EFFECT OF FINITE LAYER THICKNESS ON THE ANGLE OF INTERNAL FRICTION OF SANDY SOILS

A Thesis
By
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ABSTRACT

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EFFECT OF FINITE LAYER THICKNESS ON THE ANGLE OF INTERNAL FRICTION OF SANDY SOILS

Md. Jahangir Kabir.

The present study investigates the effects of sample length and diameter on triaxial friction angle of sand. This was done in an aim to establish the relationship between physical dimensions of sand sample tested and bearing capacity factors in particular reference to a footing resting on a sand layer of finite thickness. The end condition of the both triaxial sample and bearing sand layer were maintained identical. Drained triaxial tests were performed on three selected sand samples of Bangladesh. The initial density of each of the soil sample were maintained approximately constant during triaxial test by allowing free fall of the particles to the sample mould. A special arrangement was designed and constructed for the purpose.

Triaxial tests were done using three sample diameters varying from 38mm to 71mm and five sample lengths in the range of 38mm to 178mm. As such 14 sample moulds have been designed and fabricated.

The maximum and minimum densities of the sand samples were determined using the method of Kolbuszewski (1948). Calibration for density against height of fall were carried out in order to obtain uniform density of triaxial sample. Sample end platens of triaxial apparatus were covered with sand paper (grade 3) to achieve rough end conditions. A vacuum pressure of approximately 1 N/cm² was used during preparation and placement of the sample. Two specimen were tested for each of the triaxial test using cell pressures of 10 N/cm² and 14 N/cm² respectively.
Results of triaxial test indicates that the apparent angle of internal friction decreases with the increase of sample length up to a certain value of L/D, where L and D are length and diameter of the sample respectively. The angle of friction then becomes constant. The L/D ratio at which the friction angle becomes constant are found to be dependent on sample diameter. It is also noticed that there is an optimum diameter at which the angle of internal friction becomes minimum for a particular soil. Beyond this limit (on either side) the apparent angle of friction increases.

Using these values of angle of internal friction the bearing capacity factors $N_y, N_q$ given by Terzaghi (1943) and Meyerhof (1963) were estimated. It was observed that these factors also vary with L/D in a similar manner of apparent angle of internal friction.

Factors were introduced to consider the effect of L/D ratio on these bearing capacity factors. The effects are termed as sample dimension factors $F_y, F_q$ for $N_y, N_q$ respectively. They are expressed by the ratio of bearing capacity factor at particular L/D to that at which these factors $N_y$ and $N_q$ becomes constant. This factors were compared with the thickness factors used by Abedin (1986) to consider the effect of layer thickness on bearing capacity factors for a strip footing resting on a sand layer of finite thickness. It is observed that the sample dimension factors and thickness factors of Abedin (1986) are of similar nature. As such the present investigation indicates that a modified triaxial test may be introduced to estimate the effect of layer thickness on bearing capacity.
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NOTATIONS

B  Width of footing
D  Diameter of triaxial sample
D_f Depth of foundation
H  Layer thickness of sand
L  Length of triaxial sample
γ  Unit weight of sand
φ  Angle of internal friction
τ  Unit weight of sand
c  Cohesion
q_{ult} Ultimate bearing capacity
σ_1, σ_3 Principal stresses
σ_x, σ_y, σ_θ Cylindrical co-ordinates
N_y, N_q, N_c Bearing capacity factors
N_y(θ), N_q(θ) Investigates bearing capacity factor using Terzaghi (1943) equation
N_y(θ'), N_q(θ') Investigates bearing capacity factor using Meyerhof (1963) equation
S_y, S_c, S_q Shape factor
σ_y, i_c, i_q Load inclination factor
b_y, b_i, b_q Base inclination factor
CHAPTER 1
INTRODUCTION

All civil engineering structures, whatever may be the size and type, ultimately rest on soil through a foundation. This interaction of structure with soil calls for the designer to look into the behaviour of foundation system consisting of foundation structure itself and the underlying soil. The design of foundation requires that the soil beneath the foundation should not be overstressed to cause bearing failure or excessive settlement. The other important consideration is the cost of foundation. All these factors emphasize the fact that the behaviour of a foundation should be designed by using proper strength parameters of the underlying soil.

Extensive research and study by numerous investigators established that the bearing capacity of a foundation can be predicted reasonably if the shearing strength parameters of the soil are known. This calls for the use of a proper laboratory testing method to find out the strength parameters. Because of the complex nature of the shearing resistance of soil, many methods of testing have been used with varying success. The shear test those are commonly used are direct, ring or double direct, and triaxial tests. Of these triaxial testing gives the most consistent and reliable results with varying soil conditions (Sower, 1979). In a
conventional triaxial test a cylindrical sample is used with a length to diameter ratio of approximately 2.0. The sample is then stressed to failure by controlling the stresses. Special triaxial tests have been developed that utilizes other shapes (rectangular, hollow cylindrical etc) of sample and with different control of stresses. The rectangular, hollow cylinder triaxial tests are primarily research tools for studying the mechanisms of soil behaviour and minimizing some of the shortcomings of the conventional triaxial test. Inspite of its limitations, triaxial test which utilizes $\sigma_2 = \sigma_3$ and axial compression continues to be the most useful methods for simulating soil and rock behaviour for a wide range of engineering problems due to its simplicity in preparing samples and controlling stresses. However, the complexity of natural soil and foundation conditions must not the sacrificed in predicting their behaviour for the sake of test simplicity. The bearing capacity problem on the other hand is really a very complex phenomenon. In this consideration, the designers should be aware in interpreting the laboratory test results particularly of triaxial tests while they are used for a special foundation problem. The similarity laws should be verified using model and field tests and accordingly the results should be interpreted.

The bearing capacity problem is a complex one. People have been trying for a long time to find out an accurate solution for the bearing capacity of a foundation. However, no accurate mathematical
analysis has been derived yet. A number of approximate methods have been developed are based on simplified representations of the complex failure surface and of the soil properties. Considering variability of soil in nature, certain simple assumptions of homogeneous and isotropic medium of semi-infinite layer have been imposed to solve the bearing capacity problem.

Prandtl (1920) first developed a systematic concept to analyze the failure mechanism of underlying soil under external load by assuming that the semi-infinite soil behaves like an ideal plastic material during the imposition of load. Later Terzaghi (1943) extended his concept, and subsequently followed by many researchers like Meyerhof (1951), B. Hansen (1957, 1970), Vesic (1973), Balla (1961). They developed bearing capacity equations and introduced bearing capacity factors which are stated to be dependent on an important parameter of soil, angle of internal friction. Though they assumed a plain strain condition of failure, almost all of them suggested the use of triaxial friction angle to estimate the bearing capacity factors. In further development on bearing capacity other physical factors like depth of foundation and shape factor of foundation, ground inclination, base inclination, load inclination and eccentricity have also been considered. The present day study reveals that the layer thickness of a soil beneath a foundation should be considered to be a very important factor in estimating its bearing capacity.

In this respect the study of Livneh (1965), Milovic et. al. (1970),
Das (1981), Al-Omari (1984) and Abedin (1986) are worth mentioning. These investigators pointed out that the bearing capacity of a foundation soil is greatly influenced not only by the thickness of layer itself but also on the roughness of the interfaces soil layer. They suggested to consider the conditions of both the interfaces of soil with the footing and underlying hard stratum with the soil layer. The mode of failure depends largely on the interface condition. The investigators introduced layer thickness factors with all the three bearing capacity factor like \( N_c \), \( N_y \) and \( N_q \). Their investigation method of evaluating soil strength parameters still restricted to the conventional triaxial test.

The investigators on bearing capacity admit that the bearing capacity will be significantly influenced by the size of the foundation. And as such, it is always desirable to make comparison of bearing capacity equations with full scale load tests. But unfortunately not enough full scale tests have been carried out to draw statistically valid conclusions. It is true that most of the time bearing capacity equations are conservative and almost in all instances conservative soil parameter estimates are used so that the resulting computation of ultimate bearing capacity is very likely to be quite underestimated.

From the study of previous literature that there are three stages in the development of failure mechanism of soil beneath a foundation. First, the soil beneath the foundation is forced downward in a dense cone or wedge. The soil below the wedge is then
forced downward and outward. Second, the soil around the foundation perimeter pulls away from the foundation, and surfaces of shear failure propagate outward from the tip of the cone or wedge. And finally a continuous full or partial failure surface extends to cause failure. The failure mechanism dictates that the effect of foundation size will be much more significant for the case of finite soil layer thickness. The depth of dense cone is expected to extend down to the rigid stratum and the failure mechanism will be dictated by the boundary interface condition. This aspect was considered only by few investigators (eg. Milovic et. al, 1970; Al-Omari, 1984; Abedin, 1986) and they tried to consider the case by performing either a model test or by formulating it using a theory of elasticity and plasticity.

It is expected that a good estimation of bearing capacity for this foundation condition (finite layer) could be achieved by designing a proper testing method to obtain the relevant soil parameters. This aspect still awaits to be investigated. As such, the present research is aimed to investigate the following using a cohesionless soil:

1. To design a modified triaxial test method which will simulate the conditions of finite soil layer and rough interface conditions of the foundation.

2. To estimate the bearing capacity factors using the shear strength parameter ($\phi$) obtained from the modified triaxial
3. To investigate the effect of layer thickness on bearing capacity in terms of the parameters obtained from the modified test method.

On order to achieve the above mentioned goals, sand samples having different diameters and lengths have been tested in a triaxial apparatus. Three sand sample of different origin were collected. Fourteen sample moulds having diameters between 3.8 cm to 7.1 cm and length between 3.8 cm to 17.8 cm were designed and fabricated. The end platens of triaxial apparatus were roughened by using sand paper to achieve the rough interface condition. An apparatus to obtain the uniform density of soil sample was designed and constructed. In total 42 triaxial test have been done using the modified method.

Organization of the thesis
The study is presented in five chapters. The first chapter is the introduction and the second essentially reviews the relevant literature. Chapter 3 describes the experimental system, test programme and procedure, Chapter 4 presents and discusses the results. Finally chapter 5 presents the conclusions of the experimental results with a recommendation for future study. The stress strain diagram and Morh's circles for triaxial tests and are presented in Appendix A & B respectively.
3) It is well established that there is a defined relationship between angle of internal friction and general bearing capacity factor as used for the case of semi-infinite soil layer. However, relation given by various investigator do not fully coincide.

4) The bearing capacity of a sand layer is greatly influenced by the thickness of the layer. In its estimation the effect of layer thickness is generally included along with the general bearing capacity factors introducing some arbitrary thickness factors. No attempt has been made to simulate the test conditions as per the condition prevailing at the field.

5) No research or study yet been reported by any investigator on these aspect, as mentioned in the previous article using Bangladesh sand.

It is expected that simple simulated routine test might define the angle of friction of soil which will enable the designers to compute the bearing capacity of soil using general bearing capacity factors.

As it is felt that a research programme should be chosen to investigate the effect of sample dimensions on angle of internal friction of selected Bangladesh sand. The concern of the study shall directed to its ultimate effect on bearing capacity factors.
CHAPTER 2

LITERATURE REVIEW

2.1 General

The present study is mainly concerned with the effects of sample dimensions on angle of internal friction of sand. These effects will intern influence the bearing capacity factors particularly for a foundation resting on a sand layer of finite thickness. This chapter contains a brief review of existing literature on angle of internal friction of cohesionless soil and bearing capacity with particular emphasis on bearing capacity of a sand layer resting on a rigid stratum.

2.2 Angle of Internal Friction of Cohesionless Soil

Cohesionless soils are composed of bulky grains; ranging in shape from angular to well rounded. Sowers (1979) presented a simplified representation of such a material subjected to normal and shear forces, Q and F, is shown in Fig 2.1. The particles are in contact at only a few points at which the stresses are extremely high, far greater than average stresses on the soil $\sigma = Q/A$ and $\tau = F/A$.

According to Sowers (1979) if the shear stress is increased, the
soil particle system responds in several ways simultaneously. First, the particles deform more or less elastically. Second, there is local crushing at the most high stressed points of contact. Third, both the elastic distortion and crushing causes slight transition and rotation of the grains. These are illustrated in Fig 2.1. The vector sum of all the small movements of each particle is the deformation of the mass. If the shear stress is increased further, two additional responses are evident. First, the particle tend to roll across one another, Fig 2.1. The second mechanism is the sliding of one grain across the other. The resistance to sliding is essentially friction, which is proportional to the confining (normal) stress. A third mechanism involves the interference and interlocking of the corners of the more angular and irregular grains.

If the shear stress sufficiently large, the statistical effect of the distortion, crushing, shifting, rolling and sliding of the grains will be continuous movement and distortion of the soil body, or shear failure.

The results of many tests of cohesionless soil demonstrate that the maximum shear stress on the failure surface at failure, \( \tau_f \), is nearly proportional to be effective normal stress on the failure plane at the moment of failure, \( \sigma_f \); the relation can expressed as

\[
\tau_f = \sigma_f \tan \phi \quad \ldots \quad \ldots \quad \ldots \quad (2.1)
\]
where, $\phi$ is called the angle of internal friction

According to Lambe (1958) presented a general concept about the friction angle of soil. Friction between soil grains is similar to friction between any surface, as for example between two blocks. This can be illustrated by Fig 2.2. When the top block slides along the bottom block, Fig 2.2, a shear force is applied to the surface of the bottom block, a friction will develop due to the normal force between the interface of the blocks. The resultant of this frictional force and normal force makes an angle with the normal. If the soil fails due to excessive shear, this angle at failure is termed as angle of internal friction and designated as $\phi$.

The angle of internal friction integrates all the factors of resistance to grain displacements; distortion, crushing, shifting, rolling and sliding. These factors depend on the soil mineral, the particle angularity, roughness, sphericity, gradation and relative density. If a soil mass is not restricted to fail at a desired surface, the failure plane will make an angle $\Theta$ with the surface of applied normal stress (or deviator stress). The angle $\Theta$ can be estimated using Mohr-Coulomb principle as

$$\Theta = 45 + \phi/2 \quad (2.2)$$

This phenomenon dictates that the soil mass subjected to failure due to shear is required to have a minimum relative dimension for its free failure. Otherwise, there would be an affect on the angle
of internal friction. There are many methods in practice to determine the angle of internal friction of sandy soil. The most reliable test for routine work is the triaxial test (Sowers, 1979). In the following article briefly outlines triaxial testing of sandy soils and its relative implications.

2.3 Triaxial Compression Test

In triaxial test stresses are controlled in three axes. Knowledge of triaxial testing has been greatly advanced by two extensive research programs of Cassagrande and Taylor at M.I.T in 1944. In this test a cylindrical specimen of soil enclosed within a rubber membrane between two stiff end platens. Hydrostatic pressure is applied by means of an enveloping fluid, that is isolated from the specimen by a membrane and an extra thrust or pull can be exerted on the end platens. During this test the relative displacement of the end platens is observed, and some times also the change of volume of the specimen and its diameter at a particular height. Testing proceeds by varying the axial load or the lateral pressure or quite commonly, by imposing relative axial displacement of the platens and observing axial load.

2.3.1 Stress Condition in Triaxial Compression Specimen

The weight of evidence from the triaxial test supports the friction criterion of C.A. Coulomb (1973). In this criteria the shear stress \( \tau \) causing slip on any plane is taken as the sum of a constant
value, termed the cohesion, and an additional amount that is proportional to the normal pressure acting across the plane. Thus:

\[ \tau = c - \sigma \tan \phi \quad (2.3) \]

relation follows a straight line in \( \sigma - \tau \) plane, in which representation of stress of a point due to O. Mohr (1971) can also be drawn. From geometry of Fig. 2.3c,

\[ \frac{\sigma_3 - \sigma_1}{2} = [CCot \phi - (\frac{\sigma_1 + \sigma_3}{2})] \sin \phi \quad (2.4) \]

where \( \sigma_1 \) and \( \sigma_3 \) are principal stress.

After rearrangement the equation becomes

\[ \sigma_1 = \sigma_3 \tan^2 (\pi/4 + \phi/2) - c \tan (\pi/4 + \phi/2) \quad (2.5) \]

It is a property of Mohr circle that the angle subtended at point B, Fig. 2.3c, represents twice the angle of inclination of the plane on which the corresponding stresses act. The critical condition is represented by point C, Fig. 2.3c, occurs on a plane the normal of which subtends an angle \( (\pi/4 + \phi/2) \) with the normal to the plane at which \( \sigma_1 \) acts. When triaxial specimen is assumed to be homogeneous the lateral and axial pressure can be related at once by equation (2.5). Harthornwaite (1960) expressed that the above solution is not unique because Coulomb’s law is insufficient to
define a failure load, a stress strain relationship is needed and he solved the problem considering the soil mass an ideal plastic material. He expressed the equation as follows:

\[ \sigma_1 = \sigma_3 (1 - \sin \phi) + c \cos \phi \]  

(2.6)

Balla (1960) found out the stress conditions in triaxial compression test he also included the slenderness effect of specimen and roughness of the end patten. He gave the following solution of normal and shear stresses in triaxial specimen, Fig. 2.4.

\[ \sigma_x = \sigma_1 \left[ 1 - \psi \phi_{\text{xy}} \right] + \sigma_3 \psi \phi_{\text{xy}} \]  

(2.7)

\[ \sigma_y = \sigma_1 \psi \phi_{\text{xy}} + \sigma_3 \left[ 1 - \psi \phi_{\text{xy}} \right] \]  

(2.8)

\[ \sigma_\theta = \sigma_1 \psi \phi_{\text{xy}} + \sigma_3 \left[ 1 - \psi \phi_{\text{xy}} \right] \]  

(2.9)

\[ \tau = \sigma_1 \psi \phi_{\text{xy}} - \sigma_3 \psi \phi_{\text{xy}} \]  

(2.10)

where \( \sigma_x, \sigma_y, \sigma_\theta, \tau \) are the stress in specimen and \( x, y, z \) are cylindrical co-ordinates.
\[ \phi_{\alpha} = Q \left[ -3\mu - 4(1 - \mu)(L/D)^2 + 12(1 - \mu)(L/D)^2(2x/L)^2 + 6\mu(2r/D)^2 - 24/\pi (1 - \mu) L/D \sum_{n=1}^{\infty} F_{\alpha} \cos n\pi (2x/L) \right] \]  

\[ \phi_{\theta} = Q \left[ 3/2 (3 - 2\mu) - 4(L/D)^2 + 12(L/D)^2 (2x/L)^2 - 3/2(3/2 (3 - 2\mu) (2r/D)^2 - 24/\pi (1 - \mu) L/D \sum_{n=1}^{\infty} F_{\theta} \cos n\pi (2x/L) \right] \]  

\[ \phi_t = 12(1 - \mu) L/D (2x/L) (2r/D) - 24/\pi (1 - \mu) L/D \sum_{n=1}^{\infty} F_t \sin n\pi (2x/L) \]  

\[ F_{\alpha}, F_{\theta}, F_t \] are expression condition on \( n \) and \( 2r/D \),

\[ \psi = f/f_{\max} \] 

\( f \) = co-efficient of surface friction

\( f_{\max} \) = maximum of the co-efficient of surface friction

Kirkpatrick and Belshaw (1968) used an extra technique to investigate the strain field in cylindrical specimen on dry sand in triaxial compression tests with or without lubrication of the end platens. These experiments have shown that rough end platens support the development of rigid cones at the ends. Lubrication
prevents formation of these cones, deformation is uniform for moderate strain Bishop and Green (1965) also studied the influence of the slenderness of the specimen and the end friction. They also arrived at the similar conclusion.

Drescher and Vardoulakis (1982) made an approximate analysis of the granular material and its geometric softening in triaxial compression test. He gave an interest particularly the magnitude of the mobilized angle of internal friction for rigid, perfectly plastic material behaviour with a linear Mohs - Columb yield condition. He applied the method of slice to determine the axial mean stress, Fig. 2.5a. He expressed the following relation of stress for uniform deformation mode

\[ \sigma_n = M_0 \cot \delta_0 \sigma_r \]  

\[ \sigma_n = \text{normal stress} \]
\[ M_0 = \text{constant determine from material behaviour} \]
\[ \sigma_r = \text{radial stress} \]
\[ \delta_0 = \text{angle of inclination of the plane where the stress vector is acting.} \]

and the axial mean stress

\[ \sigma_1 = \frac{2k^2}{M_0 \tan \delta_0} [\exp (M_0/k) - (M_0/k - 1)] \]  

(2.12)
where $P$ is the confining pressure

for nonuniform deformation with large pattern—specimen interface friction and if tilting of end pattern is prevented, rigid cones develop separating the deforming zones. Deforming regions are restricted to wedge-shaped rings separated by two rigid cones, Fig 2.5b. To determine the average axial stress, slices are taken only in the wedge-shaped deforming rings, with the sides inclined to the horizontal axis $r$ at angle $\beta$.

The average axial stress

$$\sigma_1 = -2pM(\cot \delta + \tan \beta)$$

$$\times \left[ \frac{(1+\rho_1)^2}{2-N} + \frac{\rho_1(1+\rho_1)}{N-1} - \frac{\rho_1^2(1/\rho_1+1)^N}{(N-1)(2-N)} \right]$$

(2.13)

where

$$N = 1 - M_0(\cot \delta + \cot \delta)$$

(2.13a)

But if the slenderness of the specimen in compression is small, Fig. 2.6, the inner rigid cones merge, forming an almost cylindrical core surrounded by wedge-shaped ring. Thus the short specimen one may assume two different regions of slice, an inner cylindrical region and an outer wedge-shaped region. The average axial stress may express as follows.
\[
\sigma_1 = -2P \left( \frac{\rho_1^2}{\exp(M_0/k_1) - (M_0/k_1 - 1)} + M(\cot \delta + \tan \beta) \right)
\]

\[
\frac{(1-\rho_1)^2}{2-N} + \frac{\rho_1(1-\rho_1)}{1-N}
\]

where \( k_1 = L/2R_1 \) (Fig 2.6)

Hetteler and Verdoulakis (1984) performed an experiment to study of stress and strain behaviour of dry sand in large triaxial apparatus for large specimen. His experimental data have given rise the theoretical analysis of bifurcation and imperfection sensitivity in triaxial test on dry sand summarized as follows:

1) Bulging is not possible if the slenderness of the specimen sample is smaller than critical value. If bulging occurs, then it take place close to limiting state and harding regime of the stress ratio strain curve of the tested sand.

2) Shear band formation in the compression test is only possible in the softening regime of the stress ratio strain curve.

2.3.2 Influence of end restraint on the compression strength of the cohesionless soil

Bishop and Green (1965) carried out tests on cohesionless soil to examine the influence of boundary restraint on the observed strength. Result indicated that the effect of the end restraint is to increase is to increase the apparent strength of the sample. But
the effect decreases with increasing height diameter ratio. One of the main criticisms of the triaxial test is the nonuniformity stress and deformation at all but extremely small strain. The main cause of this nonuniformity is the end restraint which develops friction at the end platens. Rowe and Barden (1964) expressed that two causes are responsible for nonuniformity: the formation of dead zone at the ends of the sample caused by the restraining effect of the radial friction forces at the end platens and the tendency to localized failure surface, particularly in dense brittle or fissured material.

2.3.3 Effect of slenderness of the test specimen

Taylor (1964) studied the effect of slenderness on the test specimen and his observation indicates that in end restraint condition variation in strength is negligible with height diameter ratio 1 to 2 and on the basis of Taylors observation triaxial specimens have been standardized at a height - Diameter ratio of 2 to 1 and the effect of end friction discounted.

Bishop and Green (1965) indicated that the apparent strength has a little significance with the usual height diameter ratio 2 to 1 with end in restraint condition but for lower ratio apparent strength increases.

Drescher and Verdulakis (1982) investigated the effect of slenderness on apparent angle of internal friction on sand in
triaxial compression test. His observation indicated that the apparent angle of internal friction increases with the decreases of the L/D ratio.

2.4 Bearing Capacity Factors

Though significant number of experimental investigations have been carried out for deriving the computation of bearing capacity of supporting sand of semi-infinite layer thickness, only very few have been developed for the case of finite layer thickness. All these investigations have been carried out considering the angle of internal friction tested by conventional triaxial test or direct shear test. The following sections briefly reviews the existing literature relating angle of internal friction (φ) and the bearing capacity factors considering both semi-infinite and finite layer thickness.

2.4.1 Angle of Internal Friction in Bearing Capacity Equations of Strip Foundation on semi-Infinite Layer

Rankine (1857) is the first who took attempt to solve the bearing capacity problems, later on Bell (1915) made his contribution to this work by modification of Rankin's solution.

Prandtl (1921) developed a solution of bearing capacity of c-φ soil. He gave the following equation.

$$q = cN_c$$  \( (2.15) \)
\[ N_c = \text{Bearing capacity factor} \]
\[ = \cot \phi \left( \tan^2 \left(45 + \frac{\phi}{2}\right) \right) e^{x_{\tan \phi}} (-1) \]  (2.15a)

\[ q_u = \text{Ultimate bearing capacity.} \]
\[ c = \text{Apparent cohesion.} \]
\[ \phi = \text{Angle of internal friction.} \]

Further investigation carried out by other investigators pointed out the discrepancies in Prandtl's theory. These called for further modification in Prandtl's bearing capacity equation.

Reissner (1924) has taken into account the effect of overburden pressure on bearing and thus Prandtl's equation is modified into a new form

\[ q_u = cN_c + qN_q \]  (2.16)

\[ q = \text{Uniform surcharge or overburden pressure} \]
\[ N_q = \tan^2 \left(45 + \frac{\phi}{2}\right) e^{x_{\tan \phi}} \]  (2.16a)
\[ N_c = \cot \phi \left( N_q - 1 \right) \]  (2.16b)

Buisman (1940) is first who introduces the contribution of soil density in Prandtl's modified bearing capacity equation. He superimposed the three factors and the bearing capacity equation appeared in a new form. Terzaghi (1943) developed a rational approach of ultimate bearing capacity equation considering the above three factors and the principle of superposition, and the
equation appeared to be:

\[ q_a = 0.5 \gamma B N_q + cN_c + qN_q' \]  

\[ N_q = \frac{a^2}{2 \cos^2 (45 + \phi/2)} \]  

\[ a = e^{0.75 \pi - \phi/2} \tan \phi \]  

\[ N_q = \frac{\tan \phi}{2} \left( \frac{k_{yy}}{\cos^2 \phi} - 1 \right) \]  

\[ q = \gamma D_f \]  

\[ D_f = \text{Depth of surcharge} \]  

\[ B = \text{width of foundation} \]  

First term in the equation 2.17 presents the contribution of bearing capacity resulting from soil density or self weight of soil, Second term is the contribution of constant component of shear strength and the third component is the contribution due to surcharge and all the above terms are mainly depended on \( \phi \) value.

Curves for \( N_q', N_q, N_c \) as a function of angle of internal friction are given in Fig. 2.7 and best available calculation of \( N_q \) was made by Lundgren and Mortensen (1953) and later by Odgaard and Christensen (1969) using rupture figure shown in Fig. 2.8. The result corresponds closely to the empirical formula.
Perhaps the most widely used values for factor $N_y$ are those obtained by Hansen (1968) and Meyerhof (1948, 1951, 1963). These values can be represented by the following approximation.

\[ N_y = 1.5(N_q - 1) \tan \phi \]  \hspace{1cm} (2.17c)

\[ N_y = 1.8(N_q - 1) \tan \phi \] \hspace{1cm} (Brinch Hansen, 1968) \hspace{1cm} (2.17d)

\[ N_y = (N_q - 1) \tan \phi \] \hspace{1cm} (Meyerhof, 1948, 1951) \hspace{1cm} (2.17e)

\[ N_y = (N_q - 1) \tan(1.4\phi) \] \hspace{1cm} (Meyerhof, 1963) \hspace{1cm} (2.17f)

The variation of bearing capacity factor $N_y$ with $\phi$ as found by several authors is shown in Fig. 2.9.

In fact superposition of bearing capacity equations is theoretically incorrect for plastic material. In case of cohesionless material, the ultimate bearing capacity can more correctly expressed by:

\[ q_u = v \left( 0.5 \gamma N_y + qN_q \right) \]  \hspace{1cm} (2.18)

$v$ = Correction factor and is function of $q/\gamma B$ and

For $\phi = 30^\circ$, the correction factor has calculated by Lundgren and Mortensen (1953) is shown $q$ versus $\gamma B$ in Fig. 2.10 and its maximum value found to be 1.17. However resulting error is in safeside, not exceeding 17% not exceeding 17% to 20% for $\phi = 30^\circ$. 

to 40° degree and zero while $\phi = 0$

Special case $\phi = 0$ (clay) and exact calculation has first given by Prandtl (1921), giving $N_\gamma = 0$, $N_q = 1$ and $N_c = \pi / 2 = 5.14 = 5$

$$q_u = 5c + q$$  \hspace{1cm} (2.19)

$c =$ untrained shear strength of soil.

Skempton (1951) extended the formula to cover more general case of foundation of finite length $L > B$ placed at a depth $D_f$

$$q_u = 5c(1+0.2 \frac{B}{L}) (1+0.2 \frac{D_f}{B}) + q$$  \hspace{1cm} (2.20)

Skempton (1951) used shape and depth factor to 'c' term in Terzaghi's equation. Hansen introduced inclined factor in Terzaghi's equation to account for the inclined load on the foundation. Introducing above factors in Terzaghi's original bearing capacity equation takes the following form.

$$q_u = 0.5 B \gamma N_\gamma S_\gamma d_\gamma i_\gamma + q N_q S_q d_q i_q + C N_c S_c d_c i_c$$  \hspace{1cm} (2.21)

$S_\gamma, S_q, S_c = \text{shape factor.}$

$d_\gamma, d_q, d_c = \text{depth factor.}$

$i_\gamma, i_q, i_c = \text{Inclination factor}$
An inclined load will always mean a reduce bearing capacity and the reduction is often very considerable. The foundation load has a vertical component \( V \) and horizontal component \( h \). \( i_c \), \( i_q \), \( i_h \) are then presented by the following equations -

\[
i_c = 0.5 - 0.5 \left(1 - \frac{h}{A_c}\right)^\frac{1}{2}
\]

\[
i_q = \left[1 - 0.5h/(V + A_c \cot \phi)\right]^5
\]

\[
i_h = \left[1 - 0.7h/(V + A_c \cot \phi)\right]^5
\]

The depth factor \( d \) according to definition is equal to unity and for small value of \( D_f/B \),

\[
d_c = 0.4 \frac{D_f}{B}
\]

\[
d_q = 1 + 2 \tan \phi \left(1 - \sin \phi\right)^2 \frac{D_f}{B}
\]

and for greater depth

\[
d_c = 0.4 \arctan \phi \frac{D_f}{B}
\]

\[
d_q = 1 + \tan \phi \left(1 - \sin \phi\right)^2 \arctan^{-1} \frac{D_f}{B}
\]

Shape factors \( S_c \), \( S_q \), \( S_h \) are calculated by following equations.

\[
S_c = 1 + \frac{N_q/N_c}{B/L}
\]

and \( S_c = 1 \) for strip footing.
Brinch and Hansen (1968) introduced all the factors to get a general bearing capacity formula in the form of

\[
q_u = 0.5 \gamma b N_s d_i b_i g + q N_s d_i b_i g_q + c N_s d_i b_i g_c
\]  
(2.22)

\[b_c, b_q, b_r = \text{base inclination factor}\]
\[g_c, g_q, g_r = \text{ground inclination factor}\]

The bearing capacity factors are based on the assumptions of incompressibility of soil. Therefore, it is only applicable to the case of general shear failure. The general shear failure is associated with dense soil of relatively low compressibility and its failure surface is continuous from the edge of the footing to the soil surface and full shear resistance of the soil is mobilised along the failure surface. To deal with compressibility Terzaghi (1943)** suggested the use of reduce strength properties as follows:

\[c' = \frac{2}{3}c\]  
(2.23)
\[\phi' = \tan^{-1} \left( \frac{2}{3} \tan \phi \right)\]  
(2.24)

Where \(c'\) & \(\phi'\) are the reduced value of \(C\) and \(\phi\)
2.4.2 Angle of internal friction in bearing capacity equation of strip foundation on finite soil layer.

Livneh (1965) presented a theory for the calculation of bearing capacity of weightless C-\( \phi \) soil layer loaded with a rigid strip footing resting on sand overlain on a rigid stratum having a smooth or rough interface. He developed the following formula -

For smooth interface

\[
q_s = \frac{A - B^1 \ln(1+2Y/R) \sin \phi + (Y/R) \cot \phi}{K + L \ln(2Y/R) \sin \phi}
\]  
(2.25)

For rough interface

\[
q_s = \frac{A - B^1 \ln(1+2Y/R) \sin \phi + (Y/R) \cot \phi}{K + L \ln(1+(2Y/R) \sin \phi) - f \cos(45+\phi/2)}
\]  
(2.26)

\[
A = \frac{2 \cos \phi}{\sin(45+\phi/2)}
\]  
(2.26a)

\[
B = \frac{1 \sin \phi}{1 + \sin \phi} - \frac{\cot \phi}{2 \sin \phi}
\]  
(2.26b)

\[
K = \frac{1 - \sin \phi}{\sin(45+\phi/2)}
\]  
(2.26c)
\[ L = \frac{1 - \sin \phi}{1 + \sin \phi} \cdot \frac{1}{2 \sin \phi} \]  \hspace{1cm} (2.26d)

\[ B \]

\[ R = \frac{1}{2 \cos (45 + \phi/2)} \]  \hspace{1cm} (2.26e)

\[ Y = H - R \sin (45 + \phi/2) \]  \hspace{1cm} (2.28f)

\[ B = \text{Width of footing} \]

\[ H = \text{Layer thickness} \]

\[ f = \text{Co-efficient of friction between the soil and the stratum} \]

He concluded that the bearing capacity of soil based on smooth stratum interface increases with increasing thickness of soil layer reaching a constant value at a certain limiting thickness. The presence of friction between the soil and the stratum increases the bearing capacity and decreases the value of the limiting thickness. Livneh did not present any experimental result to support his theory.

Vyalov (1967) developed a formula for the bearing capacity of weightless weak soil with underlying rigid base and subjected to strip load.

\[ q_u = \alpha \cdot \lambda \cdot \left( 1 + \epsilon/K \cdot (H/B - \epsilon K) \right) \]  \hspace{1cm} (2.29)

Where, \( \alpha \) and \( \lambda \) are constants which depend on and the co-efficient of friction between soil and the base.
\( \epsilon = \text{co-efficient of lateral pressure.} \)

Vyalov (1967) concluded that the bearing capacity decreases to minimum with increasing layer depth and then it starts increasing.

Kanayan (1970) performed model tests on the problem and concluded that the bearing capacity increased with decreasing H/B and beyond H/B approx. 2.0 the layer has no influence on bearing capacity.

Mandel and Salencon (1972) deduced the following bearing capacity formula for a thin layer of soil under a rough based rigid strip footing.

\[
q_u = CN_{cR} + BY N_{yR} + D_f N_{qR}
\]  \hspace{1cm} (2.28)

Where \( N_{cR} \), \( N_{yR} \) and \( N_{qR} \) are modified bearing capacity factors and are functions of \( \phi \) and \( B/d \). They introduce factors \( F_{cR} \), \( F_{qR} \) and \( F_{yR} \) to define \( N_{cR} \), \( N_{qR} \) and \( N_{yR} \) respectively as

\[
F_{cR} = \frac{N_{cR}}{N_c}
\]  \hspace{1cm} (2.28a)

\[
F_{yR} = \frac{N_{yR}}{N_y}
\]  \hspace{1cm} (2.28b)

\[
F_{qR} = \frac{N_{qR}}{N_q}
\]  \hspace{1cm} (2.28c)

Abedin (1986) also investigated the effect of finite layer
thickness on bearing capacity factors of footing with rough interface resting on sand his study concludes that bearing capacity factor increases with the decrease of H/B ratio.

The variation in the values of the factors $F_{cb}$, $F_{qB}$, $F_{yB}$ against B/H found by Mandel and Salencon (1972) and Abedin (1986) are shown in Fig 2.11 and $F_{qtr}$, $F_{ytr}$ introduced by Abedin (1986) are shown in Fig 2.12.

For smooth interface condition Mandel and Salencon’s (1972) bearing capacity factors were $N_{cs}$, $N_{qs}$. According to them, modified bearing capacity factor could still be calculated using Prandtl - Reissner formula.

$$N_{qs} = N_{cs} \tan \phi + 1$$  \hspace{1cm} (2.29)

A correction factor $F_{cs} = \frac{N_{cs}}{N_{c}}$ was introduced which is a function of $\phi$ and depth ratio. The modified bearing capacity factor $N_{ys}$ for the bearing capacity term related to the soil weight could not be calculated due to difficulties in assuming a suitable failure surface (Al-Omari, 1984) and hence the theory is not valid for cohesionless soil.

Al-Omari (1984) examined experimentally the behaviour of finite layers of sand overlying a rough or a smooth rigid stratum under strip rigid loading. For rough interface Al-Omari found that for both shallow and surface foundation the ultimate bearing capacity increased sharply with decreasing layer depth below a limiting
value \( H/B = 1.5 \). He concluded that the theory of Mandel and Salencon (1972) gives safe predictions of bearing capacity for foundations on the surface of thin layer and in close agreement with the experimental results for shallow foundation if triaxial angle of friction were used. His experimental results for layer thickness \((H/B)\) were in general agreement with theoretical prediction by Sokolovski (1965).

### 2.4.3. Influence of Roughness on Ultimate Bearing Capacity of Foundation

According to Terzaghi's method, the bearing capacity of a smooth strip footing on sand is somewhat greater than that of rough base. A complete analysis of bearing capacity of a perfectly rough strip footing on the surface of a weightiness material with cohesion and friction was first made by Prandtl (1920). He showed that the distribution of contact pressure is uniform over the base and the bearing capacity factor \( N_e \) depends on \( \phi \) only. Similar analysis for a perfectly smooth footing on the surface first given by Hencky (1923). He showed that the bearing capacity factor \( N_e \) of a smooth foundation is identical with that of perfectly rough foundation. The contact pressure distribution is also similar. In an extended theory given by Meyerhof (1951) expressed that the bearing capacity, \( qu \) is given by

\[
q_u = cN_{cq}
\]  
(2.30)
Where $N_{cq}$ is bearing capacity factor depending on foundation depth as well as internal friction angle.

According to Terzaghi (1943) bearing capacity of a foundation on a material with weight for a surface footing,

$$q = q' + q''$$  \hspace{1cm} (2.31)

$$q'' = (B/2)N_y$$  \hspace{1cm} (2.31a)

and by Meyerhof (1951) bearing capacity at any depth, is

$$q'' = (B/2)N_{q_y}$$  \hspace{1cm} (2.31b)

Where $N_y$ depends on foundation depth and $\phi$ only and $N_{q_y}$ depends on depth of foundation, $\phi$ and base roughness. Analysis of the bearing capacity of a perfectly rough strip footing on the surface of cohesionless material has been made by a numerical step by step computation for special case of $\phi = 30^\circ$ by Lundgren and Mortensen (1953). This method satisfies the stress equation of plastic theory but not necessarily the strain rate equations, and thus may underestimate the bearing capacity. In this case contact pressure on the base increases from zero at the edges to a maximum at the centre of the footing, and the average pressure gives a bearing capacity factor $N_q = 14.8$.

A step by step computation for perfectly smooth footing on the
surface has also made (\( \phi = 30^\circ \)) by Lundgren and Mortnsen (1953). He expressed that the contact pressure at the base increase from zero at the edge to a maximum near the outer quarter point and zero at the centre. The bearing capacity factor \( N_y \) is found to be about one half of that for perfectly rough base.

Meyerhof carried some model tests in 1951 at Building Research Station on model footing with a base of different roughness and result shows that the bearing capacity of a perfectly smooth footing on cohesionless material is one half that for a perfectly rough base and the ultimate load of strip footing on purely cohesive soil is not affected by the roughness of the base.

2.5 Conclusion

In view of the forgoing literature review on triaxial friction angle and its relation with bearing capacity of sand, the following conclusion can be made:

1) The angle of internal friction of sand depends on the testing condition. The prominent aspects are sample dimensions, shape and end restraints.

2) A significant volume of work has been done on sample shape and dimensions. A very few study has been reported in the literature on friction angle of soil considering the effect of sample length and dimensions for a particular end roughness.
3) It is well established that the relation between angle of internal friction factor as used for the case of however, relation given by various coincide.

4) The bearing capacity of a sand layer depends on the thickness of the layer. In its estimation, the effect of layer thickness is generally included along with the general bearing capacity factors introducing some arbitrary thickness factors. No attempt has been made to simulate the test conditions as per the condition prevailing at the field.

5) No research or study yet been reported by any investigator on these aspect, as mentioned in the previous article using Bangladeshi sand.

It is expected that simple simulated routine test might define the angle of friction of soil which will enable the designers to compute the bearing capacity of soil using general bearing capacity factors.

As it is felt that a research programme should be chosen to investigate the effect of sample dimensions on angle of internal friction of selected Bangladeshi sand. The concern of the study shall directed to its ultimate effect on bearing capacity factors.
3.1 Introduction

In the present study, primarily investigates the effect of sample dimensions on the angle of internal friction (ϕ) of soil. This was accomplished through an experimental programme consisting of various steps as outlined in Fig 3.1. The important components of this test programme were the sand sample used, Porosity calibration system, Sample moulds, testing machine etc. They are briefly described in the following articles.

3.2 The Sand

Three type of riverian sand samples collected from different locations of Bangladesh were used in the investigation. The samples were designated as Sample-1, Sample-2, Sample-3 which corresponds to the locations of Sylhet (Sunamganj), Jamuna river bed at Bhuapur and Dhaka (Savar) respectively.

The grain size distribution curves of the sands used are shown in Fig. 3.2. Fineness modulus, grain sizes $d_{10}$, $d_{30}$, $d_{60}$, uniformity coefficient ($c_u$), co-efficient of curvature ($c_c$) and specific gravity of these sand samples are presented in Table 3.1. Sylhet sand has
largest grain size compared to others. Local sand of Dhaka has the finest grain size. The fineness modulus of the sands are 2.5, 1.6 and 1.15 respectively.

The maximum and minimum porosity of the sands were determined using a cylindrical mould of 150 mm in height and 100 mm in diameter. For maximum porosity dry sample was poured in the mould through a funnel. Outlet of funnel was kept as near as possible to the sand surface. Care was taken so that the d suffers no vibration. When the d filled with sand the top was carefully trimmed off and the weight was taken to calculate the void ratio and porosity.

The minimum void ratio was determined by using the same mould with collar. Then the mould was filled with sand and a surcharge was place on it. The mould was given a vibration keeping it on a vibratory platform for five minutes. The collar was then removed and the top surface was trimmed off. Weights were taken and void ratio and porosity were calculated. Limiting densities and void ratios of sand samples are in presented in Table 3.2.
Table 3.2 Density and Void Ratio of sand.

<table>
<thead>
<tr>
<th>Sample designation</th>
<th>Location</th>
<th>Dry Density (kN/m³)</th>
<th>Void Ratio (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Maximum</td>
<td>Minimum</td>
</tr>
<tr>
<td>Sample-1</td>
<td>Sylhet</td>
<td>16.4</td>
<td>14.2</td>
</tr>
<tr>
<td>Sample-2</td>
<td>Jamuna</td>
<td>16.1</td>
<td>13.9</td>
</tr>
<tr>
<td>Sample-3</td>
<td>Dhaka</td>
<td>15.9</td>
<td>13.2</td>
</tr>
</tbody>
</table>

3.3 Calibration for Porosity of Sand

In an aim to achieve uniform density of sand sample during triaxial testing, the sand was calibrated for its density against height of fall. Special type of density pots were designed and fabricated to measure the density of soil.

Six density pots of different dimensions were used for the purpose of calibration. This was done to consider the effect of variation of density, if it happens in the samples during testing. The dimensions of the density pots are mentioned in table 3.3 on the schematic diagram of density pot in Fig 3.3.

A schematic diagram of calibration procedure shown in Fig 3.4. Sand was poured through a funnel attached to a clamp into a density pot.
from different heights. It was allowed to fall freely. Care was taken so that the pots suffer no vibration. When pot was filled the top surface was carefully trimmed off and weight was taken. Void ratio, porosity and density were calculated using the from the weight readings and volume for each height of fall. Porosities of the sands against height of fall are presented in Table 3.4 and also shown in Fig 3.5a to Fig 3.5c. It is observed that a height of approximately 675 mm gives the minimum porosity i.e maximum density for all the sands used. The variation of porosity against different diameter of mould for a height of 675 mm are presented in Table 3.5 and in Fig. 3.6. It is observed that porosities of sand do not vary significantly with mould diameter.

3.4 Triaxial Sample Mould

A schematic diagram of the mould is shown in Fig. 3.7. Moulds of three different diameters were used in this investigation. Four different heights were chosen in each group so as to maintain similar L/D ratios. Dimensions of the moulds are shown in a chart in Table 3.6.

Galvanized and cast iron pipes were used to construct the Triaxial sample moulds. The pipes were cut into segments to desired length. The segments of pipes were then splitted into two halves longitudinally. Splitted segments were again tack welded. Inner and outer surfaces were then machined using a lathe to attain proper inner and outer diameter. Provisions for application of vacuum was
provided by keeping holes on two side of the moulds, Fig 3.7. Provisions were also kept for clamping the two halves to form a circular mould.

3.5 Triaxial Testing Apparatus

The triaxial testing machine is an apparatus which is designed to represent the stress conditions that exist in soil in the field. This apparatus applies the major and minor principal stresses on the soil upto failure and thus measures the angle of internal friction of the soil using the Mohr- Coulomb principles. A special arrangement was made in the present investigation to pour sand from required height by designing a Wooden Platen and fixing it with the triaxial frame. This was done to study the effect of dimension conditions of test sample on friction angle of sand. A hole was provided in the platen to facilitate pouring of sand in the triaxial mould. The triaxial machine and sand pouring arrangement is shown in Fig .3.8. The top and bottom platens of varying dimensions were used to accommodate to different diameter samples of various diameter. Sand paper of grade 3 was glued to the platen to achieve the desired roughness.

3.6 Triaxial Test Procedure

A brief description of triaxial testing of sand sample are presented in the following article.
The two halves of the splitted mould were greased on splitted faces and clamped together to form a tubular mould. A Rubber was membrane placed inside the tubular mould. Vacuum was applied by connecting the tubes of a vacuum pump to the outside holes of the mould. Mould was properly checked for air leakage. Then it was placed on bottom platen of triaxial testing machine. A schematic diagram of triaxial testing machine is shown in Fig 3.9. Proving ring was removed and wooden platen was placed at required height (675 mm). Funnel was placed into the hole of wooden platen and sand sample was allowed to fall freely into the mould through the funnel, Fig 3.8. On filling the mould the surface was carefully trimmed off level and the top platen of the mould was placed. Application of vacuum transferred from the mould to sand sample through the saturation tube so that the sample can stand vertical. Mould was then removed. Cell was then placed and closed firmly. Cell was filled with water and cell pressure (σ_j) was applied and at the same time vacuum was released. Wooden platen was replaced by the proving ring. Cell pressure was applied and the sample was loaded to failure by applying axial strain at a desired rate.
Table 3.1 Grain size properties of sand

<table>
<thead>
<tr>
<th>Sample Designation</th>
<th>Sample-1</th>
<th>Sample-2</th>
<th>Sample-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Location</td>
<td>Sylhet</td>
<td>Jamuna River</td>
<td>Dhaka</td>
</tr>
<tr>
<td>Specific Gravity</td>
<td>2.63</td>
<td>2.6</td>
<td>2.57</td>
</tr>
<tr>
<td>Effective Grain diameter $D_{10}$ (mm)</td>
<td>0.21</td>
<td>0.12</td>
<td>0.11</td>
</tr>
<tr>
<td>$D_{30}$ (mm)</td>
<td>0.30</td>
<td>0.16</td>
<td>0.18</td>
</tr>
<tr>
<td>$D_{60}$ (mm)</td>
<td>0.50</td>
<td>0.20</td>
<td>0.25</td>
</tr>
<tr>
<td>Fineness Modulus</td>
<td>2.5</td>
<td>1.60</td>
<td>1.15</td>
</tr>
<tr>
<td>Uniformity</td>
<td>2.38</td>
<td>1.66</td>
<td>2.27</td>
</tr>
<tr>
<td>Co-efficient ($c_u$)</td>
<td>0.85</td>
<td>1.07</td>
<td>1.17</td>
</tr>
<tr>
<td>Co-efficient of curvature ($c_c$)</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Table 3.4 Porosity and relative Density of sand at Various Height of fall

<table>
<thead>
<tr>
<th>Sample Designation</th>
<th>Location</th>
<th>Height of Fall (mm)</th>
<th>Void Ratio</th>
<th>Porosity (%)</th>
<th>Relative Density (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sample-1</td>
<td>Sylhet</td>
<td>600</td>
<td>0.72</td>
<td>0.41</td>
<td>41</td>
</tr>
<tr>
<td></td>
<td></td>
<td>625</td>
<td>0.69</td>
<td>0.40</td>
<td>53</td>
</tr>
<tr>
<td></td>
<td></td>
<td>650</td>
<td>0.65</td>
<td>0.39</td>
<td>69</td>
</tr>
<tr>
<td></td>
<td></td>
<td>675</td>
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Table 3.5 Porosity of Sand for Different Density Pot Diameter (Height of Fall 675 mm)

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Table 3.6 Dimensions of Triaxial Sample Moulds

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<th>Material used</th>
<th>Mould Height in mm and L/D ratio (in parenthesis)</th>
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<tr>
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<td>Galvanized Iron (1.0)</td>
<td>38 (1.0) 57 (1.5) 76 (2.0) 95 (2.5) 114 (3.0)</td>
</tr>
<tr>
<td>51 mm</td>
<td>Galvanized Iron (1.0)</td>
<td>51 (1.0) 77 (1.5) 102 (2.0) 128 (2.5) 153 (3.0)</td>
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<tr>
<td>71 mm</td>
<td>Cast Iron (1.0)</td>
<td>71 (1.0) 107 (1.5) 142 (2.0) 178 (2.5) -</td>
</tr>
</tbody>
</table>
CHAPTER 4
EXPERIMENTAL RESULT AND DISCUSSION

4.1 General

The present study was aimed to investigate the effect of sample dimensions on triaxial friction angle of the selected and dry alluvial sands of Bangladesh. Triaxial test on cylindrical sand samples have been performed. Sand samples were collected from three different locations of Bangladesh namely Sylhet (Sunamganj), Jamuna (Bhuapur) and Dhaka (Savar). The relative density of the sand samples tested were maintained at approximately 76 percent for Sylhet sand, 78 percent for Jamuna sand and 72 percent for Dhaka sand. The density of test specimen has been achieved by allowing free fall of the sand particles, as suggested by Kolbuszewski (1948) using a specially designed arrangement.

In this investigation three sample diameters of 38 mm, 51 mm and 71 mm, were used and eleven sample lengths varying from 38 mm to 178 mm. As such, fourteen triaxial sample molds has been designed and fabricated. A number of 42 triaxial tests have been carried out. The reproducibility of the results of each of the test has been ascertained by repeating a test on at least three specimen. The tests were performed using two cell pressures of 10 N/cm² and 14 N/cm². A vacuum of 1 N/cm² has been used while preparing the test specimen. The end of the edge platens for the triaxial sample were
has been roughened by using sand papers of grade 3. The friction angle of the sand paper was determined from direct shear test and found to be $40^\circ$.

It is found that the triaxial friction angle depends on the L/D ratio and then the values of friction angle used to predict the bearing capacity factor used in bearing capacity equation of layered soil. In this chapter these findings are discussed and compared with the result of similar investigations.

4.2 Effect of Sample Length on angle of internal friction:

Triaxial friction angle as determined from the experimental investigation are presented in the Table 4.1. They are also shown in Fig 4.1a to Fig 4.1c against sample length for various sample diameter. The Figs indicate that the angle of internal friction, for a particular sample diameter decreases with increasing sample length up to a certain limit. After that the angle of internal friction remains constant. The sample length at which the angle of internal becomes constant is dependent on sample diameter. It is longer for larger sample diameter. This reveals that the end condition affects the value of deviagric stress when the sample length is short. After certain length of the sample (for given diameter) the end condition do not affect the deviagric stress at failure. This observation suggests that the estimation of angle of internal friction from triaxial test should consider the geometry of failure plane. The observation also supports the general
principle of the development of failure plane that the orientation of the failure surface is governed by the length-diameter ratio (L/D) of the sample. The angle of internal friction is plotted against length-diameter (L/D) ratio in Fig 4.2a to Fig 4.2c. It is observed that the estimated angle of internal friction becomes constant only when the L/D ratio exceeds the value of 2.0. According to Kirkpatrick and Belshaw (1968) rough end plattens in a triaxial test support the development of rigid cone at the ends, Fig 2.5 and Fig 2.6. Bishop and Green (1965) had also arrived at a similar conclusion. Drescher and Verdulakis (1982) reports that when the specimen platten friction is large and tilting at end platten is prevented, rigid cone develope separating the deforming zones. According to them, if the slenderness of the specimen is small, the inner cone merge, forming an almost cylindrical core surrounded by a wedge shaped ring, Fig 2.6. The findings of the previous investigator and the present study indicate that the cones are formed only after the densification of the region. For small slenderness of the sample the merging of the densified cones results in a higher stiffness of the sample as a whole. The failure of the sample s occurs due to breaking of this densified soil and consequently the computed angle of internal friction increases.

4.3 Effect of Sample Diameter on Angle Of Internal Friction:

The results of triaxial friction angle are plotted against sample diameter for various L/D ratio in Fig 4.3a to Fig 4.3c. It is to be mentioned that three sample diameter of 38 mm, 51 mm, 71 mm are
used in this present investigation. It is a point of interest to note that estimated friction angle for 51 mm diameter gives the minimum value for a particular L/D ratio. The values for 38 mm diameter sample is little higher than 51 mm diameter sample. But this difference is not significant. On the other hand the angle of internal friction for 71 mm diameter sample is significantly higher than those of the other two. This behaviour seems to a deviation from the general concept on the effect of diameter on angle of internal friction that the angle of internal friction varies with the sample diameter. This behaviour could only be explained when the failure pattern of the sample is known. This was out of scope of the present investigation and it needs further research for a reasonable explanation of this behaviour.

4.4 Effect of Sample dimensions on Bearing Capacity Factor

Bearing capacity factors $N_y$ and $N_q$ are estimated by using observed angle of internal friction using Terzaghi (1943)’s formula (Eq 2.17a, Eq 2.17b) and Meyerhof (1963)’s formula (Eq 2.16b, Eq 2.17f). Results are presented in Table 4.4. They are also shown graphically in Fig 4.4a to Fig 4.4f against L/D ratio. Results indicate that both the bearing capacity factors $N_y$ and $N_q$ decrease with the increase of L/D ratio and becomes constant while the L/D ratio is greater than 2.0. The bearing capacity factors were not found to be independent of dimensionless L/D ratio rather they were found to vary with the diameter $D$, of the sample. However this variation is not significant. The minimum (optimum) values of these
factors were found for a sample diameter of 51mm.

Sample dimension factor $F_y$ and $F_q$ are introduced to consider the effect of L/D ratio on the bearing capacity factors $N_y$ and $N_q$ respectively. They are defined as

$$F_y = \frac{N_y}{N'_y} \quad (4.1)$$

$$F_q = \frac{N_q}{N'_q} \quad (4.2)$$

Where $N_y$ and $N_q$ are general bearing capacity factors at any L/D ratio of the triaxial sample tested. $N'_y$ and $N'_q$ are the bearing capacity factors while the L/D ratio exceeds 2.0. Values of $F_y$ and $F_q$ are shown in Table 4.5a and Table 4.5b.

The sample dimension factors $F_y$ and $F_q$ are presented in Fig 4.5a to Fig 4.5f. It indicates that the dimension factors $F_y$ and $F_q$ decrease sharply with the increase of L/D ratio upto its (L/D) value of 2.0. Then they become constant.

In estimating the bearing capacity factor $N_y$ due to a strip footing on a sand layer of finite thickness overlying a rough rigid stratum Al-Omari (1984), Abedin (1986) introduced a thickness factor. Similar factor can be obtained for bearing capacity factor $N_q$ in the study of Mendel and Salencon (1972) (Eq 2.28a to Eq 2.28c). They also worked on foundations on a sand layer of finite thickness. Abedin (1986) made the study with a sand having
triaxial friction angle of 36.5° while Mandel and Salencon (1972) went for a theoretical study supported by experimental observation. The thickness factor obtained by Abedin (1986) also incorporated in Fig 4.5a to Fig 4.5f. It shows that the nature of variation of thickness factor with H/B ratio (where H is the layer thickness, B is the width of footing) is very much similar to the variation of dimension factor $F_T$ and $F_q$ with $L/D$ as used in the present study. This observation indicates that the bearing capacity of a foundation on a finite soil layer can be estimated by performing triaxial test of the nature as proposed in the present study.
Table 4.2 Angle of Internal Friction for Various Sample Length and L/D Ratio

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<th>Sample Diameter D (mm)</th>
<th>Sample Length L (mm)</th>
<th>L/D</th>
<th>Angle of Internal Friction °</th>
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Table 4.4a Bearing Capacity Factor $N_y$ for Various L/D Ratio.

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Table 4.4b  Bearing Capacity Factor $N_q$ for Various L/D Ratio.

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Table 4.5a Sample Dimension Factor $F_y$ for Various L/D Ratio

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Table 4.5b Sample Dimension Factor $F_q$ for Various L/D Ratio

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<th>Jamuna Sand</th>
<th>Dhaka Sand</th>
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<tbody>
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<td>$F_q(W)$</td>
<td>$F_q(T)$</td>
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CHAPTER 5

CONCLUSION

The present research programme was aimed to investigate primarily the effect of sample dimensions on the angle of internal friction for rough ended triaxial sample. The results of the study triggers to introduce a factor with the observed triaxial value to consider this effect. It further investigates the relationship and the similarity between this factor considered in the present study and the factors introduced by other researchers to count for the effect of layer thickness on bearing capacity factors of soil.

Drained triaxial test have been carried out on three selected alluvial sand samples of Bangladesh. An initial relative density of approximately 75 percent were maintained for the samples. Fourteen sample moulds were designed and constructed having varying lengths and/or diameter. A total of 42 triaxial tests were done. Each test was performed using two cell pressures of 10 N/cm² and 14 N/cm². On the basis of this experimental observation the following conclusion can be drawn:

1) Both length and diameter of a triaxial test sample have individual effects on the triaxial friction angle in the range of cell pressure (10 - 14 N/cm²) used in the present investigation. The variation of friction angle could be as high as 7° for small diameter (38mm) samples. The variation is
comparatively smaller for larger diameters.

2) The angle of internal friction of a soil for particular L/D ratio has a minimum value at a particular diameter. This critical diameter was found to be 51mm for all cases. This observation could not be explained using the limited test results.

3) The angle of internal friction of a soil decreases up to a certain L/D ratio, then it becomes constant. This limiting L/D depends upon the sample diameter D and was found to be approximately 2.0.

4) The observed bearing capacity factors \( N_y \) and \( N_q \), as calculated by using Terzaghi's (1943) and Meyerhof's (1963) formula and the observed value have similar nature of variation with L/D as that of angle of internal friction \( \phi \).

5) The dimension factors introduced \( F_y \) and \( F_q \), introduced to take into account of the effect of L/D ratio on bearing capacity factors shows that at approximately a L/D ratio of 2.0 these values are independent both L/D and D and becomes equal to unity. The nature of variation of these factors are similar to those obtained by Abedin (1986) to consider the effect of layer thickness on the bearing capacity factors due to a rough based strip footing resting on a sand layer overlying a rough surfaced rigid stratum. As such these approach of triaxial test
may be adopted to estimate the design parameters for foundations resting on a soil layer of finite thickness. However, its generalisation requires further study and research.

**Recommendation of future study**

The present study initiates the scope of using triaxial test using to estimate the bearing capacity of a sand layer of finite thickness. To generalise the results of the present investigation, the following points may be considered for future study and research.

1) To find out the effect of sample diameter on angle of internal friction of the soil.

2) To study the effect of end roughness on the angle of internal friction.

3) To investigate the effect of length on the angle of internal friction using various length for particular sample diameter.

4) To perform out laboratory model tests on footing in order to find out the relation between predicted and observed bearing capacity factors. In the prediction the value of friction angle as obtained by using the present modified method should be used.
REFERENCES


MANDEL, J. and SALECON, J. (1972), "Force Portante d'un soil sur une Assise Rigide (etude theorique)". Geotechnique 22, No.1, pp. 79-93.


FIGURES
Fig. 2.1 Mechanisms for deformation and shear in a soil consisting of bulky grains (a) Shear in granular mass showing potential particle movements (b) Mechanisms of resistance, deformation and movement in grains. (After Sowers, 1979)

Fig. 2.2 Shear of soil in a box.
Fig. 2.3 Stress Condition in Triaxial Compression Specimen.

(Haythornthwaite 1960)
FIG. 2.4  Stress Condition in Triaxial Compression Specimen
(Balla, 1960)
Fig 2.5 Slices for Uniform and Non Uniform Deformation mode for Moderately Slender Specimen. (Drescher & Vardoulakis, 1982)
Fig. 2.6 Slices for non-uniform deformation mode for short specimens. (Drescher & Vardoulakis, 1982)
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Fig. 2.8 Lundgren-Mortensen rupture figure for calculation of $N_f$. Vertical load on heavy earth. (no surface load)
Fig. 2.9 Variation of bearing capacity factor $N_f$ with $\phi$

Fig. 2.10 Correction factor for bearing capacity of a shallow foundation.
FIG. 2-11 MANDEL AND SALENCON'S THEORY
FOR ROUGH INTERFACE.
Fig. 2.12 Thickness Factor for Rough Interface

(a) Surface Footing (D/B = 0.0)

(b) Shallow Footing (D/B = 0.5)
Fig. 3.1 Outline of Research Programme
FIG. 3.2 GRAIN SIZE DISTRIBUTION CURVES OF INVESTIGATED SANDS.
Table 3.3 Dimensions of Density Pot

<table>
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<th>Pot No.</th>
<th>Diameter (mm)</th>
<th>Height (mm)</th>
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<tr>
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Fig. 3.3 Density Calibration Pot
Fig 3.4 Porosity Calibration Procedure
FIG. 3.5a Height of Fall $V_s$ Porosity (Sylhet Sand)
FIG. 3.5.c Height of Fall Vs Porosity (Dhaka Sand)
Fig. 3.6 Porosity Vs Pot-Diameter
Fig 3.7 Triaxial Sample Mould
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Fig. 3.9 Triaxial Testing Procedure
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Fig. 4.2a  Length Diameter Ratio Vs Angle of Internal Friction
(Sylhet Sand)
Fig. 4.2b Length-Diameter Ratio Vs Angle of Internal Friction
(Jamuna Sand)
Fig. 4.2c  Length Diameter Ratio Vs Angle of Internal Friction
(Dhaka Sand)
Fig. 4. 3. a Effect of Sample Diameter on Angle of Internal Friction (Sylhet Sand)
Fig. 4.3b Sample Diameter Vs Angle of Internal Friction
(Jamuna Sand)
Fig. 4.3c Sample Diameter Vs Angle of Internal Friction (Dhaka Sand)
Using Terzaghi (1943) equation

Using Meyerhof (1963) equation

For dia 38 mm

For dia 51 mm

For dia 71 mm

Fig. 4.4.a Length Diameter Ratio Vs Bearing Capacity Factor $N_y$ (Sylhet Sand)
Fig. 4.4b Length–Diameter Ratio vs Bearing Capacity Factor $N_\alpha$
(Jamuna Sand)
Fig. 4.4c Length Diameter Ratio Vs Bearing Capacity Factor $N_\phi$

(Dhaka Sand)
Fig. 4.4.1 Length-Diameter Ratio Vs Bearing Capacity Factor Nq (Sylhet Sand)
Fig. 4.4.d Length-Diameter Ratio Vs Bearing Capacity Factor Nq
(Dhaka Sand)
Using Terzaghi (1943) equation
Using Meyerhof (1963) equation

For dia 38 mm.
For dia 51 mm
For dia 71 mm.

Fig. 4.4.e Length-Diameter Ratio Vs Bearing Capacity Factor Nq
(Jamuna Sand)
Using Terzaghi (1943) equation
--- Using Moyerhof (1963) equation
--- Mandel and Salencon (1972)
--- Abadin (1986)

Fig. 4.5.a Length-Diameter Ratio Vs Dimension Factor $F_Y$
(Sylhet Sand)
Fig. 4.5.b Length-Diameter Ratio Vs Dimension Factor $F_q$

(Sylhet Sand)
Fig. 4.5.c  Length-Diameter Ratio Vs Dimension Factor $F_Y$

(Jamuna Sand)
Using Terzaghi (1943) equation
Using Meyerhof (1963) equation
Mandel and Salencon (1972)
Abedin (1986)

Fig. 4.5d Length-Diameter Ratio Vs Dimension Factor Fq
(Jamuna Sand)
**Fig. 4.5.e** Length-Diameter Ratio Vs Dimension Factor $F_y$

(Dhaka Sand)
Using Terzaghi (1943) equation
Using Meyerhof (1963) equation
Mandel and Salencon (1972)
Abedin (1986)

Fig. 4.5.f Length-Diameter Ratio Vs Dimension Factor Fq
(Dhaka Sand)
APPENDIX - A
FIG. A-1

FIG. A-2
Sylhet Sand

$D = 3.8 \text{ cm}, \ L = 7.6 \text{ cm}$

$D = 3.8 \text{ cm}, \ L = 9.5 \text{ cm}$

**FIG. A-3**

**FIG. A-4**
Sylhet Sand
D = 3.8 cm, L = 11.4 cm

Sylhet Sand
D = 5.1 cm, L = 5.1 cm

FIG. A-5
FIG. A-6
Sylhet Sand

$D = 5.1 \, \text{cm}, \ L = 7.6 \, \text{cm}$

$D = 5.1 \, \text{cm}, \ L = 10.2 \, \text{cm}$

**FIG. A-7**

**FIG. A-8**
Sylhet Sand

\[ D = 5.1 \text{ cm}, \; L = 12.7 \text{ cm} \]

\[ D = 5.1 \text{ cm}, \; L = 15.3 \text{ cm} \]

**FIG. A-9**

**FIG. A-10**
Sylhet sand

\[ D = 7.1 \text{ cm}, \quad L = 7.1 \text{ cm} \]

---

Sylhet Sand

\[ D = 7.1 \text{ cm}, \quad L = 10.7 \text{ cm} \]

---

**FIG. A-11**

Deviator Stress in N/cm²

- 10 N/cm²

Strain

---

**FIG. A-12**

Deviator Stress in N/cm²

- 10 N/cm²

Strain
FIG. A-13

Sylhet Sand

\[ D = 7.1 \text{ cm}, \ L = 14.2 \text{ cm} \]

FIG. A-14

Sylhet Sand

\[ D = 7.1 \text{ cm}, \ L = 178 \text{ cm} \]
FIG. A-15

D = 3.8 cm, L = 3.8 cm

FIG. A-16

D = 3.8 cm, L = 5.7 cm
FIG. A-19

D = 3.8 cm, L = 11.4 cm

FIG. A-20

D = 5.1 cm, L = 5.1 cm

Devilator Stress in N/cm²

Strain

10 N/cm²

14 N/cm²
FIG. A-22

D = 5.1 cm, L = 10.2 cm

FIG. A-21

D = 5.1 cm, L = 7.6 cm
FIG. A-23

FIG. A-24
FIG. A-25

FIG. A-26
Jamuna Sand
D = 7.1 cm, L = 14.2 cm

FIG. A-27

Jamuna Sand
D = 7.1 cm, L = 17.8 cm

FIG. A-28
Dhaka Sand

$D = 3.8 \text{ cm}, \ L = 7.6 \text{ cm}$

$D = 3.8 \text{ cm}, \ L = 9.5 \text{ cm}$

**FIG. A-31**

**FIG. A-32**
Dhaka Sand

D = 3.8 cm, L = 11.4 cm

Dhaka Sand

D = 5.1 cm, L = 5.1 cm

FIG. A-33

FIG. A-34
Dhaka Sand

D = 5.1 cm, L = 12.7 cm

Dhaka Sand

D = 5.1 cm, L = 15.3 cm

FIG. A-37

FIG. A-38
Dhaka sand
$D = 7.1$ cm, $L = 7.1$ cm

Dhaka Sand
$D = 7.1$ cm, $L = 10.7$ cm

FIG. A-39

FIG. A-40
Dhaka Sand

D = 7.1 cm, L = 14.2 cm

Dhaka Sand

D = 7.1 cm, L = 17.8 cm

FIG. A-41

FIG. A-42
Sample - Sylhet sand
Dia - 3.8 cm.
Length - 3.8 cm.

Fig. B-1

Sample - Sylhet sand
Dia - 3.8 cm.
Length - 5.7 cm.

Fig. B-2
Sample - Sylhet sand
Dia - 3.8 cm.
Length - 7.6 cm.

Fig. B - 3

Sample - Sylhet sand
Dia - 3.8 cm.
Length - 9.5 cm.

Fig. B - 4
Sample - Sylhet sand
Dia - 3.8 cm.
Length - 11.4 cm.

Fig. B - 5

Sample - Sylhet sand
Dia - 5.1 cm.
Length - 5.1 cm.

Fig. B - 6
Sample - Sylhet sand.
Dia - 5.1 cm.
Length - 7.6 cm.

Fig. B-7

Sample - Sylhet sand
Dia - 5.1 cm.
Length - 10.2 cm.

Fig. B-8
Sample - Sylhet sand
Dia - 5.1 cm.
Length - 12.7 cm.

Fig. B-9

Sample - Sylhet sand
Dia - 5.1 cm.
Length - 15.3 cm.

Fig. B-10
Sample - Sylhet Sand
Dia - 7.1 cm.
Length - 7.1 cm.

FIG B-11

Sample - Sylhet Sand
Dia - 7.1 cm.
Length - 10.7 cm.

FIG B-12
Sample - Sylhet Sand
Dia - 7.1 cm.
Length - 14.2 cm.

Sample - Sylhet Sand
Dia - 7.1 cm.
Length - 17.8 cm.
Sample - Jamuna sand
Dia - 3.8 cm.
Length - 3.8 cm.

Fig. B-15

Sample - Jamuna sand
Dia - 3.8 cm.
Length - 5.7 cm.

Fig. B-16
Sample: Jamuna sand
Dia = 3.8 cm.
Length = 7.6 cm.

Fig. B-17

Sample: Jamuna sand
Dia = 3.8 cm.
Length = 9.5 cm.

Fig. B-18
Sample - Jamuna sand
Dia - 3.8 cm.
Length - 11.4 cm.

Fig. B-19

Sample - Jamuna sand
Dia - 5.1 cm.
Length - 5.1 cm.

Fig. B-20
Sample - Jamuna sand
Dia - 5.1 cm.
Length - 7.6 cm.

Fig. B-21

Sample - Jamuna sand
Dia - 5.1 cm.
Length - 10.2 cm.

Fig. B-22
Sample - Jamuna sand
Dia - 5.1 cm.
Length - 12.7 cm.

Shear stress (N/cm²)

Fig. B-23

Sample - Jamuna sand
Dia - 5.1 cm.
Length - 15.3 cm

Shear stress (N/cm²)

Normal stress (N/cm²)

Fig. B-24
Sample - Jamuna Sand
Dia - 7.1 cm.
Length - 7.1 cm.

Fig. B-25

Sample - Jamuna Sand
Dia - 7.1 cm.
Length - 10.7 cm.

Fig. B-26
Sample - Jamuna Sand.
Dia - 7.1 cm.
Length 14.2 cm.

Fig. B-27

Sample - Jamuna Sand
Dia - 7.1 cm.
Length 17.8 cm.

Fig. B-28
Sample - Dhaka sand
Dia - 3.8 cm.
Length - 3.8 cm.

Fig. B-29

Sample - Dhaka sand
Dia - 3.8 cm.
Length - 5.7 cm.

Fig. B-30
Sample - Dhaka sand
Dia - 3.8 cm.
Length - 7.6 cm.

Fig. B-31

Sample - Dhaka sand
Dia - 3.8 cm.
Length - 9.5 cm.

Fig. B-32
Sample - Dhaka sand  
Dia - 3.8 cm.  
Length - 11.4 cm.

Fig. B-33

Sample - Dhaka sand  
Dia - 5.1 cm.  
Length - 5.1 cm.

Fig. B-34
Sample - Dhaka sand
Dia - 5.1 cm.
Length - 7.6 cm.

Fig. B-35

Sample - Dhaka sand.
Dia - 5.1 cm.
Length - 10.2 cm.

Fig. B-36
Sample - Dhaka sand
Dia - 51 cm.
Length - 12.7 cm.

---

Sample - Dhaka sand
Dia - 51.1 cm.
Length - 15.3 cm.
Sample - Dhaka Sand
Dia - 7.1 cm
Length - 7.1 cm.

FIG B-39

Sample - Dhaka Sand
Dia - 7.1 cm
Length - 10.7 cm.

FIG B-40
Sample - Dhaka Sand
Dia - 7.1 cm.
Length - 14.2 cm.

FIG B-41

Sample - Dhaka Sand
Dia - 7.1 cm.
Length - 17.8 cm.

FIG B-42