ANALYSIS AND DESIGN OF PLAIN AND JACKETED STONE COLUMNS IN CLAYS

A Thesis

By

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TO MY PARENTS

ABSTRACT

A novel approach on the analysis and design methods for plain and newly envisaged jacketed stone columns in clay media is presented in this thesis. It considers nonlinear mechanical behaviour of the constituent materials as well as compatibility of deformation amongst these. To characterise the nonlinear mechanical behaviour of the constituent materials, the hyperbolic model forwarded by Kondner (1963) was used. The cavity expansion theory presented by Ladanyi (1963) is used here to formulate the desired equations for prediction of lateral resistance mobilized in the clay media. A new suction triaxial test was developed to characterise the mechanical behaviour of the aggregates which was proved to be very beneficial especially in case of large size specimens.

As a part of these studies a literature review is presented to depict the state of the art as well as to identify the limitations of the existing design methods. To depict the applicability of the proposed analysis and design methods, illustration examples are presented. In the examples, the constituent materials include stone aggregates, a geogrid jacket liner and a soft clay from the south-east region of Bangladesh. A comparative study revealed that the existing methods may lead to unsafe designs. Moreover, the jacketed column was seen to provide a considerably stiffer foundation than its plain counterpart.

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NOTATION

Ac	plan area of soft clay per column
As	surface area of stone column
Ast	cross sectional area of stone column
a_a , b_a	design parameter of aggregates
a_r , b_r	design parameter of reinforcing element
a_s , b_s	design parameter of clay media
c , c_u	undrained cohesion of clay
d	diameter of stone column
D_f	depth of the top of column from ground surface
d_f	size of footing
d_p	size of stone column
Es , E	modulus of elasticity of soil
Ep	deformation modulus of pile
Fc , Fq	cavity expansion factor
G	shear modulus of soil
H	thickness of clay layer
I_ρ	displacement influence factor
In	relative rigidity index of soil
Ir	rigidity index
K	ratio of minor and major principal stresses
Ko	cofficient of earth pressure at rest
Kpc	Rankine coefficient of passive resistance
Kps	coefficient of passive resistance of granular material
Lo	original length of stone column
Nc, Nq, N_γ	bearing capacity factors

Pv	vertical load Applied on the top of stone column
P_f	fritional resistance along stone column-clay interface
P_h	lateral resistance from both clay and reinforcing element
P_r	lateral resistance offered by reinforcing element
P_s	lateral resistance offered by clay media
P'_m	precompression pressure
q	effective mean normal stress
q_u	ultimate stress carried by stone column
R	frictional force
Ru	ultimate frictional force
r	current radius of expanded cavity
r_o	original radius of stone column
S	settlement of stone column
s	settlement of stone column for each strip
Su	undrained shear srength of clay
$S\bar{u}$	average undrained shear strength of clay
u	pore pressure
v	displacement at any point of clay media
Z	depth of stone column from ground surface
α	adhesion factor
γ	unit weight of soil
γ_i	initial shear strain of soil
γ_s	undrained shear strain of soil
δ	radial deformation of column
δ/r_o	dimensionless deformation ratio
ε	axial strain

Δ	volumetric strain
ΔL	change of length of aggregates sample
ξv	volumetric change factor for a cylindrical cavity
ν	poisson's ratio
ρ	distance of a point measured from the centre of cavity
ρ	distance of a cylindrical surface from the centre of cavity
σ_{ρ}	total principal stress in radial direction
σ_t	total principal stress in circumferential direction
σ_R	principal stress in radial direction
σ_{Rl}	passive resistance of soil
σ'_R	effective radial stress
σ_{Ro}	total insitu radial stress
σ'_{Ro}	effective insitu radial stress
σ_{θ}	principal stress in circumferential direction
σ_c	vertical stress in soft clay
σ_s	vertical stress in compacted stone column
σ_3	confining pressure
τ_s	skin friction along stone column - clay interface
ϕ	angle of shearing resistance of soil
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CHAPTER ONE

INTRODUCTION

1.1 GENERAL

Stone column foundations, a fairly recent ground improvement technique, have been used successfully in several projects throughout the world. The scope and the range of their applications with the development of theoretical aspects is growing rapidly. It has now reached at a stage where design methods have changed from empirical to rational approach.

Almost all of the existing design methods of stone columns are based on linear consideration of material behaviour and limit state principles. This may lead to considerable error as mobilisation of resistances in the constituent materials depends both on realistic nonlinear behaviour and degree of deformation.

In this study a new approach to analysis and design of plain and newly envisaged jacketed stone column is presented. A jacketed stone column may be produced by incorporating a geotextitle or geogrid jacket linear around a plain column. This will produce a stiffer column than its plain counterpart. The proposed analysis and design methods are based on the considerations of nonlinear behaviour of the materials under compatible deformation. Newly test methods and constitutive relations are also established to characterise the constitutive materials. Finally, design examples are produced for plain and jacketed stone columns.

1.2 HISTORICAL BACKGROUND

One of the oldest historical examples of the use of stone column foundations is found in the 1830's where French military engineers used it to support heavy foundations of iron works at the artillery arsenal in Bayoune (Hughes et al, 1975). However, the modern origins of the stone column foundations truely began 59 years ago in the 1930's in Germany by their Russian Emigre, Sergei Stenermann and Wilhem Degen as a by product of the technique of vibroflotation of cohesionless soils both above and below the water table (Glover, 1982). After that, an empirical design method was suggested by Thornburn and Macvicar (1960). Later Greenwood (1970) proposed a method using Rankine passive pressure coefficients. Vesic (1972) proposed the cavity expansion theory which offered theoretical basis for design of stone columns. Hughes and Withers (1974) and Hughes et al (1975) have made a systematic study of the failure mechanism of granular piles. They suggested bulging as the most likly mode of failure and proposed a method of estimation of the load carrying capacity of these piles. Poulos et al (1976) and Balaam et al (1977) presented settlement analysis for granular piles, using finite element method. Madhav and Vitkar (1978) cosidered a general shear failure type of mechanism for analysis of soft soil reinforced by a gravel trench or pile. Rao and Bhandari (1977) suggested an improvement of skirting the gravel pile in order to improve its load carrying capacity. Englehardt and Golding (1975) and Moh et al (1981) suggested use of stone column foundation to minimize liquifaction by Datye and Nagaraju (1981) suggested methods on installation techniques of stone columns. Madhav (1982), Datye (1982), Nayak (1982) and Mitchell and Huber (1982) have also worked on the various aspects of improvement of stone column foundations.

1.3 EXISTING METHODS

The analysis and design of stone column foundations are based on a number of advanced theories developed by several researchers. The existing methods of evaluating load carrying capacity of stone column are based on limit state analyses, Passive failure theory (Greenwood, 1970), Expansion of a cylinder (Gibson and Anderson, 1961), Cavity expansion theory (Vesic, 1972), Pile formula and General shear failure (Madhav and Vitkar, 1978). Settlement prediction methods range from empiricism based on experience to nonlinear finite element analyses. The available methods are Elastic continuum approach, Reduced stress methods, Approach of Hughes et al (1975) and Finite element approach. The existing methods for evaluation load carrying capacity and settlement advocate the use of material parameters obtained from routine geotechnical engineering tests. In almost all the cases nonlinear behaviour and compatibility of deformation amongst these are not considered. Details of the existing theories are described in Chapter 2 .

1.4 LIMITATIONS OF EXISTING METHODS

Almost all of the existing theories, on which design of stone columns are based, use limit state analyses and linear material behaviour. In these cases single parameter representation using ultimate values of material behaviour have been taken into cosideration. As compatibility of deformation of the constituent materials are not taken into account, this may lead to considerable error in predicting the mechanical behaviour. This is because the mobilization of the resistances of the constituting materials are dependent on the degree of deformation. Also constituting materials like aggregates, soils and reinforcing element show nonlinear behaviour under wide ranges of strain conditions, which are not considered in the existing design methods. These are the inherent shortcomings of the existing theories which are being used for predicting the load carrying capacity and settlement of stone columns.

1.5 SCOPE OF WORK

In Bangladesh there is a tendency, in the recent year, towards using granular (sand) piles instead of conventional pile foundations in soft ground conditions. Amongst the various ground improvement techniques, stone columns (granular piles) could generally be an ideal choice from the point of view of technical feasibility as well as in terms of low energy utilization. Advantages of stone columns over other conventional methods of ground improvement may be describes as the following:

i) Moderate increase in load carrying capacity.

ii) Significant reduction in settlement.

iii) Being granular and freely draining, consolidation settlement is accelerated and post construction settlement will be minimized.

iv) Installation is relatively simple and involves low energy input or manual labour.

v) Increase in resistance to liquefaction.

vi) Cost effective.

There is a wide scope of using stone column foundations as a ground improvement technique. Therefore, the development of design methods of stone column considering the realistic nonlinear stress-strain behaviour of constituting materials and deformation compatibility between these are required.

Due to general flexibility of plain stone columns the load carrying capacity is governed by the settlement considerations. Introduction of "Jacketed" stone columns, incorporating a geotextile or geigrid jacket liner around plain columns may be envisaged which will provide not only with a much stiffer column but also act as a separation between the aggregates and clay, preventing intermixing.

1.6 OBJECTIVES OF THIS RESEARCH

In light of the dicussion presented in this chapter the objectives of this research may be outlined as follows:

i) Literature review to depict the state of the art as well as to show the limitations of the existing design methods.

ii) Development of a new design method for plain as well as newly envisaged jacketed stone columns based on realistic nonlinear analyses, replacing the existing methods which are based on simplified assumptions.

iii) Characterization of relevant design parameters of the constituent materials influencing the functional behaviour.

iv) Development of testing techniques and analyses methods relevant to present studies.

v) A design example is produced to show the applicability of the proposed method as well as to compare this with the existing methods.

CHAPTER TWO

LITERATURE REVIEW

2.1 GENERAL

Stone columns, consisting of granular materials compacted in cylindrical holes, have been used as a technique for improving the shear strength and consolidation characteristics of soft clays since 19th century. A brief account of historical development has been presented previously in section 1.2. The modern developments may be traced from the early 1960's when vibroflotation technique was used for forming stone columns in cohesive soils. This was accompanied by simultaneous development of design methods. An empirical design was suggested by Thornburn and Macvicar (1960). This was followed by Greenwood (1970), who proposed a method using Rankine passive earth pressure coefficients. Vesic (1972) proposed the cavity expansion theory which is the main theoretical basis of estimating yield stress or the maximum vertical stress in the stone column beyond which excessive deformations would occur. Hughes and Withers (1974) and Hughes et al (1975) have made a systematic study of the failure mechanism, using radiographic techniques, of granular piles. They suggested bulging as the most likely mode of failure, and proposed a method of estimating load carrying capacity of the pile. Poulos et al (1976) and Balaam et al (1977) presented settlement analysis of granular piles using finite element method. Madhav and Vitkar (1978) considered a general shear failure type of mechanism for the analysis of soft soil reinforced by a gravel trench or pile. Rao and Bhandari (1977) suggested an improvement of skirting the gravel pile in order to improve its load carrying capacity. Balaam et al (1976) presented an analysis of rigid rafts supported by granular piles. Englehardt and Golding (1975) and Moh et al (1981) adopted stone columns to minimize the possibility of liquefaction. Datye and Nagaraju (1981) described their experience with different types of installation of granular piles. Glover (1982) outlined the details of the process of stone column installations and the ground suitable for improvement by this method. He also discussed briefly, case histories of two projects. Madhav (1982) presented the improvement of stone column providing skirt around the pile, reinforcing the granular medium, introducing a rigid plug near the top, providing a footing relatively longer than the diameter of the pile. Datye (1982) presented the methodology for testing and analysis of stone column which will minimize the empirical elements in design of stone column and also discussed briefly case histories of five projects. Nayak (1982) introduced the new technique of estimating peripherial concentration and load testing of stone column. Mitchell and Hubber (1982) produced a case study in which stone column foundations were used in a wastewater treatment plant in the U.S.A.

2.2 DESIGN PHILOSOPHY

Theoretical design of stone column with respect to yield load or settlement criterion is a complicated matter. This is due to interaction of a number of factors governing the constituting materials behaviour. Furthermore, load carrying capacity and settlement would very much depend on the method of construction and failure mechanisms of these columns. As a result an impression seems to prevail that the design approach of stone column is highly empirical, this notion is not true. Although the methods of design of stone column is less understood, it is as empirical as the design of pile foundation.

2.3 MODES OF FAILURE

Stone column may suffer failure in a number of modes. They are described in the followings (Ref. Fig.2.1):

i) Bulging: This type of failure shown in Fig.2.1a may be attributed to the plastic failure of an expanding cylindrical cavity.

ii) Pile type failure: Failure may occur by shear failure in end bearing or in skin friction as in case of conventional piles (Fig.2.1b).

iii) General shear failure: Failure like a shallow footing with stone column providing additional support (Fig.2.1c).

Hughes et al (1975) have showed that the first mode of failure is the most common one (Fig.2.2). Their experimental result showed clearly that the ultimate strength of an isolated column loaded at its top only, is governed primarily by the maximum lateral reaction of the soil round the bulging zone and that the extent of vertical movement within the column is limited. Their experiments using radiographic method, revealed that the bulging of the pile occurs near the top at a depth approximately equal to half to one diameter of the pile, as the lateral confinement is minimum there. The radial deformation decreases with depth and appears to be negligible beyond a depth greater than twice the diameter of the pile.

In a conventional pile failure, against by skin friction and/or end bearing only the equilibrium of the vertical forces on the column is considered. Clearly, if the vertical load exceeds the shear resisting forces along the side of the column and the ultimate bearing pressure at the base, the column will push through the soil. For simplicity the limiting value of the shear stresses along the side of the column are taken to be equal to the undrained shear strength of the soil.

Madhav and Vitkar (1978) proposed a general shear failure type mechanism for stone column. In practice, the failure of such category can be avoided by taking advantage of increase of soil stiffness and strength with depth and by replacing the surface layer of weak soil by a well compacted granular material or by covering the soft soil with a pad of granular material.

In a bulging type failure when the stone column yields, four zones may develop (Datye, 1982). These are, (i) arching zone, (ii) slip zone, (iii) compatibility zone with plastic and elastic condition, (iv) load transfer zone.

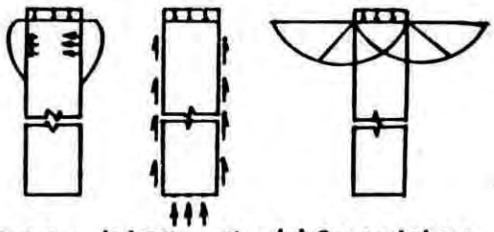


Fig. 2.1. Modes of failure

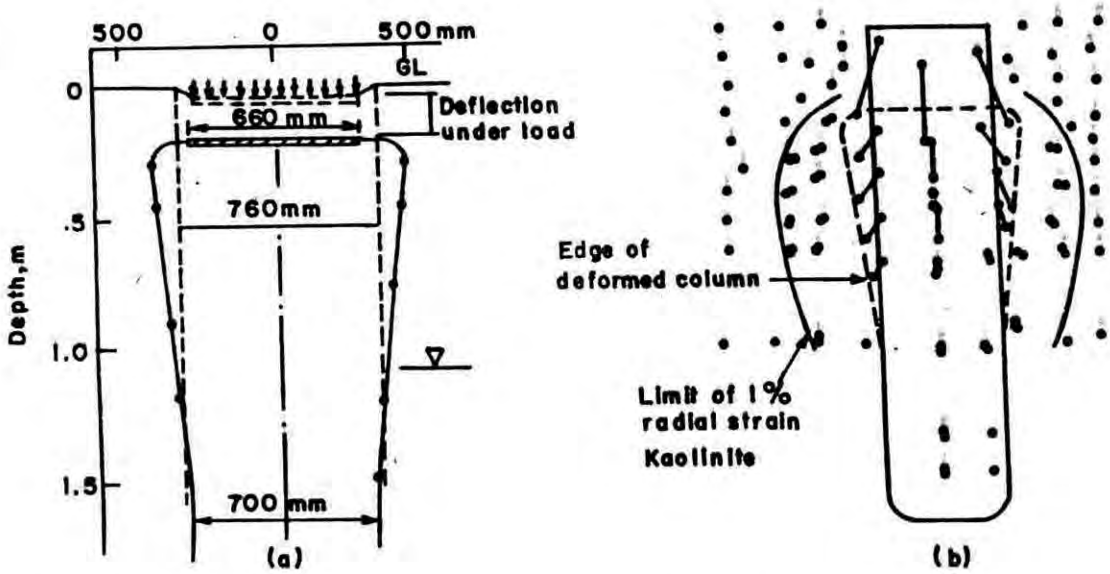


Fig.2.2. Measured shape and deflections (after Hughes et al, 1975)

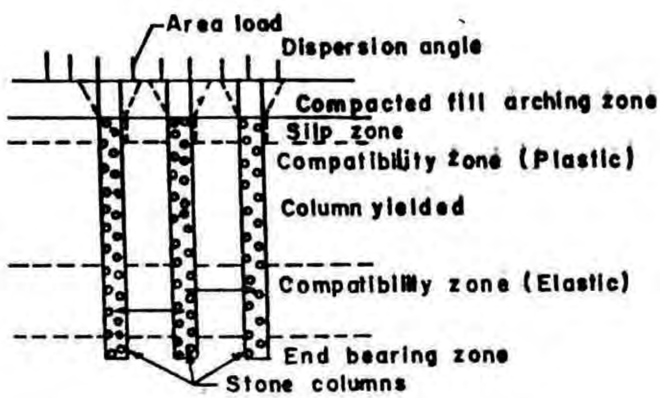


Fig. 2.3. Zone categorization.

Arching zone: When load is applied on the stone column, load transfer takes place by arching of the soil. Depending on the strength of soil, diameter of the stone columns and the depth of the stiff soil layer, the dispersion angle will vary (Fig.2.3). During installation and settlement tension cracks may develop in cohesive brittle strata. It is, therefore, advisable to rely only on the 'pad' of well compacted sandy or gravelly soil for load distribution.

Slip zone: This zone is comparatively of small depth. The slip may not occur, if top layer is of comparatively high strength due to dessication or over consolidation.

Compatibility zone: This zone may consist of a plastic and elastic subzone. (a) Plastic subzone: If the load transfers to the stone column exceeds the yield limit, the region can be defined as plastic subzone. (b) Elastic subzone: Below plastic zone, all the settlements are elastic in nature and compatibility is achieved by the load redistribution which depends on the relative compressibility of stone column and soil.

Load transfer zone: If the lower layers are of stiffer material with sufficient shear strength, the settlements in this portion are elastic as the column is not yielded.

2.4 EXISTING METHODS OF EVALUATION OF LOAD CARRYING CAPACITY

A number of methods for evaluation of load carrying capacity of stone columns are available. These include methods by different researchers developed using various approaches, which are described in the following sections.

2.4.1 Based on Passive Pressure Condition

Greenwood (1970) proposed that the surrounding clay media of the stone column can be expected to mobilize passive pressure conditions during failure. The stone column material gets compressed axially and expand laterally. He proposed the following equation for evaluation of the lateral stress.

$$\sigma_R = \gamma \, Z \, Kpc + 2c\sqrt{Kpc} \qquad 2.1$$

where, σ_R= passive resistance of the soil; γ = unit weight of the clay; c = cohesion of the clay; Z = depth of clay; Kpc = Rankine coefficient of passive resistance. The ultimate stress, q_u carried by the stone column will be

$$q_u = \sigma_R Kps \qquad 2.2$$

where, Kps = $\tan^2(45^o + \phi'/2)$; ϕ'= angle of shearing resistance of the granular material.

2.4.2 Based on Expansion of a Cylinder

Gibson and Anderson (1961) proposed the following equation to evaluate the limiting stress in a cylindrical cavity. They assumed that the surrounding clay media of the stone column will behave like an ideal elasto-plastic material.

$$\sigma'_R = \sigma_{Ro} + c_u(1 + \log_e (E/2 (1+\mu)c_u)) \qquad 2.3$$

where, σ_{Ro} = total insitu radial stress; E = modulus of elasticity of the clay; μ = Poisson's ratio of the clay; c_u = undrained cohesion of clay.

In other words the stone column can be thought of as being confined in a triaxial stress system where the cell pressure is limited. Therefore, there is an ultimate load that the column can carry. From a detailed examination of many field records of quick expansion pressuremeter test, Eqn.2.3 is modified as suggested by Hughes and Withers (1974) for the normal range of E/c_u. This may be expressed as,

$$\sigma_R = \sigma'_{Ro} + 4c_u + u = \sigma_{Ro} + 4c_u \qquad 2.4$$

Ultimate stress, $q_u = k_{ps}(\sigma_{Ro} + 4c_u)$

where, σ'_{Ro} = effective insitu radial stress; u = pore pressure.

2.4.3 Based on Cavity Expansion Theory

Vesic (1972) proposed the cavity expansion theory which constitutes the main theoretical basis of estimation of the yield stress or the maximum vertical stress in a stone column, beyond which excessive deformations would occur. The cavity expansion theory can be applied to evaluate the vertical yield stress according to the following equation.

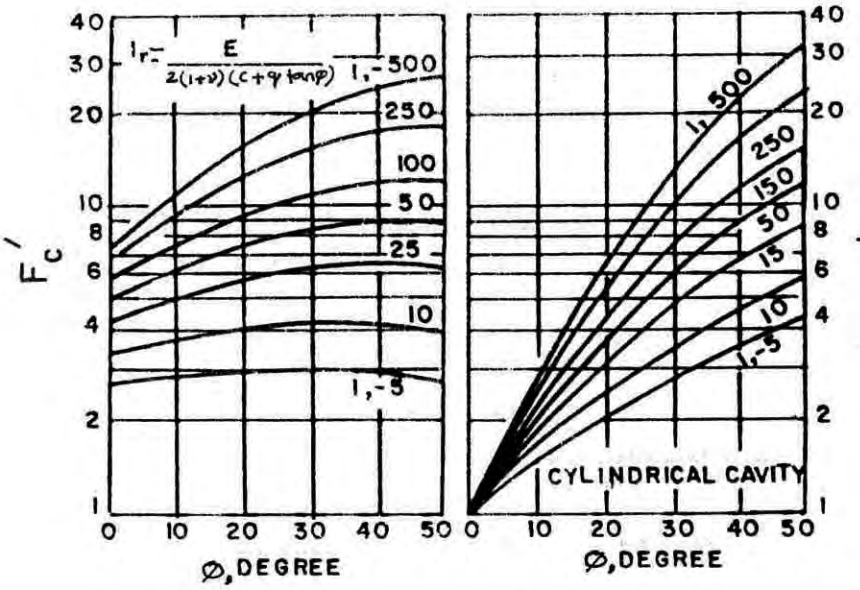


Fig. 2.4. Cavity Expansion Parameters.

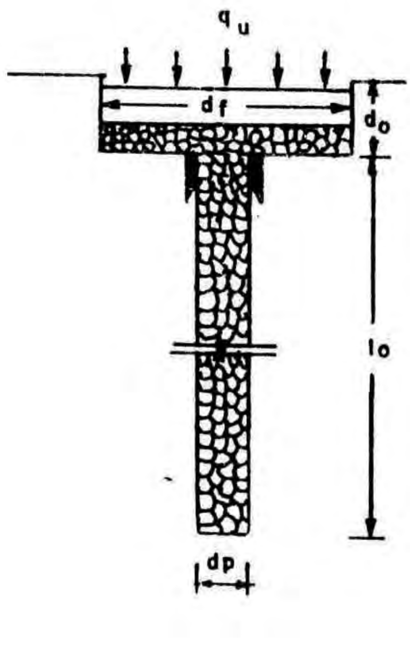


Fig. 2.5. Definition sketch.

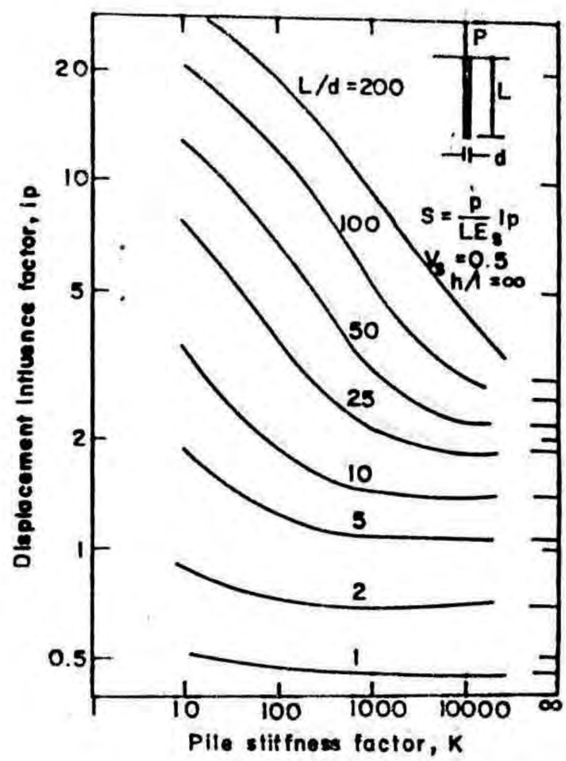


Fig. 2.6 Displacement influence factors.

$$\sigma_R = F'_c \, c_u + F'_q \, q \qquad 2.5$$

$$\sigma_\theta = \sigma_R \, N_\phi \qquad 2.6$$

where, $N_\phi = (1 + \sin\phi)/(1-\sin\phi)$; c_u = undrained shear strength of clay; q = effective mean normal stress; ϕ = angle of shearing resistance; σ_R = principal stress in the radial direction; σ_θ = principal stress in the circumferential direction; F'_c , F'_q = the cavity expansion factors for cylindrical cavity. These two parameters (F'_c and F'_q) depend on angle of shearing resistance of soil (ϕ) and the Relative Rigidity Index of the soil (Irr) shown in Fig.2.4, which is a function of Rigidity Index (Ir) and volumetric strain in the plastic region (Δ). The Rigidity Index (Ir) is a function of modulus of elasticity (E), Poisson's ratio (ν), cohesion (c), angle of shearing resistance (ϕ) and effective mean normal stress (q) of the soil. The suggested relations by Vesic are:

$$F'_q = (1+\sin\phi)(\,Irr \sec\phi\,)^{\frac{\sin\phi}{1+\sin\phi}} \qquad 2.7$$

$$F'_c = (F'_q - 1)\cot\phi \qquad 2.8$$

where, $Irr = Ir/(1 + Ir\,\Delta \sec\phi) = \xi'_v \, Ir$; ξ'_v = volume change factor for a cylindrical cavity ; $Ir = E / (2(1 + \nu)(c + q\tan\phi))$ For $\phi = 0$, and for incompressible soil ($\Delta = 0$)

$$F'_c = \ln Ir + 1$$

This is identical with the value found by Gibson and Anderson for frictionless soil.

The cavity expansion theory is a very useful tool for understanding the factors influencing the yield values of the vertical stress in the stone column and for interpreting the load test data so that the test results can be used for evaluating design parameters.

2.4.4 Based on Pile Formula

The ultimate load carrying capacity of stone column can be estimated using the conventional formula which are used to evaluate the load carrying capacity of piles. In this case, total vertical load is carried by the skin friction which developed between the pile and clay interface due to the movement of pile and end bearing which developed at the base of pile. The vertical load carried by the stone column is calculated by the following equation.

$$q_u = c_u (4(l/d) + 9)) \qquad 2.9$$

where, q_u = ultimate stress carried by the stone column; c_u = undrained shear strength of clay; d = diameter of stone column; l = length of stone column. In Eqn.2.9 it is assumed that the shaft friction is equal to the undrained shear strength (c_u) of clay. It is also assumed that the frictional resistance is constant throughout the length of stone column. The bearing capacity factor for deep foundation is taken as 9.

2.4.5 Based on General Shear Failure

If the mechanical properties of the soil are such that the strain which proceeds the failure of the soil by plastic flow is very small, the footing does not sink into the ground until the plastic equilibrium has been reached, this type of failure is called general shear failure. In case of stone column, Madhav and Vitkar (1978) proposed a general shear failure type mechanism. The equation given by them, to calculate the ultimate load carrying capacity of a granular pile similar to the bearing capacity equation of a shallow footing given by Terzaghi for ideal soil condition.

$$q_u = cNc^* + D_f Nq^* + 0.5\ BN_\gamma^* \qquad 2.10$$

where, bearing capacity factors Nc^*, Nq^* and N_γ^* depend on the frictional resistances of the granular and stabilized soils and the ratio d_f/dp, where, d_f = size of the footing; dp = size of the stone column as shown in the Fig.2.5.

2.5 SETTLEMENT ANALYSIS

Settlement predictions were made by a variety of procedures ranging from empirical formulae to nonlinear finite element analyses. The theoretical bases to predict settlement were developed by Mattes and Poulos (1969), Hughes et al (1975), Balaam et al (1977) and Balaam and Poulos (1978). In general settlement analysis of stone column, falls into two broad categories (a) when the stone column does not yield under working load, (b) when the stone column has yielded itself. The possibility of stone column yielding arises in cases where the stone columns are used primarily to reduce settlement and the structure supported by the foundation system with stone columns can tolerate the estimated settlement. Typical structures are steel tank for storage of liquid, box abutments or structures resting on rigid rafts and high embankments.

2.5.1 Elastic Continuum Approach

Elastic theory can be used for the category of settlement problems when stone column does not yield under working load. Mattes and Poulos (1969) have presented a solution for settlement of single compressible pile and it can be directly applied to granular pile as a first approximation. The first approximation which implied here is that the granular pile behaves elastically under loading. The settlement (ρ) of the top of a compressible floating pile, is given by

$$\rho = (P / (Es\ Lp)) I\rho \qquad 2.11$$

where, p = applied load; $I\rho$ = displacement influence factor, k = E_p/E_s ; E_s = modulus of elasticity of soil; E_p = deformation modulus of the pile. Fig.2.6 presents the displacement influence factors. For a compressible pile (k =10-25), the effect of finite layer depth appears to be small. This type of solution for a granular pile in a semi-infinite medium may be used.

2.5.2 Reduced Stress Methods

Reduced stress methods, proposed by Priebe (1976) and Aboshi et al (1979) are based on the concept that vertical stress concentrations on the stone columns give a reduced average stress on the soft soil. Important parameters to estimate by these methods are the stress concentration factor and the replacement ratio i.e. the proportions of the plan area that is coverd by stone columns. The replacement ratio is determined by the knowledge of stone column diameter and spacing.

The stress concentration factor depends on the relative stiffness of the stone column and soft soil. From field measurement on several projects it has been determined that the ratio of vertical stress in the stone column to that in the soft ground is in the range of 2 to 6, with values of 3 to 4 are usual. Priebe (1976) and Aboshi et al (1979) also presented methods for determination of this factor based on column properties and spacing. These can be expressed mathematically as described in the following:

A stress concentration ratio (n) may be defined as ratio of vertical stress in the compacted stone column (σ_s) to the vertical stress in the soft clay (σ_c)

$$n = \sigma_s / \sigma_c \qquad 2.12$$

If 'As' is the cross-sectional area of the stone column and 'Ac' is the plan area of soft clay per column, the following equation can be obtained

$$(As + Ac)\ \sigma = As\ \sigma_s + Ac\ \sigma_c \tag{2.13}$$

where, σ = average applied stress. A replacement ratio (a_s) may be defined as

$$a_s = As\ /\ (As + Ac) \tag{2.14}$$

by equating Eqns. 2.12, 2.13, 2.14 the following equations can be obtained.

$$\sigma_c = \sigma\ /(1 + (n-1)\ a_s) = \mu_c \sigma \tag{2.15}$$

$$\sigma_s = n\ \sigma/(1 + (n-1)\ a_s) = \mu_s \sigma \tag{2.16}$$

The consolidation settlement (s) of untreared ground would be, s = m_s σH. The consolidation settlement of the composite foundation (s') would be

$$s' = m\ \sigma_c H = m_s \mu_c \sigma H \tag{2.17}$$

and the settlement reduction ratio will be

$$\beta = \mu_c = 1/(1 + (n-1)a_s) = s'/s \tag{2.18}$$

When these methods are applied by Mitchell and Hubber (1982) to the Santa Barbara conditions it is found that the settlement of the treated ground should be about one third that of the untreated ground by the Priebe's method, where as, Aboshi et al's method predicted 40 to 50 percent of the untreated ground value, depending on the column spacing.

2.5.3 Approach of Hughes et al (1975)

Hughes et al (1975) have presented a simple method for the estimation of the settlement of the granular pile. This method is based on the assumption that the granular pile expands radially at constant value. In this method, the pile is divided into layers, and the settlement of the top of the pile is the sum of settlements of each layer.

$$\rho = \sum_{i=1}^{m} \delta_i \qquad 2.19$$

where, $\delta_i = 4\ H_i\ (\delta_{ri}/d_p)$; H_i = thickness of the ith layer; $2\delta_{ri}/d_p$ = radial strain of the ith layer. The radial strain should be calculated from the radial stress strain properties of the clay.

2.5.4 Finite Element Approach

Balaam et al (1978) have presented a finite element approach for the prediction of load settlement response of stone columns. It is assumed that the treated granular pile and soft clay materials are elasto-plastic in nature. The effect of relative stiffness of the granular pile on the load settlement relation is shown in Fig.2.7. The increase in q-value is two fold at small deformations for a four fold increase in k-values while the increase in about 70 percent at δ/d_p= 0.006. Balaam and Poulos (1978) have worked out (Fig.2.8) the load displacement relations for the following cases: (i) Adhesive slip (ii) Frictional slip and (iii) No slip. Results of all three cases agree closely in

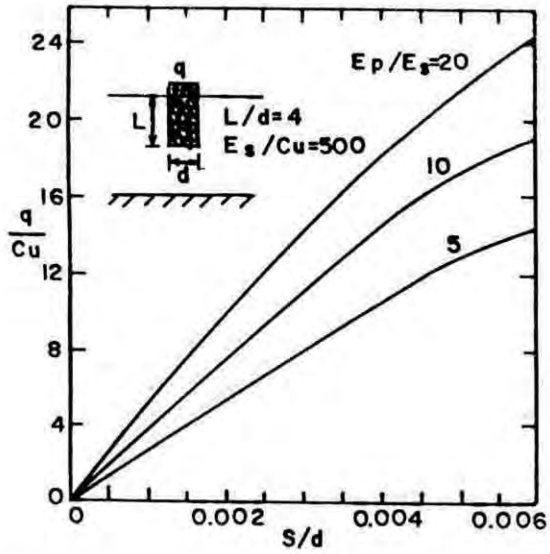


Fig. 2.7 Effect of relative stiffness Es/Ep on the load-displacement behaviour.

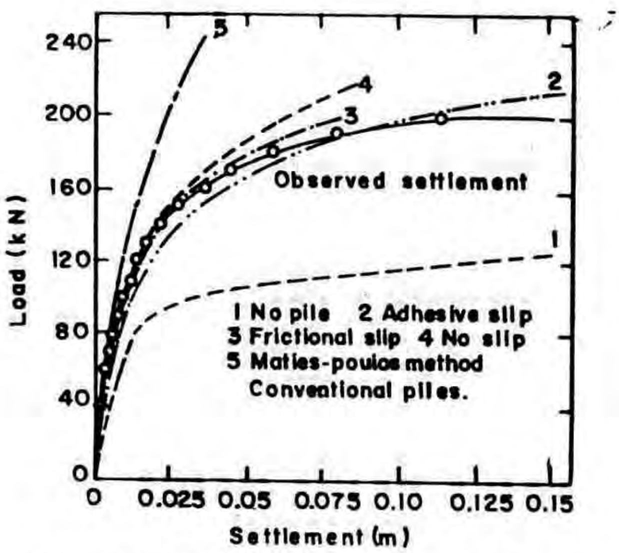


Fig. 2.8 Finite element load test results (Hughes and Withers, 1974).

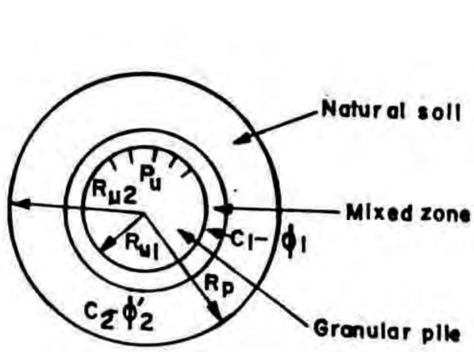


Fig. 2.9 Smear effect around granular pile.

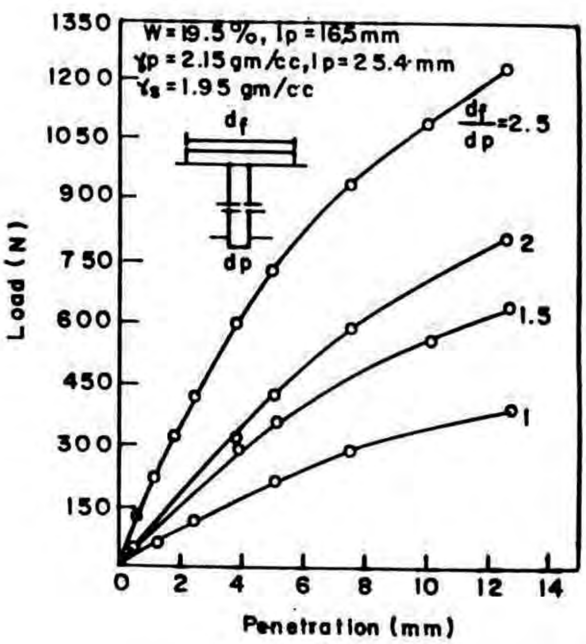


Fig. 2.10 Effect of d_f/d_p.

the initial stages while only adhesive slip is close in the latter stages of the observed response. The plastic yield extends to larger depth into the pile when the loads are smaller. However, for loads approaching the ultimate, the soil elements close to the pile yield approximately over the full length of the pile.

2.6 CONSIDERATION OF SMEAR EFFECT

Installation procedures of stone column have great effect on its load carrying capacity. While the granular pile is being installed by ramming, the gravel perticles (used to form stone column) penetrate into the surrounding soft soil. Due to this operation a zone of clay-granular mix usually develops (Fig.2.9). This effect is almost similar to the smear effect around a sand drain. Due to the pressure of mixed annulus surrounding the granular pile, the cavity expansion equation to find ultimate lateral stress will be different from the earlier case. A solution to this problem can be expressed by the following equation (Madhav, 1982).

$$\sigma_R = (1/a)q_o F_q' + c_u Fc' + c_m \cot \phi_m - c_{m2} \cot \phi_{m2} \qquad 2.20$$

where, c_m and ϕ_m are the strength parameters of the mixed zone; σ_R, q_o, F_q', F_c', c_u are represented earlier (Art. 2.4.3).

$a = (R_1/R_2)^{\frac{2\sin\phi_m}{1+\sin\phi_m}}$; R_1 = radius of granular pile; R_2 = radius upto mixed zone.

2.7 CONSIDERATION OF FOOTING SIZE

When used in conjunction with silty and sandy soils, granular piles help to increase the load carrying capacity significantly. The size of the footing which is supported by a stone column, has a great influence upon its load carrying capacity. If the footing size is larger than the granular pile, a fraction of the load is transmitted by the footing direcly into the soil surrounding the granular piles. This acts as a surcharge load and increases the load carrying capacity of the granular pile (Fig.2.10). The load carrying capacity of a single granular pile of diameter 'd_p' below a circular footing of diameter 'd_f', is obtained by

$$q_u = ((1 + \sin\phi) / (1 - \sin\phi)) (4c_u + \sigma_{Ro} + \alpha\, k_o\, q_s) (d_p / d_f)^2 + (1 - d_p^2 / d_f^2)\, q_s \qquad 2.21$$

where, ϕ = angle of internal friction of granular material; k = earth pressure coefficient at rest condition; q_s = ultimate bearing capacity of the untreated soil; $\alpha = (1 - d_p / d_f)$.

2.8 EFFECT OF SPACING AND APPLIED STRESS ON SETTLEMENT REDUCTION

From the analysis of settlement of stone column under various loads, it was found that the reduction of settlement due to stone column is greatly influenced by the following factors (Datye, 1982). These include, (i) applied stress level; (ii) spacing of stone column; (iii) yield capacity of stone column; (iv) thickness of soft layer; (v) pre-consolidation pressure;

(vi) method of construction. At low applied stress levels and when the stone columns are closely spaced, the stone column may not yield or the depth of yielded zone may be very small. In such cases, the settlement of the improved ground is a small fraction of the settlement of virgin soil. When a stone column yields the reduction of settlement depends on the stress level, the yield capacity and the spacing. Fig.2.11 shows experimental results to demonstrate the effect of spacing and yield stress on settlement. These curves show similar trend as those given by Greenwood (1970). However, the curves given by Greenwood do not seem to be dependent on stress level.

2.9 ESTIMATION OF CRITICAL LENGTH

While the length of the rigid concrete steel piles increases, the load carrying capacity of the pile increases. But in case of granular pile, the increase of length does not lead to increasing the load carrying capacity. A critical length 'Lcr' is reached beyond which there is no further increase in the load carrying capacity of the granular pile. Equating the load carrying capacity of granular pile based on bulging and pile type failures a critical pile length 'Lcr' can be determined.

From bulging, $q_u = k_{ps}(\sigma_{Ro} + 4c_u)$

From pile type failure, $q_u = c_u(4l/d + 9)$.

From these two equations 'Lcr' can be determined which is

$$Lcr = d(k_{ps}(1 + \sigma_{Ro}/4c_u) - q/4) \qquad 2.22$$

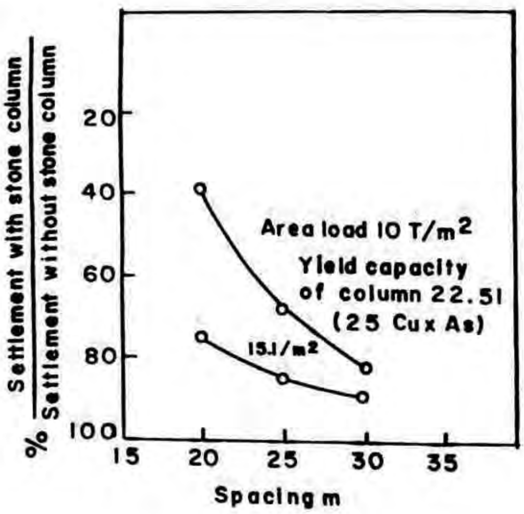


Fig. 2.11 Effect of spacing and load intensity on the settlement reduction.

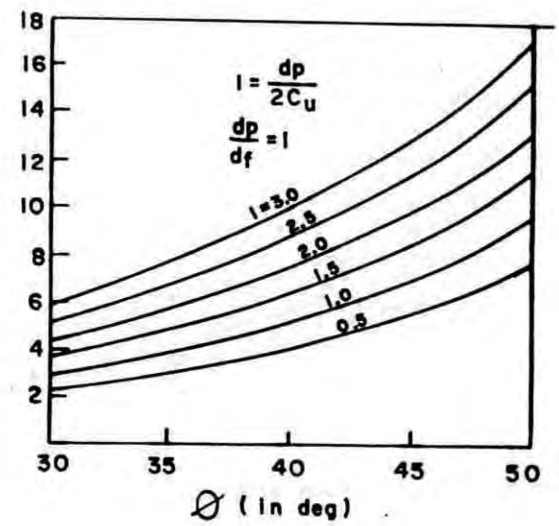


Fig. 2.12 Critical length of granular pile.

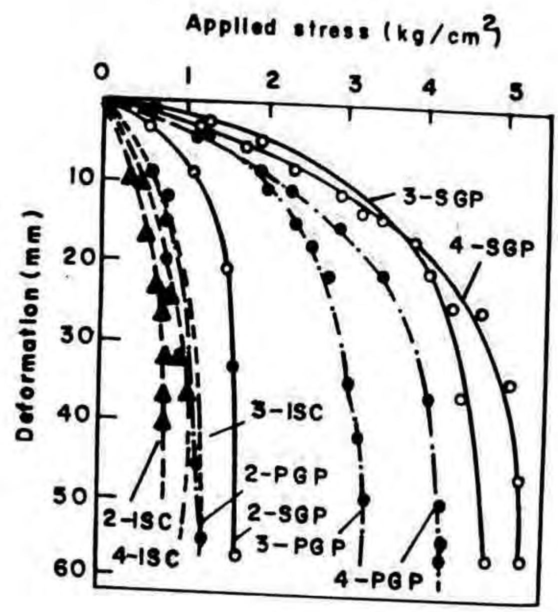


Fig. 2.13 Stress-settlement curves for skirted pile (after Rao Bhandari, 1977).

If $L_p < L_{cr}$, there is pile type failure. If $L_p > L_{cr}$, there is bulging type failure. If σ_{Rd} is calculated at a depth of d_p /2, the values of L_{cr}/d_p can be obtained from Fig.2.12.

2.10 RESISTANCE TO LIQUEFACTION

Loose natural sands and cohesionless fills frequently exsist with widely varying in-situ densities, such variations can lead to substantial differential settlement even under normal loading conditions. If, however, the liquefaction due to seismic loading occurs than the consequences could be disasterous. Phenomenon in which cohesionless soil losses strength during an earth tremor and acquires a mobility to permit large movements upto several thousands of meters. In such case stone columns can also used to minimze the possibility of liquefaction (Englehardt and Golding, 1975) . This is done in two ways. Firstly, the relative density of any confined layer of clean sand is increased and secondly, the gravel columns provide drainage paths which radialy dissipate the excess pore water pressures which lead to liquefaction. The shear strength parameters of a combined mass of stone columns and native soils are significantly higher than those existing prior to stone column installation. The resultant shear strength or improved density can safely resist horizontal force induced by a ground acceleration of .25g.

2.11 SPECIAL TYPE OF GRANULAR PILE

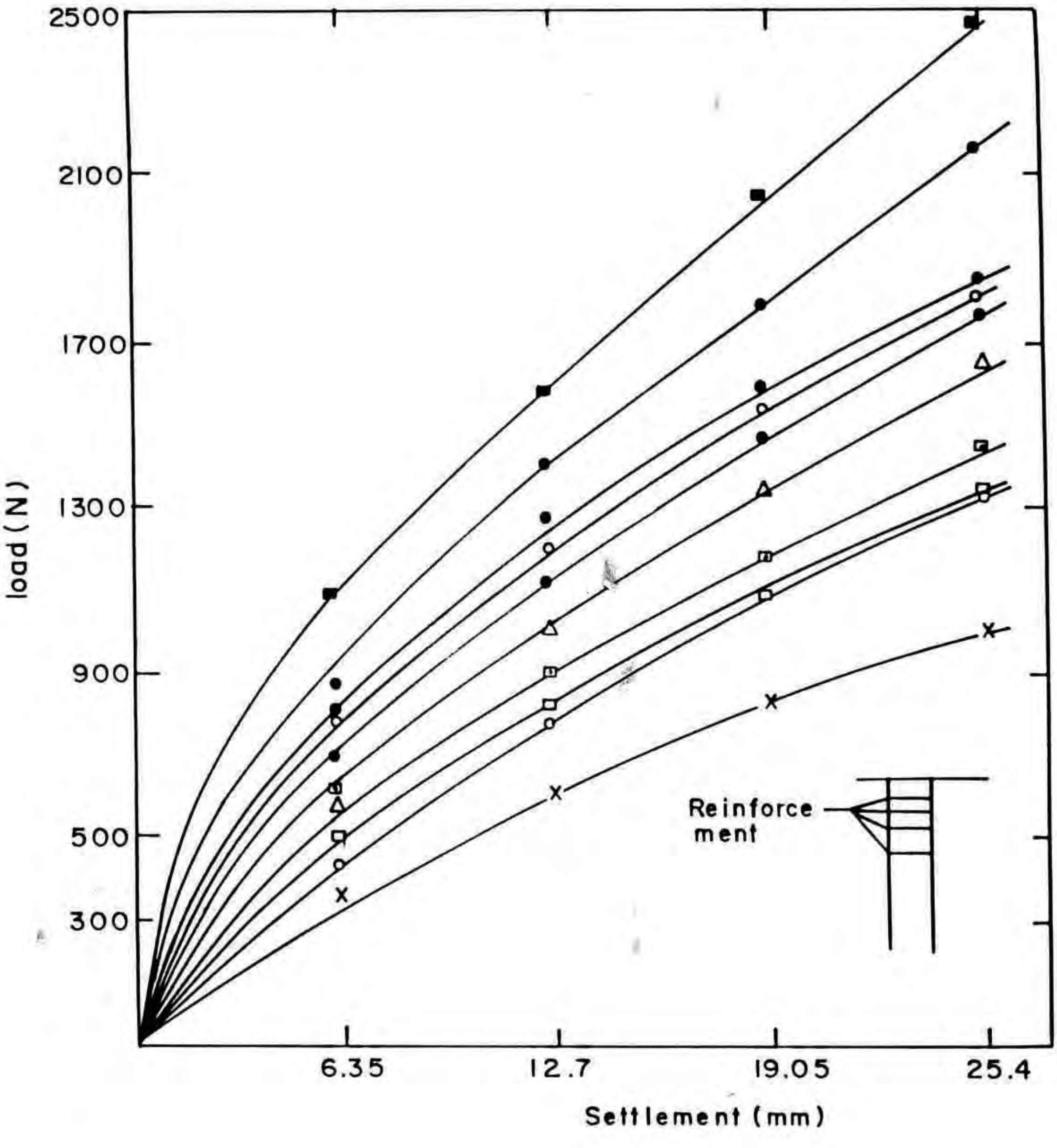
Apart from the conventional plain granular piles a number of other types are envisaged by reserchers which will offer better load deflection characteristics. These types are described in the following sections.

2.11.1 Skirted Granular Pile

Rao and Bhandari (1977) raported the load carrying capacity of granular piles provided with skirts at the top. A rigid skirt provided at the top prevents lateral deformations and thus bulging. Therefore, bulging if at all possible, can occur below the depth of the skirt, by skirting the top of the pile (upto a depth of 0.8 m) it is found that bulging at that region is prevented and increases of the order of 50 percent in the ultimate stress intensity over unskirted piles (Fig.2.13).

2.11.2 Reinforced Granular Pile

Madhav (1982) presented an alternative approach to skirting which is to provide reinforcement in the granular medium to prevent lateral strains (horizontal stiffening membrane effect). The results of small scale model tests on reinforced granular piles indicate that the larger the number of layers the higher is the improvement in the carrying capacity and the stiffness of the reinforced soil (Fig.2.14). Reinforcement increases the load carrying capacity and the stiffness by about four times. Thus, reinforced granular piles appears to be a very appropriate choice and can have advantages over skirted granular piles.



x No pile
o Single pile
□ One mesh of 2.5 cm
Δ " " 10 cm
□ Two mesh at 2.5 & 5 cm
● " " 5 & 10 cm
o Three mesh at 3.3, 6.6 & 10.0 cm
■ Four mesh at 5, 10, 15, & 20 cm
o Five mesh at 5, 7.5, 10, 15 & 20 cm
● " " 2, 4, 6, 8 & 10 cm

dp = 5 cm
df = 12 "
lp = 50 "
γp = 2.1 g/cc
γs = 2.07 "
Soil m.c = 18 %

Fig. 2.14 Effect of reinfereement.

2.11.3 Granular Pile with Rigid Plug

Madhav (1982) claimed by experiment evidence that granular pile if used in conjunction with concrete plug, at the top (the latter does not bulge laterally) can carry significantly large loads. Even if the top 15 percent of the length of pile is rigid, the load carrying capacity is doubled considering settlement. If the top 30 percent is rigid, the load carrying capacity is four times that of the pile without rigid plug (Fig.2.15).

2.12 CHARACTERISATION AND EVALUATION OF DESIGN PARAMETERS

The characteristcs of the constituting materials forming the stone column system such as stone aggregates and clay media are discussed here. The testing techniques for evaluating these design parameters are also discussed. For the existing linear limit state design methods, the parameters which are required for design of stone column foundations are the cavity expansion factors, shear strength parameters, unit weight, bearing capacity factors and the angle of internal friction of aggregates.

2.12.1 Clay Parameters

The cavity expansion factors Fc and Fq depends on initial shear strength (Su), angle of internal friction (ϕ) and shear modulus (G) of the clay. The undrained shear strength of clay (Su) can be determined by conventional unconsolidated undrained triaxial tests. The modulus of elasticity of clay can be evaluated easily from the stress-strain diagram, which is

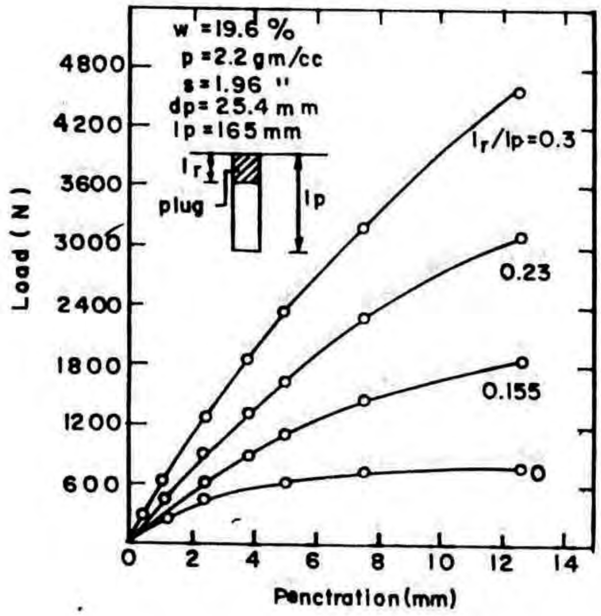


Fig. 2.15 Effect of I_r/I_p.

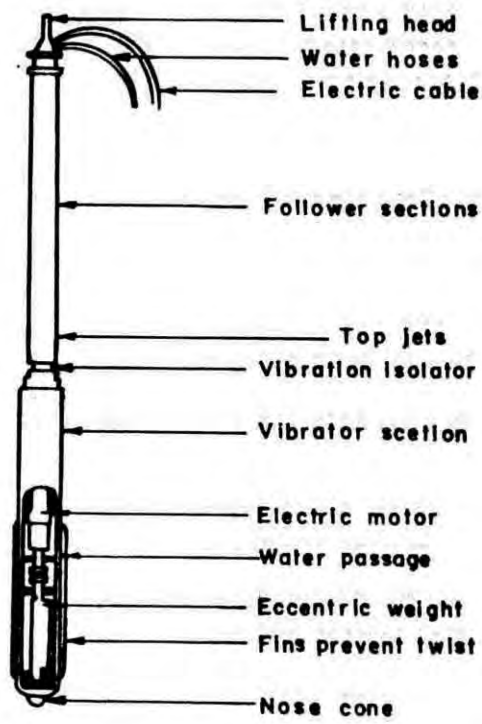


Fig. 2.16. Essential features of a vibroflot

developed by conventional tests. The bearing capacity factors (N_c, N_q and N_γ) can be found from the conventional charts. The unit weight of soil (γ) can be obtained from the laboratory test.

2.12.2 Aggregates Parameters

The design parameter of aggregates, which is required for design of stone column by the existing method is φ' (angle of internal friction). No test are yet reported for determination of this important design parameter. This values are usually taken within the range of 45^{O} to 50^{O}.

2.13 INSTALLATION TECHNIQUES

Although the scope of this research does not include installation techniques, a brief review on these are included for giving completeness to the subject matter. As mentioned earlier, in the 1930's stone columns were rediscovered as a by-product of the technique of vibroflotation for compacting granular soils. In the early 1960's the vibroflotation technique was used for forming stone column in cohesive soils (Dullage, 1969). Still in developed countries the stone columns are generally formed by using vibroflot. In India stone columns are constructed by simple bored piling equipments. Datye (1982) presented extensive studies carried out on stone columns formed by different methods of execution. He stated that rammed columns installed through cased boreholes score over stone columns formed by vibroflot.

2.13.1 Vibroflotation Techniques

The continual development and improvement of the specialist equipment along with better technical and practical appreciation makes vibroflotation probably the most effective method of compacting deep deposits of granular soils and reinforcement of cohesive material by the insertion of stone column. In the following sections, the vibroflot as well as procedures for construction of stone column using its is described.

2.13.1.1 Vibroflot

The modern technique of forming stone columns consists of first forming the hole with an instrument known as vibroflot. Essential parts are shown in Fig.2.16. It is essentially a long slender steel tube with two parts, the vibrator and the follow-up tube. The vibrator consists of a 300-400 mm diameter, hollow cylindrical body 2.0-4.5 m in length. It is connected by means of a special elastic coupling to follower tubes of slightly smaller outside diameter.

Ecentric weights in the lower part of the vibroflot are driven by an electric or hydraulic motor operating generally at 1800 rpm in a horizontal plane, which develops a horizontal centrifugal force of 100 kN. The total weight of a vibroflot is about 20 to 25 kN. The operation of the vibroflot usually requires the discharge of water at different stages from the bottom and top of the vibrator at a rate of 250 to 350 litres/minute and at a pressure of 400 to 550 kPa. Generally, water for the lower jets is carried through flexible pipes to the top of the vibrator where connection is made with passageways in the wall of the vibrator which lead to the bottom of the vibrator water for the upper jets enters at the top of the follw-up tube and fills it. The complete vibroflot assembly can be supported from a crawler crane. The diesel driven hydraulic pump, or 250 kVA portable generator, used to supply the power for the motor is usually mounted above the counter weight at the near of the same crane. Special supporting rigs have also been developed which can exert a downward hydraulic thrust to force the vibroflot into the ground.

2.13.1.2 Procedure of construction stone column by vibroflot

The construction procedure of stone columns using vibroflots is described in the following:

i) The vibroflot suspended by a crane or other support, is positioned above the selected point. With the aid of the lower water jets and its own weight, the vibroflot penetrates to the desired depth. If only air is used, the insitu soil is displaced sideways. Using water only, disturbed material is also flushed from the hole.

ii) The vibroflot is then lifted out. Coarse gravel, crushed stone or slag is tipped into the hole in increments of about 500 mm. The vibroflot is reinserted and compaction begins. Radial forces produced by the vibrator force and added material

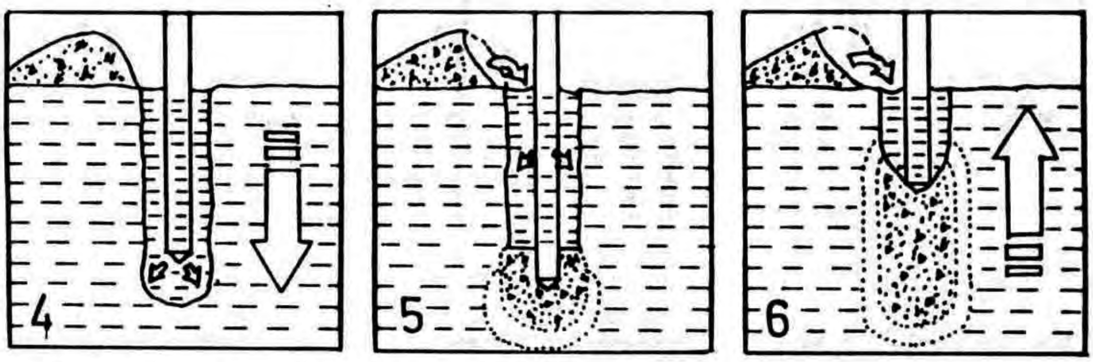


Fig. 2.17. The stone column installation.

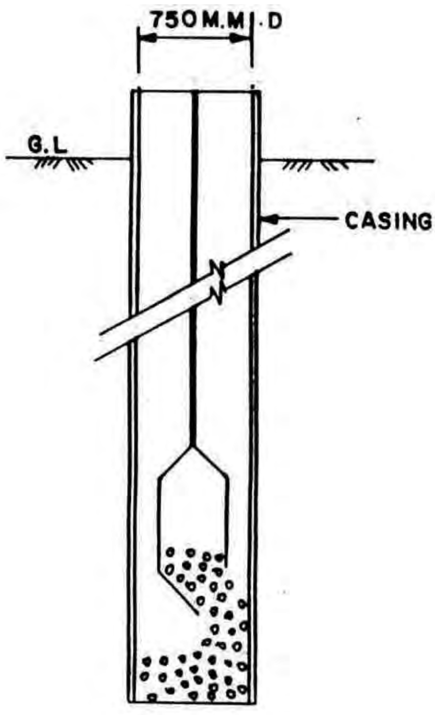


Fig. 2.19. Cased boring

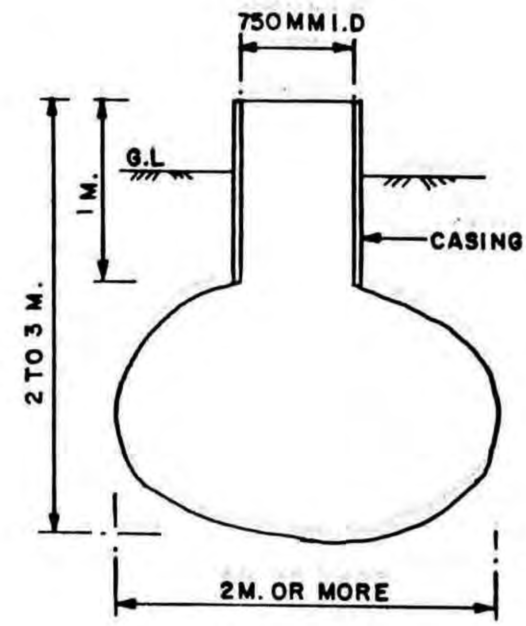


Fig.2.18. Uncased boring

horizontally out against the in-situ soil.

iii) When the required degree of compaction has been reached the vibroflot is again removed. The filling/compacting cycle is repeated step by step to the surface. Thus, a dense column of granular material interlocking with the surrounding ground is formed through the treatment zone as shown in Fig.2.17.

2.13.2 Rammed Method

To achieve optimization in construction procedure by rammed method, a number of field experiments were conducted prior to taking up actual execution work. Nayak (1982) reproted the following on this method which is based on his experiments.

2.13.2.1 Uncased borehole

In this method, a guide casing of 750 mm diameter to a depth 1 m only was adopted (Fig.2.18). Boring to required depth was carried out without casing or drilling mud to stabilise the borehole. Boring was done by bailer with flapvalve at its bottom. If this method were to be successfull, the construction time and cost could have been greatly reduced. Boring is limited to a depth of 2 to 3 m only and borehole turned out to be 2 m wide or even more, hence this method was discarded immediately.

2.13.2.2 Cased borehole with charge placed by placer

Here the casing is provided to full depth of boring. Charge of granular backfill is placed in the bore hole through placers with arrangement to open out at the bottom (Fig.2.19).

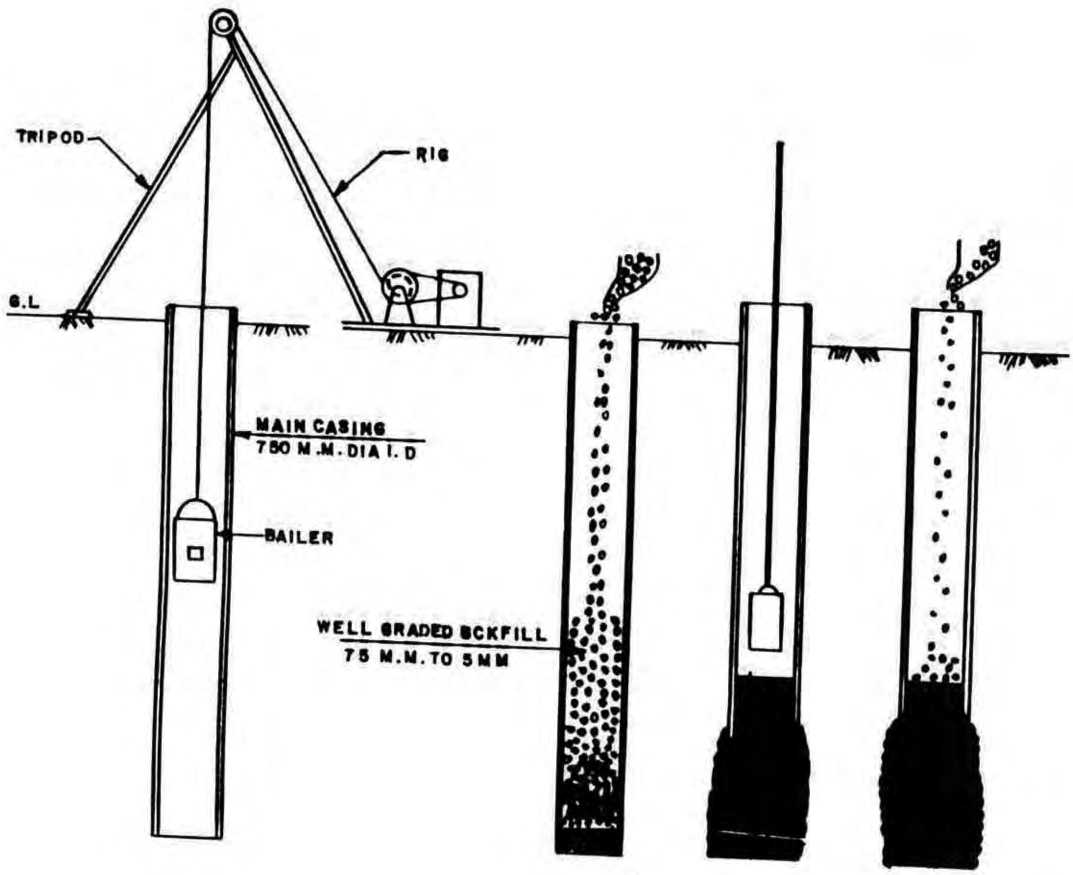


Fig. 2.20 Stages of construction of stone column by bored piling equipment.

The casing is partly with-drawn and compaction is done as illustrated in Fig.2.20. It is observed that considerable time is required in changing the wire rope from placer to rammer and rammer to placer etc.

2.13.2.3 Cased borehole with open windows

This method is similar to the method described in Art.2.13.2.2 except a number of openings (windows) of 300 mm x 200 mm were provided in the casing at 1 metre depth interval. Granular backfill should be placed in the hole through the appropriate window in the casing. But there is considerable amount of ingress of soft soil into the hole which contaminated the granular backfill and this method was also discarded.

2.13.2.4 Cased borehole with flap-door windows

This method is similar to that described in Art.2.13.2.3 except the windows were provided with hinged flap valves which open outside. These doors were kept in closed position during driving or withdrawal by screwing nut. In this method, the granular material was placed in borehole through the window close to the granular level. This casing is withdrawn by antihammering with rammer. This method was found to give functionally better quality stone columns with minimum time.

2.14 CASE STUDIES

A considerable amount of work has been performed successfully throughout the world to improve poor ground by stone column foundation system. This foundation system gives better result both in load carrying capacity and limiting settlement. A few case studies are presented here to show the applicability of stone column foundation system.

2.14.1 Belapur Bridge Abutment, India

Stone columns were used for treatment of foundations of the abutment of a major highway bridge near Bombay. The bridge is over a creek and has spans of 50 m and is designed to carry 70 tons trucked vehicles. The box abutment rests on 750 mm diametr rammed stone columns with spacing 1.7 m. The design capacity of the stone column is 25 tons, the stipulated yield value of 40 tons was confirmed by load tests. Considering load intensity of 14 t/m and plan area of box abutment is 4.5x12 m, the estimated load is 756 tons. The load per stone column was almost equal to the design load since 30 stone columns were provided. Alternatives such as piles, preloading were examined and rejected. Piles would have been subject to heavy drag forces and lateral loads due to the deformation of the soil. Preload would have required longer time for stage loading. The structural performance of the abutment is satisfactory except for a minor crack due to an unsatisfactory juntion detail. The settlement has stabilised and there is no noticeable settlement in the last five years.

2.14.2 Phosphoric Acid Tank at kandla, India

Two 10,000 tonne capacity tanks having diameter 28 m were constructed at kandla during 1972 to 1979 for storing acid. In order to achieve compatibility of the settlement of the stone column and surrounding soft soil which extends to a depth of about 16 m, the ground was preloaded for an intensity of about half the maximum design stress after installing sand drains. The settlement under the preload was about 600 mm. Rammed stone columns of 750 mm diameter were installed after removal of preload. About 3 m of soft clay was removed and a pad of sand was formed under the tank. The tank was tested under water load and filled with acid. The performance of the tank has been monitored and the settlement is stabilised in a period of about a year. Observations over a period of about 8 years since 1975 give maximum differential settlement 80 mm in 28 m. The settlement of untreated ground would have been around 1200 mm about 6 to 8 times the settlement of the improved ground. The stone column system was economical, though a conservative design approach was used in this case, because it was on of the first few major applications of stone columns in India.

2.14.3 Warehouse, Machine Shop, Kandla, India

Rammed type stone columns having 750 mm diameter and 10 m long were installed in warehouse, machine shop area in 1974. Typically two stone columns were placed under each column footing. In the rest of the area, 400 mm diameter sand drains were installed. The area was preloaded to general plinth load intensity. Soil characteristics were similar to those described in section 2.14.2. Inspite of the large difference in the load intensities on the floor and the column footings, no visible differential settlement or cracking has been observed over a period of 7 years and crane rails perform perfectly. Stone columns with sand drains helped to bring down the preload intensity for the footings, reduced cost of preload and made the preloading operation simple. Stone columns helped to mobilise the drag and accelerate consolidation.

2.14.4 Stone Column for Stabilising Embankment in Soft Soil Supporting Massive Structures, Bombay, India

In Bombay city stone columns were used for rapid construction, in a single stage, of embankments and structures forming part of a system of lagoons for sewage treatment. Stone columns were installed to a depth of 6 to 8 m in the area of soft clays. Stone column served to relieve the soil of the load and thereby prevent shear failure. It was presumed that the full reaction of the all stone column swould be mobilised before failure occur along the circular slip surface. A factor of safety 1.5 was considered. From large scale load test the yeild capacity, and vertical and lateral movements of the column was found to be within design value.

2.14.5 Waste Water Treatment Plant Santa Barbara, California

The use of stone column in the santa Barbara waste water treatment plant was the largest application of this type of foundation technique in the U.S.A., in which soil was soft esturine sediments having thickness 5 m to 15 m across the site. Stone columns, having 0,65 m diameter with lengths varying between 8 to 15 m, spacings between 1.2 m to 1.5 m in rectangular pattern were installed in the areas between structures. Lateral load tests were performed to see the adequacy of stone columns to transmit a horizontal shear equal to the design vertical loading on a column times a seismic coefficient of 0.25g with a factor of safety 2. From vertical load test it was found that a single column of a group could carry vertical load of 265 kN at a settlement less than 6 mm. The settlement have remained within acceptable limits and the foundation performance has been excellent.

2.15 LIMITATIONS OF EXISTING THEORIES

Almost all the existing theories for design of stone column are based on linear material behaviour and limit state analyses. In these methods material behaviour are characterised by single parameter representation which in most of the cases will fail to predict realistic behaviour. The limitations of the existing theories are discussed in the following sections.

2.15.1 Based on Passive Pressure Condition

i) In this method, it is assumed that passive limit state is developed in the constituent materials simultaneously but this is not possible unless they have same mechanical properties.

ii) Ultimate load carrying capacity of the stone column is calculated using equation $q_u = \sigma_R \, k_{ps}$, which is based on the yield strength of granular material and surrounding clay media. Compatibility of deformation of the constituent materials are not considered here, which will lead to considerable error.

2.15.2 Based on Pile Formula

i) In this method it is assumed that stone column behaves like a pile in transfering load to the soil. However, it is not correct because the constituting materials such as stone aggregates plays an important role in case of stone column for transfering load in the surrounding clay media.

ii) In case of stone column, the foundation system is considered as a composite system of aggregates and surrounding clay media. This is not considered for the design of stone column, based on pile formula.

iii) The radial deformation of columns govern the load carrying capacity of stone columns, which is almost negligible in case of pile.

iv) Here it is assumed that shaft friction is constant throughout the length of column and is equal to undrained shear strength of clay. However, skin friction is not constant, rather it depends on the vertical and radial displacement of column and related to undrained shear strength by the adhesion factor.

2.15.3 Based on Cavity Expansion Theory

i) Vesic's cavity expansion theory appears to be applicable for precompressed soil only i.e. to a soil which has been first subjected to a very high hydrostatic pressure and than unload.

ii) It is difficult to estimate insitu undrained shear strength of clay in the vicinity of stone column due to influence of installation procedure, which is required to determine the cavity expansion factor F_c' , F_q' .

iii) Since strength characteristics vary in the annulus around the stone column, it is difficult to assign a representative value of rigidity index which is essential for estimating the cavity expansion factors F_c' ,F_q' .

iv) Here load carrying capacity of stone column is based on the ultimate strength of aggregates and clay. Compatibility of deformation of the constituent materials is not taken into account, which is the major drawback of this method. At a given load the same deformation will never produce into the constituent materials unless they have identical mechanical properties.

v) The cavity expansion theory can not be used directly to estimate the value σv because of the above mentioned uncertainities.

CHAPTER THREE

PROPOSED ANALYSIS AND DESIGN METHOD

3.1 GENERAL

Novel analysis and design methods for well known plain stone columns and newly envisaged "Jacketed" stone columns in soft clays are presented in this chapter. The columns that are constructed using aggregates only is called plain stone columns. Stone columns wrapped by reinforcing materials may be called jacketed stone columns. This newly envisaged stone column, having jacket liner around the aggregates will provide separation, filtration and reinforcing function resulting in a stiffer column than its plain counterpart.

Several research workers have advanced theories based on limit state analyses. These include the Cavity expansion theory, Passive failure theory, Pile formula and General shear failure theory on which analysis and design of stone columns are based. The parameters of the constituent materials are obtained from routine geotechnical engineering tests. In almost all of the cases compatibility of deformation of the constituent materials are not taken into consideration. Nonlinear behaviour of the constituent materials such as aggregates and soils are also not taken into account. Therefore, there are inherent shortcomings in almost all of the existing theories that are being used to predict the load carrying capacity and settlement of stone columns. To overcome these limitations the need for a rational sound design method was envisaged. Such a design method is proposed here which is based on nonlinear behaviour of aggregates, reinforcing materials and clay media. Compatibility of deformation amongst the constituent materials in the cavity expansion mode under quasi-static loading condition is also taken into consideration. These enabled theoretical prediction of load carrying capacity and settlement of a single plain as well as jacketed stone colunms.

3.2 DESIGN CONSIDERATIONS

Design of stone column in cohesive soils is generally based on limit state analyses and linear material behaviour. Almost all the design methods are based on so called "elastic" and "ultimate" limiting state approach of the constituent materials without considering the compatibility of deformation amongst these. Plain and jacketed stone columns are composed of three materials (aggregates, reinforcing materials and clay media) possessing different physical and mechanical properties. Therefore, a realistic design method should consider the degree of mobilization of resistances in the constituent materials under compatible deformation conditions. The new design method, presented in this chapter takes these aspects into considertion and are based on the following fundamental considerations:

i) Nonlinear behaviour of the constituent materials of stone column foundation system under quasi-static loading condition.

ii) Compatibility of deformation amongst the constituent

materials into cavity expansion mode under quasi-static loading.
iii) The lateral confinement of the surrounding clay media into the cylindrical cavity expansion mode is evaluated using Ladanyi's (1963) solutions.
iv) The frictional resistances developed within stone column-clay interface due to its vertical and radial displacement is taken into consideration on the basis of the suggestion of Skempton (1966), Burland (1973), Cooke & Price (1973) and Cooke (1974).
v) Hyperbolic model (Kondner, 1963) is used to characterise the stress-strain behaviour of constituent materials. This is used as the main tool in formulating the mathematical relations.

The analyses are, however, based on the following simplified assumptions:

i) The volume of the aggregates remain constant under loading and deformation.
ii) The loading and bearing surfaces are perfectly smooth.
iii) Stone columns rest on a rigid strata.
iv) Plane strain deformation of the clay soil under undrained conditions.

3.3 GEOMETRY OF STONE COLUMN

The geometry and boundary conditions of a single plain and jacketed stone column in a soft clay media is shown schematically in Fig.3.1. The column is generally cylindrical in shape having initial length "Lo" and diameter "$2r_o$". The column

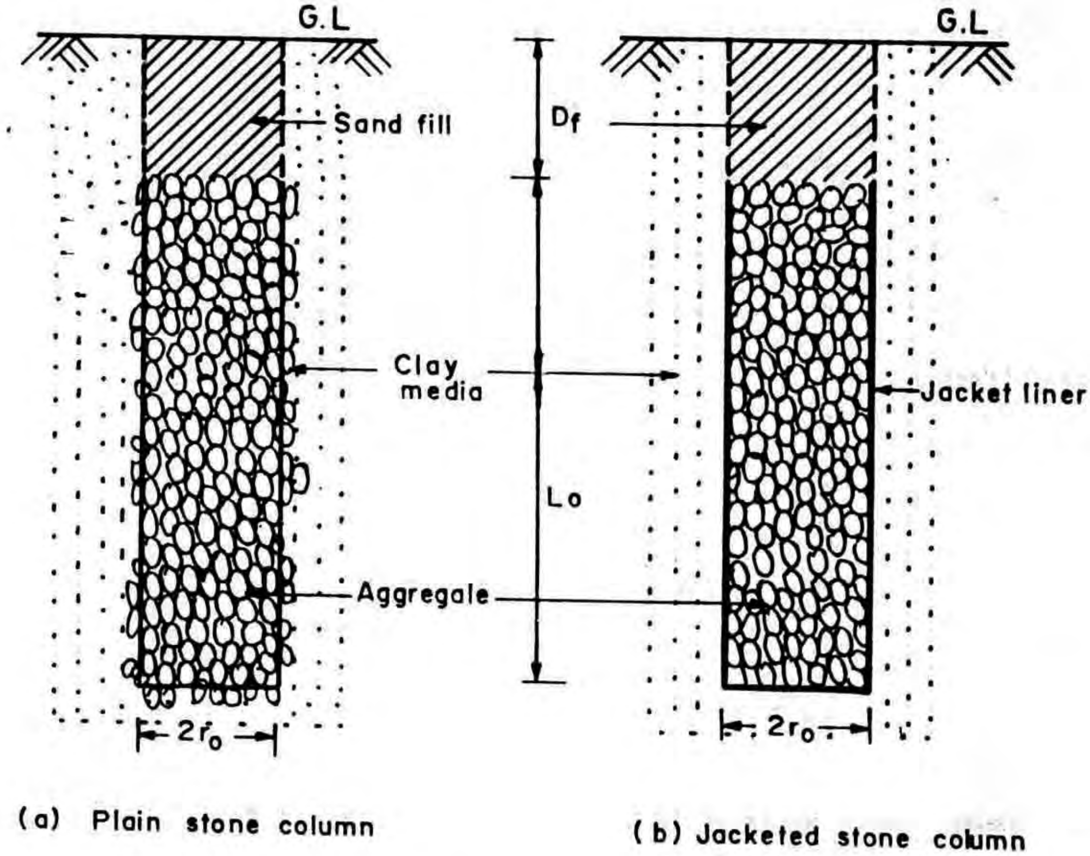


Fig. 3.1. Geometry of stone column.

rests on a rigid base and the top of column is founded at a depth 'D_f' below the ground surface of the clay layer. The cylindrical hole is filled with aggregates which come in contact with the surrounding clay media in case of plain stone column. For the jacketed stone column there is a jacket liner of reinforcing material around the aggregates column.

3.4 LOADING CONDITION

The analysis for the design of stone column is done only for quasi static type of loading. The stone column is mainly subjected to axial downward load. The weight of the aggregates is assumed to be equal to the weight of the displaced soil. The clay is assumed to be soft, saturated and normally consolidated with undrained shear strength linearly increasing with depth. The effective overburden pressure (γZ), the undrained shear strength (Su) and the Su / γ Z profiles are shown in Figs. 3.3, 3.4 and 3.5 respectively. The undeformed and deformed cross section of the column along with initial and final loading conditions on a slice of thickness 'dz', at a depth Z below the ground surface are shown in Fig.3.6. Where, γ Z is the effective overburden pressure; p_h is the lateral resistance; Pv is the vertical load on the stone column. For plain stone column p_h comes from only clay i.e. $p_h = p_s$. For jacketed stone column p_h comes from both clay media and reinforcing materials i.e. $p_h = p_s + p_r$.

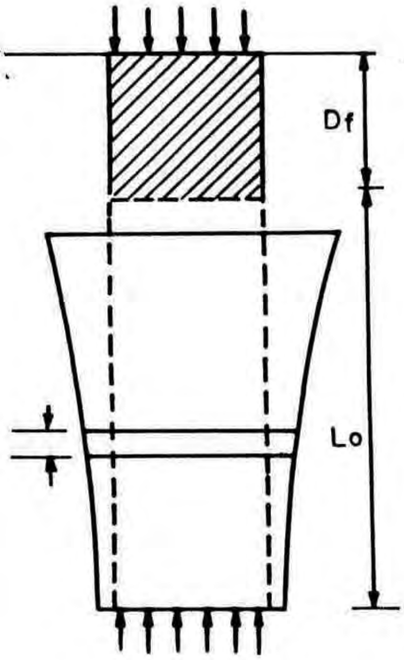


Fig. 3.2

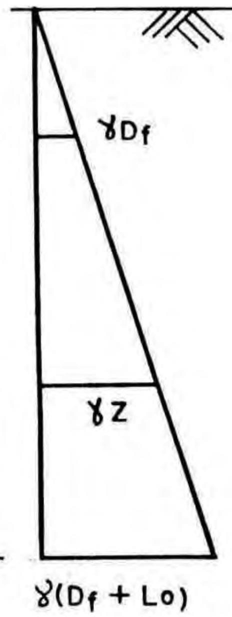


Fig. 3.3

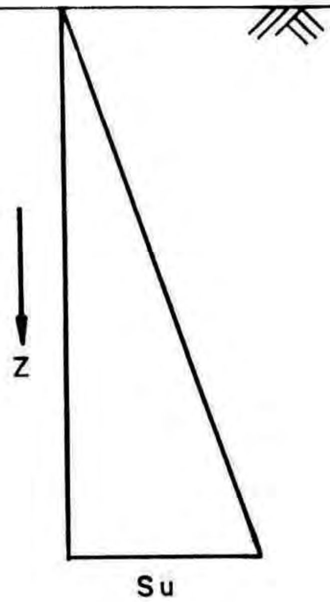


Fig. 3.4

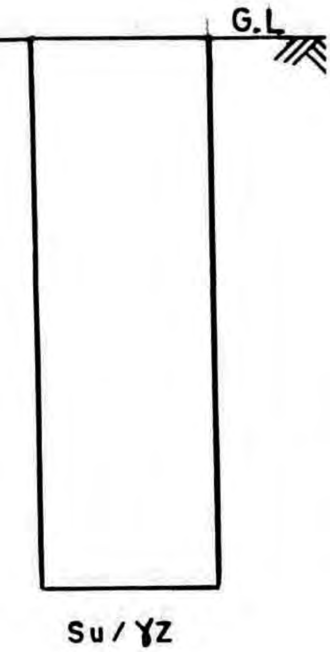


Fig. 3.5

Fig. 3.2, 3.3, 3.4, 3.5 Geometry and Strength Profile

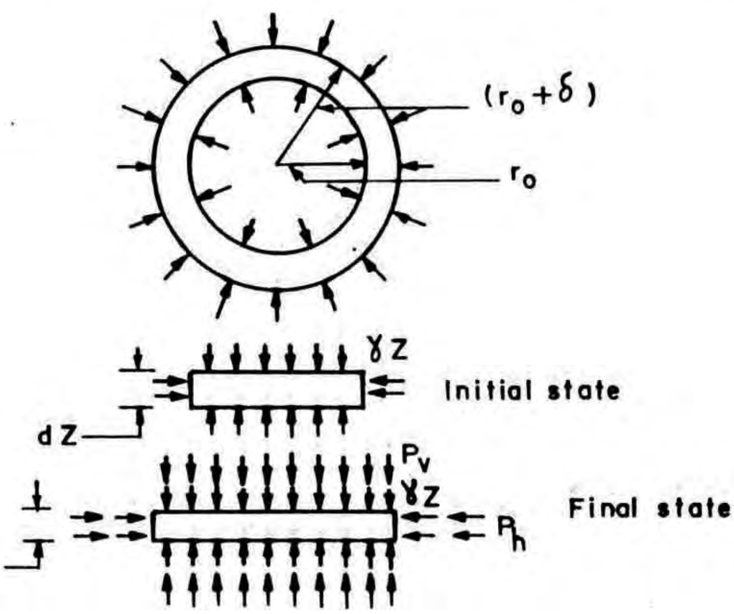


Fig. 3.6. Stress condition.

3.5 GEOMETRY OF DEFORMATION

In case of a solid pile, the lateral soil displacement compared with that in vertical direction is negligible. But the lateral deformation patterns of stone columns play an important role in the proposed analysis and design. The lateral resistance is a function of radial deformation. It is assumed that the maximum deformation will occur at the top of column and decreases with depth. The radial deformation is a function of vertical load (Pv) and the depth (Z) of column. Deformation ratio can be obtained from axial strain by the relationship which will be described later. The deformation pattern is shown in Fig.3.2.

3.6 PROPERTIES OF CONSTITUENT MATERIALS

The physical and mechanical properties of the constituent materials, i.e. aggregates, reinforcing elements and clay media are described here. The constitutive relations and mathematical formulations of behaviour for these materials in case of plain and jacketed stone column foundation systems are presented in the following sections. Hyperbolic representations of stress strain behaviour for these materials are used as these are found to obey this law to a reasonable degree of accuracy.

3.6.1 Properties of Aggregates

The properties of aggregates have a great influence on the load carrying capacity of a stone column. In this study it is assumed that aggregates possess sufficient interlocking to carry the load as well as transmitting these to the underlying strata.

For the characterisation of aggregates behaviour a new suction triaxial test methodology was developed and established, as part of this study. The basic construction elements of this apparatus and test methods are described in chapter 4. The behaviour of the aggregates are described here by K versus δ/r_ocorrelation. Here, K is the ratio of major and minor principal stresses, δ is the radial deformation and r_o is the initial radius of stone column. In thia study nearly uniform sized (9 mm to 12 mm) aggregates is used to achieve as closely as possible the assumed no volume change condition. Three tests at different suctions are performed on samples possessing identical density. The test data is obtained as deviator stress ($\sigma_1 - \sigma_3$) versus axial strain (ε) at these three different suctions. Here, σ_3 is the applied suction, is the applied vertical load and ε is the ratio between the vertical deformation (ΔL) and original length (Lo). These data are converted in to the described K versus δ/r_o relationship which is described in the next section.

3.6.1.1 Constitutive relations

The nonlinear constitutive relation (K versus δ/r_o) for stress strain behaviour of aggregates can be developed using the model forwarded by Kondner (1963). The hyperbolic equation may be presented as,

$$\varepsilon / ((\sigma_1 - \sigma_3)/ \sigma_3) = a_a + b_a \qquad 3.1$$

where, ε is axial strain in percent; σ_1 and σ_3 are the axial and confining stress respectively; and 'a_a ' and 'b_a ' are the two constants of rectangular hyperbola. To establish the constitutive relations, the following steps are recommended.

i) Plot deviator stress ($\sigma_1 - \sigma_3$) versus axial strain (ε) upto failure for different applied suctions (σ_3). The qualitative diagram is shown in Fig.3.7.

ii) Select a suitable limit strain (ε_1).

iii) Divide the deviator stresses ($\sigma_1 - \sigma_3$) by the respective suction (σ_3) and plot ($\sigma_1 - \sigma_3$)/ σ_3 versus axial strain (ε) upto the limit strain. This will trace almost a unique curve. The qualitative diagram is shown in Fig.3.8.

iv) Convert the axial strain (ε) into the deformation ratio (δ/ r_o) by the relationship, described in the next section.

v) Divide δ_r ($\delta_r = \delta/r_o$) by the respective σ_r (σ_r = ($\sigma_1 - \sigma_3$)/ σ_3) and plot δ_r/σ_r versus δ_r, which will be the rectangular hyperbolic form of the curve (Fig.3.8).

vi) The plot in step (v) will yield a straight line, the slope of which is 'b_a' and the intercepted value at $\delta/r_o = 0$ is 'a_a'. These 'a_a' and 'b_a' are the required aggregates parameters. The relevant qualitative diagram is shown in Fig.3.9.

3.6.1.2 Mathematical formulation

In suction triaxial test, using nearly uniform size of aggregates, no volume change condition is assumed. Based on this, the relationship between axial and defomation ratio can be established by the following formulation (see Fig.3.9):

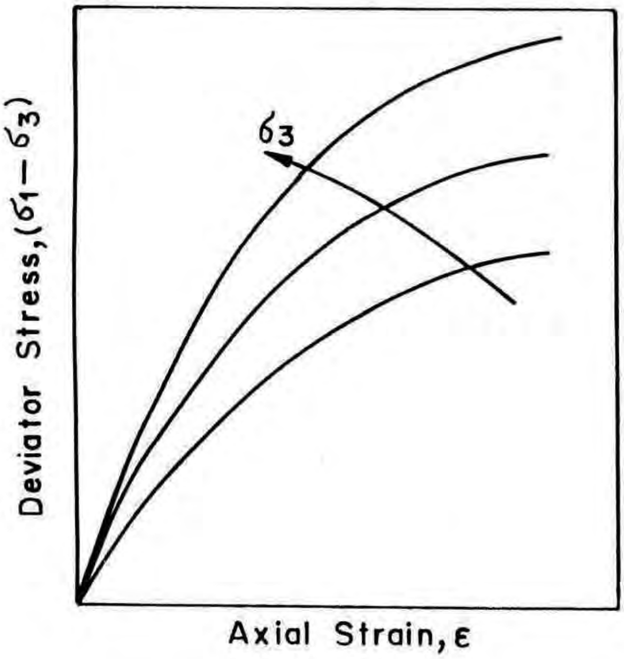


Fig. 3.7 Qualitative stress-strain diagram of aggregates.

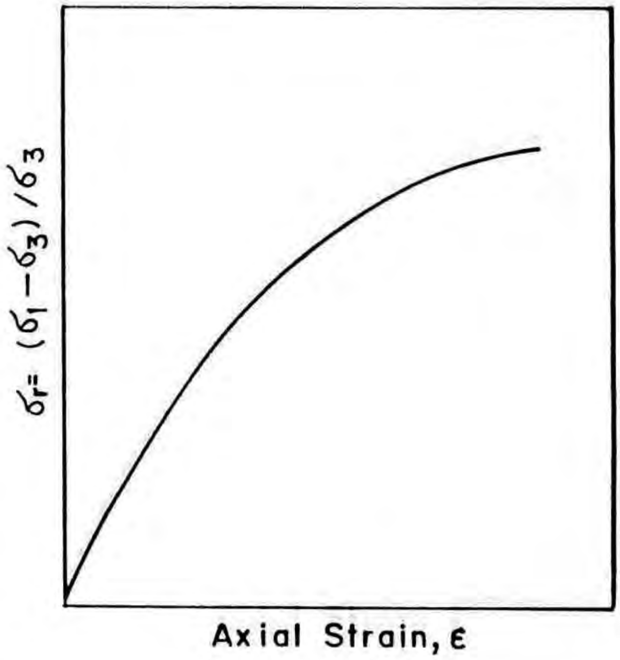


Fig. 3.8 Qualitative diagram of σ_r vs. ε of aggregates.

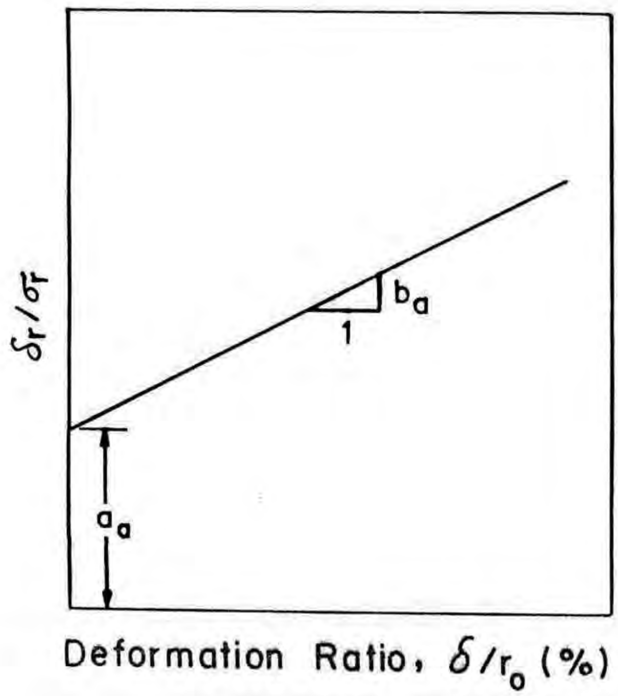


Fig. 3.9 Qualitative diagram of δ_r/σ_r vs. δ/r_o of aggregates.

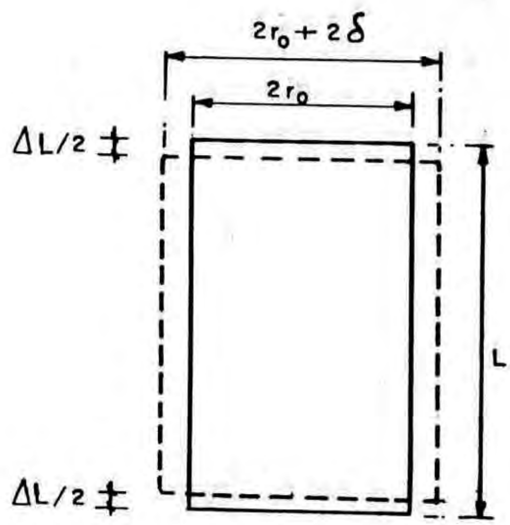


Fig. 3.10 Idealised deformation pattern of aggregates.

$$\varepsilon = 1 - (1 + \delta / r_o)^{-2} \qquad 3.2$$

The constitutive relations can be represented in hyperbolic form as follows (Fig.3.8):

$$(\delta / r_o)/((\sigma_1 - \sigma_3)/\sigma_3) = a_a + b_a(\delta / r_o)$$

by simplifying, the above expression can be written as

$$(1 - \sigma_3/\sigma_1)/(\sigma_3/\sigma_1) = (\delta/r_o)/(a_a + b_a(\delta/r_o)) \qquad 3.3$$

The Eqn.3.3 can be defined into the following, as K is defined by σ_3/σ_1 :

$$K = (a_a + b_a(\delta/r_o))/(a_a + (1 + b_a)(\delta/r_o)) \qquad 3.4$$

In this equation 'a_a' and 'b_a' are hyperbolic design parameters of aggregates. The K value can be obtained directly from laboratory test i.e. $K = \sigma_3/\sigma_1$ or from Eqn.3.4, knowing the values of parameters 'a_a' and 'b_a'. The qualitative diagram for K versus δ/r_o is shown in Fig.3.11.

3.6.2 Properties of Reinforcing Elements

Reinforcing elements will provide separation, filtration and lateral confinement for the stone column. Therefore, these have influence on the load carrying capacity of newly envisaged 'jacketed' stone columns. These will resist bulging of stone column by providing lateral confinement around the stone column. These will also prevent the loss of aggregates into the clay media. Most of the reinforcing element shows nonlinear stress strain behaviour under wide ranges of strain levels. Load (p_r) strain (ε) relations can be developed using hyperbolic representation. Axial strain (ε) can be converted into deformation ratio (δ/r_o) using the relationship describe in the next section. Then the relationship stress (p_r) versus strain δ/r_o) can be established to depict the properties of reinforcing materials. Testing techniques to evaluate the load strain behaviour of reinforcing elements have been suggested by McGown, Andrawes and Kabir (1982) and McGown et al (1984) which are described briefly in chapter 4.

3.6.2.1 Constitutive relations

To depict the nonlinear load strain behaviour of reinforcing elements, a constitutive relation is developed using the hyperbolic model proposed by Kondner (1963). To establish the constitutive relations, the following steps are recommended:

i) Plot load (P_r) versus axial strain (ε) upto the failure for reinforcing elements. A typical qualitative diagram is shown in Fig.3.12.

ii) Divide the axial strain by the corresponding load.

iii) Plot ε / P_r versus ε , which is the rectangular hyperbolic form of nonlinear load strain behaviour of reinforcing elements.

iv) The plot in step (iii) will yield a straight line. The slope of the line is 'b_r ' and the intercept at $\varepsilon = 0$ is 'a_r '. These 'a_r ' and 'b_r ' are the design parameters of reinforcing element. The relevant qualitative diagram is shown in Fig.3.13.

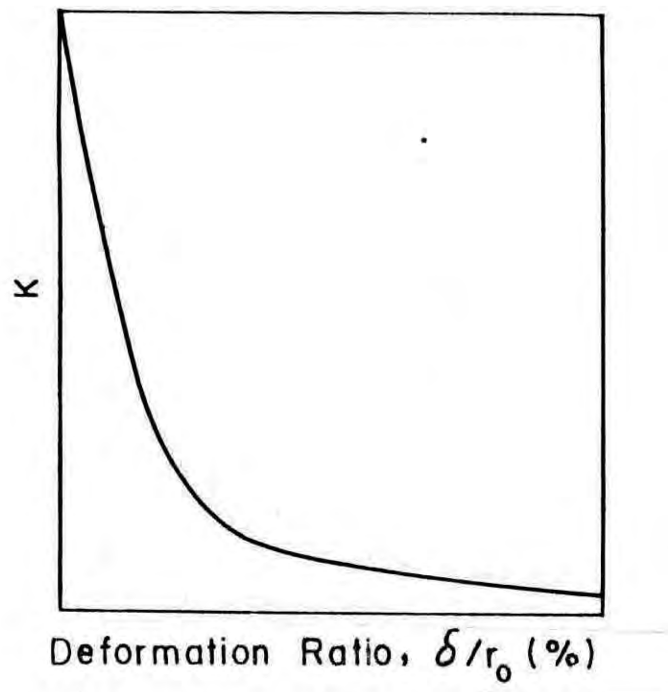


Fig. 3.11 Qualitative diagram of K vs. δ/r_o of aggregates.

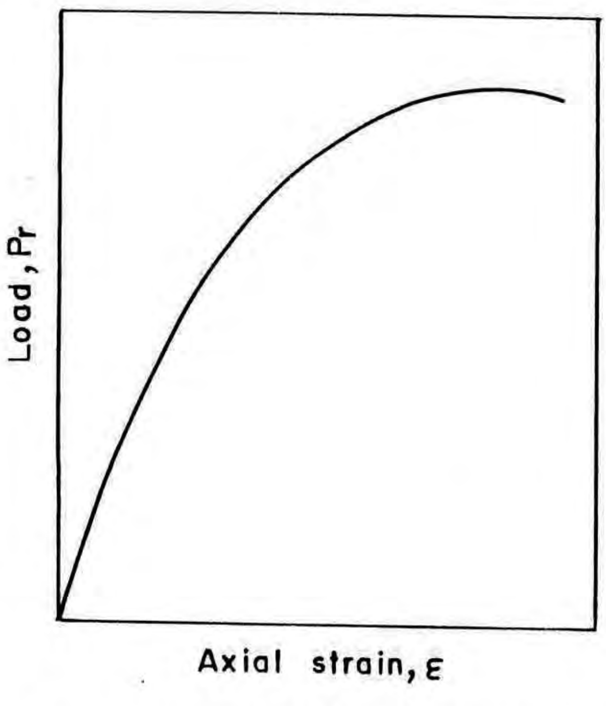


Fig. 3.12 Qualitative load-strain diagram of reinforcing element.

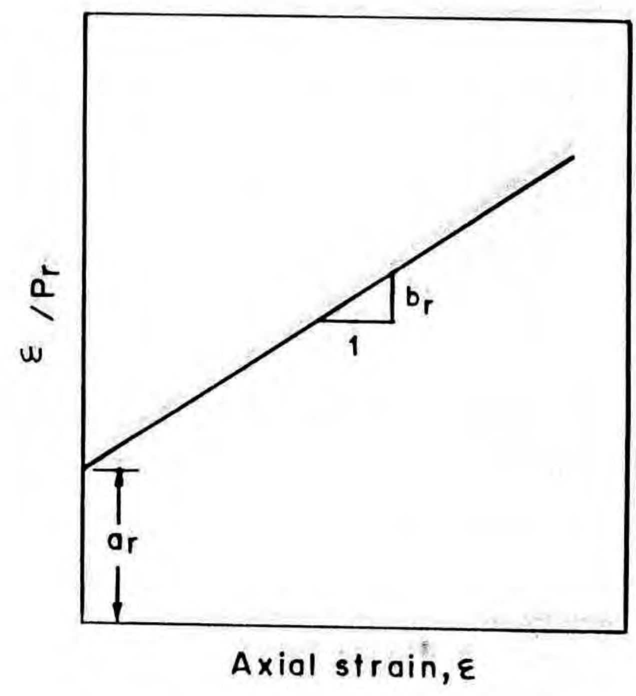


Fig. 3.13 Qualitative diagram of ε/P_r vs. ε of reinforcing element.

3.6.2.2 Mathematical formulation

Reinforcing element is used around the stone column as a jacket liner. When the column is loaded, the stress-deformation pattern of reinforcing element in the cavity expansion mode can be described by the following expression (Fig.3.14). From Fig.3.14, the relationship between axial strain (ε)and dimensionless deformation ratio (δ / r_o) can be written as ,

$$\varepsilon = (2\pi (r_o + \delta) - 2\pi r_o) / 2\pi r_o = \delta / r_o \qquad 3.5$$

The constitutive relation of load strain behaviour of reinforcing element can be presented in the hyperbolic form using the following equation (Fig.3.13):

$$P_r = \varepsilon / (a_r + b_r) \qquad 3.6$$

From Fig.3.15 the load (P_r) mobilised by reinforcing element can be expressed as

$$P_r = p_r (r_o + \delta) \qquad 3.7$$

where, p_r is the internal pressure in the cavity expansion mode taken by reinforcing element. From Eqns. 3.5, 3.6 and 3.7 the following equations can be written.

$$p_r . r_o = (\delta/r_o) / (1 + \delta/r_o) (a_r + b_r (\delta/r_o)) \qquad 3.8$$

$$p_r = (\delta/ r_o) / r_o (1 + \delta/r_o) (a_r + b_r (\delta/r_o)) \qquad 3.9$$

The relationships between $p_r . r_o$ versus δ / r_o and p_r versus δ / r_o for a particular reinforcing element and stone column can be obtained by the Eqns.3.8 and 3.9. Their qualitative diagrams are shown in Figs.3.16 and 3.17.

3.6.3 Properties of Clay Media

Since stone column is installed into clay media, the properties of the clay is very much important to evaluate the load carrying capacity of stone column. Most clay soils show nonlinear stess strain behaviour. In the proposed design method the nonlinear behaviour of surrounding clay media at different level of radial deformation is considered. So a single parameter representation by undrained shear strength (Su), for the surrounding clay soils is not adequate. In this analysis the clay is assumed to be soft, saturated and normally consolidated having undrained shear strength linearly increasing with depth. The lateral confinement given by surrounding clay media due to radial deformation of stone column, is evaluated using the cavity expansion theory forwarded by Ladanyi (1963). The frictional resistance, developed along the stone column-clay interface due to vertical and radial deformation of stone column is evaluated using the equations, developed in the next section, on the basis of Skempton (1966), Burland (1973), Cooke and Price (1973) and Cooke (1974). For analysis and design, strength parameters of clay is determined using consolidated undrained triaxial test for different confining pressures. Moreover, nonlinear constitutive relation for stress-strain behaviour of clay is considered which is also described in the next section.

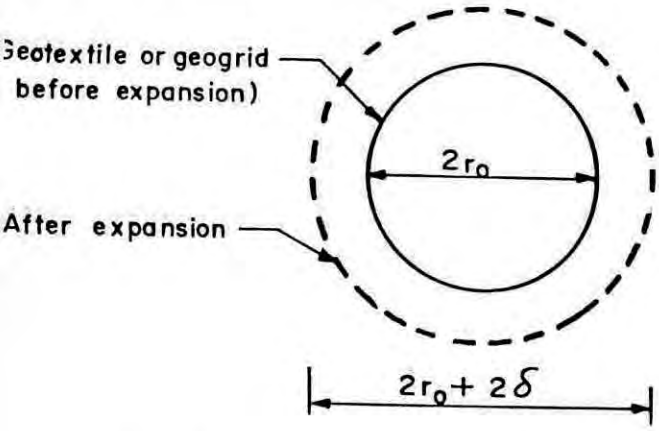


Fig. 3.14. Deformation pattern of reinforcing element.

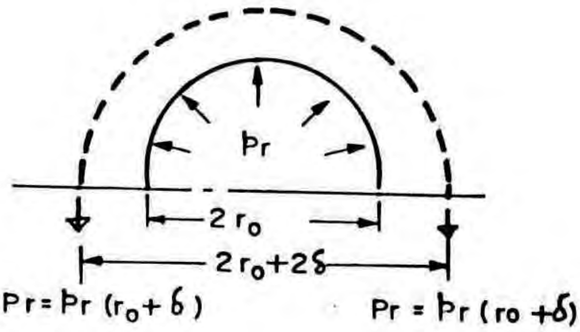


Fig. 3.15. Stress condition of reinforcing element.

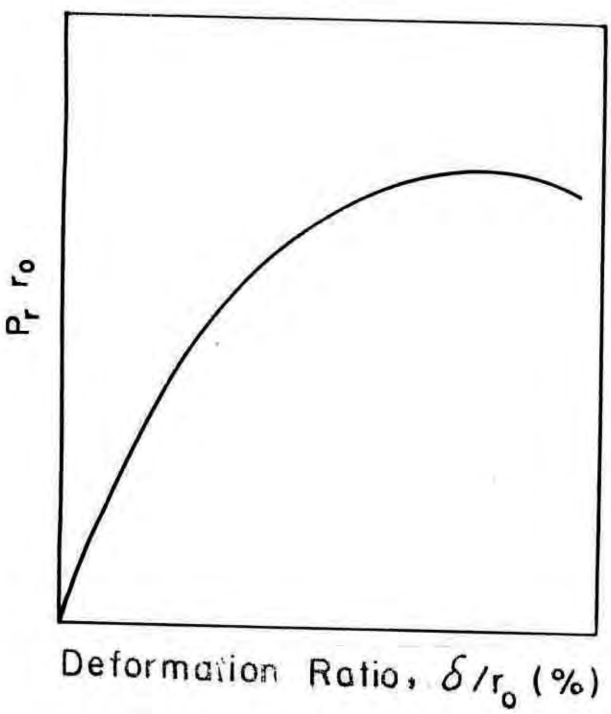


Fig. 3.16 Qualitative diagram of $p_r \cdot r_o$ vs. δ/r_o of reinforcing element.

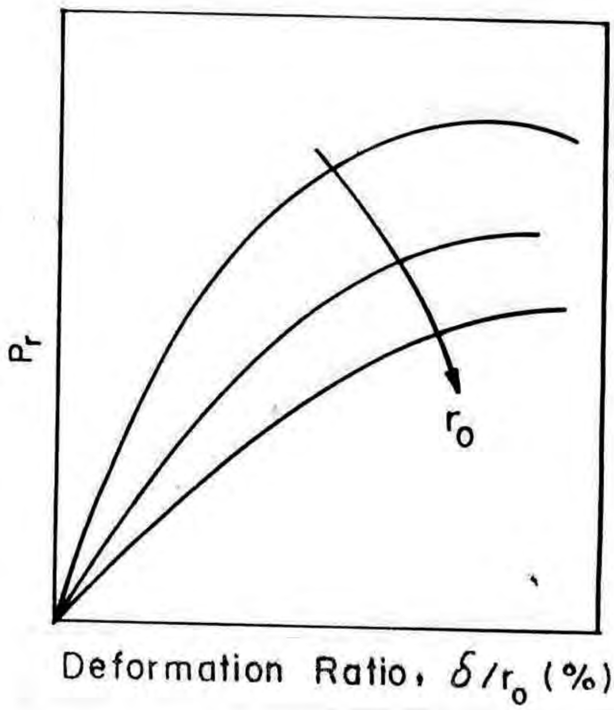


Fig. 3.17 Qualitative diagram of p_r vs. δ/r_o of reinforcing element.

3.6.3.1 Constitutive relations

The stress strain characteristics of soil are very complex due to inherent nonhomogenity and anisotropy. In veiw of this it is very difficult to predict the actual behaviour of soil under applied loads. Kondner (1963) has shown that nonlinear stress strain behaviour of cohesive soils can be approximated with a high degree of accuracy by the following equation, which is hyperbolic in nature.

$$\sigma = \varepsilon / (a + b\,\varepsilon)$$

To establish this constitutive relation the following steps are recommended.

i) Plot deviator stress ($\sigma_1 - \sigma_3$) versus axial strain (ε) upto failure or any other strain level for different confining pressure. Qualitative diagrams from consolidated undrained triaxial test for the clay sample is shown in Fig.3.18.

ii) Select a normalising strain (ε_n).

iii) Divide the deviator stresses by the corresponding confining pressures.

iv) Convert the axial strain (ε) to shear strain (γ_s) by the relationship $\gamma_s = 1.5\ \varepsilon$.

v) Plot $\sigma_r = (\sigma_1 - \sigma_3) / \sigma_3$ versus γ_s, which will almost yield a unique curve, shown in Fig.3.19.

vi) Plot γ_s / σ_r versus γ_s, which is the rectangular hyperbolic form of stress strain behaviour of clay.

vii) The plot of step (vi) will yield a straight line. The slope of the line is 'b_S' and the intercept at $\gamma_S = 0$ is 'a_S'. These 'a_S' and 'b_S' are the required clay parameters. Qualitative diagram is shown in Fig.3.20.

3.6.3.2 Mathematical formulation

Mathematical expression is developed in this section for the following cases:

i) Expression for determining lateral confinement of clay media on the basis of cavity expansion theory.

ii) Expression for determining frictional resistance along stone column-clay interface on the basis of first approximation rules.

i) Lateral resistance offered by surrounding clay of stone column:

The deformation pattern of stone column foundation system is similar to the expansion of a cylindrical cavity of finite radius in a saturated clay media. Therefore, the expression of lateral resistance given by surrounding clay media into the aggregates column can be derived on the basis of the cavity expansion theory, suggested by Ladanyi (1963). To derive this expression the following general assumptions is to be considered.

i) The clay media is homogenious, isotropic, infinite and weightless.

ii) In a first approximation, the effect of the role of strain is neglected, and it is assumed that in every point of the medium is influenced by the expansion of the cavity. The clay has the same

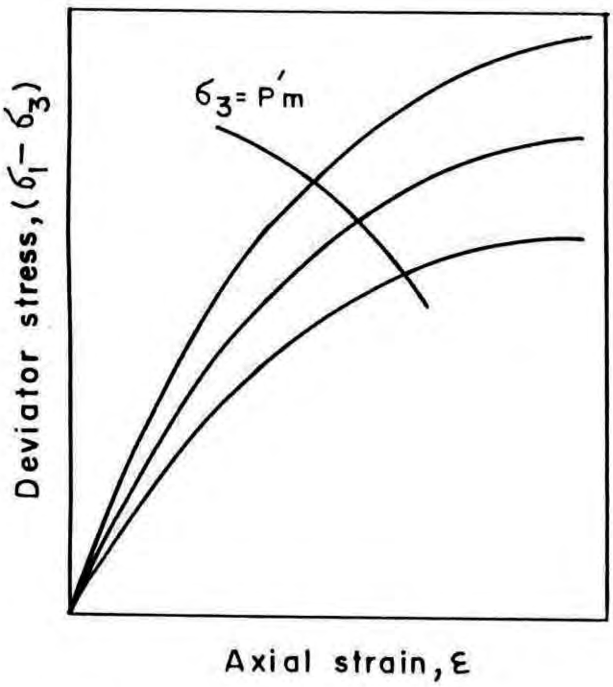


Fig. 3.18 Qualitative stress-strain diagram of clay.

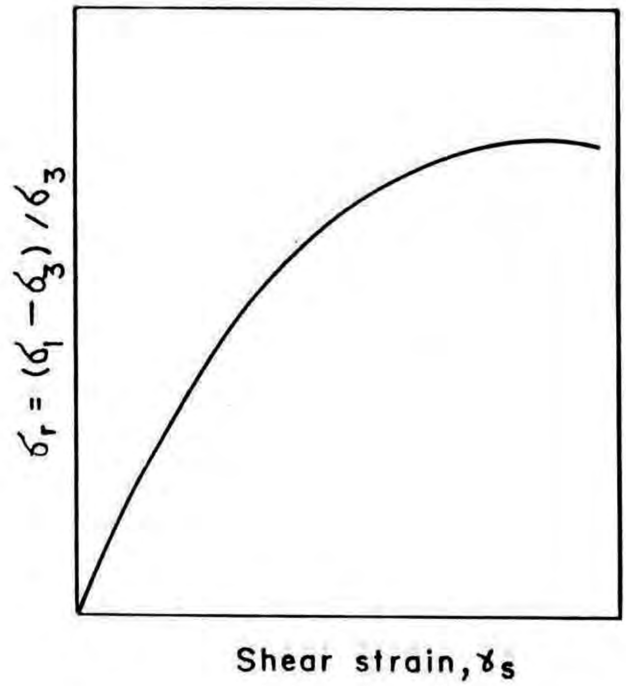


Fig. 3.19 Qualitiative diagram of σ_r vs. γ_s of clay.

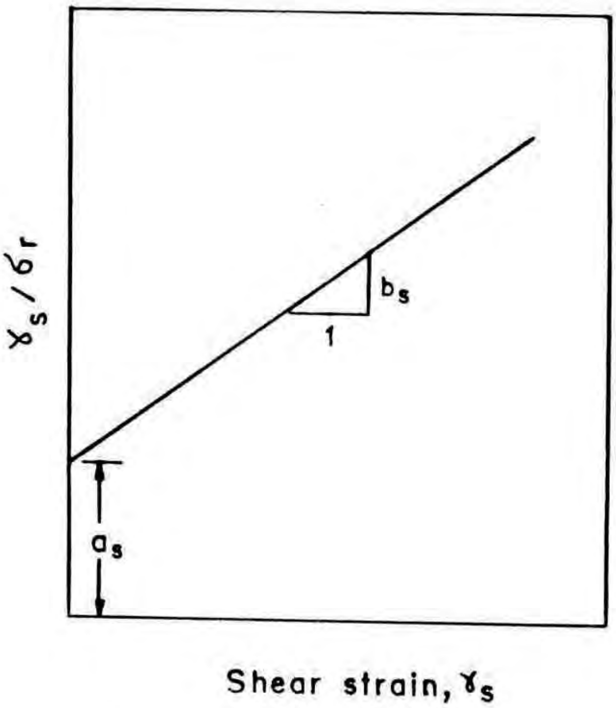


Fig. 3.20 Qualitative diagram of γ_s/σ_r vs. γ_s of clay.

stress strain characteristics.

iii)Expansion of the cavity is occuring in undrained conditions.

To develop the expression for lateral confinement of clay media, its stress strain behaviour is required, which can be established by consolidated undrained triaxial test. From these (Fig.3.18) the constitutive stress strain relationship of clay can be expressed by the following hyperbolic representation (Fig.3.20):

$$\gamma_s / q_r = a_s + b_s \gamma_s \text{, therefore,}$$

$$q_r = \gamma_s / (a_s + b_s \gamma_s) \qquad 3.10$$

where, γ_s = shear strain and it can be calculated from axial strain (ε) by the relationship $\gamma_s = 1.5\ \varepsilon$; $q_r = q / p_m$; q = deviator stress = $(\sigma_1 - \sigma_3)$; p_m' = original effective mean normal stress to which the soil was subjected immediately before undrained shear; a_s , b_s = design parameters of clay.

Ladanyi (1963) proposed that shear strain (γ_s) is a function of ρ / r by the following relationship

$$\tan \gamma_s/2 = (r/\rho)^2/(2 - (r / \rho)^2) = 1 /(2 (\rho / r)^2 -1) \qquad 3.11$$

where, ρ = distance of the point under consideration, measured from the center of the cavity; r = current radius of the expanding cavity. Eqn.3.11 can be expressed approximately by the following relationship

$$\gamma_s \approx (\rho / r)^{-2} \qquad 3.12$$

It can be shown (Fig.3.21) that the error resulting from the Eqn.3.12 instead of Eqn.3.11 is less than 10% at $\gamma_s < 0.20$ and less than 1% at $\gamma_s < 0.02$. So at low level of shear strain Eqn.3.12 can be used satisfactorily. From Eqns. 3.10 and 3.12, the following equations can be derived.

$$q / p'_m = r^2 / (a_s (\rho^2 + (b_s / a_s) r^2)) \qquad 3.13$$

As the symmetry of the problems is considered and neglecting the mass forces, the differential equation of equilibrium at a point, at a distance ρ from the center of the cavity can be expressed by the following equation in case of cylindrical cavity.

$$d\sigma_\rho / d\rho + (\sigma_\rho - \sigma_t) / \rho = 0 \qquad 3.14$$

where, σ_ρ and σ_t denote the total principal stresses in the radial and circumferential directions respectively. Substituting q instead of $(\sigma_\rho - \sigma_t)$ in the Eqn.3.14, than it becomes

$$d\sigma_\rho = - q \, d\rho / \rho$$

substitute the value of q (Eqn.3.13),the above equation becomes

$$d\sigma_\rho = - (r^2 / a_s) (p'_m / (\rho^2 + (b_s / a_s) r^2) (d\rho / \rho)$$

assume, $y^2 = (b_s / a_s) r^2$, therefore, the above equation becomes

$$d\sigma_\rho = - p'_m (r^2 / a_s) (d\rho / \rho (\rho^2 + y^2))$$

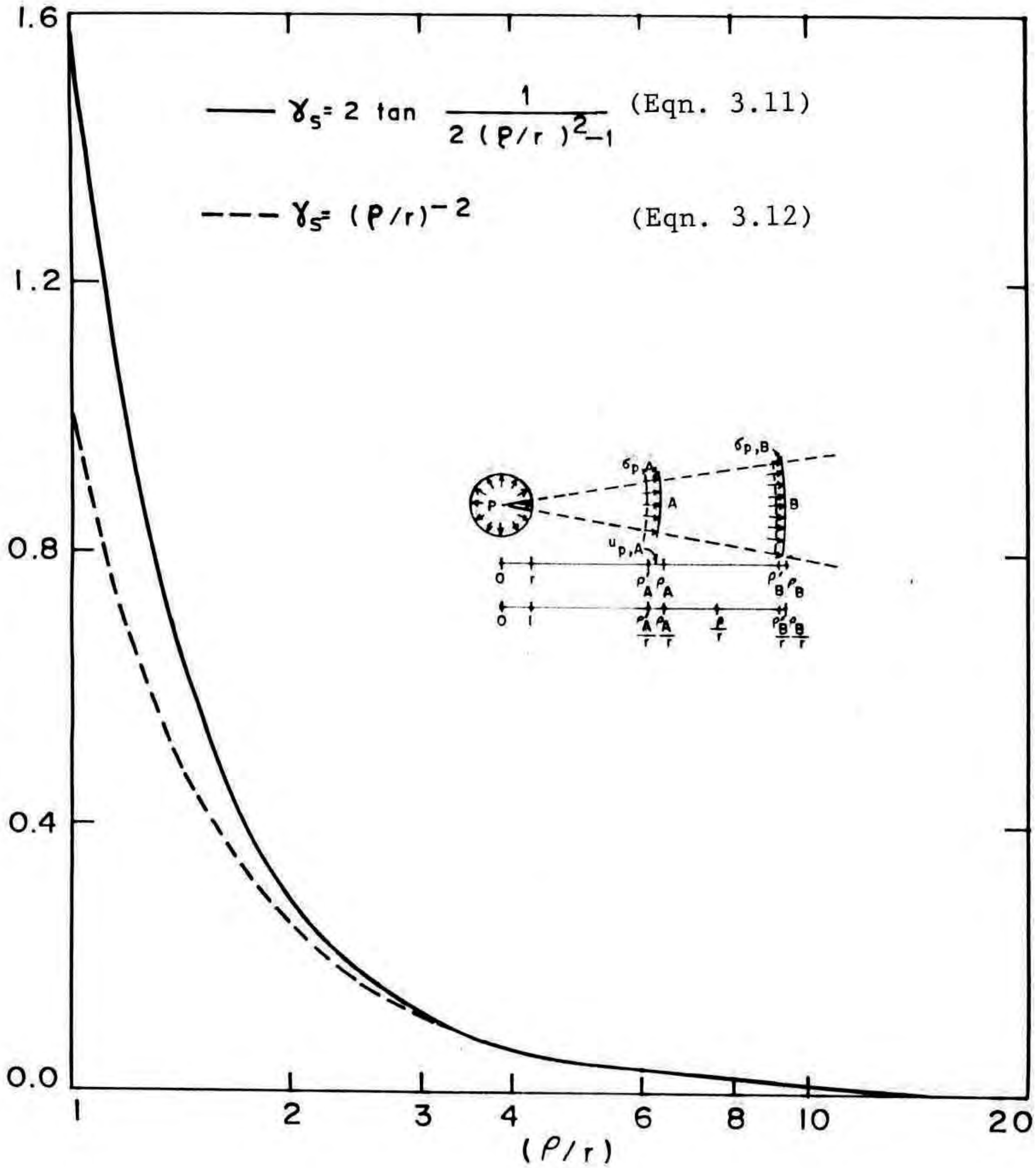


Fig. 3.21 Variation of shear strain with ρ/r (Ladanyi, 1963).

by integrating both sides of the above equation, the following equation can be written,

$$\sigma_\rho / p'_m = -(r^2 / a_s y^2) \ln \rho + (r^2 / a_s y^2) \ln \sqrt{\rho^2 + y^2} + C$$

putting the value of y^2 in the above equation, therefore

$$\sigma_\rho / p'_m = (1 / b_s) \ln \sqrt{1 + (b_s / a_s)(r / \rho)^2} + C \qquad 3.15$$

At the smallest value of shear strain (denoted as γ_i) the relationship between q/p'_m and γ_i is linear. γ_i is a point in the q/p'_m versus γ_s curve, which can be counted readily from the dial gauge during test. Considering 1 division which is equal to 0.002 mm, as upto this can be read easily from dial gauge. Height of the sample is 75 mm, therefore, the axial strain (ε) at this level is 0.000267 and the corresponding shear strain is 0.0004. So the value of γ_i can be taken as 0.0004. In fact this assumption implies that at $\gamma_s = 0.0004$, the soil is behaving like an elastic material and consequently, the Lame's solution for the expansion of a cylindrical cavity in an infinite elastic medium will be valid in this region. According to this solution, the expression for this region is

$$\sigma_\rho / p'_m = 1 + (1/2)(q / p'_m) \qquad 3.16$$

For stress strain curve q / p'_m versus γ_s , the initial modulus $\gamma_s / (q/p'_m)$ is equal to 'a_s' (Fig.3.20), infact 'a_s' is very near to the value of $\gamma_s / (q/p'_m)$ for the smallest value of shear strain (γ_i). For smalleat value of shear strain, $\gamma_i / (q /p'_m) = a_s$ or, $q / p'_m = \gamma_i / a_s$, putting this condition into

Eqn.3.16. At $\gamma_s = \gamma_i$

$$\sigma_\rho / p'_m = 1 + (\gamma_i / 2 a_s) \qquad 3.17$$

Using the relation $\gamma_i = (r / \rho)^2$ in Eqns. 3.15 and 3.17, the value of constant C can be obtained by the following equation,

$$C = 1 + (\gamma_i / 2 a_s) - (1 / b_s) \ln \sqrt{1 + (b_s / a_s) \gamma_i}$$

putting the value of C in Eqn.3.15, the expression for σ_ρ can be expressed as:

$$\sigma_\rho / p'_m = 1 + (\gamma_i / 2a_s) + (1/2b_s) \ln ((1 + (b_s / a_s) (r/ \rho)^2)$$

$$/(1 + (b_s / a_s) \gamma_i)) \qquad 3.18$$

Using the above expression (Eqn.3.18) the lateral resistance can be obtained at every point from the center of the cavity at radial direction when the cavity expand from a radius zero to 'r'. In the case of stone column, the column has a initial radius. Therefore, the expression is different from Eqn.3.18 and this can be established in a way described in the following. Let the finite radius is 'r_o', the radial deformation is 'δ' and the cavity expands from a finite radius 'r_o' to '$r_o + \delta$'. So a relationship should exist among ρ / r, r_o and δ . The relationship between ρ / r and u_ρ / ρ' for a cylindrical cavity, given by Ladanyi (1963) is,

$$\rho / r = \sqrt{(1 + u_\rho / \rho')^2 / ((1 + u_\rho / \rho')^2 - 1)} \qquad 3.19$$

where, ρ' = distance of a cylindrical surface from the center of the cavity before expansion; u_ρ = radial displacement of the same surface during the expansion of cavity from $r_o = 0$ to r. After expansion the distance of the considered surface, will be $\rho' + u_\rho$. In case of stone column foundation system, $\rho' = r_o$; $u_\rho = \delta$, hence, $\rho = r_o + \delta$. Therefore Eqn.3.19 becomes,

$$((r_o + \delta)/ r) = \sqrt{(1 + \delta /r_o)^2 / ((1 + \delta /r_o)^2 - 1)} \qquad 3.20$$

$$(r / (r_o + \delta))2 = 1 - (1 + \delta /r_o)^{-2} \qquad 3.21$$

At the surface of the expanded cavity i.e. at $\rho = r_o + \delta$, the expression for radial stress (σ_ρ) can be expressed by the following equation, obtained from Eqns. 3.18 and 3.20

$$\sigma_\rho/p'_m = 1 + \gamma_i/(2a_s) + (1/2b_s) \ln ((1 + (b_s/a_s)(r /(r_o+\delta))^2) / (1 + (b_s / a_s) \gamma_i)) \qquad 3.22$$

Putting the value of $r/(r_o+\delta)$ from Eqn.3.21, the Eqn.3.22 becomes.

$$\sigma_\rho/p'_m = 1 + \gamma_i/(2a_s) + (1/2b_s) \ln((1 + (b_s/a_s)(1 -(1 +\delta/r_o)^{-2}) / (1 + (b_s / a_s) \gamma_i)) \qquad 3.23$$

Here lateral stress in soil (σ_ρ) is represented as p and the precompression pressure (p'_m) is taken as γZ. Therefore, the Eqn.3.24 can be rewritten as

$$p_s = (1 + \gamma_i/(2a_s) + (1/2b_s) \ln((1 + (bs/as)(1 -(1 +\delta/r_o)^{-2}) /(1 + (b_s / a_s)\gamma_i)))\gamma Z \qquad 3.24$$

This is the general expression to determine the lateral resistances developed in the surrounding clay media due to the radial deformation of stone column. The qualitative diagrams of p_s $/\gamma Z$ versus δ/r_o and $p_s = f(\delta/r_o, Z)$ are shown in Figs.3.22 and 3.23.

(ii) Shaft friction along stone column-clay interface:

The frictional forces along stone column-clay interface due to vertical and radial deformation of column, have a pronounced effect on the load carrying capacity as well as settlement of stone columns. In cohesive soil the frictional forces are considerably greater than the bearing forces unless the base is enlarged by underreaming or by driving out a plug (Burland 1973). The load carrying capacity for frictional forces depends on the shear failure, takes place either stone column - clay interface or clay-clay interface. Therefore, the shear strength of clay media in which stone column is installed is the governing factor. In this analysis it is assumed that the shaft failure will occur into clay to clay interface for both plain and jacketed stone columns.

The mathematical expression to evalvate the frictional resistance is established here by a simplified manner. The first approximation rules, suggested by skempton (1966) for piles in clays is used here. Frictional force (P_f) on shaft is

$$P_f = \alpha \bar{S_u} A_s \qquad 3.25$$

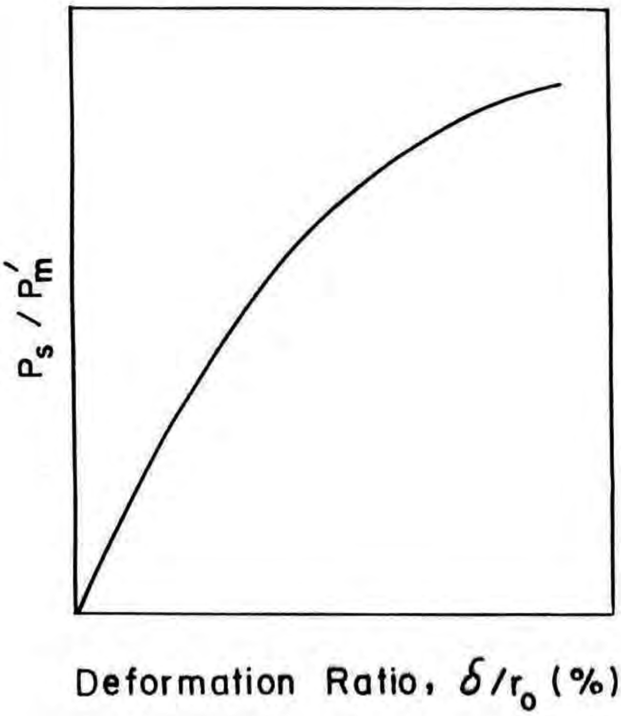


Fig. 3.22 Qualitative diagram of lateral resistance vs. deform.ratio of clay.

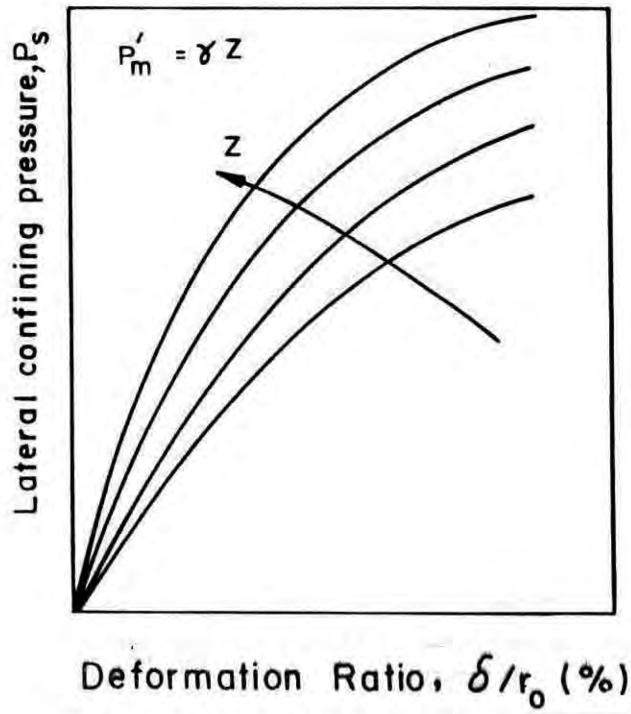


Fig. 3.23 Qualitative diagram of lateral resistance with deform. ratio and depth.

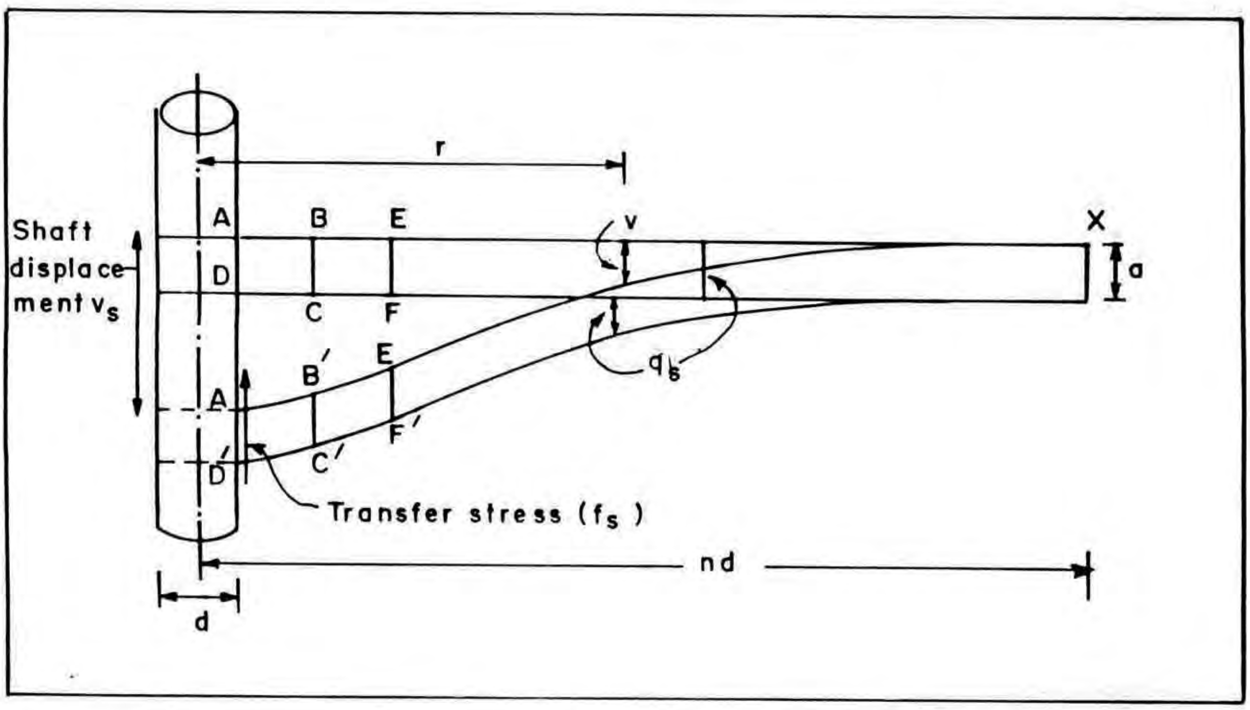


Fig. 3.24 Idealised vertical section through soil displaced by an element of a loaded friction pile (Cooke, 1974).

where, As = surface area of the shaft = π dL; d = diameter of the shaft; L = length of the shaft; α = adhesion factor; $\bar{Su}$=average undrained shear strength of clay.

Frictional force along the stone column-clay interface depends on the clay properties and its displacement. There are two load carrying components in pile, these are base bearing and skin friction. Whitaker and Cooke (1966) showed that these two components are mobilised at entirely different rates of settlement. The frictional resistance is developed rapidly and linearly and for full mobilization it requires different amount of settlement which depends on properties of clay and size of pile. So the settlement for which skin friction is mobilizied fully is to be established.

Cooke (1974) proposed a simple approach to depict the behaviour of pile due to the movement of soil adjacent to a small element of the shaft that is displaced downwand to a distance v (Fig.3.24). He suggested the following equation to calculate the displacement at any point 'r'

$$v = (d\ f_s\ /2G)\ \log_e(r/nd) \qquad 3.26$$

where, v = displacement at any point 'r' measured from the center of the shaft; f_s= shaft friction; d=diameter of the shaft; G = shear modulus of the soil; r=radial distance at any point the adjacent clay media, measured from the center line of the shaft after which the soil displacement became insignificant.

At shaft surface i.e. at $r = d/2$, the displacement 'v' is maximum i.e. $v = v_s$. Therefore,

$$v_s = (\bar{f_s}\ d /2G)\ Log_e (2n) \qquad 3.27$$

The frictional force R may be denoted as

$$R = \pi\ dL\bar{f_s} \qquad 3.28$$

where, $\bar{f_s}$ is the mean value of f_s for any elastic medium. For elastic medium $E = (1+2\nu)G$, here $\nu = 0.5$ for saturated clay media. Therefore,

$$E = 3G \qquad 3.29$$

From Eqns. 3.27, 3.28 and 3.29 the following equation can be deduced.

$$v_s = (3R/2\pi EL)\ Log_e(2n) \qquad 3.30$$

Eqn.3.30 can be rewritten as

$$v_s = (R/Ru)\ (3Ru/\ 2\pi EL)\ Log_e\ (2n) \qquad 3.31$$

where, R = ultimate frictional force = $p_f = \alpha\ As\ S\bar{u} = \alpha\pi\ dLS\bar{u}$. putting the value of R_u , Eqn.3.31 becomes,

$$v_s = (R/Ru)\ (d\ (3\alpha /2(E/S\bar{u}))\ Log_e\ (2n)\ ) \qquad 3.32$$

From Eqn.3.31 for particular values of α, $E/S\bar{u}$ and n, the settlement as a percentage of shaft diameter can be evaluated for which the skin friction is mobilised fully. The evaluation of these parameters (α, E, $S\bar{u}$) will be described at the next chapter.

Cooke and Price (1973) found experimentally that soil displacements became insignificant for values of n greater than 10. Assume, $\eta = ((3\ \alpha/(2(E/\bar{Su}))Log_e(2n)$, therefore,

$$v = \eta\ d\ (R/Ru) \quad 3.33$$

R = Ru , for full mobilization of skin friction. Therefore,

$$v_s = \eta\ d \quad 3.34$$

Eqn.3.34 shows that frictional resistance will be mobilised fully when settlement is η times of the shaft diameter. Eqn.3.25 gives the ultimate frictional force and Eqn.3.34 gives the displacement for which skin friction is mobilised fully. So from Eqns.3.25 and 3.34, the relationship between frictional force and settlement can be expressed by the following equations. Here it is assumed that the relationship between settlement and skin friction is linear upto full mobilisation. Therefore,

$$P_f = (\alpha\ As\ S\bar{u}\ v_s)\ /\ \eta\ d \quad \text{and} \quad \tau_s = (\alpha\ S\bar{u}\ v_s)/(\eta\ d)$$

Let, v_s = s = displacement, therefore,

$$\tau_s = (\alpha\ S\bar{u}\ /\ \eta\ d)\ s\ , \text{ when } s \leqslant \eta\ d \quad 3.35$$

$$\tau_s = \alpha\ S\bar{u} \qquad \text{when } s \geqslant \eta\ d \quad 3.36$$

$$P_f = \tau_s As \quad 3.37$$

by the Eqns. 3.35, 3.36 and 3.37 the frictional resistance along stone column-clay interface can be evaluated. The qualitative diagrams of skin friction versus displacement and frictional force versus displacement are shown in Figs.3.25 and 3.26.

3.7 ANALYSIS OF BEHAVIOUR OF STONE COLUMN

The overall behaviour of a single stone column foundation system under quasi static loading are presented here. The behaviour of the stone column should be dependent on the behaviour of the constituent materials. Due to downward vertical load at the top of column, it will compress longitudinally and expand radially and lateral confinement is developed from the reinforcing element and the surrounding clay media to resist the bulging of the column. The frictional resistance along the surface of column increases the load carrying capacity and resist settlement.

3.7.1 Behaviour of Aggregates Column

In stone column foundation system, aggregates column is not only a load transfering media but also a load carrying media. Due to the downward vertical load, aggregates column is compressed longitudinally and expanded laterally. Loads coming from superstructures is transfered to the rigid base through the aggregates. The compression and expansion of aggregates column depends on the physical and mechanical properties of aggregates. Here it is assumed that there is no volume change of aggregates column due to the application of load on stone column. There should be a certain limit upto which the column can expand laterally. Beyond this limit the aggregates column will fail. This behaviour of aggregates can be established by the newly

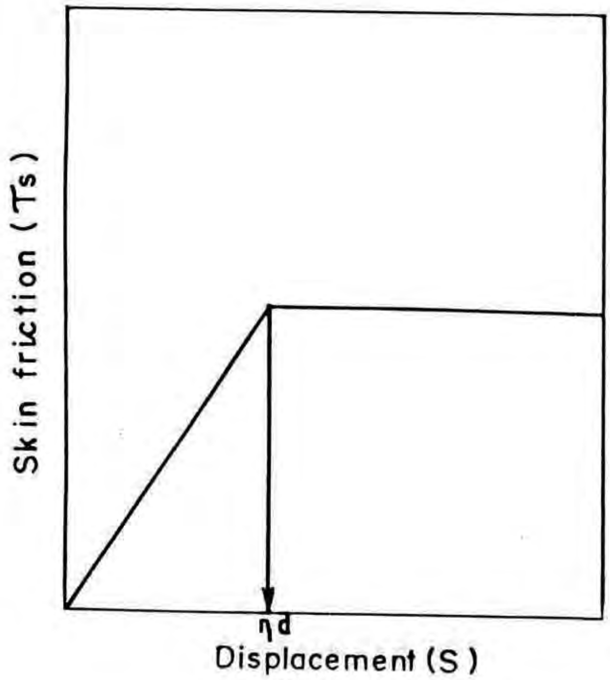


Fig. 3.25 Qualitative diagram of skin friction vs. displacement for stone column.

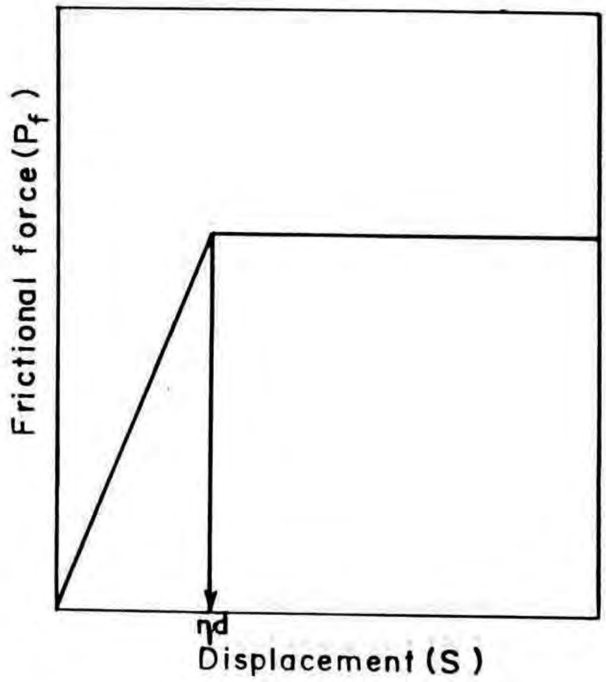


Fig. 3.26 Qualitative diagram of frictional force vs. displacement for stone column.

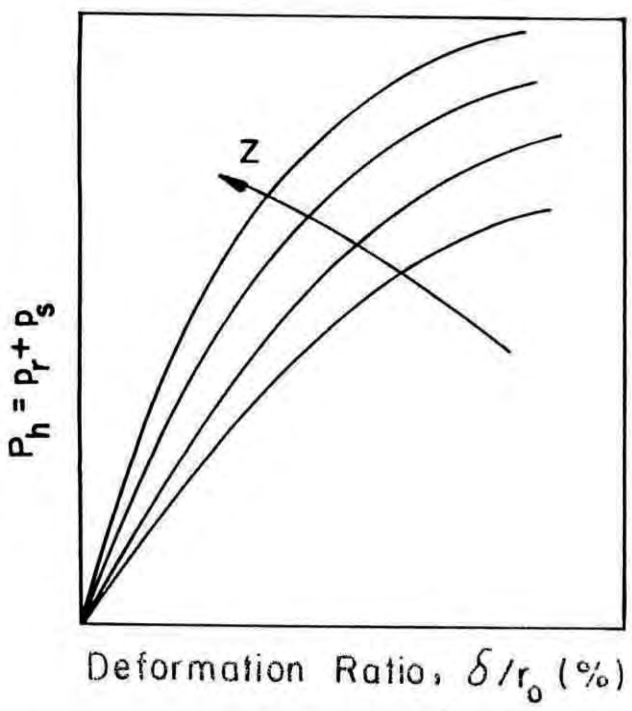


Fig. 3.27 Qualitative diagram of total lateral confinement vs. deform. ratio with depth.

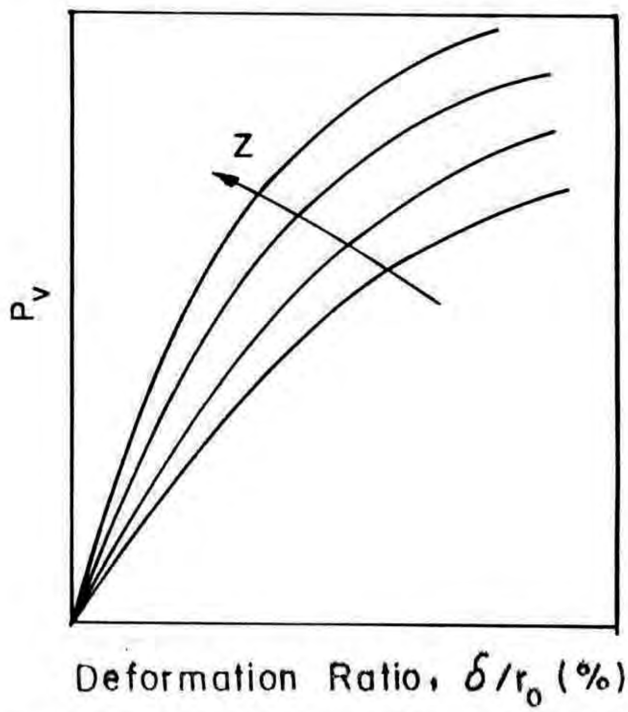


Fig. 3.28 Qualitative diagram of vertical load vs. deform. ratio with depth.

envisaged suction triaxial test and theoretically by the Eqn.3.4, which is based on test results.

3.7.2 Behaviour of Reinforcing Element

Reinforcing element around the aggregates column is used to provide seperation, filtration function and lateral confinement. It prevents the loss of aggregates during installation into the clay media. When aggregates column is going to expand radially, reinforcing element resists it to expand according to its stress strain behaviour, explain at Art.3.6.2. Here it is assumed that the mobilised radial deformation is same for aggregates column and reinforcing element. The lateral confinement by reinforcing element can be evaluated by the Eqn.3.7. This lateral confinement will increase the load carrying capacity of stone column providing additional lateral confinement with clay media.

3.7.3 Behaviour of Surrounding Clay Media

Behaviour of the surrounding clay media has pronounced influence on the load carrying capacity of stone column. The clay media is assumed to be semi-infinite, homogeneous and isotropic. Stress is developed due to the radial deformation of aggregates column under undrained condition. The surrounding clay media provides lateral confinement thorughout the column length and prevents bulging of stone column. Skin friction along stone column-clay interface increases the load carrying capacity and reduces deformation of column. Since deformation is a major design criteria, the lateral confinement and frictional resistance of clay media dictates the vertical load carrying capacity of stone column. The radial deformation of clay media is equal to the radial deformation of aggreggates column and reinforcing element. The deformation of the stone column varies along the length due to the different overburden pressures and skin frictions. The deformation is maximum at the top of stone column and minimum at the bottom. The mobilised lateral confinement of the surrounding clay media can be evaluated by the Eqn.3.24. The frictional resistance can be evaluated by the Eqns. 3.35, 3.36 and 3.37. The overall behaviour of the surrounding clay media depends on its physical and mechanical properties.

3.8 DESIGN OF STONE COLUMN

The newly developed design methods of plain and jacketed stone columns are presented here. These are based on the mobilisation of mechanical resistances in the constituent materials under compatible deformation conditions. Here constituent materials are aggregates and clay media for plain stone column and aggregates, reinforcing element and clay media for jacketed stone column. The load carrying capacity of a single stone column, its settlement, load-settlement curve and design steps are presented here.

3.8.1 Load Carrying Capacity

In this study nonlinear behaviour of the constituent materials are considered to evaluate the load carrying capacity of stone column. Here load carrying capacity means the amount of load that can be carried by a stone column for a limiting settlement. In this method deformation compatibility of the constituent materials is considered, on which vertical load carrying capacity depends. The mobilized confining pressure (p_s) from the surrounding clay media can be obtained by the Eqn.3.24. The values of p_s as a function of δ/r_o and Z, for a particular clay media, can be obtained from this equation. The lateral resistance (p_r) given by the reinforcing element around the aggregates can be obtained by the Eqn.3.9. The values of p_r as a function of δ/r_o for particular reinforcing element and 'r_o' can be obtained from this equation. In case of jacketed stone column this p_r will provide additional lateral confinement over that from the clay media, which will increase the load carrying capacity of stone column. Therefore, in case of plain stone column the lateral confinement $p_h = p_s$ and for jacketed stone column the lateral confinement $p_h = p_s + p_r$ (Fig.3.27). Now to predict the vertical load a differential strip 'dz' of stone column at a depth 'Z' from the ground level is considered. Initial and final stress conditions of this strip are shown in Fig.3.5. From this stress condition, the following equation can be written.

$$K = (p_h + \gamma Z) / (Pv + \gamma Z) \qquad 3.38$$

where, p_h = total lateral confinement from constituent materials; γ = unite weight of soil; Z = depth of stone column from the ground level; Pv = vertical load applied on the top of

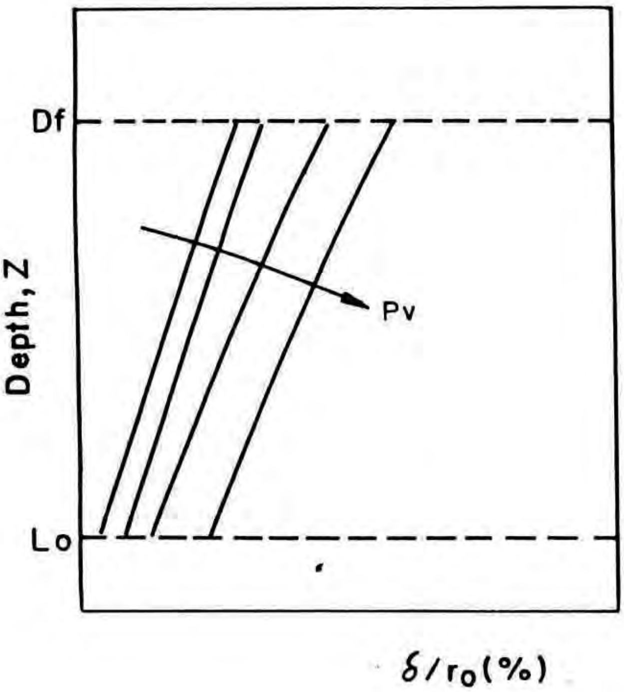


Fig. 3.29 Qualitative diagram of variation of deform. ratio with depth at different vertical load.

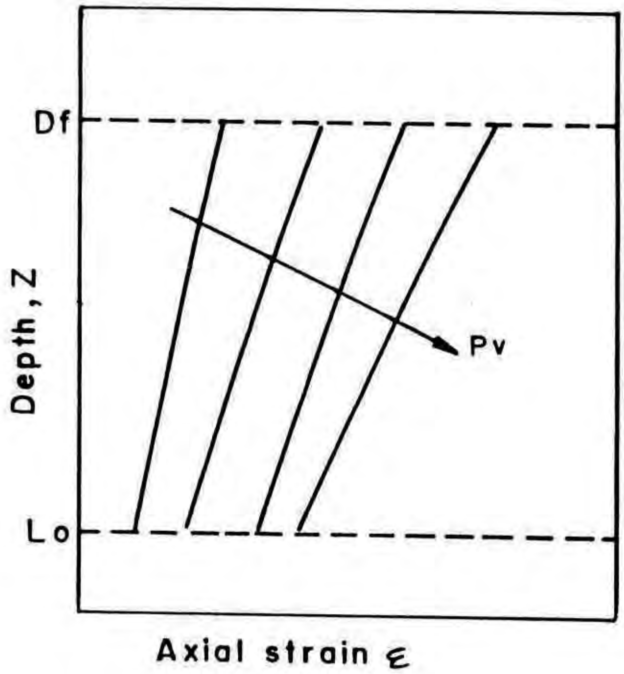


Fig. 3.30 Qualitative diagram of variation of axial strain with depth at different vertical load.

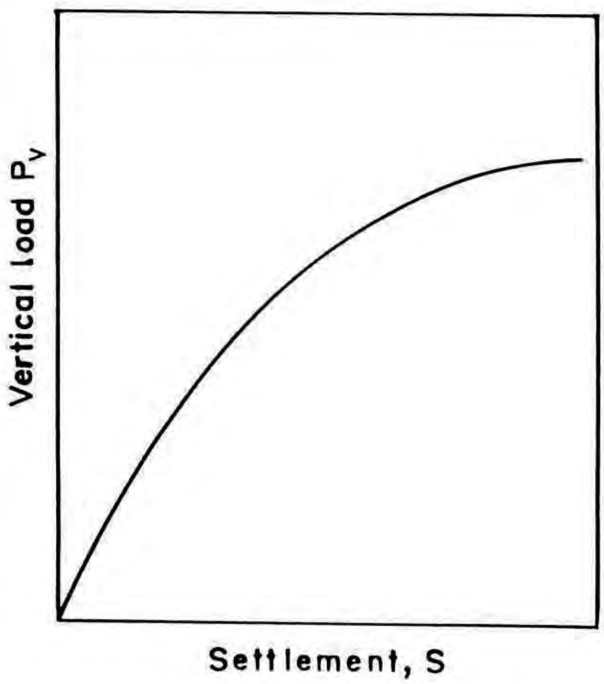


Fig. 3.31 Qualitative diagram of vertical load vs. settlement of stone column.

column. Eqn. 3.38 can be rewritten as

$$Pv = (p_h / K) + ((1 - K) / K) \gamma Z \qquad 3.39$$

Using Eqn.3.39 vertical load carrying capacity of stone column for different K i.e. for different deformation ratio at different depth (Z) can be determined. The values of Pv as a function of δ/r_o and Z can be plotted. Qualitative diagram is shown in Fig.3.28. At different depths, the curve Pv versus δ/r_o are different. From such curve the value of Pv for a stone column can be determined at different depth of column without considering skin friction. The curve (Pv versus δ/r_o) for $Z = D_f$ is the governing curve by which Pv can be determined upto the failure level. From such curve, it will be found that if the value of Pv is greater than the ultimate value of Pv at $Z = D_f$, the radial deformation at the top of the column will exceed the tolerable deformation level. So it is clear that the vertical load greater than the highest value of Pv at $Z = D_f$ can not be carried by a stone column.

In the Pv = f(δ/r_o, Z) curve skin friction along stone column-clay interface is not considered. From this curve for particular Pv, the deformation ratio (δ/r_o) as well as axial strain (ε) at different depth (Z) can be obtained. Qualitative diagrams are shown in Figs. 329 and 3.30. From the curve (Fig.3.30) for particular vertical load (Pv), the settlement can be calculated by the methods described in the next section. It gives the settlement for particular Pv in which skin friction is zero. Now the column length may be divided into some equal strips and the settlement can be calculated seperately for each strip by taking the linear variation of axial deformation. For each settlement τ_s and P_f may be calculated using Eqns. 3.35, 3.36 and 3.37, after checking whether skin friction is mobilized fully or not by Eqn.3.34. This skin friction will give different values of Pv instead of same value at different depth by the following relationships.

$$\begin{aligned} Pvi &= Pv \\ Pvi+1 &= Pvi - P_{fi} \\ &\cdots\cdots\cdots\cdots \\ Pvn+1 &= Pvn - P_{fn} \end{aligned} \qquad 3.40$$

where, i = 1 to n , n is the total number of strips considered in the column. From these values of Pv, evaluate δ/r_o i.e. ε (use Eqn.3.2) at different depths from curves Pv = f(Z, δ/r_o) (Fig.3.28). By these axial strains (ε), calculate the total settlement (S) for the load Pv using Simpson's rule. Now compare this settlement with the previous one. If it differs with the previous one than calculate new sets of settlements (s), R/Ru, τ_s, P_f, Pv, δ / r_o , ε , and total settlement (S) and continue this process untill the settlement (S) becomes very close to the previous one. In this way vertical load carrying capacity (Pv) can be evaluated for plain and jacketed stone columns.

3.8.2 Settlement Anlysis

In this study a realistic condition for determining settlement of stone column is considered. Nonlinear behaviour, plane strain condition and compatibility of deformation of the constituent materials are considered here. Here surcharge effect from soil is not considered. It is also assumed that there is no volume change in the aggregates column under deformation. So the axial strain (ε) can be calculated from δ / r_0 using Eqn.3.2. ε depends on the depth of column (Z) and the vertical load (Pv). The qualitative diagram ε = f(Pv, Z) is shown in Fig.3.30. Settlement (S) is the summation of axial strain (ε) for the whole length of the stone column. Therefore, settlement can be determined using the following equation.

$$S = \int_{D_f}^{Lo + D_f} \varepsilon \, dz \qquad 3.41$$

Here, settlement can be calculated by two ways. For each strip of stone column, settlement (s) is calculated only by taking the average axial strain of the top and bottom of each strip i.e. s = ($\varepsilon_i + \varepsilon_{i+1}$) / 2 . To calculate the total settlement (S) the Simpson's rule is used. Therefore,

$$S = d/3 \; (\varepsilon_i + 4 \, \varepsilon_{i+1} + \qquad + 4\varepsilon_n + \varepsilon_{n+1}) \qquad 3.42$$

where, S = total settlement for load Pv; d = length of each equal strip; ε_i = axial strain at different depth.

3.8.3 Load-Settlement Curve

In stone column design load-settlement curve is the final form of representation, from which the load carrying capacity of stone column can be obtained. The settlements for vertical loads can be calculated (Arts.3.8.1 and 3.8.2) for both without skin friction and skin friction considerations. From these vertical loads (Pv) and settlements (S), the load - settlement curves can be drawn for both conditions. Qualitative diagram is shown in Fig.3.31. Now from this curve, for the allowable settlement the load carrying capacity of stone column can be obtained easily. If the settlement is higher than the allowable settlement and there is no sharp break point in the load - settlement curve, the load carrying capacity can be obtained by the proposed method of Terzaghi's (1942) for pile. This method is used in this study in case of stone column. Vertical load carrying capacity of stone column can be obtained from Pv versus S curve corresponding to the settlement, which is 10% of the stone column diameter.

3.8.4 Design Steps

To determine the vertical load carrying capacity of a single stone column, the following steps are recommended.

i) Find the values of L_o , $2r_o$ and D_f according to the specification of stone column (Fig.3.1).

ii) Determine the properties of aggregates, used in the stone column by the relationship K versus δ / r_o with the help of the newly envisaged suction triaxial test (Eqn.3.4 and Fig.3.11).

iii) Determine the properties of reinforcing element used around the aggregates column by the relationship p_r versus δ/r_o (Eqn.3.4 and Fig.3.11).

iv) Determine the properties of clay media (E, $\bar{Su}$, γ_i, a_s, b_s) at which stone column is installed. Also find the value of adhesion factor (α) and value of n which is taken as 10 in this study.

v) Determine the lateral confinement (p_s) given by the surrounding clay media for different δ/r_o(Eqn.3.24 and Fig.3.23).

vi) Determine the total lateral resistance (p_h). For plain stone column $p_h = p_s$. For jacketed stone column $p_h = p_s + p_r$ (Fig.3.27).

vii) Determine the vertical load (Pv) at different depth (Z) for δ/r_o without considering skin friction (Eqn.3.39, Fig.3.27).

viii) From the curve Pv = f(δ/r_o ,Z), find δ/r_o for different Pv. Highest value of Pv is the peck value for the curve at $Z = D_f$. Convert δ/r_o into ε by the Eqn.3.2.

ix) Draw the curves Z = f(Pv, δ/r_o) and Z = f(Pv, ε) (Figs. 3.29 and 3.30).

x) Evaluate the total settlements (S) for different vertical load (Pv) using the Eqn.3.42.

xi) Draw vertical load (Pv) versus settlement (S) curve in which skin friction is not considered.

xii) Calculate the value of from the relation $\eta = (3\alpha)/(2(E/\bar{Su}))\ \mathrm{Log}_e(2n)$, which gives the settlement in percentage of column diameter for which skin friction is mobilized fully.

xiii) Divide the total length of stone column into some equal strips.

xiv) Calculate the settlement seperately for each strip by taking the average axial strain for particular Pv. Axial strain is obtained from curve Pv = f(δ/r_o , Z) (Fig.3.28).

xv) Check whether the skin friction is mobilized fully at each strip using the relationship R/Ru = s / (η d).

xvi) Determine the values of mobilized skin friction by Eqns.3.35 and 3.36.

xvii) Determine the frictional force (P_f) by Eqn.3.37 for each strip seperately.

xviii) Determine the vertical load (Pv) that actually comes on the column at different depth after considering skin friction by the Eqn.3.40. For particular Pv, it will give different Pv at different depth.

xix) Now find the values of ε for these different Pv at the respective depths using diagram Pv = f(δ/r_o,Z) (Fig.3.28) and Eqn. 3.2.

xx) Calculate the total settlement (S) for the whole length of the column for these axial strain (step xix) by the Eqn.3.42.

xxi) Compare the newly calculated settlement (S) with the previous one. If it differs from the previous one than steps xiv to xx will be continued untill the difference of the consequative settlements become negligible.

xxii) Plot load (Pv) - settlement (S) curve for the stone column in which skin friction is considered.

xxiii) Evaluate the vertical load carrying capacity (Pv) from load - settlement curve (step xi and xxii) by the method described in the section 3.8.3.

3.9 DISCUSSION

In the development of new design methods, some simplifying assumptions are made and the problems encountered are discussed in the followings.

i) In this analysis it is assumed that there is smooth rigid base at the top of stone column, as a result maximum radial deformation occurs at the top of column. In practice this conditions will be difficult to achieve.

ii) In this studies stone aggregates of almost uniform size (9 mm to 12 mm) are used and it is assumed that the volume of the stone aggregates remain constant under loading and deformation. This consideration may not be true because this is valid only for uniform sized smooth spheres.

iii) The plane strain condition of deformation of the clay media under undrained conditions is considered here, but in reality a three dimensional strain condition will prevail.

iv) In these analyses, it is observed that the K versus δ / r_o relations have a pronounced effect on the load carryiing capacity of stone column. Therefore, it is required to develop the K versus δ / r_o relation accurately using equipments which will give high degree of accuracy.

v) For accurate evaluation of frictional forces, developed along stone column clay interface, realistic numerical methods should be used. The method presented here is based on approximate rules and simplified assumptions.

CHAPTER FOUR

DESIGN PARAMETERS AND THEIR CHARACTERISATION

4.1 GENERAL

The development of appropriate constitutive relations and relevant design parameters for all the constituent materials forming stone column foundation systems are described in this chapter. A number of pieces of new test apparatuses were fabricated to evaluate design data. These include, a suction triaxial test for aggregates and conventional techniques for characterising the other constituting materials. Hyperbolic representation of stress strain behaviour for all the three materials, stone aggregates, jacketing reinforcing materials and clay soils are used, as these are found to be of reasonable degree of accuracy. In these studies nearly uniform sized stone aggregates, geotextile and geogrid jacketing elements and soft clay media are taken into consideration. Design parameters were established for these materials which are described in the following sections.

4.2 AGGREGATES PARAMETERS

In this study it is assumed that the aggregates should possess sufficient interlocking characteristics for efficient transmission of the imposed loading. As described earlier in Art.3.6.1, the aggregates properties are depicted through the relationship K versus δ / r_o . This relation is established by using test data obtained from the newly developed suction triaxial test. The test apparatuses and methodologies including newly developed suction triaxial test are described here.

4.2.1 Suction Triaxial Test

The suction triaxial test was mainly developed for performing triaxial test on coarse aggregates with specimen size of 100 in mm diameter and 200 mm in length. A conventional triaxial test for such a large specimen is very expensive and not readily available in most geotechnical laboratories. Therefore, the vacuum technique was envisaged in which suction is applied in the specimen so that the atmospheric pressure acts as the confining pressure on the enclosing rubber membrane. A maximum of upto eighty percent of the atmospheric pressure can be applied and intermediate values may be achieved by introducing leakage through the input line. A detailed description of the test apparatus and the validity of the test which is established through its comparison with conventional triaxial test is described in the following sections.

4.2.2 Development of Test Apparatus

The suction triaxial test appratus developed as part of this study comprised of components which is described in the following. These are, (i) Rubber membrane, (ii) Specimen mold, (iii) Base plate, (iv) Rubber O-ring, (v) Wooden platform, (v) Suction pump, (vii) Suction measuring dial, (viii) Strain measuring dial and (ix) Loading device. The suction triaxial test apparatuses are presented in Plates 4.1 and 4.2 and Fig.4.1.

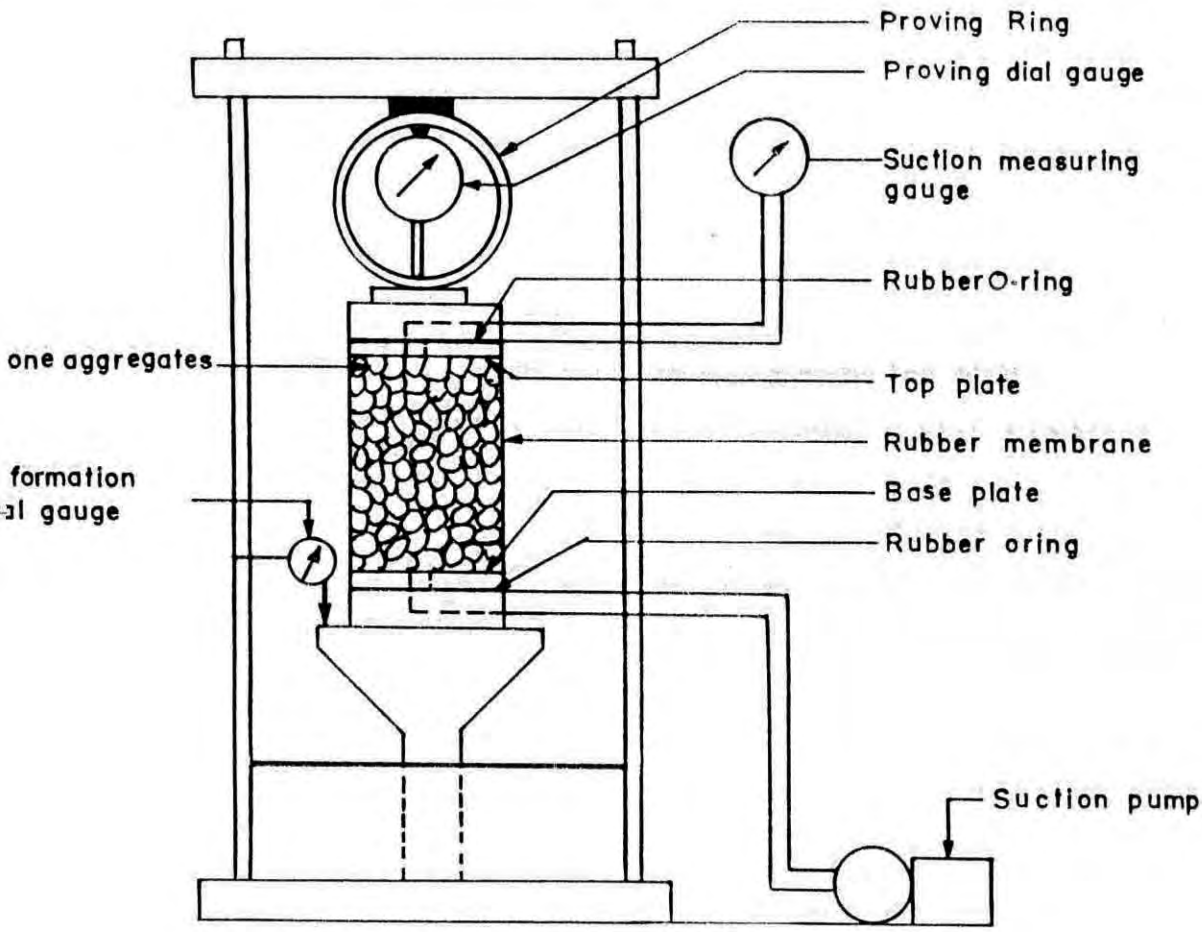
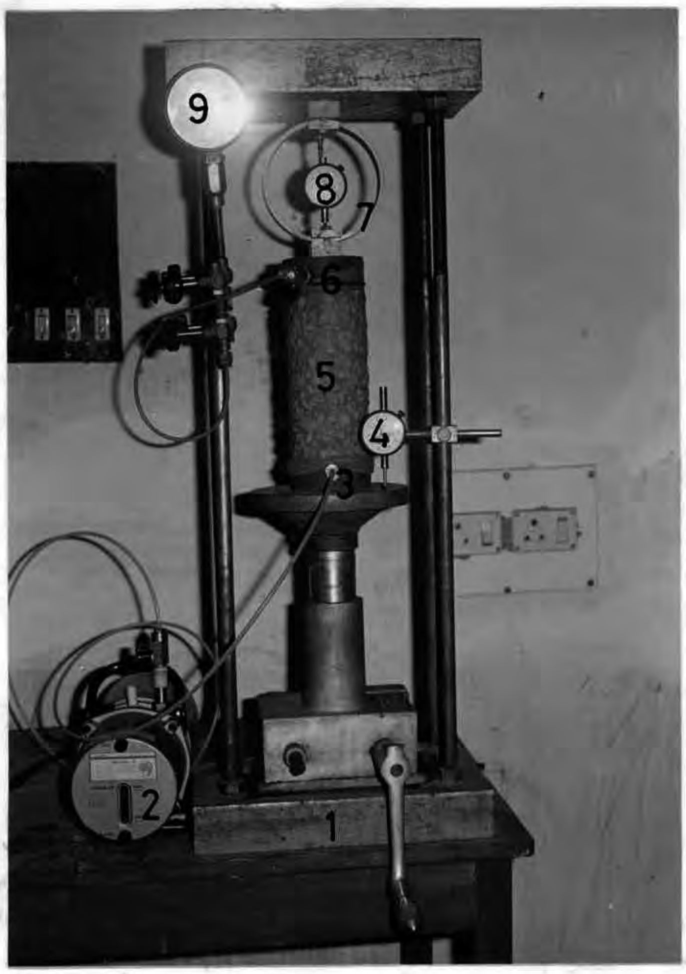


Fig. 4.1 Schematic diagram of suction triaxial test apparatus.

1. Specimen mold
2. Wooden platform
3. Base plate
4. Rubber membrane
5. Rubber O-ring
6. Stone aggregates
7. Suction measuring dial
8. Suction pump

Plate: 4.1 Photograph of different components required to perform suction triaxial test.



1. Loading platform
2. Suction pump
3. Base plate
4. Strain dial gauge
5. Stone aggregates specimen
6. Rubber O-ring
7. Proving ring
8. Load dial gauge
9. Suction measuring dial
10. Suction control device

Plate: 4.2 Photograph of suction triaxial test apparatus.

(i) Rubber membrane: A larger than normal sized rubber membrane is required to accomodate the large sized sample. Rubber membrane of 100 mm diameter and 250 mm long was procured from abroad which is not available in most of the geotechnical engineering laboratories. Due to the sharp angularity of aggregates, the right type of membrane is to be used which does not allow any leakage at the time of applying suction and loading. At the same time it should be ensured that the membrane should not provide any extra confinement from itself while testing.

(ii) Specimen mold: In this test special type of mold is required for preparation of specimens. The required size of mold is 100 mm in diameter and 225 in mm height. The extra 25 mm above the 200 mm is required to place the top and bottom base plates. The mold is split cylindrical type. To hold the membrane uniformly throughout the interior surface of the mold, suction is applied through a mechanism similar to that used in conventional triaxial test. In this study PVC pipe is used to make the mold. It is divide into two equal parts and they are held together using O-rings at the top and bottom with grease sealant at the interface.

(iii) Base plate: Two base plates are required in this apparatus, with one at the top and the other at the bottom of mold. Suction is applied through one, while the other is used to monitor its magnitude. They also ensure the uniform distribution of load on the sample applied through the loading frame. These plates are made from cast iron having 100 mm diameter and 25 mm thickness . There is an opening of nominal diameter at the centre of each plate, which is carried upto the periphery to apply or measure suction within the sample. A vaccum pump is used for application of suction which is measured using a suction dial. These are connected to the specimen through the port holes in the base plates.

(iv) Rubber O-ring: At least two rubber O-rings are used for sealing the specimen with the base plates. The diameter of the O-rings should be less than the base plate so that it can attach the rubber membranes with the upper and lower base plates tightly to prevent leakage.

(v) Wooden platform: In this test apparatus, a platform is required around the bottom base plate on which the mold rests. There is an opening to accomodate the pipe which is attached to the suction pump. For this study, an wooden platform is used, which can be made using any other convenient material.

(vi) Suction pump: A suction pump is required to apply the desired amount of suction in the aggregates sample. There must be a device attached with the pump by which the amount of suction applied on the sample can be controlled.

(vii) Suction measuring dial: Since the applied suctions are different, a suction measuring device is required to monitor this. A conventional suction measuring dial is attached to the upper base plate by a pipe with air tight sealings at all junctions. The suction measuring scale in this dial is in FPS and SI units. A suction control device is also attached to it to acheive the desired suction.

(viii) Strain measuring dial: To measure the axial deformation of the aggregates sample a strain dial gauge is required. It should be attached at any convenient location to readily measure the deformation of sample due to its vertical movement. The attachment of strain dial is similar to that of conventional triaxial test.

(ix) Loading device: The loading system of the suction triaxial test is similar to that of a conventional triaxial test. In this system the load is applied on the sample through a calibrated proving ring. The upper portion of the loading frame is fixed and the lower plate can be moved upwards at required deformation rates.

4.2.3 Comparison of Suction and Conventional Triaxial Test

The validity of using suction triaxial test instead of widely accepted conventional triaxial test, to characterise granular material is established here. To establish the applicability of suction triaxial test , a test scheme was run by the conventional triaxial test apparatus. In this test scheme suction (confining pressure) and covnevtional (water) confining pressure are applied on the same sample seperately. A sand sample of 38 mm diameter and 88 mm length is tested in both cases. The amount of pressure which is applied for the tests is 62 kPa.

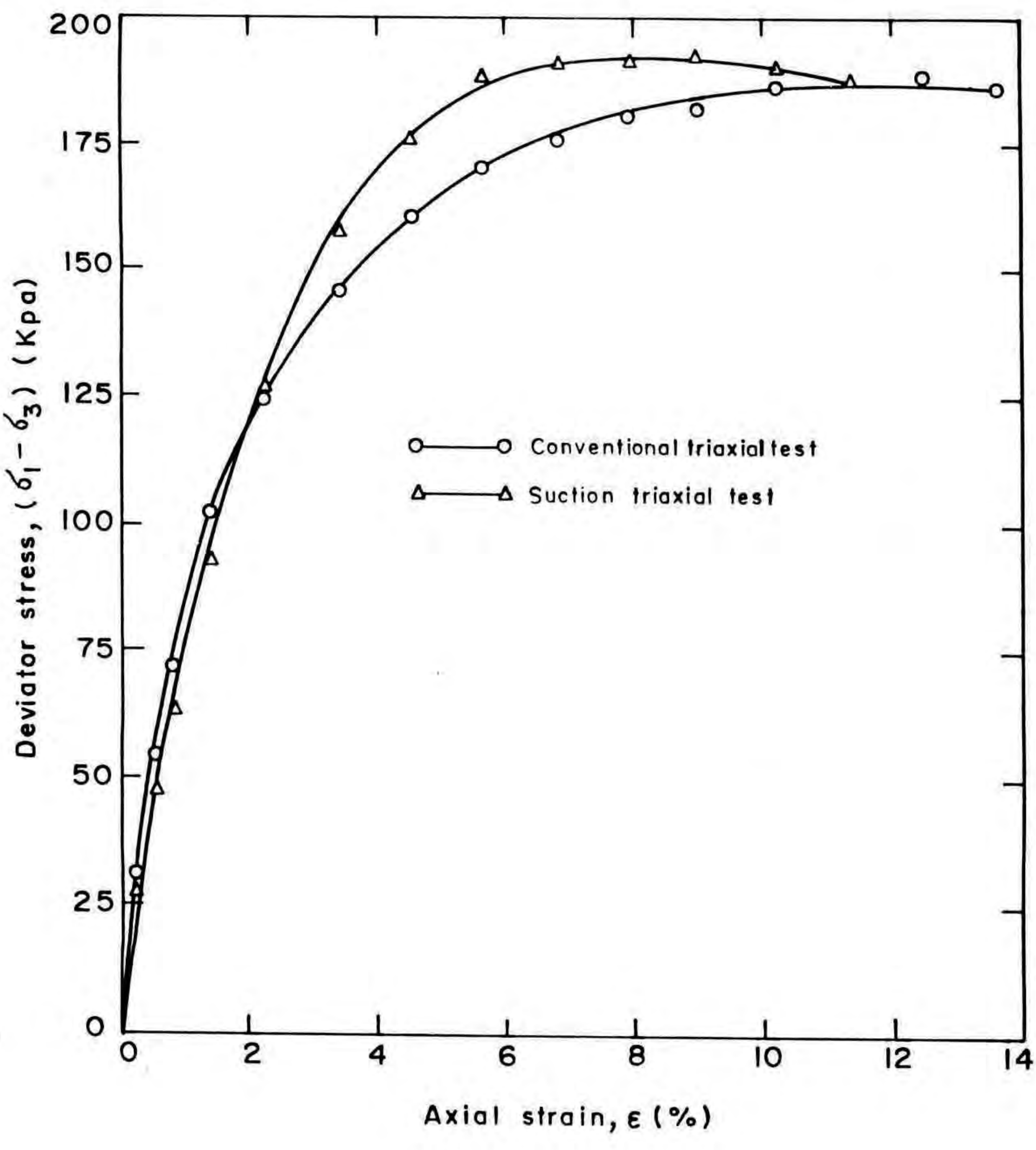


Fig. 4.2 Stress - strain behaviour of sand by conventional and suction triaxial test.

Three samples are tested for both cases to achieve reproducable result. From these tests results the deviator stress versus axial strain behaviour of sand sample for both cases are established which are presented in Tables. 4.1 and 4.2 and Fig.4.2. These figures show that the variation in stress strain diagram for the both cases is very negligible. Therfore, it can be concluded that the newly developed suction triaxial test can be used satisfactorily to characterise granular material instead of conventional triaxial test.

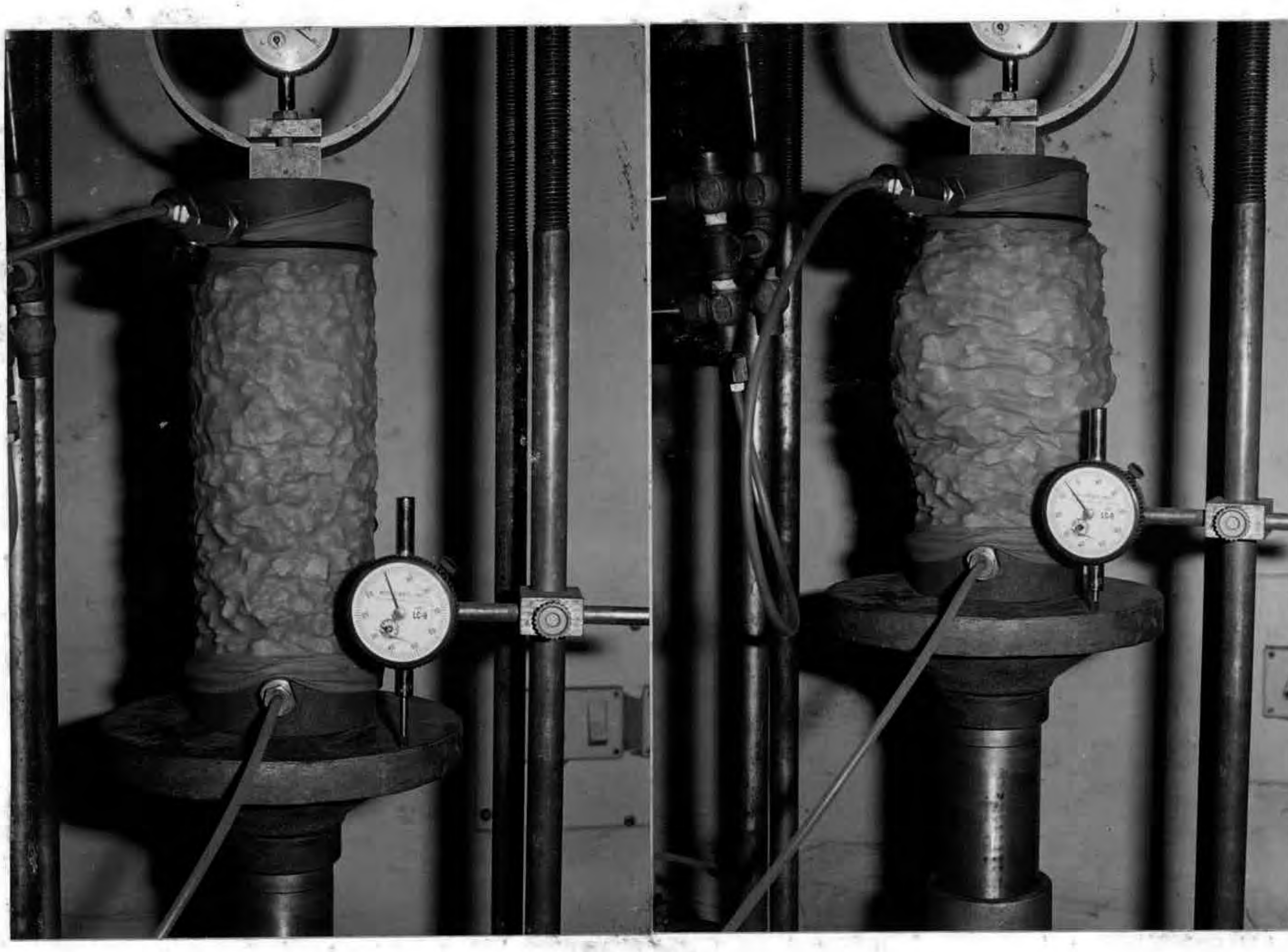
4.2.4 Test Procedures

The test procedures of suction triaxial test is similar to that of conventional triaxial test. To perform this test the strain control approach is used. The test has been found somewhat sensitive to the rate of strain, but a strain rate between 1/2 and 2 percent per minute appears to yield satisfactory results (Bowles, 1970). Negative confining pressure is applied by suction pump into the sample upto the desired magnitude. The stone aggregates sample before and after loading conditions are shown in Plate 4.3. The following procedures are recommended to perform the suction triaxial test for stone aggregates successfully.

i) Prepare the aggregates sample using the conventional equipments.

ii) Weigh the required aggregates to prepare the sample so that the sample density can be uniquely maintained.

iii) Place the bottom base plate at the middle of the loading



(a) Before loading (b) After loading

Plate: 4.3 Photograph of stone aggregates specimen on suction triaxial test before and after loading.

device. There is a pipe connection at each base plate through which suction can be applied and monitored later on.

iv) Surround the bottom base plate by the wooden platform.

v) Place the mold on the wooden platform around the bottom base plate. The interior surface of the mold is covered by the rubber membrane which is folded over the top and bottom edges. Silicone grease along the split surfaces are used for efficient vacuum sealing. The split mold is held tightly together by the rubber O-rings, are at the top and the other at the bottom.

vi) Small amount of suction is applied to hold the membrane uniform throughout the whole length of the interior surface of mold.

vii) Place the aggregates in the mold and use a temper to maintain the desired geometry and density.

viii) Determine the proper height and diameter of the specimen by standard measurements.

ix) Place the top base plate having suction measuring dial at the top of mold. Roll the membrane off the mold into the top platen and seal it with the help of rubber O-ring that hold the mold together. Remove the wooden platform and roll the membrane off the mold into the bottom platen and seal it with the help of the other rubber O-ring.

x) Suction is applied into the specimen, using the suction pump, upto the desired magnitude, throgh the bottom platen while the mold is removed.

xi) Examine the membrane for holes and leaks. If any are found, the sample must be remade using a new membrane.

xii) Attach a deformation dial gauge to the machine so that the sample deformation can be measured. Set the dial gauge at zero, than manually compress and release the dial plunger several times and observe the zero reading.

xiii) The sample is set to the loading frame.

xiv) Check the deformation dial gauge, load dial gauge and magnitude of applied vacuum for initial readings.

xv) Load the sample through the compression machine manually at the earlier mentioned strain rate and take simultaneous load and deformation readings upto the failure or desired strain level.

xvi) Release the vacuum and remove the sample from loading frame.

xvii) Prepare the new specimen and repeat this test at different suctions as required. At least three specimens should be tested for a particular suction.

4.2.5 Specification of Aggregates

Large sized aggregates sample can be tested by the newly developed suction triaxial test. Here stone aggregates are used to evaluate its mechanical properties. Since the basic assumption for analysis and design of stone column is that their is no volume change during loading and deformation, nearly uniform sized stone aggregates are used to meet with this assumption. In this study suction triaxial test is performed for the crushed stone aggregates passing 12 mm sieve and retained on 9 mm sieve.

4.2.6 Evaluation of Design Parameters

Design parameters of aggregates in case of stone column foundation system can be evaluated by the suction triaxial test. The test data is found in the form of deviator stress ($\sigma_1 - \sigma_3$) versus axial strain (ε) for different suction (σ_3). From this axial strain, dimensionless deformation ratio (δ/r_o) can be obtained using Eqn.3.2. From these test data the design parameters 'a_a' and 'b_a' can be evaluated, as described in Art.3.6.1.1. Using design parameters 'a_a' and 'b_a', the relationship between K and δ/r_o can be obtained by the Eqn.3.4 and as described in Art.3.6.1.2. The relationship K versus δ/r_o can be obtained from Eqn.3.4 and also from test results. From this relationship K value can be obtained for any dimensionless deformation ratio.

Another design parameter of aggregates is the angle of internal friction (ϕ') which can be evaluated from the test results obtained from suction triaxial test. The value ϕ' can be achieved by the conventional procedure. In this case Mohr's circle is established and the ϕ' value is obtained from the failure envelope.

4.2.7 Values of Design Parameters

In these studies stone aggregates of nearly uniform size (9 mm to 12 mm) are used. The test is performed for three values of suctions, these are 17 kPa, 34 kPa and 69 kPa. The test data in the form of deviator stress ($\sigma_1 - \sigma_3$) and axial strain (ε) for different suctions are shown in Tables.4.3 and 4.4 and

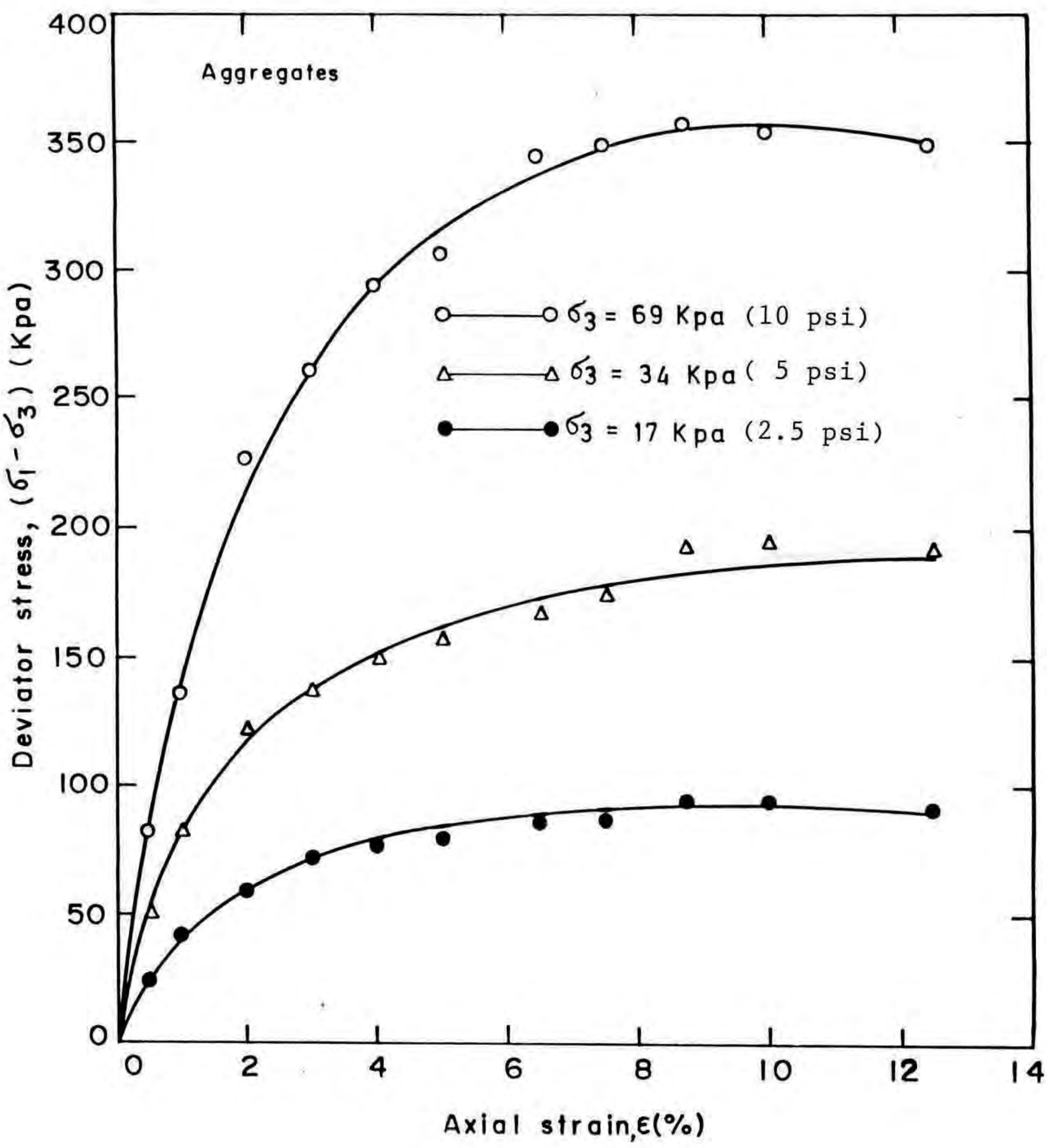


Fig. 4.3 Stress - strain behaviour of stone aggregates produced by suction triaxial test.

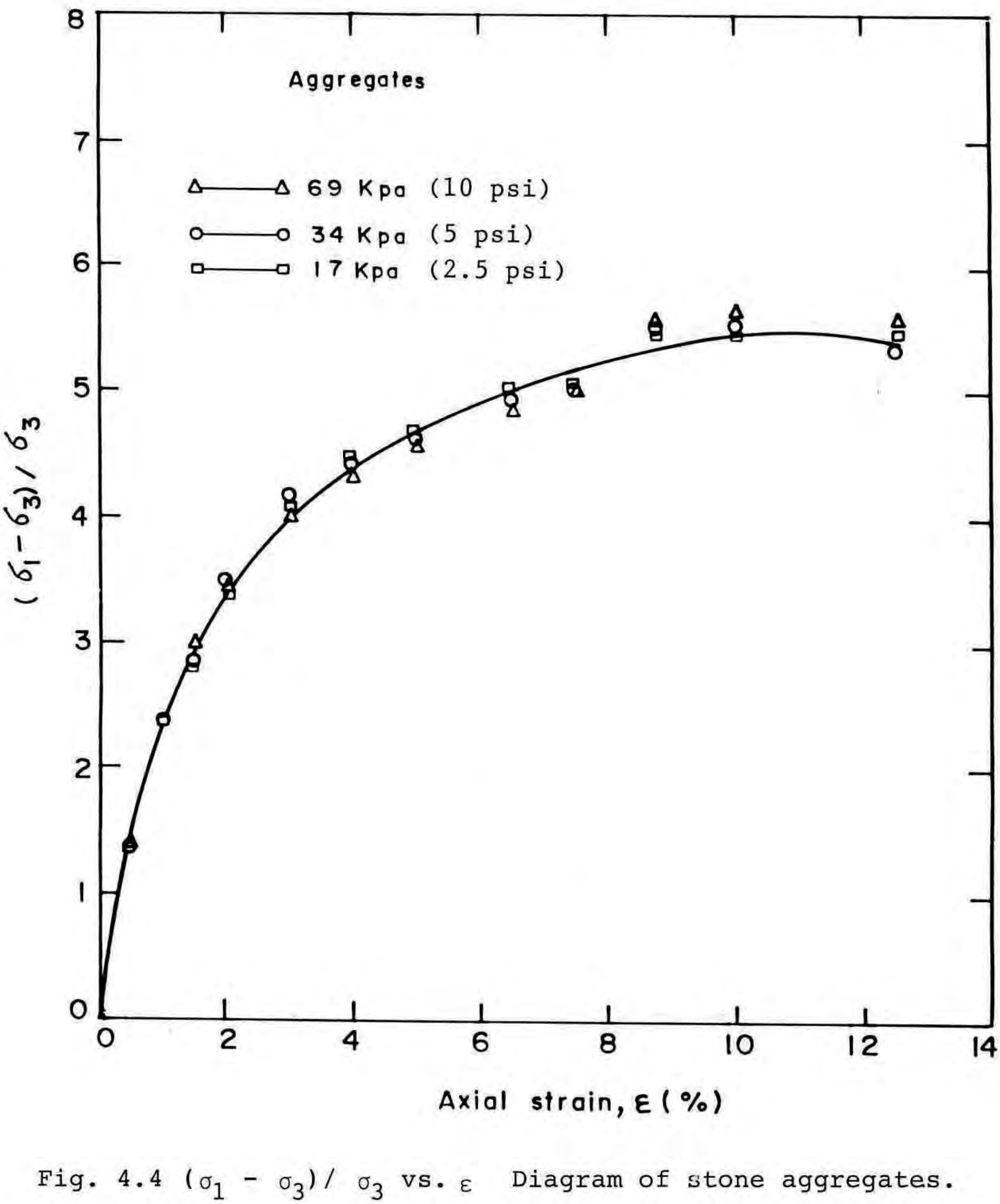


Fig. 4.4 $(\sigma_1 - \sigma_3)/\sigma_3$ vs. ε Diagram of stone aggregates.

Fig.4.3. The calculated data for the establishment of design parameters 'a_a' and 'b_a' and K versus δ / r_o relationship are shown in Tables.4.5 and 4.6. The $(\sigma_1 - \sigma_3)/\sigma_3$ versus σ_3 and $\delta_r/((\sigma_1 - \sigma_3)/\sigma_3)$ versus δ / r_o relation are shown in Figs. 4.4 and 4.5 respectively. Fig.4.5 gives a straight line, the intercept of this line at $\delta/r_o=0$ will give 'a_a' the slope of this line is 'b_a'. The values of these parameters are found to be 0.00145 m/kN and 0.160 m/kN respectively. Using these two parameters the K versus δ / r_o curve can be obtained from Eqn.3.4 which is very close to the curve K versus δ / r_o obtained from the test result direcly, shown in Table.4.7 and Fig.4.6. The value of angle of internal friction (ϕ') of aggregates can be obtained from the failure envelope shown in Fig.4.7. From this figure the ϕ' value of these stone aggregates is found to be 47°.

4.3 PARAMETERS OF REINFORCING ELEMENTS

Reinforcing elements are used as jacket liners in jacketed stone columns. The required design parameters of reinforcing elements and the techniques of their evaluation are discussed here. The test data to evaluate the design parameters is taken from Kabir (1984). In this study, two types of reinforcing elements are characterised, these include a geotextile and a geogrid.

4.3.1 Required Tests, Preparation of Samples and Test Procedures

To evaluate the design parameters of reinforcing elements, load-strain relationship is required. The load-strain

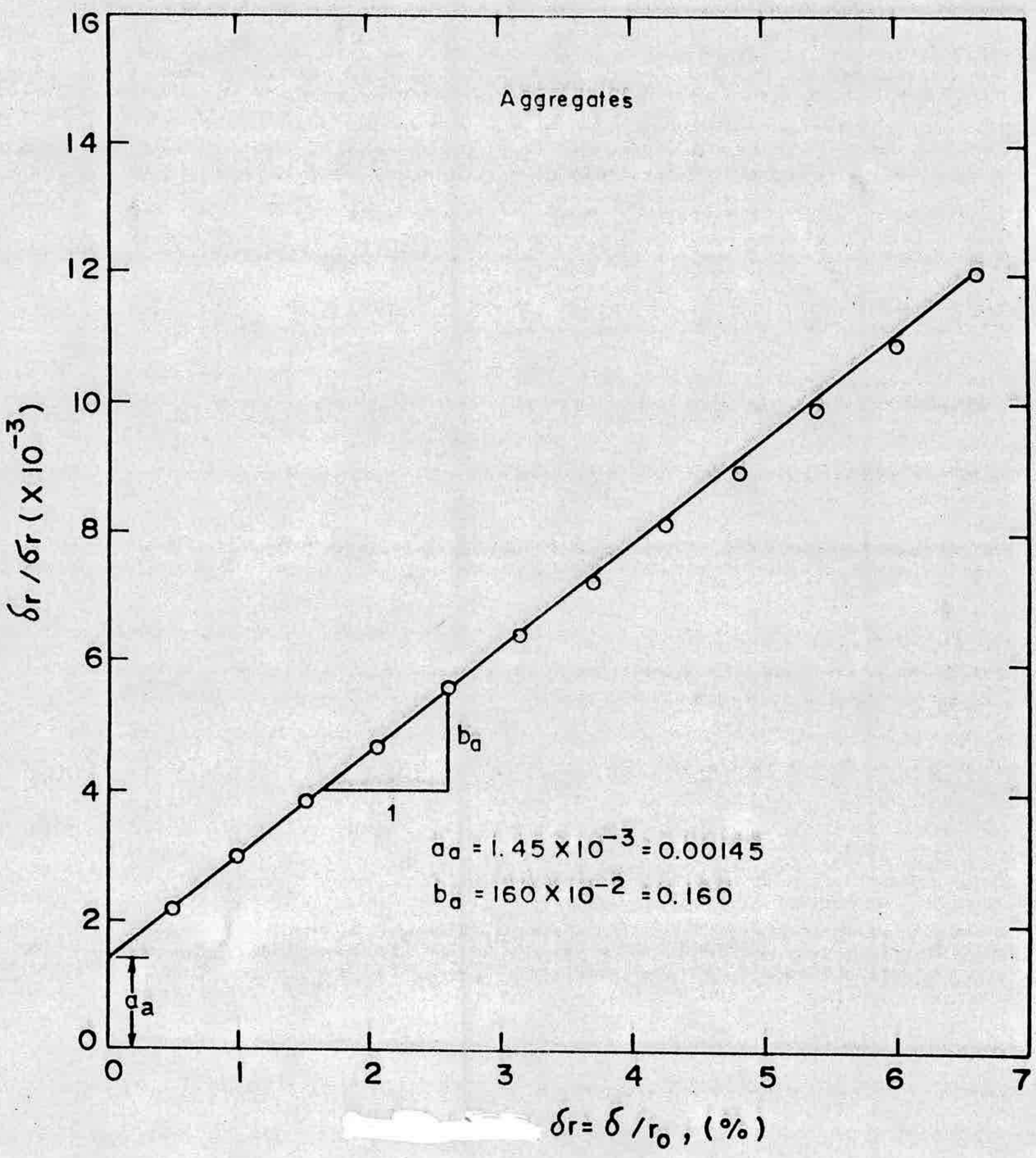


Fig. 4.5 Hyperbolic representation of stress-strain diagram of stone aggregates.

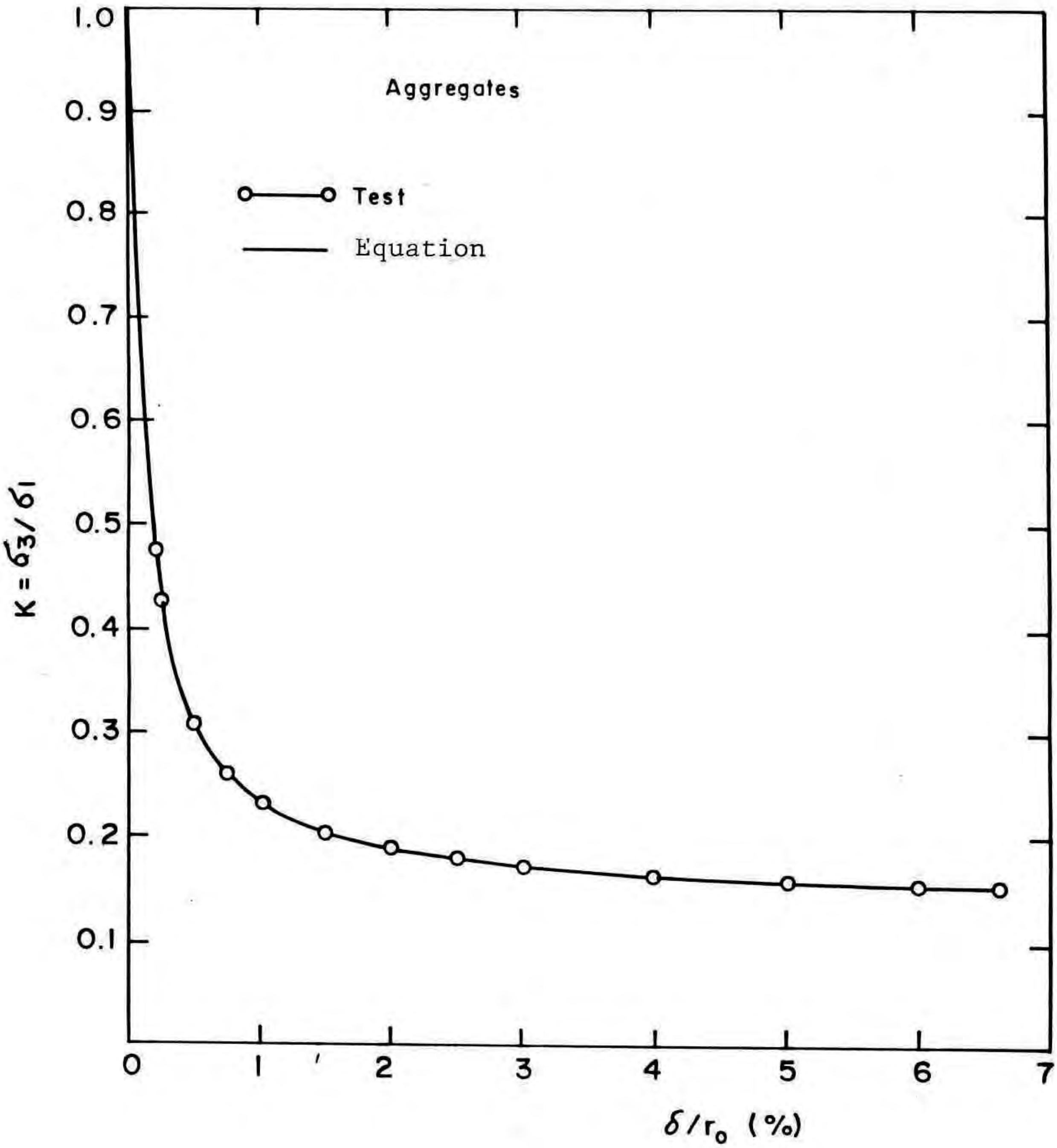


Fig. 4.6 K vs. deform. ratio diagram of stone aggregates.

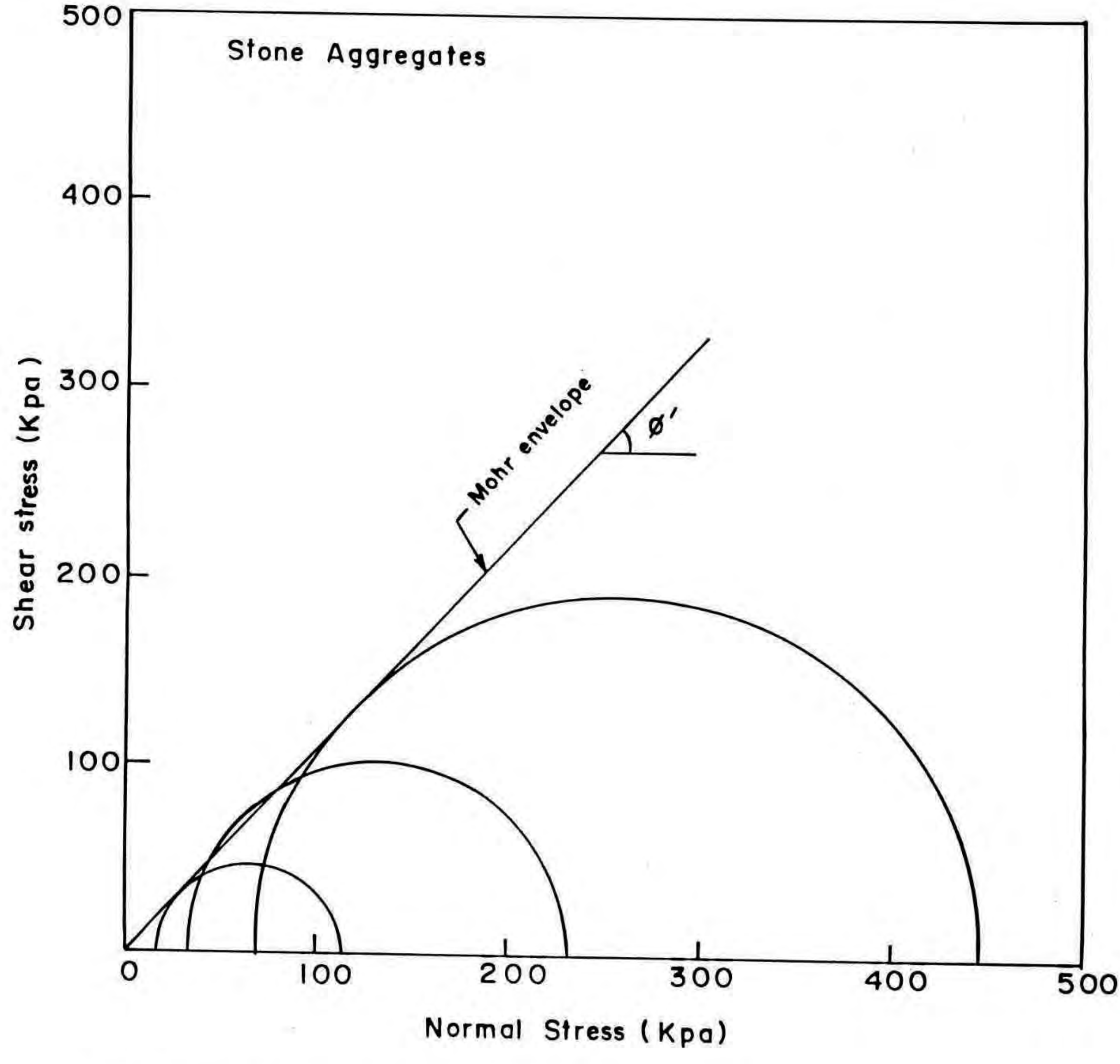


Fig. 4.7 Mohr envelope of stone aggregates.

relationship for reinforcing materials can be generated using the test methods proposed by Andrawes et al (1984) and McGown et al (1984). The preparation of samples for test is somewhat laborious and follows the newly developed techniques and specifications. Details of these techniques and specifications are given by Kabir (1984). The test set-ups required to evaluate the load-strain behaviour of geotextiles and related products are shown in Fig.4.8 (Kabir, 1984). The set-ups mainly comprises upper and lower clamp and a loading device to induce direct tensile loads on specimens. To perform in-isolation tests the following procedures are adopted (see Fig.4.8).

i) The top clamp of the specimen is connected through a pin and saddle mechanism to the top load cell (no.2) which in turn is connected to the top beam "B" of a triaxial loading frame.
ii) The base "P" of the triaxial mechine, which houses the bottom load cell (no.1), is raised to connect it to the bottom clamp of the specimen via another pin and saddle mechanism.
iii) The chosen scan interval and start time is set in the data logger. Following this, the key switch is set to the run position and the teletype switched on almost simultaneously.
iv) After the first scan, which is printed out on the paper roll of the teletype, the motor of the loading machine is started to load the specimen at a preselected rate of strain.
v) The test is continued upto the breaking point of the specimen or upto a desired percentage of strain (usually 40%) whichever is encountered first.

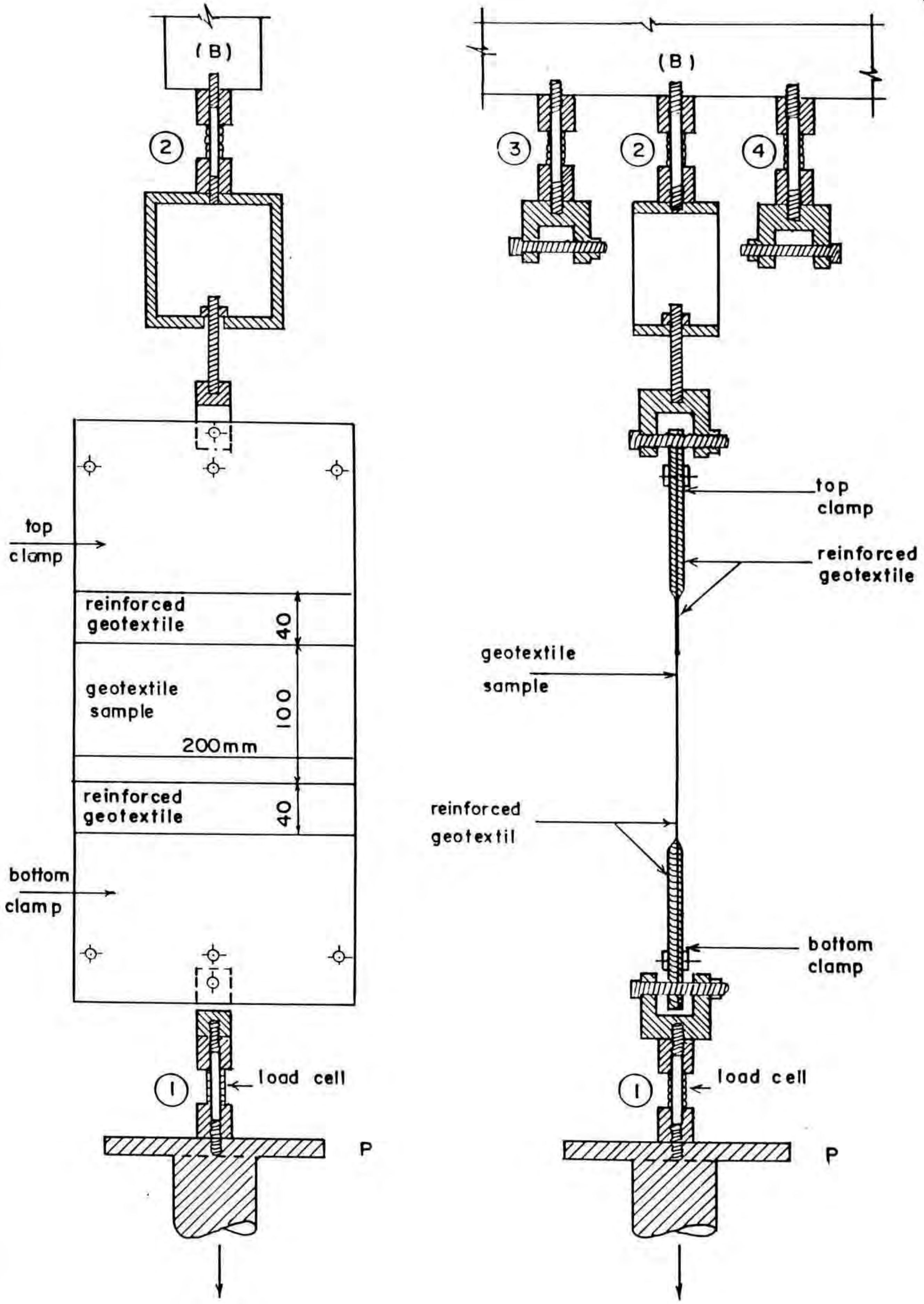


Fig. 4.8 In-isolation test set-up(after Kabir, 1984).

vi) The failure mode of the specimen is noted, the specimen is then unloaded.

4.3.2 Evaluation of Design Parameters

Design parameters of reinforcing elements, in case of jacketed stone column foundation system can be established using load-strain behaviour. The developed relationship between lateral resistance and radial deformation is given by Eqns.3.8 and 3.9 and described in Art.3.6.2.2. The required design parameters are 'a_r ' and 'b_r '. These are different for different types of reinforcing materials but unique for a particular type. Using Eqns.3.8 and 3.9 the curves $p_r.r_o$ versus δ/r_o and p_r versus δ/r_o can be established. From these curves, the lateral resistance(p_r) mobilized in reinforcing materials for a particular radial deformation (δ/r_o) can be evaluated.

4.3.3 Values of Design Parameters

Design parameters for a geotextile and a geogrid in case of jacketed stone column are presented here. The load strain data (Kabir, 1984) and relevant calculation procedures are shown at Tables.4.8 and 4.9. The relationships of load (p_r) versus strain (ε), ε/p_r versus ε , $p_r.r_o$ versus δ/r_o and p_r versus δ/r_o (for r_o = 250 mm) are presented in graphical forms by the Figs. 4.9, 4.10, 4.11 and 4.12 respectively. From Fig.4.7 the parameters 'a_r' and 'b_r' for the geotextile and the geogrid can be obtained from the intercepts (at $/r_o = 0$) and slopes of the lines. The values of these parameters are found to be, a_r =

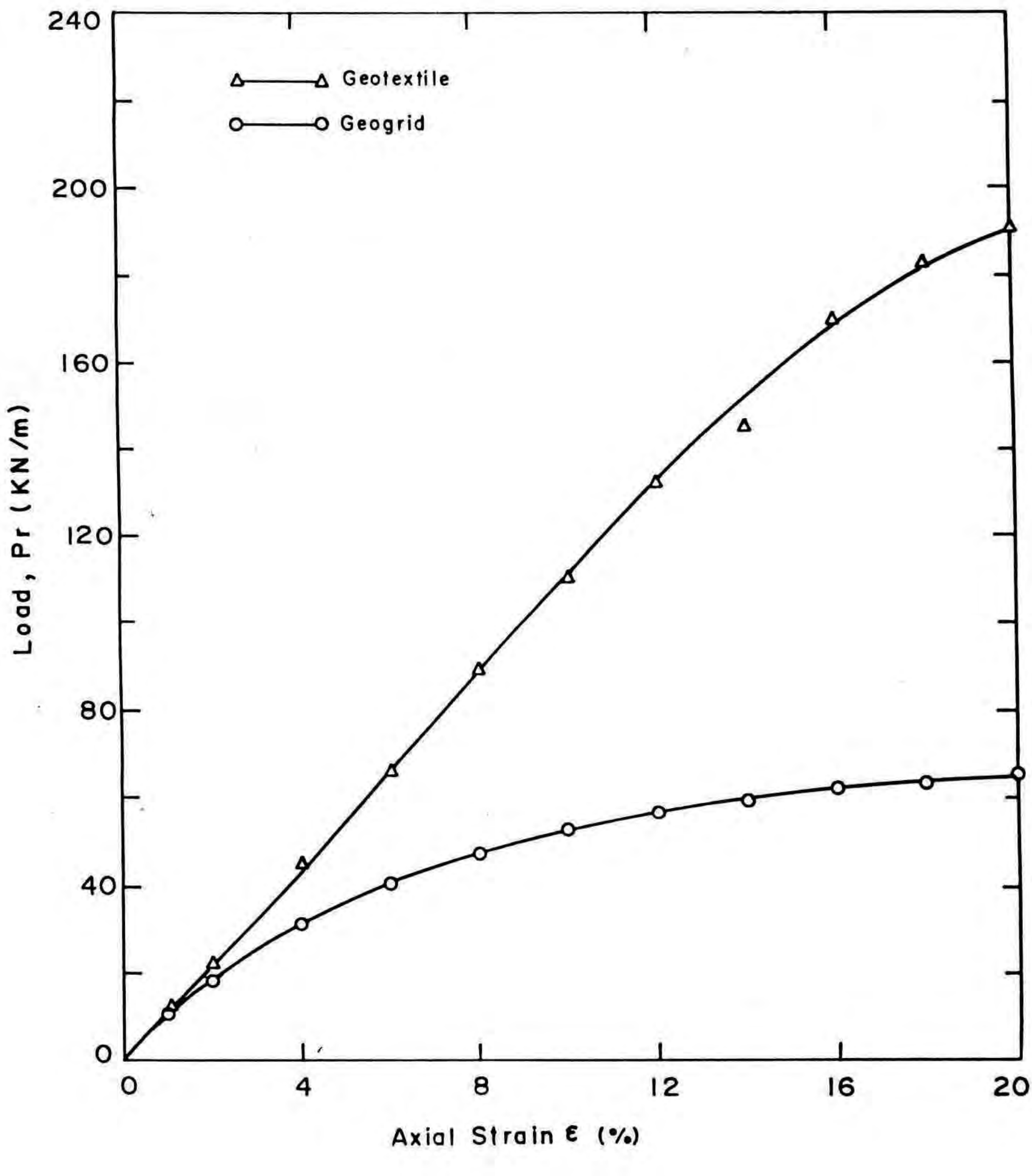


Fig. 4.9 Load strain diagram of geotextile and geogrid.

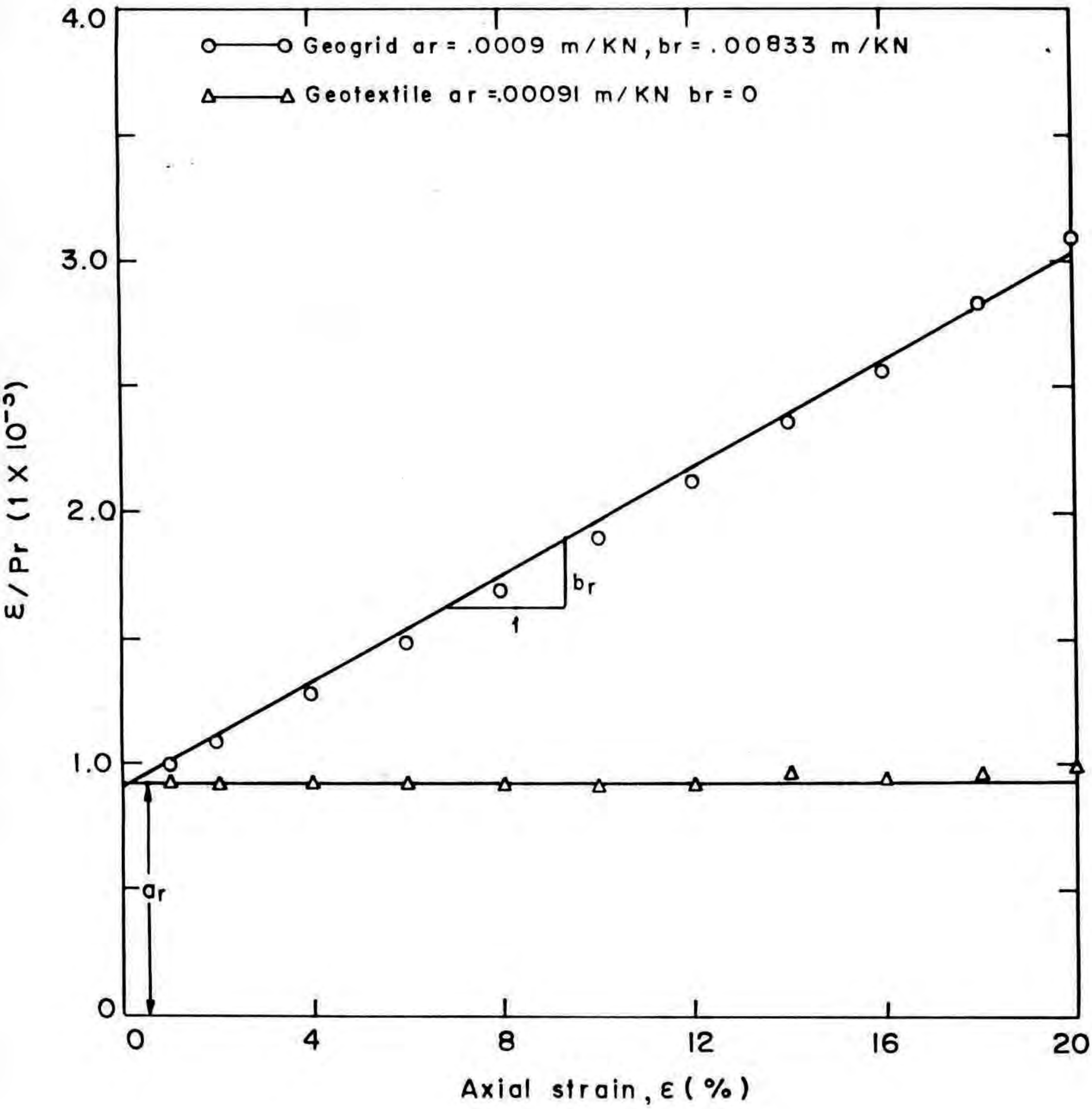


Fig. 4.10 Hyperbolic representation of load strain diagram of geotextile and geogrid.

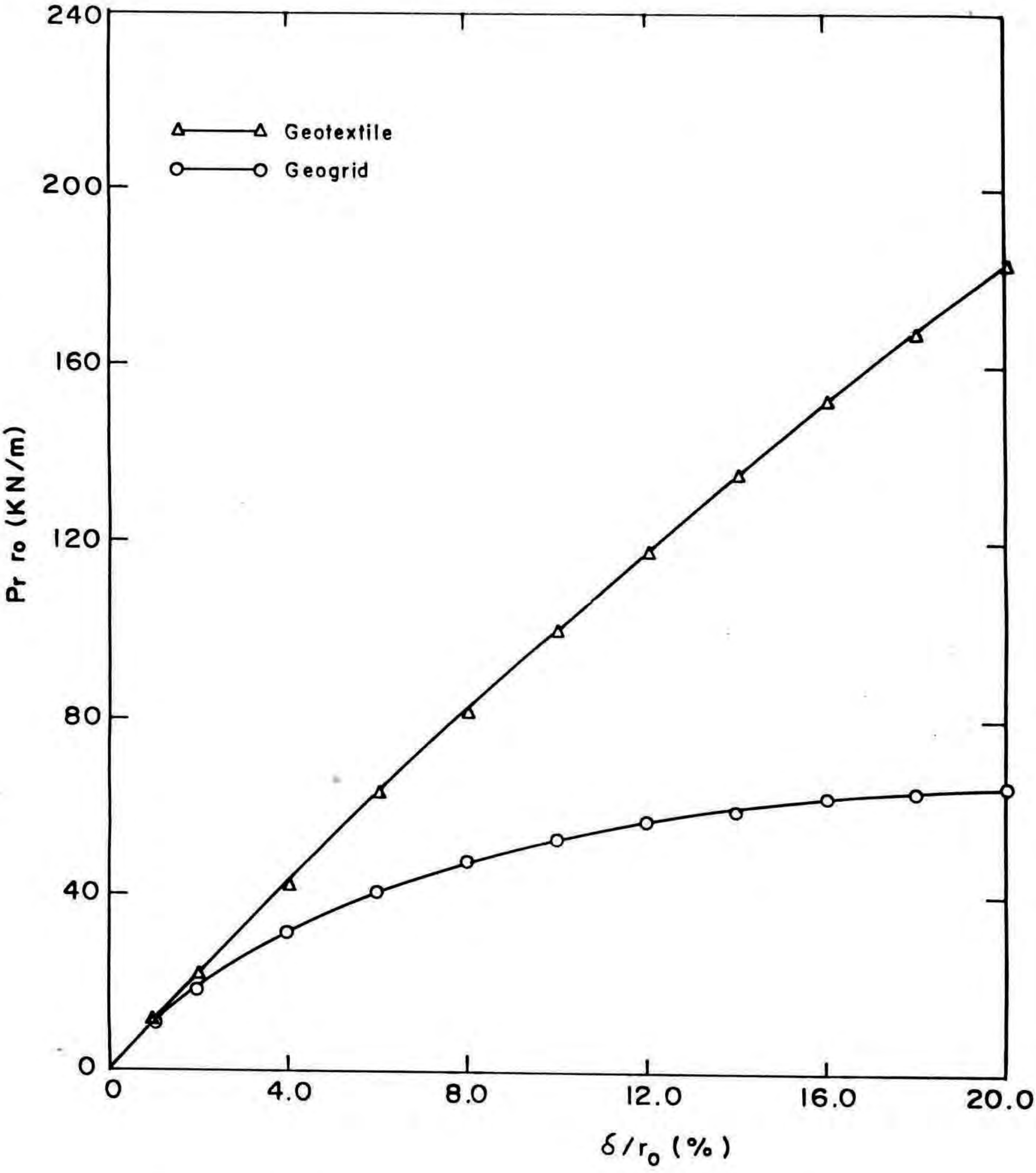


Fig. 4.11 $P_r r_o$ vs. δ / r_o diagram of geotextile and georid.

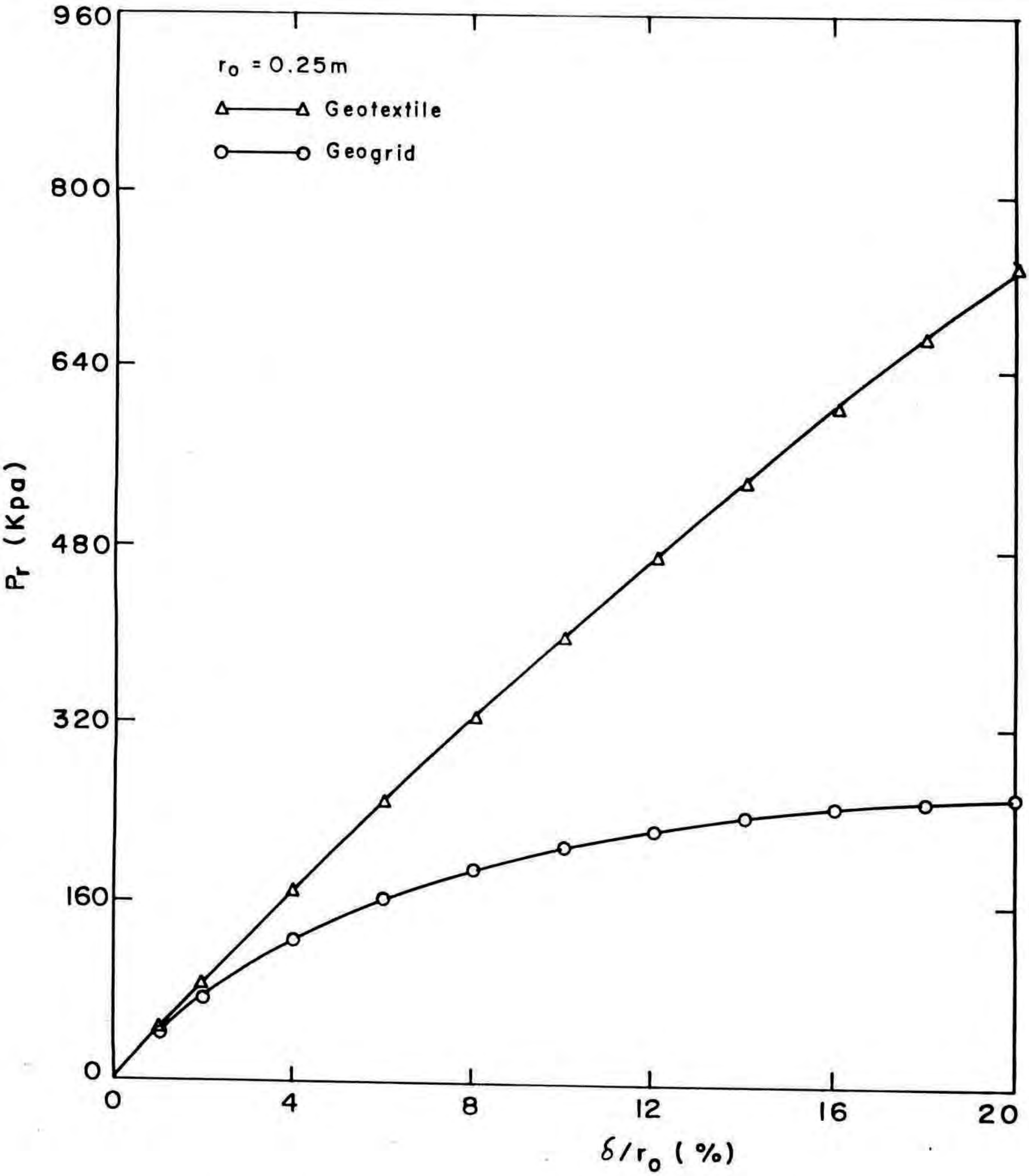


Fig. 4.12 P_r vs. δ/r_o diagram of geotextile and geogrid.

0.00091 m/kN, b_r = 0.0 and a_r = 0.0009 m/kN, b_r = 0.00833 m/kN for the geotextile and the geogrid respectively.

4.4 CLAY PARAMETERS

The properties of surrounding clay media are very important for the design of stone column foundation as the stiffness of the stone column is dependent on the mobilisation of lateral confinement from the clay. The required design parameters of clay and test methods for their evaluations are presented here. Typical values of design parameters for a clay media from the coastal region of Bangladesh are presented here.

4.4.1 Test Requirements and Methods

Undrained stress strain behaviour of clay is the main basis from which design parameters are established. The deviator stress versus shear strain relationship is used as the main input in the cavity expansion theory which has been used to develop design procedures for plain and jacketed stone column. For these, required tests data are generated from the conventional consolidated undrained triaxial test. The collection and preparation of samples are done using conventional approach.

4.4.2 Evaluation of Design Parameters

The behaviour of the surrounding clay media of stone column is presented in these studies by the relationship amongst p_s , δ / r_o and Z (Eqn.3.24 ,Fig.3.23). The method of development of this relation is described in Art.3.6.3.2. Here consolidated undrained triaxial test data are obtained as deviator stress ($\sigma_1 - \sigma_3 = q$) versus axial strain (ε) relation at different initial consolidation pressure (σ_3), simulating the effective overburden pressure (γZ). From this, q / σ_3 versus γ_s relationship is established as an unique curve using the relation $\gamma_s = 1.5\,\varepsilon$, appropriate for undrained condition. Then q/σ_3 versus γ_s is represented in hyperbolic form to obtain the parameters 'a_s' and 'b_s'. These parameters are required to establish $p_s / \gamma Z$ versus δ / r_o relation using Ladanyi's cavity expansion theory.

The clay parameters E, Su , ν and α are required to evaluate the frictional forces (P_f) mobilised along stone column-clay interface. The modulus of elasticity (E) can be evaluated from the stress strain diagram of soil. Here secant modulus is considered. The method of evaluation of E is shown in Fig.4.13. Average undrained shear strength (Su)can be evaluated by the conventional triaxial test. The Poisson's ratio (ν) is taken as 0.50, considering the clay media is saturated. The evaluation of another important parameter, adhesion factor (α), is described in the following.

Adhesion factor (α): The ultimate adhesion, between pile and clay is generally related to undrained shear strength (Su) by the simplified relation α Su. Much of the uncertainity to calculate the pile load capacity lies in the selection of a value of α . This can be back analysed from the results of pile load

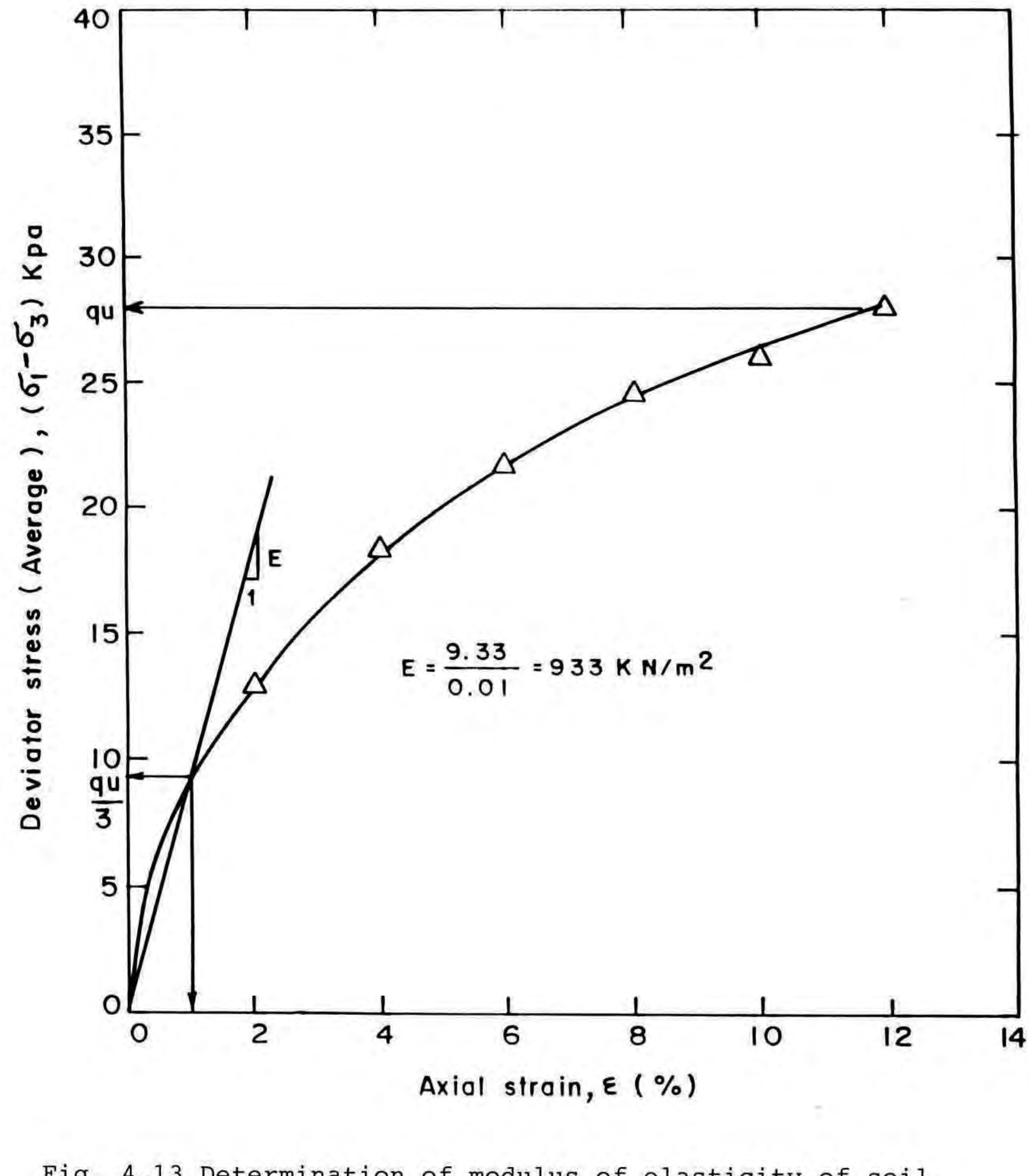


Fig. 4.13 Determination of modulus of elasticity of soil.

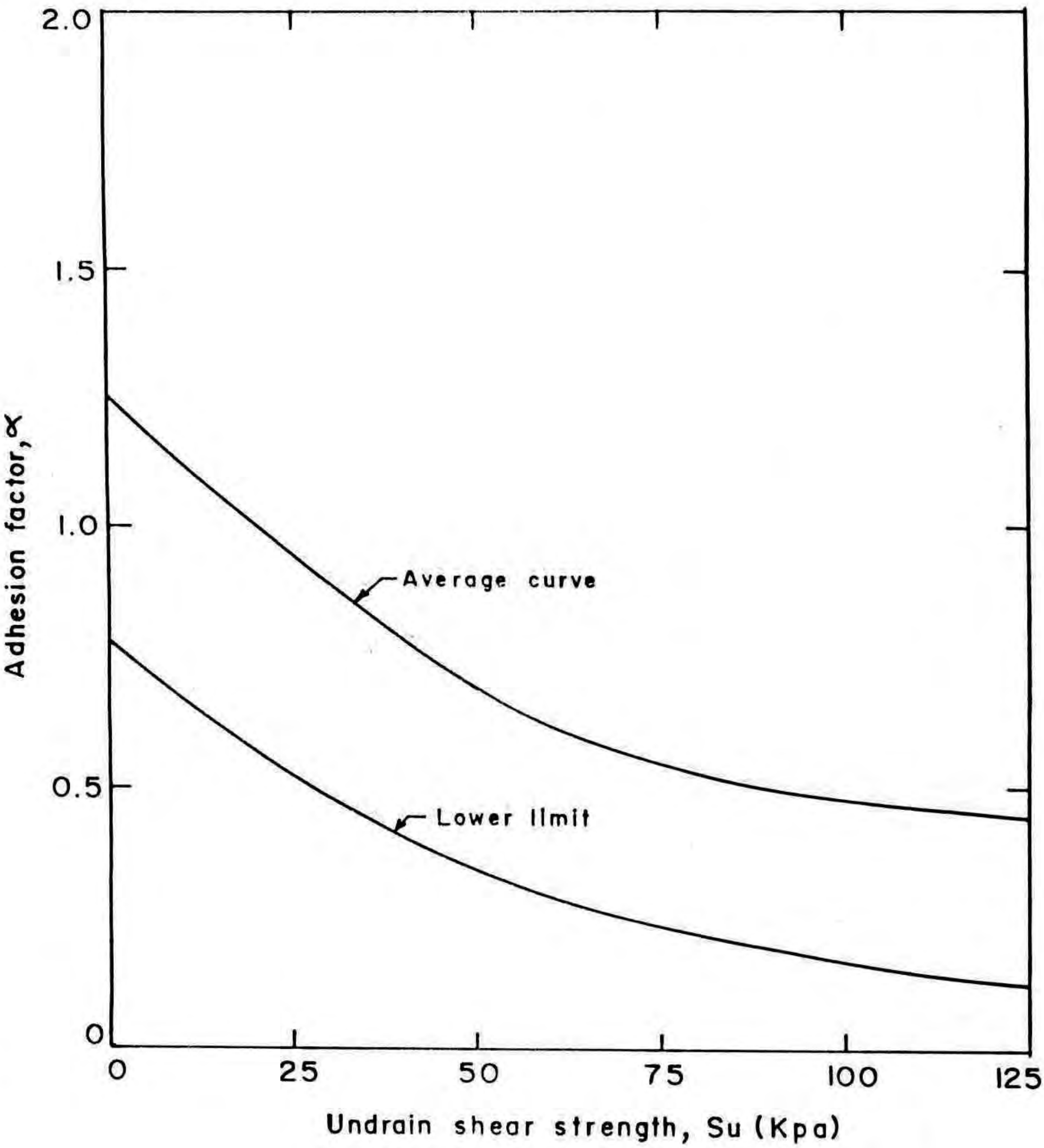


Fig. 4.14 Relation between adhesion factor for driven piles and undrained shear strength of clay (after Tomlison, 1970).

tests. Accumulated data suggested that the value of α for the soft clays ($Su \leqslant 50$ kPa) is about unity, for stiff clays, however, α appears to be very vaŗiable and may be influenced by the pile geometry and the nature of any strata overlying the stiff clay (Tomlinson, 1970). After Tomlinson, the variation of α with Su are shown in Fig.4.14 (Simons & Menzies, 1977). In this figure the drop of adhesion factor with increasing undrained shear strength of the clay is most marked.

4.4.3 Values of Design Parameters

Design parameters were evaluated using the consolidated undrained triaxial data of a clay, which was collected from a site situated at the south-east region of Bangladesh. The test data are presented here in Table.4.10 and Fig.4.15. Using the test data, the design parameters of clay, 'a_s' and 'b_s' were obtained by the proposed evaluation techniques (Art.3.6.3.1). The calculated data are shown in Tables.4.10 and 4.11. The developed curves $(\sigma_1 - \sigma_3)/\sigma_3$ versus ε , γ_s and $\gamma_s / ((\sigma_1 - \sigma_3)/\sigma_3)$ versus γ_s are shown in Fig. 4.16 and 4.17 respectively. From Fig.4.17 the design parameters 'a_s' and 'b_s' can be obtained from the intercept (at $\gamma_s = 0$) and slope of the line. The values of these parameters 'a_s' and 'b_s' are found to be 0.095 and 1.667 respectively. Using the values of 'a_s' and 'b_s' the relationship $p_s / \gamma Z$ versus δ / r_o and p_s versus δ / r_o ,Z can be evaluated. These are shown at Table.4.12 and Figs. 4.18 and 4.19 respectively.

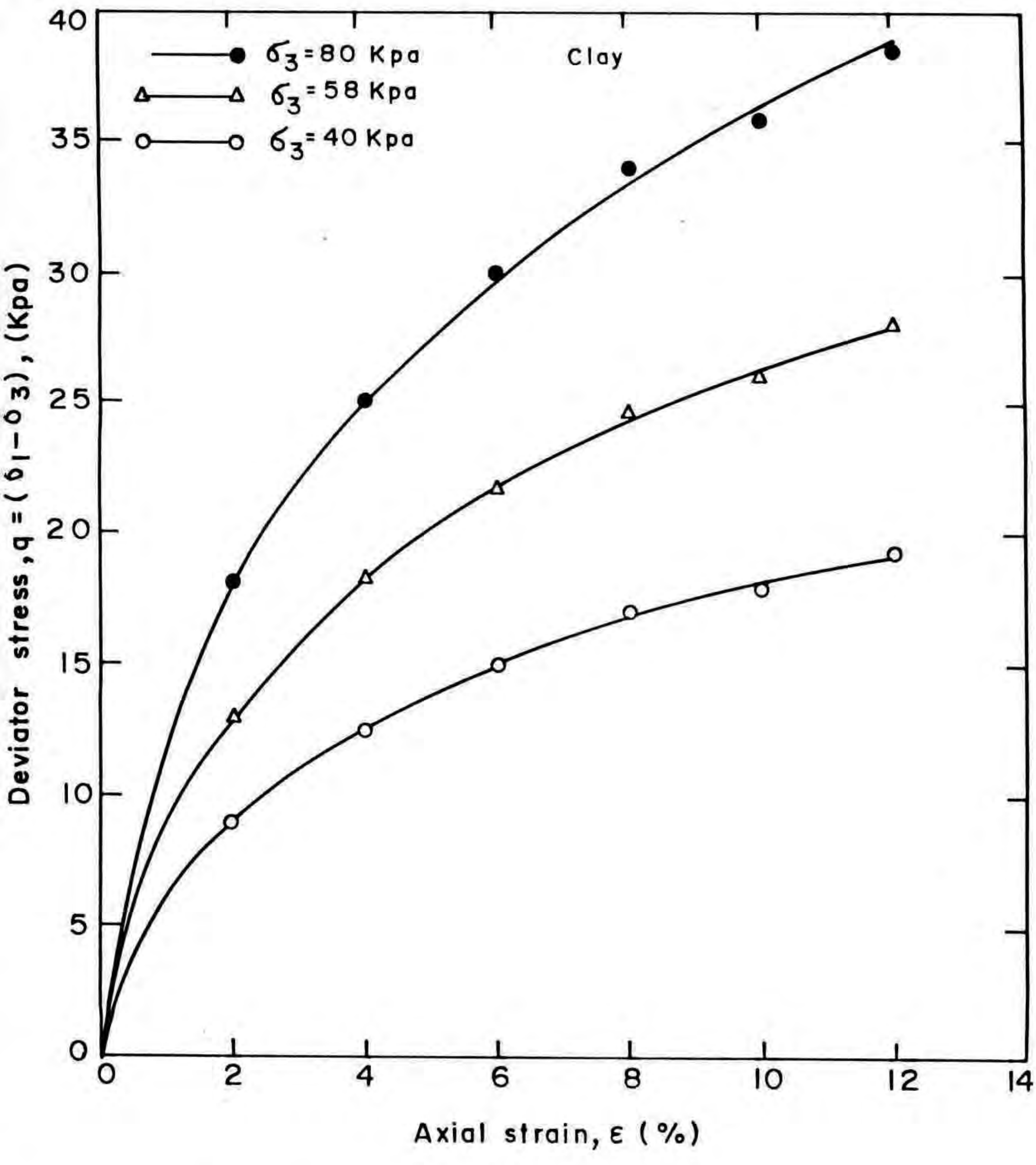


Fig. 4.15 Stress-strain diagram of clay from south-east region of Bangladesh.

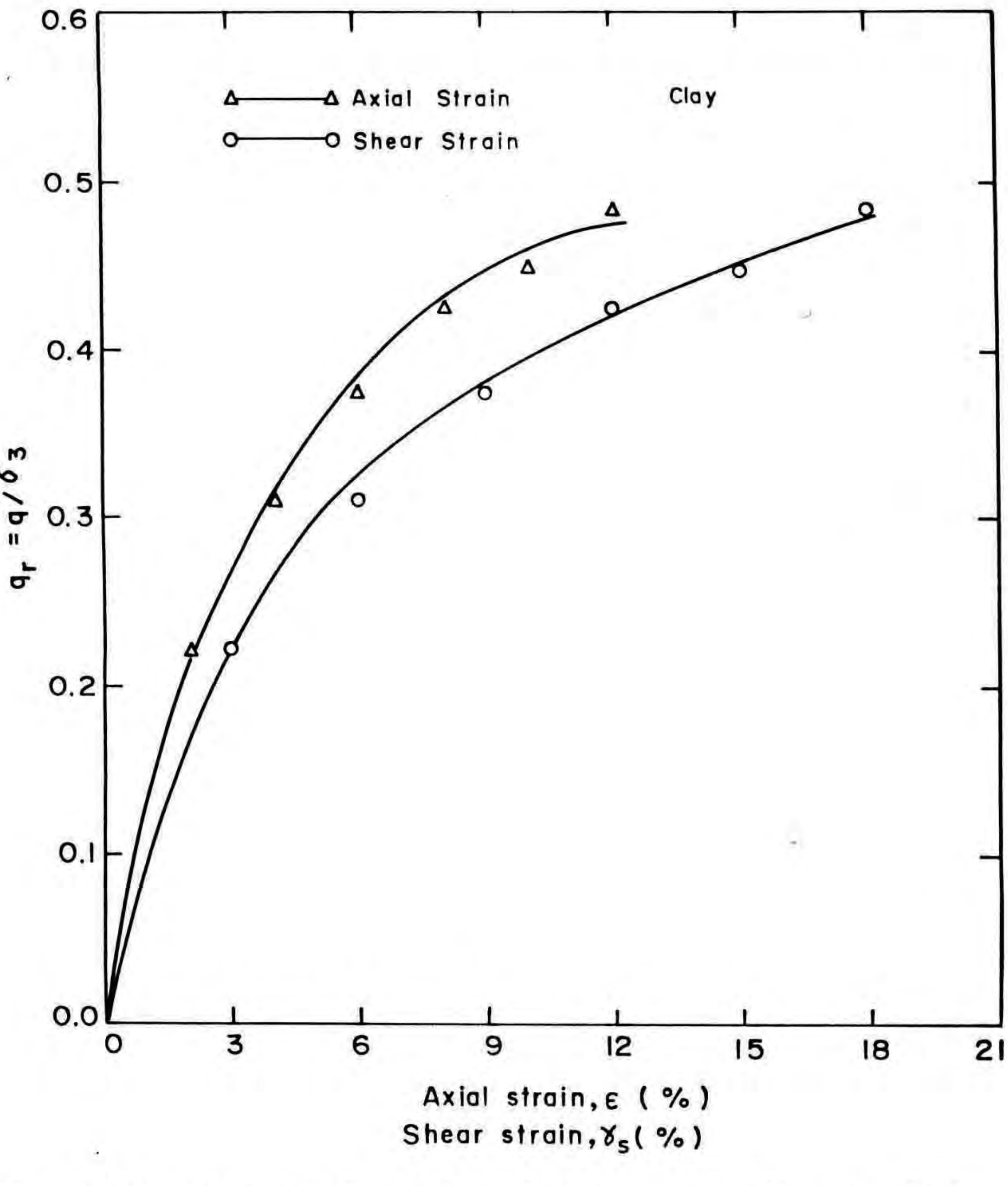


Fig. 4.16 q_r vs. axial strain and shear strain diagram of clay.

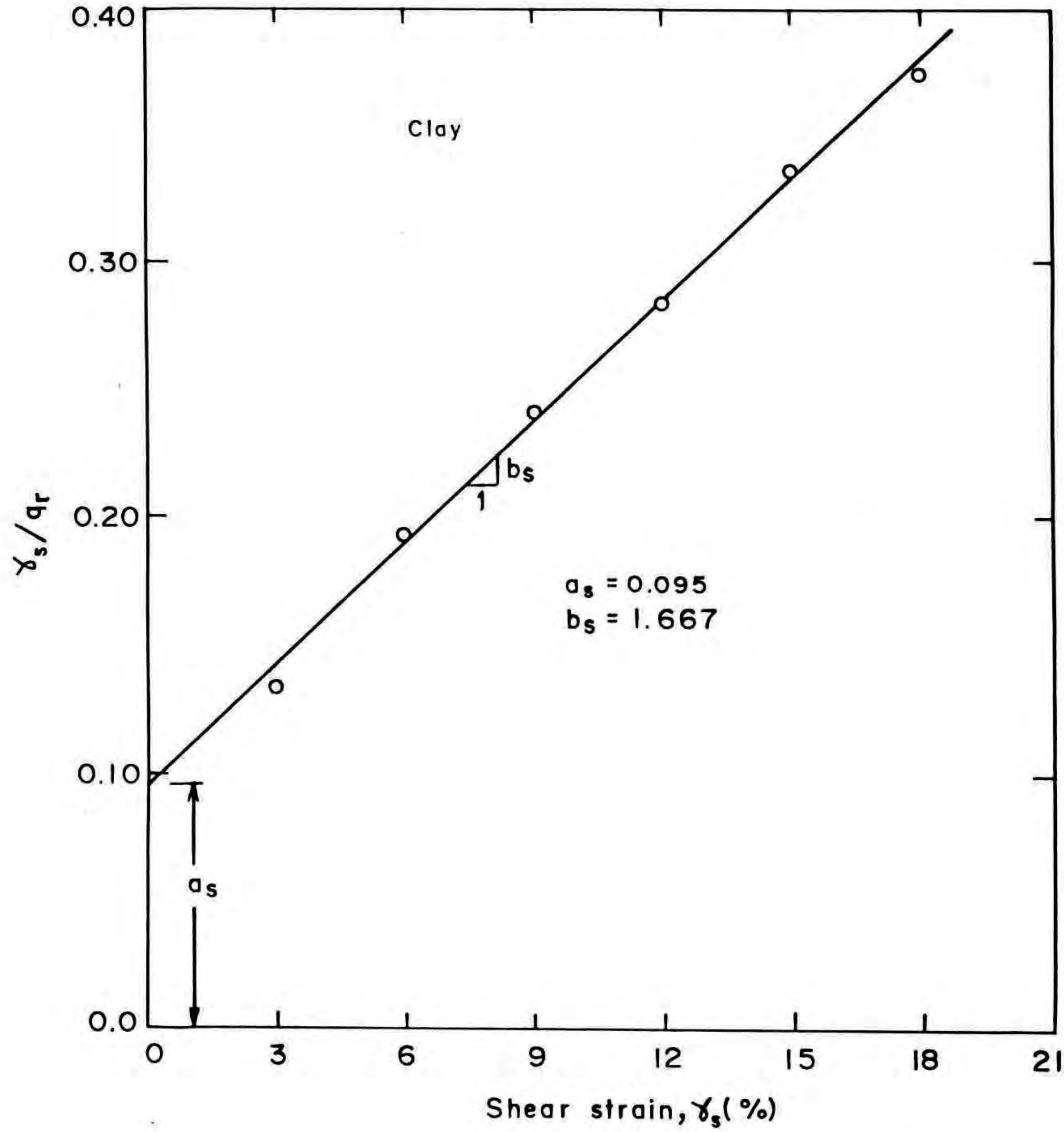


Fig. 4.17 Hyperbolic representation of stress strain behaviour of clay.

The average undrained shear strength (Su) of clay can be obtained from Fig.4.15, which is found to be 14 kPa. The modulus of elasticity of the soil (E) can be evaluated from Fig.4.13 which is found as 933 kPa. The value of adhesion factor (α) for $\bar{Su}$ = 14 kPa is found greater than 1.0 (Fig.4.14), but in these analysis it is taken as 1.0. Using these design prameters in the Eqns. 3.34, 3.35 and 3.36, skin friction (τ_s) versus settlement (s) and frictional force (P_f) versus settlement (s) can be evaluated. These are presented in Table.4.13 and Figs.4.20 & 4.21

4.5 DISCUSSION

To characterise the mechanical properties of stone aggregates, a new test method, suction triaxial test is introduced in this studies. The limitations and problems encountered during this test is described by the following:

i) The newly developed suction triaxial test, to characterise the aggregates behaviour, functioned satisfactorily though it is very difficult to maintain the constant suction throughout the test.

ii) It becomes very difficult to avoid system leakage due to the poor localy manufactured equipments.

iii) Due to the sharp angularity of stone aggregates it is difficult to keep the rubber membrane free of leakage.

iv) In this test, though there is no limitation of sample size, the suction beyond 100 kPa can not be applied.

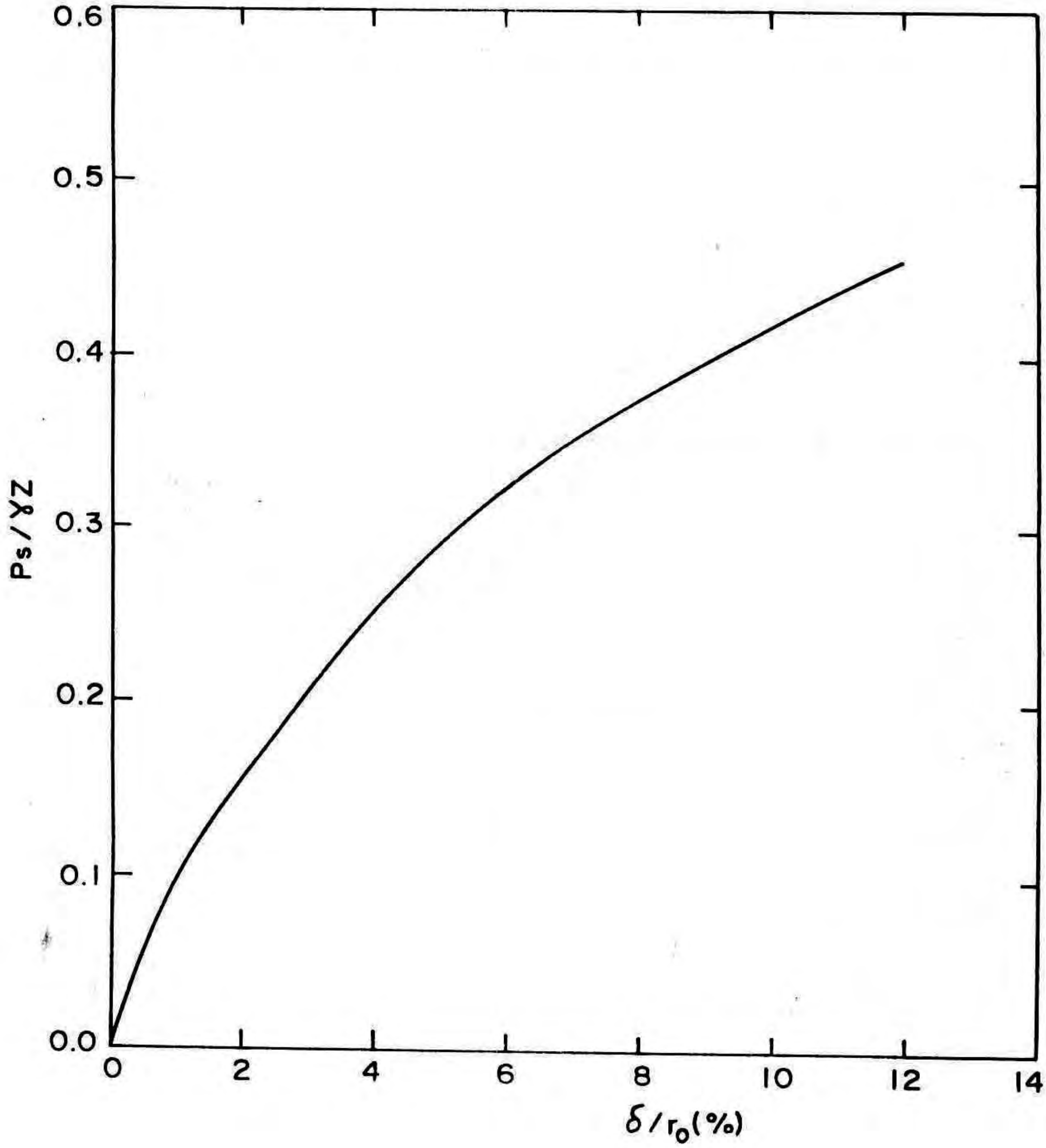


Fig. 4.18 Lateral resistance vs. deform.ratio diagram of clay.

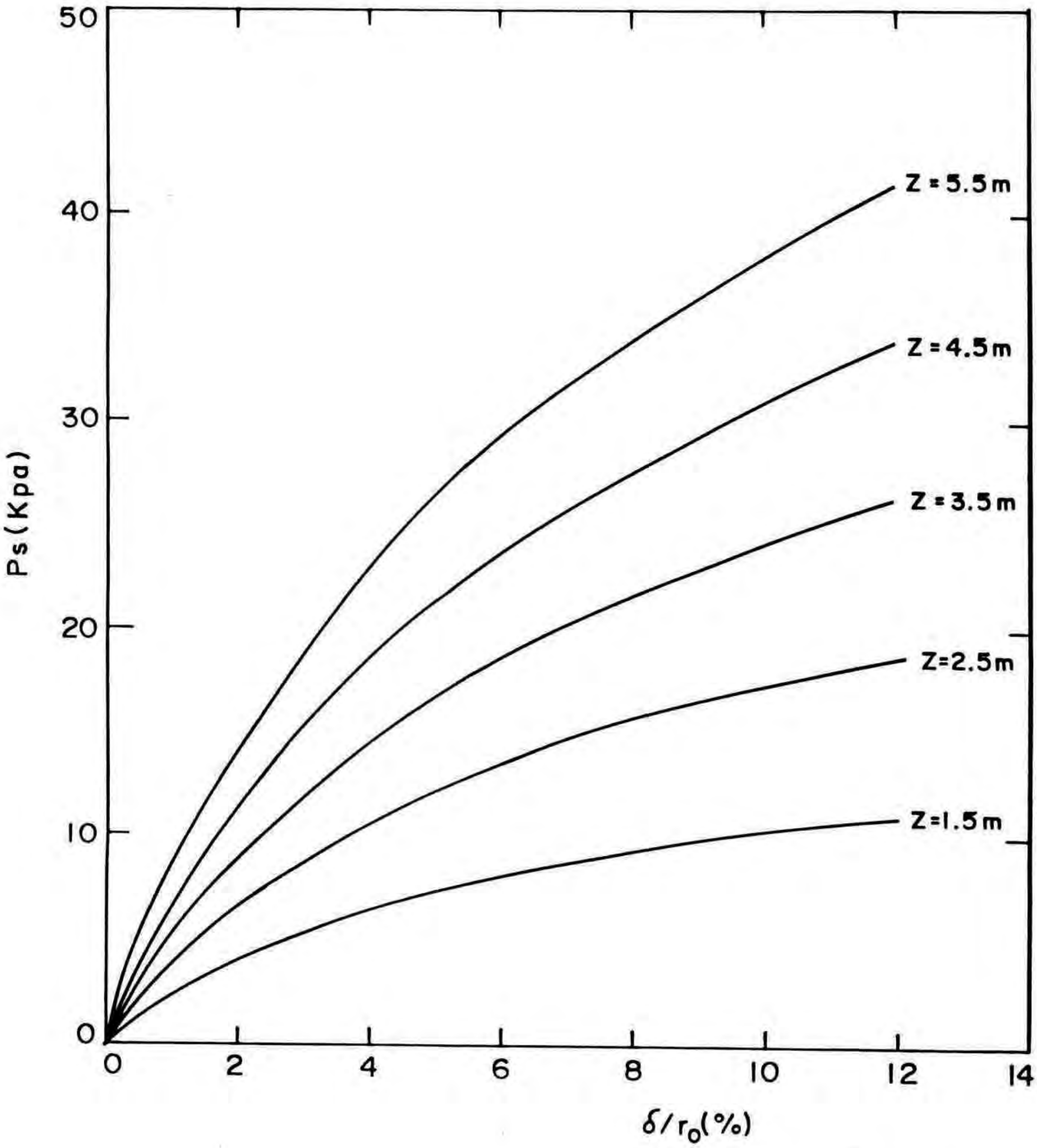


Fig. 4.19 Lateral resistance vs. deform. ratio of clay at different depth of stone column.

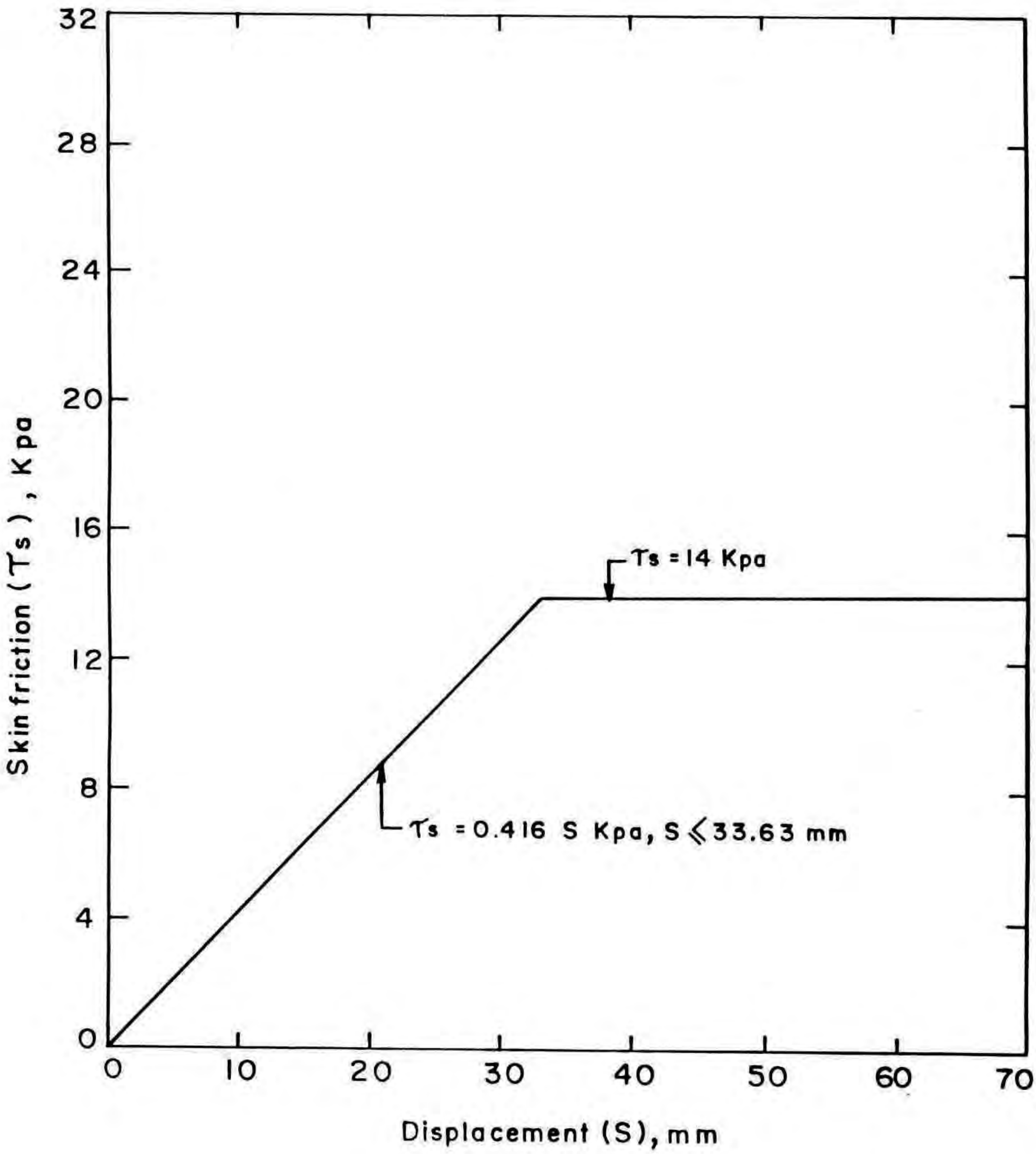


Fig. 4.20 Skin friction vs. displacement diagram develops along stone column - clay interface.

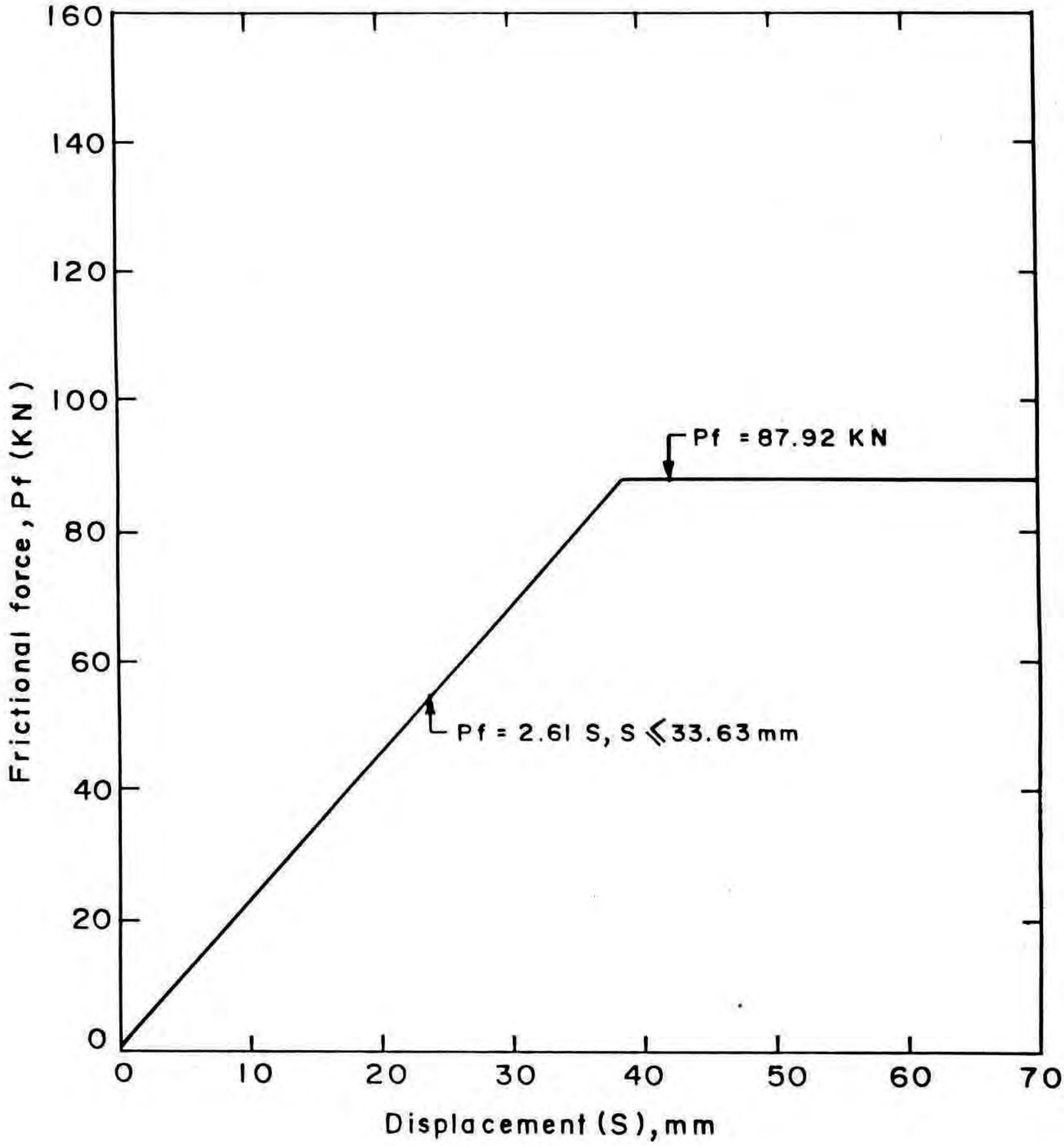


Fig. 4.21 Frictional force vs. dispalcement diagram develops along stone column clay interface.

CHAPTER FIVE

DESIGN EXAMPLE

5.1 GENERAL

A new approach to analysis and design of plain and jacketed stone column is reported in this thesis. To depict the selient freatures of this new approach, design examples are produced, which will help professionals in this field to grasp these approaches. Examples of analysis and design of plain and jacketed stone columns are produced showing use of stone aggregates, geogrid jacketing material and soft clay obtained from the south east region of Bangladesh. A comperative study of the new method with the existing methods is also produced.

5.2 SIZE OF STONE COLUMN

The diameter and length of a stone column is normally dictated by the consistancy and extent of soft clay media. Normally if a relatively stronger strata is encountered at a reasonably shallow depth, stone column foundation is carried upto that strata. In this example a 4 m long stone column having 500 mm diameter and founded at a depth of 1.5 m below ground level is considered. The base of the column is assumed to be seated on a smooth rigid stratum.

5.3 AGGREGATES

In these examples nearly uniform size of stone aggregates (9 mm to 12 mm) is used to achieve as closely as possible the assumed no volume change condition. The design parameters of aggregates are evaluated by suction triaxial test, described at Art.4.2.1. The relevant K versus δ / r_o relation are presented in Table.4.7 and Fig.4.5.

5.4 REINFORCING ELEMENT

For the design of newly envisaged jacketed stone column, a geogrid is used as the reinforcing element to act as jacket linear around the aggregates. The evaluation of design parameters of geogrid is described in Art.4.3.2. The $p_r r_o$ versus δ / r_o, p_r versus δ / r_o relationships are produced in Table.4.9 and Figs. 4.11 and 4.12.

5.5 CLAY MEDIA

A soft clay media from south-east region of Bangladesh is used in these examples. The clay media is soft, saturated and normally consolidated having undrained shear strength linearly increasing with depth. A rigid strata at a depth of 5.5 m below the existing ground surface is assumed. The consolidated undrained triaxial test data for the clay are presented in Table.4.10 and Fig.4.15. The mobilised lateral confinement from the surrounding clay media due to the radial deformation of column, p_s as a function of δ / r_o are presented in Table.4.12 and Figs. 4.18 and 4.19. Mobilised skin friction (τ_s versus s) and frictional resistance (p_f versus s) along stone column-clay interface are presented are in Table.4.13 and Figs.4.20 and 4.21.

5.6 DESIGN OF A PLAIN STONE COLUMN

The proposed design method is used here to establish the load versus settlement behaviour of a plain stone column from which the load carrying capacity at specified settlement can be obtained. The design steps described previously in Art.3.8.4 is followed and the relevant design parameters of the constituents materials are obtained from Arts. 4.2.6 and 4.4.3. The relationships Pv = f(δ / r_o , Z) and ε = f(Pv, Z) are established and presented in Tables. 5.1 and 5.2 respectively and Figs.5.1 and 5.2 respectively. The vertical pressure (Pv) versus settlement (S) diagram is presented in Table.5.3 and Fig.5.3. This relationship is established by neglecting any skin friction along the shaft of the stone column. Next the effect of skin friction is considered by methods described earlier in Art.3.6.3.2 and 3.8.4. The relevant pressure (Pv) versus settlement (S) relation are presented in Table.5.8 and Fig.5.4. The results of appropriate trials at different pressures are presented in Tables. 5.4 through 5.7 from which Table.5.8 is established. The pressure at 50 mm (ten percent of diameter 500 mm) settlement for a plain stone column without considering and considering skin friction was found to be 137 kPa and 185 kPa respectively. This is equivalent to loads of 26.90 kN and 36.30 kN respectively on the stone column.

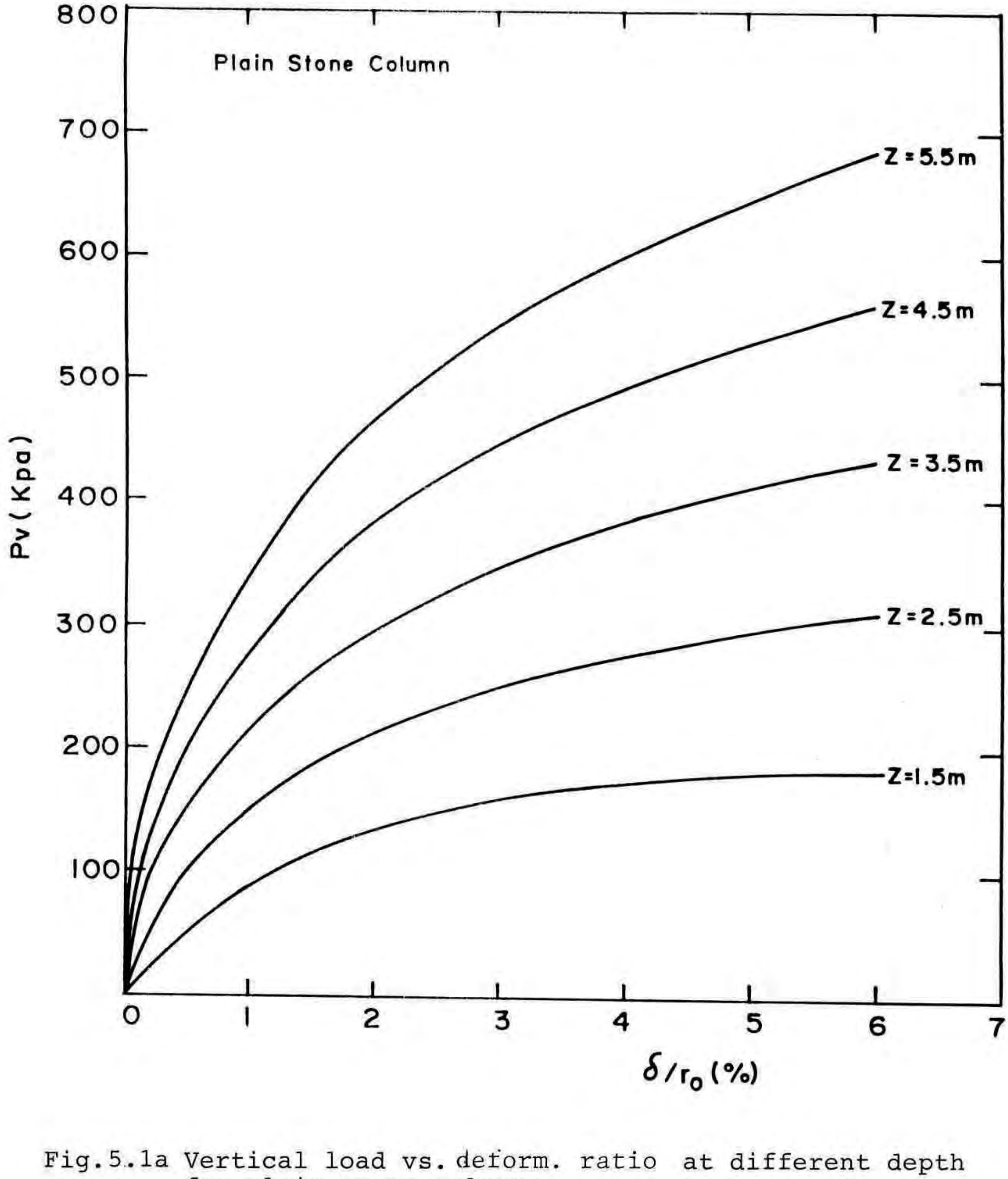


Fig.5.1a Vertical load vs. deform. ratio at different depth for plain stone column.

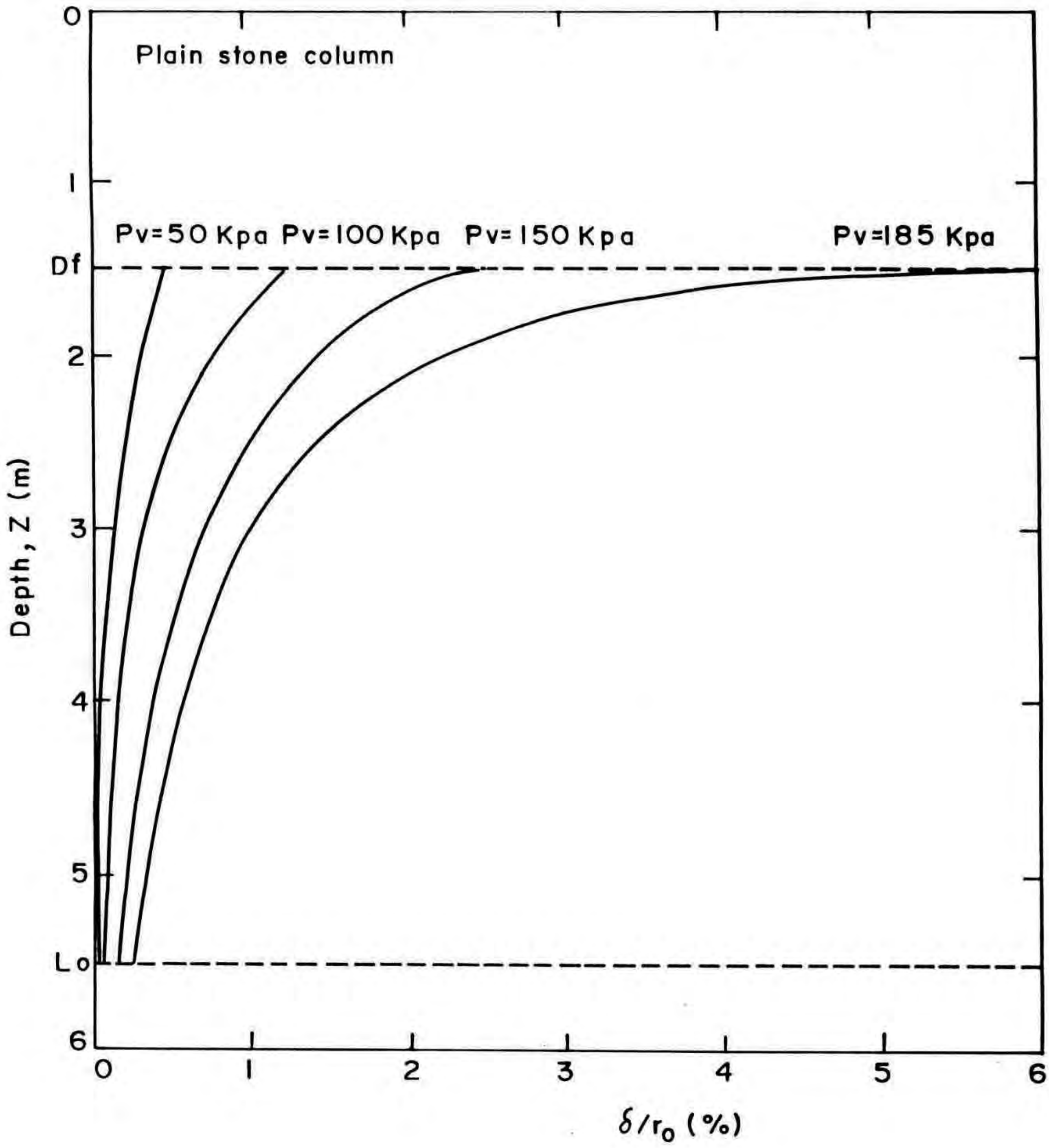


Fig. 5.1b Vertical load vs. deform. ratio at different depth for plain stone column.

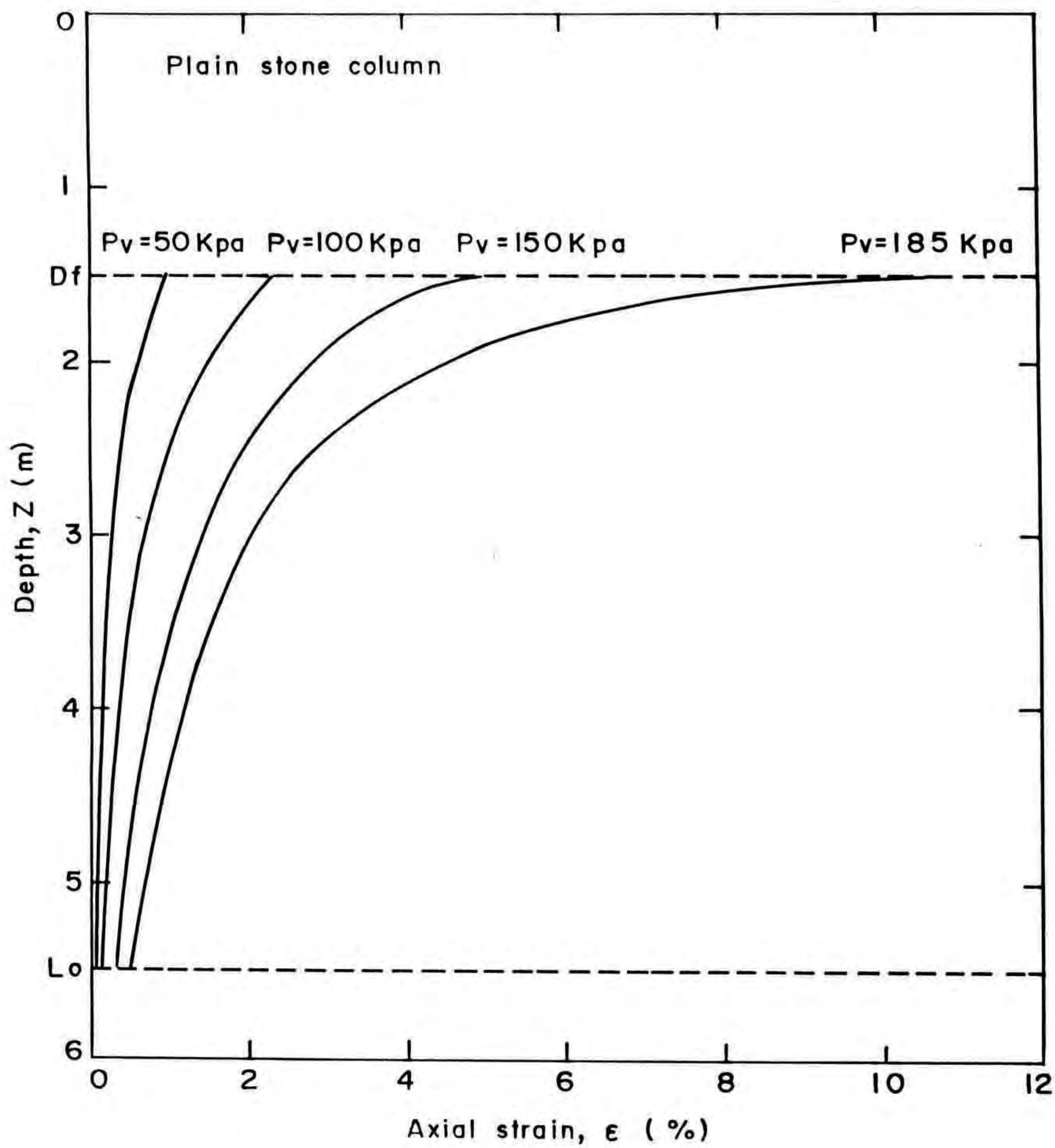


Fig. 5.2 Variation of axial deformation for different vertical load throughout the length of plain stone column.

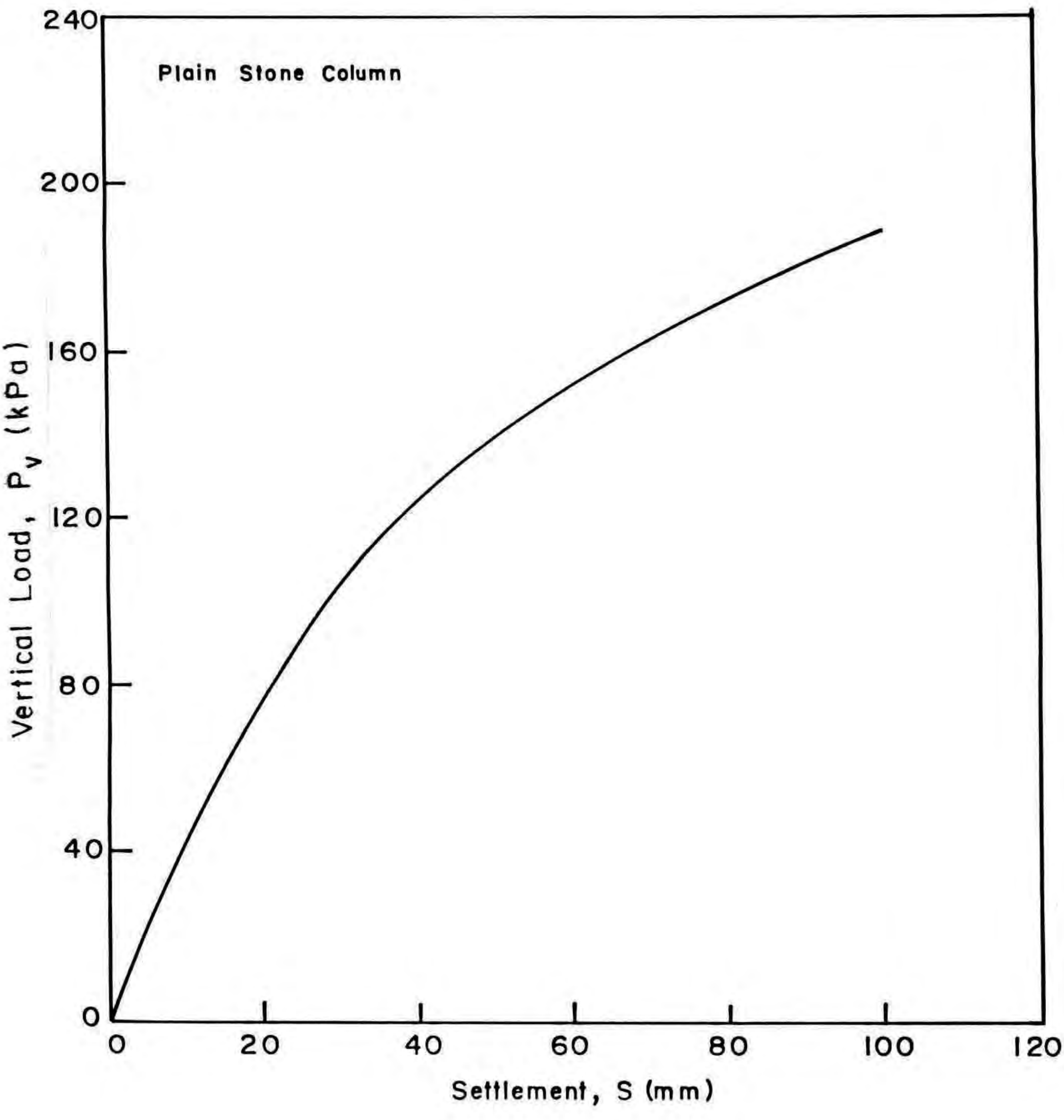


Fig. 5.3 Vertical load vs. settlement diagram for plain stone column without considering skin friction.

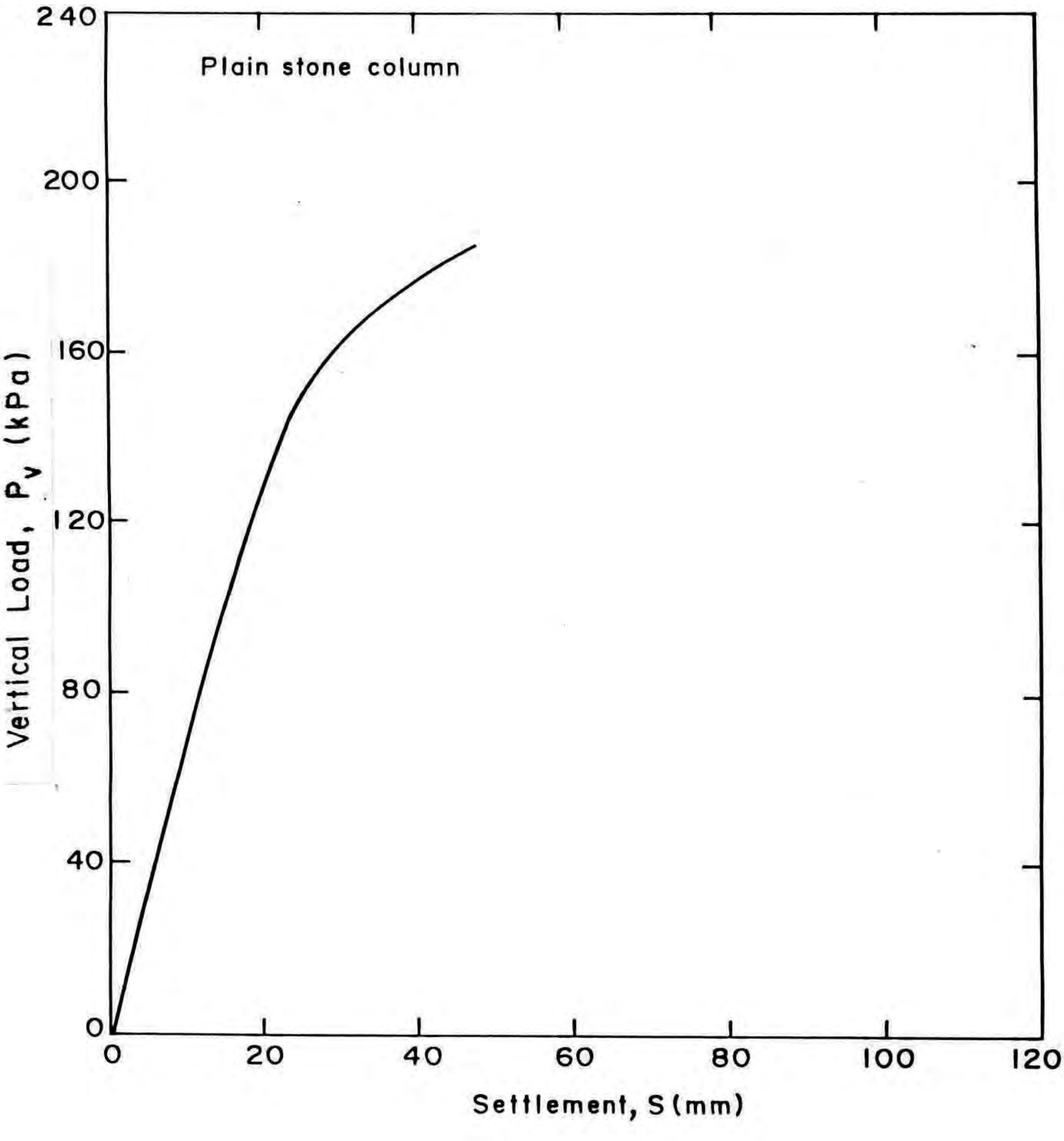


Fig. 5.4 Vertical load vs. settlement diagram for plain stone column considering skin friction.

5.7 DESIGN OF JACKETED STONE COLUMN

A jacketed stone column of similar dimension and materials as the plain stone column in Art.5.6 was considered. A geogrid jacket liner was assumed around the plain stone column to provide additional confining effect as described previously in Art.3.8.1. The property and design parameters of the geogrid is presented in Art.4.3.2. The relationships $p_h = f(\delta / r_o, Z)$, $Pv = f(\delta / r_o, Z)$ and $\varepsilon = f(Pv, Z)$ for this stone column were established and are presented in Tables.5.9, 5.10 and 5.11 respectively and Figs. 5.5, 5.6 and 5.7 respectively. The relevant pressure (Pv) versus settlement (S) relationship for this stone column without considering skin friction is presented in Table.5.12 and Fig.5.8. Considering skin friction by the methods described earlier in Arts.3.6.3.2 and 3.8.4, the relevant pressure (Pv) versus settlement (S) relation is established. This is presented in Table.5.17 and Fig.5.9. The results of appropriate trials at different pressures are presented in Tables. 5.13 through 5.16, from which Table.5.17 is established. The pressure at 50 mm settlement for the jacketed stone column was established as 258.90 kPa and 352.70 kPa respectively for cases without considering and considering skin friction. This is equivalent to loads of 50.80 kN and 69.30 kN respectively on the stone column.

5.8 PLAIN STONE COLUMN VERSUS JACKETED STONE COLUMN

Vertical load carrying capacity of plain and jacketed stone columns for the design example, described earlier, is

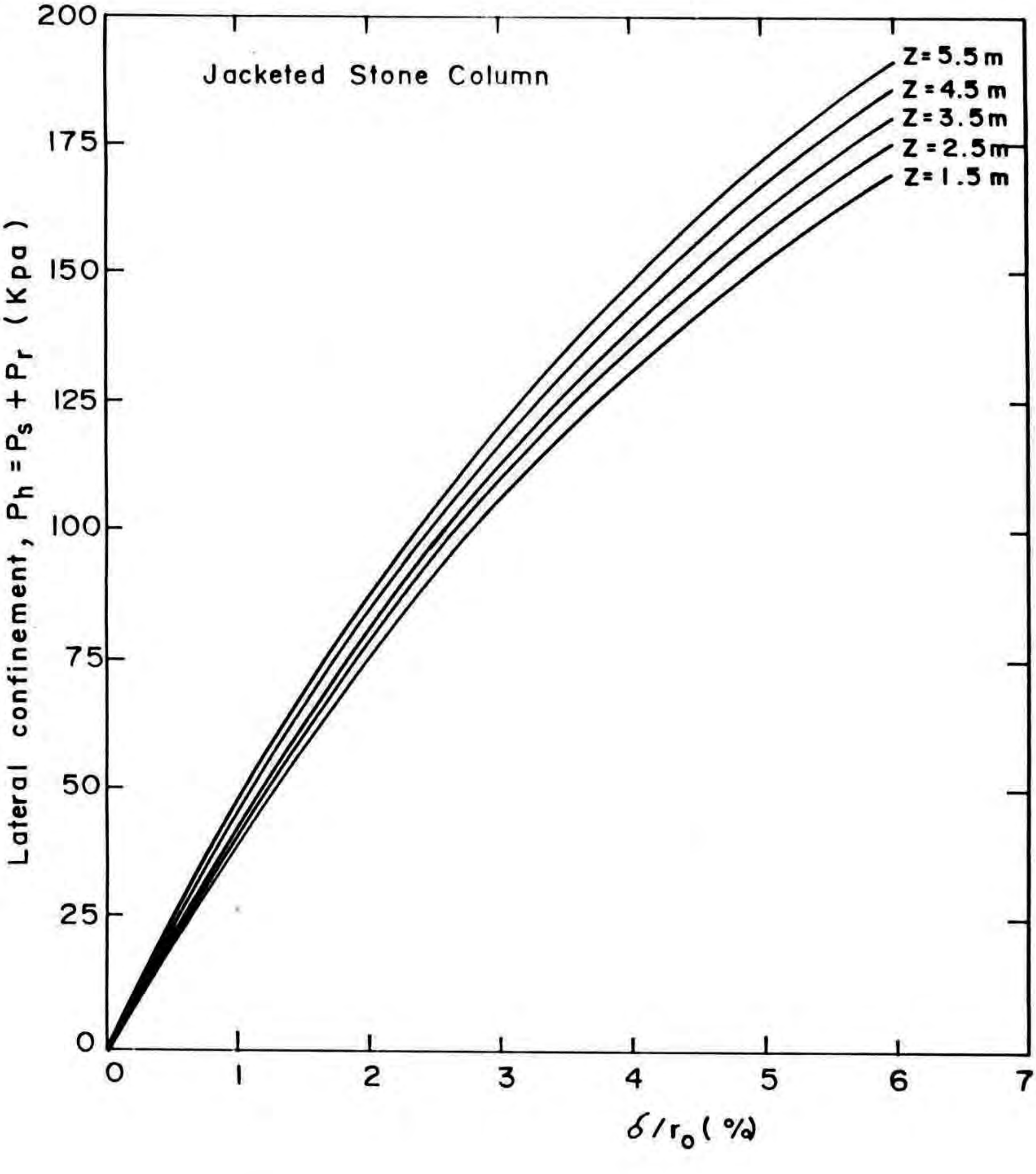


Fig. 5.5 Total lateral confinement vs. deform. ratio diagram of jacketed stone column.

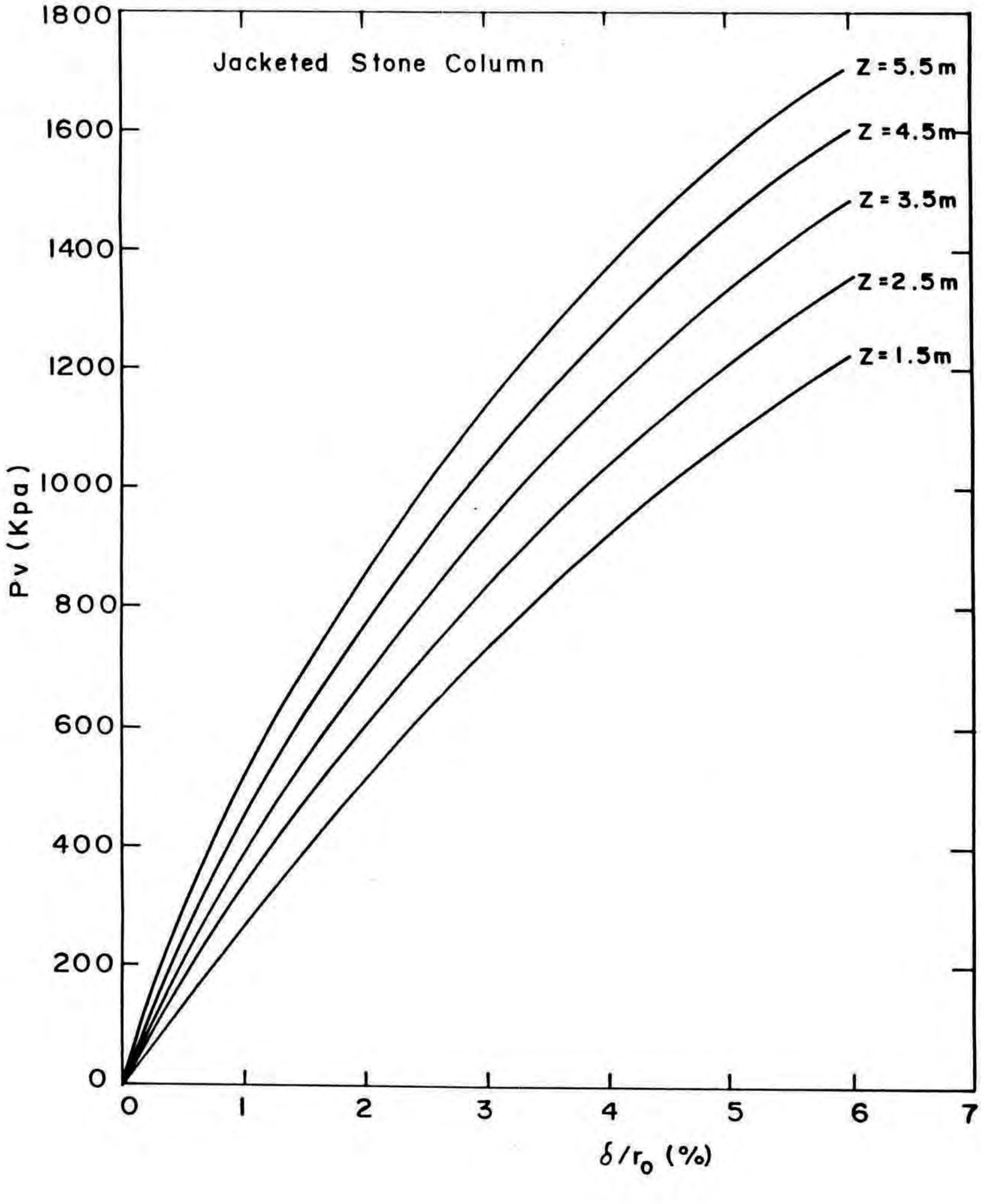


Fig. 5.6a Vertical load vs. deform. ratio at different depth of jacketed stone column.

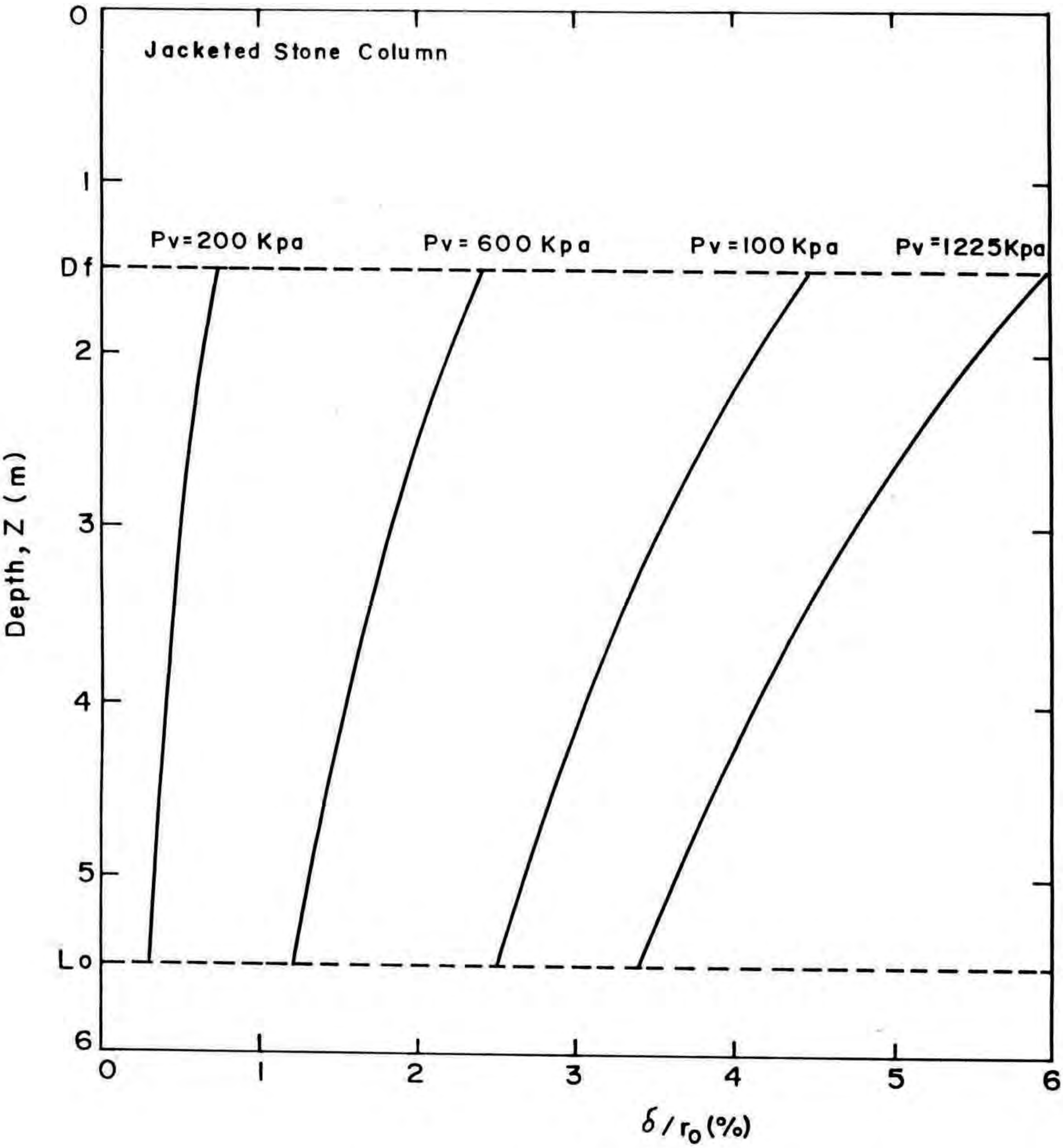


Fig. 5.6b Vertical load vs. deform. ratio at different depth of jacketed stone column.

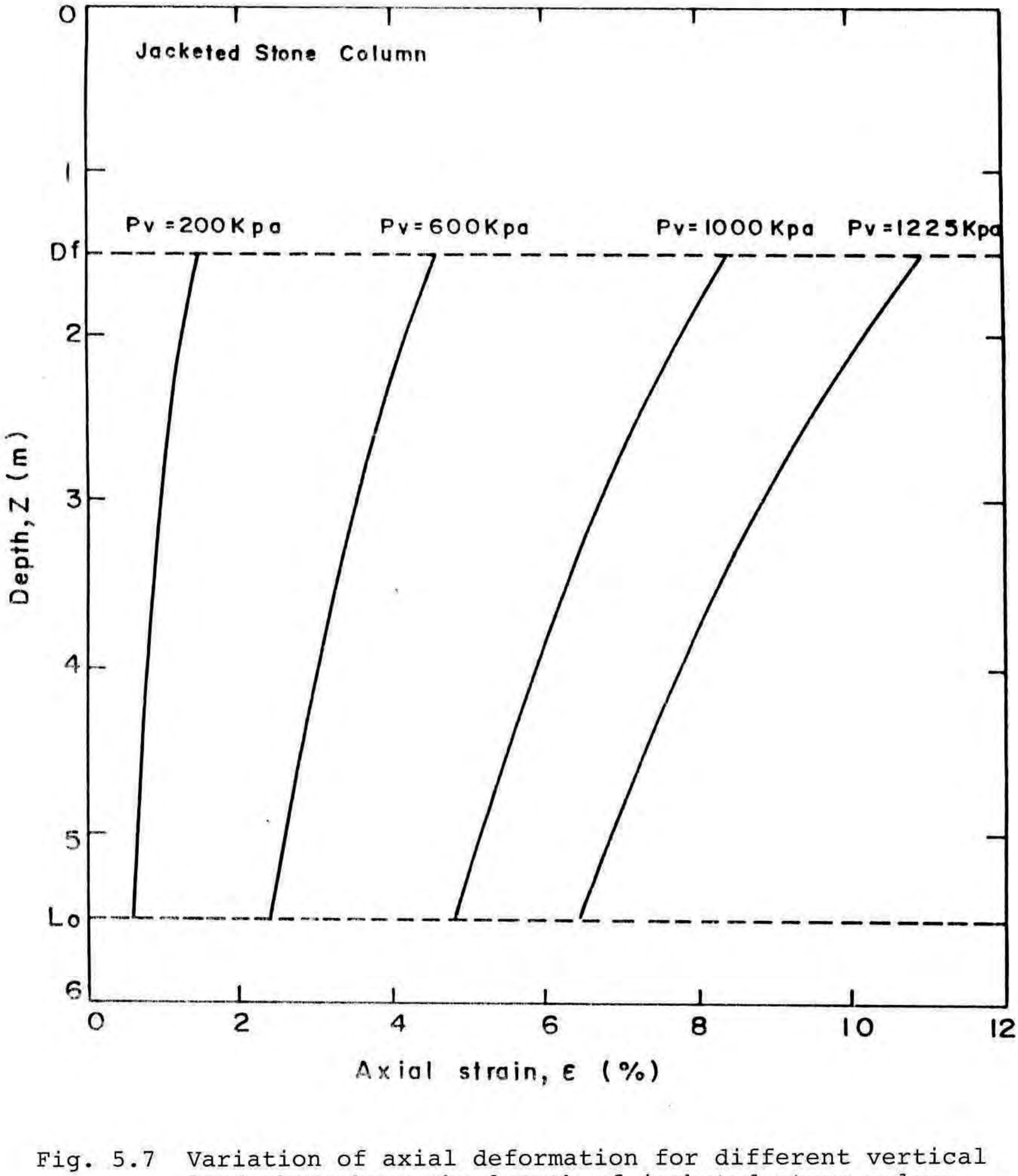


Fig. 5.7 Variation of axial deformation for different vertical load throughout the length of jacketed stone column.

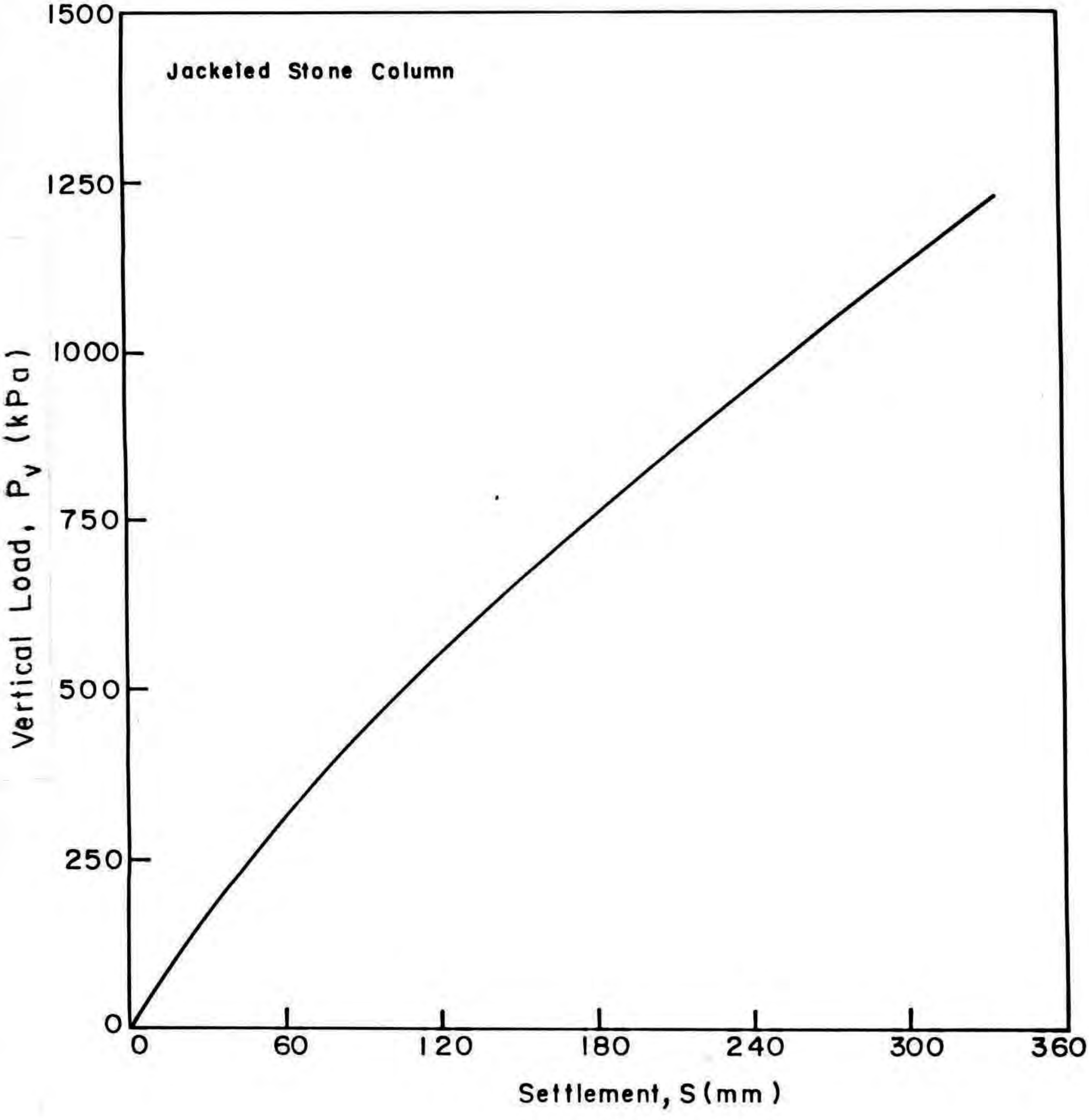


Fig. 5.8 Vertical load vs. settlement diagram for jacketed stone column without considering skin friction.

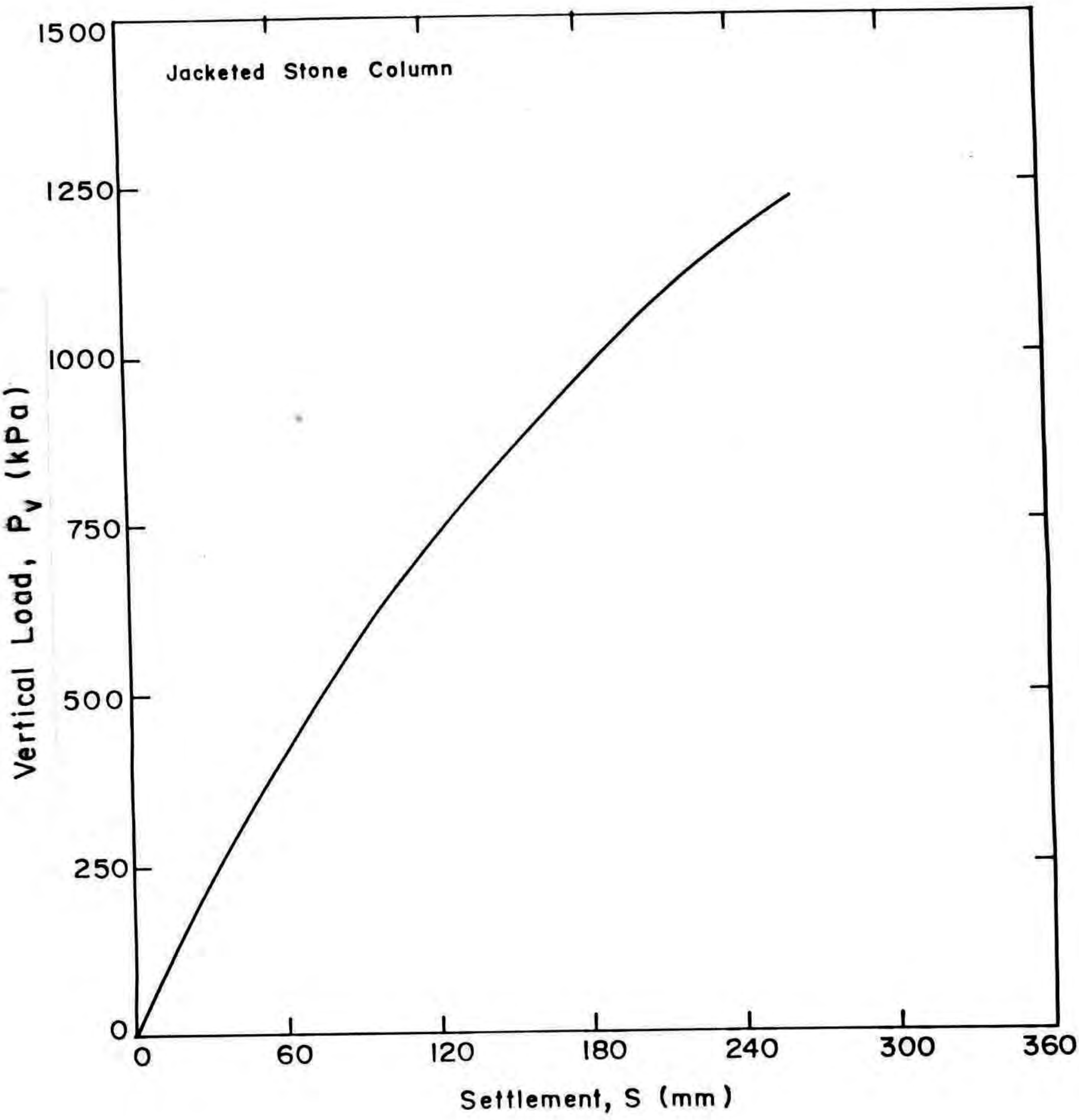


Fig. 5.9 Vertical load vs. settlement diagram for jacketed stone column considering skin friction.

compared in this section. The proposed nonlinear design methods, without considering and considering skin friction conditions, are used to evaluate the load carrying capacity. The vertical loads on plain and jacketedstone columns are found to be 26.90 kN and 50.80 kN respectively, when skin friction is neglected. When skin friction is considered, the vertical loads on plain and jacketed stone columns are found to be 36.30 kN and 69.30 kN respectively. From the above mentioned vertical loads on both stone columns, it can be concluded that the newly envisaged jacketed stone column system may be designed to provide with a much stiffer foundation compared with its plain counterpart. In case of jacketed stone column, the total lateral confinement comes from both reinforcing element and clay media. Here, it is found that the contribution of reinforcing element (geogrid) on lateral confinement is almost eighty percent (Table.5.18 and Fig.5.10).

5.9 COMPARISON OF PROPOSED METHOD WITH EXISTING METHODS

Vertical load carrying capacity of stone columns using the existing limit state methods and the proposed nonlinear method, is compared here. For comparison only plain stone column is considered because jacketed stone column is envisaged newly in this studies. The design example from which the values are evaluated is described earlier in this chapter. Evaluated lateral resistance and vertical loads (Art.5.6 and Appendix-1), using the existing as well as the proposed method are presented in Table.5.19. From this table it is found that there is a wide variation of load carrying capacity of plain stone column,

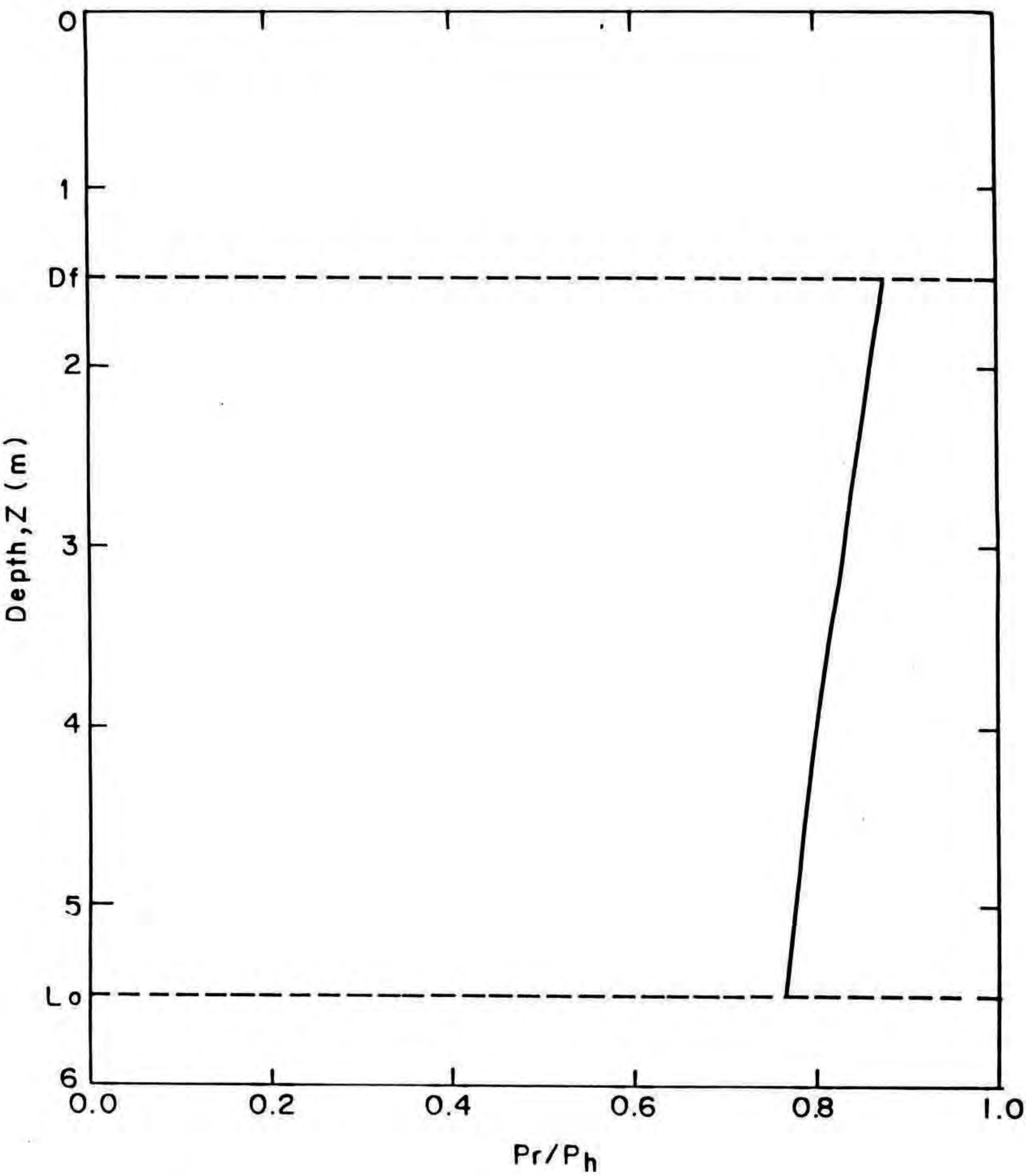


Fig. 5.10 Contribution of reinforcing element on lateral confinement over clay media for jacketed stone column.

obtained from different methods. Most of the existing methods predict much greater values than the proposed method. The reason is that, in the existing methods the ultimate strength values of each constituent materials are assumed to be mobilized fully. Whereas the values obtained by the proposed method is based on those mobilized at a vertical deformation of ten percent of the stone column diameter under full deformation compatibility condition. For the existing methods the N_{ϕ} (= 1/K) value of aggregates is 6.49, it was to be 3.83 by the proposed method. The mobilized lateral resistance from clay media varied from 28 to 84 kPa for the existing methods but it was found to be only 26 kPa in case of the proposed method. Therefore, the existing design methods will lead to unsafe design of stone column.

CHAPTER SIX

CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

The research reported in this thesis, is concerned about the development of nonlinear design method for plain and newly envisaged jacketed stone column foundations. Testing techniques have been developed to obtain the design parameters of aggregates. Data interpretation and analysis techniques have been developed to obtain design parameters of aggregates, geotextile or geogrid and clay. On the bases of the studies presented in this thesis, the following conclusions can be made:

i) In existing designs, single parameter representation of the mechanical behaviour of constituent materials are used. These for aggregates and clays are ϕ' and Su respectively. Design of stone columns based on these limit values are not adequate.

ii) The proposed analysis and design method is based on realistic nonlinear behaviour of constituent materials and compatibility of deformation amongst these and therefore, depicts the foundamental load transfer mechanism of the stone column foundation system.

iii) The existing limit state design method may lead to unsafe designs, specially at low level of deformation.

iv) In data analysis techniques using hyperbolic representation of stress strain behaviour of constituent materials such as aggregates, reinforcing element and clay are found to be of reasonable degree of accuracy.

v) The suction triaxial test, newly developed as part of this studies, was used satisfactorily for the characterisation of aggregates and was proved to be very beneficial specially in case of large size samples.

vi) The newly envisaged geogrid jacketed stone column system resulted in a much stiffer foundation compared with its plain counterpart.

vii) In this analysis the cavity expansion theory proposed by Ladanyi (1963) was used very successfully for the development of equations to predict lateral resistance mobilised by surrounding clay media of stone column.

6.2 RECOMMENDATIONS FOR FUTURE REASEARCH

Several aspects of the work presented in this thesis require further study. Some of the important areas of the further research may be listed as follows:

i) In this study nearly uniform size of aggregates are used and no volume change condition is adopted. It is requires for future researchers to work with different size of aggregates and to find out the proper grading of aggregates for which the capacity of stone column can be increased.

ii) The effect of specimen geometry, on the design parameters from suction triaxial test should be properly investigated as only an unique geometrical shape was considered in this studies.

iii) Design parameters and design charts can be developed for different types of soft soils and different types of jacketing elements such as jute which is locally available. This will be the interesting research work on this topics.

iv) Here it is assumed that stone column rests on the rigid strata. Further studies regarding the different conditions such as floating stone column will be very beneficial.

iv) Research work is to be performed considering the effect of creep behaviour of the constituent materials on the load carrying capacity of stone column.

v) It will be the interesting future research work on this thesis to perform a field load test considering all assumptions taken in the study and to draw a comparison between theoretical and field load-settlement diagram.

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TABLES

Table: 4.1 Conventional Triaxial Test on Sand.

Sample type : sand   Weight of sample : 150 lbs.
Dia of sample : 38 mm   Confining pressure (σ_3): 62 Kpa
Height of sample: 88 mm

Deformation dial 1 div. = .01 mm	Load dial 1 div.=1 lb	Axial strain ε (%)	Deviator stress ($\sigma_1 - \sigma_3$) Kpa
0	0.0	0.0	0.0
25	8.0	0.28	31.27
50	14.0	0.56	54.56
75	18.5	0.85	71.89
100	22.0	1.14	85.24
125	26.5	1.42	102.38
150	29.0	1.70	111.73
175	31.0	1.99	119.08
200	32.5	2.27	124.48
225	34.0	2.56	129.84
250	36.0	2.84	137.08
275	37.5	3.125	142.38
300	38.5	3.41	145.74
350	41.0	3.98	154.29
400	43.0	4.55	160.86
450	44.5	5.11	165.49
500	46.0	5.68	170.04
600	48.2	6.82	176.02
700	50.0	7.95	180.38
800	51.0	9.01	181.87
900	53.0	10.23	186.47
1000	54.0	11.36	187.60
1100	55.0	12.50	188.61
1200	55.0	13.64	186.16

Table: 4.2 Suction Triaxial Test on Sand.

Sample type : sand
Dia of sample : 38 mm
Height of sample: 88 mm
Weight of sample : 150 lbs.
Confining pressure (σ_3): 62 Kpa

Deformation dial 1 div. = .01 mm	Load dial 1 div.=1 lb	Axial strain ε (%)	Deviator stress (σ_1 - σ_3) Kpa
0	0.0	0.0	0.0
25	7.0	0.28	27.36
50	12.2	0.56	47.55
75	16.2	0.85	62.95
100	21.0	1.14	81.36
125	24.0	1.42	92.738
150	27.5	1.70	105.95
175	31.0	1.99	119.08
200	33.0	2.27	126.40
225	36.0	2.56	137.48
250	38.0	2.84	144.70
275	40.0	3.125	151.87
300	41.6	3.41	157.48
350	44.8	3.98	168.59
400	47.0	4.55	175.82
450	49.5	5.11	184.09
500	50.8	5.68	187.79
600	52.2	6.82	190.63
700	53.0	7.95	191.21
800	53.8	9.01	191.86
900	54.0	10.23	189.99
1000	54.0	11.36	187.60
1100	55.0	12.50	181.75
1200	53.0	13.64	179.39

Table : 4.3 Suction Triaxial Test of Stone Aggregates.

Aggregates type : Stone aggregates
Aggregates size : 9 mm to 12 mm
Diameter of sample : 100 mm
Hieght of sample : 200 mm

Deformation dial reading (1 div. = .001 in)	Load dial reading (1 div. = 1 lb)		
	σ_3= 17 Kpa	σ_3= 34 Kpa	σ_3= 69 Kpa
0	0	0	0
10	6	10	15
20	20	50	80
40	43	90	150
60	58	120	250
80	76	150	310
100	85	180	335
120	91	195	360
140	102	210	400
160	110	225	420
180	120	240	450
200	130	250	468
240	135	258	490
280	140	266	525
320	145	281	560
360	149	290	575
400	153	300	590
440	156	305	620
480	160	315	650
520	166	325	370
560	168	335	680
600	170	342	685
700	190	385	765
800	192	395	770
1000	192	400	780
1100		410	800
1200		410	800

Table : 4.4 Suction Triaxial Test of Stone Aggregates.

Aggregates type : stone aggregates
Aggregates size : 9 mm to 12 mm
Diameter of sample : 100 mm
Height of sample : 200 mm

Axial deformation (inch)	Axial strain ε (%)	Corrected area (sq. in)	Deviator stress ($\sigma_1 - \sigma_3$) Kpa σ_3=17 Kpa (2.5 psi)	σ_3=34 Kpa (5 psi)	σ_3=69 Kpa (10 psi)
0	0	0	0	0	0
0.01	0.125	12.58	3.28	5.47	8.21
0.04	0.50	12.63	23.45	49.10	81.80
0.08	1.00	12.70	41.24	81.40	135.66
0.10	1.25	12.73	46.01	97.43	181.33
0.12	1.50	12.76	49.13	105.30	194.37
0.16	2.00	12.83	59.01	120.86	225.61
0.20	2.50	12.89	69.47	133.60	250.11
0.24	3.00	12.93	71.78	137.17	260.53
0.32	4.00	13.09	76.30	147.86	294.67
0.40	5.00	13.23	79.67	156.22	307.23
0.52	6.50	13.44	85.07	166.56	343.40
0.56	7.00	13.52	85.64	170.77	346.60
0.60	7.50	13.59	86.19	173.40	347.31
0.70	8.75	13.78	95.03	192.56	382.63
0.80	10.00	13.97	94.69	194.81	379.76
1.00	12.50	13.36	92.08	191.84	374.09
1.10	13.75	14.57		193.80	378.21
1.20	15.00	14.79		191.00	372.68

Table : 4.5 Suction Triaxial Test of Stone Aggregates.

Aggregates type : stone aggregates
Aggregates size : 9 mm t0 12 mm
Diameter of sample : 100 mm
Height of sample : 200 mm

Axial strain ε (%)	$(\sigma_1 - \sigma_3)/\sigma_3$		
	σ_3 = 17 Kpa	σ_3 = 34 Kpa	σ_3 = 69 Kpa
0	0	0	0
0.125	0.19	0.160	0.12
0.50	1.36	1.42	1.19
1.00	2.39	2.36	1.97
1.25	2.67	2.83	2.63
1.50	2.85	3.06	2.82
2.00	3.42	3.51	3.27
2.50	4.03	3.88	3.63
3.00	4.17	3.98	3.78
4.00	4.41	4.29	4.28
5.00	4.62	4.53	4.46
6.50	4.94	4.83	4.98
7.00	4.97	4.96	5.03
7.50	5.00	5.03	5.04
8.75	5.52	5.59	5.55
10.00	5.49	5.65	5.51
12.50	5.34	5.57	5.43
13.75		5.75	5.49
15.00		5.54	5.41

Table: 4.6 Suction Triaxial Test of Stone Aggregates.

From graph ($\sigma_1 - \sigma_3$) / σ vs. ε		$\delta_r = \delta / r_o$ (%)	δ_r / σ_r
Axial strain ε (%)	$\sigma_r = (\sigma_1 - \sigma_3)/\sigma_3$		
1.0	2.30	0.504	2.19
2.0	3.36	1.015	3.02
3.0	4.00	1.535	3.84
4.0	4.40	2.062	4.69
5.0	4.65	2.598	5.59
6.0	4.93	3.142	6.37
7.0	5.10	3.695	7.25
8.0	5.25	4.257	8.148
9.0	5.40	4.828	8.94
10.0	5.47	5.409	9.89
11.0	5.50	6.00	10.91
12.0	5.49	6.60	12.02

Table: 4.7 Suction Triaxial Test of Stone Aggregates.

Dimensionless deformation ratio, δ/r_o (%)	K (Equation)	K (Test)
0.0	1.00	1.00
0.25	0.425	0.480
0.50	0.310	0.303
0.75	0.261	0.261
1.00	0.234	0.229
1.50	0.204	0.201
2.00	0.189	0.185
2.50	0.179	0.177
3.00	0.172	0.169
4.00	0.164	0.163
5.00	0.159	0.156
6.00	0.155	0.154
6.60	0.154	0.154

Table: 4.8 Load-Strain Data of Reinforcing Element.

Axial strain ε (%)	Load Pr (kN/m)		ε/Pr (%)	
	Geogrid	Geotextile	Geogrid	Geotextile
0.0	0.0	0.0	0.0	0.0
0.10	1.20	1.10	0.083	0.091
0.25	2.70	2.75	0.0926	0.091
0.50	5.25	5.50	0.095	0.091
1.00	10.00	11.00	0.100	0.091
2.00	18.40	21.60	0.109	0.092
4.00	31.25	44.30	0.128	0.0903
6.00	40.50	66.20	0.148	0.0906
8.00	47.30	88.20	0.169	0.091
10.00	52.50	110.50	0.190	0.090
12.00	56.50	132.00	0.212	0.091
14.00	59.40	145.00	0.236	0.096
16.00	61.75	170.00	0.259	0.094
18.00	63.60	182.00	0.283	0.098
20.00	65.00	190.00	0.308	0.10

Table: 4.9 Data Calculation for Reinforcing Element.

Axial strain ε (%) or Deform. ratio δ/r_o (%)	$P_r\, r_o$(KN/m)		P_r (kPa) for r_o= 0.25 m	
	Geogrid	Geotextile	Geogrid	Geotextile
0.0	0.0	0.0	0.0	0.0
0.10	1.10	1.10	4.40	4.39
0.25	2.71	2.74	10.84	10.96
0.50	5.28	5.47	21.12	21.88
1.00	10.07	10.90	40.28	43.60
2.00	18.38	21.55	73.52	86.20
4.00	31.19	42.27	24.76	169.08
6.00	40.44	62.20	161.76	148.80
8.00	47.29	81.40	189.16	325.60
10.00	52.46	99.90	209.84	225.60
12.00	56.40	117.74	225.60	470.96
14.00	59.44	134.95	237.76	539.80
16.00	61.77	151.57	247.08	606.28
18.00	63.58	167.63	254.32	670.52
20.00	64.95	183.15	259.80	732.60

Table: 4.10 Consolidated Undrained Triaxial Test of Clay.

Axial strain ε (%)	Deviator stress ($\sigma_1 - \sigma_3$) kpa			$q_r = q/\sigma_3$, $q = (\sigma_1 - \sigma_3)$		
	σ_3=40 Kpa	σ_3=58 Kpa	σ_3=80 Kpa	σ_3=40 Kpa	σ_3=58 Kpa	σ_3=80 Kpa
0.0	0.0	0.0	0.0	0.0	0.0	0.0
2.0	9.0	13.0	18.00	0.225	0.224	0.225
4.0	12.50	18.25	25.00	0.3125	0.314	0.3125
6.0	15.00	21.75	30.00	0.375	0.375	0.375
8.0	17.00	24.60	34.00	0.425	0.424	0.425
10.0	17.90	26.00	35.80	0.4475	0.448	0.447
12.0	19.30	28.00	38.50	0.4825	0.483	0.4812

Table: 4.11 Calculated Consolidated Undrained Triaxial Test Data of Clay.

Axial strain, ε (%)	Shear strain, γ_s(%)	$q_r = q/\sigma_3$	γ_s / q_r
0.0	0.0	0.0	
2.0	3.0	0.224	0.134
4.0	6.0	0.310	0.194
6.0	9.0	0.374	0.241
8.0	12.0	0.424	0.283
10.0	15.0	0.448	0.335
12.0	18.0	0.483	0.373

Table: 4.12 Lateral Resistance Developed in the Surrounding Clay Media of Stone Column.

Deformation ratio δ/r_o (%)	p_s/γ_s	p_s (Kpa) Z=1.5 m	Z=2.5 m	Z=3.5 m	Z=4.5 m	Z=5.5 m
0.0	0.0	0.0	0.0	0.0	0.0	0.0
1.0	0.09	2.23	3,71	5.20	6.68	8.17
2.0	0.156	3.86	6.44	9.01	11.58	14.16
3.0	0.209	5.17	8.62	12.07	15.50	18.97
4.0	0.253	6.26	10.44	14.61	18.79	22.96
5.0	0.290	7.18	11.97	16.76	21.53	26.30
6.0	0.323	7.99	13.23	18.65	23.98	29.31
8.0	0.376	9.31	15.51	21.71	27.92	34.12
10.0	0.419	11.26	17.28	24.20	31.11	38.02
12.0	0.455	11.26	18.77	26.28	33.78	41.29

Table: 4.13 Skin Friction (τ_S) Versus Settltement (S) and Frictional Force (P) Versus Settlement (S) Relation for d = 500 mm and L = 4 m

Settlement S(mm)	Skin friction (kPa)	Frictional force P_f (kPa)
0.0	0.0	0.0
1.0	0.416	2.613
2.0	0.832	5.226
5.0	2.08	13.064
10.0	4.16	26.13
15.0	6.24	39.19
20.0	8.32	52.26
30.0	12.48	78.38
33.65	14.0	87.92
40.0	14.0	87.92
50.0	14.0	87.92

Table: 5.1 Calculated Value of P_v for Plain Stone Column.

δ/r_o (%)	P_v (Kpa)				
	Z = 1.5 m	Z = 2.5 m	Z = 3.5 m	Z = 4.5 m	Z = 5.5 m
1.0	90.802	151.336	211.871	272.405	332.939
2.0	127.176	211.959	296.743	381.527	466.310
3.0	149.103	248.505	347.907	447.309	546.711
4.0	164.588	274.313	384.039	493.764	603.490
5.0	176.464	294.106	411.749	529.391	647.034
6.0	186.040	310.067	434.094	558.120	682.147

Table: 5.2 Deformation Ratio Strain for Different Value of P_v at Different Depth (For Plain Stone Column).

Depth Z (m)	Dimensionless deformation ratio, δ/r_o (%)				
	P_v =0.0	P_v =50 Kpa	P_v =100 Kpa	P_v =150 Kpa	P_v =185 Kpa
1.5	0.0	0.50	1.20	2.55	6.00
2.5	0.0	0.20	0.50	1.00	1.45
3.5	0.0	0.10	0.225	0.50	0.75
4.5	0.0	0.05	0.125	0.28	0.45
5.5	0.0	0.03	0.05	0.10	0.25

Depth Z (m)	Axial strain, ε (%)				
	P_v =0.0	P_v =50 Kpa	P_v =100 Kpa	P_v =150 Kpa	P_v =185 Kpa
1.5	0.0	0.99	2.34	4.91	11.00
2.5	0.0	0.40	0.99	1.97	2.84
3.5	0.0	0.20	0.45	0.99	1.48
4.5	0.0	0.10	0.25	0.56	0.90
5.5	0.0	0.06	0.10	0.32	0.50

Table: 5.3 Calculated Load-Settlement Values for Plain Stone Column Without Considering Skin Friction

Vertical load Pv (kPa)	0.0	50	100	150	185
Settlement S (mm)	0.0	11.50	27.67	57.80	98.07

Table: 5.4 Calculation of Vertical Load (Pv) and Settlement (S) (Plain Stone Column).

Vertical load (Pv) = 50 kPa i.e. 9.815 KN
Skin friction (τ_s) = 0
Settlement (S) = 11.50 mm

<u>1st</u> <u>Trial</u>, Considering skin friction

Z(m)	s(mm)	R/Ru	τs(kPa)	P_f(kN)	Pv(kN)	Pv(kPa)	ε(%)	S(mm)
1.5					9.82	50.00	0.99	
2.5	6.95	<1.0	2.89	4.54	5.28	26.90	0.20	
3.5	3.00	<1.0	1.25	1.96	3.32	16.91	0.10	7.30
4.5	1.50	<1.0	0.62	0.98	2.34	11.92	0.05	
5.5	0.80	<1.0	0.33	0.52	1.81	9.22	0.01	

<u>2nd</u> <u>Trial</u>, Considering skin friction

Z(m)	s(mm)	R/Ru	τs(kPa)	P_f(kN)	Pv(KN)	Pv(kPa)	ε (%)	S(mm)
1.5					9.82	50.00	0.99	
2.5	5.95	<1.0	2.48	3.89	5.93	30.21	0.21	
3.5	1.50	<1.0	0.624	0.98	2.90	14.77	0.12	7.60
4.5	0.75	<1.0	0.312	0.49	2.41	12.28	0.05	
5.5	0.30	<1.0	0.125	0.20	2.22	11.31	0.01	

Table: 5.5 Calculation of Vertical Load (Pv) and Settlement (S) (Plain Stone Column).

Vertical load (Pv) = 100 kPa i.e. 19.63 kN
Skin friction (τ_s) = 0
Settlement (S) = 27.67 mm

1st Trial, Considering skin friction

Z(m)	s(mm)	R/Ru	τ_s(kPa)	P_f(kN)	Pv(kN)	Pv(kPa)	ε (%)	S(mm)
1.5					19.63	100.0	2.34	
2.5	16.65	<1.0	6.93	10.88	8.75	44.60	0.35	
3.5	7.20	<1.0	2.99	4.70	4.05	20.63	0.10	13.80
4.5	3.50	<1.0	1.46	2.29	1.76	8.97	0.05	
5.5	1.75	<1.0	0.73	1.14	0.62	3.16	0.01	

2nd Trial, Considering skin friction

Z(m)	s(mm)	R/Ru	τ_s(kPa)	P_f(kN)	Pv(KN)	Pv(kPa)	ε (%)	S(mm)
1.5					19.63	100.0	2.34	
2.5	13.45	<1.0	5.60	8.80	10.83	55.15	0.45	
3.5	2.25	<1.0	0.94	1.47	9.35	47.63	0.15	15.90
4.5	0.75	<1.0	0.312	0.49	8.87	45.18	0.08	
5.5	0.30	<1.0	0.125	0.196	8.67	44.17	0.01	

3rd Trial , Considering skin friction

Z(m)	s(mm)	R/Ru	τ_s(kPa)	P_f(kN)	Pv(kN)	Pv(kPa)	ε (%)	S(mm)
1.5					19.63	100.0	2.34	
2.5	13.95	<1.0	5.80	9.12	10.51	53.54	0.40	
3.5	3.00	<1.0	1.25	1.96	8.55	43.56	0.14	14.90
4.5	1.15	<1.0	0.48	0.75	7.80	39.74	0.06	
5.5	0.45	<1.0	0.19	0.29	7.51	38.26	0.01	

4th Trial, Considering skin friction

Z(m)	s(mm)	R/Ru	τ_s(kPa)	P_f(kN)	Pv(KN)	Pv(kPa)	ε (%)	S(mm)
1.5					19.63	100.0	2.34	
2.5	13.70	<1.0	5.70	8.95	10.68	54.40	0.40	
3.5	2.70	<1.0	1.12	1.76	8.92	45.40	0.14	15.00
4.5	1.00	<1.0	0.416	0.65	8.27	42.13	0.07	
5.5	0.35	<1.0	0.146	0.23	8.04	40.96	0.01	

Table: 5.6 Calculation of Vertical Load (Pv) and Settlement (S) (Plain Stone Column).

Vertical load (Pv) = 150 kPa i.e. 29.45 kN
Skin friction (τ_s) = 0
Settlement (S) = 57.80 mm

<u>1st</u> <u>Trial</u> , Considering skin friction

Z(m)	s(mm)	R/Ru	τs(kPa)	P_f (kN)	Pv(kN)	Pv(kPa)	ε (%)	S(mm)
1.5					29.45	150.0	4.91	
2.5	34.40	1.0	14.00	21.99	7.46	38.00	0.30	
3.5	14.80	<1.0	6.16	9.67	0.0	0.0	0.0	27.55
4.5	7.75	<1.0	3.22	5.06	0.0	0.0	0.0	
5.5	4.40	<1.0	1.83	2.88	0.0	0.0	0.0	

<u>2nd</u> <u>Trial</u>, Considering skin friction

Z(m)	s(mm)	R/Ru	τs(kPa)	P_f (kN)	Pv(KN)	Pv(kPa)	ε(%)	S(mm)
1.5					29.45	150.0	4.91	
2.5	26.06	<1.0	10.84	17.02	12.43	63.00	0.50	
3.5	1.50	<1.0	0.62	0.98	11.45	58.00	0.20	25.77
4.5	0.0		0.0	0.0	11.45	58.00	0.10	
5.5	0.0		0.0	0.0	11.45	58.00	0.02	

<u>3rd</u> <u>Trial</u> , Considering skin friction

Z(m)	s(mm)	R/Ru	τ s(kPa)	P_f (kN)	Pv(kN)	Pv(kPa)	ε(%)	S(mm)
1.5					29.45	150.0	4.91	
2.5	27.05	<1.0	11.25	17.67	11.78	60.00	0.50	
3.5	3.50	<1.0	1.46	2.29	9.49	48.00	0.15	25.27
4.5	1.50	<1.0	0.62	0.98	8.51	43.00	0.09	
5.5	0.60	<1.0	0.25	0.39	8.12	41.00	0.01	

Table: 5.7 Calculation of Vertical Load (Pv) and Settlement (S) (Plain Stone Column).

Vertical load (Pv) = 185 kPa i.e. 36.30 kN
Skin friction (τ_s) = 0
Settlement (S) = 98.07 mm

<u>1st Trial</u>, Considering skin friction

Z(m)	s(mm)	R/Ru	τs(kPa)	P_f(kN)	Pv(kN)	Pv(kPa)	ε(%)	S(mm)
1.5					36.30	185.0	11.00	
2.5	69.20	1.0	14.00	21.99	14.31	14.31	0.65	
3.5	21.60	<1.0	8.99	14.11	0.20	1.02	0.01	45.40
4.5	11.90	<1.0	4.95	7.78	0.0	0.0	0.0	
5.5	7.00	<1.0	2.91	4.57	0.0	0.0	0.0	

<u>2nd Trial</u>, Considering skin friction

Z(m)	s(mm)	R/Ru	τs(kPa)	P_f(kN)	Pv(KN)	Pv(kPa)	ε (%)	S(mm)
1.5					36.30	185.0	11.00	
2.5	58.25	1.0	14.00	21.99	14.31	73.00	0.60	
3.5	3.30	<1.0	1.37	2.16	12.15	62.00	0.20	47.50
4.5	0.05	<1.0	0.02	0.03	12.12	61.00	0.10	
5.5	0.0		0.0	0.0	12.12	61.00	0.05	

<u>3rd Trial</u>, Considering skin friction

Z(m)	s(mm)	R/Ru	τs(kPa)	P_f(kN)	Pv(kN)	Pv(kPa)	ε (%)	S(mm)
1.5					36.30	185.0	11.00	
2.5	58.00	1.0	14.0	21.99	14.31	72.90	0.60	
3.5	4.00	<1.0	1.66	2.61	11.70	59.60	0.18	47.33
4.5	1.50	<1.0	0.624	0.98	10.72	54.61	0.10	
5.5	0.75	<1.0	0.312	0.49	10.23	52.11	0.04	

Table: 5.8 Calculated Load-Settlement Values for Plain Stone Column Considering Skin Friction

Total vertical load, Pv (kPa)	0.0	50.0	100.0	150.0	185.0
Settlement, S (mm)	0.0	7.50	15.00	25.50	47.50

Table: 5.9 Lateral Resistance form Clay and Geogrid with Respect to Deformation Ratio and Depth of Column (Jacketed Stone Column).

Deformation ratio δ/r_o(%)	Lateral resistance, $p_h = p_r + p_s$ (kPa) Z = 1.5 m	Z = 2.5 m	Z = 3.5 m	Z = 4.5 m	Z = 5.5 m
1.0	42.533	44.037	45.541	47.046	48.550
2.0	77.443	80.050	82.656	85.262	87.869
3.0	106.542	110.025	113.508	116.991	120.474
4.0	131.065	135.273	139.481	143.689	147.897
5.0	151.918	156.741	161.564	166.387	171.210
6.0	169.781	175.137	180.492	185.847	191.203

Table: 5.10 Calculated P_v for Different Depth and Deformation Ratio (Jacketed Stone Column).

Deformation ratio δ/r_o (%)	Vertical load, P_v (kPa) Z = 1.5 m	Z = 2.5 m	Z = 3.5 m	Z = 4.5 m	Z = 5.5 m
1.0	263.16	323.67	384.20	444.74	505.27
2.0	516.99	601.77	686.55	771.34	856.12
3.0	736.74	836.15	935.55	1034.95	1134.35
4.0	925.03	1034.75	1144.48	1254.21	1363.93
5.0	1086.67	1204.31	1321.96	1439.60	1557.24
6.0	1226.06	1350.09	1474.11	1598.14	1722.17

Table: 5.11 Deformation Ratio and Axial Strain for Different P_v at Different Depth (Jacketed Stone Column).

Depth	Dimensionless deformation ratio , δ/r_o (%)				
Z (m)	P_v =0.0	P_v =200 Kpa	P_v =600 Kpa	P_v =1000 Kpa	P_v =1225 Kpa
1.5	0.0	0.74	2.40	4.50	6.00
2.5	0.0	0.56	2.00	3.80	5.15
3.5	0.0	0.48	1.70	3.30	4.40
4.5	0.0	0.40	1.43	2.85	3.85
5.5	0.0	0.30	1.22	2.50	3.40

Depth	Axial strain, ε (%)				
Z (m)	P_v =0.0	P_v =200 Kpa	P_v =600 Kpa	P_v =1000 Kpa	P_v =1225 Kpa
1.5	0.0	1.46	4.63	8.43	11.0
2.5	0.0	1.11	3.88	7.19	9.56
3.5	0.0	0.96	3.32	6.29	8.25
4.5	0.0	0.80	2.80	5.47	7.28
5.5	0.0	0.60	2.40	4.82	6.47

Table: 5.12 Calculated Load-Settlement Values for Jacketed Stone Column Without considering Skin Friction.

Vertical load Pv (kPa)	0.0	200	600	1000	1225
Settlement S (mm)	0.0	38.73	134.63	254.90	337.77

Table: 5.13 Calculation of Vertical Load (Pv) and Settlement (S) (Jacketed Stone Column).

Vertical load (Pv) = 200 kPa i.e. 39.26 kN
Skin friction (τs) = 0
Settlement (S) = 38.73 mm

<u>1st</u> <u>Trial</u> , Considering skin friction

Z(m)	s(mm)	R/Ru	τ s(kPa)	P_f (kN)	Pv(kN)	Pv(kPa)	ε (%)	S(mm)
1.5					39.26	200.0	1.46	
2.5	12.85	<1.0	5.30	8.40	30.86	157.0	0.87	
3.5	10.35	<1.0	4.30	6.76	24.10	123.0	0.55	25.33
4.5	8.80	<1.0	3.70	5.75	18.35	93.0	0.35	
5.5	7.00	<1.0	2.90	4.57	13.78	70.0	0.16	

<u>2nd</u> <u>Trial</u>, Considering skin friction

Z(m)	s(mm)	R/Ru	τ s(kPa)	P_f (kN)	Pv(KN)	Pv(kPa)	ε (%)	S(mm)
1.5					39.26	200.0	1.46	
2.5	11.65	<1.0	4.85	7.61	31.65	161.0	0.89	
3.5	7.70	<1.0	3.20	5.03	26.62	136.0	0.58	27.60
4.5	4.50	<1.0	1.87	2.94	23.68	120.0	0.45	
5.5	2.55	<1.0	1.06	1.67	22.01	112.0	0.30	

<u>3rd</u> <u>Trial</u> , Considering skin friction

Z(m)	s(mm)	R/Ru	τ s(kPa)	P_f (kN)	Pv(kN)	Pv(kPa)	ε (%)	S(mm)
1.5					39.26	200.0	1.46	
2.5	11.75	<1.0	4.89	7.68	31.58	161.0	0.89	
3.5	7.35	<1.0	3.06	4.80	26.78	136.0	0.58	27.40
4.5	5.15	<1.0	2.14	3.36	23.42	119.0	0.44	
5.5	3.75	<1.0	1.56	2.45	20.97	107.0	0.28	

Table: 5.14 Calculation of Vertical Load (Pv) and Settlement (S) (Jacketed Stone Column).

Vertical load (Pv) = 600 kPa i.e. 117.78 kN
Skin friction (τ_s) = 0
Settlement (S) = 134.68 mm

1st Trial , Considering skin friction

Z(m)	s(mm)	R/Ru	τ s(kPa)	P_f(kN)	Pv(kN)	Pv(kPa)	ε (%)	S(mm)
1.5					117.78	600.0	4.63	
2.5	42.55	1.0	14.00	21.99	95.79	488.0	3.03	
3.5	36.00	1.0	14.00	21.99	73.80	376.0	2.92	85.67
4.5	30.60	<1.0	12.43	20.00	53.80	274.0	1.14	
5.5	26.00	<1.0	10.82	16.99	36.81	187.0	0.55	

2nd Trial, Considering skin friction

Z(m)	s(mm)	R/Ru	τ s(kPa)	P_f(kN)	Pv(KN)	Pv(kPa)	ε (%)	S(mm)
1.5					117.78	600.0	4.63	
2.5	38.30	1.0	14.00	21.99	95.79	488.0	3.03	
3.5	24.75	<1.0	10.29	16.17	79.62	406.0	2.12	94.00
4.5	25.03	<1.0	6.36	10.00	69.62	354.7	1.53	
5.5	8.45	<1.0	3.51	5.52	64.00	326.5	1.09	

3rd Trial , Considering skin friction

Z(m)	s(mm)	R/Ru	τ s(kPa)	P_f(kN)	Pv(kN)	Pv(kPa)	ε (%)	S(mm)
1.5					117.78	600.0	4.63	
2.5	38.30	1.0	14.00	21.99	95.79	488.0	3.03	
3.5	25.75	<1.0	10.71	16.82	78.97	402.0	2.06	92.20
4.5	18.25	<1.0	7.59	11.92	67.05	341.0	1.45	
5.5	13.01	<1.0	5.45	8.56	58.49	298.0	0.99	

4th Trial, Considering skin friction

Z(m)	s(mm)	R/Ru	τ s(kPa)	P_f(kN)	Pv(KN)	Pv(kPa)	ε (%)	S(mm)
1.5					117.78	600.0	4.63	
2.5	38.30	1.0	14.00	21.99	95.79	488.0	3.03	
3.5	25.45	<1.0	10.59	16.63	79.16	403.0	2.06	92.93
4.5	17.55	<1.0	7.30	11.47	67.69	345.0	1.48	
5.5	12.20	<1.0	5.07	7.97	59.72	304.0	1.09	

Table: 5.15 Calculation of Vertical Load (Pv) and Settlement (S) (Jacketed Stone Column).

Vertical load (Pv) = 1000 kPa i.e. 193.60 kN
Skin friction (τ_s) = 0
Settlement (S) = 254.90 mm

<u>1st Trial</u> , Considering skin friction

Z(m)	s(mm)	R/Ru	τ s(kPa)	P_f(kN)	Pv(kN)	Pv(kPa)	ε (%)	S(mm)
1.5					196.30	1000.0	8.43	
2.5	78.10	1.0	14.00	21.99	174.31	888.0	6.18	
3.5	67.40	1.0	14.00	21.99	152.32	776.0	4.54	190.20
4.5	58.80	1.0	14.00	21.99	130.33	664.0	3.17	
5.5	51.45	1.0	14.00	21.99	108.34	552.0	2.16	

<u>2nd Trial</u>, Considering skin friction

Z(m)	s(mm)	R/Ru	τ s(kPa)	P_f(kN)	Pv(KN)	Pv(kPa)	ε (%)	S(mm)
1.5					196.30	1000.0	8.43	
2.5	73.05	1.0	14.00	21.99	174.31	888.0	6.18	
3.5	53.60	1.0	14.00	21.99	152.32	776.0	4.54	190.56
4.5	38.55	1.0	14.00	21.99	130.33	664.0	3.17	
5.5	26.65	<1.0	11.09	17.41	112.92	575.0	2.26	

Table: 5.16 Calculation of Vertical Load (Pv) and Settlement (S) (Jacketed Stone Column).

Vertical load (Pv) = 1225 kPa i.e. 240.50 kN
Skin friction (τ_s) = 0
Settlement (S) = 337.77 mm

1st Trial , Considering skin friction

Z(m)	s(mm)	R/Ru	τs(kPa)	P_f(kN)	Pv(kN)	Pv(kPa)	ε (%)	S(mm)
1.5					240.50	1225.0	11.00	
2.5	102.80	1.0	14.00	21.99	218.51	1113.0	8.34	
3.5	89.05	1.0	14.00	21.99	196.52	1001.0	6.29	263.0
4.5	77.65	1.0	14.00	21.99	174.53	889.0	4.67	
5.5	68.75	1.0	14.00	21.99	152.54	777.0	3.41	

2nd Trial, Considering skin friction

Z(m)	s(mm)	R/Ru	τs(kPa)	P_f(kN)	Pv(KN)	Pv(kPa)	ε (%)	S(mm)
1.5					240.50	1225.0	11.00	
2.5	96.70	1.0	14.00	21.99	218.51	1113.0	8.34	
3.5	73.15	1.0	14.00	21.99	196.52	1001.0	6.29	263.0
4.5	54.80	1.0	14.00	21.99	174.53	889.0	4.67	
5.5	40.40	1.0	14.00	21.99	152.54	777.0	3.41	

Table: 5.17 Calculated Load-Settlement Values for Jacketed Stone Column Considering Skin Friction

Total vertical load, Pv (kPa)	0.0	200.0	600.0	1000.0	1225.0
Settlement, S (mm)	0.0	27.00	92.00	190.00	263.00

Table: 5.18 Contribution of Using Reinforcing Element (Geogrid) on Lateral Confinement for Jacketed Stone Column

Deformation ratio, δ/r_0 (%)	p_r/p_h				
	Z = 1.5 m	Z = 2.5 m	Z = 3.5 m	Z = 4.5 m	Z= 5.5 m
1.0	0.945	0.915	0.884	0.856	0.830
2.0	0.950	0.919	0.890	0.862	0.837
3.0	0.951	0.921	0.893	0.866	0.841
4.0	0.952	0.922	0.894	0.868	0.844
5.0	0.952	0.923	0.896	0.870	0.845
6.0	0.953	0.924	0.896	0.870	0.846

Depth	Z = 1.5 m	Z = 2.5 m	Z = 3.5 m	Z = 4.5 m	Z = 5.5 m
Average p_r/p_h	0.95	0.92	0.89	0.86	0.84

Table: 5.19 Comparison of Existing Methods and Proposed Method

Design Methods	Mobilized $N\phi$ (= 1/K) of aggregates	Mobilized lateral resistance of clay p_S (kPa)	Vertical load Pv (kN)
Existing methods: Based on			
i) Cavity expansion theory	6.44	84.00	168.00
ii) Passive pressure theory	6.44	28.00	97.00
iii) Pile formula	--	--	112.70
iv) Genral shear failure	--	--	68.70
Proposed method:	3.83	26.00	36.30

APPENDIX

APPENDIX-I

Evaluation of vertical loads of plain stone column by the existing methods:

1. Column dimensions:

    Length of column, $L = 4.0$ m

    Diameter of column, $d = 500$ mm

    Depth of top of column from ground surface, $D_f = 1.5$ m

    Column depth at middle from ground surface, $Z = 3.5$ m

2. Aggregates properties:

    Aggregates type : Stone aggregates

    Aggregates size : 9 mm to 12 mm

    Angle of internal friction, $\phi' = 47^0$

3. Clay properties:

    Type : Soft clay

    Location: South-east region of Bangladesh

    Unit weight, $\gamma = 16.50$ kN/m

    Possion's ratio, $\nu = 0.50$

    Undrained shear strenght, Su = 14 kPa

    Modulus of elasticity, Es = 933 kPa

A. Existing method based on cavity expansion theory:

$$q = \gamma Z = 16.5 \times 3.5$$
$$= 57.75 \text{ kPa}$$
$$N_\phi = \tan^2(45^0 + \phi'/2)$$
$$= 6.49$$

$$Ir = E / (2 (1 + \nu)(Cu + q \tan\phi))$$

$$= 22.20$$

Therefore, $Fc' = 6.0$ (From Fig.2.4)

$Fq' = 1.0$ (From Fig.2.4)

$$= Fc' \; Cu + Fq' \; q$$

$$= 141.75 \text{ kPa}$$

Therefore, $q_u = \sigma_R N_\phi$

$$= 913 \text{ kPa}$$

$q_{net} = 855$ kPa

Therfore, vertical load on column Pv = 168 kN

B. Existing method based on passive pressure theory:

$$q_u = Kps \; \sigma_R$$

$$= \gamma Z \; Kpc + 2 \; c \quad Kpc \qquad (Kpc = 1.0)$$

$$= 85.75 \text{ kPa}$$

Therefore, $q_u = 552$ kPa

$q_{net} = 495$ kPa

Therefore, vertical load on stone column, Pv = 97 kN

C. Existing method based on pile formula

$$q_u = Cu (4 (L/d) + 9)$$

$$= 574 \text{ kPa}$$

Therefore, vertical load carrying capacity of stone column,

Pv = 112.70 kN

D. Based on general shear failure

Mitchell and Katti (1981) suggested from their experience that the load carrying capacity of stone column is 25 Cu.

Therefore, $q_u = 25\ Cu$

$= 350$ kPa

Therefore, vertical load carrying capacity of stone column ,

$Pv = 68.70$ kN