Rigidity index
I_B	=	Brittleness index
k_v	=	Bjerrum's correction factor for in-situ vane shear strength
K_v	=	Coefficient of permeability in vertical direction
K_o	=	Coefficient of earth pressure at rest
K_H	=	Coefficient of permeability in horizontal direction
K_{ps}	=	Rankine passive pressure coefficient of stone column
K_w	=	Coefficient of permeability of sand drain
K_p	=	Rankine passive pressure coefficient of the soil
L	=	Length of intake filter
L'	=	Lengh of lime column block
Le	=	Length of geotextile beneath the sloping embankment face
Lc	=	Length of geotextile beneath the full height section of the embankment
L_s	=	Length of each slice
L_H	=	Length of the stone/lime column
M_{RG}	=	Additional restoring moment due to geotextile
m_j	=	Janbu's modulus number
m_v	=	Coefficient of volume decrease
Nc	=	Bearing capacity factor
n	=	Stress concentration factor
n_r	=	Ratio of diameter of zone of influence to diameter of sand drain
n'	=	Number of columns
n_p	=	Coefficient of surcharge
n_h	=	Number of drainage paths
N	=	Normal force/reaction
P_o	=	Effective overburden pressure
Pc	=	Preconsolidation pressure
P_L	=	Limit pressure
ΔP	=	Increase in effective pressure of mid depth of clay layer
P_f	=	Final design load
Ps	=	Surcharge load
Q	=	Quantity of lime slurry flow
q	=	Net foundation pressure
q_{ult}	=	Ultimate bearing pressure
q_a	=	Average applied external stress
q_{allow}	=	Allowable bearing capacity

q	=	Mean isotropic stress at the average depth of bulge
q_c	=	Cone resistance
q_g	=	,Applied stress due to load W_g on lime column
r	=	Radial distance from centre of drain
r_w	=	Radius of drain well
R_f	=	Friction ratio
S	=	Spacing between columns/sand drains
S_u	=	Undrained shear strength of soil
ΔSu	=	Increase in undrained shear strength
S_p	=	Peak shear strength
Sr	=	Residual shear strength
S	=	Shear resistance of soil at the base of slice
SR	=	Settlement ratio
T_T	=	Total tension in the geosynthetic layers
T_{reqd}	=	Required strength of geosynthetic reinforcement
$t90$	=	Time required for 90% response in clay
t	=	Time for consolidation
t_f	=	Time in field for specified degree of consolidation
t_L	=	Time in laboratory for same degree of consolidation
t_{SR}	=	Time of removal of surcharge load
t_s	=	Secondary compression time
t_p	=	Time at which consolidation settlement are complete under design load
t_b	=	Thickness of band drain
T	=	Maximum torque
T_v	=	Time factor for vertical consolidation
T_f	=	Shear strength of soil
T_r	=	Time factor for radial flow
T_{oct}	=	Increase in octahedral shear stress
V_L	=	Viscocity of lime slurry
u	=	Excess pore water pressure
$u_e(z)$	=	Excess pore water pressure at any depth Z
u_{eo}	=	Initial excess pore pressure under surcharge load
U	=	Average degree of consolidation
U_{f+s}	=	Average degree of consolidation under design load and surcharge load
Uz	=	Degree of consolidation for vertical flow
Ur	=	Degree of consolidation for radial flow
Vo	=	Initial volume of pressuremeter
W	=	Weight of slice
Wd	=	Weight of pounder
Wg	=	Applied load on group of lime columns
z	=	Depth below the ground surface
z_g	=	Depth of geotextile below the top of embankment
T_n, T_{n+1}, E_n, E_{n+1}	=	Boundary interslice forces
γ	=	Unit weight of soil
γ_w	=	Unit weight of water

γ_l	=	Unit weight of lime slurry
γ_p	=	Perimeter shear stress
σ	=	Average external applied stress
σ_s	=	Stress in the stone column
σ_c	=	Stress in the surrounding soil of stone column
σ_p	=	Passive resistance of a single stone column
σ_{RL}	=	Ultimate lateral stress
$\bar{\sigma}_{RO}$	=	Effective in-situ radial stress
$\overline{\sigma}$	=	Effective stress
$\Delta\sigma$	=	Increase in effective vertical stress
$\sigma_1, \sigma_2, \sigma_3$	=	Principal stressess
$\Delta\sigma_1, \Delta\sigma_2, \Delta\sigma_3$	=	Changes in principal stresses due to applied load
$\bar{\Delta}\sigma_{oct}$	=	Increase in octahadrel normal stress
$\bar{\sigma}_{vo}$	=	Initial vertical effective stress
σ_{ho}	=	Initial horizontal effective stress
ρ	=	Settlement of a single column
ρ_{group}	=	Settlement of group of columns
ρ_t	=	Total settlement
ρ_{ie}	=	Initial settlement without considering yield
ρ_f	=	Final consolidation settlement under design load
ρ_{f+s}	=	Final consolidation settlement under design load and surcharge load
ρ_c	=	Consolidation settlement
ρ_i	=	Immediate settlement including yield
ρ_s	=	Secondary settlement
$\rho_{(t)}$	=	Time rate of settlement
ϕ	=	Angle of internal friction of soil
ϕ_r	=	Residual angle of friction of soil
ϕ'	=	Angle of internal friction of soil based on effective stresses
ϕ_s	=	Angle of internal friction of stone column
θ_s	=	Inclination of each slice above failure surface with horizontal
α	=	Settlement reduction factor
ν	=	Poisson's ratio
β	=	Inclination of failure surface with horizontal
μ_{cp}' μ_{oc}'	=	Bjerrum's correction factor
μ	=	Angle of friction between soil and geotextile


GEOTECHNICAL INVESTIGATIONS, TESTING
SELECT EMBANKMENT DIMENSIONS & MATERIAL
STABILITY ANALYSIS
F.S > 1.2
NO
CHANGE DIMENSIONS
YES
YES
ESTIMATE SETTLEMENTS
NO
PROCEED WITH CONSTRUCTION
YES
SETTLEMENT WITHIN ACCEPTABLE LIMITS
NO
SELECT
GROUND IMPROVEMENT TECHNIQUES

INTRODUCTION

Planned development efforts over the last four decades have considerably enlarged and improved the road network in India, increasing it more than fourfold. The Network, which now forms an important component of the infrastructure for transportation, provides comfortable mobility to a large section of the population, stimulates their economic and social activities, and draws their great participation into the national life. However, increasing level of economic activity has resulted in a large increase in the volume of traffic on the entire road network. Further, the axle loads have also tended to increrase. To meet the demands for the increased traffic volumes and intensities, a quantum jump in highway construction is in the offing.

This expansion of the road network involves construction of expressways, highways, feeder roads and road bridges in widely dispersed areas with totally different sub-soil conditions, each of which present unique problems to road scientists and engineers. The problems faced by the engineers in the construction of roads and road bridges in the coastal and delta areas of the country arise primarily out of the low shear strength and high compressibility of the soft clay sub-soils. As a result, road embankments may fail or experience long term settlements. At the same time, consideration of road geometry and smooth traffic flow require that the roads in coastal and delta areas need to be located on embankments of substantial height.

Improvement of the load response behaviour of the soft sub-soil has become necessary if embankments are to be built economically and pavements with high serviceability made on them and an economic life ensured for such pavements. The know-how relating to ground improvement, therefore, has become vitally important to India.

The body of knowledge on ground improvement is scattered over a large number of published and unpublished works from India and abroad. A need was, therefore, felt for some time to consolidate the research findings on this subject in a single reference document to help research scientists and practising engineers in solving problems of road construction on soft ground.

This report seeks to synthesise the theoretical, experimental and empirical research findings concerning ground improvement techniques to-date. While there is stress on practical aspects in the report to help engineers directly appply the techniques in the field, the related theoretical aspects have also been covered and the areas needing further research are highlighted where appropriate. A number of case studies from India as well as other countries have been included.

1. CONSOLIDATION BY VERTICAL DRAINS

1.1. Introduction

Vertical drains have been in use for almost half a century to promote rapid consolidation of thick soft clay deposits, where preloading alone will be inefficient. The first installation of this technique was done in California in 1934. Uptill early 1970's large diameter drains of the order 50 cm were used. This caused a considerable smear problem around the drains. The smear problem was overcome in Netherlands during 1950's with the development of jetted drains. The method was not a great success and it was costly. Large jetting pumps were required and difficulties of disposing large quantities of water was a major drawback. The alternative to large diameter sand drains was the much smaller band shaped drains of cardboard, first developed and used by Kjellman (1948). These band drains proved susceptible to degradation and did not gain widespread acceptance. In the 1970's the rising costs of providing large quantities of suitable sand and the great technical advances in the manufacture of man made fabrics led to the development and use of a variety of synthetic band drains, manufactured from polyethylene, PVC, polypropylene and polyester, etc. 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.

1.2. Advantages of Vertical Drains

The main advantages of vertical drains in soft clay are:

(1) It accelerates 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.

(2) 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.

(3) Many soft clay deposits contain sand and silt seams. Instability of embankment on these strata is sometimes due to horizontal spread of excess pore pressure along these partings. Vertical sand drains relieve these excess pore pressure and avoid the occurrence of instability.

1.3. Efficiency of Vertical Drains

The efficiency of vertical drains technique is assessed in terms of the increased rate achieved in primary consolidation over the consolidation rate that would have occurred without the drains. According to Bjerrum (1972), the satisfactory performance of vertical drains at five sites has established a satisfactory efficiency factor value between 0.6 and 0.8.

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. Rowe (1968) concluded that for achieving a satisfactory efficiency of a vertical drain technique detailed soil investigation should be carried out which should include continuous core sampling of the soil strata, in-situ permeability measurements at low hydraulic head and laboratory consolidation tests on large diameter specimens

1.4. Types of Vertical Drains and Installation Techniques

There are four types of vertical drain used in engineering practice and are as follows,

(i) Sand drains
(ii) Sand wicks
(iii) Carboard drains
(iv) Synthetic drains

1.4.1. 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 1.1. The most common are closed mandrel and open mandrel methods.

Closed mandrels consists 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 sufficient 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 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 major problem in the formation of sand drains is formation of cavities due to bulking of sand. This can be avoided by using saturated sand at the time of placement.

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.

1.4.2. Sandwicks

Sand wicks are ready made small diameter sand drains prepacked in filter stocking. In early days woven jute canvas was used as filter stocking, but presently polypropylene woven and melt bonded fabrics are used. Sand wicks were first used in India by Dastidar et.al. (1969) followed by Subbaraju et.al. (1973).

Table 1.1. Common Methods of Installation (McGown et.al., 1982)

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 (shall and auger) methods with or without casing Hard 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 prefabricated drains

The installation can be done by a variety of methods as listed in Table 1.1, depending upon the soil conditions, the simplest being by hand auger. Economy is achieved in the amount of sand required. They are particularly attractive in developing countries where labour costs are low. The making and filling of jute canvas and subsequent installations in soft clay is done by hand.

1.4.3. Cardboard drains

The first person to use cardboard drains was Kjellmann (1948). The early cardboard drains were 100 mm wide and 3mm thick. A typical cross section of cardboard drain is shown in Fig. 1.1. They are inserted into the ground by means of a mandrel which is then removed. Channels in the cardboard facilitate the removal of water. A 100 mm x 3 mm cardboard drain is equivalent to a 50 mm diameter sand drain. They are easy to instal and cause little soil disturbance. The specially processed cardboard has a long life and are quite durable.

1.4.4. Synthetic drains

There are several types of synthetic drains widely in use. They have replaced cardboard drains. Table 1.2 gives a list of synthetic drains with dimensions and material used.

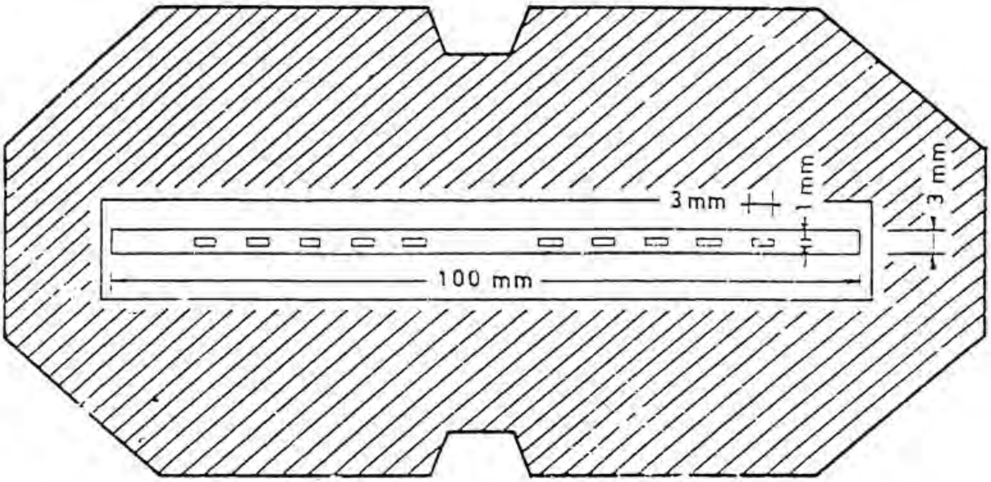


Fig. 1.1. Cross-section of Cardboard Drain and Insertion Mandrel

Table 1.2. Properties of Different Vertical Drains (McGown et.al., 1982)

Drain type	Appoximate dimensions (mm)		Materials		Approximate permeability of filter cm/s	Assumed nominal diameter (mm)
	Width	Thickness	Core	Filter		
Kjellman	100.0	3.0	Card-board	Card board	$1x10^{-5}$	66
Mebra Paper Filter	95.0	3.2	Polye-thylene	Treated Paper	$6x10^{-7}$	63
Mebra Polypro-pylene Polyester	95.0	3.4	Polye-thylene	Pro-pylene or polye-ster	$2x10^{-2}$	63
Geodrain	95.0	4.0	Polye-thylene	Treated paper	$6x10^{-7}$	63
Coolbond	300.0	4.0	Non-woven	Polyes-ter 650	$3x10^{-2}$	194
Alidrain	100.0	7.0	Plastic	Cellu-losic	$3x10^{-4}$	68
Castle Drain Boards	94.6	2.6	Polye-olefin	Non-woven engineer-ing fabric	$2x10^{-2}$	62

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 Fig. 1.2. 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 drains continuity.

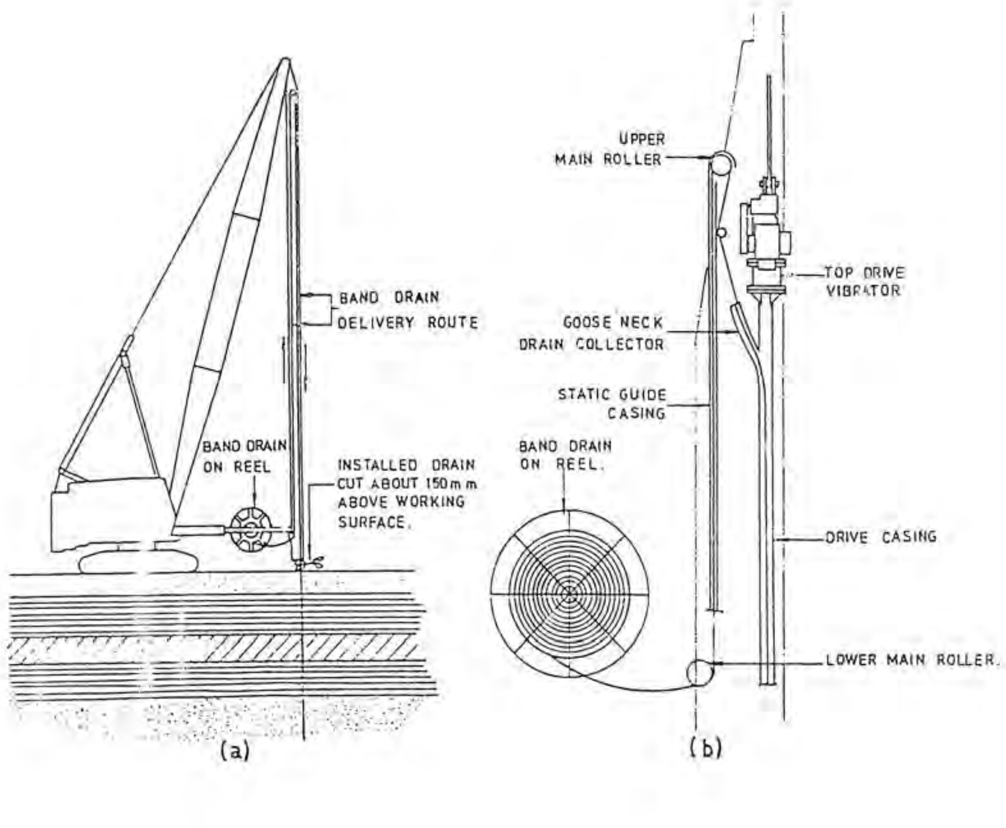


Fig. 1.2. Schematic Sketch of a Band Drain Installation Rig: (a) General Arrangement of Installation; Rig: (b) Band Drain Delivery Arrangement

1.5. Spacing and Depth of Vertical Drains

The vertical drains are usually installed in triangular or square grid pattern with spacings 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 given by McGown et.al., (1982) is illustrated in Fig. 1.3.

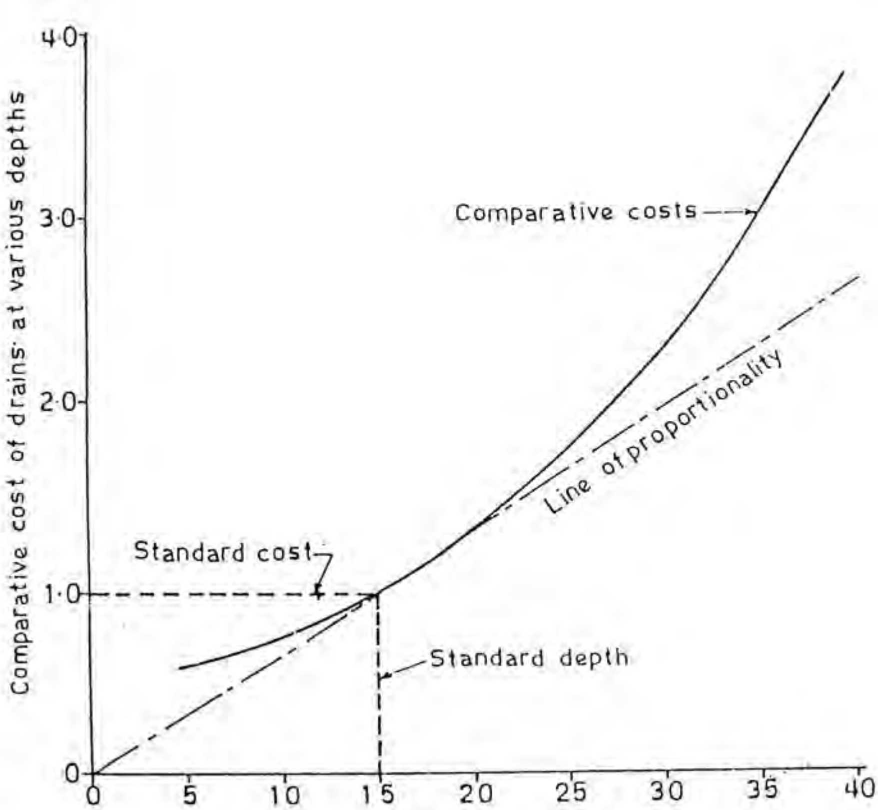


Fig. 1.3. Relationship Between Costs and Depth of Synthetic Drains Taking the Cost for 15 m Depth as the Unit Cost

1.6. Theoretical Considerations

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 theoretical design of vertical drains is based upon the independent bahaviour of each drain in the centre of cylindrical soil mass (Fig. 1.4). Many theories proposed by Barron (1948), Kjellman (1948), Chaput et.al. (1975), and Hansbo (1979) are based upon various assumptions, about the homogenity of soil, variations with time of permeability, coefficient of consolidation, smear effects, creep effect and loading conditions.

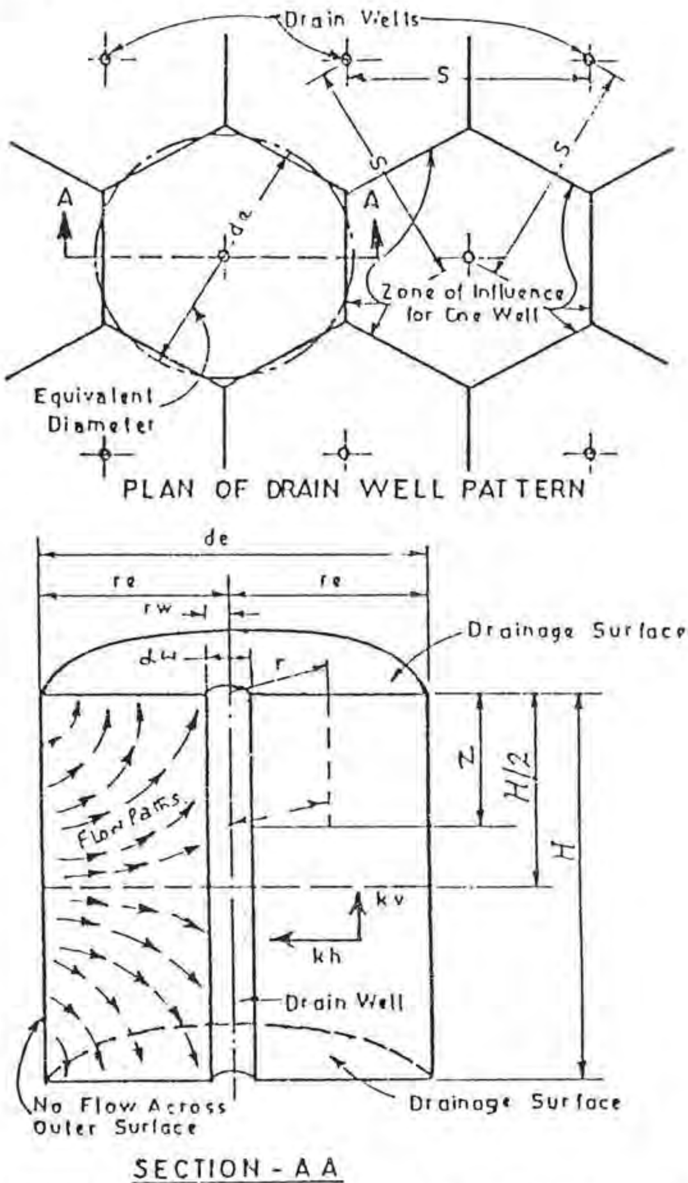


Fig. 1.4. Plan of Drain Well Pattern and Fundamental Concepts of Flow within Zone of Influence of Each Well

Consolidation of the cylindrical body of soil around a vertical drain is three dimensional and is dependent on following parameters, namely, coefficient of radial consolidation Cr, coefficient of vertical consolidation Cv, pore pressure u, radial distance from drain r, and time t. The equation which governs this relationship is as follows:

$$\frac{\delta u}{\delta t} = Cv \frac{\delta^2 u}{\delta z^2} + \left(Cr \frac{\delta^2 u}{\delta r^2} + \frac{1}{r} \cdot \frac{\delta u}{\delta r} \right) \qquad \text{Eqn. 1.1}$$

Carillo (1942) had shown that overall average degree of consolidation, is expressed by the following relationships

$$(1 - \bar{U}) = (1 - Ur)(1 - Uz) \qquad \text{Eqn. 1.2}$$

Where

$\bar{U}$ = Average total consolidation
U_z = Average vertical 1 - D degree of consolidation
U_r = Average radial degree of consolidation

The average degree of consolidation in the vertical direction is based on the one dimensional theory proposed by Terzaghi (1943)

$$Uz = 1 - \frac{8}{2\pi} \sum_{N=o}^{N=oo} \frac{1}{(2N+1)^2} \exp \left\{ (2N+1)^2 \pi^2 \frac{cv.\,t}{4H} \right\} \qquad \text{Eqn. 1.3}$$

This is expressed by an independent dimensionless variable called the Time factor for vertical consolidation, T_v.

$$T_v = cv.t / (H/2)^2 \qquad \text{Eqn. 1.4}$$

The average radial consolidation is based on Barron's (1948) solution of the following governing equation:

$$\frac{\delta u}{\delta t} = cv \left\{ \frac{\delta^2 u}{\delta r^2} + \frac{1}{r} \frac{\delta v}{\delta r} \right\} \qquad \text{Eqn. 1.5}$$

The equation was solved by Barron (1948) for uniform vertical strain and is as follows.

$$U_r = 1 - \exp\{-8\,Tr/F(n_r)\} \qquad \text{Eqn. 1.6}$$

where $T_r = C_r t/dc^2$

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

The relationship of U_v with time factor T_v and U_r for a range of n_r values is shown in Fig. 1.5.

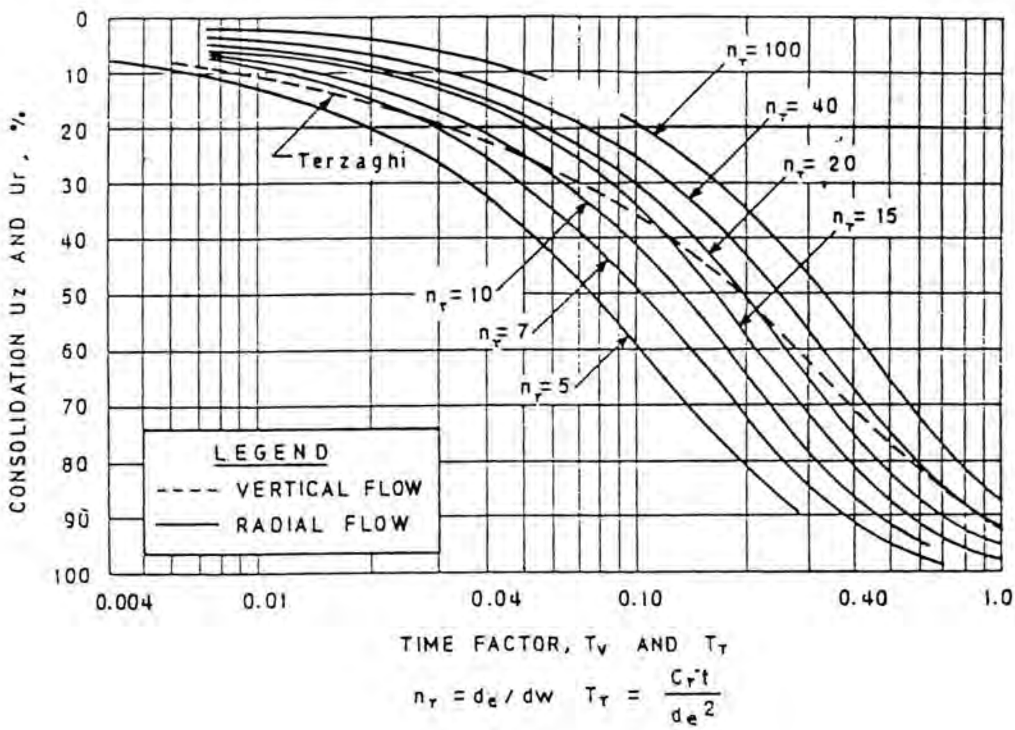


Fig. 1.5. Theoretical Solution for the Consolidation of Clay,Containing Vertical Drains (Barron, 1948)

The diameter of the equivalent cylinder of soil surrounding each drains '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. This is illustrated in Fig. 1.6.

In a square grid pattern

$$d_e = 1.13\ S \qquad \text{Eqn. 1.7}$$

and, In a triangular grid pattern

$$d_e = 1.05\ S \qquad \text{Eqn. 1.8}$$

Where S = spacing of vertical drains

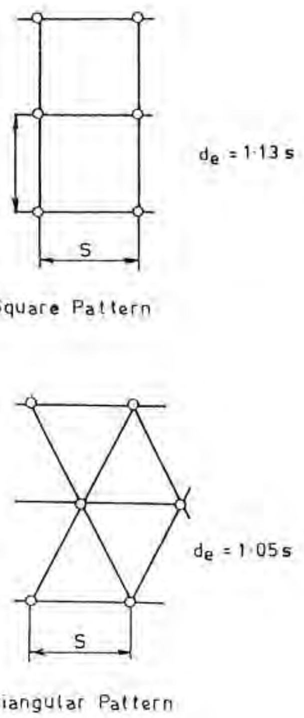


Fig. 1.6. Zone of Influence of Drain Well

Kjellman (1948) proposed that the spacing of vertical drains be fixed by considering only radial consolidation.

In case of band drains, the flow pattern around the drain is different from the cylinderical case. This problem is solved by modelling the band drain as an equivalent cylindrical drain.

Hansbo (1979) suggested that the equivalent diameter could be estimated from the consideration of the drain surface area. The equivalent diameter 'deq' is given by

$$deq = 2\,(b_b + t_b)/\pi \qquad \text{Eqn. 1.9}$$

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

Thus, for a typical band drain 100 mm wide by 4 mm thick, the equivalent diameter will be approximately 66 mm. Van den Elzen et.al., (1980) has proposed that a factor $\pi/4$ should be applied to this estimate. By applying this factor the equivalent diameter of 66 mm reduced to about 52 mm. A back calculated equivalent diameter from field installation of 100 mm band drains (Humpheson & Davies, 1981) was about 50 mm.

Atkinson et.al. (1982) state that probable error using the above analytical techniques in the estimation of time required for 90 per cent consolidation is about 17 per cent, but this error may be considered insignificant when compared with the errors arising from the effects of smear, well resistance and in correct assessment of soil drainage parameters. Atkinson et.al. (1982) also conclude that provided smear, well resistance and the soil drainage parameters are assessed and the drain geometry matches that assumed by the theories, both the Barron and Kjellman methods of assessing appropriate vertical drain spacings will give reasonable results.

1.7. 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.

This effect of smear was considered by Barron (1948). Barron considered the smeared zone as a homogeneous material having soil properties different from those of the in-situ soil. For analysis, Barron assumed the ratio of permeabilities of undisturbed and smeared zone as ten. According to him that if thickness of the smeared zone was 1/6 of the drain radius, the time to achieve a particular degree of consolidation would be increased by about 20 per cent. If the thickness of the smeared zone was increased to twice the drain radius then the effect would approximately double the consolidation time.

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. Field and laboratory experience by fabric manufacturers have shown that the optimum filter for drains used in clayey soils has an average pore size of about 10 - 20 μm.

1.8. Drain Resistance

The Terzaghi's and Barron's equations discussed earlier assumed unresrticted 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. However, for practical vertical drain installations for which n_r is approximately 7 to 15 and for de/H<1.0, the effect on the consolidation behaviour due to resistance of the vertical drains should not be significant.

1.9. 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. The number of soil parameters required depends upon the design method chosen. The analysis proposed by Kjellman (1948) is simple and requires only the knowledge of horizontal coefficient of consolidation. In the case of the analysis proposed by Barron (1948) their requirements vary. When the drain resistance is ignored, data regarding the values of coefficients of horizontal and vertical consolidation in the horizontal and vertical direction is adequate. On the other hand if drain resistance is taken into account, the values of the coefficients of permeability in the horizontal and vertical directions are also required in addition.

The horizontal and vertical coefficient of consolidation are measured directly in laboratory by conducting oedometer tests on undisturbed soil specimens. However, generally the laboratory measurement seriously underestimate the field values of coefficient of consolidation but give a better estimate of coefficient of volume decrease (m_v). Therefore a better method of estimating coefficient of consolidation values is by combining laboratory m_v values with field permeability measurements. For the determination of coefficient of permeability, constant head test yield more representative values. The soil investigation programme should also include measurement of strength and determination of index properties as they are essential for overall evaluation of soil characteristics as well as that of any improvements occurring due to the installation of vertical drains.

1.10. Cost Effectiveness

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.

In considering the cost of vertical drains the cost of sand is a major component in sand drains. 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.

Rate of placement and depth of treatment are also crucial factors to be considered. The overall placement cycle on the project is controlled by time spent in moving machinery from drain to drain and drilling and subsequent placing of drains. McGown et.al., (1982) suggest that drains at 1-2.5 m spacing and 10 -20 m depth allow for the highest production rates for most installation methods. Thouand band drains of 15 m length per installation rig can be conveniently placed per working week. The optimum depth for highest production and lowest cost is about 15 m. The guide to relative cost of sand drains sand wicks and band drains is given in Table 1.3.

Table 1.3. Relative Cost Per Unit Area of Site Treated by 15 m Deep Drains (McGown et.al., 1982)

Drain type	Common equivalent diameters; (m)	Square spacing; m*	Site area treated by each drain m2	Relative cost per 15 m deep drain %	Relative cost per unit area site treated m" #
Sand drains	300	2.22	4.93	790	380
Sand wicks	65	1.62	2.62	160	145
Band drains	50	1.55	2.40	100	100

* Assuming $t = 1/2$ year for $U = 90\%$ and $C_h = 5m^2$/year.

\+ Assuming band drains -100% cost'

\# Excluding all on site establishment costs.

1.11. Case Histories

1.11.1. Case history-1

Vertical sand drains were installed for improvement of soft clay for construction of an important industrial plant of Guariya State of Sao Paulo, Brazil. (Silveria et.al., 1977)

The subsoil consisted of a very thick bed of about 43 m of dark grey marine clay. The clay was very soft in consistency upto 35 m. This is followed, uptill 43 m, by a medium stiff deposit. The subsoil was interbedded with two thin strata of 1.5 m thick clayey sand layer at 18m and 35 m depth. Ground water was at surface. Several laboratory tests were conducted. The unconfined compression strength ranged from 1.6t/m^2 to 3.0t/m^2 at surface. The soil profile and laboratory test parameters obtained are shown in Fig. 1.7.

The requirment was to establish a fill to an elevation of 3.0 m to guard against high tides and to provide support for yards, parking lots, ponds, tanks and light structures with unit loads not exceeding 5.0t/m^2. Maximum differential settlement allowed for the structures was 1:200. The construction of the required fill would have caused large settlements extending for a period of more than twenty years.

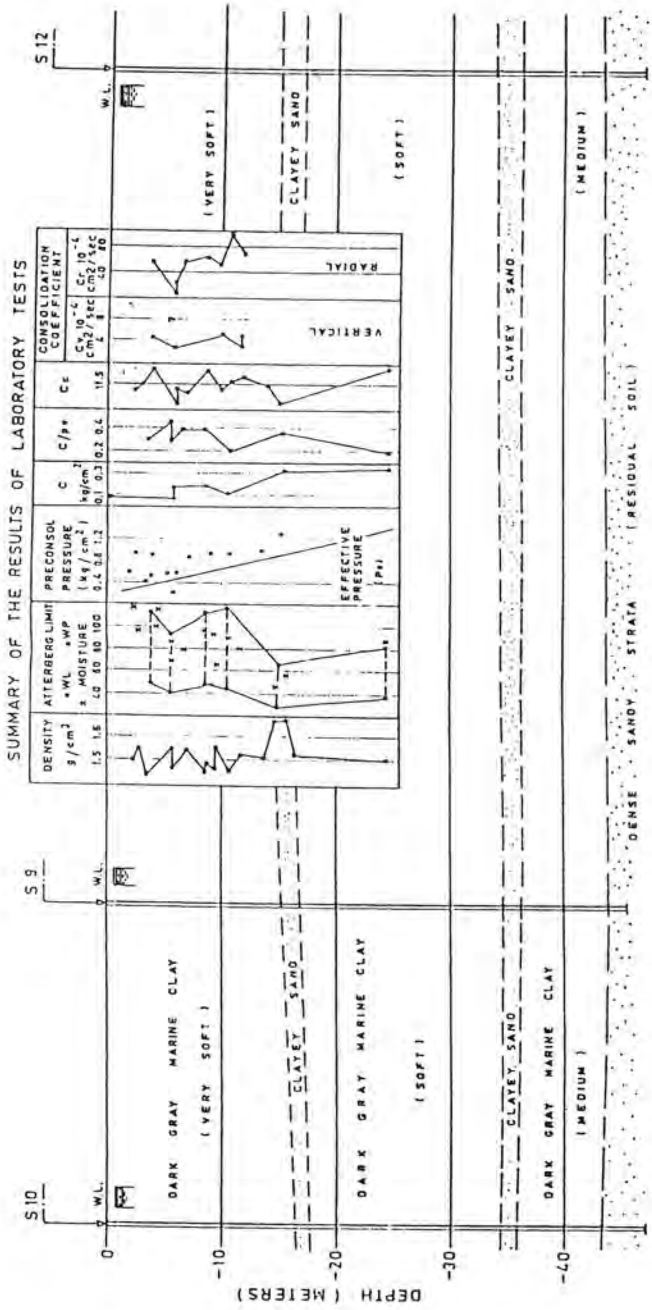


Fig. 1.7. Soil Profile and Parameters (Silveria et. al., 1977)

Vertical sand drain with preloading was considered an economical alternative. For the success of this technique following factors were considered.

(a) Rate of consolidation of soil
(b) Spacing of drains
(c) Method of installation

Result of laboratory tests showed that total settlement of about 148 cm will take place in 20 years, when no ground treatment is done. For sand drains at 3 m spacing the time required for the completion of 90 per cent settlement reduced to about 5 months. Two experimental field tests were done and the results confirmed the laboratory test data and theoretical design considerations. Non displacement sand drains were installed to reduce the effect of smear around the drains. The construction phase is shown in Fig. 1.8. The settlement stakes and piezometers were installed to record the settlements and dissipation of pore pressure beneath the loaded area. Although the settlements after the placement of backfill and the installation of sand drains has been fairly small but rapid rate of settlement was observed during the preloading and immediately afterwards. About 65 per cent to 80 per cent of the final total measured settlement took place during the period of preloading and subsequent first month. At the end of scheduled three months period of preloading, settlement at a number of places had essentially ceased. Therefore, surcharge fills were removed and taken to another area. The settlement observations are shown in Fig. 1.9.

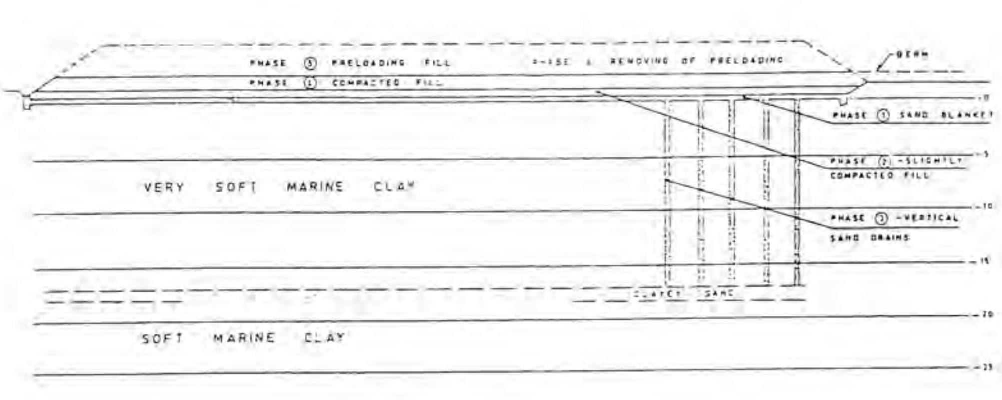


Fig. 1.8. Construction Phases (Silveria et.al., 1977)

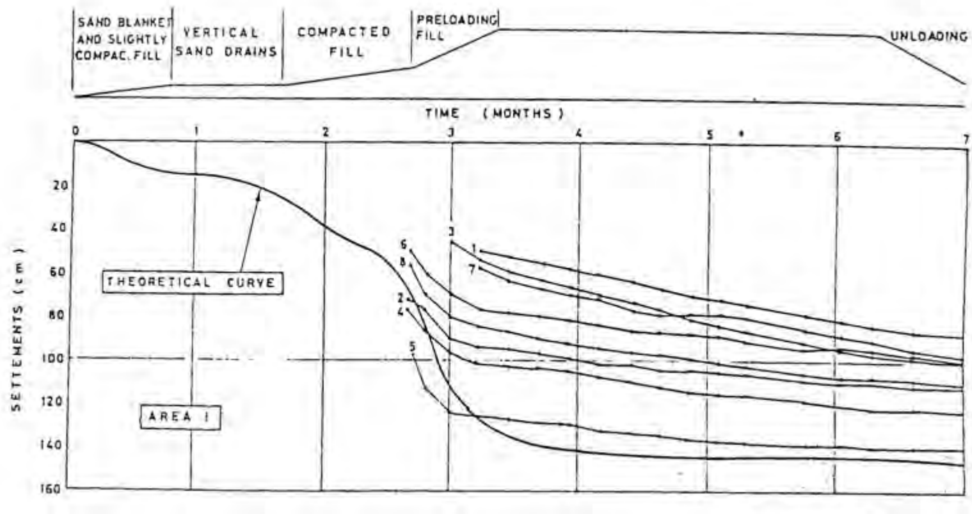


Fig. 1.9. Settlement Observations (Silveria et.al., 1977)

Silveria et.al. (1977) conclude that the performance of the structures constructed indicated that the method of improving soft ground by vertical sand drains and construction sequence was effective and successful.

1.11.2. Case history - 2

Vertical sand drain were installed for improvement of soft clay for construction of road embankment on soft clay for construction of road embankment for Eastern Express Highway between Sion and Thana in Bombay, (Mehra & Natarajan, (1962).

A section of the 22.5 km long Eastern Express Highway between Sion and Thana is routed over a length of 701 m of tidal flats. The depth of soft clay was of the orders of 3.6-4.5m. The unconfined compression strength was 0.15 kg/cm^2, compression index 0.6 and a initial void ratio 2.6. Underlying the soft clay stratum is an unfissured and relatively impervious layer of soft rock or murrum. The proposed embankment was 24.4 m wide at top with an average height of 2.1 - 2.4 m with side slopes of 2:1. Computations for the rate of consolidation under worst sections indicated that 90 per cent consolidation under embankment load would take place in not less than 20 years. In other section, computations indicated 9 years as the minimum time period to accomplish 90 per cent consolidation.

Vertical sand drains with use of cinder, a light weight material for embankment construction was considered as an economical alternative in terms of time and money.

In the embankment design due consideration was given to the creep of soft clay which causes progressive subsidence of embankment under constant stress. From laboratory test result the ratio of creep strength to the ultimate strength of clay was 0.6. The load intensity due to the weight of the embankment was kept lower than 0.6 of the bearing capacity calculated on the basis of ultimate strength.

A sand blanket of 0.45 m thick was placed over the ground surface. After a partial height of about 1.5 m was built, vertical sand drains were installed. The diameter was 25 cm and were installed in a triangular pattern with a spacing of 3m. The length of drains varied from 3.6m to 4.5 m. Three thousand drains were installed. The schematic sketch of embankment construction is shown in Fig. 1.10. A time lapse of 6 months was given for dissipation of pore water pressure and subsequent gain in strength. Therefore the remaining height of cinder embankment plus 4.5 cm of road pavement was completed making use of this increased strength. The embankment was fully instrumented. The typical observed and calculated time settlement curves are shown in Fig. 1.11 The computed and the observed magnitude and rate of settlement are in close agreement.

The use of light cinder as a material for highway embankment construction opens up new avenues for the profitable utilisation of this waste product whose disposal has posed serious problems.

1.11.3. Case history - 3

Large scale field trials were undertaken by CRRI, (1971) with the objective of improving the soft marine clay subsoil to support the load of 9 m height of iron ore stack at the Visakhapatnam Port. The field trials involves both the use of sand drains and sand wicks in combination with preloading. The objective of field trials was to check whether the anticipated strength gain of the soft clay due to preloading in combination with vertical drains was adequate to carry the desired load and was an economical method of improving the soft soil.

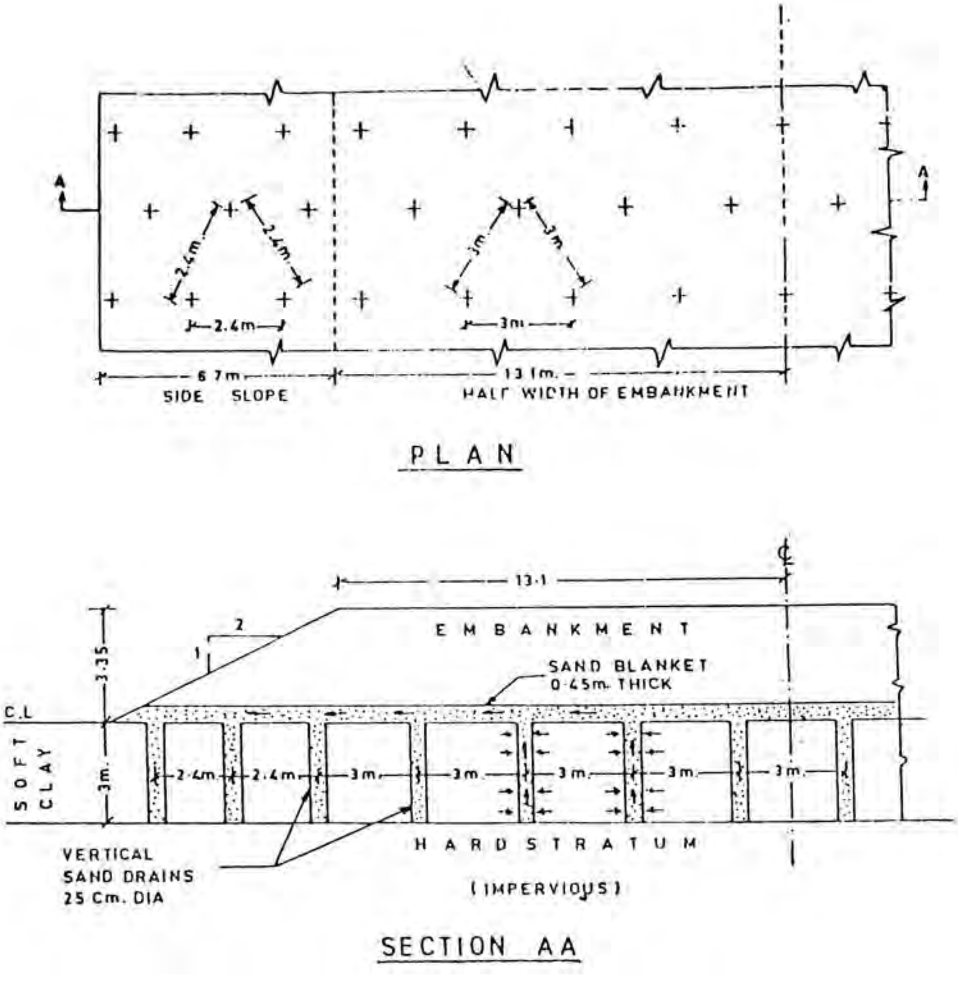


Fig. 1.10. Embankment Construction with Sand Drains (Mehra & Natarajan, 1962)

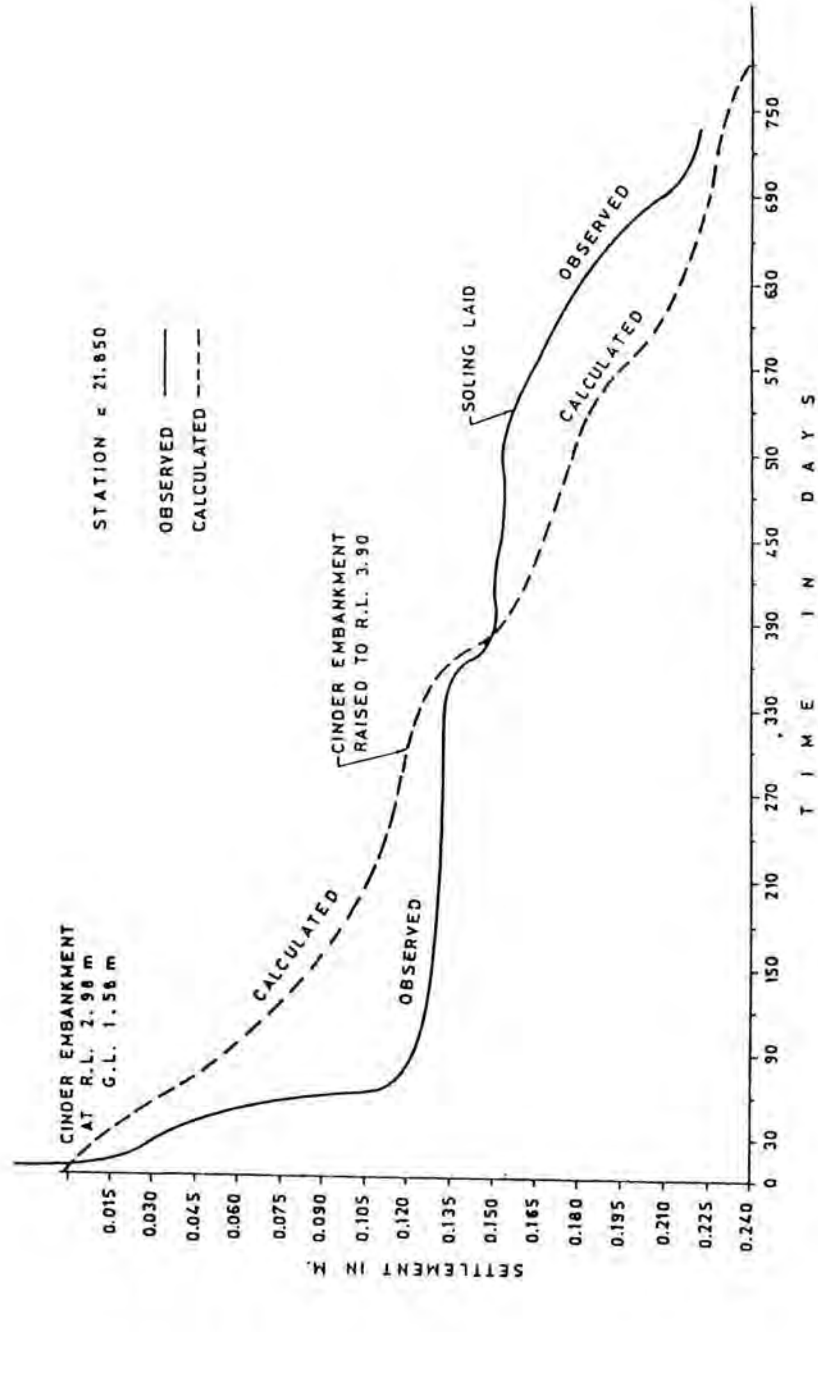


Fig. 1.11. Typical Time Settlement Curves Eastern Express Highway (Mehra & Natarajan, 1962)

Based on geotechnical investigation the substrata, it was found that upto a depth of 6m a sand deposit exist and this was followed by soft marine clay extending upto a depth 18 m below the ground level. The soft clay was followed by a deposit of stiff clay upto a depth of 21.0 m. Three test areas were made viz. Test Area 1, Test Area 2, Test Area 3.

Test Area - 1: In this area 35.5 cm diameter sand drains were installed at 2.14 m metres c/c in a triangular pattern penetrating up to the full depth of soft clay layer. In all 116 sand drains were installed.

Test Area - 2: In this area 6.35 cm diameter sand wicks were installed at 1.22 m c/c in a triangular pattern penetrating to the full depth of the soft clay layer. In all 357 sand wicks were installed.

Test Area - 3: No Sand drains or sand wicks were installed in this area. The fill was placed in this test area in the same way as in other two test areas. The test areas were located at 61 metres c/c. The test embankment in all the test areas was loaded to a height of 9.0 m in stages. The test areas were instrumented by installing Casagrande open stand pipe piezometers, plateform type settlement gauges and displacement stakes. The observed settlements are shown in Table 1.4. The variation of undrained shear strength in the soft clay deposit was determined by performing in-situ vane tests both before and after the ground improvement. Based on in-situ Vane shear tests the Su/Pc ratio for the Visakhapatnam clay was of the order 0.30. The comparison of strength gain before and after loading in sand drain and sand wick area is shown in Tables 1.5 and 1.6.

Table 1.4. Observed Settlements

Test Areas	Lapse of time after full loading (months)	Observed settlement	
		Maximum (mm)	Average (mm)
	2	1006	925
	3	1069	993
Test Area 3	4	1110	1023
(Untreated)	5	1156	1071
	6	1179	1105
	2	1166	1036
Test Area 2	3	1280	1148
(Sand Wicks)	4	1361	1219
	5	1443	1298
Test Area 1	2	1260	1158
(Sand Drains)	3	1361	1255

Table 1.5. Comparison of Strength Before and After Loading in Sand Drain Area

R.L. (m)	Depth below the top of the clay deposit (m)	Undrained shear strength kPa Before loading	170 days after loading	Strength gain
-2.51				
-2.97	(top of clay)			
	0.45	17.12	29.01	11.88
-3.88	1.06	19.57	31.02	11.45
-5.41	3.00	28.86	37.90	12.23
	(quarter depth)			
-6.93	4.41	29.35	39.43	10.08
-8.45	5.94	33.76	40.70	6.95
-9.98	7.46	36.20	42.46	6.26
-9.98	9.00	38.65	44.28	5.63
	(three-quarter depth)			
13.03	10.50			

Based on the above field trial preloading technique in conjuction with sand drains/sand wicks was used successfully for stacking the iron ore upto 7.5m height in about 4-5 years.

1.12. Conclusions

(1) 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.

(2) The effects of smear and drain resistance should be considered in the design of vertical drains. The effects of smear caused by installing the drains by displacement method can largely be overcome by the correct choice of drain filter fabric. Internal resistance of the drain can have a large effect on the consolidation process. The cost benefit of closer spacing at shallow depth against increased depth of treatment should be a prime design consideration.

(3) Many installation methods exist but as shown in Table 1.1, they are grouped largely in three categories (i) Displacement method, (ii) Drilling method, (iii) washing methods. Generally the drains are installed by any of these methods depending upon the site conditions and availability of equipment.

(4) Synthetic fabric drains have shown wide acceptance. They have the lowest overall cost amongst other types of drains. They are less bulky and easier to handle at site. However they require sophisticated installation techniques.

(5) Predictions of rates of consolidation cannot be made reliably because of difficulty in determining representative value of coefficient of radial

Table 1.6. Sandwick Area (Test Area 2)
Comparison of Strength Before and After Loading

R.L. (m)	Depth below the top of the clay deposit (m)	UNDRAINED SHEAR STRENGTH - kPa						Gain in undrained shear strength in a period of 170 days - kPa		
		BORE HOLE 'A'			BORE HOLE 'B'					
		Before loading*	170 days after loading	Gain in Strength	Before loading	175 days after loading	Gain in Strength	Maximum	Minimum	Average
-2.9	0	15.6	27.9	12.3	13.7	26.5	12.7	12.7	12.2	12.5
-3.3	0.76	21.6	29.4	7.8	19.6	29.4	9.8	9.8	7.8	8.8
-5.9	3.05 (quarter depth)	27.0	32.3	5.4	24.5	32.3	7.8	7.8	5.4	6.6
-9.0	6.10	31.9	34.3	2.5	30.9	34.8	3.9	3.9	2.5	3.2
-12.0	9.15	36.3	38.2	2.0	36.0	38.7	2.0	2.0	2.0	2.0
-15.0	12.19 (bottom of soft clay)	41.2	41.2**	—	41.2	41.2**	—	—	—	—

* estimated by interpolation
** extrapolated probable values.

consolidation,C_r. There is inherent uncertainity in accounting for the effects of smear and disturbance during drain installation.

(6) In most field situations, 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 techniques.

(7) Synthetic fabric drains have not yet been widely used in India. It is suggested that laboratory and field investigation concerning the use of these type of drains may be taken up.

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2. STONE COLUMNS TECHNIQUE

2.1. Introduction

Among the recent developments which involve strengthening of soft sub-soils, Stone Column Technique has proved to be an effective technique. Stone Column are used to support structures overlying both very soft to firm cohesive soils and also loose silty sands. Stone Column consist of granular material compacted in-situ in long cylindrical bore-holes. They have been found to be economical and effective in a number of countries, especially the developing countries.

The development of this technique has proceeded from field to the laboratory. Exact solutions for evaluation and prediction of the behaviour of soils strengthened by stone column are not available. However, on the basis of case histories and extensive laboratory studies, a conservative semi-emperical designs approach is adopted in practice.

Stone Columns were used in 1830 by French Military Engineers to support the heavy foundations of the iron works for the artillary arsenal at Bayonne that was founded on soft estuarine deposits. The Columns were two metres deep, 0.2 m in diameter and supported loads of 10 kN each. They were constructed on a progressively finer grid. The Columns reduced the expected settlement by a factor of four.

The Stone Columns were subsequently forgotten and their modern origins truly began 50 years ago, in Germany, with the development of the idea of compacting cohesionless soils both above and below the water table by a vibrator. However, in early 1960's, vibration technique was used for improving cohesive soils with better results. Methods of installation of densely compacted sand columns were developed in Japan concurrently which led to the development of vibro compozer method, Murayama (1958). Presently, with increased capacities of vibroflots and alternative method of construction, such as, the rammed stone columns resulted in economic installation of stone columns upto 20 m length, 0.5 m to 1.5 m diameter which can support loads upto 300 kN, Hughes et.al. (1975), Carran (1974), Mckeena et.al. (1975), CRRI (1986).

2.2. Advantages of Stone Columns

Advantages of stone columns over other conventional methods of ground improvement are :

(i) It provides increase in load carrying capacity.

(ii) It provides a significant reduction in total and differential settlements.

(iii) Being granular and freely draining, consolidation settlements are accelerated and post construction settlement are minimised.

(iv) Installation is relatively simple and involves low energy input on manual labour. It is suited particularly for developing countries.

(v) It is cost effective.

(vi) It increases the resistance of soil to liquefaction.

(vii) It helps in improving the slope stability of both embankments and natural slopes.

2.3. Installation and Construction Techniques

The installation of stone columns is more of an art rather than an exact science, therefore, it requires a judicious quality control of various parameters, in the field. A description of the installation techniques, and the necessary quality control for the satisfactory performance, is given.

Installation of stone columns is carried out by vibroflotation and ramming techniques. Sand compaction piles which are similar in behaviour to stone columns are installed by vibro-compozer method.

2.3.1. Vibroflotation process

Vibro-replacement (wet process) and vibro displacement (dry process) are two techniques of this process for the installation of stone columns with the vibroflot. Essential features of vibroflot are shown in Fig. 2.1.

(i) Vibro-replacement Installation (wet process)

Fig. 2.2 illustrates the use of vibrator to form the stone columns. 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. After forming the hole of the required depth, the hole is flushed out repeatedly by raising and dropping the probe several times.

An important factor in successfully forming the stone columns is to keep water flowing from the jets all the times. A flow rate of 11-15 m^3/hr is maintained throughout the construction.

The hole formed by the vibrator is backfilled in stages with granular materials with specified characteristics. Usually, the borehole is backfilled in 1 m 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. In this way, a compacted stone columns is formed. Because of the lateral displacement of the stone during vibration, the completed diameter of the column is always greater than the initial diameter of the borehole.

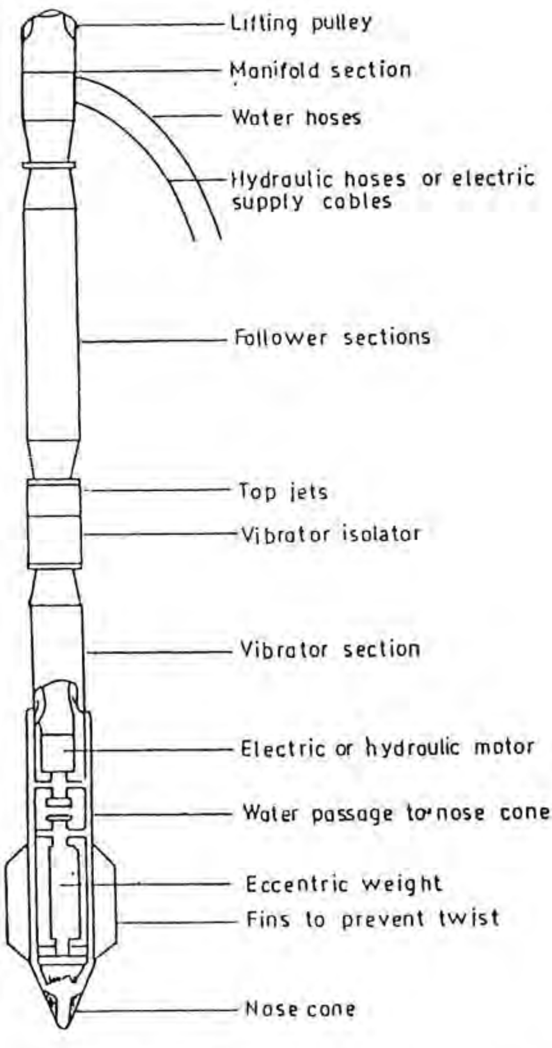
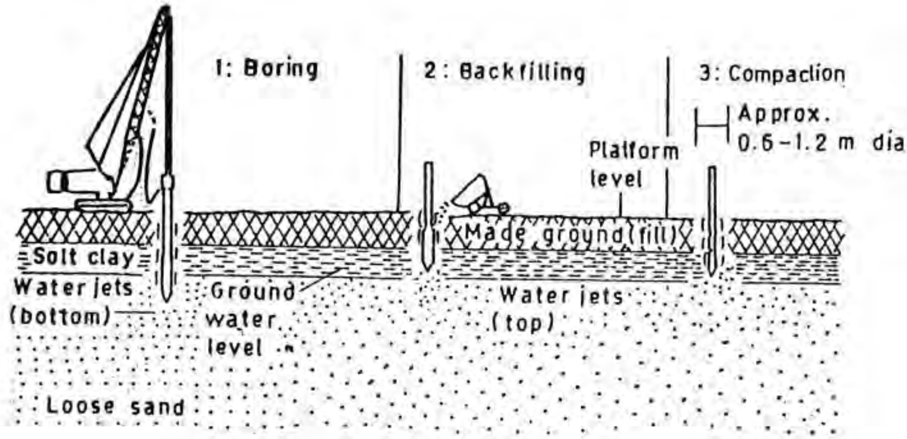


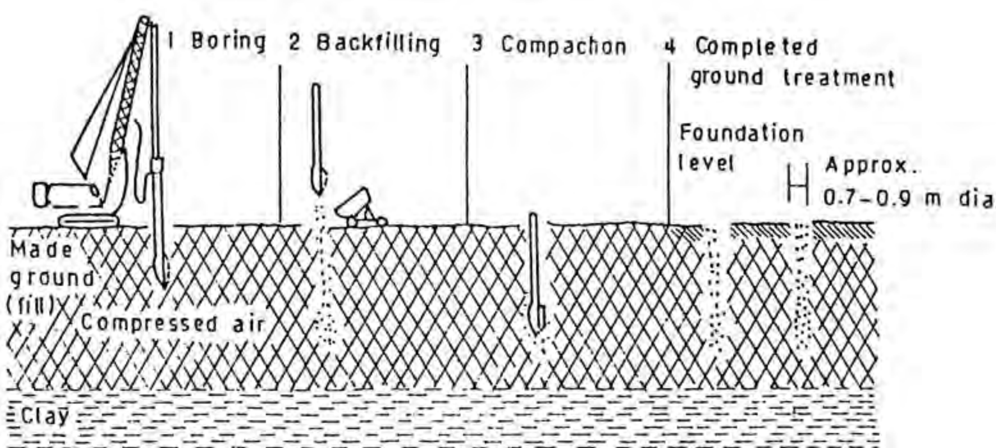
Fig. 2.1. Essential Features of Vibrofloat (After Greenwood & Thomson, 1984)

This method should be used at sites consisting of very soft soils, unsupported holes and where high ground water conditions exist. This procedure also prevents the setting up of excess pore-water pressure during initial penetration. The average reported construction rate are 1-2 m/min. for excavation and 0.5-1 m/min. for backfill and compaction.

The principal disadvantage of wet technique is that it involves 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.



Vibroreplacement Technique



Vibrodisplacement Technique

Fig. 2.2. Vibrofloatation Process - Sequence of Operations

(ii) 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. Although, no jetting water is used in this method, it is essential to use compressed air to break the suction which develops when the vibrator is withdrawn from the soft saturated cohesive soil. The compressed air generates a back pressure in the hole, thus, improving the stability of the hole.

When the vibrofloat penetrates into the ground, it tends to displace the soil laterally resulting in either a compaction of the soil or a heave at the surface. The borehole diameter formed is of the same order as that of the vibrofloat. Hence, it has to be taken out to facilitate the backfilling of the borehole, with granular material. The procedure for compaction of the backfill remains the same as described in the wet process method.

The major limitation lies in the difficulty in forming uncontaminated stone columns because the radial displacement seriously distorts and remoulds the surrounding soil, inducing excess pore water pressure which cannot dissipate rapidly.

(iii) Rammed Stone Columns

The technique is illustrated in Fig. 2.3. 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. If a single section of casing is used then it is provided with flat door windows generally at 1.0 m 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 of recommended size is placed in the borehole through the window close to ground level by chute. These windows basically eliminate the need of scaffolding platform to place charge through the casing and thus reduces the cost of construction. The amount of backfill is restricted such that a column of not more than 2 m height is formed at a time. After the backfill is placed the casing is withdrawn by not more than 1.25 m. Withdrawing of casing is done by antihammering with the hammer. Generally, a hammer of 15 kN is used for compaction falling through a height of 1.4-2.0 m to impart an energy of 20-30 kN per blow. Next charge is placed and the casing is withdrawn further and compaction is carried out till stone column upto ground level is formed.

Alternatively, the columns are installed through a driven tube with a dispensable shoe. It is displacement type of method. The method is advantageous as there is little ingress of water and installation procedure is fast. The method of compaction remains the same as described above.

Equipments commonly used for bored and driven piles are used for rammed stone columns. Light bored piling equipment with 20-40 HP winches can be used.

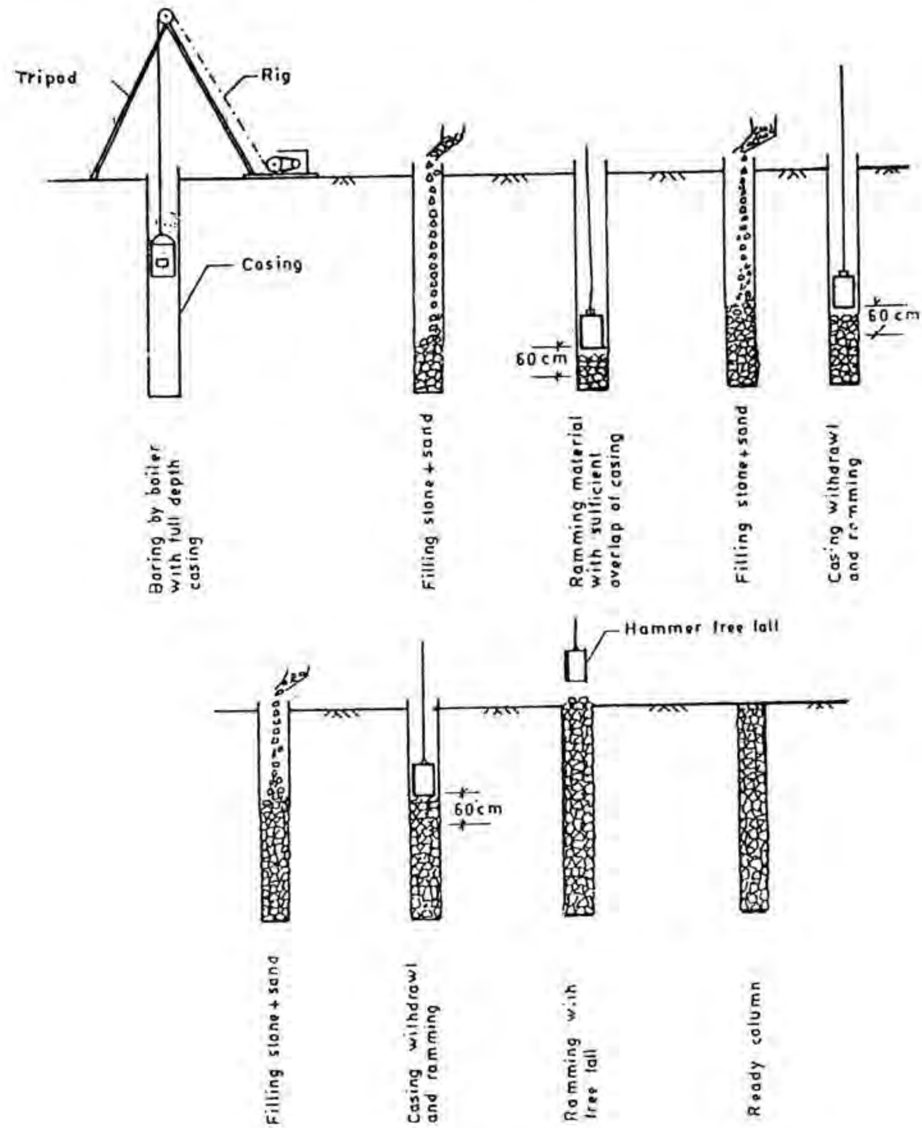


Fig. 2.3. Installation of Stone Columns Through Cased Boreholes (Datye, 1982)

2.3.3. Sand compaction piles

The method of sand compaction piles was introduced in Japan in 1958 (Murayama, 1958). The vibratory and compaction piles are installed by means of equipment which consists of a crane, vibrator, installation pipe and accessories, such as, sand supply, suspending wire and compressed airline. The installation process is shown in Fig. 2.4. The casing pipe is installed on a suitable amount of gravel and mixture at the prescribed point on ground surface. The casing is driven downward by the operation of the vibrator and a simultaneous relaxing of the suspending wire. The granular mass works as a wedge and is prevented from enlarging inside the casing because of the unique Iris structure of the casing tip. 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.

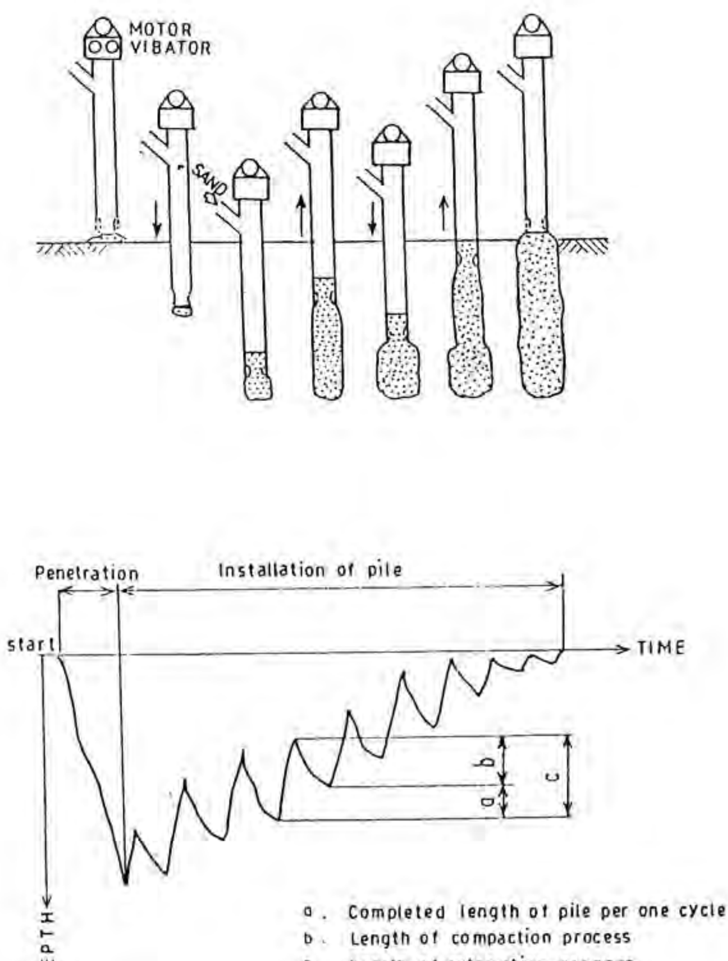


Fig. 2.4. Method of Installation of Sand Compaction Piles

2.4. Backfill Gradation

The granular material should develop a high angle of internal friction under the energy of compaction. Crushed stone, chemically inert, hard and preferably angular in shape should be used. A mixture of crushed stone of 23-75 mm in size and medium sand in the proportion of 5:1 to 2:1 by volume gives a well compacted mass with sand filling the voids of the crushed stone, Datye et. al. (1981). The crushing strength of aggregate should be high so that the aggregate does not get crushed under the energy of compaction.

Single sized gradations should not be used because the voids in such a backfill gets filled with clay slurry which prevents them from acting as drains.

2.5 Estimation of Stone Column Diameter and Spacing

The diameter of the stone column depends upon the undrained shear strength of the soil, gradation of the stone aggregate and construction method.

Stone columns are usually constructed in an equilateral pattern.Typical column spacing are usually 1.2 - 2.5 m. A minimum of 1.5 m spacing is imposed because of potential construction problems. Higher concentration of columns is provided near the periphery of the foundation to compensate for reduced capacity of the column. The capacity of columns near the periphery is less than the central ones as they do not receive the full surcharge effect.

2.6. Construction Control

The degree of compaction then can be achieved depends upon several factors, such as, (i) consistency of the surrounding soil, (ii) gradation, (iii) shape and quality of granular backfill, (iv) size of borehole, (v) type of vibrofloat (vi) weight, height of fall and number of blows of rammer in case of rammed columns and (vii) type of construction method.

In vibro-constructed columns, control of quality is based primarily on the electric current consumed during the compaction and the consumption of the aggregate. Generally repenetration of vibrofloat in the column is continued till it develops a minimum reading of at least 40 ampere more than the free standing ampere draw on the motor. The next change is placed subsequently.

In case of rammed stone columns, the degree of compaction is governed by the set criteria, i.e., penetration of rammer into the filled material for a given number of blows. Control of set along with the measurement of the consumption of aggregate ensures a uniform quality. A typical specification specifies the weight of rammer as 1.5 KN, height of drop as 1 m and a set of cm for 25 blows.

2.7. Soil Stone Column Interactions

Stone Columns interact with the intervening soil so as to increase the average shear strength. By virtue of the rigidity endowed by the coarse backfill, the stone columns help to reduce the overall compressibility of the treated soil. At the same time, they act as drains and accelerate dissipation of pore pressures.

The complex and undefined nature of soil-column interaction enables only a qualitative understanding of the load response behaviour of the composite strengthened soil. 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 stresses are transferred by arching action. Depending on the strength of the soil and the diameter of the stone column the dispersion angle in arching action varies. The remainder of the load is carried by soft cohesive soil. The large initial vertical stresses on the top of the stone columns produce large radial stresses and strains in the roughly cylindrical walls of the columns, thereby, mobilising the passive resistance of the surrounding soil. The magnitude of the radial strains necessary to develop the maximum passive soil resistance, is low, because of the considerable radial displacement of the soil that has already taken place during the formation of stone columns. The vertical strains in the stone columns produced by the initial applications of the loads cause transfer of load from the yielding column to the soil. As the consolidation of the intervening soil takes place, load is transferred from the soil to the stone columns. 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.

A simplified load transfer mechanism proposed by Datye (1982) is shown in Fig. 2.5.

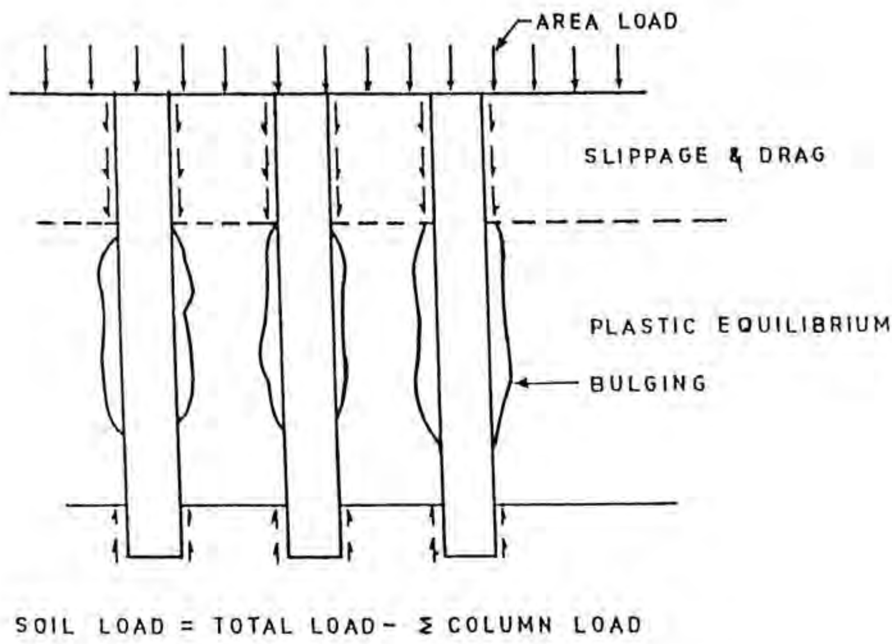


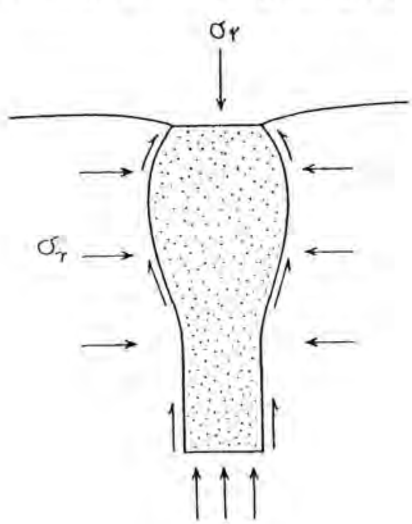
Fig. 2.5. Simplified Load Transfer Mechanism (Datye, 1982)

2.7.1. Failure mechanisms

Based on above load transfer conditions, it may be stated that stone column may fail by any of the following modes :

(1) General shear failure
(2) Pile action failure
(3) Bulging failure

The possible modes of failure in a single and group of stone columns are shown in Figs. 2.6 and 2.7.



STRESSES ACTING ON THE STONE COLUMN

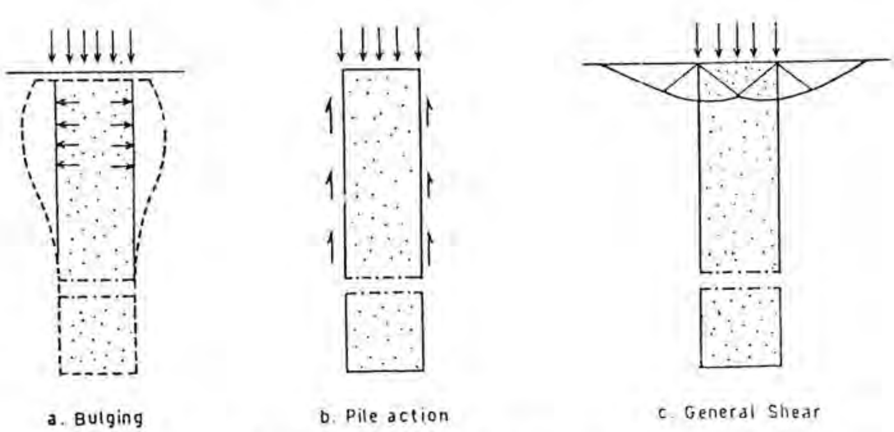


Fig. 2.6. Modes of Failure of Single Stone Column

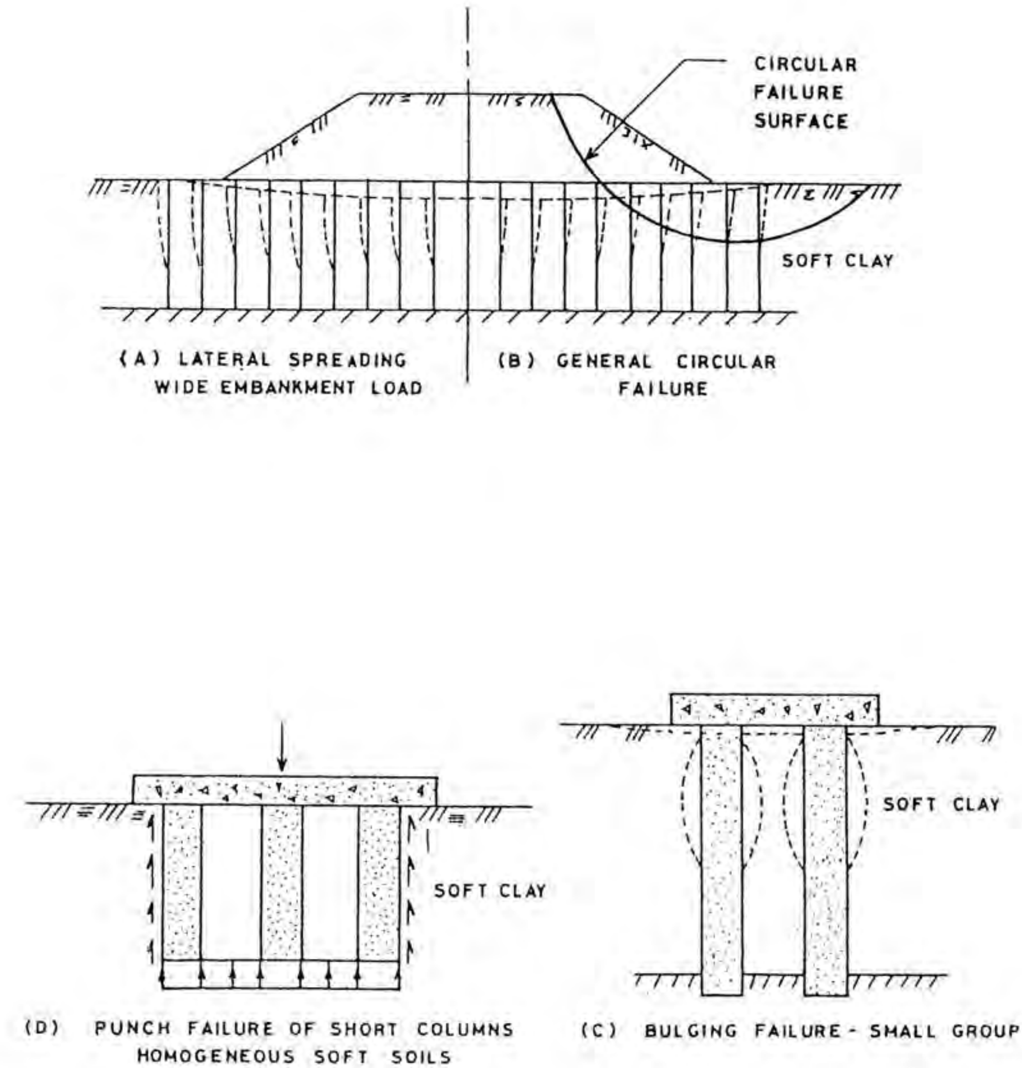


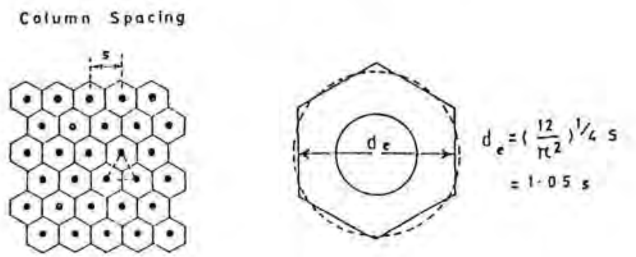
Fig. 2.7. Failure Modes of Group of Stone Columns

2.8. Determination of Ultimate Load Capacity

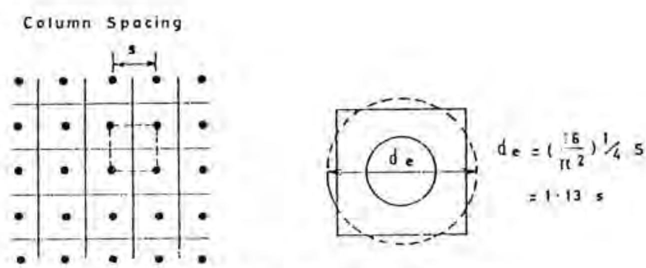
In the calculation of utimate bearing capacity the possible modes of failure should be considered. Th . bearing capacity depends upon the strength parameters of the compacted material, column diameter and properties of the sub-soil.

2.8.1. Unit cell concept

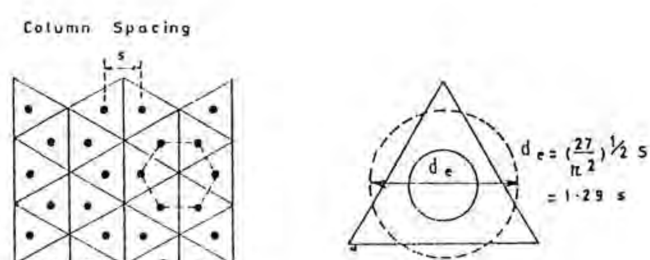
Stone columns are constructed in an equilateral triangular pattern. A tributary area surrounding the stone column is associated with it. Although, a tributary area forms a regular hexagon about the stone column, it can be closely approximated as an equivalent circle having the same area. The resulting equivalent cylinder of material having the diameter d_e enclosing the tributary soil and one stone column is known as unit cell. Various stone column arrangements showing the domain of influence of each column is shown in Fig. 2.8.



(A) TRIANGULAR ARRANGEMENT OF STONE COLUMNS



(B) SQUARE ARRANGEMENT OF STONE COLUMNS



(C) HEXAGONAL ARRANGEMENT OF STONE COLUMNS

Fig. 2.8. Various Stone Column Arrangements Showing the Domain of Influence of Each Column

2.8.2. Ultimate capacity of a single stone column

(i) Based on Passive Pressure Condition

Greenwood (1970) presented an approach for estimating the ultimate load of a single column. The passive resistance of a uniform soil is given by :

$$\sigma_P = \Upsilon Z\ K_p + 2su\ \sqrt{K_p} \qquad \text{Eqn. 2.1}$$

Assuming the column material to have yielded and the vertical stress to be the principal stress, the ultimate stress on the column is :

$$q_{ult} = \sigma_p.\ K_{ps} \qquad \text{Eqn. 2.2}$$

Υ - Unit weight of soil
su - Undrained shear strength of soil
σ_p - Passive resistance of soil
Kp - Rankine passive pressure coefficient of soil
K_{ps} - Rankine passive pressure coefficient of stone column
Z - Depth below ground surface

(ii) Based on Expansion of Cylinder

Hughes and Withers (1974) presented a method for predicting the ultimate load of a stone column which has been developed from the results of the plasticity theory. The ultimate strength of a column is governed by the maximum lateral resistance of the soil around the zone of column which bulges. This observation has been used in development of the model. In their approach the elastic plastic theory given by Gibson and Anderson (1961) for a frictionless material and for an infinitely long expanding cylindrical cavity has been used for predicting the undrained and ultimate lateral stress.

$$\sigma_{rl} = \sigma_{r}o + su\ (1 + \ln \frac{E}{2\ su\ (1+\nu)}) \qquad \text{Eqn. 2.3}$$

The result of quick pressure meter tests show that above equation can be approximated by

$$\sigma_{rl} = \sigma_{r}o + u + 4\ su \qquad \text{Eqn. 2.4}$$

If the granular material in the bulged zone has yielded, then the ultimate vertical stress that the column can carry is given by :

$$q_{ult} = K_{p}S\ \ (\sigma_{r}o + 4su + u\ ) \qquad \text{Eqn. 2.5}$$

Where

σ_{rl} - Ultimate lateral stress
σ_{ro} - Effective radial stress

E - Young modulus of soil
υ - Poisons ratio of soil
K_{ps} - Rankine pressure coefficient of stone column
u - Pore water pressure
q_{ult} - Ultimate bearing pressure

(iii) Based on Cavity Expansion Factors

Vesic (1972) has presented a theory for the expansion of a cylindrical cavity in an infinite soil taking into consideration the soils having both friction and cohesion. The cylinder is assumed to be infinitely long and the soil is assumed to behave as a rigid plastic material.

The ultimate lateral resistance developed by the surrounding soil is expressed as :

$$\sigma_{rl} = cF'_c + q\,F'_q \qquad \text{Eqn. 2.6}$$

The cavity expansion factors F'_c and F'_q are shown in Fig. 2.9. These factors are functions of angle of internal friction of surrounding soil and the rigidity index. The rigidity index is expressed as :

$$Ir = \frac{E}{2(1+\nu)\,(c + q\tan\phi)} \qquad \text{Eqn. 2.7}$$

The ultimate vertical stress is given by :

$$q_{ult} = (\,cF_{c'} + qF_{q'}\,)\,K_{ps} \qquad \text{Eqn. 2.8}$$

where

c - Cohesion intercept
q - Mean isotropic stress at average depth of bulge
$F_{c'}, F_{q'}$ - Cavity expansion factors
E - Young modulus of soil
ν - Poiseons ratio
Ir - Rigidity index
ϕ - Angle of internal friction of soil

2.8.3. Ultimate capacity of group of stone columns

The ultimate bearing capacity of a stone column is predicted in practice, by estimating the capacity of a single column by the various methods described in the preceding section and multiplying the capacity of the single column by the number of columns in the group.

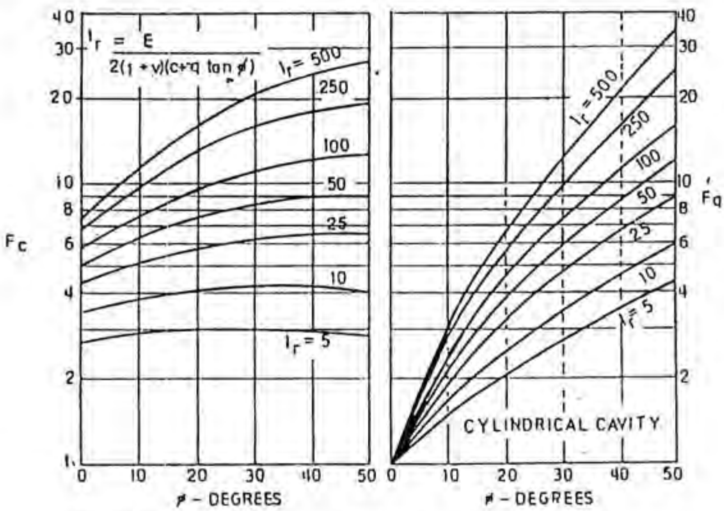


Fig. 2.9. Vesic Cavity Expansion Factors (After Vesic, 1972)

2.9. Settlement Prediction

Theories to predict settlements of ground reinforced with stone columns is subject to several uncertainties, primarily because the installation techniques modify and influence the stress strain behaviour of the column and the surrounding soil. Present available methods for calculating settlement can be classified as :

(1) Approximate methods which make important simplifying assumptions.

(2) Sophisticated methods based on elasticity and or/plasticity theory (such as finite element method) which model the material and boundary conditions.

Various methods to predict settlement of the stone column reinforced ground are reviewed in the following section.

2.9.1. Settlement of single stone column

(i) Elastic Continuum Approach

Mattes and Poulos (1969) have presented a solution for predicting the settlement of single compressible pile and it can be directly applied to the stone column as a first approximation. The settlement on the top of the compressed pile is given by :

$$\rho = \frac{P}{E_s L_H} Ip \qquad \text{Eqn. 2.10}$$

where

P - Applied load

E_s - Young's modulus of the column
Ip - Displacement influence factor given in Fig. 2.10
L_H - Length of stone column

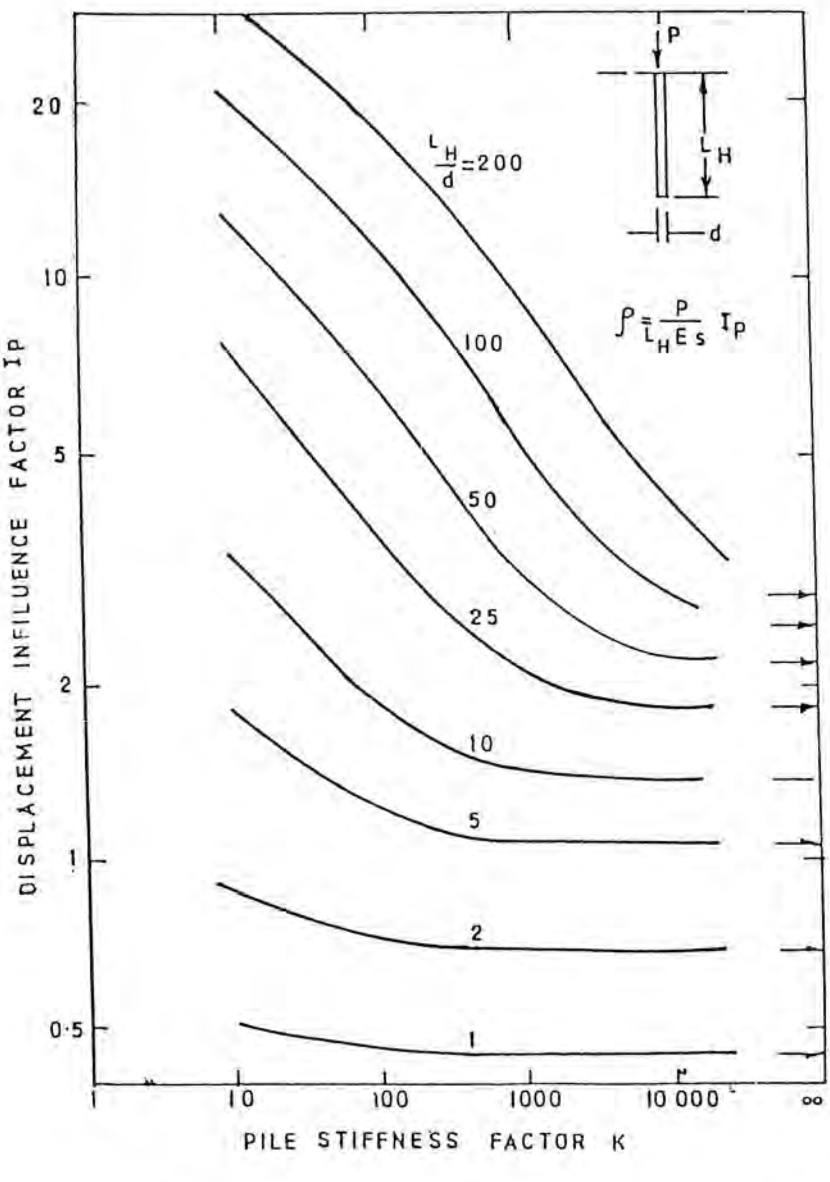


Fig. 2.10. Displacement Influence Factors (After Mattes & Poulus, 1969)

(ii) Hughes and Withers Approach (1975)

Hughes et. al. (1975) outlined an approach which enables the prediction of the settlement characteristics of an isolated stone column. The main assumption is that the column expands radially as settlement occurs with constant volume being maintained. The calculation is simplified by dividing the column into layers, the total settlement being the sum of the contributions of each layer.

$$\rho = \sum_{i=1}^{m} \delta i$$

$$\rho i = 4\, H\, \rho_{ri/d} \qquad \text{Eqn. 2.11}$$

ρi - Settlement of the layer considered
H - Thickness of layer considered
$\rho_{ri/d}$ - Radial strain of the layer considered

(iii) Equilibrium Method

The equilibrium methods described by Aboshi (1979) is a method used in Japan for estimating the settlement of the sand compaction piles.

This method offers a simple realistic approach to the estimation of the reduction in settlements. The following assumptions have made in development of this method :

(1) Unit cell concept is valid
(2) Total vertical load applied equals the load on the column and the soil
(3) Vertical displacement in the column and the soil are equal.
(4) Uniform vertical stress exists throughout the column length.

The change is vertical stress in the clay due to the applied load is equal to

$$\sigma c = \mu_c . q_a \qquad \text{Eqn. 2.12}$$

where

q_a - Average applied external stress
μ_c - Ratio of stress in soil to the average stress

The settlement ratio of the ground treated with stone column to that experienced by the untreated ground as obtained from Aboshis' method is given in Fig. 2.11.

(iv) Priebe Method

The method proposed by Priebe (1976) for estimation of reduction in settlement, uses unit cell approach. The stone columns is assumed to be in a state of plastic equilibrium under

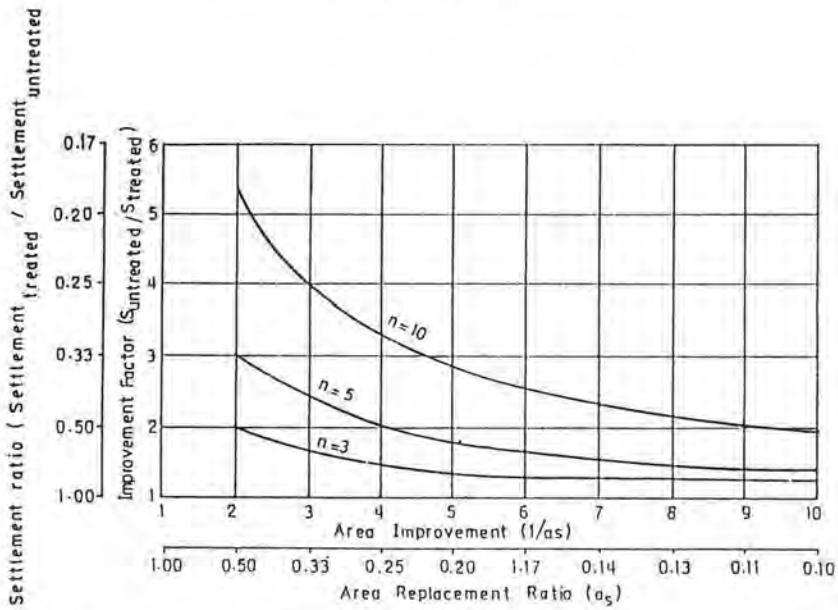


Fig. 2.11. Maximum Reduction in Settlement from Equilibrium Method (Aboshi, 1979)

triaxial stress state. The soil is considered as an elastic material. The stone column is assumed to be in compressible, and the change in volume within the soil is directly related to the vertical shortening of the column.

The settlement ratio of the ground treated with stone column to that experienced by the untreated ground as a function of area replacement ratio (a_s) and angle of internal friction of stone column ϕ_s is given in Fig. 2.12.

2.10. Recent Developments in Stone Columns

A large number of field installations of stone columns have been carried out in India in the past decade using rammed column technique as well as vibrofloat technique. However, very few experimental investigations on different aspects of stone column behaviour have been taken up nor are the available theoretical developments adequate to provide satisfactory predictions concerning the immediate as well as long term settlement behaviour of soft clay improved with stone columns. Laboratory studies on the use of stone column for soft ground improvement have been taken up by CRRI in 1986 and are continuing (Rao et. al. 1988, 1989, 1990). Following different aspects have been studied.

I. Laboratory studies concerning the load carrying mechanism of stone columns

(a) Drainage aspect of stone columns
(b) Critical depth of stone columns for load transfer

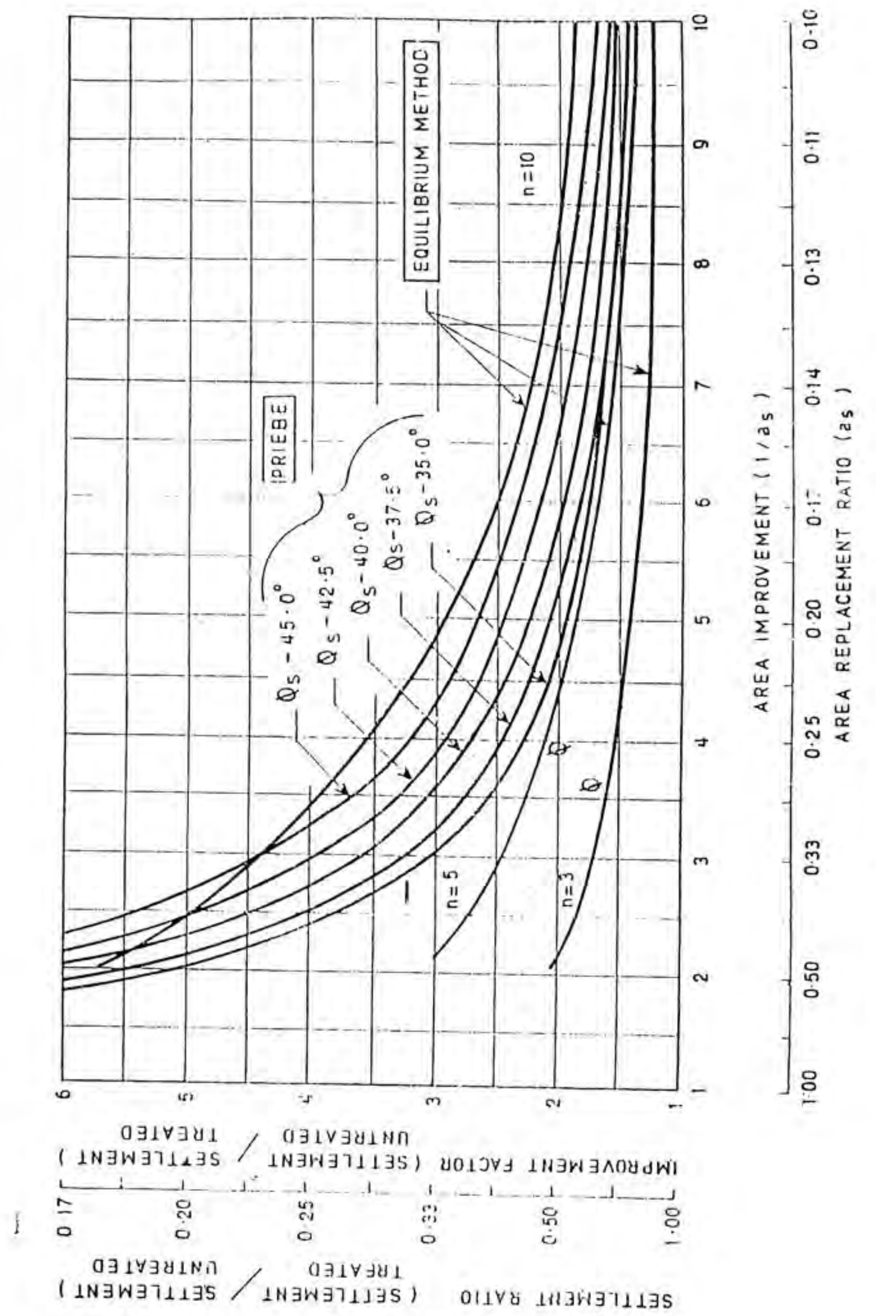


Fig. 2.12. Settlement Reduction due to Stone Column, Priebe and Equilibrium Method

II. Studies concerning new and innovative stone columns

(a) Load carrying capacity and settlement response of reinforced stone columns.
(b) Load carrying capacity of stone columns of composite diameter and composite materials

I. Laboratory studies concerning the load carrying mechanism of stone columns

(a) Drainage aspect of stone columns

Laboratory tests on stone columns clearly established the importance of the drainage function of the stone columns and the consequent strength gain in the soft clay. Rapid and slow loading tests were conducted on a group of three and four stone columns. (The results indicate that there is a decrease in water content and accompanying increase in the shear strength of the soft soil due to the drainage function of the stone columns).

Fig. 2.13 summarises the results from a number of tests carried out on soft clay at different initial moisture contents subject to slow loading. The test results indicate a significant increase in the bearing capacity of the stone columns soil system. The proportional increase in the shear strength is higher where the initial strength is lower. The results show that it is possible to use stone columns to improve the load bearing capacity of very soft clays with an initial undrained shear strength of the order of 10-15 kPa in conjunction with the precompression technique.

(b) Critical depth of stone columns for load transfer

Critical length of the stone columns is defined as the shortest column length which can carry the ultimate load regardless of the settlement. Studies have indicated that the yield load increases with increase in column length upto six times the diameter of stone columns (6D) and thereafter remained constant. Based upon mathematical analysis developed by Hughes et.al. (1974), the critical length of pile was predicted to be 3 to 4 D. Mokashi et.al. (1976) stated 2.85 D as the critical length. Ranjan (1988) states that the critical length extends upto 4 D to 5 D. The present experimental study shows the critical depth to be 6 D.

II. Studies concerning new and innovative stone columns

(a) Load bearing capacity of reinforced stone columns

Stone columns reinforced with peripheral restrainment extending upto a depth of 2 D have been found to give nearly 2.5 times the bearing capacity of conventional or unreinforced stone columns.

Stone columns reinforced with perforated metallic discs spaced at intervals of 1 D and placed upto a depth of 5 D have shown 2.5 times the bearing capacity of conventional stone columns. Typical results are shown in Fig. 2.14. Load settlement curves of reinforced stone columns depict an increased rigid behaviour of the stone column.

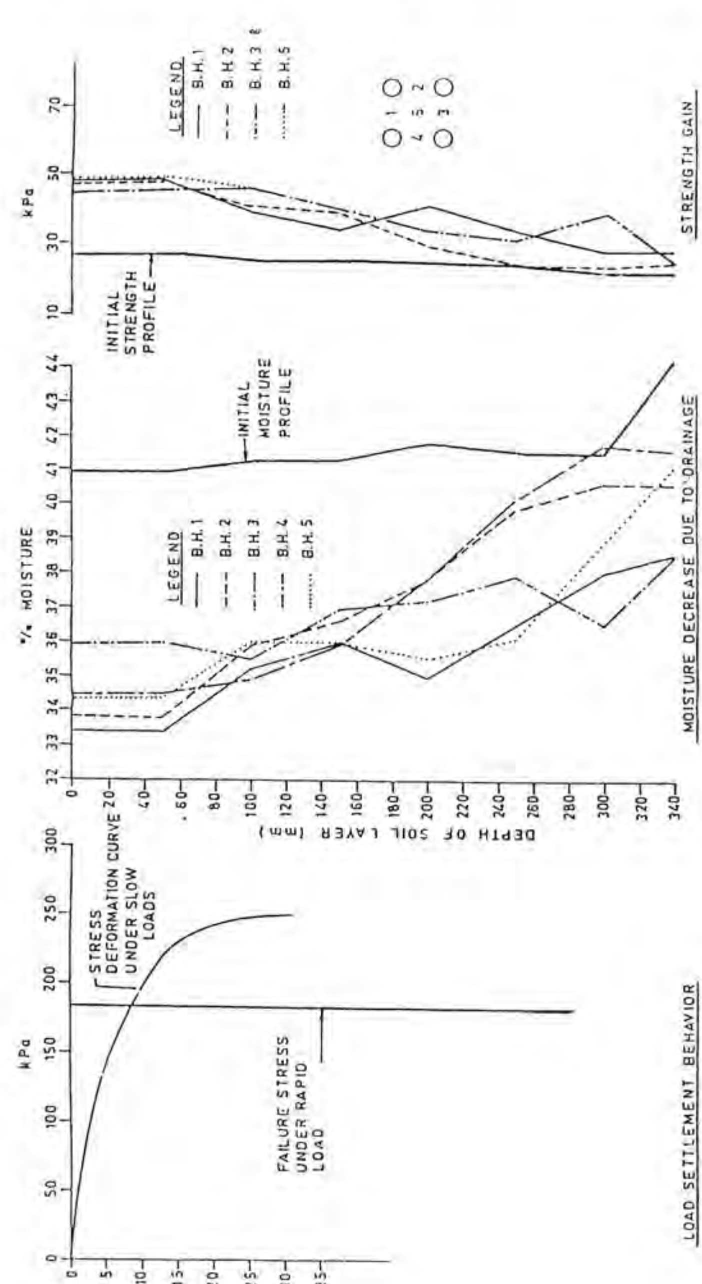


Fig. 2.13. Four Stone Columns in Square Pattern (Drainage Function)

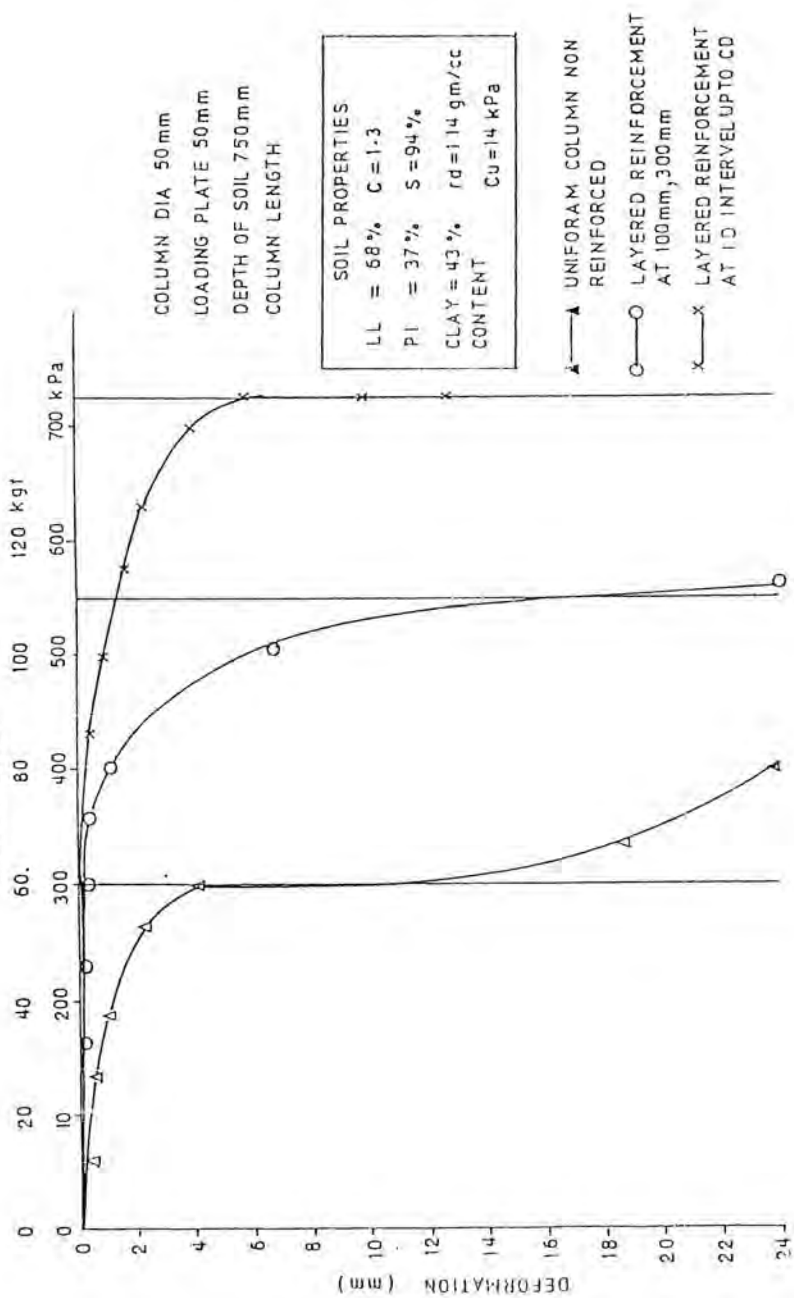


Fig. 2.14. Full Length Column with Layered Reinforcement

Studies have also revealed that immediate settlements of stone columns with peripheral reinforcement as well as layered reinforcements under loads are greatly reduced.

(b) Composite stone columns

Studies based on critical length concept led to the development of composite columns concept. Since only the top 6 D of the stone column acts in mobilising the resistance and load dispersion, it is possible to reduce the diameter of the stone columns below 6 D depth as well as replace the costly stone metal in this part, with less expensive sand. Such columns are termed "Composite columns" in this study. The load carrying and settlement characteristics are shown in Fig. 2.15 and their response is close to conventional stone columns. Significant saving in material can be achieved if the diameter of the stone columns is uniform upto a depth of 6 D and is thereafter reduced to 0.5 D as shown in Fig. 2.16.

2.10.1. Improvement in design methodology (stability analysis)

Rao et. al. (1990) conducted a study on the use of stone columns for improving the stability of embankments on soft clay. In their approach the stone columns were considered as shear pins leading to increase in the average shearing resistance along the potential slip surface. Average shear stress method was used in which weighted average material properties within the unit cell are evaluated. Design curves in the form of stability charts are presented in Fig. 2.17 for varying sub-soil strength characteristics and stone column configurations. The design curves simplify the design process and enable a rapid choice of optimum diameter and spacing of stone columns.

2.11. Illustrative Example

Design a stone column foundation system for the following sub-soil and loading conditions.

Depth of soft clay	-	10.0 m
Undrained shear strength of soft clay	-	20 kPa
Water table	-	Ground Level
γ_{sat}	-	15 kN/m^3
m_v soil	-	0.0008 m^2/kN
m_v stone	-	3.3×10^{-5} m^2/kN
ϕ_s	-	42^o

Embankment material

$\gamma_{sat} = 2.0$ t/m^3
$\phi = 30$

Loading intensity from embankment and granular platform = 100 kPa
Width of embankment = 20 m

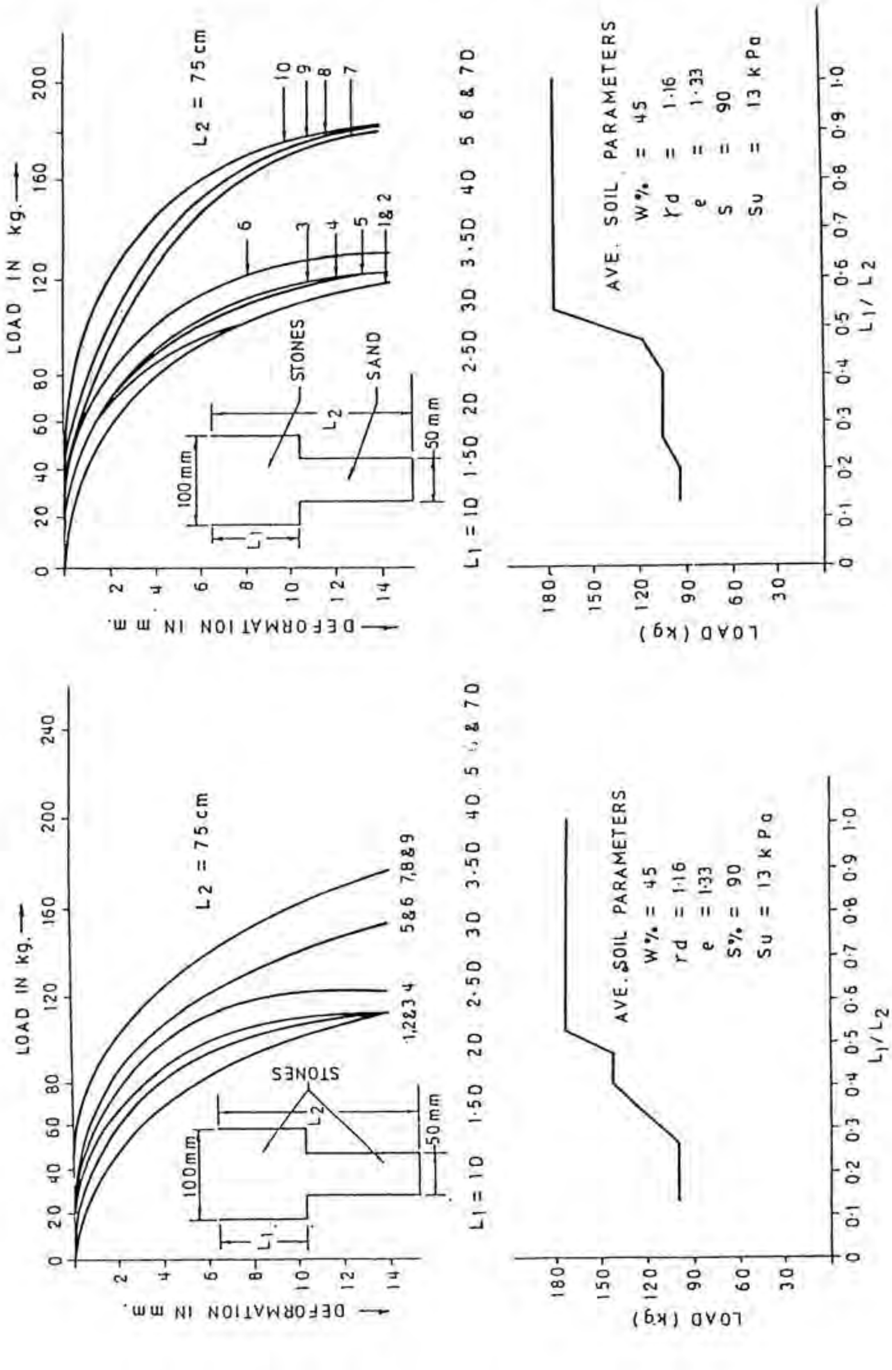
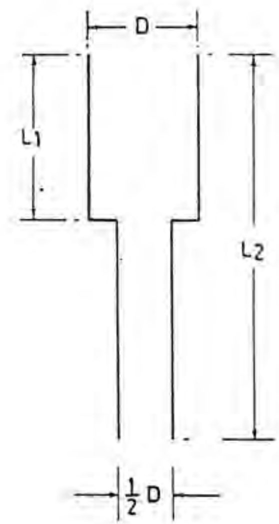


Fig. 2.15. Load Deformation Behaviour of Composite Stone Columns



L1	L2	SAVING
10 D	5 D 6 " 7 "	38·0 30·0 22·0
15 D	5 D 6 " 7 "	49·0 45·0 37·0
20 D	5 D 6 " 7 "	56·0 52·0 48·0

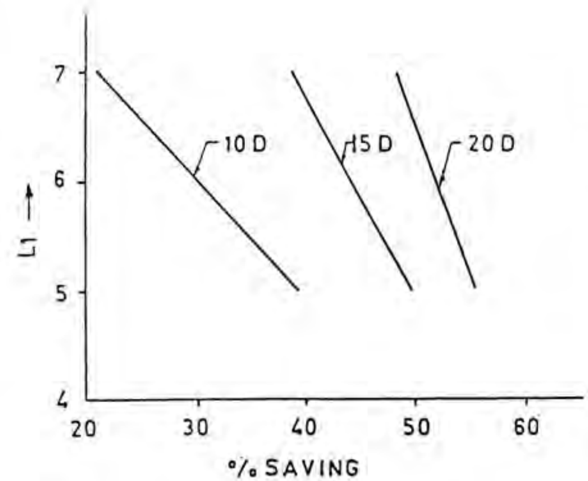


Fig. 2.16. Per cent Saving in Aggregate for Composite Stone Columns

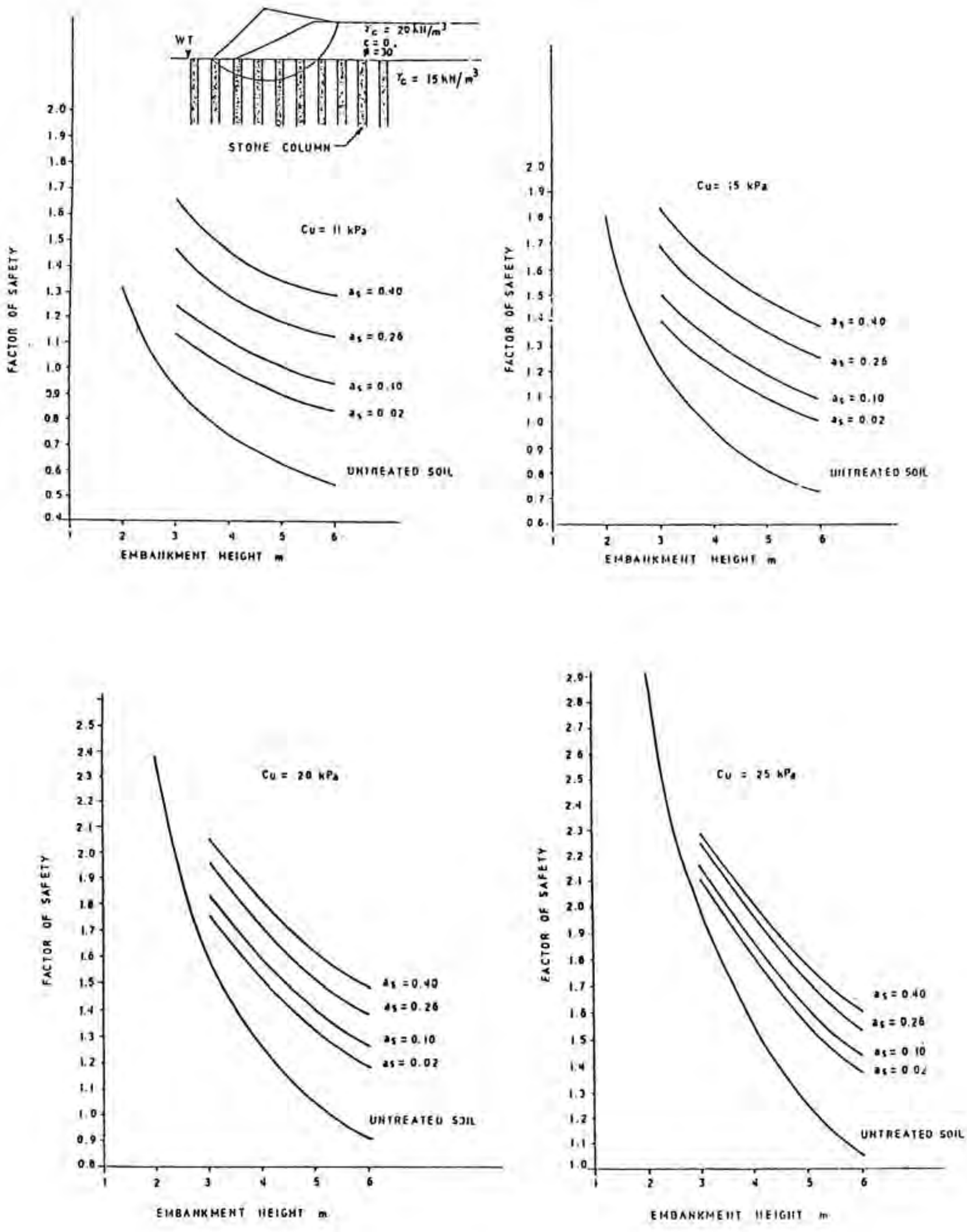


Fig. 2.17. Design Curves

The diameter and spacing of stone columns depends primarily upon the undrained shear strength of soft soil. From practical constructions the following arrangement of stone column is proposed :

Stone column pattern	-	Triangular lay out
Diameter of stone column	-	0.8m (finished)

Estimation of spacing based on stone column yield strength (Bearing Capacity consideration)

The yield load capacity of stone columns is given by the sum of the following components.

(A) Load capacity resulting from the resistance offered by the surrounding soil against bulging.

(B) Bearing support provided by the soil in between the stone columns.

(C) Increase in lateral resistance due to surcharge effect.

Capacity from component (A) is estimated as follows

$$\sigma_y = N_\phi (\sigma_{ro} + 4su).$$

$$N_\phi = \tan^2 (45 + \phi s/2)$$

$$\sigma_{ro} = K_o . \sigma_{vo}$$

Where

σ_y - Yield stress on stone column
σ_{ro} - Effective radial stress
k_o - Coefficient of lateral earth pressure (may be taken as 0.6)
σ_{vo} - Effective/ vertical stress (at depth of 4-5 times the diameter of stone column)
su - Undrained shear strength of soil
N_ϕ - $\tan (45 + 42/2) = 5.04$
σ_{vo} - $5 \times 0.8 \times 0.5 = 20$ kPa
σ_{yo} - $5.0 (0.6 \times 20 + 4 \times 20) = 460$ kPa

Area of stone column = 0.5 m^2

Yield load	=	460 x 0.5 = 230 kN
Factor of safety	=	1.5
Safe load on single stone column	=	153 kN

The bearing support provided by surrounding soil is estimated as

$q_{ult} = c.\ Nc$

$q_{ult} = 5.14 \times 20 = 102.8$ kPa

Factor of safety = 3

$$q_{safe} = \frac{102.8}{3} = 34.2 \text{ kPa}$$

Area of the unit cell, assuming stone columns are installed in a triangular pattern with spacing S is $0.867\ S^2$

Area of each stone column	=	$(\pi/4) \times 0.8^2 = 0.50\ m^2$
Area of the tributary soil surrounding the stone column	=	$(\pi/4)\ (0.867\ S^2 - 0.50)\ m^2$
Safe load capacity of the tributary soil	=	$(0.867\ S^2 - 0.50) \times 34.2$ kN

The surcharge effect (Component C) is estimated as follows:

Increase in radial stress:

$$= \frac{q_{allow}}{3} (1 + 2\ Ko)$$

$$= \frac{34.2}{3} (1 + 2 \times 0.6) = 25 \text{ kPa}$$

Increase in ultimate cavity expansion stress = σ_{ro}, F_q'

F_q'	=	1 for $\phi = 0$
σ_{ro}	=	$25 \times 1 = 25$ kPa
Ultimate stress	=	$\sigma_{ro}\ .\ N\phi$
	=	$25 \times 5.04 = 126$ kPa

Ultimate Load Capacity

$= (\pi/4) \times 0.8^2 \times 126$

$= 63.3$ kN

$$\text{Safe Load Capacity} = \frac{63.3}{1.5} = 42.2 \text{ kN}$$

The safe load carried by each unit cell is the sum of components A, B & C.

$$= 153 + (0.867S^2 - 0.50)\ 34.2 + 42.2$$

$$= 178.1 + 29.65\ S^2$$

The imposed load intensity on each unit cell due to embankment intensity of 100 kPa is $0.867\ S^2 \times 100 = 86.7\ S^2$.

Equating the two equations

$$178.1 + 29.65\ S^2 = 86.7\ S^2$$

$$S = 1.76 \text{ m}$$

Based on bearing capacity consideration provide 0.80 m diameter stone columns in a equilateral pattern at a spacing of 1.76 m.

Stability Considerations

The improvement in the factor of safety achieved by the use of stone columns has been evaluated by the average shear stress method. In this method weighted average material properties within the unit cell are evaluated. Design curves developed by Rao et.al. (1990) and discussed in the preceeding sections have been used.

Refer the Design curves (Fig. 2.17)

Su = 20 kPa

Embankment loading intensity 100 kPa

This corresponds to 5 m height of embankment

Spacing = 1.76 m

Area of stone column	=	$0.50\ m^2$
Area of unit cell	=	$2.68\ m^2$
Area replacement ratio	=	0.5/2.68 = 0.18

Factor of safety of 5.0 m embankment on untreated soil = 1.03

Factor of safety of 5.0 m embankment on stone column reinforced ground ($a_s = 0.18$) = 1.45

$$\text{Stability Improvement factor} = \frac{1.45}{1.03} = 1.4$$

Settlement Considerations

The settlement have been determined by estimating an equivalent coefficient of volume compressibility for the soft soil reinforced with stone columns. (Rao and Ranjan, 1985) and method proposed by Aboshi (1979) viz., equilibrium method. The stone columns are assumed to be installed to the full depth of soft clay.

The total settlement of the composite ground is given by

ρ_{group}	=	meqΔ P. H (Rao & Ranjan, 1985)
m_{eq}	=	a_s m_v stone + (1- as) mv soil
ρ_{group}	=	Settlement of the composite ground
m_{eq}	=	Coefficient of equivalent volume compressibility
Δ P	=	Pressure increment at mid depth of soft soil
H	=	Depth of soft clay
m_v(stone)	=	Coefficient of volume compressibility of stone column
m_v soil	=	Coefficient of volume compressibilty of soil
a_s	=	Area replacement ratio
m_{eq}	=	$0.18 \times 3.3 \times 10^{-5} + (1\text{-}0.18) \times 0.0008$
	=	6.6×10^{-4} m^2 /kN
ΔP	=	80 kN /m^2
ρ_{group}	=	$6.6 \times 10^{-4} \times 80 \times 10$
	=	0.52 m

Settlement of untreated ground 0.64 m

Estimation of settlement from equilibrium method (Aboshi, 1979)

The improvement factor read from the curves in Fig. 2.11 is of the order 1.20 using $a_s = 0.18$ and n = 1.5.

Settlement of the soft soil improved with stone columns is estimated as 0.64/ 1.20 = 0.53 m

2.12. Case Histories

Three typical case histories of successful applications of stone columns for improvement of soft ground are present.

Case History - 1

Portions of an embankment fill for intercharge ramps were supported an vibro-constructed stone columns at Hamption, Virgina (Goughnour and Bayuk, 1979). Fill height in the area reinforced by stone columns were 10.70 m.

The general soil profile revealed 3.7 m to 4.6 m of very soft silts and clays overlaying loose to medium compact silty sands. Consolidation tests preformed on undisturbed samples of these very soft soils showed compression indices C_c ranging upto 1.04 with void ratio 'e' of 2.693. The coefficient of vertical consolidation 'C_v' was determined to be as low as 7.02×10^{-3} m/day for loadings comparable to the maximum proposed embankment heights. Effective strength parameters were established from C_u tests. The results of these tests indicated that very soft soils have effective strength parameters of $c' = 2.4$ kN/m^2 and $\phi' = 26^{\circ}$.

The stone columns were installed by the vibro-replacement techniques using crushed angular granite as backfill. The gradation used was as follows :

Sieve size (cm)	Percentage passing by weights
6.3	100
3.8	65-79
1.9	6-10
1.2	1-5

The stone columns were driven on an average to a depth of 6.4 m. The average stone column diameter was 1.1 m. Vertical load tests were conducted on a large group of columns. A total of 45 stone columns were installed for the load test. The large group was load tested to using 401 t of dead load and settlement after 54 hours under the centre of group was 79 mm and total settlement of after 130 days was 300 mm. A stress concentration ratio of 30 has been reported which is fairly good. Shear strength and soil profiles are shown in Fig. 2.18.

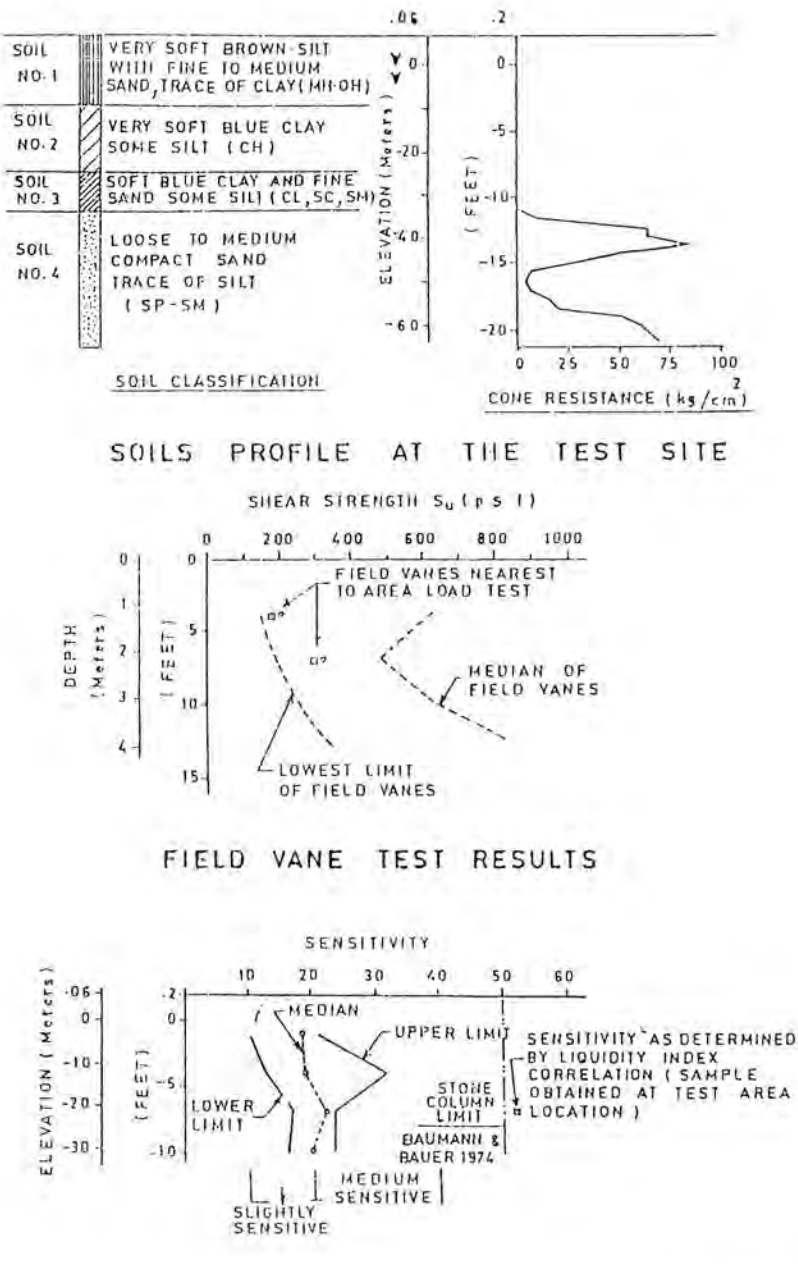


Fig. 2.18. Field Vane Sensitivity Results (Goughnour & Bayuk, 1979)

Case History - 2

Stone columns were used to support a large sewage treatment facility at Santa Barbara, California (Engelhart- Golding, 1975). The stone columns served the purpose of improving the site to withstand an earthquake of maximum horizontal acceleration of 0.25 g and also to provide an acceptable load- deformation response under the imposed loads. The site, in general, is underlain by recent estuarine deposits of soft to firm clays and silts and clayey sands. The design load for each column was 30 T. Each column penetrated the recent estuarine deposits into older marine soils and the length varied from 9 m to 15 m. A 0.3-0.9 m thick distribution blanket of compacted sandy gravel was used to transfer the structural loads to sand columns.

The stone column diameters ranged from 0.8 m - 1.2 m. A trangular pattern of stone columns was used. The pattern and spacing varied from 2.1 m equilateral triangle to 1.2 m x 1.5m isosceles triangular pattern depending upon the sub-surface conditions. The stone columns were constructed using the following gradation :

Sieve size (mm)	Per cent passing (by weight)
75	95-100
63	86-95
25	26-40
12	14-33

Twenty-eight vertical load tests and direct shear tests were conducted at the site. The results of one vertical load is shown in Fig. 2.19. Tests were conducted upto 1.3 to 1.5 times the design load. The columns settled by 6 mm under a design of 30 T. The results of SPT carried out before and after installation of stone columns is given in Fig. 2.20. SPT tests indicate that after stone column construction the relative density of in-situ soil increased to 92 per cent. Sands having relative densities of this magnitude are considered not susceptible to liquefaction.

Case History - 3

To improve the bearing capacity of soft marine clay in the ore stack area at Vishakhapatnam Port, to enable stacking of iron ore upto a height of 9.0 m, CRRI (1986) proposed the use of stone column for improving the load carrying capacity of the soft marine clay deposit. The undrained shear strength of the soft clay deposit in the experimental area was of the order of 2.5 to 3.0 t/m^2 and could support iron ore stock of 3.0 m height only. CRRI suggested field test on a single column and group of columns for evaluating the design. Stone column of 1.0 m diameter were installed upto a depth of 18.0 m below the existing surface elevation, at which depth a harder clay layer with an undrained shear strength of 15 t/m^2 was encountered. Nineteen stone columns were installed in an equilateral pattern at a spacing of 2.0 m in the experimental stack area. The test area was adequately instrumented with piezometers and settlement stakes.

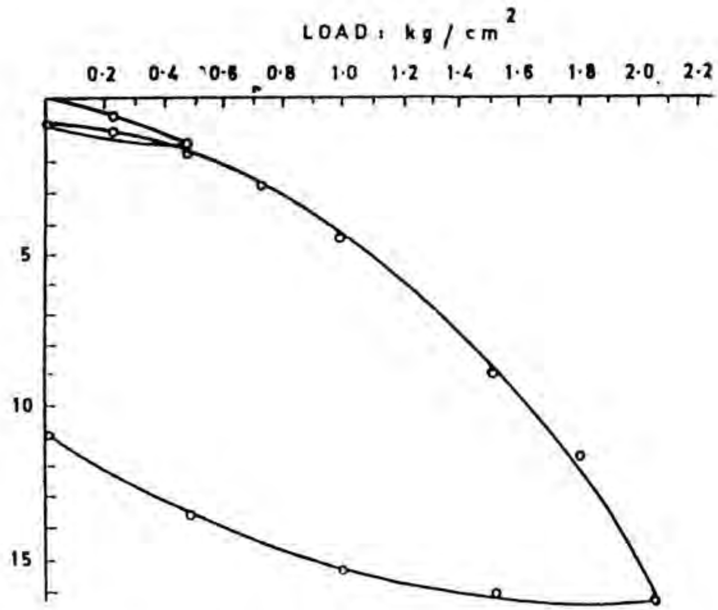


Fig. 2.19. Vertical Load Tests on Stone Columns at Santa Barbara (Engelhardt & Golding, 1974)

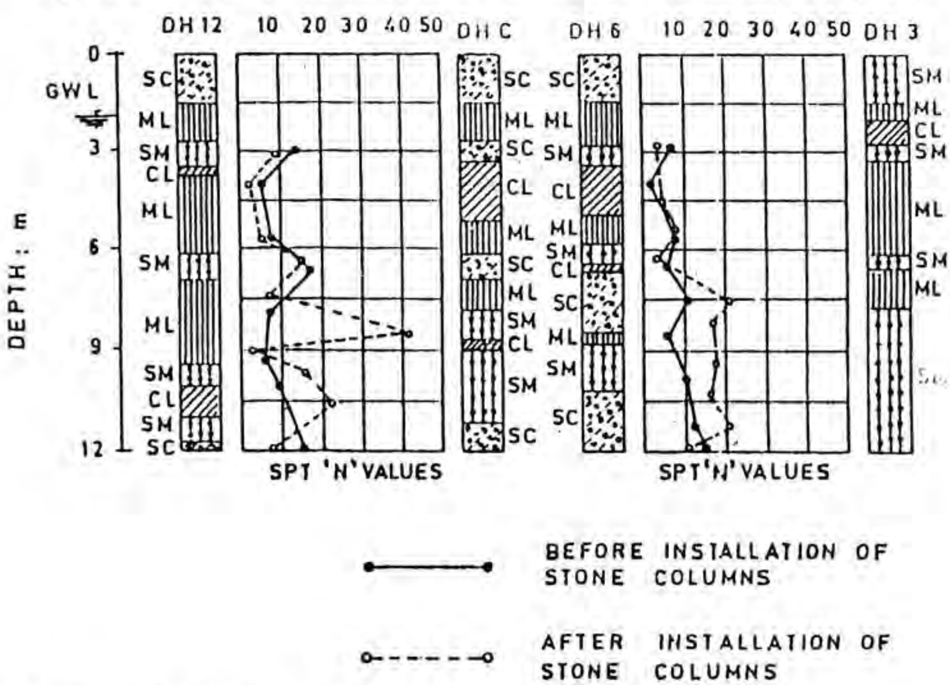


Fig. 2.20. Sub-surface Conditions at Santa Barbara Including the Effect of Stone Columns Improvement of Native Soil (Engelhardt & Golding, 1974)

Load test was conducted on the following arrangements :

(1) Single column - A test plate of 1.0 m diameter was used. Maximum load of 100 t was applied.

(2) Unit Cell Approach - A test plate of 2.10 m diameter was used. A maximum load of 180 t was applied.

(3) Group of Three Columns - A test plate of 3.60 m diameter was used. A maximum load of 180 t was applied.

In a single column load test, the column settled 10 mm under a load of 100 t. In Unit Cell Load Test the column settled 20 mm under a load of 180 t. In a group of three columns the group settled 4 mm under a load of 180 t. The above preliminary test gives an indication that the soft clay strengthened with stone columns can easily support 10 m of ore. The static load test is designed to verify this and give the final allowable ore height.

Presently, the site is being preloaded with iron ore upto 9.0 m height. The experimental stretch has been adequately instrumented piezometers and settlement stakes.

2.13. Conclusions

Following point can be concluded :

(1) Stone columns provide an economical alternative of improving the sub-soil condition.

(2) A detailed field and laboratory testing is required for successful design and construction of stone columns.

(3) Theories developed from the unit cell concept predict more closely the field observations than theories developed for isolated stone columns.

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3. GROUND IMPROVEMENT BY LIME STABILISATION

3.1. Introduction

Soil improvement with lime has been in practice since early 1930's. Soil properties change with the addition of lime. It has been observed that addition of 1-2 per cent lime, brings about modifications in the soil by reducing its plasticity, and improving its workability through cationic exchange, agglomeration and flocculation. Improvement in shear strength, compressibility and permeability characteristics is brought about by pozzolanic reactions resulting in the formation of cementing compounds. The rate at which chemical reactions and the formation of compounds take place is dependent on the quality of lime added, the clay minerals present in the soil, and the period of curing. Other factors which contribute to the lime-soil interaction are the degree of pulverisation and compaction.

The physico-chemical reactions in the soil are rather complex and can only be broadly generalised. Soil stabilisation with lime was used in the field of highways, rail roads, airport construction and bearing layers. This method is also employed in the construction of embankments, in improving slopes, backfill for bridge abutments and retaining walls, in soil improvement under foundation slabs and for lime piles for foundation, in excavation pits and in slope stabilisation. Lime slurry/lime flyash slurry injection is used to strengthen existing weak or less compacted subgrade and embankments. Compacted quick lime with gypsum has been found useful for rapid development of strength of very soft clays.

3.2. Lime Soil Interaction

On adding hydrated lime to a clayey soil, an immediate reduction in plasticity is experienced. This reduction in plasticity occurs with 1 - 2 per cent of lime in clay. It also improves the workability of the soil. Above a certain percentage of lime, there is no appreciable reduction of plasticity and this per cent of lime is called lime fixation point. The lime fixation requirement (L_m) of a clay is determined by the equation

$$L_m = \frac{\text{per cent of clay } (< 2\,\mu\text{ m})}{35} + 1.25 \qquad \text{Eqn. 3.1}$$

Lime soil reactions are more controversial than accepted. The basic mechanism identifiable in soil-lime mixes are (1) Base exchange (ion exchange), (2) Flocculation, (3) Cementation due to pozzolanic reactions and (4) Carbonation. Base exchange and flocculation are immediate reactions whereas pozzolanic reactions and carbonation are long range reactions. Strength due to carbonation is not appreciable and the major part of the strength is derived from pozzolanic reactions. When small quantity of lime is added, the lime is used up in flocculation and base exchange reactions.

A geotechnical engineer is more concerned with the pozzolanic/cementation activity of a soil lime mix. There are a number of complex compounds formed, viz., Tobermorite, calcium silicate, hydrate, calcium alluminate hydrate.

3.3. Lime Columns

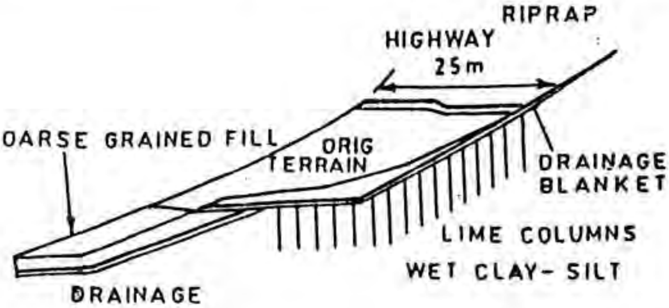
Lime columns are made by mixing a predetermined quantity of lime (hydrated or unslaked) to a mass of soil and compacted in the boreholes at a fixed density. Columns may be produced 'in situ' by providing measured quantity of unslaked lime into a soft clay mass through a rotary drill equipment equipped with a special auger bit, both to advance to the desired depth and to mix the soil and admixture thoroughly during withdrawal.

Lime columns have been widely used in Sweden and Japan. The mixing bit used in Japan is usually somewhat larger and more complex in comparison to the egg beater mixing tool used in Sweden which is simple (Fig. 3.1). The ability of the equipment to distribute the admixture thoroughly for the full required depth and to mix it uniformly across the column, is a pre-requisite for success of lime columns.

A lime column dia of 0.5 m is standard in Sweden for columns upto 10 m long. Diameters upto 1.75 m and depth upto 60 m, have been used in Japan. In addition to lime or cement, special chemicals are used in Japan. These columns have been used in Japan for harbour structures, whereas, in Sweden the use of lime columns has been usually to improve soft sensitive clays. Small diameter lime columns (80-150 mm) spaced at 0.5 m to 3 m have been used in Austria for slope stabilisation.

When quick lime (CaO) is used as the stabiliser, the heat of hydration can be substantial and drying of surrounding ground due to this and consumption of water by hydration can be significant. Admixture contents are of the same order (5 -15 per cent by dry soil weight) as for the conventional lime stabilisation. Since the soils to be treated have very high water content, use of unslaked lime is significant from strength gain criterion. For a normally consolidated clay from Sewden, the strength gain was from 2 to 7 times immediately after mixing with 6 and 12 per cent quick lime (Broms and Boman, 1979). The strength further increased to 13 to 82 times the initial strength of 10 kPa, after 1-3 years. The initial water content was about 60 per cent. Strength increase of 10 to 20 times the untreated value have been noted typically. Decrease of compressibility of the stabilised soil has been observed. The rate of hardening is influenced by ground temperature. The permeability increase is 100 to 1000 times the original soil and these columns also served as potential drains for excess pore pressure dissipation and consolidation of soil between the columns (Boman and Broms, 1979).

A possible disadvantage with lime column is that certain plants are adversely affected by lime when the lime content is very high. Also, a relatively long time is required (appx. 3 months) before the desired strength is achieved. Basic disadvantage should, however, be compared with advantages of the method. The foundation can be made very light and heavy construction equipment can be used because the bearing capacity of the soil is increased by lime. Also, noise and vibration can be reduced compared with pile driving. The costs when lime columns are used as foundation can be compared with driven precast concrete piles. Only thin concrete slabs are required as compared with a conventional slab supported on piles.



LIME COLUMNS UNDER A POTENTIAL SLIDE

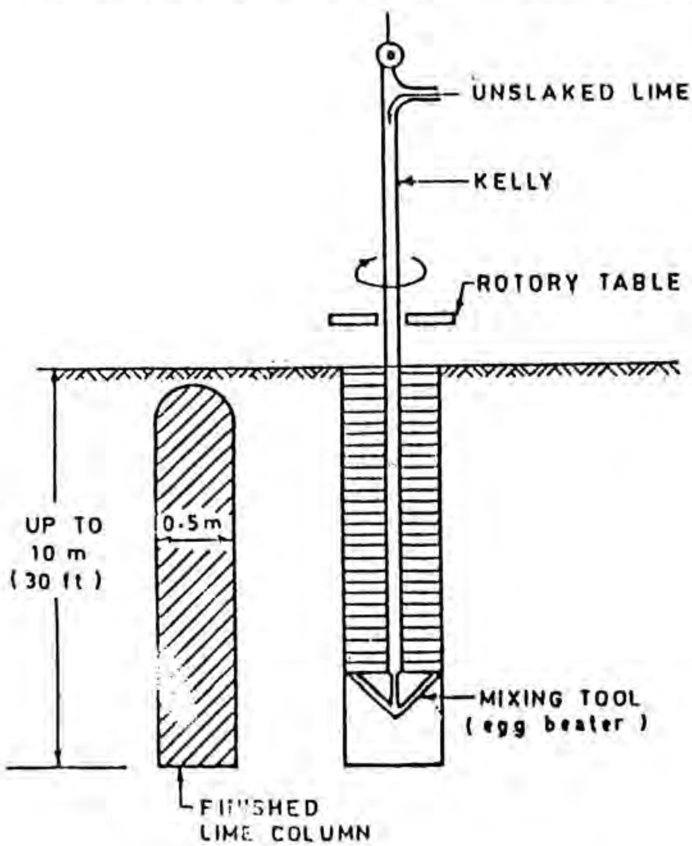


Fig. 3.1. In-situ Construction of a Lime Column

3.3.1. Application of lime columns

Lime columns are used to support light structures, control differential settlements, reduce total settlements, improve the stability of slopes and decrease the skin friction below the pile supported buildings or below bridge abutments. The lime piles will also carry a part of the weight of the soil that otherwise will be transferred to the structural piles when the soil around the structure settles (Broms and Boman, 1976). Lime piles are also used to reduce the lateral earth pressure in case of retaining structures. These have been used as sheet piles in deep excavations. Lime piles have also found use in controlling failure due to bottom heave.

3.3.2. Laboratory investigations and field tests

It is generally not possible to estimate in advance the amount of lime required to reach a certain strength increase without laboratory investigation. The amount of lime required for stabilisation of a clay depends mainly on the critical water content of the clay and the organic content. However, it is advantageous to use more of lime than is absolutely necessary. This is important to reduce the effects due to leaching, filteration of acid waters and other chemicals from surrounding soils. For very soft clays, 5 to 10 per cent of unslaked lime by dry weight is sufficient. A strength increase of 10 to 20 times the original strength is normal.

It is always advisable to have a few field installations of lime columns and bearing capacity tested at different intervals of curing period. It is observed that strength gain by using finely powdered lime is more than that from coarse grained lime.

3.3.3. Design of foundations on lime columns

The bearing capacity of the lime column is minimum immediately after the installation of lime columns, and in general, one third of the strength is developed after one month, half the strength after two months and it takes about 1-2 years for full strength to develop (Broms and Boman, 1979). Confining pressure from surrounding soils also help in increasing the strength of lime columns.

3.3.4. Settlement reduction and differential settlement

Differential Settlements may occur primarily due to shear failure in the unstabilised soil between the columns. Broms and Boman(1979) have given a formula to evaluate perimeter she stress along the perimeter of the reinforced block,

$$\gamma_p = \frac{W_g}{2.(B' + L').L_H} < \frac{s_u}{F} \qquad \text{Eqn.. 3.2}$$

and when the shear stress is less than the shear strength of surrounding soil, the differential settlement is less and within permissible limits. It is suggested that the factor of safety should be at least 1.5, then the equation 3.2 becomes,

$$\frac{L_H}{B'} > \frac{0.75\,q}{su\left(\frac{B'}{L'} + 1\right)} \qquad \text{Eqn.. 3.3}$$

W = weight of structure, B',L',L_H are the width, length and height of the reinforced block respectively. q ($Wg/B'.L'$) = average applied load. At $q/s_u = 2.0$, $B'/L' = 1.0$ and F = 1.5, the ratio $L_H/B' = 0.75$, the required length of column at $B' = 8$ m is thus 6 m. If this condition is satisfied, the differential settlement is minimum.

Total settlement is reduced with lime columns. The reduction factor is the ratio of the total maximum settlement for an area without and with lime column is given by the following equations:

$$\alpha = \frac{1}{\frac{E\,col}{E\,clay}(1+\rho)} \qquad \text{where } \rho = \frac{n'\,A\,col}{B'.\,L'} \qquad \text{Eqn. 3.4}$$

B', L' and L_H are width, length and height of reinforced block (Fig. 3.2).

3.4. Typical Case Histories

Sand and quick limes have been used by Chummar (1985) to improve the bearing capacity of marine clays and highly swelling clays. 25 to 30 per cent quick lime mixed with sand was used for cast in-situ 100 mm piles and properties of the clay in a radius of 200 mm were found to have changed. These piles were used to improve the foundation soil of a building which developed cracks due to foundation failure during the ground floor construction. Similar technique was used to stabilise the railway embankment which was sinking as the same was founded on a soft marine clay. Both the projects were undertaken in Tamilnadu (India).

The diffusion of lime into the surrounding soil around a lime pile upon flooding has been investigated by Ratnam et.al. (1985). Lime piles of 100 mm in dia were made in the ground and top 1 m was kept open. The spacing was kept at 12 d. The columns were flooded for two weeks. Samples were taken after 2 weeks at different lateral distances and vertical depths. It was found that the lime had diffused upto a lateral distance of 4 d. The same trend was not observed at deeper layers and the lime diffused to a far lesser extent. This was attributed to availability of abundant water in the top layer due to surface inundation. Diffusion of lime and its attendant beneficial effects are significant upto a radial distance of 1 m to 1.2 m and as such a column spacing of 2m, c/c is quite effective (Figs. 3.3, 3.4, 3.5 and 3.6).

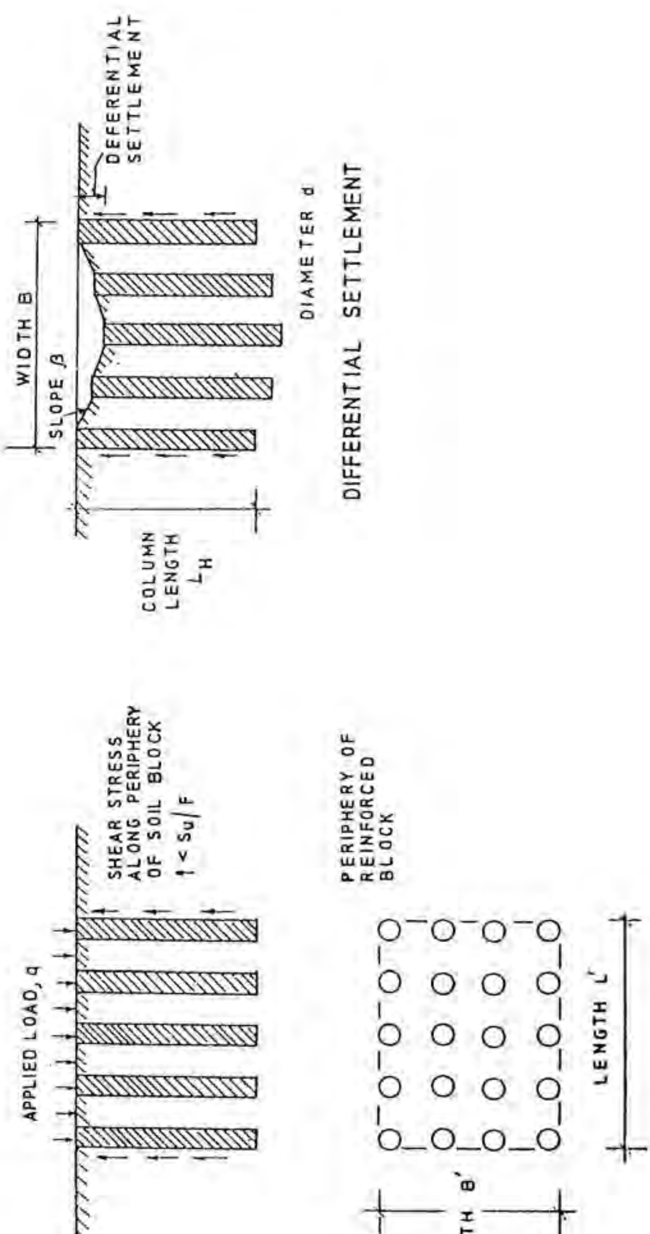


Fig. 3.2. Shear Stress Along Periphery of Soil Block Reinforced with Lime Columns

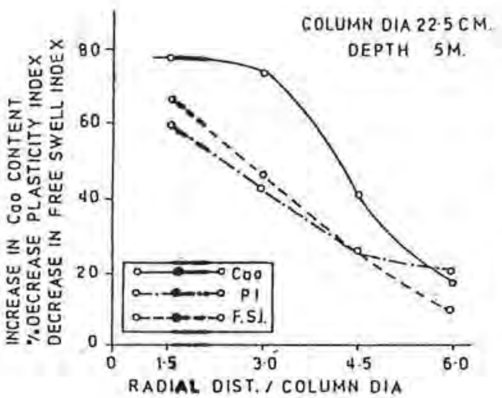


Fig. 3.3. Effect of Lime Diffusion from Field Lime Columns

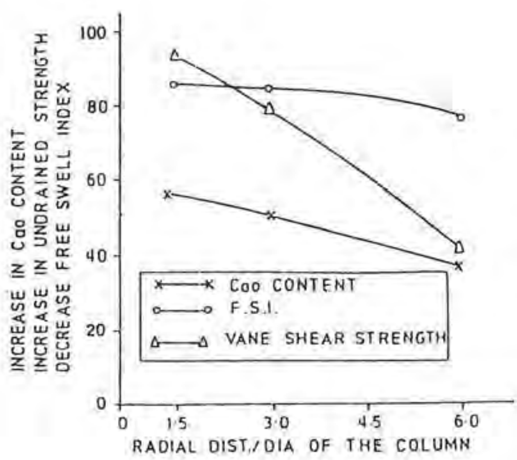


Fig. 3.4. Amelioretion Effect of Lime Soil Columns

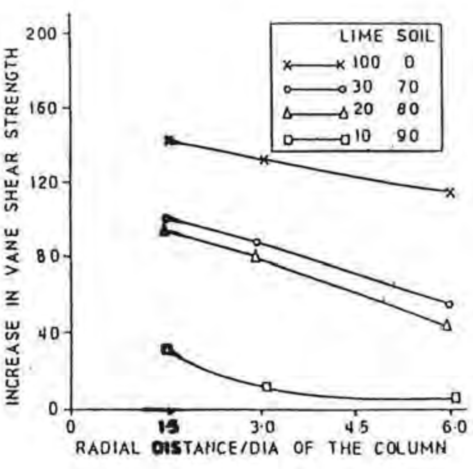


Fig. 3.5. Effect of Lime Diffusion from Lime Soil Columns on Undrained Strength

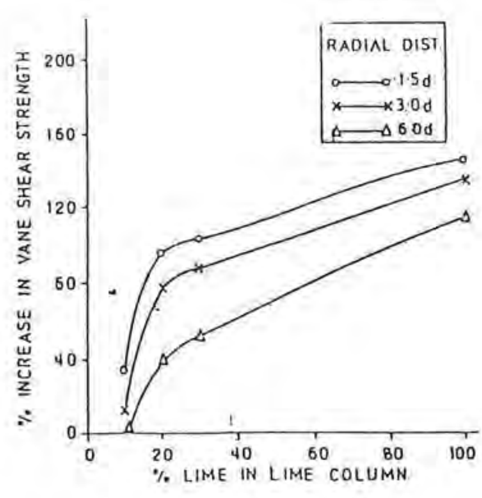


Fig. 3.6. Effect of Percentage Lime in Lime Columns on Undrained Strength

(Ratanam, M.V. et al., 1985)

Theoretical assessment of quick lime pile behaviour was made by Holyman and Mitchel (1983). The authors (Holyman and Mitchel, 1983) presented a mathematical model describing the physical phenomena involved in the stabilisation of the soil mass with the help of pure quick lime piles. The finite difference on dimensional formulation makes provision for the speed of the slaking of lime piles, expansion of the column, development in lateral pressures in the soil and subsequent consolidation of the soil in the radial direction. Physical parameters are deduced from the tests published by Kurado, et.al. (1980). The parametric analysis conducted substantiates the importance of the horizontal permeability of the soils being treated, the fineness of the lime used and its condition of placement.

Quick lime piles were successfully used by Bredenberg, et. al. (1983) to (i) decrease long-term settlement to an acceptable value, (ii) improve stability during construction stage to required levels, and (iii) decrease the water content for part of the excavated mass, thus making them suitable for backfilling at the same time as the excavation soil volume that had to be removed was decreased. The lime piles were made in a 10 m thick layer of very soft sensitive clay over which a cargo terminal building was erected. The ground level of the building was raised to 2-3 meters and under this overburden. Anticipated settlement was 1 m and the allowable settlement to building was only 15 cm. The terminal building was made to rest on point bearing piles. The soft clay was treated with lime piles to restrict the long term settlement and to eliminate negative skin friction on piles due to consolidation of soft clay under the over burden.

Besides the settlements, the authors (Bredenberg et. al., 1983) found lime piles useful for excavating basements. Also, due to excavation, stability of a railway line close to the site was endangered and lime piles helped in increasing the stability. Use of ten per cent lime by dry weight of clay in columns gave tenfold increase in strength. The authors (Bredenberg et. al., 1983) have developed an equation to find time rate of settlements as,

$$\rho(t) = 1.13 \frac{q_a}{E_{clay}} su(t) \qquad \text{Eqn. 3.5}$$

where, q_a = average applied stress, t = time at which settlement is required.

G. Holm (1983) used gypsum and quick lime as admixture for making columns. He observed that mixing gypsum-quick lime soil, the columns developed three times more strength than the lime- soil columns. Also, the range of clays possible to stabilise is extended by using gypsum together with lime. Clays with the natural water contents upto 140 per cent can be stabilised with good effect. It takes approximately three years to reach the same strength without gypsum. The mixing ratio of lime to gypsum ought to be 75 to 25 when long-term stabilisation is required. For immediate stabilisation, the mixing ratio 50 to 50 is preferred. The formation of ettringite (calcium sulphate alluminate) having needle shaped crystals gives high strength on stabilisation.

3.5. Lime Slurry Injections

Pressure injection of lime slurry for ground improvement is a new method for ground improvement. Two types of injection methods are in vogue, viz. injecting lime slurry through boreholes and injecting through hollow rods pushed into the ground. It is a form of grout and limited success has been reported by various researchers. However, it is different from normal grouting. In the normal grouting chemicals are injected into voids in granular soil and voids are filled with chemical grout. In lime slurry injection method, the slurry is forced into the soil horizontally, rupturing the existing structure and forcing the slurry along fissures, cracks and seams. The lime slowly migrates further into sub-soil by diffusion.

The technique of strengthening weak soil deposits by lime slurry injection under high pressure at different depths is a recent one. Trials so far have demonstrated enough promise for a larger application. Till now, lime stabilisation is being achieved by mix-in-place method on the ground and with this method, it is possible to stabilise a layer of small thicknes. This method of stabilisation cannot be used where weak deposits of larger thickness have to be improved. Where existing embankments showing signs of failure due to inadequate compaction have to be stabilised, this method is well suited to achieve the desired results. Thus, when there are thick deposits of soft soil forming the subgrade which need to be stabilised, the technique of stabilisation by injecting lime slurry under high pressure comes as effective and handy tool for ground improvement. This technique has also been successfully used for deep soil stabilisation. Lime slurry when injected under pressure into the soil mass, spreads into the adjoining crevices and vacant spaces in the soil mass. A network of lime slurry paths is thus formed as a result of spreading in different directions. The lime in the slurry reacts with the soil and thereby, strengthening process takes place. Though, the technique of 'in-situ' stabilisation of fine grained soils to a depth of 3 metres has been achieved successfully by drilling holes and filling then with lime and lime slurry, yet this technique has limitations because of low permeability of fine grained soil and low rate of migration. The technique of injection has been used to improve the bearing capacity of weak soils, to reduce their plasticity and swelling characteristics of expansive clays, minimise differential settlement under structures and floor loads. This method has also found application in stabilising the failed embankment slopes at an economical cost.

3.5.1. Techniques of injection

Lime or lime-flyash slurry is injected into the soil under pressures of 350 to 1000 kPa. Slurry is injected through 38 mm to 41 mm diameter nozzles. The injection pipes are pushed into the ground and lime slurry containing 30 per cent solids is injected into the subgrade to refusal at 300 to 450 mm intervals (Thomson and Robnett, 1976). Equipment capable of pushing the injection pipes to depths of 40 m or more have been developed in recent years. Generally the injection rods are pushed into the soil at about 30 cm intervals. At each depth, the lime slurry is injected to refusal. Refusal occurs when

(1) Soil will not take additional slurry,

(2) Slurry is running freely on the surface either around the injection pipe or out of the previous injection holes, or

(3) Injection has fractured or distorted the surface.

Although, there is substantial variability in the amount of slurry that can be injected, a normal take is about 120 litres per metre of injected depth. The nature of the soil being treated will influence the quantity of slurry that can be injected.

The normal lime water slurry composition is 0.3 to 0.4 kg/litre of water with a wetting agent added in accordance with manufacturers recommendation. Based on extensive field experience, the above slurry composition has proved to be satisfactory.

Although, injection pressures as high as several hundred kPa can be developed with most lime slurry injection equipment, the majority of the work is injected with pressure range of 350 kPa to 1400 kPa. In this pressure range it is normally possible to disperse the maximum amount of slurry into the soil.

Spacing of 1 m to 1.5 m on centres are common in pressure injection treatment for building foundation work. Spacings of 1.5 m are also typical for treatment of rail road subgrades.

Injection depths are variable but current equipment is capable of injecting to depths of approximately 3 m to 15 m. Write (1973) has indicated that a treatment depth of 2 m is normally sufficient for foundation treatments. This depth compares reasonably well with 1.5 m depth suggested by Holtz (1969). The general guide is to inject to a depth sufficient to be below the zone of critical moisture change in the expansive soil deposit.

If surface of the PIL treated soil deposit is exposed it is common practice to mix the free surface lime available into the soil to a depth of 15 to 20 cm. The stabilised layer further contributes to the process of retarding the moisture loss from the underlying soil.

Field studies in which PIL treated soils have been excavated, show that the PIL slurry is forced along fractured zones, cracks, fissures, bedding planes, root lines, coarse textures seams in varved clays, seams and fractures effected by the pressure slurry injection process, or other passages in the soil mass. The field observations indicate water slurry will not permeate an intact fine grained mass.

Therefore, it is apparent that there are two major treatment mechanisms of concern relative to PIL. The first is the ability to permeate the soil mass with the stabilising additive (in this case a lime-water slurry) and the second is the process whereby, following PIL treatment, the lime translocates and modifies the soil adjacent to the lime seams.

3.5.2. Theoretical aspects of lime injection (Lundy et. al., 1968)

When the basic theory of permeability is combined with Darcy's law of fluid flow in a soil mass, the total quantity of fluid that can be forced into a soil mass during a given interval of time can be approximated by the following equation.

$$Q = \frac{g.\gamma_L}{V_L} \left(\frac{P}{L}\right) \quad K.A.t. \qquad \text{Eqn. 3.6}$$

where

A = Cross-sectional area over which pressure acts

V_L = Viscosity of fluid
g = acceleration due to gravity
P = Pressure head
K = $C.d\,m^2$, intrinsic permeability of medium,where C is shape factor and d_m is average pore size of soil.
L = length over which pressure head acts
Q = quantity of fluid flow
γ_L = density of fluid
t = time of pressure injection

The equation (1) indicate that the following major factors control the quantity of fluid that is injected, (a) fluid viscosity, (b) injection pressure and time, and (c)intrinsic permeability of the soil medium.

Since the lime slurry is not an ideal fluid, but rather, a particulate suspension, the pore size distribution of the soil mass is an important consideration in the permeation process. Successful injection of lime slurry into the soil mass would require that channels larger than the lime particles be present.

The inherent pore size of most of the fine-grained soils is quite small as compared to lime particle size. Thus, appreciable lime slurry movement through these pores is questionable. Johnson (1968) recommends that groutability ratio as calculated by using equation (2) be greater than 20 to 25 for successful cement grouting.

$$\text{Groutability ratio} = \frac{D_{15}\ \text{Soil}}{D_{85}\ \text{Grout}} \qquad \text{Eqn. 3.7}$$

D_{15} Soil = Particle size for which 15 per cent of the soil fraction is finer, and

D_{85} grout = Particle size for which 85 per cent of the cement grout is finer.

It is apparent that to inject lime slurry successfully by pressure injection method into fine grained soils, natural channels and passages larger than the lime particles must be present in the soil mass. Such channels may be present as a result of (a) inherent pore structures of the soil mass, (b) cracks, fissures, seams and root holes present in the soils or, (c) jetting or tearing of the soil effected by the pressure injection process. Eventhough the permeability of the soil, resulting from inherent soil pore structure may be low, the permeability or conductivity may be very high due to fissures, seams cracks, varves, etc. When this condition exists, the potential for successful lime slurry injection of a soil mass is greatly enhanced.

However, effectiveness of lime slurry injection method is not free from controversies. There are conflicting reports about the effectiveness of the technique. This method is not applicable in all cases but it is useful in appropriate situations. Lime slurry may not penetrate through intact fine soil mass unless pressure is high enough to create fractures in the soil.

3.5.3. Applications of lime slurry injection

The application of the method, though limited, has a great scope in India. Normally, the village roads which run on embankments are seldom compacted and in due course of time and due to ribbon development, these roads have to be strengthened. Instead of excavating the embankment and compacting the same to allow heavy vehicular traffic, the method of lime slurry injection becomes economical and time saving. The pressure injection of lime slurry compacts the soil at deeper layers, increases the in-situ density, and increase the soil strength due to pozzolanic/chemical reactions. Flood protection embankments which get eroded every year due to floods can be strengthened by the use of lime slurry injections. Road embankments which are under-compacted can also be compacted by this technique.

3.5.4. Typical case histories

Utility of lime slurry injection to stabilise poorly compacted embankments had been undertaken by Malhotra et.al. (1987). Lime slurry was injected at depth of 0.5, 1.0, 2.0 and 2.5 m in the embankment under a pressure of 3-4 kg/cm^2 with the help of an A-rod having perforated tip which is inserted in the body of the embankment with the help of a hammer to the desired depth. The slurry used for injection was containing 30 per cent lime. Migration of lime, both vertically and laterally, was determined by taking out samples of treated soil at different lateral distances and at different depth away from the point of injection of lime slurry and increase in strength was measured by performing pressure-meter and plate load tests. The results show that the lime migrates laterally to a distance of about 0.75 metre. The extent of lateral migration could be improved by increasing the pressure of injection. Pressure-meter tests indicate a tangible increase in values of limit pressure and modulus of deformation parameters used in determining strength.

Lundy et.al. (1968) conducted pressure injection of lime slurry in a soft glacial clayey silt deposit to strengthen the foundation to a depth of 6 m lying under a 12 m high embankment in Pennsylvania, (U.S.A.). High pressure lime slurry injection was moderatelysuccessful in forcing slurry uniformally into the soft clayey silt deposit. 30 per cent slurry was injected at 2000 kPa -4000 kPa pressure. Eight to twelve gallon of slurry were injected at every 20 cm interval as nozzle moved down. The injection nozzle was tapered to help prevent leakage up along the drill rod and to a self sealing hole as the rod was advanced into the soil. The samples were recovered after one year. There was no noticeable change in the mechanical properties of the soil except increase in the shear strength of the soil.

Joshi et.al. (1980), has attempted pressure injection of lime-fly-ash slurry for deep 'in-situ' stabilisation of fine grained soils to a depth of 13 m. A 13 metre high and 250 m, long embankment constructed in early 1900's to support a double main line track between Cape Girerdear Missouri and Gale, United States, had to be strengthened as the track failed repeatedly even at a very reduced speeds of 32 km/h. The embankment is located over river flood plain. The settlement of the track occurred every year in spring season. The embankment material consisted of silts, silty clays and sands. The top six metre of embankment consisted of cindered blast, as the top of embankment had to be surfaced every year. The fill material underlying the ballast is loose silty and highly plastic clays. The lime slurry was injected in two stages to penetrate the shear failure plane, tension cracks and voids in the subgrade and embankment fill beneath the ballast. Lime-flyash slurry was injected to fill the voids in the deep ballast and cinder pockets to reduce their water holding capacity. The embankment, after treatment performed exceptionally well over the last three years and no maintenance was required.

3.6. Conclusions

1. Lime deposited in seams by lime slurry injection, not only reacts with the soil adjacent to the lime seam, but also with the clay, away from the seam. The lime migrates by diffusion or translocation and alters the clay properties and improves the strength to some extent.

2. Shear strength increase occurs in the 'in-situ' soils treated by lime slurry. It takes one month to one year to notice this change, which depends on the type of soil. The stabilisation can be carried out to a deeper layer if equipment is available to force drills rods at the required depth and lime slurry can be injected at very high pressure as more pressure is required at deeper layer.

3. Only the soil which had completely reacted with lime show increase in strength. Since diffusion of lime is slow in clayey deposits under water table, it takes a year or so to achieve sufficient strength increase. Submerged clays takes a very long time to react with hydrated lime slurry. This may be more than a year.

4. Lime has been found to travel upto 75 cm in radial from the point of injection in under compacted silty soil embankment. Twenty- five per cent gain in strength is observed in terms of limit pressure and k-value after 28 days of injection.

5. To-date little research has been conducted on the effects of high- pressure soil stabilisation on soft clayey soils and under compacted embankments. Though undisturbed strength increase have been measured in case of silty soils injected with lime slurry, no one has tried to measure the undisturbed strength in case of clayey soils.

6. The field injection equipment should be modified considerably to ensure that the injected slurry is more effectively and accurately forced into the subsurface.

7. Actual test embankment should be constructed on treated and untreated test areas to study the effects of consolidation on the migration of the lime slurry and to observe the stability of treated v/s untreated areas.

8. More extensive sampling and testing programme providing longer curing periods between injection and testing are suggested to obtain more laboratory data, documentation of strength increase.

9. Introduction of additives into the slurry to increase the speed of reaction, increases of migration and increase in the strength of the treated soils should be attempted and analysed.

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4. USE OF GEOSYNTHETICS FOR GROUND IMPROVEMENT OF SOFT SOIL

4.1. Introduction

Over the last decade, the use of geotextile of all types, more often termed geosynthetics, has recorded a tremendous increase. Geosynthetics are being increasingly employed in Highway Engineering to facilitate the construction of road embankments, ensure better performance of the road pavement and reduce maintenance. Table 4.1 shows the consumption of geosynthetics for different application 'in U.S.A. The figures listed in Table 4.1 indicate that the use of geosynthetics increased by 12 per cent roughly every year. The growth rate in Europe and South East Asia is also comparable. The potential of geotextiles has caught the attention of Indian Engineering community as well.

Table 4.1. North American Geotextiles Market (Million Square yards) 1986-1990

				Forecast Projected	
Application	1986	1987	1988	1989	1990
Asphalt Overlays	79	90	101	112	123
Separation/ Stabilization	70	78	88	98	108
Drainage	35	37	41	44	47
Lining Systems	15	17	19	24	30
Erosion Control	13	14	16	18	20
Silt Fances	12	14	16	18	20
Reinforcement	11	14	16	19	21
Total Market	235	264	297	333	369

Bulk of geosynthetics presently in use are manufactured from petroleum derivatives, such as, polyamides, polyethylenes, polypropylenes and polyesters. Geosynthetics have appreciable tensile strength and yet are flexible. By and large they have good permeability characteristics. Highly impermeable geosynthetics are termed as geomembranes. Geosynthetics of different types as listed below are being used for various avil engineering applications:

(a) Woven

(b) Non-woven

(i) Needle punched

(ii) Spun bonded

(iii) Melt bonded

(iv) Resin bonded

(v) Knit Fabrics

(c) Geomembrances

(d) Geogrids

The main functions of geosynthetics are reinforcement, filtration, drainage and separation, John (1987).

In the present context the use of geosynthetics for the construction of embankments on soft soil is under review where the embankment itself is basically stable, but where problems arises due to the inadequate strength of the foundation soil on which the fill rests. Generally, soft soils are associated with high water table conditions where the removal and replacement of soft soil is a difficult task.

4.2. Ground Improvement by Geosynthetics

Geosynthetics are employed as a horizontal reinforcement at the base of the embankment for improving the load carrying capacity of soft soil. The main function intended is

(i) To prevent mixing of the borrow fill and soft sub-soil resulting from local bearing capacity failure. (Separation function).

(ii) To prevent excessive vertical and horizontal deformations. (Reinforcing function).

(iii) To compensate for low shear strength of the soil that might lead to a slip circle failure. (Reinforcing function).

Earlier non-woven geotextile with relatively low tensile strength and relatively low tensile stiffness were used for this application. Such geosynthetics, however, did not fulfill all the functional requirement elaborated above and, as such, their use was not satisfactory. More recently, the development of geotextiles with high tensile strength and a high tensile stiffness values has made the application of geotextiles in the prevention of slip circle failures a more realistic proposition.

4.3. Design Methodology

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

(i) Bearing capacity failure

(ii) Slip failure

(iii) Elastic deformation

(iv) Pullout failure

(v) Lateral spreading

Fig. 4.1 illustrates the various modes of failure.

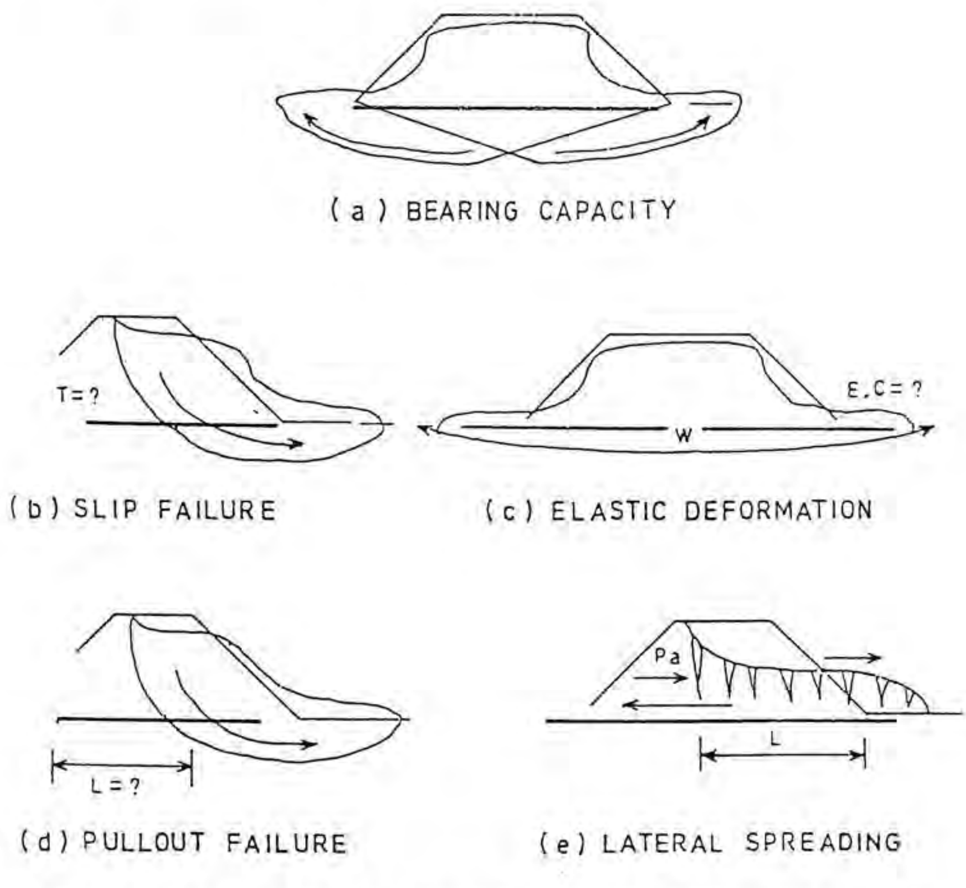

Fig. 4.1. Various Modes of Failure of Embankment Supported by Geosynthetic on Soft Sub-soil

4.3.1. 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 geofabric. The allowable bearing capacity is computed from the following formula:

$$q_{allow} = s_u \, Nc / F \qquad \text{Eqn. 4.1}$$

q_{allow} = Allowable bearing capacity
s_u = Undrained shear strength
Nc = Bearing capacity factor
F = Factor of safety

4.3.2. Slip failure

The benefit of geotextile as a horizontal reinforcement at the base of the embankment can be quantified by using any standard methods of stability analysis by slices. The method of slices is discussed in Chapter 2 (IRC HRB Special Report No.13, 1994). The method of slices is basically a semi-graphical technique where a potential slip zone is sub-divided into slices on a scale drawing or on a coordinate system using a microcomputer program. The disturbing and resisting moments for each slice are assessed individually. There are summed to give the factor of safety of the potential slip circle being investigated.

The factor of safety based on Fellenius method for stability is as follows,

$$F_u = \Sigma \, \{ c'Ls + \tan\phi' \, (W\cos\theta s - u\,L_s)\} / - W\sin\theta\, s \qquad \text{Eqn. 4.2}$$

The disturbing moment is given by

$$M_D = \Sigma \; R\,W\,\mathrm{Sin}\,\theta s$$

For an embankment with geotextile reinforcement at base the geotextile tensile resistance offered by the geotextile provides an additional restoring moment (M_{RG}). Referring to Fig. 4.2 this is evaluated by the following expression.

$$M_{RG} = R\,T_T\,\mathrm{Cos}\,\theta$$

Where T_T is the total tension in the geotextile layers.

The addition of this restoring moment modifies the F.S by

$$F_R = \frac{\Sigma\,\{c'\,Ls + \tan\phi' \;(W\cos\theta s - u\;Ls)\} + T_T\;\mathrm{Cos}\,\theta}{\Sigma\,W\sin\theta s} \qquad \text{Eqn. 4.3}$$

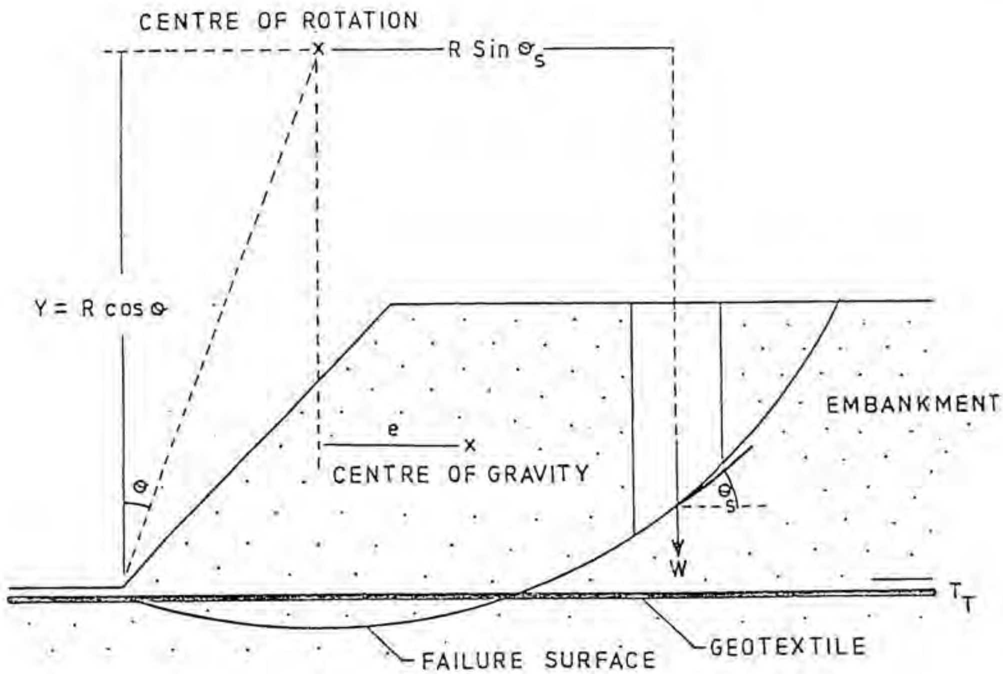

Fig. 4.2. Stability Analysis of Embankment with Geosynthetic at Base

From Eqn. 4.2 and 4.3 ;

$$F_R - F_u = \frac{T_T \cos\theta}{\sum W \sin\theta s} \qquad \text{Eqn. 4.4}$$

where

F_R = Factor of safety with geosynthetic reinforcement
F_u = Factor of safety without geosynthetic reinforcements

Hence F_R - F_u gives the improvement in the F.S. due to the use of Geosynthetics.

When a geosynthetic fabric is placed on a soft clay and the embankment load is applied, the fabric experience tensile strain, induced by the stabilising force T_T. In addition, the fabric also experiences strain due to settlement of the soft clay under the applied load. The strain due to settlement of the soft clay may be estimated by considering peak settlement to occur under full height of embankment section and zero settlement to occur at the embankment toe. If the strain in the geosynthetic fabric due to the settlement exceeds that induced by the stabilising force

T_T than the tensile force corresponding to the strain due to settlement of the soft clay shall be used for the assessment of factor of safety in tension.

Koerner et.al. (1987) carried out a study on the effect of the strength of the geofabric on the slope angle and sub-soil strength using limit equilibrium approach keeping a Factor of Safety of 1.3, Fig. 4.3. Following important points were concluded from the study.

(i) Increasing the slope angle of embankment requires gradually increasing geosynthetic strength. The decrease in the strength of sub-soil also requires correspondingly increase in the geosynthetic strength.

(ii) The live loads have a significant effect in requiring high geosynthetic strengths.

4.3.3. Elastic deformation

The magnitude of elastic deformation allowed by the geosynthetic will govern the deformation of the embankment. Large deformation will cause embankment cracking and loss of overall stability. Fowler et. al. (1986) have recommended 10 per cent value of the maximum strain at the required stress level. Modulus of geosynthetic required is

$$E_g = 10.\ T_{reqd} \qquad \text{Eqn. 4.5}$$

E_g = Young's modulus of Geosynthetic
T_{reqd} = Required strength of geosynthetic reinforcement

4.3.4. Pullout failure

Sufficient length of geotextile on each side of the failure surface should be provided to ensure a safe bond between the geotextile and the soil. Geogrids should be preferably used because a high soil to soil contact through the grid apertures ensures that the geogrid is well bonded to the soil.

Refer Fig. 4.4, the frictional bond developed on both faces of the length of geotextile within the failure zone is given by

$$FB = 2\gamma\ (1/2\ Le + Lc).\ z_g.\ \tan\mu \qquad \text{Eqn. 4.6}$$

Where

FB = Frictional bond
μ = angle of friction between soil and geotextile
z_g = depth of geotextile below the toe of embankment
γ = unit weight of soil
Le = length of geotextile beneath the sloping embankment face
Lc = length of geotextile beneath the full height section of the embankment

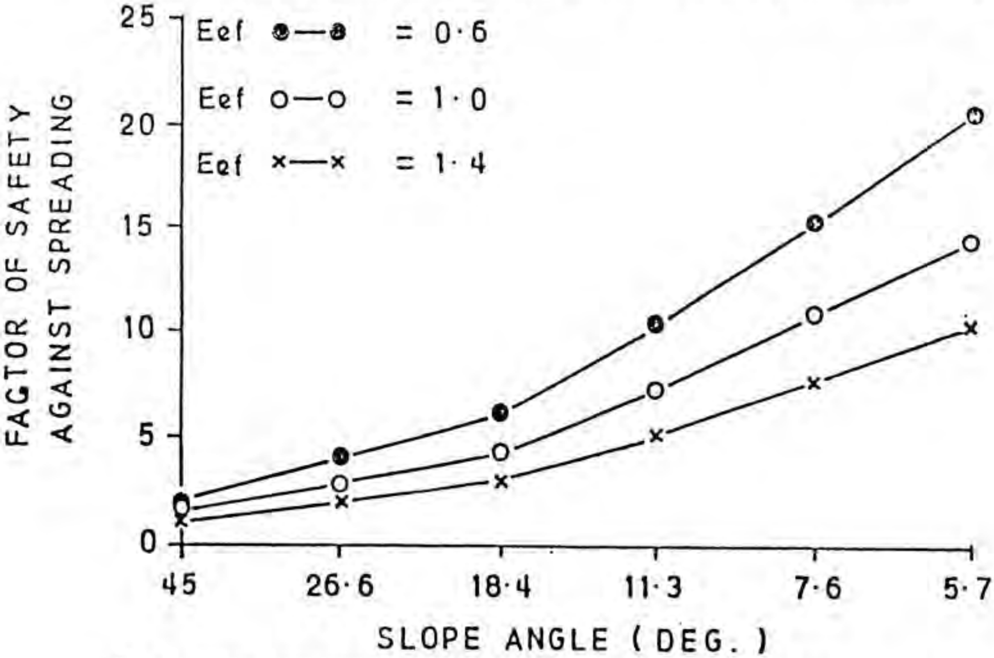

Fig. 4.3. Lateral Spreading Tendency Vs Slope Angle and Geosynthetic Friction Efficiency (Koerner, 1987)

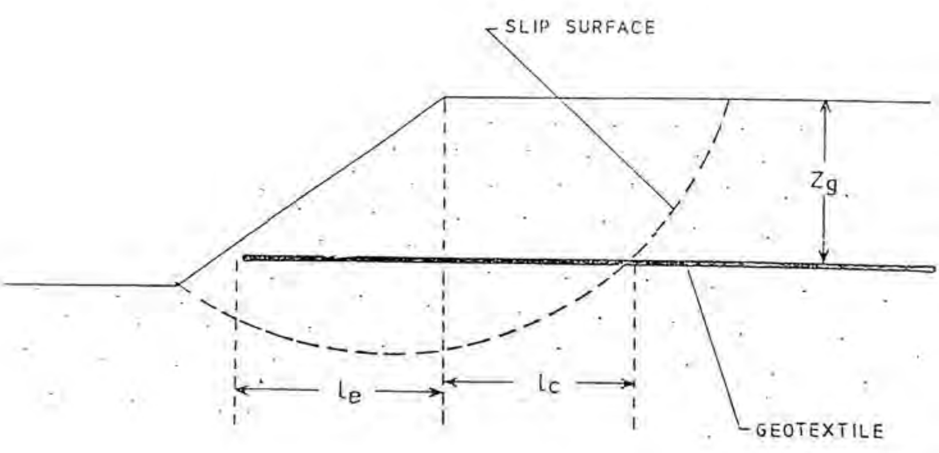

Fig. 4.4. Geotextile Bond Analysis

The factor of safety against pull out failure is given by

$$F_{(B)} = F_B/T_T \quad ..... \quad ..... \qquad \text{Eqn. 4.7}$$

T_T = Tension in the geosynthetic
$F_{(B)}$ = Factor of safety against pull out failure

The factor of safety against pull out should not be less than 2.0.

When the calculated value for the safety factor in pull out exceeds in tension, it indicates that full theoretical bond cannot be mobilised without inducing an excessive tensile force in the geotextile.

A number of efficiency relationships have been developed based on the laboratory pullout tests on geosynthetics. The geosynthetic is pulled out of surrounding soil under applied normal stress and the resulting strength is compared to the shear strength of the soil itself. The efficiency relationship for different geosynthetics proposed by Koerner (1987) are as follows:

E_{ef} (geogrid)	=	1.3 to 1.5
E_{ef} (geotextile)	=	0.8 to 1.2
E_{ef} (geomembrane)	=	0.5 to 0.7

The geogrids provide an excellent anchorage in soil with the soil filling up the apertures of the geogrid. However, when E_{ef} is greater than one, the failure may occur in the surrounding soil. Therefore, in design E_{ef} value is not assumed greater than one.

4.3.5. Lateral spreading

The mechanism of lateral spreading is schematically shown in Fig. 4.1. Tension cracks are generally observed on the surface of embankment. Koerner (1987) studied the tendency of 4.0 m high embankment to lateral spreading versus slope angle for various efficencies 'E_{ef}'. This is illustrated in Fig. 4.4. The different curves for different efficiency values are typical of geogrids, geotextiles and geomembranes. It can be deciphered from the curves that the tendency to lateral spreading becomes severe for steep slope angles and very smooth geosynthetic surfaces.

4.4. Orientation of Principal Stresses Under Embankment Loading

In linear embankments the major principal stresses are directed towards slope faces of the embankment and minor principal stresses are parallel to the axis of the embankment. The minor principal stresses are generally taken 50 per cent of the major principal stresses. The major principal stresses are handled by the strong warp direction of the geosythentic as shown in Fig. 4.2 . It is in the minor principal stress direction where seams are required to transfer loads from the edge of one role of geosynthetic to another. Hence consideration of seam strength is important in the choice of geotextile. In areal embankments where length and width are both extremely large the principal stress directions are quite undefined. In such cases major principal stress is considered to occur throughout the geosynthetic requiring a balanced geosynthetic design. In such cases seam strength usually governs.

4.5. Construction Procedure

Before placing the geotextile, adequate site preparation is necessary. In case ditches are present they should be lined with a geotextile sheet extending at least 3m beyond the edges of the depression.

During the construction it is essential that the geotextile is put gradually in tension to obtain the full benefit of its tensile strength. In case where the geotextile 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 geotextile rather than mobilise the tensile force in the geotextile.

The following sequence is recommended for construction of embankments on soft soil with geotextile as a base layer, John (1987).

(a) Place the geotextile layer

(b) Place fill at the embankment edges

(c) Fold over the geotextile ends

(d) Place more fill at the embankment edges

(e) Place fill in the central region

(f) Raise the height of the embankment edges

(g) Complete embankment construction

The sequence of operations is illustrated in Fig. 4.5.

In practice the movement of soft soil beneath the geotextile reinforcement is very large and there is a risk of opening and initiating a failure even when the overlap width is substantial. Therefore, the geotextiles are laid and stitched together. The stitching thread should have strength and durability characteristics that either match or exceed those of the geotextile.

After the geotextile has been unrolled and site stitching completed, it should be lightly tensioned by hand to remove any wrinkles or folds. An initial fill cover of minimum 30 cm should be provided over the geotextile before any plant can traverse the area. The rate of placement of fill should ideally proceed at the same pace along each edge of the embankment. This minimises the risk of lateral sliding of the fill and the geotextile over the sub-soil.

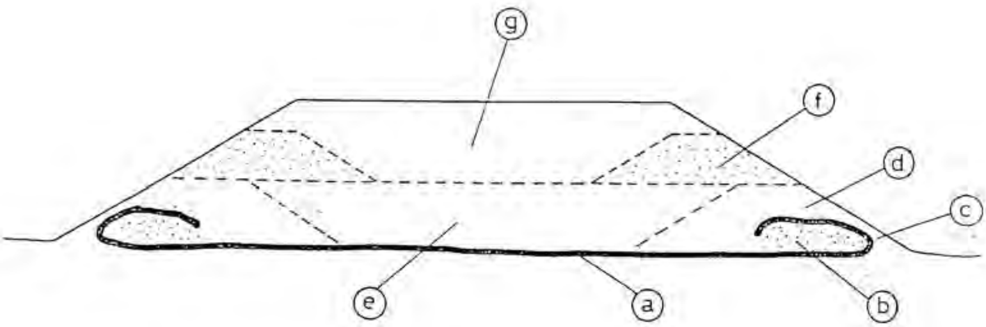

Fig. 4.5. Recommended Embankment Construction Sequence (John, 1987)

Lack of experience with this technique can very easily initiate a circular slip failure. The use of plant producing high contact pressure should also be avoided particularly during early stages of the embankment construction.

4.6. Seam Strength

During the construction as described in the preceding section the geosynthetic are placed in rolls and sewn in the field. The strength of the stiched joint often called seam strength is less than the strength of the continuous geosynthetic fabric. It therefore becomes a limiting strength requirement in the design and it should not be ignored.

4.7. Case Histories

4.7.1. Case histories - 1

Road construction in South East Alaska involved extensive crossings of muskeg terrain (saturated soft peaty soil). A test section of construction of fabric reinforced roads across muskeg was located about 14 km south of Pelersburg, Alaska, Bell et.al. (1977). The average depth of soft muskeg peat was about 3m. The shear strength ranged from 2.4 to 17.2 kPa. The saturated water content was 960 per cent. The section was planned to illustrate the differences in granular embankment thickness if any, with and without geotextiles. The various sections were constructed to show the effects of fabric. Basically it consisted of granular fill placed directly on muskeg and granular fill on both a single and double layer of fabric.

The fabric used was a nonwoven, needle punched, spunbonded polypropylene. Bell et. al. (1977) conclude the following points:

(i) The main function of the fabric is to prevent local bearing failures

(ii) When shear failures do not occur the embankment settlement is essentially the same with or without fabric and independent of fabric type.

(iii) Other condition being equal, the tension in the fabric depends upon the modulus of the fabric. The tension increases as modulus of the fabric increases.

4.7.2. Case history - 2

Construction of a 6.62 m high bridge approach embankment over weak marsh deposits in Mobile, Alaska was made possible through the use of a variety of geosynthetics resulting in an estimated savings of U.S. $ 600,000, Lockett et. al. (1988). Bridge deterioration and updated vertical clearance requirement for the waterway necessitated the building of a new bridge in vicinity of existing bridge having a vertical clearance of 42.68 m and a main span length of 237.80 m, on Tennessee - Tombigbee Waterway. In order to avoid the sharp curve on the east approach of the old bridge, the new facility was extended to a length of 2222.86 m with a curve alignment that swung across a dredge disposal area and marshland on Blakely Island. Trial boring along the shoulder of the existing embankment indicated a preconsolidated condition of the marsh deposits due to the imposed stresses of the existing highway. The surface of marsh outside the existing embankment was extremely soft and it was impossible to transverse by foot. The shear strength of the virgin marsh was estimated to vary linearly from 0 at surface to 10.4 kPa at the bottom of the stratum.

Preliminary calculations showed that settlement from a combination of both foundation and consolidation could range from 1.22 m to 2.13 m beneath the design embankment and would require several years to achieve 90 per cent consolidation. Therefore sand wicks with preloading were specified to accelerate the consolidation process. This would also eliminate post construction settlement and negative skin friction on the abutment piling. The weak shear strength of the marsh deposits dictated that stage construction be used to avoid deep seated shear failure. Calculations showed that the time required for each stage to reach a degree of consolidation where the gain in shear strength was capable of supporting the next embankment segment would stretch the construction schedule beyond acceptable limits. It was, therefore, decided to use geosynthetics at the base of embankment to achieve the desired factor of safety during interim period of consolidation. With the use of geosynthetics as a basal reinforcement, the embankment was built at a controlled rate to the full embankment height and surcharge height without interim waiting period. Six layers of geogrids were required to achive a minimum factor of safety of 1.20 (from stability analysis) and laid. Geogrid did not extend throughout the full width of the embankment. Sand wicks were installed at a spacing of 1.22 m in a triangular pattern. The embankment was constructed at a rate of 0.30 m/day. A typical section of the reinforced embankment is shown in Fig. 4.6.

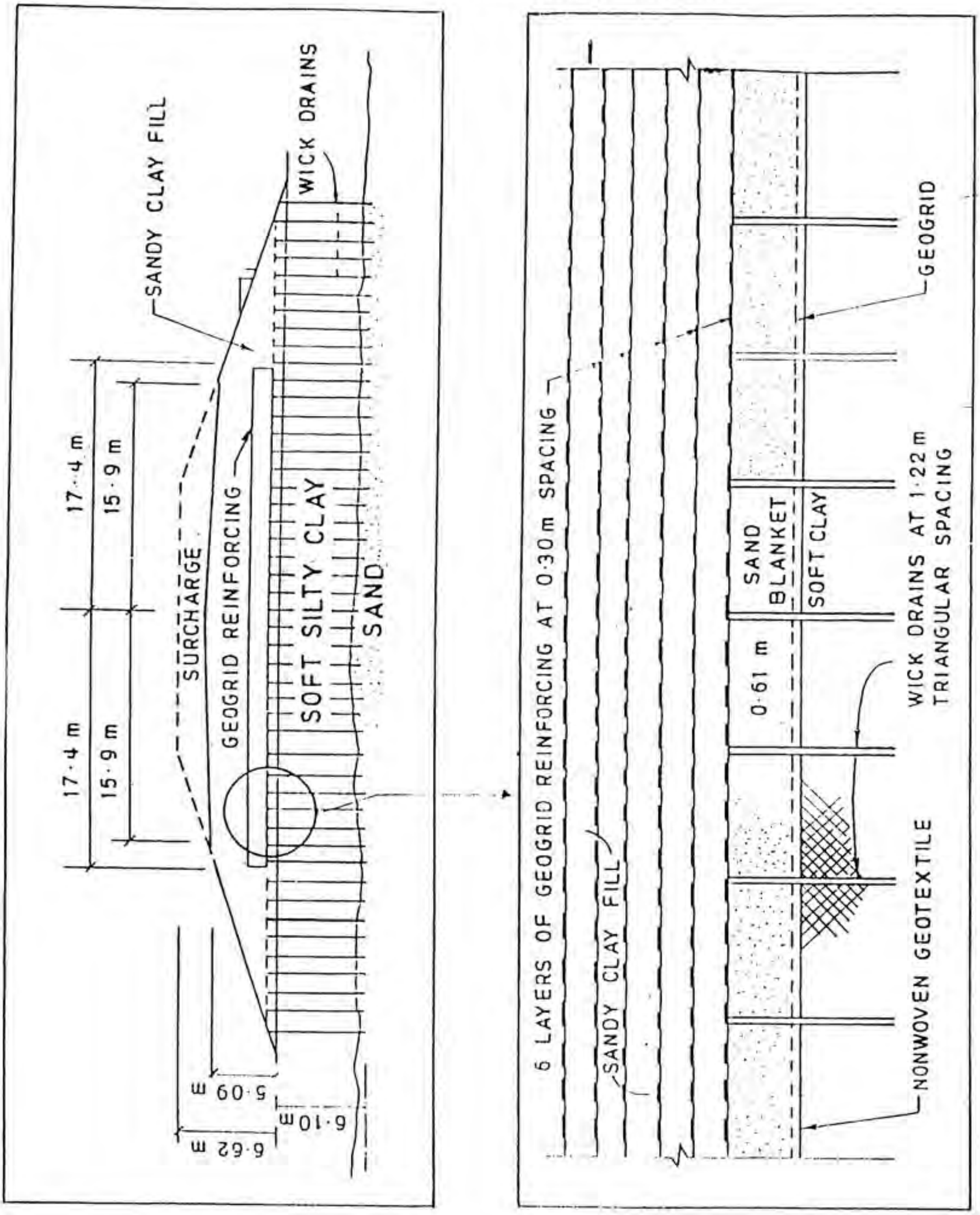

Fig. 4.6. Typical Section of Reinforced Embankment (Lockett et.al., 1988)

The embankment was adequately instrumented with piezometers and settlement plates. Both the piezometers and the settlement data showed that the 100 per cent consolidation mark was achieved within the specified surcharge time frame. No evidence of embankment cracking, lateral spreading or shear displacements in the marsh either during or after embankment construction has been reported.

4.7.3. Case history - 3

Nhava Sheva port project near Bombay is the first of its kind in India where geotextiles have been extensively used Dabke et. al. (1988). About one million square metre of geotextiles was used. Geotextiles of varying strength were used for separation filteration and soil reinforcement function.

The project involved construction of deep water bulk and container berths, roads and reclamation bunds. The bunds as well as the reclamation was to be done on very soft marine clay. The depth of the soft clay varied from 6 to 20 m in depth. The compression index varied from 0.66 to 0.96 and shear strength 0.6 to 20 kPa. The natural moisture content was close to liquid limit. The layout of the project is shown in Fig. 4.7. Since the sub-soil is very soft in nature therefore embankment construction would have necessitated use of side berms and flatter side slopes together with small lift thicknesses and sufficiently long waiting periods for the required consolidation to occur. This traditional method would have resulted in long construction time. Large quantities of fill material would also have been required. This would have resulted in an enormous cost. Therefore considering the overall requirements, woven geotextiles were specified. Selection of geotextiles was done on the basis of soil conditions, superimposed loads and project time schedule. The stability of embankment bunds with geotextiles was checked for following four failure modes.

i. Bearing capacity
ii. Internal stability
iii. Foundation stability
iv. Overall stability

The increase in shear strength of soil as the excess pore water pressure dissipates from soils was considered in the design to achieve economy . Geotextile of different strength were selected for various section of the road bunds and reclamation bunds. The selection of geotextile was based primarily on the functional requirement of the geotextile at different location. Low strength geotextile was applied as separation layer under road bunds R 1, R 2, R 3, R 4, R 6 & R 7 where ground was not generally subjected to flooding. In the road bund portion R 8 where creeks traverse the road section high strength geotextile of 12 t/m was used due to localized poor conditions of the soil. As a filter and separation layer geotextile of 2.75 t/m x 2.75 t/m was applied in S1 & S2 areas. The guide bund and S3 bund required multiple layer of geotextile or a single layer which would provide 200 kN/m at 5 to 6 per cent total strain for 10 years. A woven polyester fabrics with ultimate short term strength of 400 kN/m was selected to provide 200 kN/m design force. The details of geotextiles used are shown in Table 4.2.

SL. NO.	AREA	CLASSI-FICATION	LIQUID LIMIT %	PLASTIC LIMIT %	COHESION kg/cm2	COMPRESSION INDEX	INITIAL VOID RATIO
1.	ROAD R1 to R8	CH	106	40	0·0775	0·883	2·535
2.	GUIDE BUND	CH	102	39	0·088	0·74	2·553
3.	S1 S2 & S3	CH	109	37	0·0567	0·902	2·862

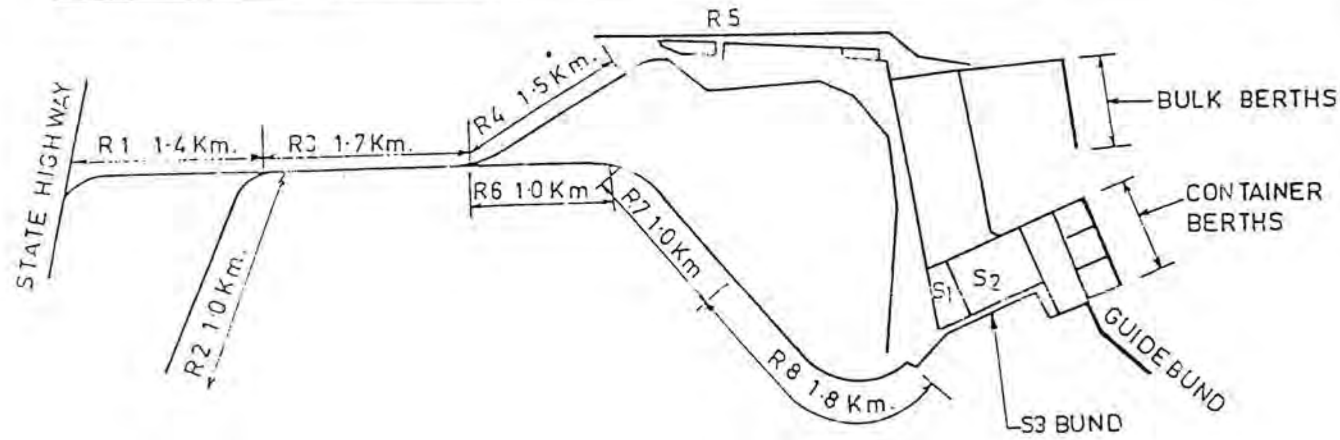

Fig. 4.7. Location Plan of Nhava Sheva Port

Table 4.2. Details of Geotextiles

SECTION	FUNCTION	STRENGTH		FABRIC	MATERIAL	ELONGATION AT BREAK %		WEIGHT OF FABRIC g/m^2
		WARP	WEFT	WARP	WEFT	WARP	WEFT	
R1 to R7	Separation Layer	1.4	0.9	Poly-Propylene	Poly Propylene	16	13	65
R8	Reinforcement		4	Polyester	Polyamide	10	20	300
S3 BUND	Reinforcement	40	8	Polyester	Polyamide	10	20	850
S3 ROAD	Reinforcement	40	8	Polyester	Polyamide	10	20	850
S1 AREA	Drainage	2.8	8	Polyester	Polyester	12-15	12-15	110
S2 AREA	Drainage	2.8	2.8	Polyester	Polyester	12-15	12-15	110
GUIDE BUND	Reinforcement	40	9	Polyester	Polyamide	10	20	850

4.8. Construction Aspects

Geotextile were spread with its warp direction perpendicular to the road bund. After spreading the geotextile was shown by electric sewing machine in French seam. Seams were made parallel to warp yarns. During rains when electric sewing machine failure, then 0.5 m to 1.0 m laps were provided depending upon the condition of ground, 'U' shaped pins made out of 12 G galvanised wire, 500 mm long were used to anchor the fabric to the ground at 500 mm centres. A drainage blanket of 600 mm was provided. Therefore, band drains were installed.

In S1 and S2 areas, no band drains were required. Geotextiles were laid on the soft ground and the fill material was placed by end dumping by trucks. However, this procedure resulted in local overloading leading to formation of mudwaves, ballooning of geotextiles and also tearing of geotextiles in some places. The problem was overcome to a limited extent by placing lesser quantities of fill materials during low tides.

In deeper areas of S2, the geotextiles was placed during high tide. The fabric 2.75 t/m x 2.75 t/m was floated out during high tide using a pontoon. The fabric was held in position by ropes anchored to the ground. The fabric was stitched by men in boats. Sand bags were kept on the geotextile to keep it in place. In shallow areas of R8, S1 & S2 the geotextile was spread manually during low tides and stitched. The details of geotextiles used in various sections are shown in Table 4.2.

Pontoons were used for spreading the geotextile under water for guide bunds in the sea. 20 x 50 m sizes of geotextiles was placed with ropes attached to it. Floats were used to keep the fabric floating. The geotextiles were stitched in the floating position. The geotextiles was weighed down with MS hooks and concrete blocks. Sand bags were thereafter put by divers to keep the geotextile in position. The fill material was subsequently put slowly.

Dabke et.al. (1988) conclude that judicious use of geotextiles for reclamation work and bund construction on soft marine clay resulted in savings in fills material, construction time and overall cost.

4.9. Conclusions

Geosynthetics are being increasingly employed in Highway Engineering to facilitate construction of road embankments on soft soils. Geosynthetics are employed as a horizontal reinforcement at the base of the reinforcement for construction embankment on soft clay. Their main functions are

(1) To prevent mixing of the borrow fill with soft sub-soil resulting from local bearing capacity failure.

(2) To prevent excessive vertical and horizontal deformations.

(3) To compensate for low shear strength of the soil that might lead to a slip circle failure.

The quantity and the mass of fill material also gets reduced. Lot of experience in the construction procedure is essential to make this technique effective. Lack of experience with this technique can very easily initiate a circular slip failure.

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5. DYNAMIC CONSOLIDATION

5.1. Introduction

This technique was introduced in France by Louis Menard (1972) and was initially known as Heavy Tamping. Its field of application then covered principally compacton of ballast fills or natural sandy gravel soils. However, recently this technique has been used for improving saturated clays or alluvial soils. From then onwards the technique has taken the name 'Dynamic Consolidation'.

Dynamic consolidation involves repeated dropping of heavy weights on the ground surface from great heights following a well defined pattern as regards time and space appropriate to the site. A key feature of this technique is its ability to engender a substantial depth, normally of the order of 20-25 m below the ground surface. 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.

The method has been successfully applied on many types of soils ranging from rockfill to clayey material and even peat. Due to the development of new sturdy hoisting devices, substrata as deep as 40 m has been improved using the technique.

The method has been used for various types of projects, e.g., embankments on soft soil, tank farms, storage areas, industrial estates, marine and other reclamation development projects.

5.2. Design and Practical Aspects

It is important to ascertain before using this technique with some precision, the improvement that may be expected in the foundation soil. Operating method, total energy, number of phases etc.are essential for the successful design and implementation of the method.

5.2.1. Preliminary investigation

Before considering the site for dynamic consolidation a detailed soil investigation programme should be formulated. The soil investigation should include:

(a) Sufficient boreholes for stratigraphic description representative of the site. In-situ tests like pressuremeter test, vane shear test should also be carried out.

(b) Laboratory tests including dynamic consolidation tests in dynamic oedometers should be carried out, Menard et.al., (1975). Although it may be difficult in the laboratory to simulate the real behaviour of the soil, the results so obtained have permitted fairly accurate determination of the anticipated response of the soils to the heavy tamping.

5.2.1.1. Dynamic oedometers

This apparatus is patented by Menard, et.al. (1976) and is shown in Fig. 5.1. It is a composite form of a triaxial apparatus and a conventional oedometer. It measures successive static consolidation of a 30 cm sample under the influence of static and dynamic loads. The pore water pressure, lateral pressure and corresponding settlements are measured as a function of time. The test permits the determination of saturation energy and number of phases to obtain certain densification and delay between passes, etc. with reasonable accuracy.

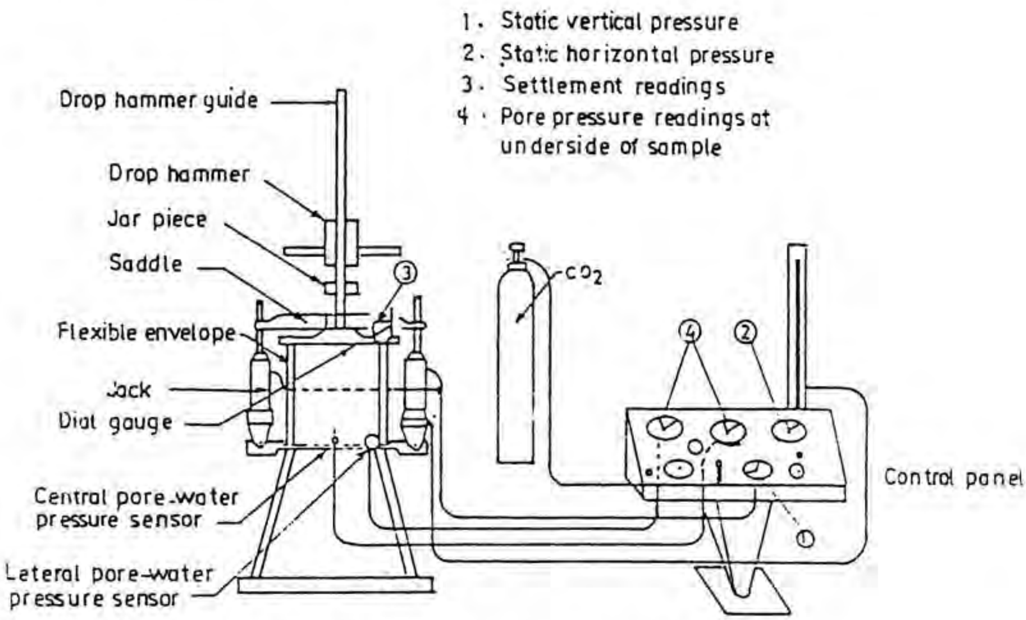

Fig. 5.1. Dynamic Oedometer (Patented) (Menard et.al., 1972)

5.2.2 Methodology of technique in field

The technique is performed by dropping repeatedly heavy pounders of 10-40 tons from a height of 10 to 40 m on the ground surface. The pounders used for heavy tamping may be concrete blocks, steel plates or thick steel sheets filled with concrete or sand. The pounders are usually square or circular in plan and have dimensions of upto a few meters depending on the weight required, and the material. The drop weights are hoisted using crane hoists.

For a large area, several repetitions at points spaced several meters apart in the grid pattern, are applied. A typical treatment will result in an average of 2 to 3 blows per metre . The representative equipment used is shown in Fig. 5.2. Two to three coverages 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 time interval between coverages may range from a day, for freely draining coarse sands, to weeks for fine grained soils. The ground surface is usually levelled between the coverages. To ensure uniformity and high density in the near surface zone, surface ironing is done by small impacts of the pounder over the entire surface. Zheng et. al. (1980) determined the effective deformation defined as the volume of crater less the volume of adjacent heave from displaced soil. In the clay the deformation was 30 per cent effective and in the sand it was 62 per cent effective. When the site is compacted uniformly over the whole surface, a peripheral fringe appears with characteristics which are intermediate between the exterior non compacted and the interior compacted zones. The fringe follows the line of perimeter and has a width equal to twice the thickness of the layer being consolidated. It is necessary to provide width relative to the area to be effectively prepared.

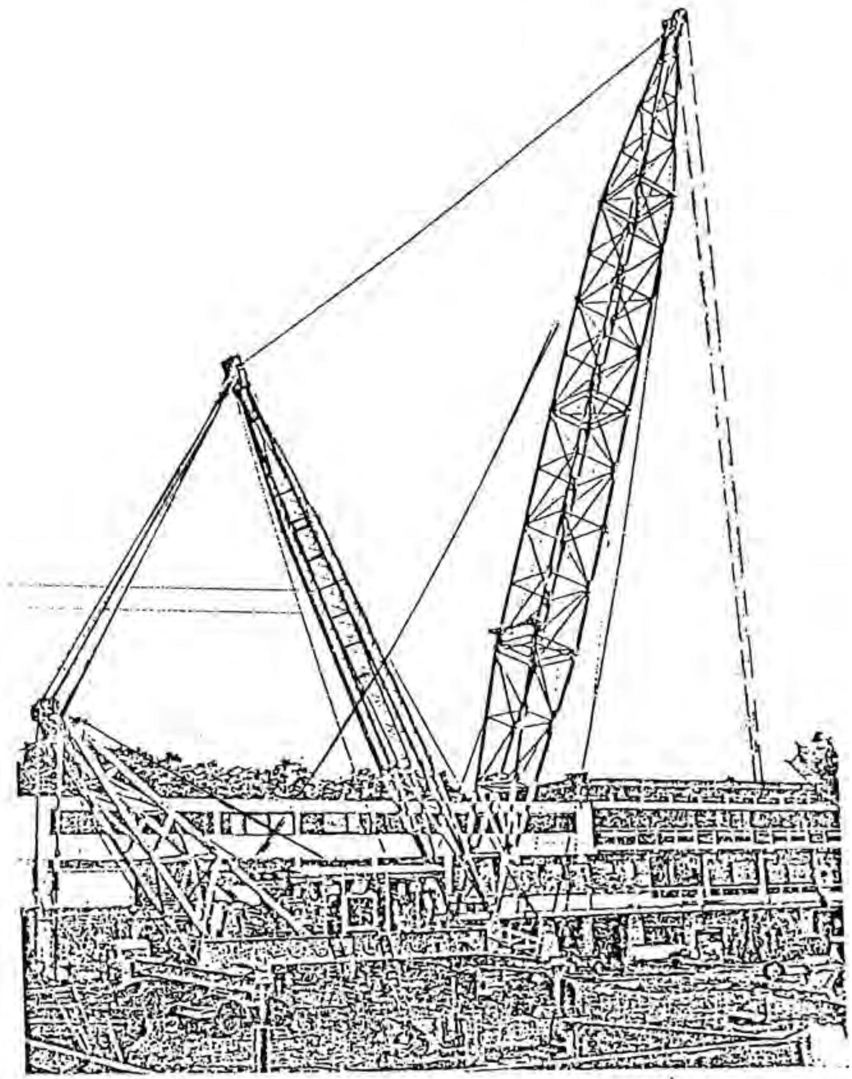

Fig. 5.2. Representative Equipment used in Dynamic Consolidation

5.2.3. Influence Depth

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. A crane drop is less efficient than a free drop. Soft clay and peat have a damping influence on dynamic forces, therefore, they cannot be strengthened to the same level as coarser materials. Many emperical relations have been suggested by different Authors.

Menard and Broise (1975) proposed the relationship

$$D_m = \sqrt{W_d . H_d} \qquad \text{Eqn. 5.1}$$

Where

D_m = Maximum depth of influence in m
W_d = Falling weight in metric tons
H_d = Height of drop in metres

Leonards et.al. (1980) analysis seven cases and concluded that:

$$D_m = 1/2 \sqrt{W_d . H_d} \qquad \text{Eqn. 5.2}$$

Luckes (1980) analysis eight cases and concluded that

$$D_m = (0.65 \text{ to } 0.80) \sqrt{W_d . H_d} \qquad \text{Eqn. 5.3}$$

A number of field experiences are summarised in Fig. 5.3, Mitchell (1981). The plotted points show the results of heavy tamping on soils ranging from silts to clean sands and rubble fill.

5.2.4. Site preparation

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 to a depth of 2-3 m and back filling with sand and gravel. Perforated plastic pipes are placed at the bottom of the trench.

5.2.5. Compaction control

Compaction control during the tamping operation is carried out by adopting the following procedure:

(i) Pressuremeter and penetration tests are used for the measurement of strength and compressibility. They are carried before compacting and after each compaction round. The test results are greatly influenced by the delay at which they are carried out after each pass. Generally for coarse grained soils, delay of 3-4 days and for fine grained soils a minimum delay of 3-4 weeks should be observed.

(ii) Piezometers should be installed at different locations to determine the minimum delay between each pass.

(iii) Gamma densitometer and water content measurement of samples should be done to check the variation of dry density.

(iv) Topographical measurements of the ground surface should be done for determination of the overall variations of dry density.

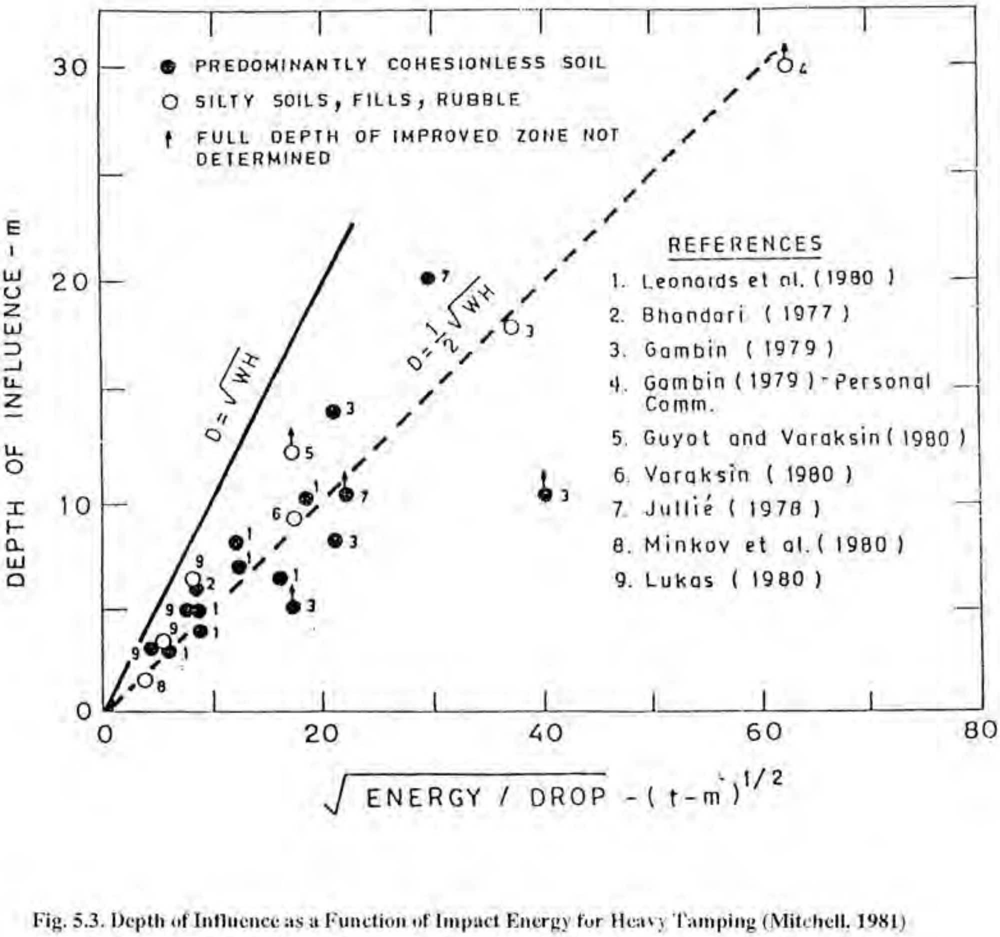

Fig. 5.3. Depth of Influence as a Function of Impact Energy for Heavy Tamping (Mitchell, 1981)

5.2.6. Settlement reduction

The tamping produces a true pre-settlement (Wc) well beyond the settlement (Wo) which would have occurred as a result of the weight of construction only without any preliminary consolidation. The ratio Wc/Wo measured after dynamic consolidation varies between 2 to 3 compared with the values of 0.8 - 0.9 usually obtained in the case of traditional static preloading. The secondary settlements are also reduced.

5.2.7. Vibrational effects

The vibrations produced by the impact are relatively large and may prohibit the employment of this technique in urban areas. The usual frequencies of the vibrations caused by tamping vary between 2 - 12 HZ. The wave velocity is very low in the zone liquefied by tamping and increases as it moves away and becomes normal at great distance. A 30 m from the point of impact the vertical and horizontal velocities of the soil particles remain much below the value of 50 mm/sec. which is generally acceptable for ordinary buildings. The amplitude of the vibrations is slightly influenced by the height of fall of the pounder, but increases appreciably with the area of contact.

5.3. Theoretical Considerations

5.3.1. Menard and Broise (1975) have proposed a theory which forms the basis of the success of this technique.

They assumed the following:

(a) Compressibility of saturated soils due to presence of micro bubbles.

(b) The gradual liquefaction under repeated impacts.

(c) The changes of permeability of a soil mass due to the presence of fissured and/or the state of near liquefaction and the state played by absorbed water.

(d) Thixotropic recovery

(a) Compressibility: Research has shown that most quaternary soils contain gas in the form of micro bubbles, the content varying between 1 - 4 per cent. It is difficult to measu this gas but the immediate settlement of 50 -60 cm under impact, indicates its presence and it can be seen coming from the surface.

(b) Liquefaction: As energy is applied to the soil under repeated impact, the gas gradually becomes compressed. The soil starts to react as an incompressible material as the percentage of gas by volume approaches zero. At this stage the liquefaction of the soil begins to take place. The energy level required to reach this stage is termed as saturation energy.

In practice, liquefaction in natural soils occurs gradually. It is, therefore, imperative to know the precise level of energy corresponding to this threshold condition which is essential to develop high pore water pressures as well as high permeabilities. The dynamic oedometer is used in laboratory for this purpose. Once the saturation energy is achieved, further application of energy would be entirely wasteful.

(c) Permeability: When the conditions are close to liquefaction, large hydraulic gradients are developed and permeability of soil increases. This phenomenon, according to Menard et.al. (1975), is general and apparent in all soils. A very slight local increase in pore water pressure is sufficient to start the tearing of the solid fissures by splitting and quite naturally, the flow of water concentrates in these newly created fissures. By concentrating the tamping energy at regular grid locations vertical fissures are created which are distributed regularly around the impact point. These preferential drainage areas are generally perpendicular to the direction of lowest stress. Fountains of water which under certain geological condition appear near the craters a few hours after tamping are indicated and fed by this flow network.

(d) Thixotropic recovery: During tamping operation, a considerable fall in shear strength is noted, the minimum being recorded when the soil is liquefied or approaching liquefaction. The soil skeleton is completely torn and the adsorbed water which pays an important role in stiffening of the structure is partially transformed into water. As pore water pressure is dissipated a large increase in shear strength and deformation modulus is noted. This is due to the closer contact between the particles as well as the gradual fixation of new layers of absorbed water. The general response of the ground as a function of time after a coverage based on foregoing point is illustrated in Fig. 5.4.

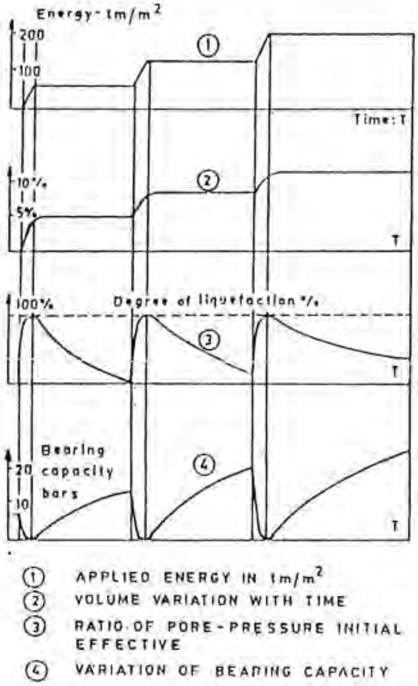

Fig. 5.4. Ground Response with Time After Successive Coverages of Dynamic Consolidation (Menard et. al., 1975)

5.3.2. Gabin theory (1979)attributes the compaction of ground due to shock wave generated in the ground under the influence of the impact of loads. According to Gabin's theory the compression wave mostly travels through the water phase and shakes the soil skeleton by successively increasing and decreasing the pore pressure untill the soil skeleton dislocates. The shear waves or Raleygh travel at a lower velocity and subsequently rearrange dislocated grains into a dense state. Consequently in submerged soils the shear waves are the densification waves.

5.4. Case Histories

5.4.1. Case history - 1

For a warehouse contruction in Singapore a 3.0 - 4.5 m depth of peaty clay overlain by a deposit of 3.5 -6.0 m of medium stiff clay posed a problem for foundation contruction, (Ramaswamy et. al., 1979). The sub-soil compression index varied from 0.23 to 0.81. Void ratio varied from 0.81 to 3.11 and the shear strength of the soil varied from 11 kN/m^2 to 72.3 kN/m^2 The main requirement criteria for the treatment of the site were as follows:

(i) Allowable bearing capacity of treated soil upto 2.0 m should not be less than 10 t/m^2 with FOS of 3.

(ii) Total settlement not to exceed 50 mm under a loading intensity of 10 t/m^2.

(iii) Post construction differential settlement not to exceed 1 in 1000 under the loading intensity of 10 t/m^2.

With the above minimum requirement criteria, dynamic consolidation was considered to be an economical ground improvement technique.

Based on the pre-engineering studies the entire site was divided into three zones as shown in Fig. 5.5. The entire site was covered with a layer of sand 0.50 -0.75 m thick in order to facilitate the moving of hoisting machines. A pounder of 1.83 m x 1.83 m in size weighing 15.5 tonnes with a height of fall of 30.50 m was used. Four passes and 4-6 blows per pass was used. The spacing of the pounder blows was 3.65 m centre to centre in all the passes. Sufficient time of the order of 3 to 4 weeks, was allowed for the dissipation of pore water pressure between subsequent passes. Fig. 5.6 shows the typical development and dissipation of pore water pressure. After completing the required number of passes the treated ground was made level by a continuous pass generally known as the ironing pass. Considerable improvement in the soil properties has been reported. Fig. 5.7 shows the bearing capacity of sub-soil before and after treatment. Maximum total settlement of 48 mm under a loading intensity of 10 t/m^2 has been reported.

5.4.2. Case history - 2

Construction of an embankment to carry a new dual carriageway road across the flood plain of the River Wey, Guilford, U.K. presented a stability problem due to the restricted width of land available and low undrained shear strength of alluvial silts (Charles et. al., 1981). The undrained shear strength of soft soil was 2.5 t/m^2 and dynamic consolidation was considered an economical alternative to improve the undrained shear strength. The entire area was covered by a one metre layer of granular material. Drainage trenches were excavated and then filled with granular material. Piezometers were installed to monitor the pore pressure. A pounder of 4 m base area weighing 20 tons with a heights of fall of 20.0 m was used.

Two passes and 10-15 blows per pass was used. Craters upto two metres deep were created in the surcharge and then backfilled. The average total energy input over the whole area was 160 t/m. Fig. 5.8 shows the typical development and dissipation of pore water pressures.

Considerable improvement in the soil properties has been reported. The undrained shear strength increased to the order of 3 - 4 t/m^2 after two months of tamping.

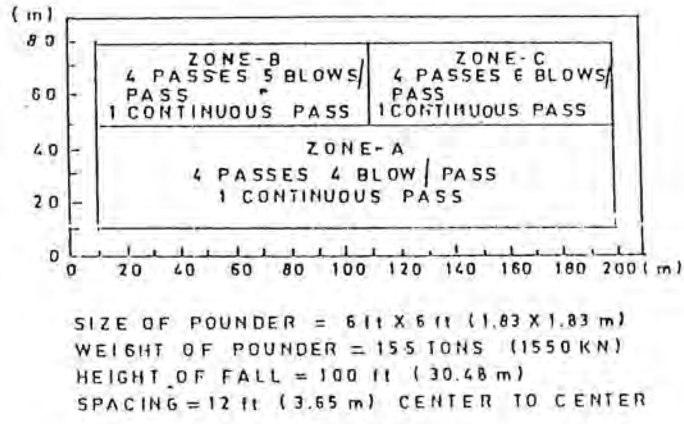

Fig. 5.5. Site Showing Zones, Number of Passes in Each Zone and Number of Blows Per Pass (Ramaswamy et.al., 1979)

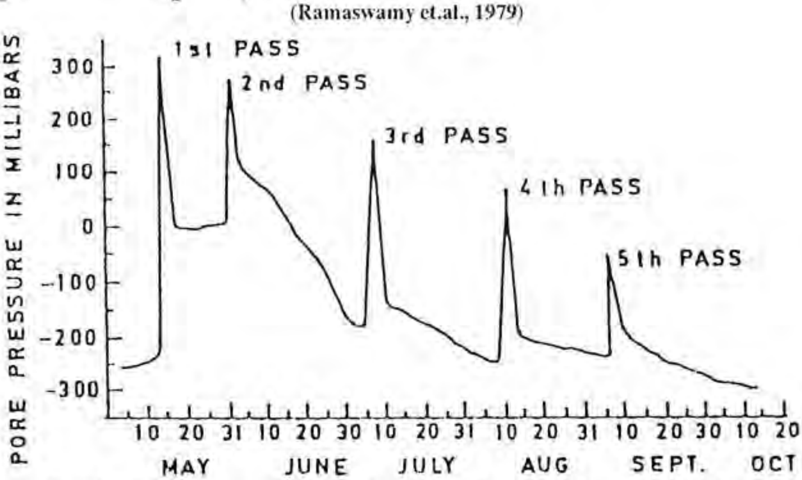

Fig. 5.6. Development and Dissipation of Pore Pressure Piezometer : Station 2 (Ramaswamy et.al., 1979)

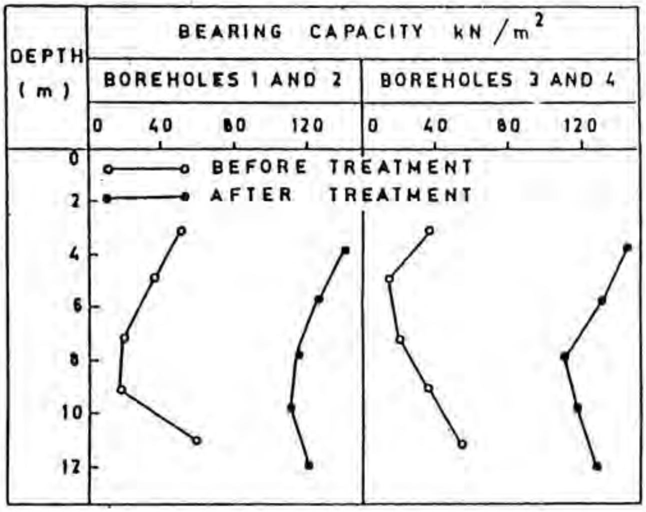

Fig. 5.7. Bearing Capacity Before and After Treatment (Ramaswamy et.al., 1979)

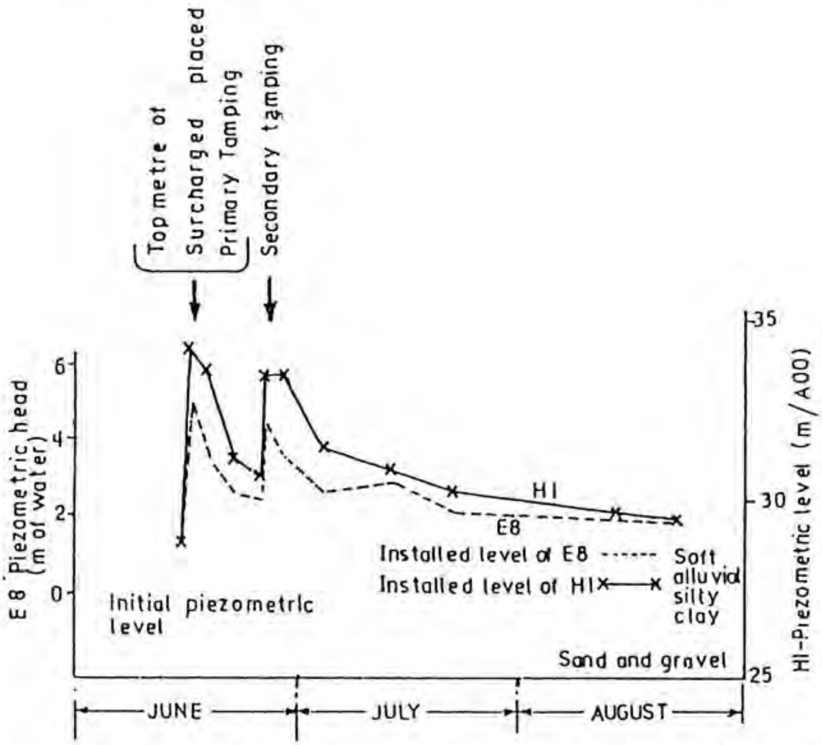

Fig. 5.8. Overall Pore Pressure Response to Dynamic Consolidation of Soft Alluvial Soil at Guilford (Charles et.al., 1981)

5.4.3. Case history - 3

Consolidation and compaction piling was done to improve sub-soil condition for a refinery complex at Bongaigaon in Assam, (Bhandari et. al., 1977). The general conditions at site consisted of 0.6 m of top silty sand followed by about 10 m by a deposit of loose to medium dense saturated sand mixed with gravel and cobbles about a depth of investigation of 18.5 m. A geological and seismotectonic study of the area revealed that Bongaigaon lies in an active seismic zone. Therefore liquefaction of the loose saturated sand during an earthquake was a major concern. To counter liquefaction, the sub-soil was suggested to be densified to 80 per cent relative density. To achieve this, following criteria was adopted. SPT value will be not less than 10 at surface and not less than 25 at 10.0 m depth with a linear variation between them. Compaction piles and dynamic consolidation were adopted to densify the trial tank areas. For dynamic consolidation compaction point were spaced at 5.0 m in primary pass with 10 blows/pass. Fig. 5.9 shows the pre and post Nc values from the dynamic cone penetration test after the first phase of tamping. It is seen that the effect of dynamic consolidation was limited to 4.5 m below the ground surface. Since the compaction did not penetrate to a desired depth of 6.0 m, a secondary pass was done at a spacing of 2.5 m with 10 blows/pass. A total subsidence of 28.0 cm has been reported. Post densification SPT values after the second phase of tamping is shown in Fig. 5.10 and show significant improvement upto a depth of 6.0 m. Post densification SPT values decreased in the dense strata. It is likely that the hammering effect of the pounding weight shears the dense sand and causes it to dilate, thereby, resulting in reduction of N values.

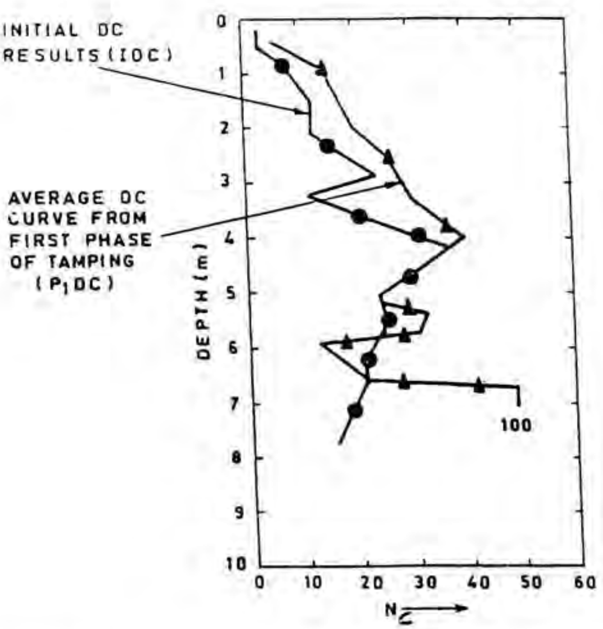

Fig. 5.9. Comparison of N_c Values Before and After First Phase of Tamping in Dynamic Consolidation (Bhandari et. al., 1977)

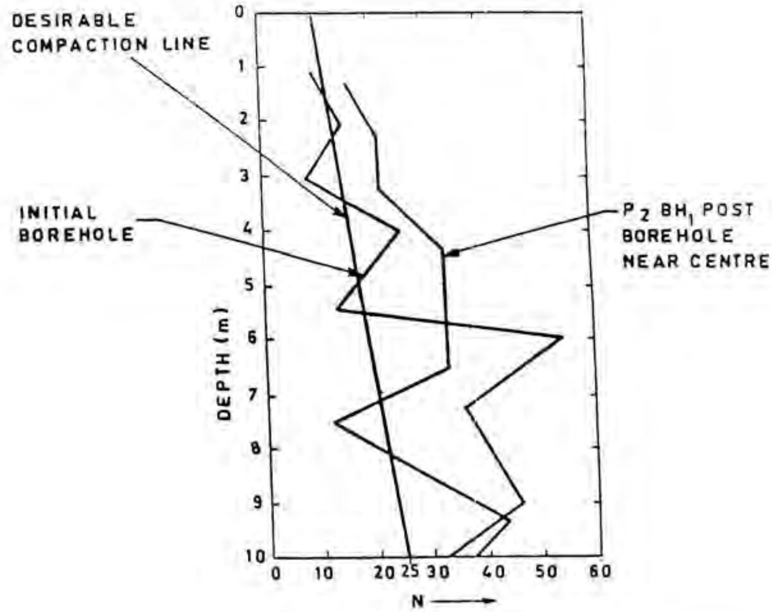

Fig. 5.10. Comparison of N Values Before and After Second Phase of Tamping in Dynamic Consolidation (Bhandari et.al., 1977)

5.5. Conclusions

(1) Dynamic consolidation is an effective ground improvement technique for loose sands, soft clays and peats.

(2) Unless a very large site is required for treatment it would probably be not economical to use this technique.

(3) Theories have been proposed by various authors for the technique, however the mechanism is not yet clearly understood. Further laboratory and field studies are required.

(4) Laboratory and field studies should be conducted in different soils and conditions to determine the depth of influence vs impact energy.

(5) Laboratory studies for estimating the long term settlements of the improved ground are required to be carried out.

(6) Laboratory studies to determine and analyse the compressibility of soil due to microbubbles, saturation level, energy required, and extent of thixotropic recovery achieved under the dynamic stresses.

(7) Laboratory studies to determine the state of stress in soil under dynamic loading conditions.

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6. INSTRUMENTATION AND MONITORING

6.1. Introduction

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:

(i) Build up and dissipation of pore pressures.

(ii) Rate and magnitude of the vertical settlements of the sub-soil under the applied embankment loads.

(iii) Horizontal spreading, if any, of the sub-soil under the applied loads.

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. Monitoring and observational procedure must be adopted with due care, if the same is to be successful. According to Peck (1969), "The observational method is not without its pit falls and limitations. It should not be used unless the designer has in mind a plan of action for every unfavourable situation that might be disclosed by the observations. The observations must be reliable and must reveal the significant phenomenon, and, must be so reported as to encourage prompt action".

The concept of design and construction of an earth structure based on observational procedure and instrumentation, involves the following steps.

(a) Design of the earth structure with the help of available field and laboratory data concerning soil parameters.

(b) Prediction of the response of the structure 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.

(c) 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. It may also be possible to optimise the design as compared to the one initially envisaged. In addition, the wealth of field data generated, would form the basis on which the present theories can be evaluated, improved and brought closer to the actual conditions.

6.2. Prediction of Pore Pressures

Various theories have been proposed in the past to evaluate the pore pressures induced by total stress variations in clay masses, Skempton (1954), Henkel (1960) and Burland (1971).

6.2.1. Skempton's Method

Skempton (1954) gives the following equation for the estimation of pore water pressures:

$$\Delta u = B\,[\Delta\sigma_3 + A\,(\Delta\sigma_1 - \Delta\sigma_3)] \qquad \text{Eqn. 6.1}$$

where, A, B are called Skempton's pore pressure coefficients.

The application of the stresses $\Delta\sigma_1$ and $\Delta\sigma_3$ on a soil element under embankment loading may be considered as taking place in two stages as shown in Fig. 6.1. Firstly, the soil element is subjected to an equal all round increment of pressure and secondly, it is subjected to a deviator stress increment $(\Delta\sigma_1 - \Delta\sigma_3)$. The pore pressure coefficient A,B are measured experimentally in the undrained triaxial test, Skempton (1954). The coefficient B is sensitive to the degree of saturation. This is overcome by considering the soil as fully saturated. This will result in the value of B equal to unity. Coefficient A is a function of over consolidation ratio, the stress level and the sensitivity of the clay. The selection of representative value of A for a given problem is not easy. Typical values of pore pressure coefficient A at failure proposed by Skempton (1954) for different types of soil are given in Table 6.1. One of the major limitation associated with the Skempton's equation is that it applies strictly to axi-symmetric problems. It is generally the plane strain conditions that prevails under road embankments. The wide experience gained with the application of Skempton's coefficient has shown that the resulting pore pressure predictions are of variable quality and cannot be fully relied upon, Leroueil et.al. (1978).

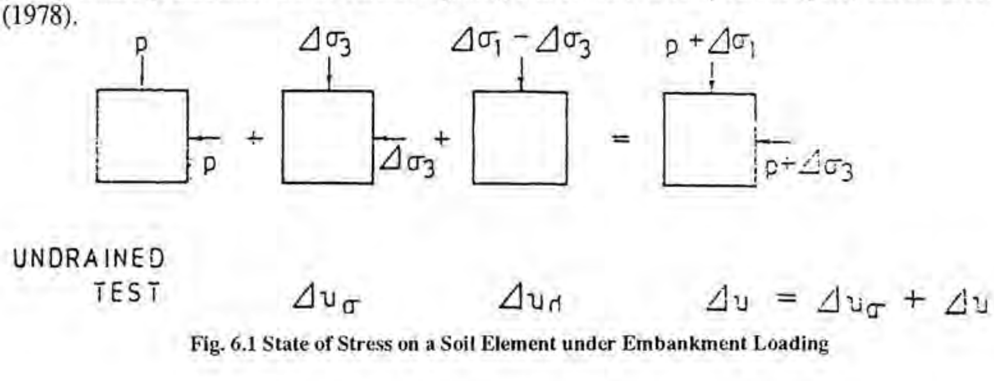

Fig. 6.1 State of Stress on a Soil Element under Embankment Loading

Table 6.1. Typical Value of Pore Pressure Coefficient 'A' at Failure (Skempton, 1954)

Type of Clay	Pore Pressure Coefficient 'A'
Clays of high sensitivity	+ 0.75 to + 1.5
Normally consolidated clays	+ 0.5 to + 1.0
Compacted sandy clays	+ 0.25 to + 0.75
Lightly over consolidated clays	0 to + 0.5
Compacted clay gravels	- 0.25 to + 0.25
Heavily over consolidated clays	- 0.5 to 0

6.2.2. Henkel's Method

A more general formula applicable to any stress state was proposed by Henkel (1960)

$$\Delta u = \frac{\Delta\sigma_1 + \Delta\sigma_2 + \Delta\sigma_3}{3} + a\sqrt{(\Delta\sigma_1 - \Delta\sigma_2)^2 + (\Delta\sigma_2 - \Delta\sigma_3)^2 + (\Delta\sigma_1 - \Delta\sigma_3)^2} \qquad \text{Eqn. 6.2}$$

$$\Delta u = \Delta\sigma_{oct} + 3a.\ \tau_{oct}$$

where

a = Henkel's pore pressure parameter
$\Delta\sigma_{oct}$ = Increase in octahedral normal stresses
$\Delta\tau_{oct}$ = Increase in octahedral shear stress

The usefulness of this equation is that it enables to predict the excess pore water pressure associated with plane strain loading conditions.

For uniaxial test condition where ($\Delta\sigma_1 - \Delta\sigma_3$) can be substituted by $\Delta\sigma_1$; $\Delta\sigma_2$ and $\Delta\sigma_3$ are zero and comparing this equation with Skempton's equation relationship between 'a' and 'A' may be obtained.

$$A = \left(\frac{1}{3} + a\sqrt{2}\right) \;;\; a = \frac{1}{\sqrt{2}}\left(A - \frac{1}{3}\right) \qquad \text{Eqn. 6.3}$$

The difficulties associated with the measurement and selection of A & B has already been discussed in the previous section. However, the difficulty associated with the measurement and relation of Skempton pore pressure coefficients also apply to selection of Henkels pore pressure parameters. Therefore, the quality of pore pressure prediction is still variable.

The above two methods are generally used to estimate the total changes in pore pressure due to changes in total stresses assuming elastic behaviour of soil. Their applications to field problem, such as, embankment foundations where plastic flow or yield can develop locally, has resulted in inaccurate realistic assessment of pore water pressure. However, the application of these techniques in practice has apparently not led to serious problems.

6.2.3. Burland method

Burland (1971) applied the concept of limit and critical state to plane strain problems to account for plastic flow and local yielding of soil. Burland (1971) stated that local yielding prior to failure would result in increase in rate of pore water pressure generation so as to maintain the effective stress condition on the limit state line of the clay upto failure.

All the methods described above apply strictly to the cases where no rotation of the principle stress axes occur. Broms and Casbarian (1965) have shown that stress axes rotation has a strong effect on the pore pressure. Generally, no rotation of principle stress axes occur below the center line of embankment. At other points rotation of axis occur and this should result in wrong estimate of pore pressures.

6.3. Prediction of Settlement

The estimation of magnitude and rate of settlements of sub-soil under embankment loads are discussed in detail in Chapter 2 of IRC HRB Special Report No.13.

6.4. Prediction of Lateral Deformations

The prediction of lateral displacement under the toe of an embankment has become recently possible with the development of finite element techniques. However, experience to date has not been very successful, Poulous (1972) used FEM technique to predict the distribution of lateral displacement with depth under the toe of an embankment and compared with the observed values. It is shown in Fig. 6.2. The Fig. 6.2 indicates that there is a wide variation between the observed and predicted values. This is mainly due to the assumption of a truly undrained behaviour of clay soil under embankment loads.

6.5. Observational Procedure

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 times 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. These parameters, generally, control the stability of embankment. The different types of instruments used to measure the above parameters are stated in the next section.

6.6. Monitoring of Pore Water Pressures

The monitoring of pore water pressures is done by piezometers.

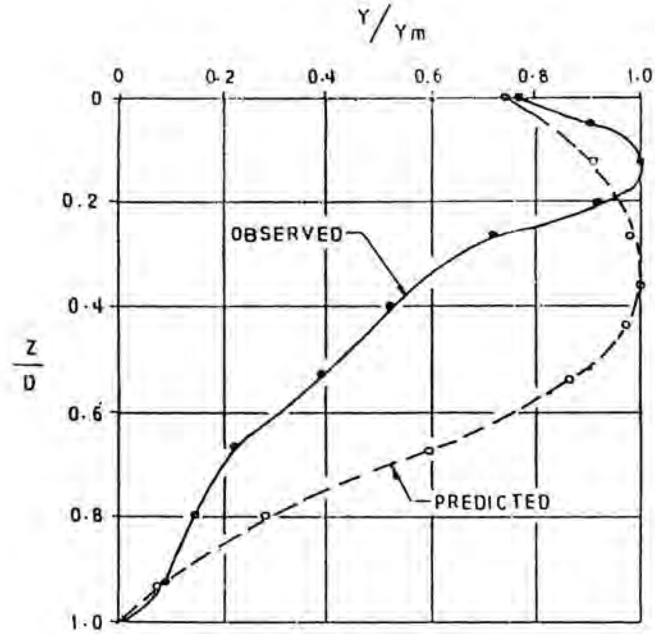

Fig. 6.2. Predicted and Observed Distribution of the Lateral Displacement with Depth under the Toe of an Embankment (After Poulos, 1972)

6.6.1. Piezometers

A piezometer is an instrument for measuring pressure in the pore water. 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:

1. Hydraulic piezometers
2. Pneumatic piezometers
3. Electrical piezometers

6.6.2. 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 and as such the application of the same is described below.

6.6.2. (a) Casagrande open stand pipe piezometer

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. Fig.6.3 shows the Casagrande piezometer and its installation. In order to record a given incremental pore water pressure within the ambient soil. large volume of water is required to flow into the piezometer unit. The time required for equalisation 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. Methods of estimating time lag are presented by Hvorslev (1951), Terzaghi & Peck (1967). Brooker and Lindberg (1965), evaluate time lag effects for twin tube hydraulic piezometers. Branch (1982), Penman (1960), Premchitt (1981) and Vanghman (1974) report on the effects for various types of piezometers.

The response time of open stand pipe piezometer can be estimated from equation given by Penman (1960).

$$t_{90} = 3.3 \times 10^{-6} \; \frac{d^2_s \; \mathrm{Ln} \left[(L/D_i) + \sqrt{1 + (L/D_i)^2}\right]}{K_i L} \qquad \text{Eqn. 6.4}$$

where

t_{90}	=	time required for 90 per cent response in days
d_s	=	inside diameter of the stand pipe in cm
L	=	length of intake filter in cm
D_i	=	Diameter of intake filter in cm
K_i	=	Permeability of soil in cm/sec.

A response time corresponding to 90 per cent equalisation of the out of balance pore pressure should be realistically assessed.

Typical specifications for Casagrande piezometer are as follows:

(i) The piezometer may be of 38mm dia and 300mm in length

(ii) The air entry value should be of the order of 0.3kg/cm^2.

(iii) The standpipes should be more than 16mm in diameter.

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. (In Casagrande piezometer the pore pressure is read by rise in water in the open stand pipe, whose level is measured by a probe).

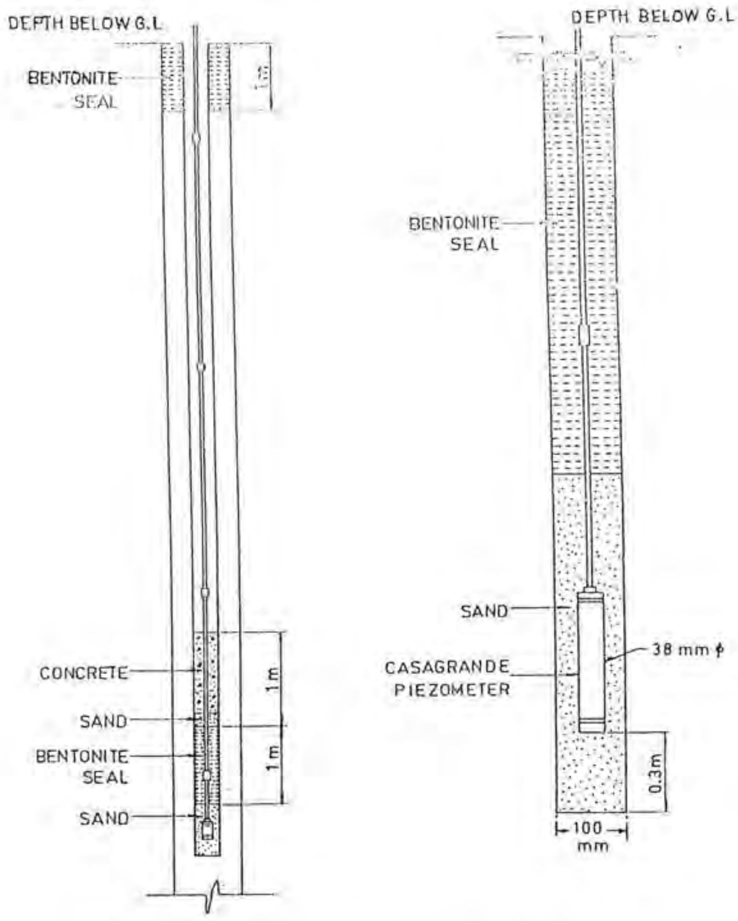

Fig. 6.3. Installation Details of Open Stand Pipe Piezometer (Casagrande Type)

6.6.3. Pneumatic piezometers

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. Fig.6.4 gives details of a Pneumatic piezometer.

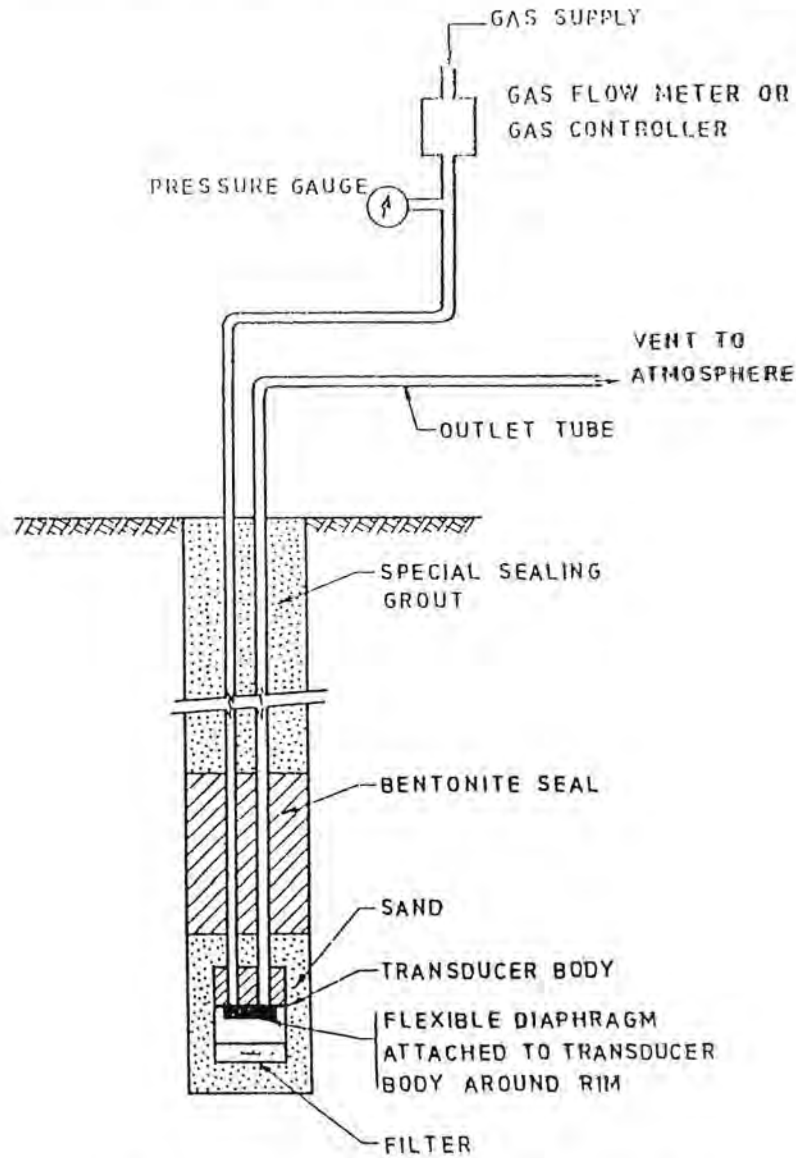

Fig. 6.4. Schematic of Pneumatic Piezometer Installed in a Borehole

6.6.4. Electrical piezometers

The electrical piezometer mostly contain 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 piezometers, especially the Casagrande type is installed in a borehole, at a particular depth and the piezometer tip is sealed to prevent surface water interfearing with the pore water, electrical piezometers can be pushed in the soft soil with special equipment to the desired depth. The electrical piezometer requires to be calibrated before installation. These instruments cannot be calibrated after being installed.

The details of installation procedures for different types of piezometers are discussed in Hanna (1985).

6.6.5. Frequency of installation

Piezometers are generally installed along the centre line of the embankment. Generally, a decision has to be made on whether to install the instruments before or after adopting a particular ground improvement technique. Installing piezometers before adopting the ground improvement techniques ensures that base line data is obtained but there is a potential risk of piezometers getting damaged. It is also proposed that 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.

6.7. Monitoring of Settlement and Heave

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. Various types of settlement gauges are commercially available. These are as follows:

1. Platform type settlement gauge
2. Liquid level gauge
3. Magnetic heave/settlement gauge

6.7.1. Platform type settlement gauges

A platform type settlement gauge is relatively simple device for monitoring settlements by taking levels at the top of the platform at suitable intervals 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 skiu friction, the stem above the platform level is encased in a freely moving casing pipe. A typical sketch of settlement gauge is shown in Fig. 6.5.

6.7.2. Magnetic settlement gauges

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.

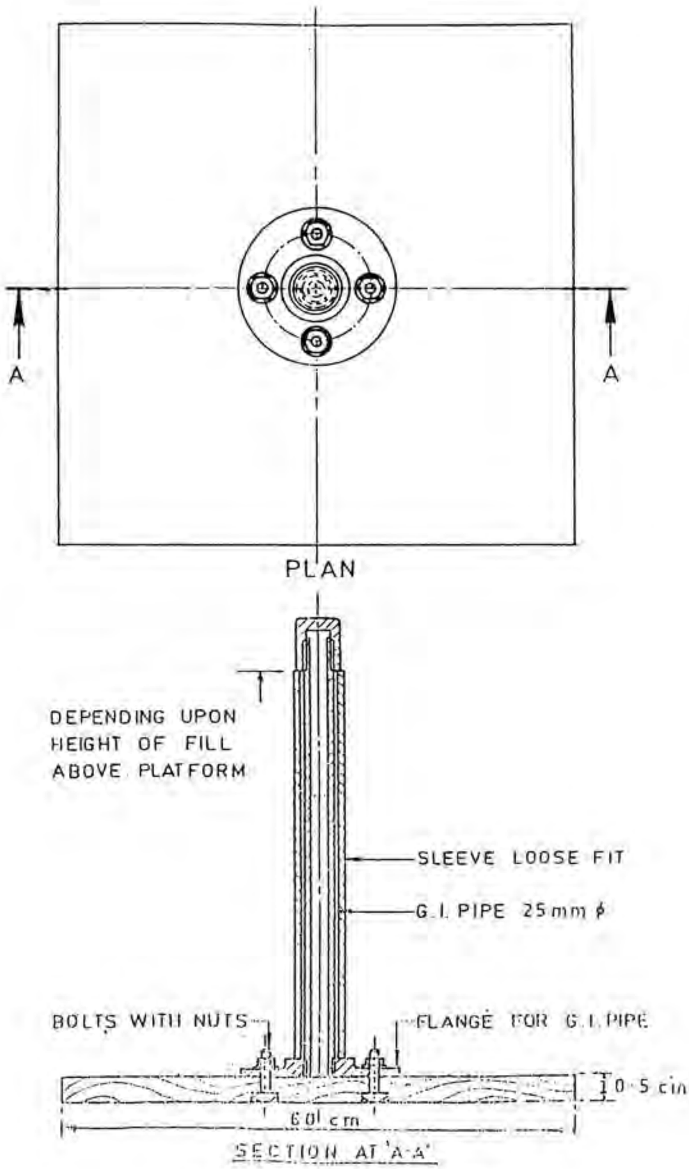

Fig. 6.5. Platform Settlement Gauge

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 Fig. 6.6. 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.

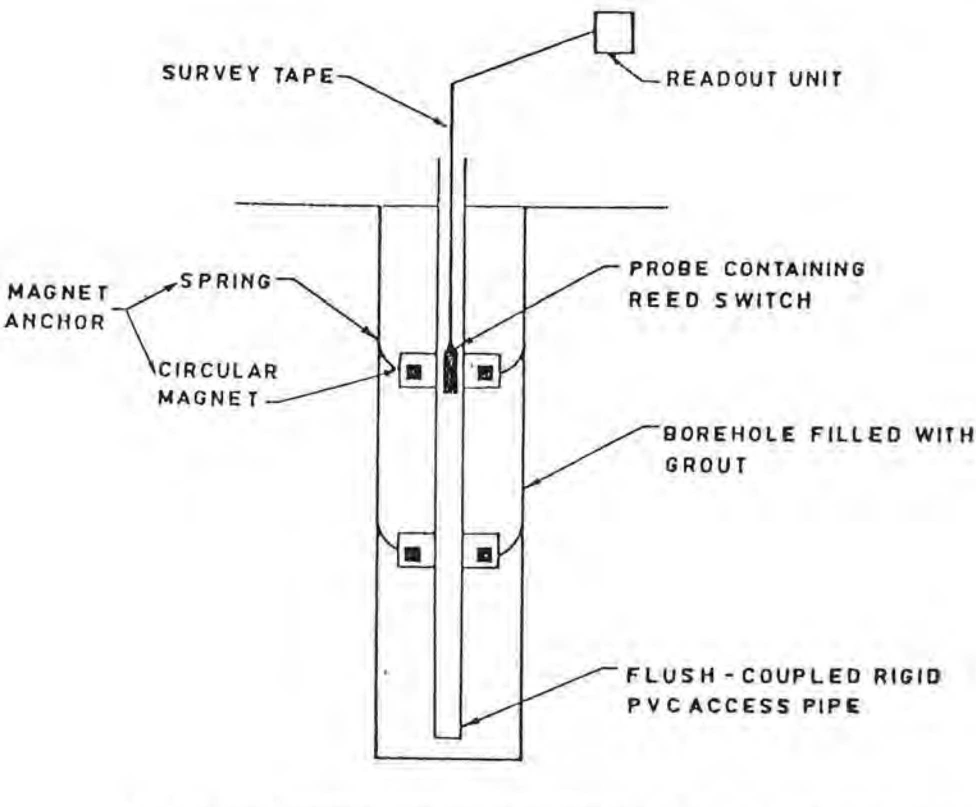

Fig. 6.6. Schematic of Probe Extensometer with Magnet/Reed Switch Transducer in a Borehole

6.7.3. Water overflow settlement gauges/liquid level gauges

To achieve a better order of accuracy, the possible alternative to settlement stakes or platform types of 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, Fig. 6.7. 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 airtube 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.

The details of installation procedures for different types of settlement gauges are discussed in Hanna (1985).

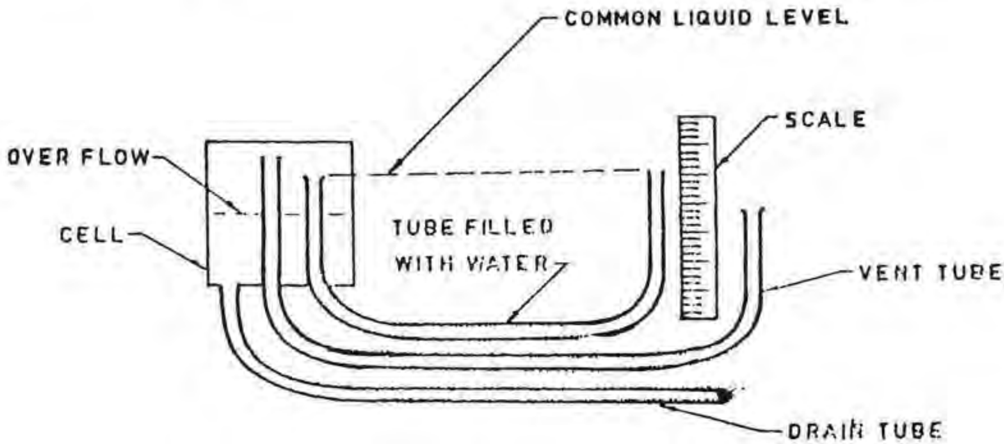

Fig. 6.7. Schematic of Overflow Settlement Gauge

6.7.4. Frequency of installation

The settlement gauges may be installed along the alignment of the embankment at interval of 500m. The gauges may be installed along the centre line of the embankment. The magnetic rings may be installed at 3.0m depth intervals in the soft clay. The platform settlement gauges may be installed at 0.5m below the ground surface.

6.8. Monitoring of Horizontal Movements

Horizontal movements are measured with inclinometers. Inclinometers are installed, preferably near the toe of the embankment as shown in Fig. 6.8. These instruments enable quantitative measurement of lateral movements of soft clay. The instrument not only gives lateral movement, but also reveals direction of movement. Working principle of an inclinometer is shown in Fig. 6.9.

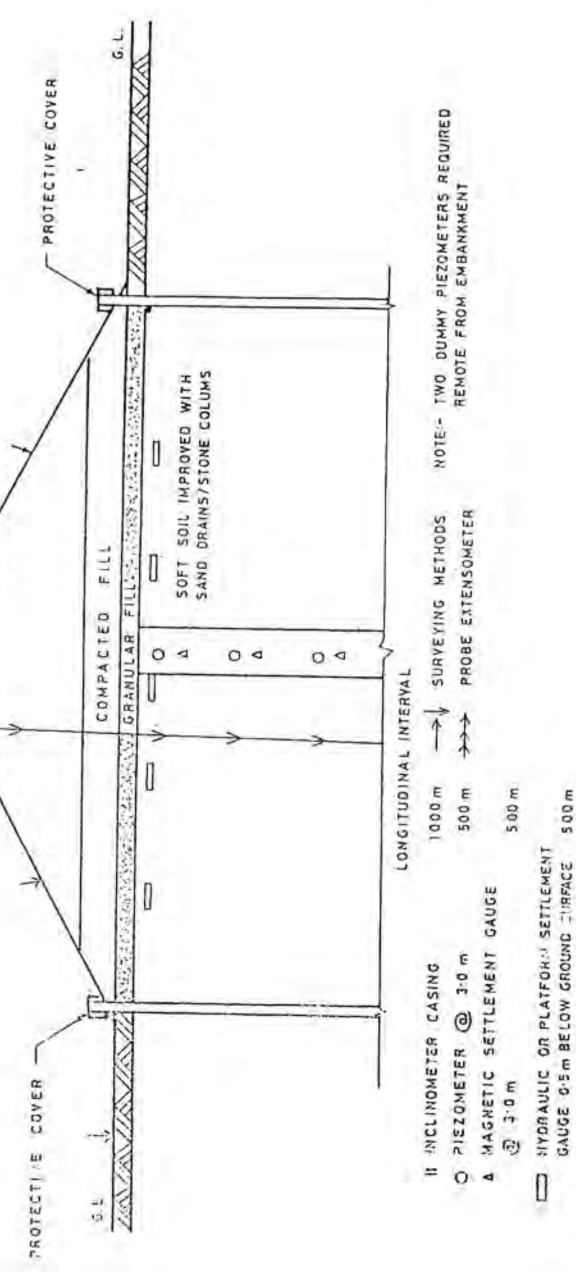

Fig. 6.8. Layout of Instrumentation Between Embankment

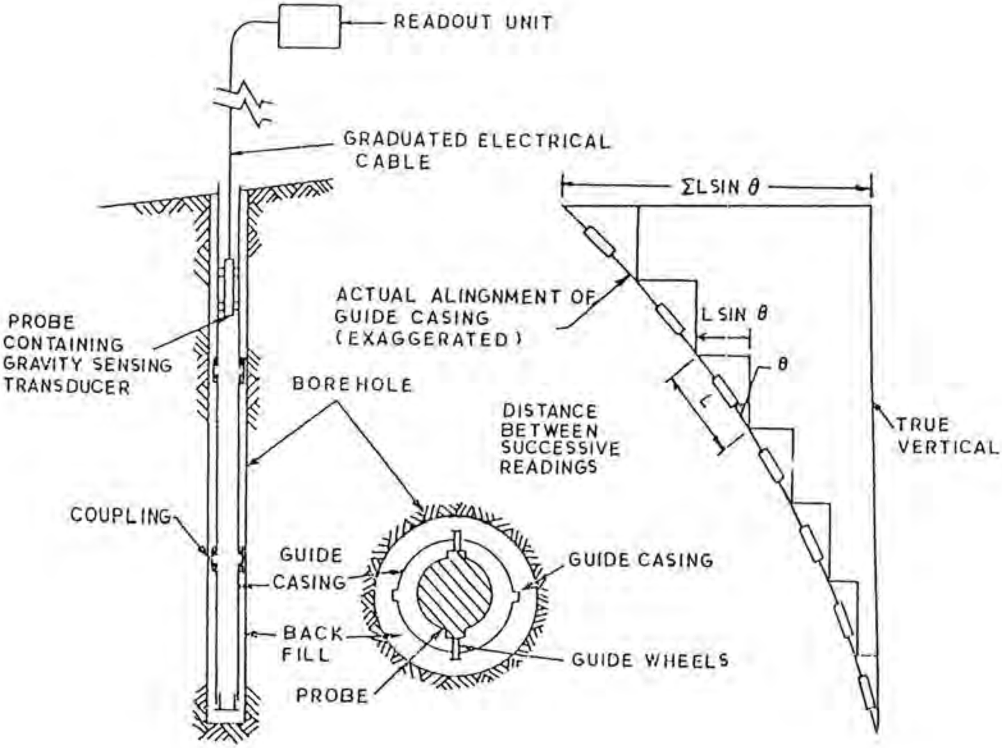

Fig. 6.9. Schematic Sketch of Inclinometer

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:

(i) A permanent guide casing installed in a near vertical alignment

(ii) A portable probe containing a gravity sensing transducer

(iii) Portable readout unit for power supply and indication of probe inclination

(iv) 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:

(i) Inclinometer with force balance accelerometer transducer,

(ii) Slope indicator with suspended pendulum working on the principle of wheat stone bridge,

(iii) Inclinometer with bonded resistance strain gauge, and

(iv) Vibrating wire transducer inclinometer.

The details of the installation procedures of inclinometer are discussed by Hanna (1985).

6.8.1. Frequency of Installation

It is proposed to install guide casing units at 1000m intervals along the length of the embankment. These would be located near both toes of the embankment, in a staggered arrangement. The casing unit shall be installed to the full depth of soft clay. The casing tube may be installed at an off-set distance of 20m from the centre line of the embankment. The reading can be taken by lowering the probe in the guide casing.

The instrumentation details are shown in Fig. 6.8.

6.9. Frequency of Observations

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

i. Readings shall be taken daily in stretches where filling operation is in progress. Weekly readings shall be taken during the same period in other stretches.

ii. Fortnightly readings shall be taken after the desired fill height is achieved.

6.10. 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. 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. In this context it is of interest to note that field data related to embankments on soft ground is relatively scarce in India. Systematic collection of such data would facilitate checking on the design for future use. 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 geotechnical engineer would be essential.

6.11. Precautions against Pilferage

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. It may be necessary to install casing pipe in small lengths. The casing pipe may be terminated close to the level of fill height achieved during the stage construction. The casing pipe may be extended gradually as the fill height increases. The top of the casing pipe should be covered by a pipe cap. Finally, the casing pipe should be terminated close to the road surface. The top of casing pipe may be encased in a protective box firmly embedded in the ground surface and having a suitable locking arrangement.

6.12. 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.

(i) Optical
(ii) Mechanical
(iii) Hydraulic
(iv) Pneumatic, and
(v) Electrical

The instruments installed for embankment is 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.

6.13. Layout of Instruments

Fig. 6.8 shows a general pattern of layout of instrumentation for monitoring behaviour of soft soil under the embankment. 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 slope indicator 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.

6.14. Back Analysis Approach

In the last four decades field instrumentation and full scale testing techniques have been increasingly used to gather evidence on the in-situ behaviour of soils. In an engineering science development in material is so complex as soil can only be achieved by constant reference to field observations, Terzaghi (1936). The interpretation of the complex field data to support to existing theories or to develop new, has been made easy with the use of computer and numerical techniques like finite element methods. The numerical back analysis is now a preferred method of validating theories or more frequently for evaluating soil parameters to be used in accepted methods

6.14.1. Approach of back analysis

The principle of back analysis may be expressed in either the modified forms of Lambe's (1973) equation.

Observation + Soil Parameter	=	Validated theory
Observation + Theory	=	Empirical soil parameter

These equations are handled through back analysis which must be executed meticulously for yielding reliable results. The back analysis of any case history involves a series of assumptions, each of which in itself is a potential source of error.

6.14.2. Steps in back analysis

(i) Simplify the geometry of the problem and soil stratigraphy

(ii) Assume an idealised soil response, i.e., drained or undrained

(iii) Select an analytical model, i.e., Linear or non-linear elasticity, isotropy or anisotropy, Mohr-Coulomb criterion or plastic flow rules. This step requires considerable idealisation.

(iv) Conduct laboratory tests for determination of input soil parameters

(v) Using the above data and parameters, evaluate the stability and calculate the settlements. The values can be compared with the observed field data.

In majority of the cases the first trial results in wide variations of computed and field observations. Thereafter, some of the earlier assumptions in soil response, boundary conditions analytical models or input parameters are modified until a reasonably good fit is achieved between computations and observations. The general practice is to adjust a few key input parameters and try to obtain a good fit for one or two parameters of the field behaviour. Once a satisfactory fit is obtained for a particular problem as part of field behaviour for which the back analysis was done, than the theory of model used or the methodology for determining the input parameters are accepted. This methodology has several shortcomings. The most serious shortcoming is that the basic soil mechanics principles are generally not satisfied by the results.

When dealing with the results of back analysis, we suffer from the same weaknesses as identified by Terzaghi (1936), i.e., our conclusions are founded on unbalanced evidence. As soon as a good fit is obtained for a parameter under investigation, no serious attention is paid to the quality of best fit for other parameters.

Nevertheless, in the last two decades considerable progress has been achieved on various aspects of geotechnical problems because of back analysis.

6.14.3. Guidelines for the proper use of back analysis

Leroueil et. al. (1981) have specified guidelines for the proper use of back analysis for obtaining meaningful results and avoiding costly errors.

(i) All aspects of soil behaviour should be integrated into one effective stress dependent response. It is wrong to split this response into separate limited phenomenon, such as, pore water response, strength mobilisation and strain development. All parameters should be considered in totality and in a consistent manner.

(ii) Soil behaviour is more complex than the available theories. Thus, experience should always be given precedence. Theories and concepts which provide a framework for the qualitative interpretation of observations should be preferred to complex analytical solutions.

(iii) The first priority should be given to the unbiased field observations of all conceivable parameters pertinent to a given problem before attaching too much importance to numerical analyses.

(iv) Once such data is available, a qualitative analysis of the data from a variety of case histories should be made to develop an understanding of the nature of problem. Otherwise there is a great danger of analysing the wrong problem.

(v) Implicit and explicit assumptions in theories or analytical methods must be clearly identified and checked against the actual soil behaviour as obtained from field and laboratory investigations.

(vi) All input parameter which are naturally variable must be identified. In most cases assumption on key input parameters will lead to non-reliable conclusions of the back analysis.

(vii) Soil mechanics principles must apply to all the results of back analysis. All interrelated parameters must be back analysed with equal success or with a consistent error to allow an empirical correction. A back analysis, successful on only 50 per cent of the parameters, is nothing but 100 per cent wrong.

(viii) Time is systematic parameter in all geotechnical problems. The validity of back analysis should be, therefore, checked at various stages of the problems under investigation before drawing any conclusion.

6.15. Case Histories

Case History-1

The ore handling yard at Vishakhapatnam Port is underlain by a 12m thick layer deposit of soft marine clays (CRRI 1975). The soft clay is overlain by a 6m thick sand layer. Iron ore is brought from mines and stacked over four strips of 1000m x 35m in this area for export to other countries. Instability of the foundation soil was experienced as indicated by heaving up of the adjoining ground wherever the ore stack height was of the order of 3 to 4m. For its export commitments and as the mechanised system is capable of handling approximately 2000 tonnes per hour of iron ore, stacking heights had to be increased to 9m in each stock pile area. This required the soft clay to be artificially strengthened so as to sustain high ore stack loads. The problem of strengthening the soft clay was assigned to CRRI. A report was submitted to Visakhapatnam Port Trust (VPT) which included recommendations for strengthening the soft clay with sand drains and preloading technique, as well as, monitoring the process of consolidation with extensive instrumentation. These instruments were to identify unanticipated and the most unfavourable field conditions ahead of time so that a course of action or modification of the design could be taken at the most appropriate time table. The different instruments used to monitor the various parameters are as follows:

Parameter	Instruments	Remarks
Deformation of soft clay	Settlement stackes	For monitoring settlements
	Hydraulic settlement gauge	-do-
	Inclinometer, suspended pendulum type based upon wheatstone bridge principle	For monitoring lateral displacements of soft clay

Pore water pressure	Casagrande piezometer	Extensively used as the same is indigeneously available.
	Bishop's twin tube piezometer Electrical piezometer	Only a few of them were imported and installed.

Fig. 6.10 gives plot of settlements recorded under different preloading heights. Fig. 6.11 shows the increase in pore water pressure with time due to preloading. Fig. 6.12 shows lateral deformation recorded by the inclinometer.

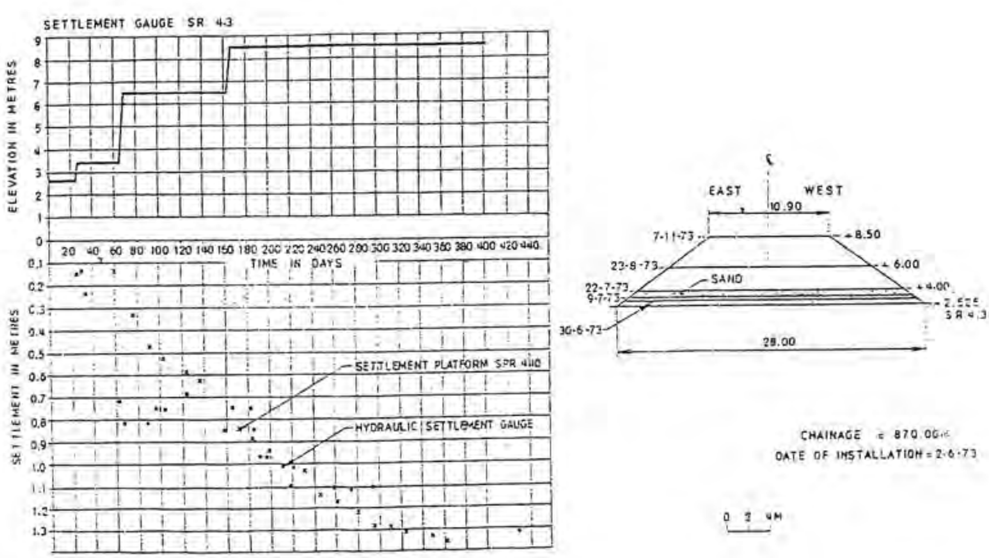

Fig. 6.10. Load Settlement - Time Curves

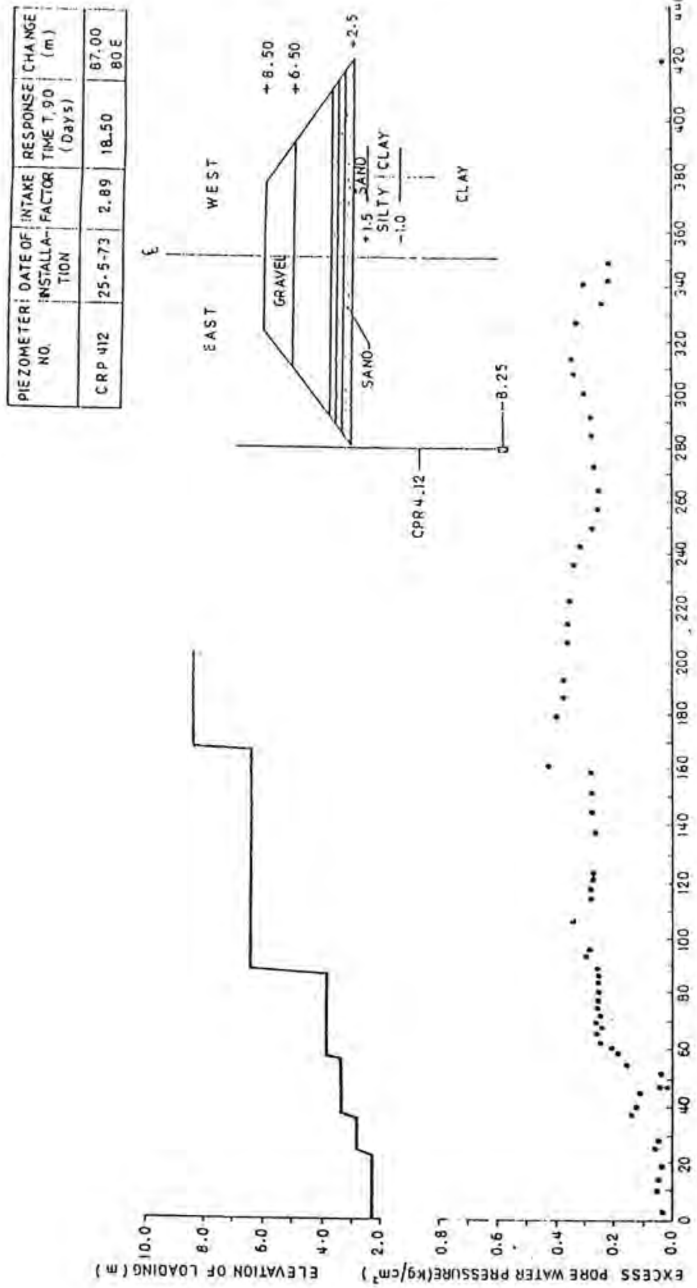

Fig. 6.11. Load - Excess Pore Pressure - Time Curve

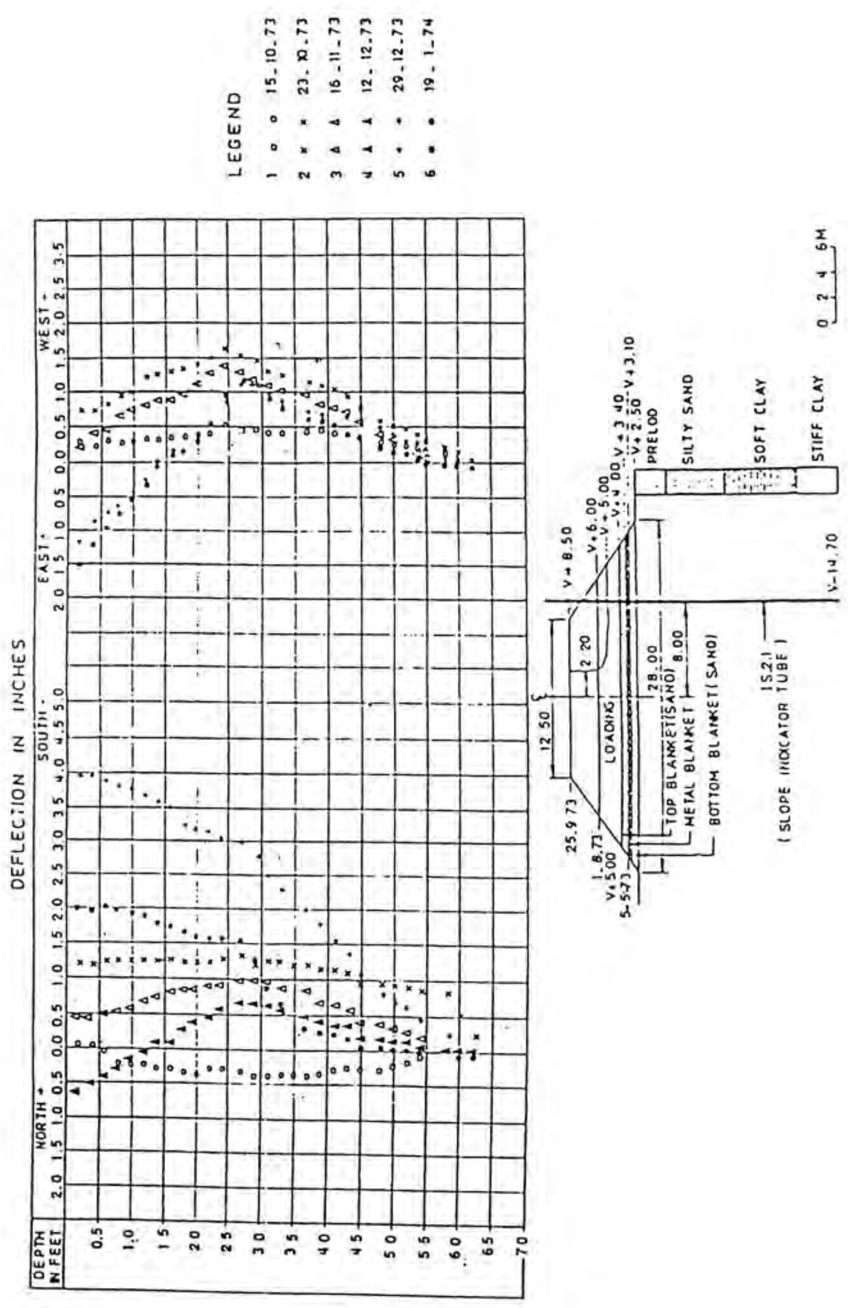

Fig. 6.12. Lateral Ground Movement - Time Curves

Case History-2

An embankment of a height of 10m was constructed over a soft alluvial deposits extending upto a depth of 11m (Simons et. al. 1975). The soil profile is shown in Fig. 6.13. The settlement analysis showed that the soft alluvium shall undergo a consolidated settlement of more than 1m under the embankment loads and it will take nearly 15 years to complete. To accelerate the rate of consolidation the ground was treated with sand drains and the embankment was raised in stage construction.

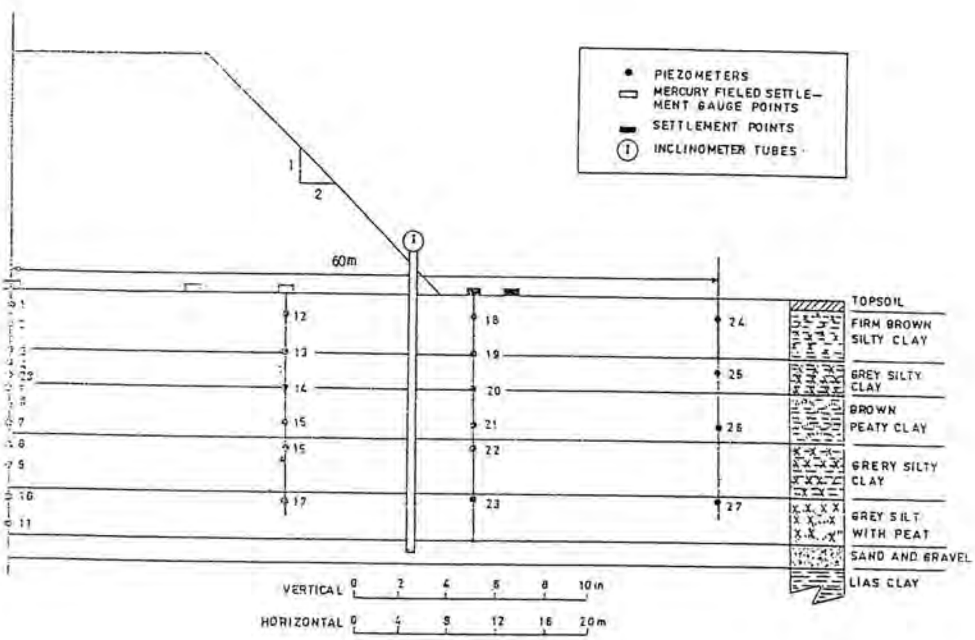

Fig. 6.13. Cross Section of Embankment Showing Location of Instruments

The stability calculation indicated that the shear strength of the soil could support a safe height of 6m without stability problem. For construction beyond this height the control was exercised with data obtained from instruments. The embankment was raised upto 8.4m and the construction was stopped for a period of 13 months to allow the consolidation process to take place. Mainly, the pore pressures were monitored. The details of instrument installed for control measures are shown in Fig. 6.13 and are as follows:

(i) **Settlement Gauges**: These consisted of mercury field settlement gauges under the embankment and settlement points beyond the embankment for monitoring of heave/ settlement.

(ii) **Piezometers**: These were made of ceramic filter candles attached to twin lengths of polythene coated nylon tubes. These tubes were connected through an arrangement to a de-airing unit. These were used for measuring pore pressures.

(iii) **Inclinometers**: For measuring lateral deformations plastic casing tubes were installed at a distance of 2m from the toe within the boundaries of the embankment. Inclinometer readings were taken at regular interval of time with the readout instrument.

The control of the earthwork above the height of 6m was exercised through measurement of pore pressures. The settlement gauges measured the settlement of the foundation soil. The inclinometer was used to control the lateral movement of soft soil under the embankment loads.

The development of pore pressure due to embankment loads was estimated with Skempton's and Burland method. The construction of embankment from 5.3m to 8.4m was completed in a time of eleven days. During this period dissipation of excess pore pressure was not expected because of relatively short period of time, however, it is reported that significant dissipation did occur in the upper 2m of sub-soil. The prediction of pore water pressure by Burland method gave closest approximation with recorded values in the upper layers. All the methods were found to over estimate the pore pressure recorded with piezometers in the upper layers. At greater depths the predicted value under estimated the observed value and the Burland method showed maximum deviation. Fig. 6.14 gives the plot of difference between predicted increase in pore pressure with three given methods and the pore pressure actually measured with piezometers. The closest approximation of pore pressure increase is made with the Skempton's method. The settlement analysis was done using the conventional two dimensional consolidation employing a computer programme. The settlements were also computed from the method based upon in-situ measurement of permeability. The results of theoretical prediction of settlement with time and observed settlements with settlement gauges are given in Fig. 6.15. It is seen that settlement computed with the in-situ measurement of permeability gave a very close approximation with the observed settlements.

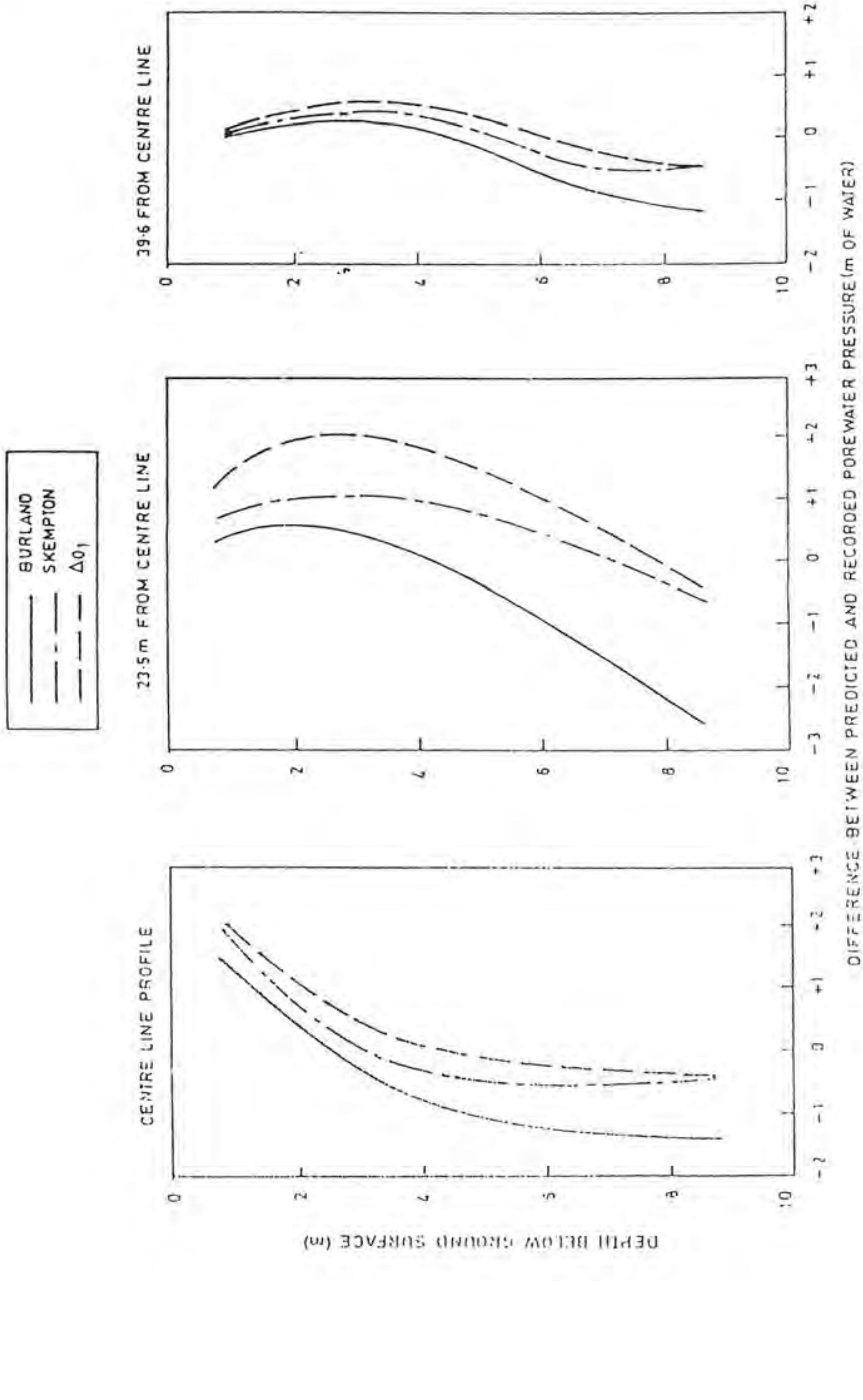

Fig. 6.14. Comparison of Measured and Predicted Increases of Pore Water Pressure for Increment in Embankment Height from 5.3-8.4 Metres

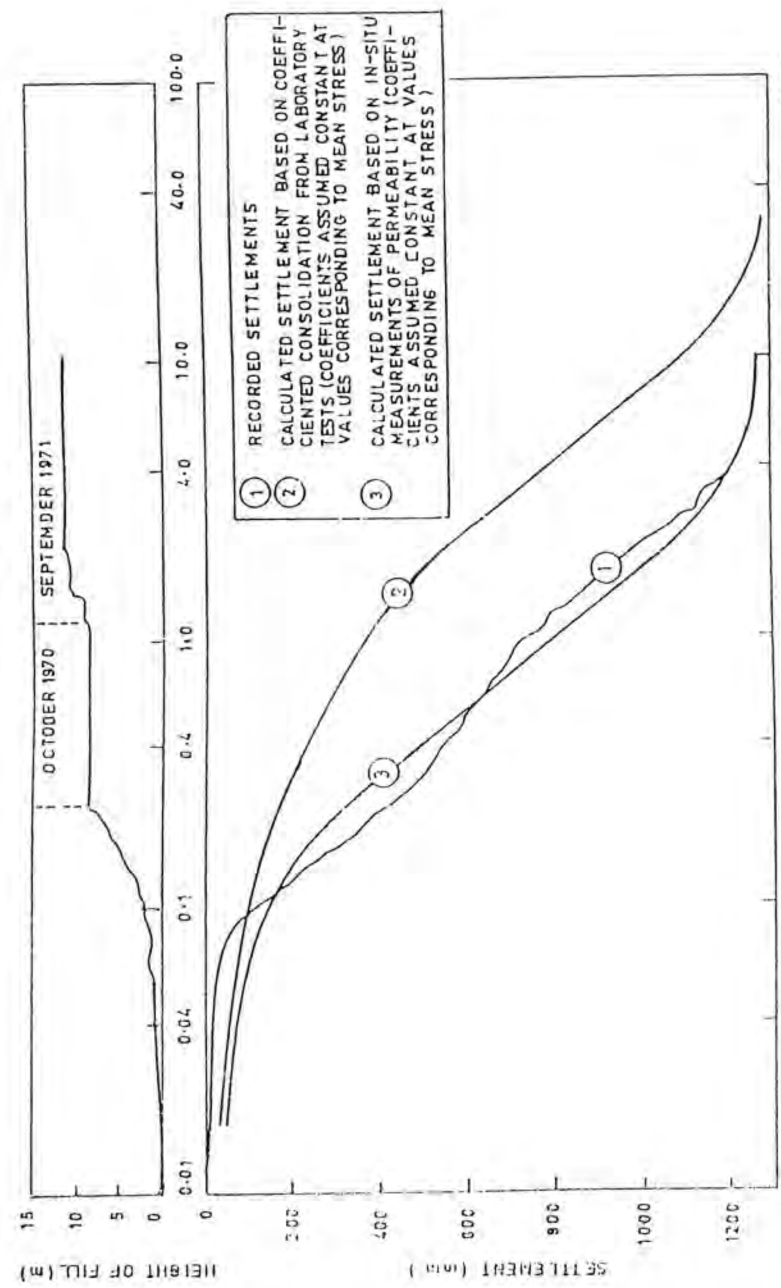

Fig. 6.15. Recorded Settlement Compared with the Calculated Relations Between Settlement and Time Using Coefficients of Consolidation from both Laboratory and In-situ Tests

Case History-3

Lambe (1962) discusses in detail the predictions of excess pore water pressure in the soft sub-soil under preloads. The prediction of the excess pore pressures developed due to the preload are compared with the field observations.

Lagunilas Preloads' Creole Petroleum Corporation was faced with the necessity of constructing three oil tanks, very heavily loaded and the same were expected to run at full capacity. The stability analysis showed a very low factor of safety of 0.6. Another tank built on almost similar sub-soil conditions failed during test loading due to shear failure. The applied load, in this case was, of the designed load. The tank was later founded on piles. The study of the site for engineering values showed that a substantial saving of the more than $ 375000 (in 1960) could be made if the sub-soil at three sites were improved by preloading technique.

The sub-soil conditions required that the preloading was carefully monitored to avoid a shear failure. The sub-soil was, therefore, extensively instrumented for the following reasons:

(i) Control of the preloading,

(ii) Obtain data for the design of tank foundations, and

(iii) Provide information for future construction in the area. The maximum pore pressure increase were computed and the same were compared with the observed values.

Fig. 6.16 shows the cross-section of the preload and the sub-soil conditions. The undrained shear strength of the clay was 290 kPa to 360 kPa. Fig. 6.17 shows plots of total stress, pore pressures and effective stresses under the centre of preload. It is seen that the ratio of induced vertical stress to initial existing effective stress at the centre of the clay was approximately three, which is a very large value. Therefore, the embankment heights were raised taking precautions on the rise in piezometric levels of pore water.

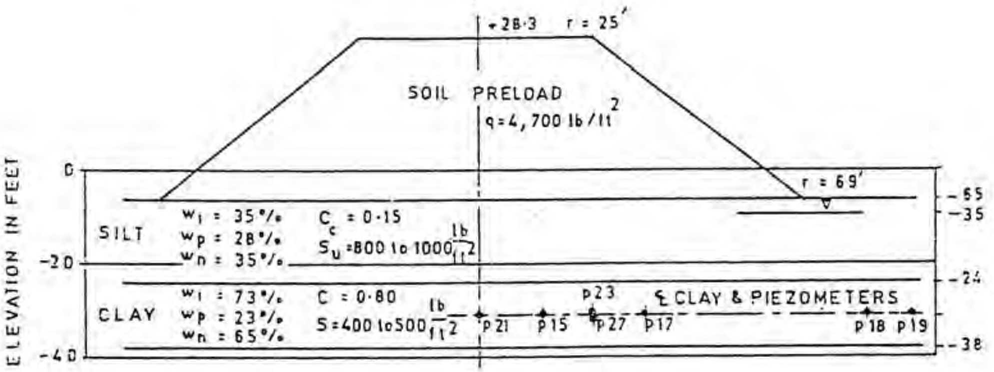

Fig. 6.16. Lagunillas Preload

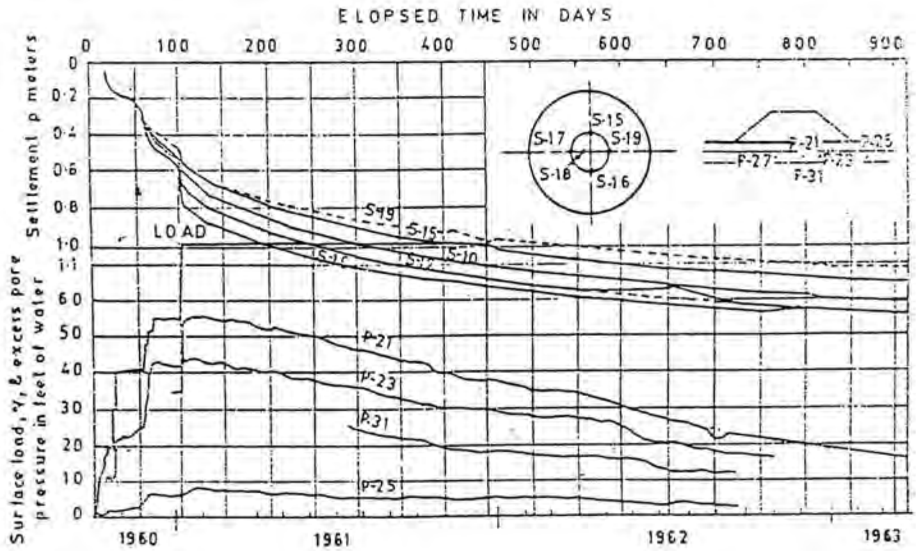

Fig. 6.17. Lagunillas Preload S-II

The prediction of the rise in pore water pressure was made with different methods as shown in Fig. 6.18. The piezometers installed to measure the increase in pore pressures were improved version of Casagrande type piezometer having tube connections which were sealed into Bourden pressure gauges at each end. Fig. 6.17 shows locations of seven piezometres, the readings of which are analysed. The piezometer installed in silt did not show appreciable rise in pore pressure as the silt had very high permeability.

Fig. 6.18 shows the comparison between predicted and measured pore pressures at the centre of the clay layer for an undrained loading. The comparison shows a very close agreement between predicted and measured pore pressures. The closeness of this agreement is remarkable in view of the fact that the computed shear stress at most of the piezometers far exceed, the shear strength of clay.

Case History-4

The construction of a trial embankment at Athlone. Ireland leading to controlled failure and subsequent back analysis has been reported by Dauncey et. al. (1987). As a part of embankment design for construction of bridge embankment accross River Shannan et Athlone, Ireland instrumental trial embankments were constructed to confirm the design strength and consolidation parameters. The soil profile at trial embankment section and the undrained shear strength profile is shown in Fig. 6.19. Field vane test gave sensitivities in the range of 2 to 30. Consolidation tests indicated an preconsolidation pressure generally about 14 KN/m^2, above the estimated in-situ vertical effective stress. The stability of the embankment at each stage of construction was based on the undrained shear strength of the soft soil using Su/Po relationship. Shear strengths were based on vane shear tests and design values of Su/Po were carried from consolidated undrained triaxial tests as shown in Table 6.2. 1.0m thick sandy gravel platform was placed on the ground surface. This acted both as a horizontal drainage layer and as well as, a working platform. Prefabricated band drains were installed in an isosceles grid at a spacing of 1.1m and thereafter instrumentation was installed. The embankment was constructed in stages till failure. The location of failure surface as determined from slope indications and inclinometer readings, are shown in Fig. 6.20. The average increase in the effective vertical stress at the time of failure was about 20 kN/m^2. Back analyses was carried out to establish the mobilized su/po profile. Janbu's method was used for stability analysis. The parameters used in analysis are shown in Table 6.2. The pore pressures recorded by piezometers were used to calculate the vertical effective stress below the embankment. The su/po mobilized derived from back analysis is shown in Fig. 6.21. It is evident from Fig. 6.19 that the observed performance of the embankment is consistent with the increase in shear strength and the su/po values used for design are shown in Table 6.3. The performance of the trial embankment was satisfactorily predicted by effective stress analysis based on pore pressure measurements.

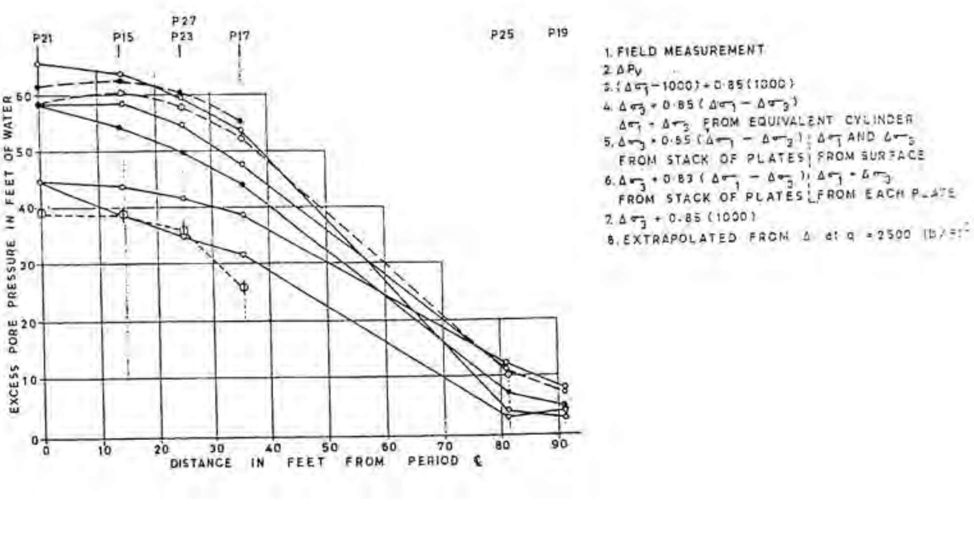

Fig. 6.18. Comparison of Predicted and Measured Pore Pressures

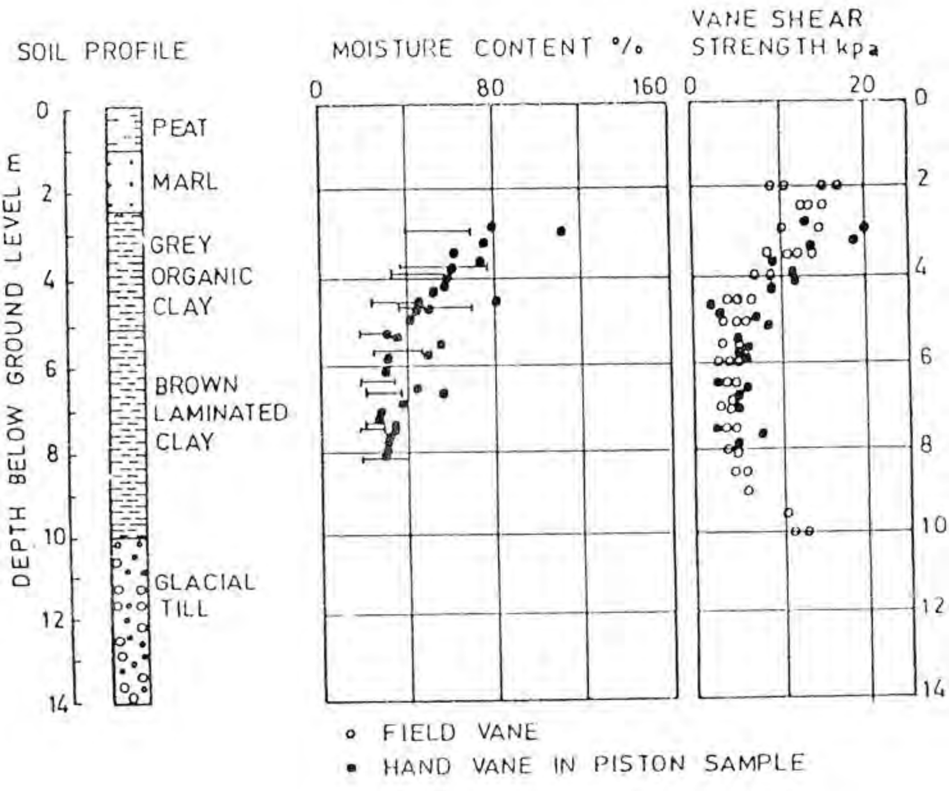

Fig. 6.19. Soil Profile at Trial Embankment Location (Dauncey et.al., 1987)

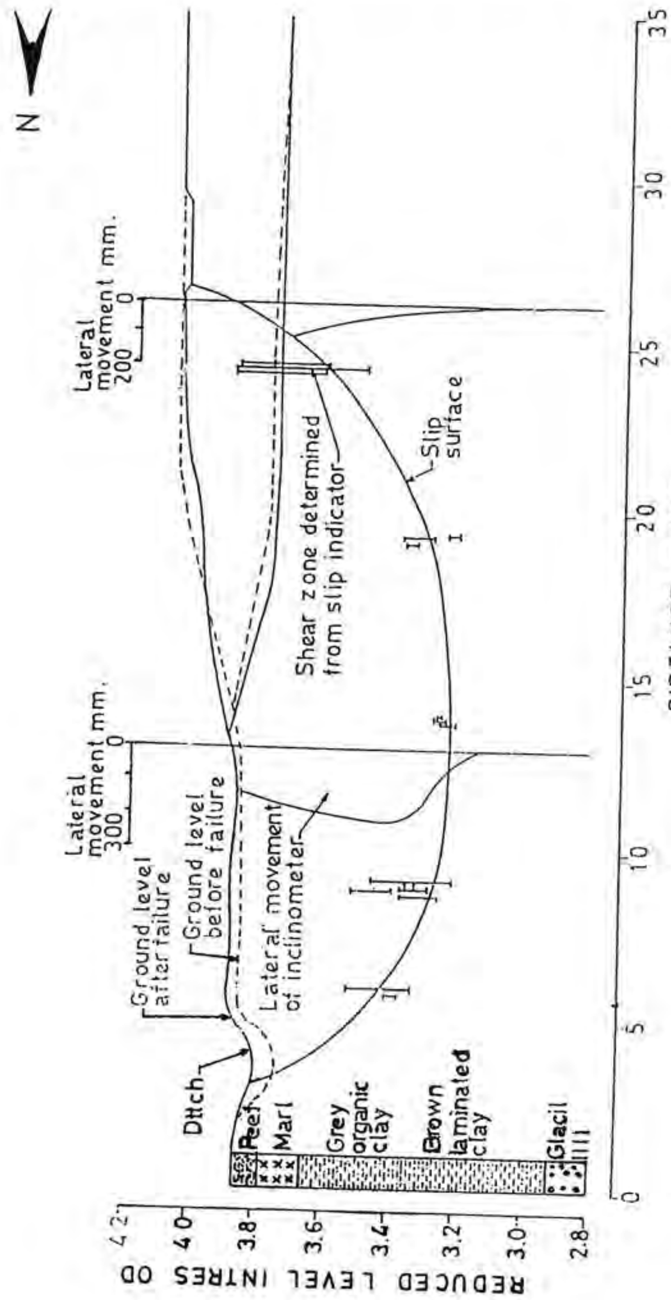

Fig. 6.20. Location of Failure Surface (Dauncey et. al., (1987)

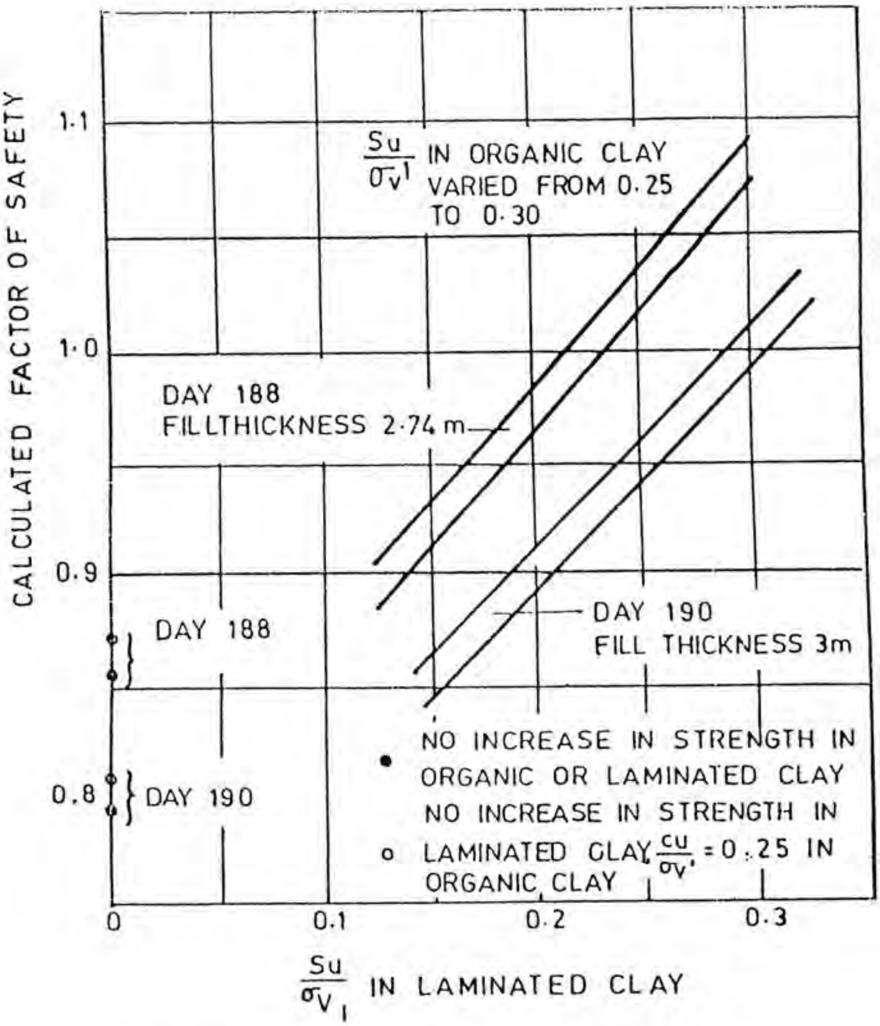

Fig. 6.21. Results of Back Analysis, (Dauncey et. al., 1987)

Table 6.2. Parameters Used in Embankment Design (Dauncey, P.C. et. al., 1987)

	Y kN/m^3	Initial su /kPa	su/po
Peat	10.0	15.0	0.50
Marl	12.5	12.5	0.25
Organic clay	15.7	10.0	0.25
Laminated clay	19.0	5.0	0.20

Table 6.3. Parameters Used for Back Analysis (Dauncey, et. al., 1987)

	Y kN/m^3	Initial su/kPa	su/po	ϕ
Uncompacted fill	20.2	—	—	32
FDM	19.2	—	—	30
Peat	10.0	15.0	0.5	35
Marl	12.5	12.5	0.25	34
Organic clay	15.7-16.35	13.5-5	Varied	32
Laminated clay	19.0-19.35	5.0	Varied	30

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