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CORRELATION BETWEEN PENETROMETER TEST VALUE AND BEARING CAPACITY FOR GRANULAR SOIL

MD. GOLAM QUADER

Department of Civil Engineering
Khulna University of Engineering & Technology (KUET)
Khulna, Bangladesh

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MASTER OF ENGINEERING

By

MD. GOLAM QUADER

Department of Civil Engineering
Khulna University of Engineering & Technology (KUET)
Khulna, Bangladesh

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Md. Golam Quader

Approved as to Style and Contents by the Examination Committee:

Professor Dr. Md. Abul Bashar — **Chairman**
Supervisor
Department of Civil Engineering
Khulna University of Engineering & Technology

Professor Dr. Quazi Hamidul Bari — **Member**
Head
Department of Civil Engineering
Khulna University of Engineering & Technology

Professor Dr. Muhammed Alamgir — **Member**
Department of Civil Engineering
Khulna University of Engineering & Technology

Professor Md. Nuruzzaman — **Member (External)**
Department of Civil Engineering
Dhaka University of Engineering & Technology
Gazipur

13 January, 2007

ABSTRACT

In the present study, an attempt to investigate the correlation between hand penetrometer test value and bearing capacity of granular soil especially sand, was under consideration. To achieve this goal artificial sand beds of thirteen samples were prepared in an open test bed above the ground surface, and on these samples, direct shear test and field density test were carried out to find out the friction angle and the field density of each sample. Bearing capacity of sandy soil were predicted from Terzaghi's bearing capacity equation for strip footing. Three penetrometers of different diameters (18.75 mm, 25 mm and 31.25 mm) were especially fabricated and used to find out penetrometer test value. The test procedure of these penetrometers is similar to standard penetration test (SPT) by split spoon sampler, but diameter of penetrometers, weight of hammer and height of fall were different.

To correlate between hand penetrometer test value, friction angle and bearing capacity, seven equations have been established for hand penetrometer of three different diameters. It has been found that there established better correlations between friction angle and hand penetrometer test value for three penetrometers with high coefficient of correlations. In the investigation it has observed that there exists a best correlation between friction angle and bearing capacity of granular soil (sand). Finally it has been found that there are good correlations between hand penetrometer test value and bearing capacity of granular soil for any diameter of penetrometer with coefficient of correlation from 0.79 to 0.96.

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DECLARATION

This is to declare that the project entitled "Correlation Between Penetrometer Test Value and Bearing Capacity for Granular Soil" that is being submitted by the author for the award of the degree of M. Engineering to Khulna University of Engineering & Technology (KUET), Khulna is a record of project work carried out by him under the supervision and guidance of Dr. Md. Abul Bashar, Professor, Department of Civil Engineering, KUET, Khulna. The results embodied in this project have not been submitted to any other University for the award of any Degree or Diploma.

Supervisor

(Dr. Md. Abul Bashar)
Professor
Dept. of Civil Engineering
Khulna University of Engineering & Technology
Khulna

Author

(Md. Golam Quader)
Roll-971002

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B	Breadth of foundation
b_c	Base inclination factor
b_q	Base inclination factor
b_γ	Base inclination factor
c	Cohesion
c_u	Cofficient of uniformity
D	Depth of foundation
D_{min}	Minimum depth of foundation
d_c , d_q , d_γ	Depth factor
g_c, g_q, g_γ	Ground surface inclination factor
i_c, i_q, i_γ	Inclination factor
N_h	Hand penetrometer test value (No. of blows/30 cm)
N_c	Dimensionless bearing capacity factor
N_q	Dimensionless bearing capacity factor
N_γ	Dimensionless bearing capacity factor
N_R	Recorded value of penetration
N_o	Correct value for over burden pressure
P'	Effective overburden pressure
q	Intensity of loading
q_{ult}	Ultimate bearing capacity
q_a	Allowable bearing capacity
q_u	Unconfined compressive strength
s_c, s_q, s_γ	Shape factor
w	water content
x	x is used in equation in place of N_h
y	y is used in equation in place q_{ult}
γ	Unit weight of soil
σ'	Effective stress
ϕ	Angle of internal friction or friction angle

CHAPTER 1

INTRODUCTION

1.1 General

For the construction of multi-storied buildings, highway, bridge, tower, overhead water tank, industrial plants, etc., sub-soil investigation is very important to know the soil type, consistency or relative density and ground water table. To design the selection of foundation type and depth of foundation of any super-structure, it is essential to know the bearing capacity, settlement of sub-soil layers. For these cases, field tests and/or laboratory tests are performed. But it takes more than one month for field and laboratory tests. Now-a-days, N-value from standard penetration test (by split spoon sampler) is widely used for determining bearing capacity of soil but it also takes long time as well as costly.

The site engineer is often faced with the problem of ascertaining the in-situ bearing capacity of soil. To find out in-situ bearing capacity of soil, standard penetration test, plate load test, etc. are performed which are time-consuming and expensive. A soil survey can never cover the entire site. Based on the parameters obtained from the soil investigation safe bearing capacity is calculated, which is subsequently specified in the drawings. The site engineer has to verify quickly whether the specified value is available or not. But the time involved in the process of laboratory tests would make it rather impractical. To overcome these difficulties a hand penetrometer has been developed which is simple to handle and operate. K L Sanyal (1987) has developed a correlation between hand penetrometer test value N_h and standard penetration test N value. It is particularly useful to the field engineer to determine the bearing capacity of the excavated strata. It can also be used for compaction control. For lightly loaded shallow foundation, the instrument can be useful in which case owner is not at all interested to perform costly soil investigation work. To avoid these difficulties present investigation was undertaken hand penetrometer of three different diameters (18.75 mm, 25 mm and 31.25 mm) to find out the bearing capacity of granular soil.

1.2 Area of The Project Work

The layer of granular soil of various types are not available in this locality. So, it was under consideration to prepare artificial sand bed in a manually constructed test bed like a box above the ground surface. For this project work disturbed samples of Sylhet sand, Kushtia sand and local sand (Paigram koshba sand) were collected from the local business center of sand. A test bed was prepared at the test yard of the Civil Engineering Department in KUET campus.

1.3 Objective of The Project Work

The present study has been undertaken to achieve the following principal objectives:

To investigate the relationship between hand penetrometer test value and bearing capacity, Terzaghi's empirical equation was used to determine the bearing capacity by using friction angle from direct shear tests on granular soils. The field tests, laboratory tests and predicted values were used in order to achieve the following objectives:

(i) To correlate between hand penetrometer test value, N_h and friction angle, ϕ of granular soil.

(ii) To correlate between friction angle, ϕ and bearing capacity of granular soil.

(iii) To correlate between N_h and ultimate bearing capacity, q_{ult}.

CHAPTER 2

REVIEW OF LITERATURE

2.1 General

The foundation of any structure transmits the load of the structure to the soil underneath. A rational design of the foundation is based upon the bearing capacity of soil and hence determination of bearing capacity of soil is very important for a foundation engineer. If the soil near the surface is capable of adequately supporting the structural loads it is possible to use either footings or a raft. A footing is a relatively small slab giving separate support to part of the structure. A footing supporting a single column is referred to as an individual footing, one supporting a group of columns as a combined footing and one supporting a load bearing wall as a strip footing. A raft is a relatively large single slab, usually stiffened, supporting the structure as a whole. If the soil near the surface is incapable of adequately supporting the structural loads, piles or piers are used to transmit the loads to suitable soil at greater depth. Foundation level should be below the depth which is subjected to frost action (around 0.5 m in the United Kingdom) and, where appropriate, the depth to which seasonal swelling and shrinkage of the soil takes place.

A foundation must satisfy two fundamental requirements: (i) the factor of safety against shear failure of the supporting soil must be adequate, a value between 2.5 and 3.0 normally being specified; (ii) the settlement of the foundation should be tolerable and, in particular, differential settlement should not cause any unacceptable damage nor interfere with the function of the structure. So, the bearing capacity may be defined as the largest intensity of pressure which may be applied by a structure or a structural member to the soil which supports it without causing excessive settlement or danger of failure of the soil in shear. In other words, the allowable bearing capacity (q_a) is defined as the maximum pressure which may be applied to the soil such that the two fundamental requirements are satisfied.

Damage due to settlement may be classified as architectural, functional or structural. In the case of frame structures, settlement damage is usually confined to the cladding and finishes (i.e., architectural damage): such damage is due only to the settlement occurring subsequent to the application of the cladding and finishes. In some cases, structures can be designed and constructed in such a way that a certain degree of movement can be accommodated without damage. In other cases a certain degree of cracking may be inevitable if the structure is to be economic. It may be that damage to services, and not to the structure, will be the limiting criterion.

Based on observations of damage in buildings, Skempton and MacDonald (1956) proposed limits for maximum settlement at which damage could be expected and related maximum settlement to angular distortion. The angular distortion between two points under a structure is equal to the differential settlement between the points divided by the distance between them. No damage was observed where the angular distortion was less than 1/300: for individual footings this figure corresponds roughly to a maximum settlement of 50 mm on sands and 75 mm on clays. Angular distortion limits were subsequently proposed by Bjerrum (1963) as a general guide for a number of structural situations as shown in Table 2.1. It is recommended that the safe limit to avoid cracking in the panel walls of framed structures should be 1/500. In the case of load bearing brick wall the criteria recommended by Polshin and Tokar (1957) are generally used. These criteria are given in terms of the ratio of deflection to the length of the deflected part and depend on the length-to-height ratio of the building: recommended deflection ratios are within the range $0.3x10^{-3}$ to $0.7x10^{-3}$. In the case of buildings subjected to hogging the criteria of Polshin and Tokar (1957) should be halved.

The above approach to settlement limits is empirical and is intended to be only a general guide for simple structures. A more fundamental damage criterion is the limiting tensile strain at which visible cracking occurs in a given material. A comprehensive discussion of settlement damage in buildings has been presented by Burland and Wroth (1975).

Table 2.1 Angular Distortion Limits

1/500	Structural damage of general buildings expected
1/250	Tilting of high rigid buildings may be visible
1/300	Cracking in panel walls expected Difficulties with overhead cranes
1/500	Limit for buildings in which cracking is not permissible
1/600	Overstressing of structural frames with diagonals
1/750	Difficulties with machinery sensitive to settlement

2.2 Determination of Bearing Capacity Using Empirical Equations

Bearing capacity equations have been developed by several methods. Some of them are mentioned below:

(a) Rankine's equation, (b) Prandtl's equation, (c) Terzaghi's equation, (d) Meyerhof's equation, (e) Brinch Hansen's equation, (f) Vesic's equation, (g) Balla's equation, (h) Skempton's equation, (i) Caquot and Kerisel equation, (j) Frohlich's equation, etc.

2.2.1 Rankine's Equation

Rankine (1885) gave the theory of bearing capacity considering the plastic equilibrium of two adjacent soil elements, one immediately beneath the footing and the other just beyond the edge of the footing. The following equation gives an approximate value of the ultimate bearing capacity q_{ult} of the cohesionless soil.

$$q_{ult} = \gamma D\left[\frac{1+\sin\phi}{1-\sin\phi}\right]^2 \tag{2.1}$$

As the equation (2.1) does not give reliable values, it is rarely used for the determination of the ultimate bearing capacity of the soils. It has been superseded by Terzaghi's theory and other theories which give more dependable values. Rankine did not consider cohesion intercept (c') of the soil. The theory gives the bearing capacity of the soil as zero if D = 0. This is contrary to experience. These are the limitations of the theory. Equation (2.1) is occasionally used to determine the minimum depth of foundation, $D_{min.}$ It can be as

$$D_{\text{min.}} = \frac{q}{\gamma}\left[\frac{1-\sin\phi'}{1+\sin\phi'}\right]^2 \tag{2.2}$$

Where q is the intensity of loading at base.

2.2.2 Terzaghi's Equation

An analysis of the condition of complete bearing capacity failure, usually termed general shear failure, can be made by assuming that the soil behaves like an ideally plastic material. The concept was first was developed by Prandtl, later extended by Terzaghi, Meyerhof and others. Terzaghi (1929) derived a general bearing capacity equation from a modification of equations proposed by Prandtl as given below.

$$q_{ult} = cN_c + \gamma DN_q + 0.5\gamma BN_\gamma \tag{2.3}$$

The parameters N_c, N_q and N_y are the dimensionless bearing capacity factors.

From equation (2.3) it can be concluded that for purely cohesive soils ($\phi = 0$) the bearing capacity of soil depends upon the cohesion of the soil, the depth of footing below the ground level and the unit weight of the soil but it is independent of the width of the footing as the width B does not appear in the following equation.

$$q_{ult} = cN_c \tag{2.4}$$

In the case of cohesionless soil ($c = 0$), q_{ult} becomes as follows:

$$q_{ult} = \gamma DN_q + 0.5\gamma BN_\gamma \tag{2.5}$$

Equation (2.5) shows that ultimate bearing capacity equation of cohesionless soil that depends on unit weight of the soil γ,width of the footing B and depth of the footing D.

Terzaghi also gave the semi-empirical equations for square, circular and rectangular footings which are mentioned below:

(a) For circular footing on c-ϕ soil: $q_u = 1.3cN_c + \sigma'. N_q + 0.3\,\gamma BN_\gamma$ (2.6a)

where B = diameter of the footing.

(b) For square footing on c-ϕ soil: $q_u = 1.3cN_c + \sigma'. N_q + 0.4\,\gamma BN_\gamma$ (2.6b)

where B = width or length of footing.

(c) For rectangular footing on c-ϕ soil:

$$q_u = cN_c\,(1 + 0.3B/L) + \sigma'. N_q + 0.4\,\gamma BN_\gamma \tag{2.6c}$$

(d) For rectangular and square footing on non-cohesive soil:

$$q_u = \sigma'. N_q + 0.4\,\gamma BN_\gamma \tag{2.6d}$$

(e) For circular footing on non-cohesive soil:

$$q_u = \sigma'. N_q + 0.3\,\gamma BN_\gamma \tag{2.6e}$$

2.2.3 Prandtl's Equation

Using the theory of plasticity, Prandtl (1921) developed expressions for the ultimate bearing capacity for a strip footing. For purely cohesive soils ($\phi' = 0$), Prandtl's analysis gives the following equation for the ultimate bearing capacity,

$$q_{ult} = (\pi + 2)\;c_u \;= 5.14\;c_u \tag{2.7}$$

The theory is applicable for the footings at the surface. For the footing at a depth (D) below the surface, an allowance can be made by increasing the bearing capacity by γD . Hence for strip footing on cohesive soil,

$$q_{ult} = \;5.14\;c_u + \gamma D \tag{2.8}$$

2.2.4 Meyerhof 's Equation

Meyorhof (1963) proposed the equation on consideration of shape, depth and inclination factors,

For vertical load: $q_{ult} = cN_cs_cd_c + qN_qs_qd_q + 0.5\gamma B'N_\gamma s_\gamma d_\gamma$ (2.9a)

For inclined load: $q_{ult} = cN_cd_ci_c + q\,N_qd_qi_q + 0.5\gamma B'N_\gamma d_\gamma i_\gamma$ (2.9b)

2.2.5 Hansen's Equation

Hansen (1970) proposed the bearing capacity equation on consideration of shape, depth and other factors,

$$q_{ult} = cN_c s_c d_c i_c g_c b_c + q\, N_q s_q d_q i_q g_q b_q + 0.5\gamma B' N_\gamma\, s_\gamma d_\gamma i_\gamma\, g_\gamma\, b_\gamma \qquad (2.10)$$

2.2.6 Vesic's Equation

Vesic (1973, 1975) supported the Hansen's equation for shape, depth and other factors but changed the value of $N_\gamma = 2\ (N_q + 1)\tan\phi$. N_c and N_q are same as Meyerhof.'s values.

2.2.7 Skempton's Equation

Skempton (1951) observed that the factor N_c increases with the ratio D/B. He found that for purely cohesive soil, N_c has a maximum value of 9 for square or circular footing and 7.5 for strip footing. His recommendations may be summarized as follows:

(i) When $D = 0$, $N_c = 5.14$ (for strip footing) (2.11a)

$N_c = 6.2$ (for square or circular footing) (2.11b)

(ii) At depths $D/B < 2.5$, $N_c = \left(1 + 0.2\dfrac{D}{B}\right)[N_C]_{surface}$ (2.11c)

Value of $[N_c]_{surface}$ may be roughly taken as 5 for surface strip footing and as 6 for square or circular footing.

(iii) At depths $D/B > 2.5$, $N_c = 1.5[N_C]_{surface}$ (2.11d)

(iv) For rectangular footing:

$$[N_c]_{rect} = \left(1 + 0.2\frac{B}{L}\right)[N_c]_{strip} \qquad (2.11e)$$

or, $$[N_c]_{rect} = 5\left(1 + 0.2\frac{B}{L}\right)\left(1 + 0.2\frac{D}{B}\right) \qquad (2.11f)$$

2.3 Determination of Bearing Capacity in the Field

The bearing capacity of soil can be determined both by in-situ test and The in-situ field tests for a project may consist of any one or more of the following tests:

(i) Standard penetration test (SPT) in boreholes,

(ii) Dynamic cone penetration test (DCPT),

(iii) Static cone penetration test (CPT),

(iv) Vane shear test

(v) Plate load test

2.3.1 Bearing Capacity Based on N-Value

The standard penetration resistance is determined at a number of selected points at intervals of 75 cm in the vertical direction or change of strata if it takes place earlier and the average value beneath each point is determined between the level of base of the footing and the depth equal to 1.5 to 2 times the width of foundation. In computing the value, any individual value more than 50 percent of the average calculated shall be neglected and average recalculated (the values for all loose seams shall however be included). Knowing the value of N, the value of ϕ is read from Fig. 2.1. The ultimate bearing capacity is then calculated from the chosen empirical formula.

The standard penetration number, N is corrected for dilatancy correction and over burden correction.

(i) **Dilatancy Correction**

Silty fine sand and fine sands below the water table develop pore pressure which is not easily dissipated. The pore pressure affects the resistance of the soil and hence the penetration number (N). Terzaghi and Peck recommended the following correction in the case of silty fine sands when the observed value of N_R exceeds 15,

$$N_c = 15 + 1/2\,(N_R - 15) \qquad (2.12)$$

where, N_R = Recorded value and N_c = Corrected value.

If $N_R \leq 15$, $N_c = N_R$

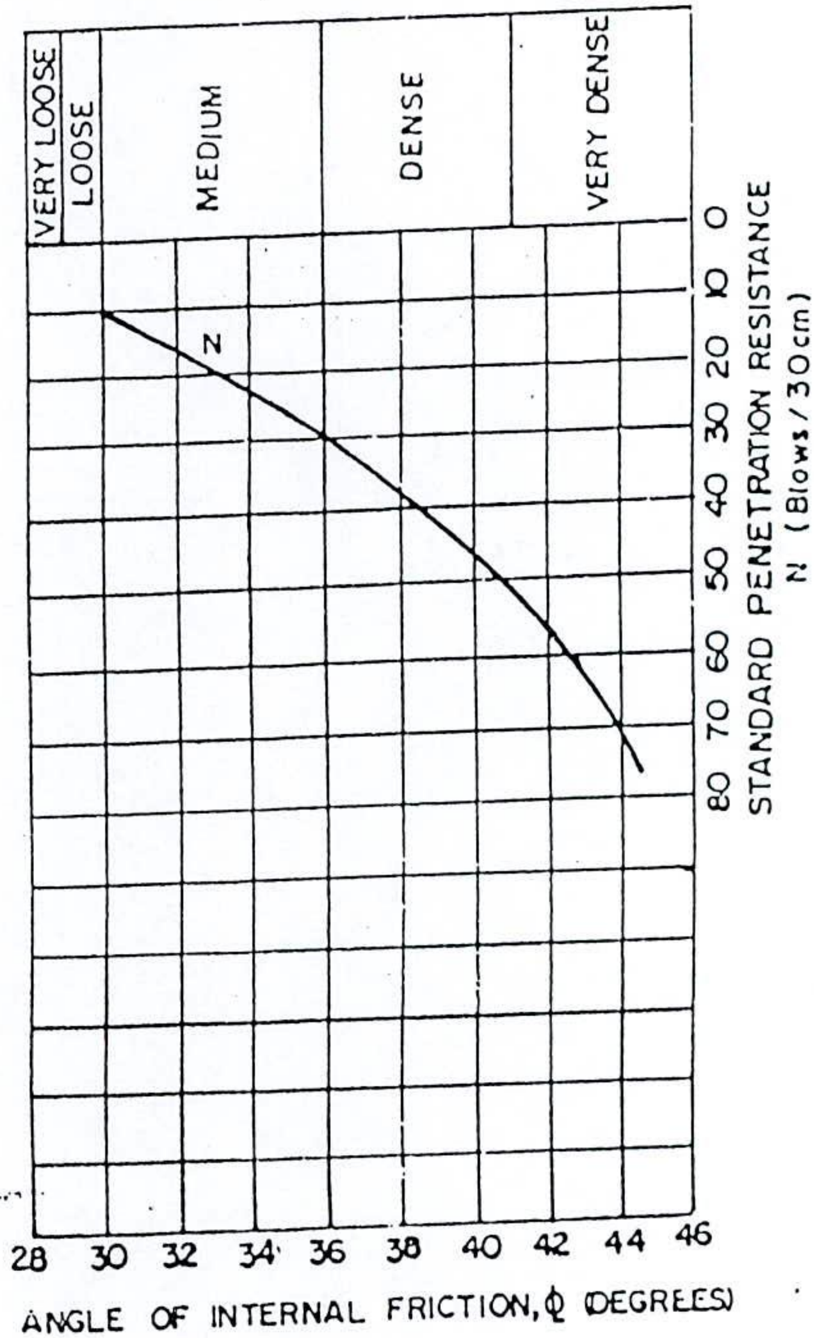

FIG. 2.1: RELATIONSHIP BETWEEN ϕ AND N.

ii) **Overburden Correction**

For a constant density index, the N value increases with increasing effective over burden pressure for which correction have been proposed by Gibbs and Holtz (1957), Peck, Thornburn (1963), Whitman and others. Peck proposed that N values be reported at a reference over burden pressure 100 kN/m^2, and the normalized value of N be expressed as

$$N_0 = C_n N \qquad (2.13a)$$

Where N_0 = Correct value for overburden effect.

N = actual value, C_n = normalizing factor,

$$C_n = 0.77 \log_{10} (200/P') \qquad (2.13b)$$

Where P' = effective over burden pressure (kN/m^2) at the test level.

The above correction is valid for $P' > 25\ kN/m^2$

2.3.2 Bearing Capacity Based on Static Cone Penetration Test

The static cone point resistance, q_c is determined at a number of selected points at intervals of 10 cm to 15 cm as per IS: 4968 (Part III, 1976). The observed values are corrected for the dead weight of sounding rods. Then the average value at each one of the locations is determined between the level of the base of the footing and the depth equal to 1.5 to 2 times the width of the footing. The average of static cone point resistance value is determined for each one of the location and the minimum of the average value is used in the design.

2.3.3 Plate Load Test

In this test the sand is loaded through a steel plate at least 300 mm square, readings of load and settlement being observed up to failure or to at least 1.5 times the estimated allowable bearing capacity. The load increments should be approximately one-fifth to one-fourth of the estimated allowable bearing capacity. The test plate is generally located at foundation level in a pit at least 1.5 m square. The test is reliable only if the sand is reasonably uniform over the significant depth of the full-scale foundation. Minor local weaknesses near the surface will influence the results of the test while having no appreciable effect on the full-scale foundation. On the other hand, a weak stratum below the significant depth of the test plate but within the significant depth of the foundation would have no appreciable effect on the performance of the foundation.

Settlement in a sand increases as the size of the loaded area increases and the main problem with the use of plate bearing tests is the extrapolation of the settlement of a test plate to that of a full-scale foundation. The required correlation appears to depend on the relative density, particle size distribution and stress history of the sand, and at present there is no reliable method of extrapolation. Bjerrum and Eggestad (1963), for example, from a study of case records, showed that there is a considerable scatter in the relationship between settlement and the size of the loaded area for a given pressure. Ideally, plate bearing tests should be carried out at different depths and using plates of different sizes in order that extrapolations may be made, but this is generally ruled out on economic grounds: further problems would be introduced if the tests had to be carried out below water table level.

Since a load test is of short duration, consolidation settlements can not be predicted. The test gives the value of immediate settlement only. If the underlying soil is sandy in nature immediate settlement may be taken as the total settlement. If the soil is clayey type, the immediate settlement is only a fraction of the total settlement. Load tests, therefore, do not have much significance in clayey soils to determine allowable pressure on the basis of settlement criterion. If the soil is not homogeneous to a great depth, plate load tests give very misleading results. So, this test is not at all recommended in soils which are not homogeneous at least to a depth equal to 1.5 to 2 times the width of the prototype foundation. The procedure of plate load test is described below.

Procedure of Load Test:

(a) Excavate a pit of size $5B_p$ x $5B_p$ at the depth of foundation, d_f. Here Bp = width of plate.

(b) Use a size of rigid steel plate = 0.30 m square = 1ft x 1ft (thickness = 25mm)

(c) Make a hole (30 cm x 30 cm) at the centre of pit.

(d) The plate is firmly seated in the hole, and if the ground is slightly uneven, a thin layer of sand is spread underneath the plate.

(e) Load may be applied by hydraulic jack from loading platform above ground surface.

(f) First apply a seating load of 7 kPa which is released after some time.

(g) Load increment will be about 20% of the estimated safe load or 10% of ultimate bearing capacity.

(h) Settlement of the plate is observed by at least 2 dial gauges fixed at diametrically opposite ends, with sensitivity of 0.02 mm.

(i) Settlement is recorded for each increment of load after an interval of 1, 5, 10, 20, 40, 60 min. and thereafter at hourly intervals. These hourly observations are continued for clayey soils until the rate of settlement is less than 0.2 mm or 0.25 mm / hr.

(j) After this, the next load increment is applied.

(k) The maximum load that is to be applied corresponds to 1.5 times the estimated ultimate load or to 3 times the proposed allowable bearing pressure.

(l) The test should continue until a total settlement of about 25 mm or the settlement, at which the soil fails, whichever is earlier, is obtained.

(m) After the load is released, the elastic rebound of the soil should be recorded.

(n) From the test results, a load-settlement curve should be plotted in normal scale & log-log scale.

(o) From this curve ultimate bearing capacity can be determined.

(p) For clayey soils, ultimate bearing capacity of footing $q_{u(f)} = q_{u(p)}$.

(q) For sandy soil, $q_{u(f)} = q_{u(p)} \times (B_f / B_p)$.

(r) For clayey soils, settlement of footing, $S_r = S_p \times (B_f / B_p)$

(s) For sandy soils, settlement of footing,

$$S_f = S_p \times [\{B_f (B_p + 0.3)\} / \{B_p (B_f + 0.3)\}]^2$$

Where B_f and B_p in metres.

In order to determine the safe bearing capacity it would be normally sufficient to use a F.S. of 2 to 2.5 on ultimate bearing capacity.

If the water table is already above the level of the footing, it should be lowered by pumping and the bearing plate sealed after the water table has been lowered just below the footing level. Even if the water table is located above 1 m below the base level of the footing, the load test should be made at the level of the water table itself.

2.4 Bearing Capacity of Sands

In this section the term sand includes gravelly sand, silty sands and non-plastic silts. Most sand deposits are non-homogenous and the allowable bearing capacity of shallow foundations is limited by settlement considerations except in the case of narrow footings. In most situations the allowable settlement is reached at a pressure for which the factor of safety against shear failure is greater than 3. in the case of narrow footings, however, shear failure may be the limiting consideration. Other factors being equal, the pressure that will produce the allowable settlement in a dense sand will be greater than that to produce the allowable settlement in a loose sand. Settlement in sand is rapid and occurs almost entirely during construction and initial loading. Settlement, therefore, should be estimated using the dead load plus the maximum live load.

Differential settlement between a number of footings is governed mainly by variations in the homogeneity of the sand within the significant depth and to a lesser extent by variations in foundation pressure. According to Terzaghi and Peck (1967) settlement records indicate that the differential settlement between footings of approximately equal size carrying the same pressure is unlikely to exceed 50% of the maximum settlement. If the footings are of different size the differential settlement will be greater. The maximum settlement of footings carrying the same pressure increases with increasing footing size. There is no appreciable difference between the settlement of square and strip footings of the same width. For a given pressure and footing size the settlement decreases slightly with increasing footing depth below ground level due to the fact that the lateral confining pressure will be greater. In most cases, even under extreme variations of footing size and depth, it is unlikely that differential settlement will be greater than 75 % of the maximum settlement. A few cases have been reported, however, in which the differential settlement was almost equal to the maximum settlement.

A reasonable design criteria for footings on sands is an allowable maximum settlement of 25 mm. The differential settlement between any two footings is then likely to be less than 20 mm. Differential settlement may be decreased by reducing the size of the smallest footings, provided the factor of safety with respect to shear failure remains above the specified value.

The allowable bearing capacity of a sand depends primarily on the relative density, stress history, the position of the water table relative to foundation level and the size of foundation. Of secondary importance are particle shape and size distribution. The relative density has a dominating influence on the magnitude of settlement for a particular foundation and on the value of the shear strength parameter ϕ'. The magnitude of the settlement is also influenced by the stress history of the deposit, i.e., whether the sand is normally loaded or preloaded. The water table position affects both the settlement and the ultimate bearing capacity. If the sand within the significant depth is fully saturated the effective unit weight is roughly halved, resulting in a reduction in lateral confining pressure and corresponding increase in settlement: the reduced effective unit weight will also result in a lower value of ultimate bearing capacity.

Due to extreme difficulty of obtaining undisturbed sand samples for laboratory testing and to the inherent heterogeneity of sand deposits, the allowable bearing capacity is normally estimated by means of correlations based on the results of in-situ tests. The tests in question are plate bearing tests and dynamic or static penetration tests.

2.5 Measures of Correlation

It can be determined in a qualitative manner how well a given line or curve describes the relationship between variables by direct observation of the scatter diagram itself. If we are to deal with the problem of scattering of sample data about lines or curves in quantitative manner it will be necessary for us to devise measures of correlation. The measure of linear relationship between two variables X and Y is estimated by the sample correlation coefficient r, where

$$r = \frac{\sum xy}{\sqrt{\left[\left(\sum x^2\right)\left(\sum y^2\right)\right]}} \tag{2.14}$$

Where $x = X - X_{mean}$ and $y = Y - Y_{mean}$

For the value of r between -1 and +1 we must be careful in our interpretation. For example, value of r equal to 0.3 and 0.6 only mean that we have two positive correlations, one some what stronger than the other. It is wrong to conclude that $r = 0.6$ indicates a linear relationship twice as good as that indicated by the value $r = 0.3$, on the other hand, if we consider r^2, then $100 \times r^2$ % of the variation in the values of Y may be accounted for by the linear relationship with the variable X. Thus a correlation of 0.6 means that 36 % of the variation of the random variable Y is accounted for by differences in the variable X.

2.3.4 Correlation Between Hand Penetrometer Test Value and Bearing Capacity of Cohesive Soil

Prodip (2001) has established a relationship between hand penetrometer test value and bearing capacity of cohesive soil in terms of unconfined compressive strength, q_u. He used three types of hand penetrometer (diameters of 18.75, 25.00 and 31.25 mm) of the same sizes as the author used for the present project. He found from his study that the coefficient of correlations for the hand penetrometers of diameters 18.75 mm (0.75 in.), 25.00 mm (1.0 in.) and 31.25 mm (1.25 in.) are respectively 0.49, 0.97 and 0.64. This result indicates that correlation is very well for 25 mm diameter hand penetrometer. Prodip (2001) presented the following correlations for the cohesive soil.

$$Y = 4.6013\, X + 39.585 \quad \text{for 18.75 mm dia. Penetrometer} \tag{2.15a}$$

$$Y = 4.9569\, X + 9.9766 \quad \text{for 25.00 mm dia. Penetrometer} \tag{2.15b}$$

$$Y = 2.3373\, X + 27.8820 \quad \text{for 31.25 mm dia. Penetrometer} \tag{2.15c}$$

CHAPTER 3

EQUIPMENTS AND INSTRUMENTATION

3.1 General

For carrying out the tests required in this investigation a number of instruments and equipments have been used. The basic equipments and instruments used for the investigation are described in this chapter.

3.2 Equipments Used

A soil survey, no matter how detailed, can never cover the entire site. The recommendations obtained from the soil investigation safe bearing capacity is calculated. But real problem lies with the site engineer who has to verify quickly whether the specified value is available or not. Undisturbed soil sample could undoubtedly be taken for laboratory tests, but the time involved in the process would make it rather impacted. In situ tests such as standard penetration test (SPT), cone penetration test (CPT), plate load test are even more time consuming and too expensive to make them a practical proposition. Hence to overcome these difficulties hand penetrometer (specially designed) will be helpful which is very simple to handle and operate. It is possible to correlate the value of hand penetrometer test value (N_h) with friction angle (ϕ) which is the main parameter to determine the bearing capacity of granular soil. Thus it may be helpful to predict the bearing capacity of soil by specially designed hand penetrometer in many cases such as compaction control project, lightly loaded shallow foundation in which case the owner is not at all interested to perform costly soil investigation work etc. Three numbers of hand penetrometer of diameters 0.75" (18.75mm), 1" (25mm) and 1.25" (31.25mm) were used which were made by MS rod. The total length of each penetrometer is about 1.5 m and total system is shown in Figure 3.1. A collar is subject to the middle third of the rod. The rod is divided into two parts. Upper part is one third and lower part is two third of the total length of the rod. One end of the lower part of the rod is tapered to a cone while the other end is threaded.

Both ends of upper part of the rod are threaded. A circular steel disc weighting 10 kg is allowed to slide freely through the rod. An adjustable nut is then adjusted so that the distance from the top the disc to the under side of the nut is exactly 375 mm. At the site the instrument is then hold vertically by one person to the spot where the test is required. A second person is then required to lift the disc slowly up to the bottom of the nut and allow it to fall freely from that height to the top of the collar. This test procedure is almost like standard penetration test but hammer weight, height of fall and fabrication of instrument are not same as split spoon sampler. The number of blows required for the initial 15 cm is ignored to allow for any soil disturbance. The number of blows required for the penetration of the last 30 cm of the rod is taken as the penetration test value or penetration number (N_h) of the penetrometer.

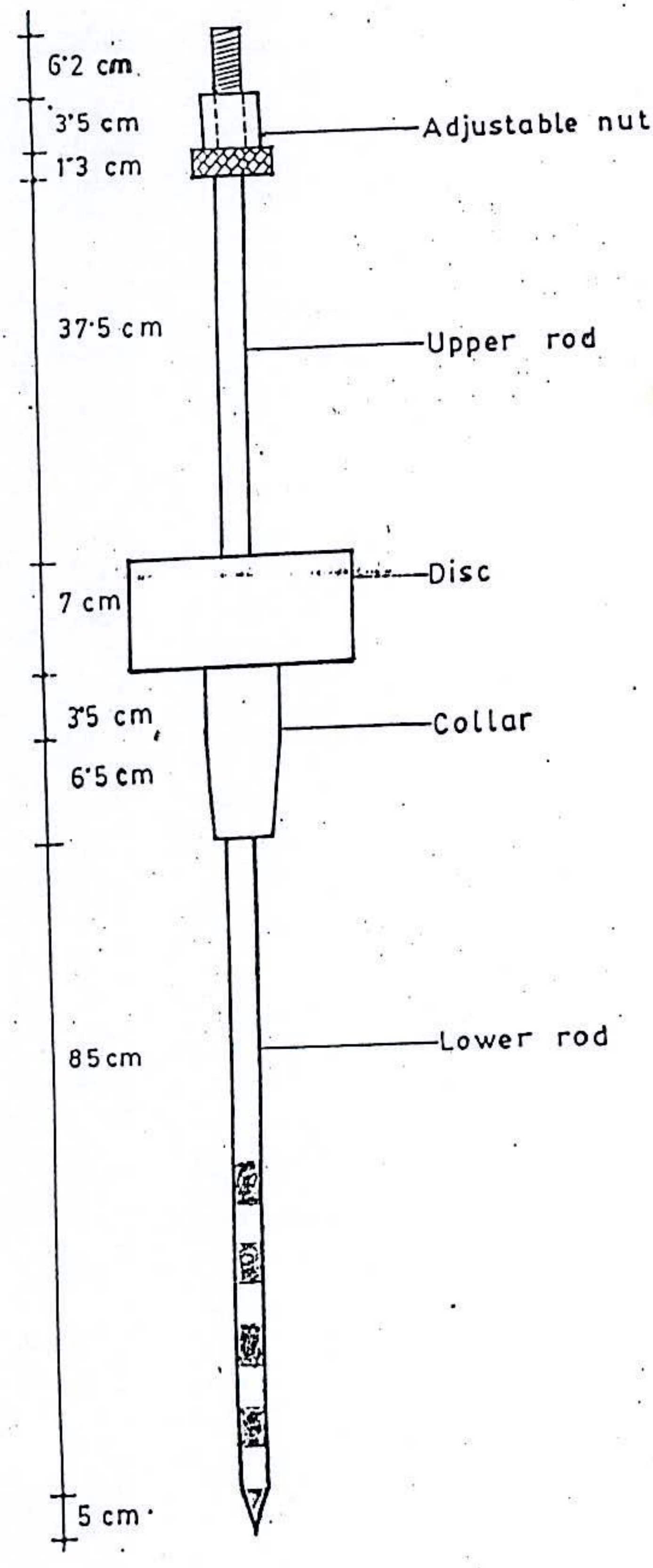

FIG. 3·1(a) HAND PENETROMETER.

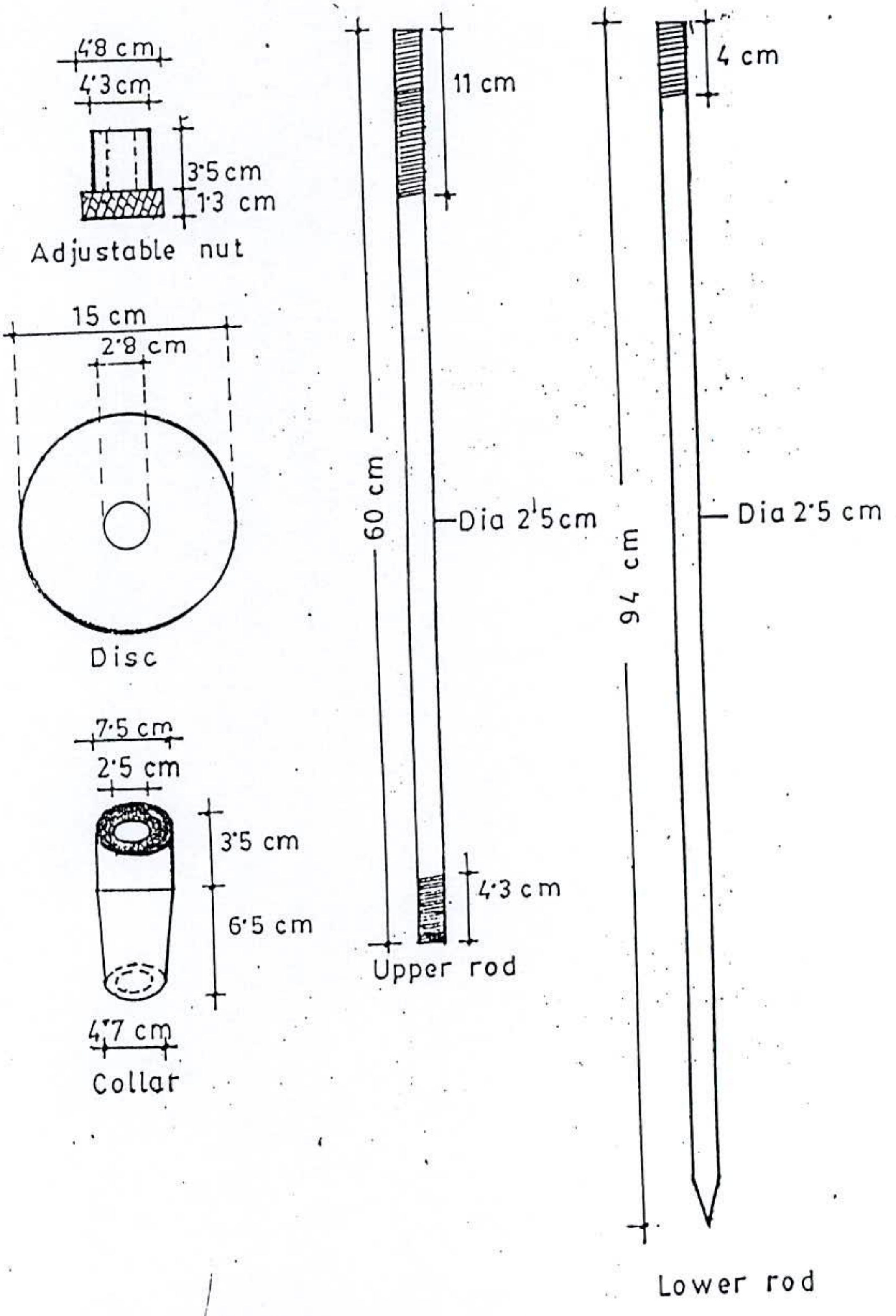

FIG. 3·1(b) DETAILS OF THE HAND PENETROMETER

3.3 Preparation of Model Test Bed

A model open test bed of length 90 cm, width 90 cm and height 105 cm was fabricated near the laboratory of Civil Engineering Department, KUET as shown in Fig. 3.2. The bottom of the test bed was constructed by cement concrete and the walls of 12.5 cm thickness on four sides of the test bed was constructed by brick. Two small holes were provided near the bottom in each side of the test bed to escape excess water from bed.

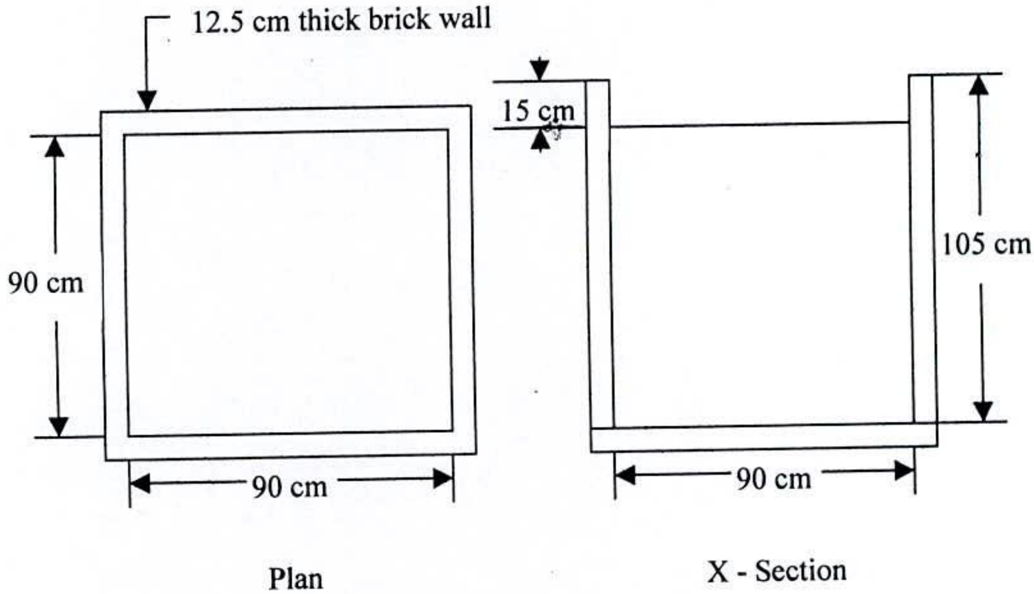

Fig. 3.2 Open Test Bed to Prepare Artificial Sand Bed Above Ground Surface

CHAPTER 4

LABORATORY INVESTIGATIONS

4.1 General

The use of disturbed samples of soils for testing would be very desirable in the investigation of their behavior. Such samples are seldom uniform due to the complex geological conditions acted upon them and as such, from the test results on such samples, it is rather difficult to generalize the behavior of soils. Therefore, to study any specific effect on the behavior of soils, it is considered essential to use uniform reconstituted samples prepared under controlled conditions in the laboratory (Hvorslev, 1960). The laboratory investigations made on selected samples have been described in details in this chapter.

4.2 Soil Samples Used for the Test

In this investigation sand samples were used as granular soil. Three sizes of sand namely, Sylhet sand, Kushtia sand and local sand were selected for this purpose. Disturbed samples were collected from local business center at Fultala. To prepare 13 types artificial samples, these three types of samples were mixed with different proportions to obtain different fineness moduluses.

4.3 Preparation of Samples

Practically it was not possible to obtain granular field having sand in the upper layers in the region of south-west of Bangladesh. For this region an artificial sand bed was prepared in a box above the ground level. Thirteen sand beds were prepared in the box of which 3 by original collected sands and 10 by mixing three collected sands among them in various proportions. The mixing work was done manually. The original and mixing 13 sand beds are designated by sample S1, sample S2, sample S3 up to Sample S13. Samples designation and different mixing proportions of thirteen beds are shown in Table 4.1.

Table 4.1 Samples Designation and Mixing Proportions of Sand Bed

Sample Designation for Each Sand Bed	Mixer of Granular Soils	Mixing Ratio of Sand Bed
S1	Sylhet sand	-
S2	Sylhet sand : Kushtia sand	3:1
S3	Sylhet sand : Local sand	3:1
S4	Sylhet sand : Kushtia sand	1:1
S5	Sylhet sand : kushtia sand	1:3
S6	Sylhet sand : Kushtia sand : Local sand	1:1:1
S7	Sylhet sand : Local sand	1:3
S8	Sylhet sand : Local sand	1:1
S9	Kushtia Sand	-
S10	Kushtia sand : Local sand	3:1
S11	Kushtia sand : Local sand	1:1
S12	Kushtia sand : Local sand	1.3
S13	Local sand	-

4.4 Determination of Fineness Modulus of Samples

From grain size analysis, fineness modulus of each sand sample was determined. For this test, No. 4 (4.76 mm opening size), No. 8 (2.38 mm opening), No.16 (1.19 mm opening), No.30 (0.59 mm opening), No.50 (0.297 mm opening) and No.100 (0.149 opening) sieves were used for sieving 500 gm oven-dried sample of each sand bed. Detailed calculations of thirteen tests on thirteen sand beds (S1 to S13) are mentioned in the Tables 4.2 to 4.14. Table 4.15 shows the Fineness Modulae (F.M.) of different samples.

Table 4.2 Fineness Modulus Test on Sample S1

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained(gm)	Cumulative % Retained	% Finer
S1	4	4.75	0	0	0	100
	8	2.36	4.6	4.6	0.92	99.08
	16	1.18	38.2	42.8	8.56	91.44
	30	0.60	139.2	182.0	36.40	63.60
	50	0.30	241.5	423.5	84.70	15.30
	100	0.15	64.5	488.0	97.60	2.40
F.M. of Sample S1 = 2.28						

Table 4.3 Fineness Modulus Test on Sample S2

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S2	4	4.75	0	0	0	100
	8	2.36	7.3	7.30	1.46	98.54
	16	1.18	29.9	37.20	7.44	92.56
	30	0.60	103.7	140.90	28.18	71.82
	50	0.30	225.8	366.70	73.34	26.66
	100	0.15	104.1	470.80	94.16	5.84
F.M. of the Sample = 2.05						

Table 4.4 Fineness Modulus Test on Sample S3

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S3	4	4.75	0	0	0	100
	8	2.36	6.1	6.1	1.22	98.78
	16	1.18	32.4	38.5	7.70	92.30
	30	0.60	112.7	151.2	30.24	69.76
	50	0.30	188.9	340.1	68.02	31.98
	100	0.15	97.8	437.9	87.58	12.42
F.M. of the Sample = 1.95						

Table 4.5 Fineness Modulus Test on Sample S4

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S4	4	4.75	0	0	0.00	100
	8	2.36	3.6	3.60	0.72	99.28
	16	1.18	25.0	28.60	5.72	94.28
	30	0.60	79.4	108.00	21.60	78.40
	50	0.30	236.5	344.50	68.90	31.10
	100	0.15	121.5	466.00	93.20	6.80
F.M. of the Sample = 1.90						

Table 4.6 Fineness Modulus Test on Sample S5

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S5	4	4.75	0	0	0	100
	8	2.36	2.7	2.70	0.54	99.46
	16	1.18	11.4	14.10	2.82	97.18
	30	0.60	44.9	59.00	11.80	88.20
	50	0.30	227.9	286.90	57.38	42.62
	100	0.15	169.3	456.20	91.24	8.76
F.M. of the Sample = 1.64						

Table 4.7 Fineness Modulus Test on Sample S6

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S6	4	4.75	0	0	0	100
	8	2.36	4.5	4.5	0.90	99.10
	16	1.18	18.4	22.9	4.58	95.42
	30	0.60	66.6	89.5	17.90	82.10
	50	0.30	181.2	270.7	54.14	45.86
	100	0.15	154.4	425.1	85.02	14.98
F.M. of the Sample = 1.63						

Table 4.8 Fineness Modulus Test on Sample S7

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S7	4	4.75	0	0	0	100
	8	2.36	0.2	0.20	0.04	99.96
	16	1.18	0.3	0.50	0.10	99.90
	30	0.60	13.5	14.00	2.80	97.20
	50	0.30	242.8	256.80	51.36	48.64
	100	0.15	202.8	459.60	91.92	8.08
F.M. of the Sample = 1.46						

Table 4.9 Fineness Modulus Test on Sample S8

Sample Designation	Sieve No (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S8	4	4.75	0	0	0	100
	8	2.36	5.5	5.5	1.10	98.90
	16	1.18	28.1	33.60	6.72	93.28
	30	0.60	81.2	85.20	17.04	82.96
	50	0.30	133.2	218.40	43.68	56.32
	100	0.15	147.5	365.90	73.18	26.82
F.M. of the Sample = 1.42						

Table 4.10 Fineness Modulus Test on Sample S9

Sample Designation	Sieve No (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S9	4	4.75	0	0	0	100
	8	2.36	2.8	2.8	0.56	99.44
	16	1.18	1.3	4.10	0.82	99.18
	30	0.60	7.2	11.30	2.26	97.74
	50	0.30	185.2	196.50	39.30	60.74
	100	0.15	207.6	404.10	80.82	19.18
F.M. of the Sample = 1.24						

Table 4.11 Fineness Modulus Test on Sample S10

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S10	4	4.75	0	0	0	100
	8	2.36	0	0	0	100
	16	1.18	1.0	1.00	0.20	99.80
	30	0.60	6.8	7.80	1.56	98.44
	50	0.30	161.6	169.40	33.88	66.12
	100	0.15	199.0	368.40	73.68	26.32
F.M. of the Sample = 1.09						

Table 4.12 Fineness Modulus Test on Sample S11

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S11	4	4.75	0	0	0	100
	8	2.36	0	0	0	100
	16	1.18	0.8	0.80	0.16	99.84
	30	0.60	10.0	10.80	2.16	97.84
	50	0.30	150.3	161.10	32.22	67.84
	100	0.15	185.4	346.50	69.30	30.70
F.M. of the Sample = 1.04						

Table 4.13 Fineness Modulus Test on Sample S12

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S12	4	4.75	0	0	0	100
	8	2.36	3.5	3.50	0.70	99.30
	16	1.18	2.2	5.70	1.14	98.86
	30	0.60	3.8	9.50	1.90	98.10
	50	0.30	63.9	73.40	14.68	85.32
	100	0.15	277.0	350.40	70.08	29.92
F.M. of the Sample = 0.89						

Table 4.14 Fineness Modulus Test on Sample S13

Sample Designation	Sieve No. (ASTM)	Sieve Opening (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % Retained	% Finer
S13	4	4.75	0	0	0	100
	8	2.36	0	0	0	100
	16	1.18	0.3	0.30	0.06	99.94
	30	0.60	2.0	2.30	0.46	99.54
	50	0.30	32.8	35.10	7.02	92.98
	100	0.15	267.5	302.60	60.52	39.48
F.M. of the Sample = 0.68						

Table 4.15 Fineness Modulus of Different Sand Samples for Sand Beds

Sample designation for each sand bed	F.M.
S1	2.28
S2	2.05
S3	1.95
S4	1.90
S5	1.64
S6	1.63
S7	1.46
S8	1.42
S9	1.24
S10	1.09
S11	1.04
S12	0.89
S13	0.68

4.5 Grain Size Analysis of Granular Soil Samples

From sieve analysis grain size curves of thirteen samples were drawn. Figs. 4.1 to 4.13 show the grain size curves of thirteen granular soil samples which are prepared for thirteen sand beds. For grain size analysis, according to ASTM sieve Nos. 4, 8, 16, 30, 50 and 100 were used to sieve the granular soil (sand).

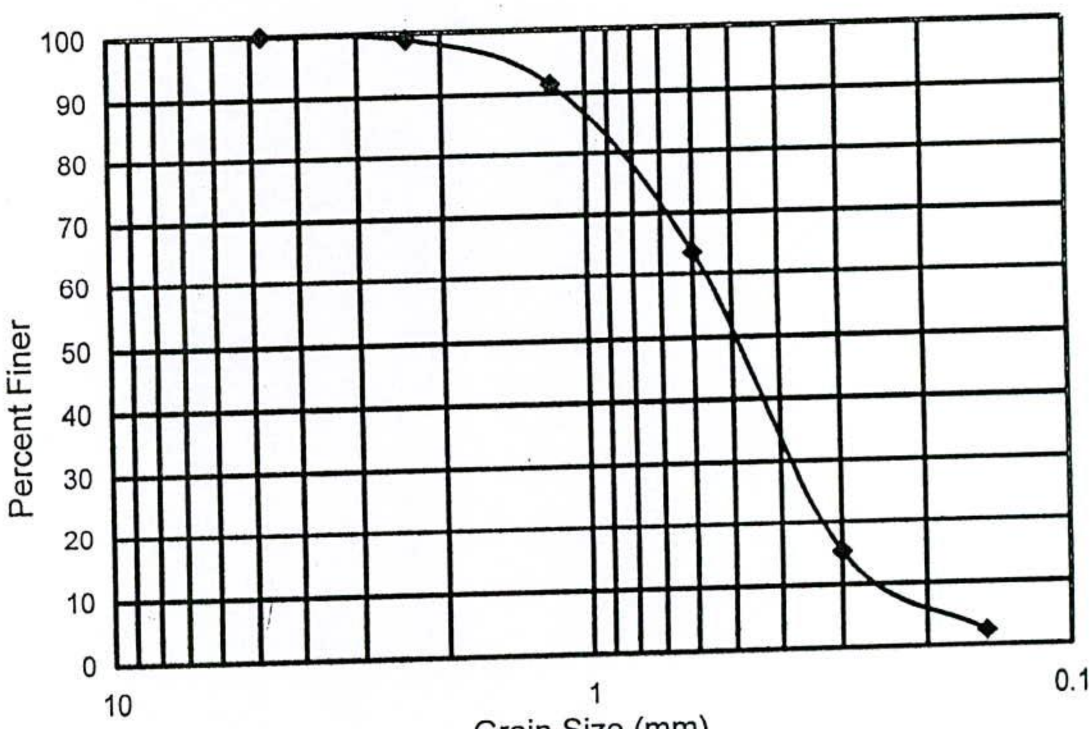

Fig. 4.1 Grain Size Distribution Curve of Sample S1

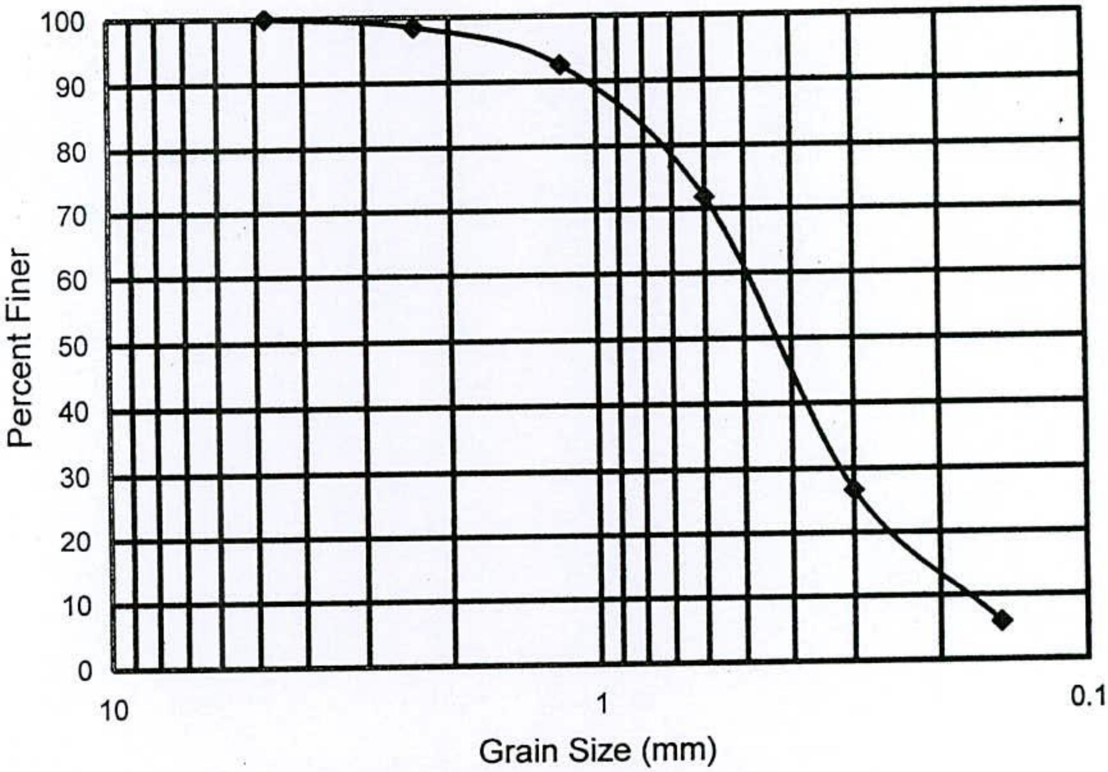

Fig. 4.2 Grain Size Distribution Curve of Sample S2

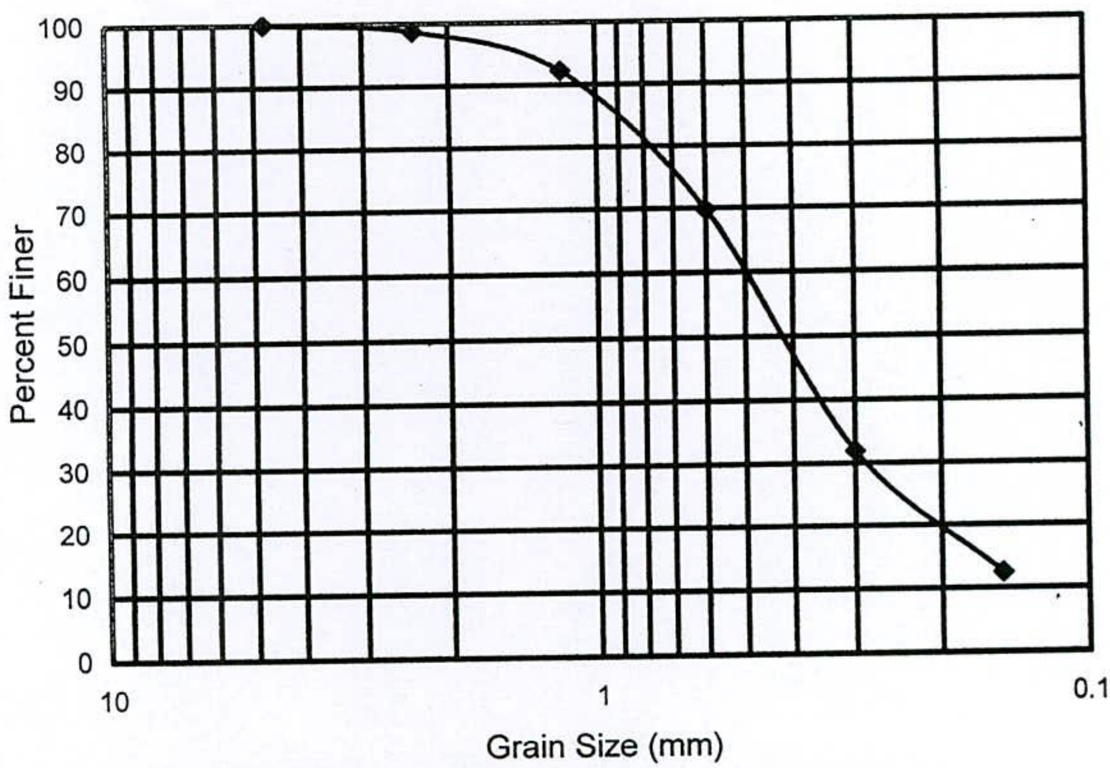

Fig. 4.3 Grain Size Distribution Curve of Sample S3

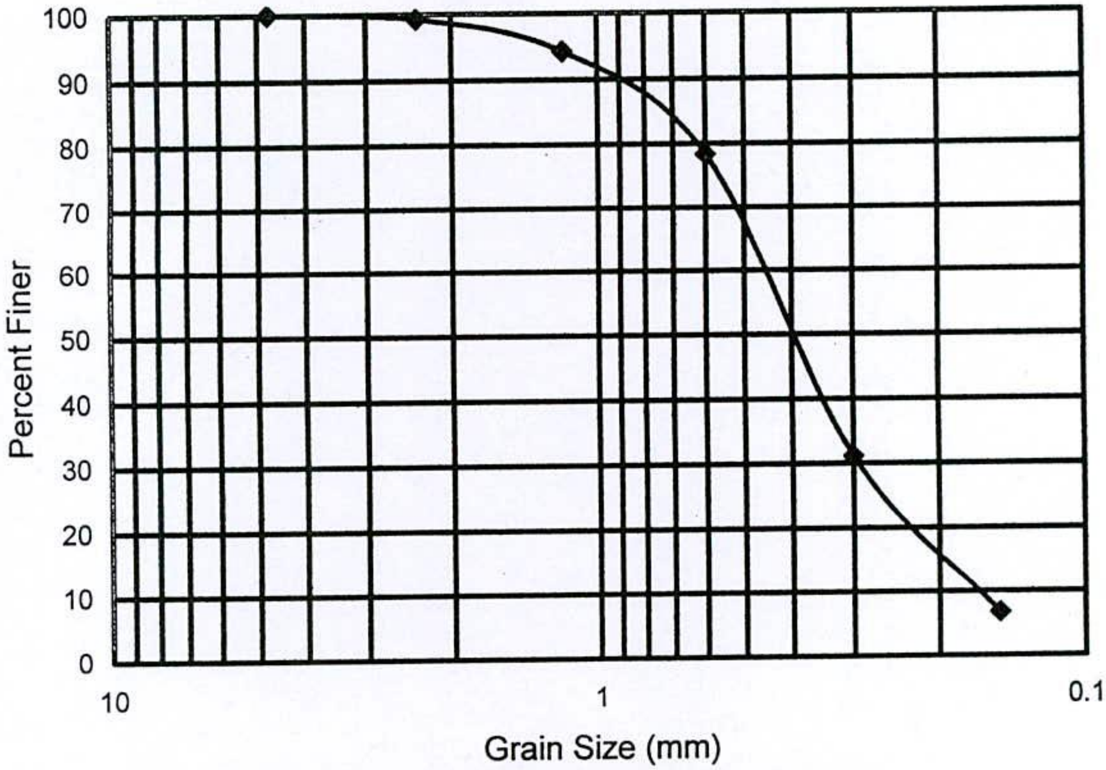

Fig. 4.4 Grain Size Distribution Curve of Sample S4

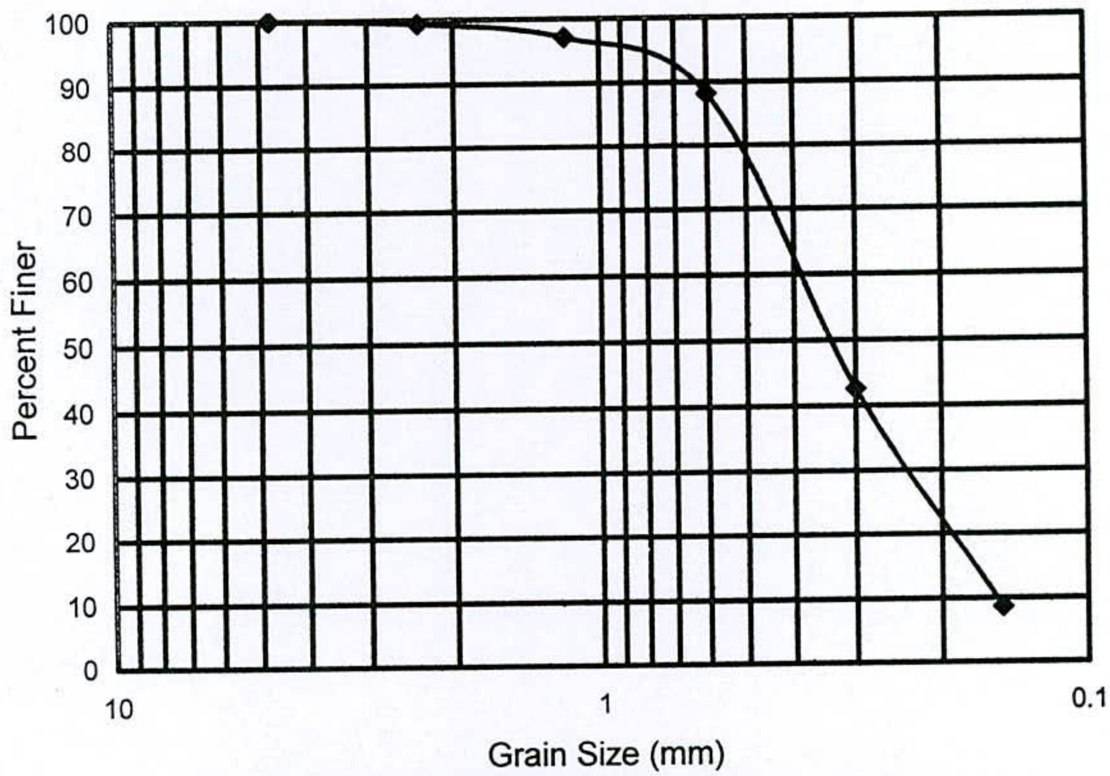

Fig. 4.5 Grain Size Distribution Curve of Sample S5

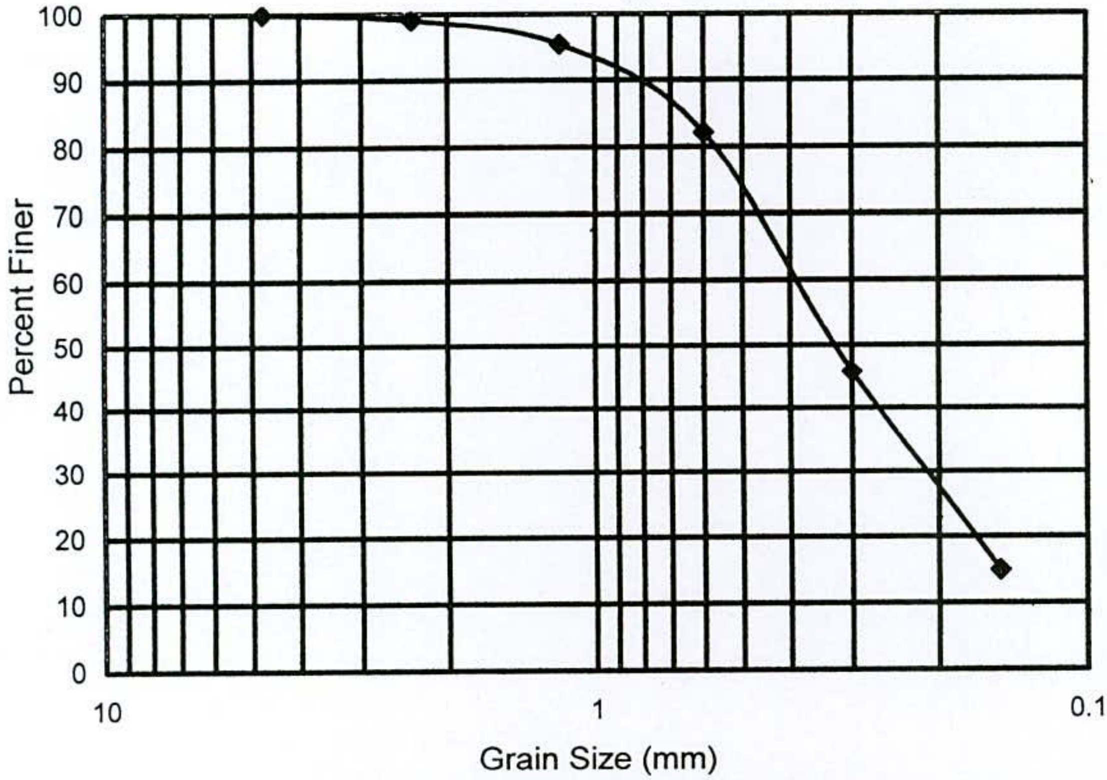

Fig. 4.6 Grain Size Distribution Curve of Sample S6

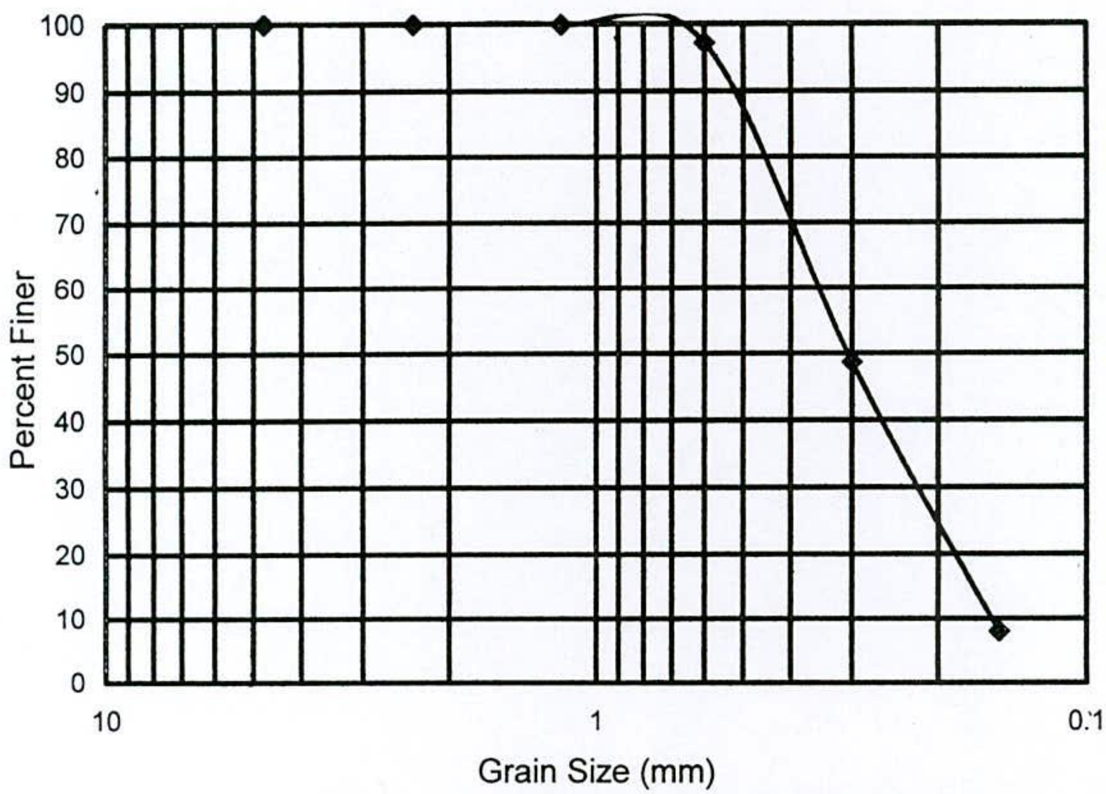

Fig. 4.7 Grain Size Distribution Curve of Sample S7

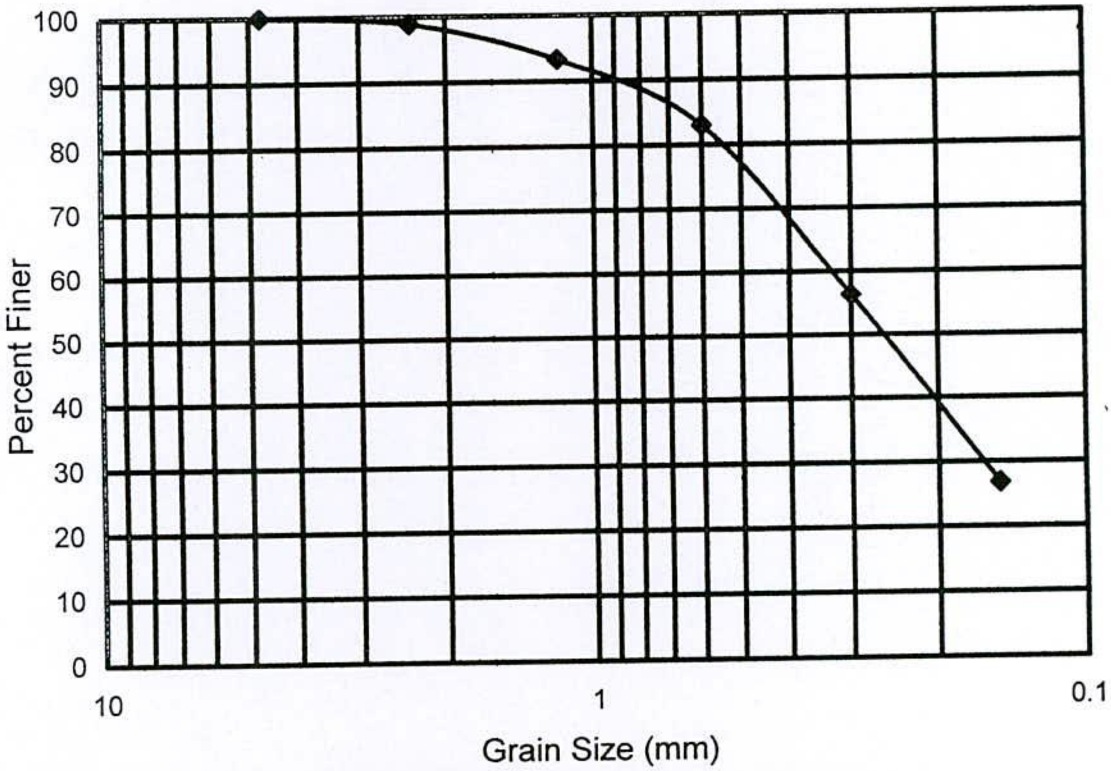

Fig. 4.8 Grain Size Distribution Curve of Sample S8

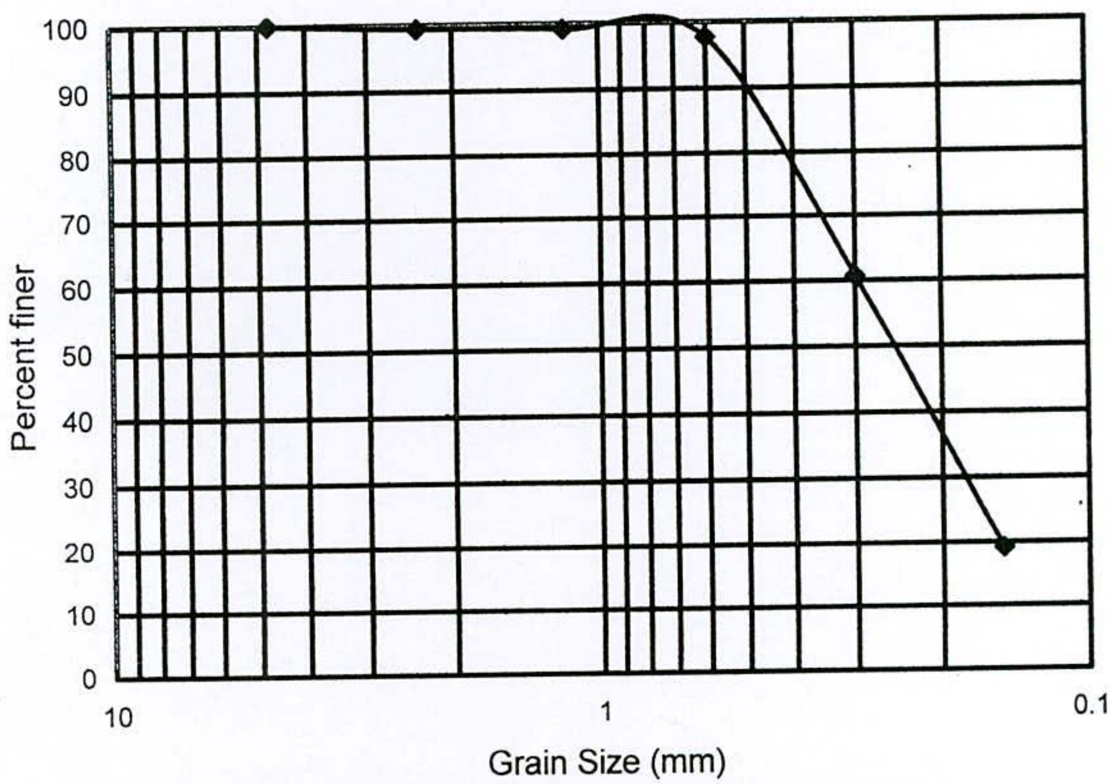

Fig. 4.9 Grain Size Distribution curve of Sample S9

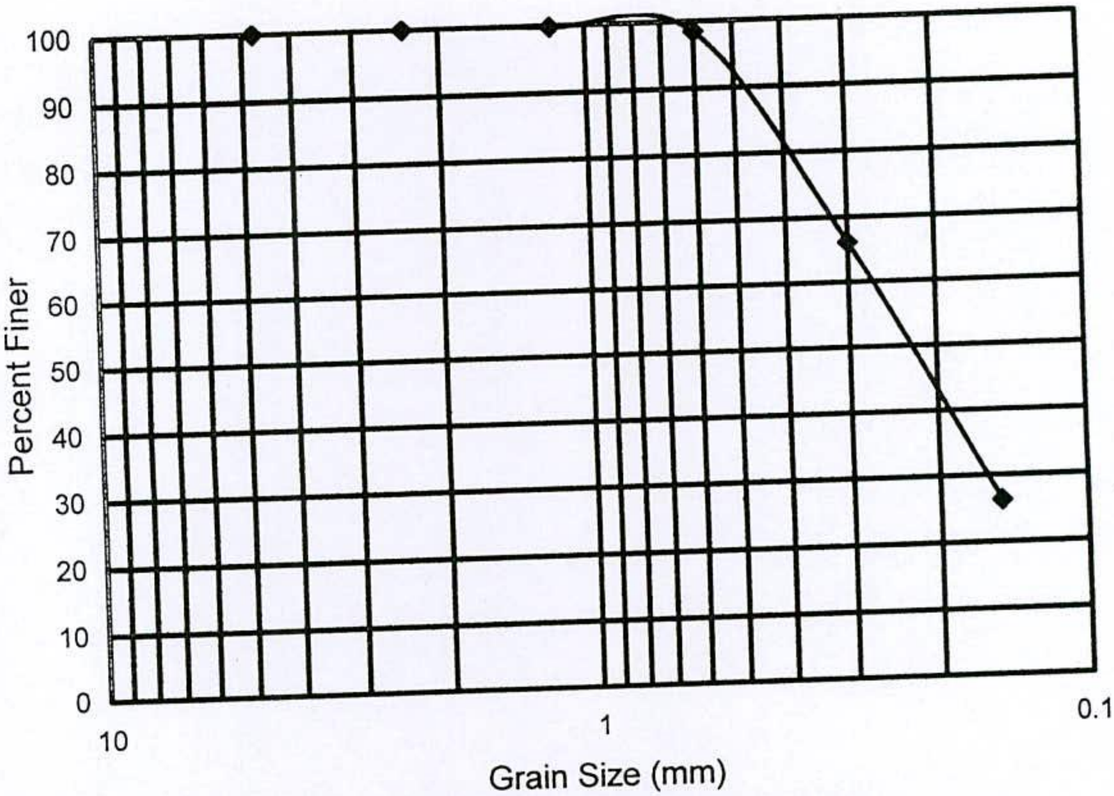

Fig. 4.10 Grain Size Distribution Curve of Sample S10

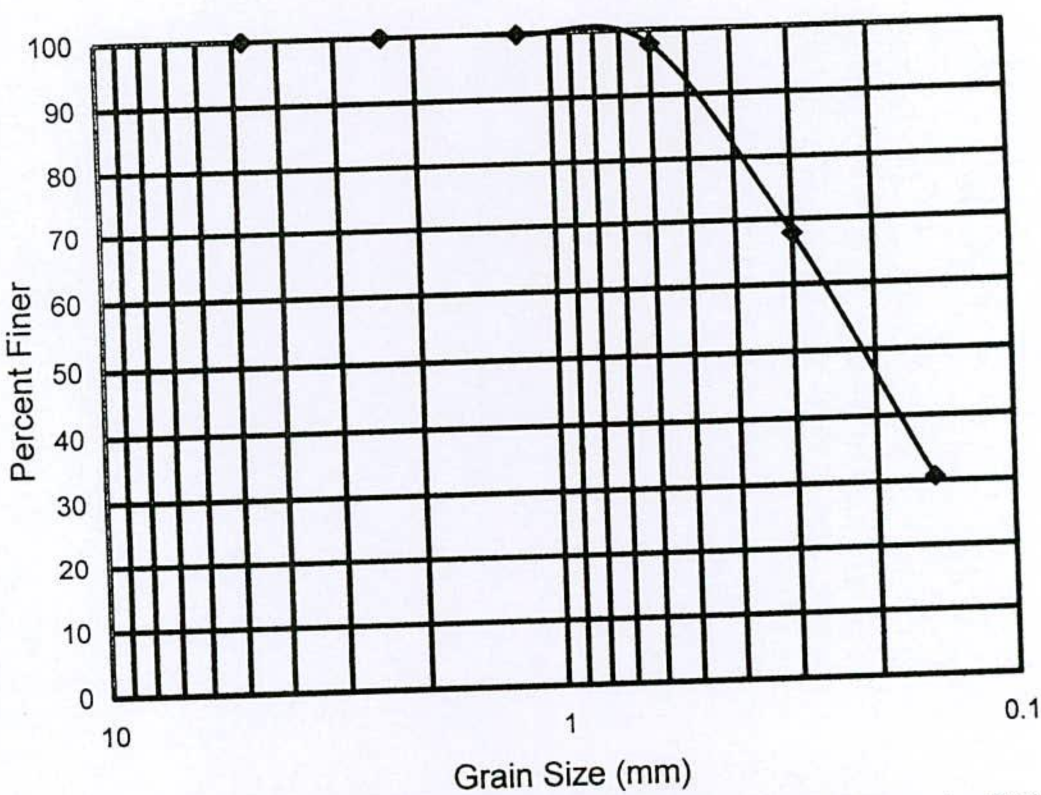

Fig. 4.11 Grain Size Distribution Curve of Sample S11

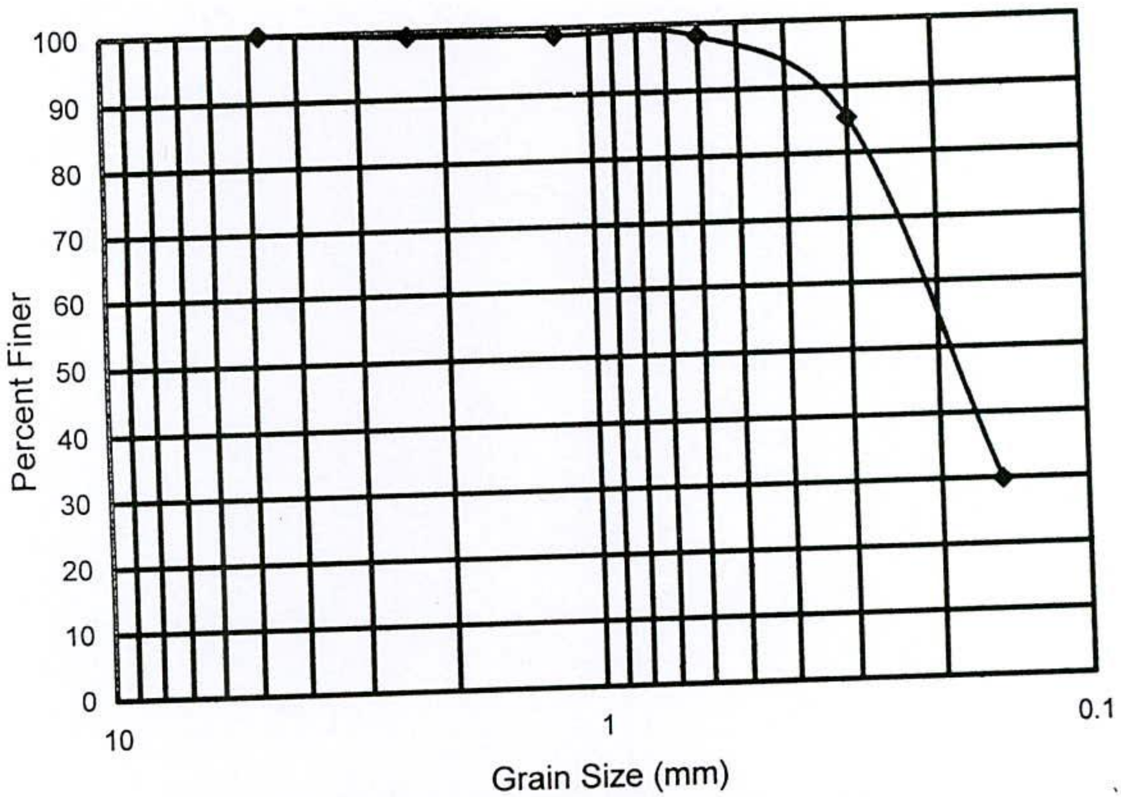

Fig. 4.12 Grain Size Distribution Curve of Sample S12

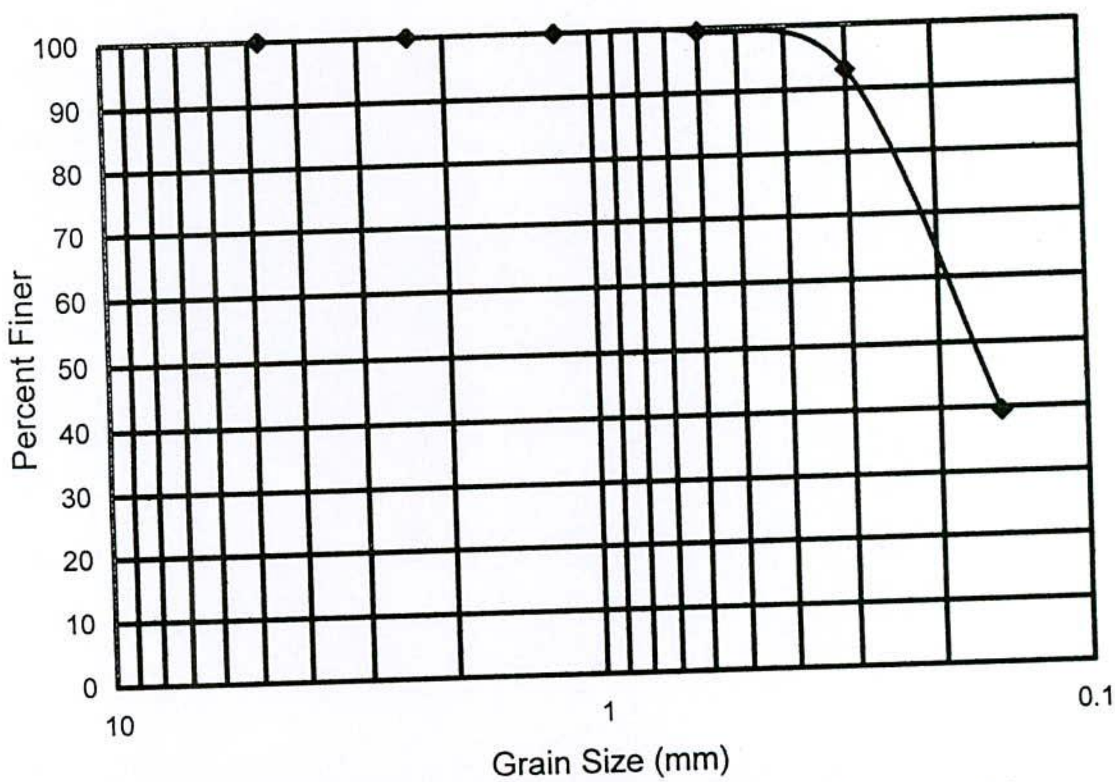

Fig. 4.13 Grain Size Distribution Curve of Sample S13

4.6 Preparation of Sand Bed

Before performing the penetration test and field density test, artificial sand bed was prepared in the open test bed by filling each type of sand with proper compaction of each 15 cm layer. The compaction was done with vibration and hammer manually. The top 15 cm of the test bed was kept empty. Fig. 4.14 shows the compaction in the sand bed.

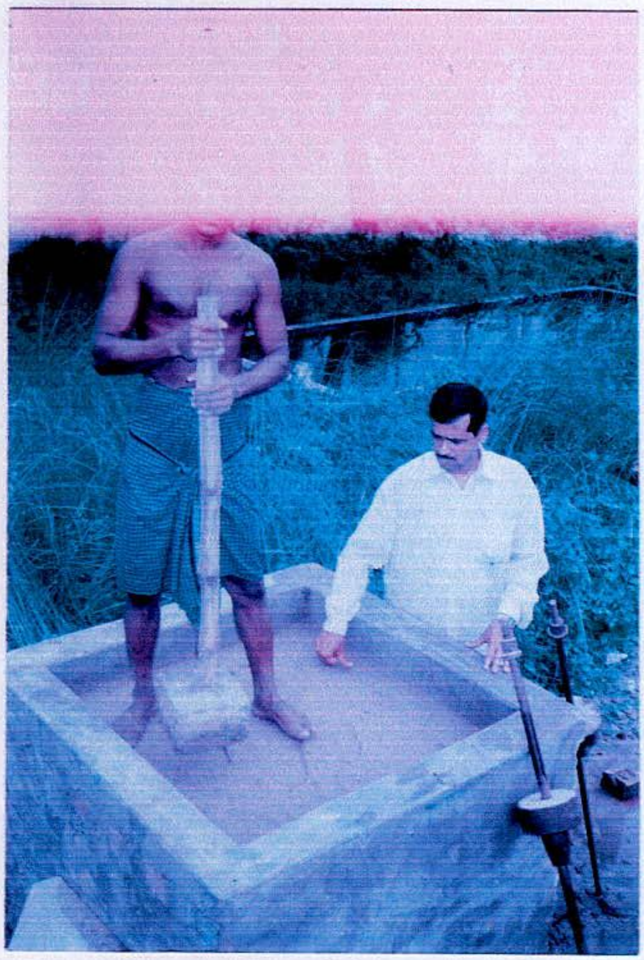

Fig. 4.14 Mechanical Compaction for the Preparation of Sand Bed

4.7 Laboratory and Field Tests

Direct shear test was performed in the laboratory to find out friction angles of all the sand beds and field density test by sand replacement method was performed on each prepared sand bed in artificial test bed to evaluate the bearing capacity of the sand bed. To determine the bearing capacity of each sand bed, Terzaghi's formula was under consideration. The tests were discussed in the following articles:

4.7.1 Direct Shear Test

Direct shear test was performed on samples collected from each sand bed in the artificial test bed to determine the friction angle of that sample. This test was repeated on three samples collected from each sand bed. The difference in three test was so small that it could be neglected. However, average value was under consideration. In direct shear test normal loads were 10 kg, 20 kg and 30 kg.

4.7.2 Determination of Field Density

To find out field density of the each filling sand in the test bed, sand replacement method was conducted for compacted sand in the test bed. In the sand pouring cylinder Ottawa sand was used which was run into the hole from the cylinder by opening the valve till the hole and the cone below the valve is completely filled. Fig. 4.15 shows the operation of pouring Ottawa sand to the hole in sand bed through cylinder and cone to measure the field density by sand replacement method. A typical result sheet was presented in the Table 4.16.

4.8 Hand Penetrometer Test

To investigate the bearing capacity by hand penetrometer, specially fabricated penetrometer was used. The upper rod, lower rod and collar of the hand penetrometer were set on the top of the compacted sand bed. The circular disc of weight 10 kg was slided through the upper rod to rest on the collar. The nut was then adjusted so that the height from the top of the disc to the bottom level of the nut of 37.5 cm. A check nut on the top of the adjustable nut is preferable. This prevent movement of the original nut. The instrument thus set was then held vertically by one person to the test bed.

Another person is then required to lift the disc slowly up to the bottom of the nut and allow it to fall freely from that height to the top of the collar. This procedure was repeated until the rod penetrated 45 cm into the soil. The number of blows required for first 15 cm penetration was ignored to allow for any soil disturbance. The number of blows required for the penetration of the last 30 cm of the rod was taken as the N_h value (hand penetrometer test value) of the particular penetrometer. Fig. 4.16 shows the penetration of hand penetrometer by free falling disc.

Three tests were performed by three different hand penetrometers at different places in each artificial test bed. For each sand bed, three tests were repeated by a particular hand penetrometer.

Fig. 4.15 Operation of Pouring Ottawa Sand to the Hole in the Bed

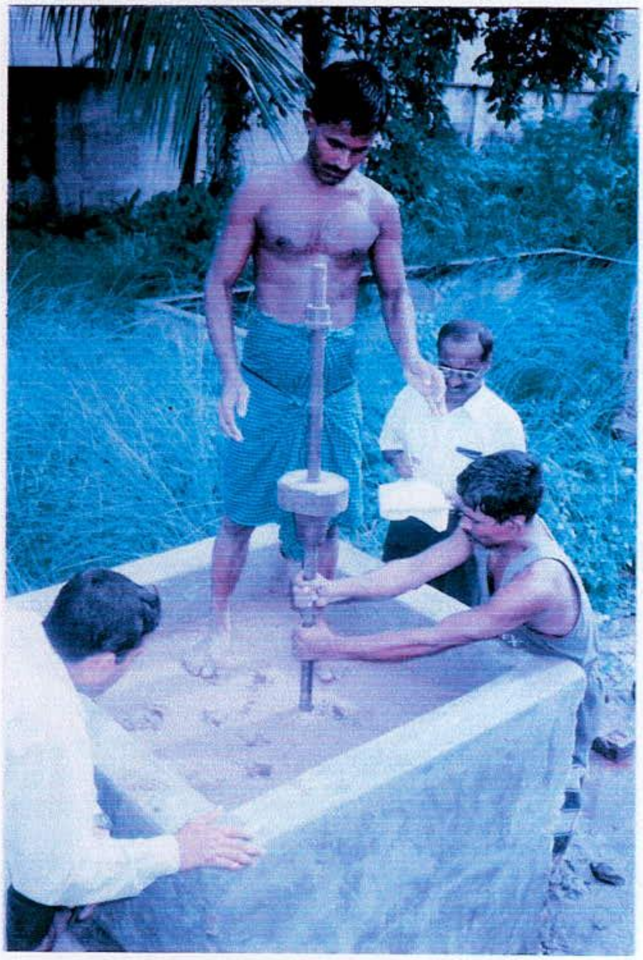

Fig. 4.16 Penetration of Hand Penetrometer by Free Falling Disc

Table 4.16 Determination of Field Density of Granular Soil for Sample S3 (A Typical Calculation Sheet)

Serial No.	Description	Weight
	Volume of Test Hole	
1.	Weight of apparatus filled with sand	5038.20 gms
2.	Weight of apparatus and remaining sand	2164.20 gms
3.	Weight of sand in hole, plate and cone	2874.00 gms
4.	Weight of sand in cone and plate	1620.00 gms
5.	Weight of sand in hole	1254.00 gms
6.	Bulk density of sand	1.48 gms/cc
7.	Volume of test hole	847.30 cc
	Wet Density	
8.	Weight of moist soil from hole plus tare	2037.00 gms
9.	Weight of tare	418.50 gms
10.	Weight of moist soil	1618.50 gms
11.	Wet Density	1.91 gm/cc
	Field Moisture Content and Dry Density	
12.	Weight of wet sample plus tare	141.90 gms
13.	Weight of dry sample plus tare	117.30 gms
14.	Weight of water in sample	24.60 gms
15.	Tare No. and weight	22.50 gms
16.	Weight of dry soil	94.80 gms
17.	Moisture content (item 14 / item 16) x 100	25.95 %
18	Dry Density (item 11) / (1.0 plus item 17)	1.52 gm/cc

The details calculation to find out field density of all the samples are mentioned in Appendix-I.

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1 General

The results in the laboratory investigations of friction angles and hand penetrometer test values on granular soil (especially sand) are represented at this chapter. This chapter deals mainly with the presentation and discussion on the relationships of penetrometer test value versus bearing capacity in terms of friction angle, and penetrometer test value versus friction angle.

5.2 Determination of Field Density

To find out field density of the each filling sand in the test bed, sand replacement method was conducted for compacted sand in the pit as discussed in the Article 4.6.2. Table 5.1 shows the field densities for thirteen samples.

Table 5.1 Determination of Field Densities of Sand Samples Compacted in the Test Bed

Sample Designation	Wet Density in Field, γ (gm/cc)	Dry Density in Field, γ_d (gm/cc)
S1	1.93	1.76
S2	1.80	1.68
S3	1.91	1.52
S4	1.87	1.69
S5	1.76	1.62
S6	1.83	1.66
S7	1.87	1.63
S8	1.96	1.70
S9	1.78	1.62
S10	1.94	1.67
S11	1.64	1.51
S12	1.79	1.57
S13	1.64	1.40

5.3 Hand Penetrometer Test Values

Tables 5.2 to 5.14 show the hand penetrometer test value (N_h) for sample S1 to S13 respectively. Tables 5.2 to 5.14 also show the hand penetrometer test value (N_h) for three hand penetrometers of different sizes for thirteen sand samples (S1 to S13) of granular soils from the test bed.

Table 5.2 Hand Penetrometer Test Values for Sample S1

Dia. of Hand Penetro Meter	1st test				2nd test				3rd test				N_h
	1st layer	2nd layer	3rd layer	2nd+ 3rd layer	1st layer	2nd layer	3rd layer	2nd+ 3rd layer	1st layer	2nd layer	3rd layer	2nd+ 3rd layer	Av.of Three Tests
	15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		
31.25 mm	6	21	31	52	6	20	33	53	7	21	35	56	54
25.00 mm	5	13	23	36	4	13	16	29	5	13	21	34	33
18.75 mm	3	10	12	22	3	10	11	21	3	10	12	22	22

Table 5.3 Hand Penetrometer Test Values for Sample S2

Dia. of Hand penetro meter	1st test				2nd test				3rd test				N_h
	1st layer	2nd layer	3rd layer	2nd + 3rd layer	1st layer	2nd layer	3rd layer	2nd + 3rd layer	1st layer	2nd layer	3rd layer	2nd+ 3rd layer	Av. of three tests
	15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		
31.25 mm	10	18	27	45	9	18	26	44	8	17	28	45	45
25.00 mm	6	10	13	23	6	10	1	26	5	11	15	25	25
18.75 mm	4	7	9	16	3	7	8	15	4	6	10	16	16

Table 5.4 Hand Penetrometer Test Values for Sample S3

Dia. of Hand penetrometer	1st test				2nd test				3rd test				N_h
	1st layer 15 cm	2nd layer 15 cm	3rd layer 15 cm	2nd+ 3rd layer	1st layer 15 cm	2nd layer 15 cm	3rd layer 15 cm	2nd+ 3rd layer	1st layer 15 cm	2nd layer 15 cm	3rd layer 15 cm	2nd+ 3rd layer	Av.of three tests
31.25 mm	10	17	28	45	9	18	26	44	8	17	28	45	45
25.00 mm	6	10	13	23	6	10	16	26	5	11	14	25	25
18.75 mm	4	7	9	16	3	7	8	15	4	6	10	16	16

Table 5.5 Hand Penetrometer Test Values for Sample S4

Dia. of Hand Penetrometer	1st test				2nd test				3rd test				N_h
	1st layer 15 cm	2nd layer 15 cm	3rd layer 15 cm	2nd+ 3rd layer	1st layer 15 cm	2nd layer 15 cm	3rd layer 15 cm	2nd+ 3rd layer	1st layer 15 cm	2nd layer 15 cm	3rd layer 15 cm	2nd+ 3rd layer	Av.of three tests
31.25 mm	6	14	28	42	5	19	36	55	6	17	29	56	44
25.00 mm	3	10	16	26	3	9	19	28	4	10	18	28	27
18.75 mm	2	8	13	21	2	8	13	21	3	8	12	20	21

Table 5.6 Hand Penetrometer Test Values for Sample S5

Dia. of Hand Penetrometer	1st test				2nd test				3rd test				N_h
	1st layer 15 cm	2nd layer 15 cm	3rd layer 15 cm	2nd+ 3rd layer	1st layer 15 cm	2nd layer 15 cm	3rd layer 15 cm	2nd+ 3rd layer	1st layer 15 cm	2nd layer 15 cm	3rd layer 15 cm	2nd+ 3rd layer	Av.of three tests
31.25 mm	8	18	25	43	8	17	27	44	6	15	22	37	41
25.00 mm	4	11	13	24	5	9	13	22	5	10	14	24	23
18.75 mm	2	9	9	18	3	8	9	17	3	7	8	15	17

Table 5.7 Hand Penetrometer Test Values for Sample S6

Dia. of Hand Penetrometer	1^{st} test				2nd test				3rd test				N_h
	1^{st} layer	2^{nd} layer	3^{rd} layer	2^{nd}+ 3rd layer	1^{st} layer	2^{nd} layer	3^{rd} layer	2^{nd}+ 3rd layer	1^{st} layer	2^{nd} layer	3^{rd} layer	2^{nd}+ 3rd layer	Av. of three tests
	15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		
31.25 mm	6	17	23	40	6	17	21	38	8	17	24	41	40
25.00 mm	4	10	13	23	4	9	14	23	5	10	13	23	23
18.75 mm	3	7	10	17	2	6	10	16	2	7	9	16	16

Table 5.8 Hand Penetrometer Test Values for Sample S7

Dia. of Hand Penetrometer	1^{st} test				2nd test				3rd test				N_h
	1^{st} layer	2^{nd} layer	3^{rd} layer	2^{nd}+ 3rd layer	1^{st} layer	2^{nd} layer	3^{rd} layer	2^{nd}+ 3rd layer	1^{st} layer	2^{nd} layer	3^{rd} layer	2^{nd}+ 3rd layer	Av. of three tests
	15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		
31.25 mm	9	20	26	46	7	15	20	35	8	19	20	39	40
25.00 mm	5	13	14	27	5	11	13	24	5	12	16	28	26
18.75 mm	3	8	8	16	3	7	6	13	3	6	7	13	14

Table 5.9 Hand Penetrometer Test Values for Sample S8

Dia. of Hand Penetrometer	1^{st} test				2nd test				3rd test				N_h
	1^{st} layer	2^{nd} layer	3^{rd} layer	2^{nd}+ 3rd layer	1^{st} layer	2^{nd} layer	3^{rd} layer	2^{nd}+ 3rd layer	1^{st} layer	2^{nd} layer	3^{rd} layer	2^{nd}+ 3rd layer	Av. of three tests
	15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		
31.25 mm	6	16	18	34	6	17	27	44	6	15	20	35	38
25.00 mm	5	10	15	25	3	8	12	20	4	9	12	21	22
18.75 mm	3	7	8	15	2	6	8	14	2	7	7	14	14

Table 5.10 Hand Penetrometer Test Values for Sample S9

Dia. of Hand Penetrometer	1^{st} test				2nd test				3rd test				N_h
	1^{st} layer 15 cm	2^{nd} layer 15 cm	3^{rd} layer 15 cm	2^{nd}+ 3rd layer	1^{st} layer 15 cm	2^{nd} layer 15 cm	3^{rd} layer 15 cm	2^{nd}+ 3rd layer	1^{st} layer 15 cm	2^{nd} layer 15 cm	3^{rd} layer 15 cm	2^{nd}+ 3rd layer	Av. of three tests
31.25 mm	7	13	26	39	6	12	25	37	6	10	22	32	36
25.00 mm	4	7	11	18	5	8	16	24	4	7	14	21	21
18.75 mm	3	6	10	16	3	6	9	15	3	5	9	14	15

Table 5.11 Hand Penetrometer Test Values for Sample S10

Dia. of Hand Penetrometer	1^{st} test				2nd test				3rd test				N_h
	1^{st} layer 15 cm	2^{nd} layer 15 cm	3^{rd} layer 15 cm	2^{nd}+ 3rd layer	1^{st} layer 15 cm	2^{nd} layer 15 cm	3^{rd} layer 15 cm	2^{nd}+ 3rd layer	1^{st} layer 15 cm	2^{nd} layer 15 cm	3^{rd} layer 15 cm	2^{nd}+ 3rd layer	Av. of three tests
31.25 mm	6	14	21	35	5	13	16	29	5	11	18	29	31
25.00 mm	4	8	12	20	4	8	12	20	3	7	11	18	19
18.75 mm	3	5	7	12	3	6	7	13	2	6	7	13	13

Table 5.12 Hand Penetrometer Test Values for Sample S11

Dia. of Hand Penetrometer	1^{st} test				2nd test				3rd test				N_h
	1^{st} layer 15 cm	2^{nd} layer 15 cm	3^{rd} layer 15 cm	2^{nd}+ 3rd layer	1^{st} layer 15 cm	2^{nd} layer 15 cm	3^{rd} layer 15 cm	2^{nd}+ 3rd layer	1^{st} layer 15 cm	2^{nd} layer 15 cm	3^{rd} layer 15 cm	2^{nd}+ 3rd layer	Av. of three tests
31.25 mm	6	15	17	32	5	14	17	31	6	13	16	29	31
25.00 mm	4	10	11	21	4	9	10	19	3	9	12	21	20
18.75 mm	2	6	7	13	2	5	6	11	2	4	5	9	11

Table 5.13 Hand Penetrometer Test Values for Sample S12

Dia. of Hand Penetrometer	1st test				2nd test				3rd test				N_h
	1st layer	2nd layer	3rd layer	2nd+ 3rd layer	1st layer	2nd layer	3rd layer	2nd+ 3rd layer	1st layer	2nd layer	3rd layer	2nd+ 3rd layer	Av. of three tests
	15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		
31.25 mm	5	12	15	27	5	12	15	27	5	12	15	27	27
25.00 mm	4	9	10	19	4	9	9	18	4	9	9	18	18
18.75 mm	3	4	6	10	3	5	5	10	3	4	6	10	10

Table 5.14 Hand Penetrometer Test Values for Sample S13

Dia. of Hand Penetrometer	1st test				2nd test				3rd test				N_h
	1st layer	2nd layer	3rd layer	2nd+ 3rd layer	1st layer	2nd layer	3rd layer	2nd+ 3rd layer	1st layer	2nd layer	3rd layer	2nd+ 3rd layer	Av. of three tests
	15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		15 cm	15 cm	15 cm		
31.25 mm	4	9	11	20	4	9	11	20	3	9	12	21	20
25.00 mm	3	6	7	13	2	6	7	13	3	7	10	17	14
18.75 mm	1	4	5	9	2	4	5	9	2	4	5	9	9

5.4 Determination of Friction Angles of Granular Soils

Figs. 5.1 to 5.13 show the normal stress versus shear stress curves for the samples of thirteen sand beds. From these curves friction angles of all the sand beds were determined. The values of friction angles with normal stresses and shear stresses are also shown in Tables 5.15 and 5.16 which show the results of direct shear test on samples S1 to S8 and S9 to S13 respectively.

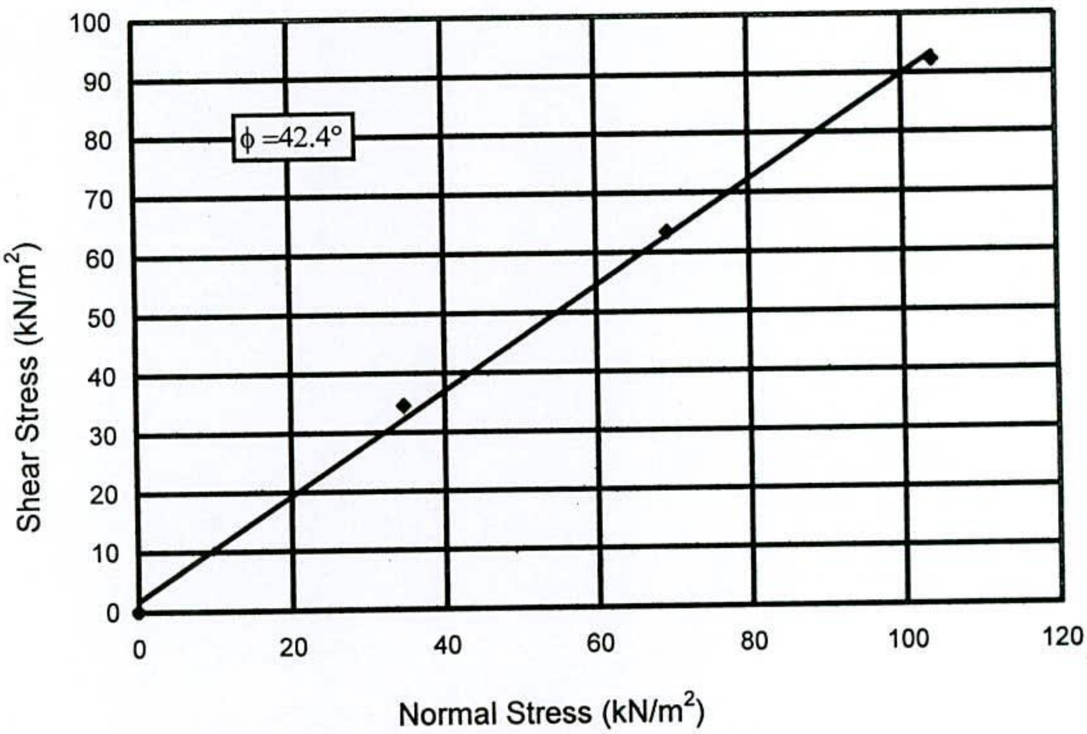

Fig. 5.1 Shear Stress Versus Normal Stress Curve of Sample S1

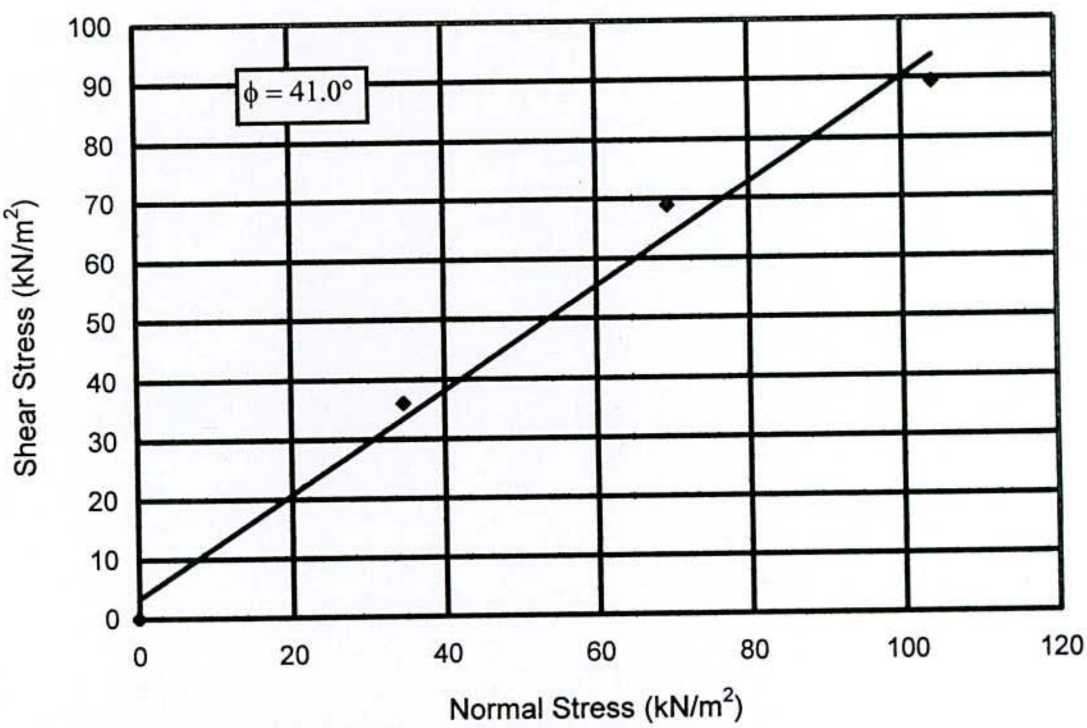

Fig. 5.2 Shear Stress Versus Normal Stress Curve of Sample S2

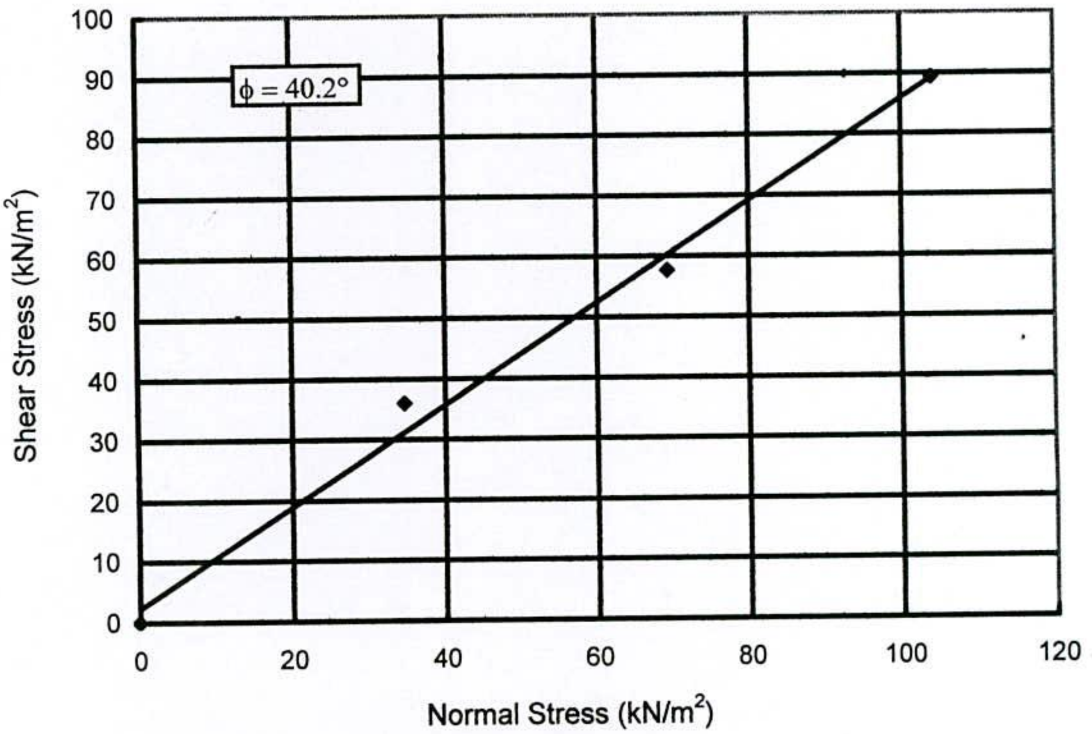

Fig. 5.3 Shear Stress versus Normal Stress Curve of Sample S3

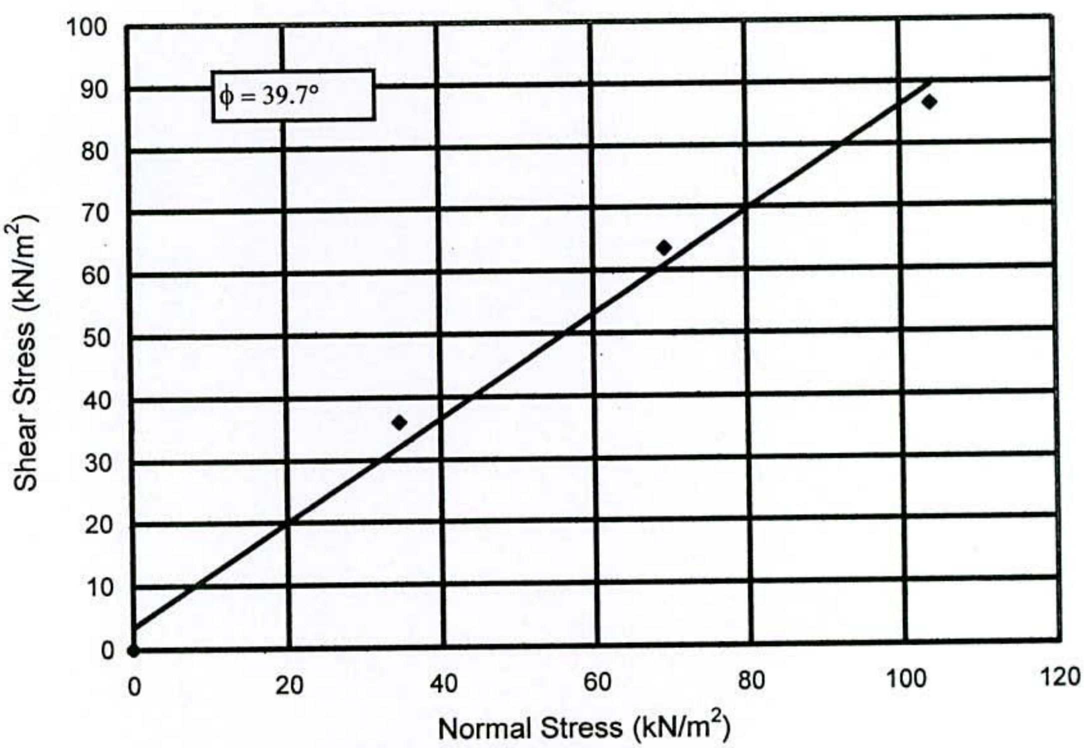

Fig. 5.4 Shear Stress Versus Normal Stress of sample S4

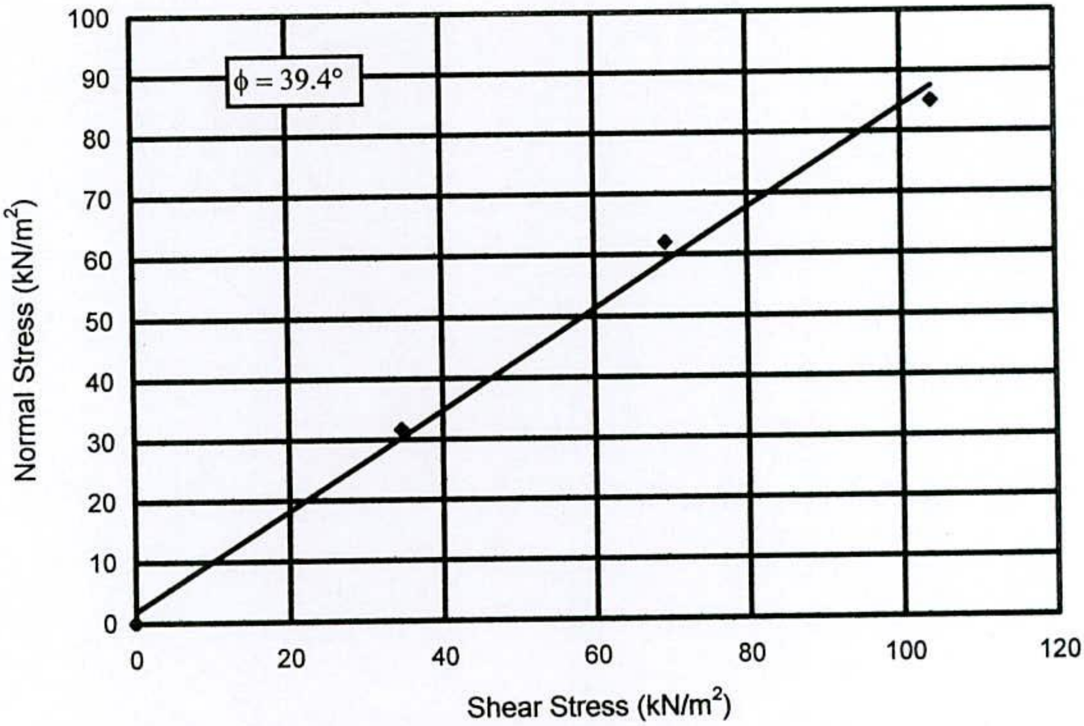

Fig. 5.5 Shear Stress Versus Normal Stress Curve of Sample S5

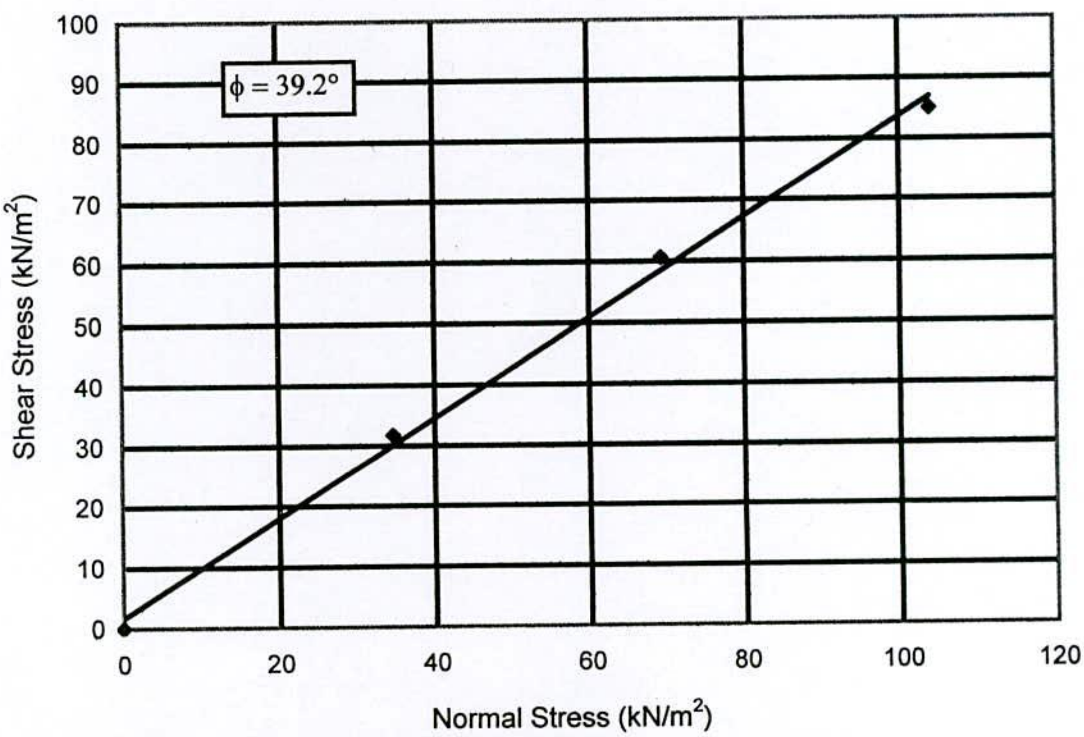

Fig. 5.6 Shear Stress Versus Normal Stress Curve of Sample S6

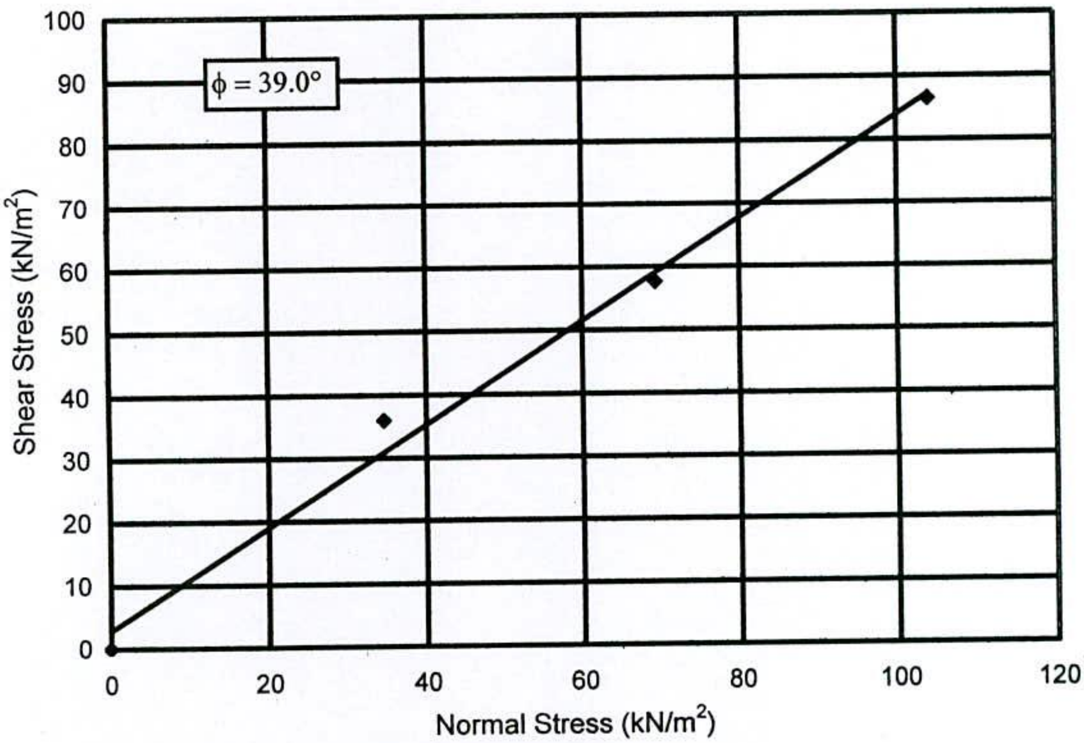

Fig. 5.7 Shear Stress Versus Normal Stress Curve of Sample S7

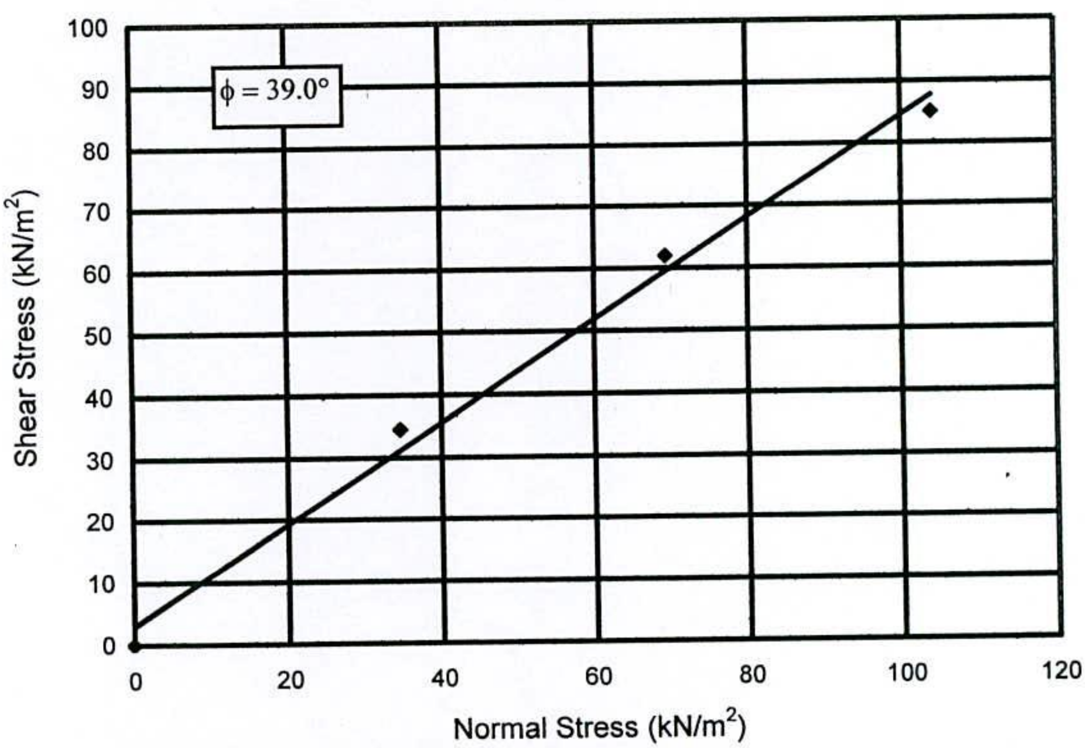

Fig. 5.8 Shear Stress Versus Normal Stress Curve of Sample S8

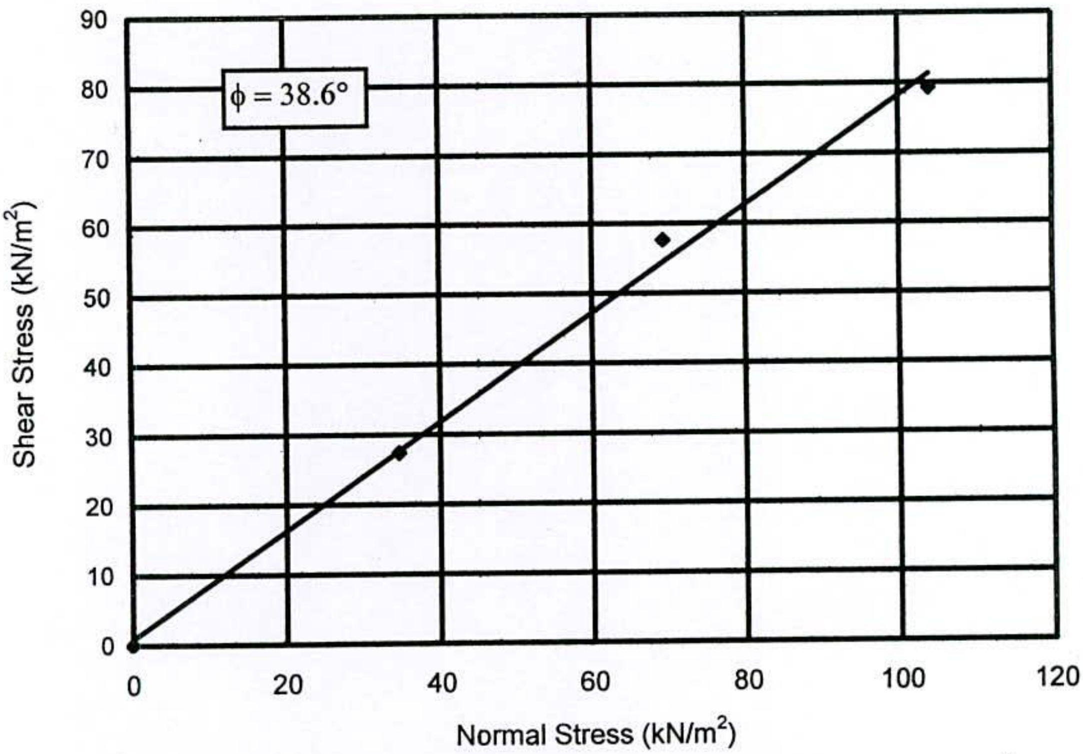

Fig. 5.9 Shear Stress Versus Normal Stress Curve of Sample S9

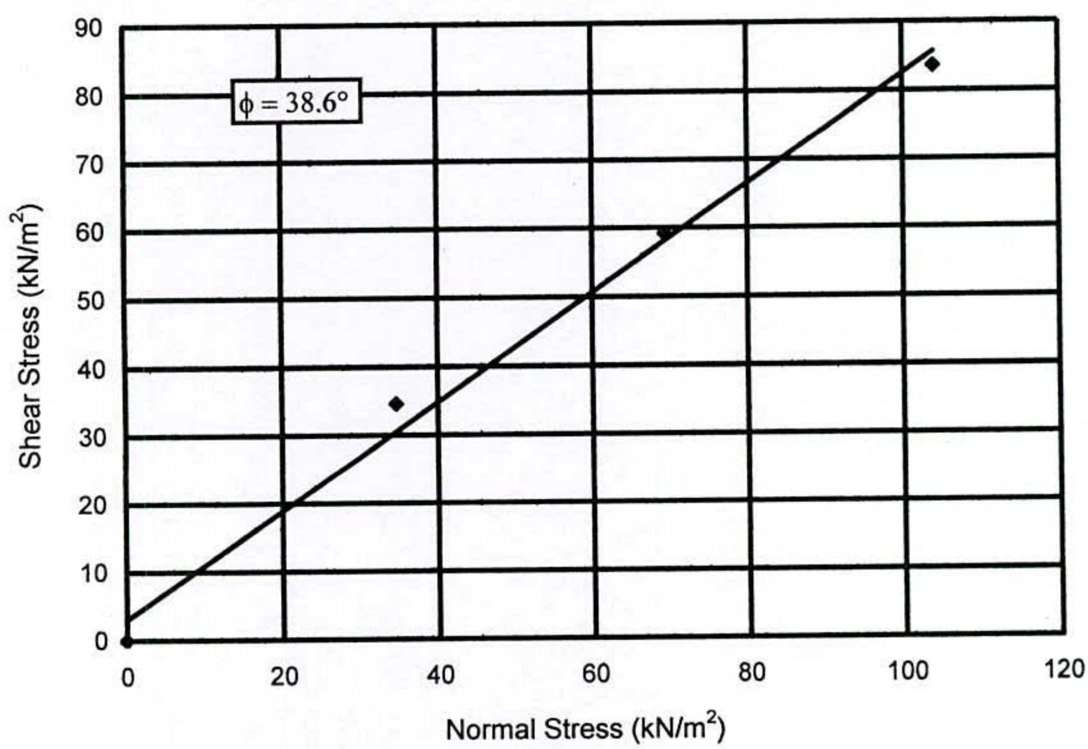

Fig. 5.10 Normal Stress Versus Shear Stress Curve of Sample S10

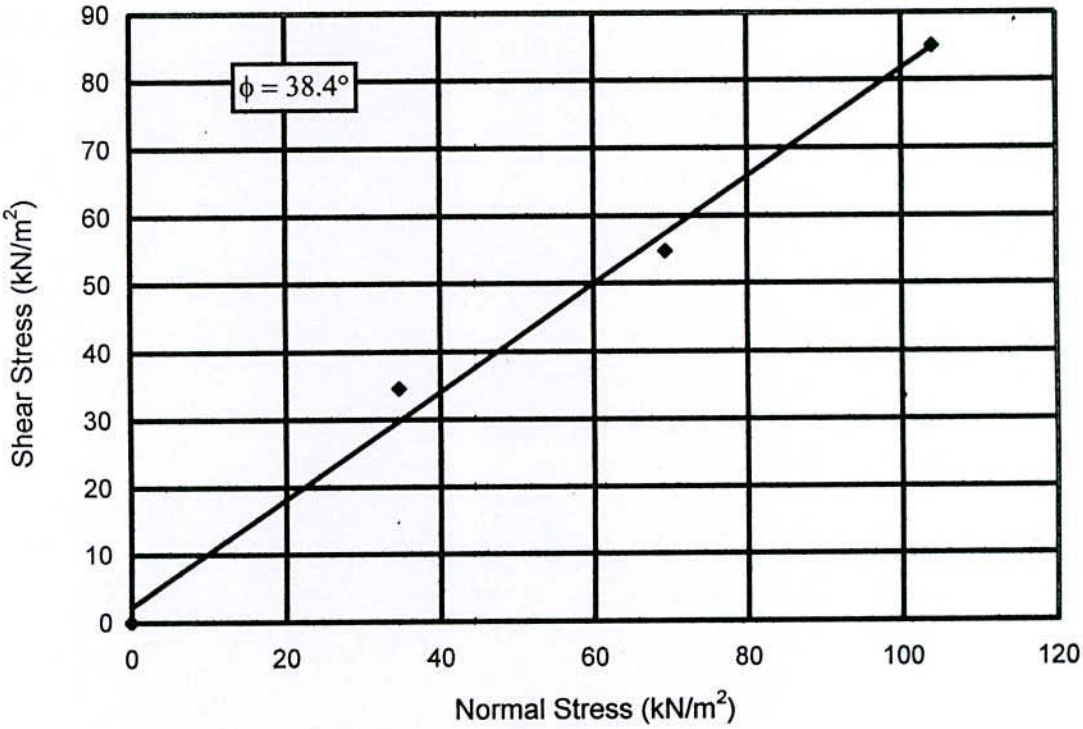

Fig.5.11 Shear Stress Versus Normal Stress Curve of Sample S11

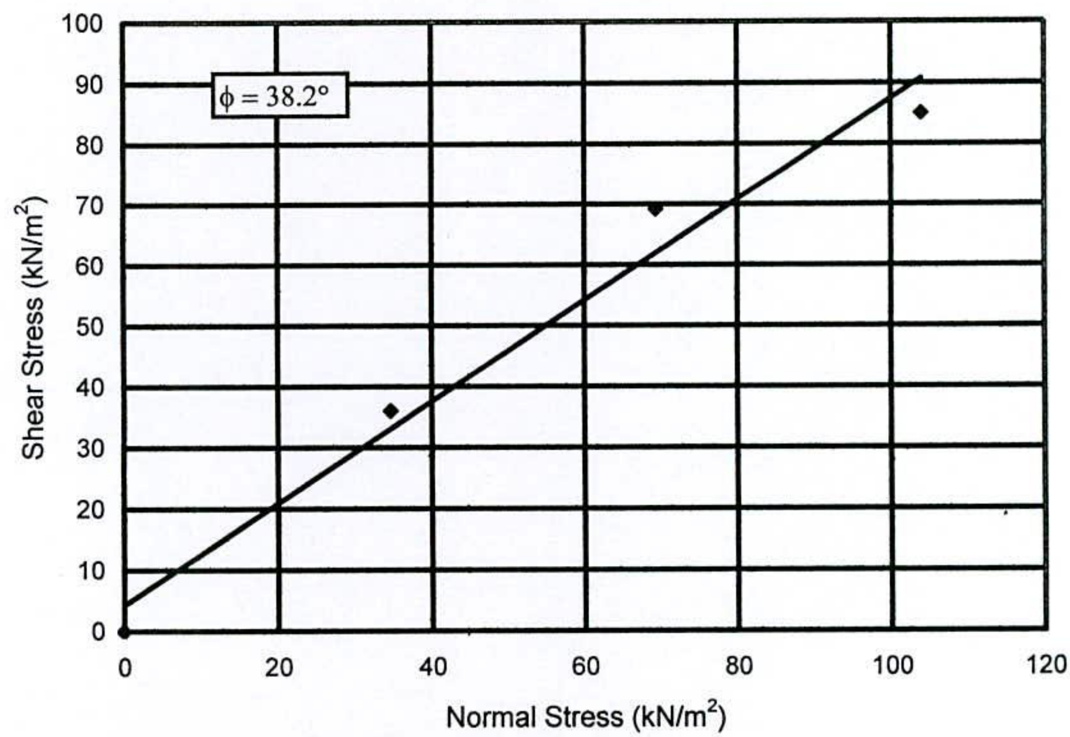

Fig. 5.12 Shear Stress Versus Normal Stress Curve of Sample S12

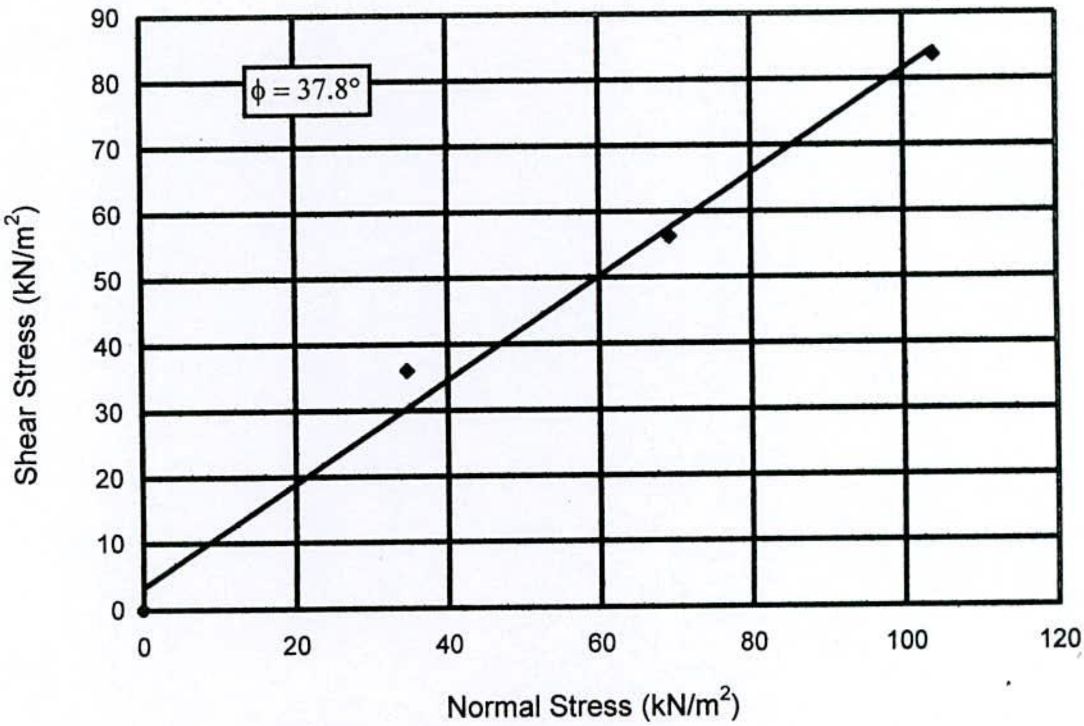

Fig. 5.13 Shear Stress Versus Normal Stress Curve of Sample S13

Table 5.15 Determination of Friction Angle, ϕ of Samples S1 to S8

Sample No.	Normal Stress, σ (kN/m^2)	Shear Stress, τ (kN/m^2)	Failure Strain, ε_f (%)	Value of ϕ in degree
S1	34.66	34.59	8	42.4°
	69.32	63.43	10	
	104.00	92.26	12	
S2	34.66	36.04	10	41.0°
	69.32	68.88	14	
	104.00	89.40	14	
S3	34.66	36.04	10	40.2°
	69.32	57.67	10	
	104.00	89.40	10	
S4	34.66	36.04	12	39.7°
	69.32	63.43	8	
	104.00	86.50	12	
S5	34.66	31.73	16	39.4°
	69.32	61.98	16	
	104.00	85.05	16	
S6	34.66	31.77	10	39.2°
	69.32	60.57	12	
	104.00	85.05	16	
S7	34.66	36.04	14	39.0°
	69.32	57.67	14	
	104.00	86.50	14	
S8	34.66	34.59	10	39.0°
	69.32	61.98	12	
	104.00	85.08	14	

Table 5.16 Determination of Friction Angle, ϕ of Samples S9 to S13

Sample No.	Normal Stress, σ (kN/m^2)	Shear Stress, τ (kN/m^2)	Failure Strain, ε_f (%)	Value of ϕ in degree
S9	34.66	27.38	12	38.6°
	69.32	57.67	18	
	104.00	79.29	20	
S10	34.66	34.59	14	38.6°
	69.32	59.12	12	
	104.00	83.60	16	
S11	34.66	34.59	14	38.4°
	69.32	54.77	14	
	104.00	85.06	16	
S12	34.66	36.04	18	38.2°
	69.32	69.19	16	
	104.00	85.05	16	
S13	34.66	36.04	8	37.8°
	69.32	56.22	8	
	104.00	83.60	12	

5.5 Relationship Between Hand Penetrometer Test Value and Friction Angle of Granular soils

No. of penetration from penetrometer test and friction angle from direct shear test were determined on each sample. Figs. 5.14 to 5.16 show the variation of friction angle, ϕ with the hand penetrometer test value, N_h for three different penetrometers. Table 5.17 also shows the values of N_h for three penetrometers of diameters 18.75 mm, 25 mm and 31.25 mm with the variation of friction angles. From Figs. 5.14 to 5.16 it is observed that friction angle increases with the increase of penetrometer test value in all penetration tests. Three constitutive equations have been established from these figures with coefficient of correlations from 0.86 to 0.91. Table 5.18 shows the constitutive equations for three penetrometers.

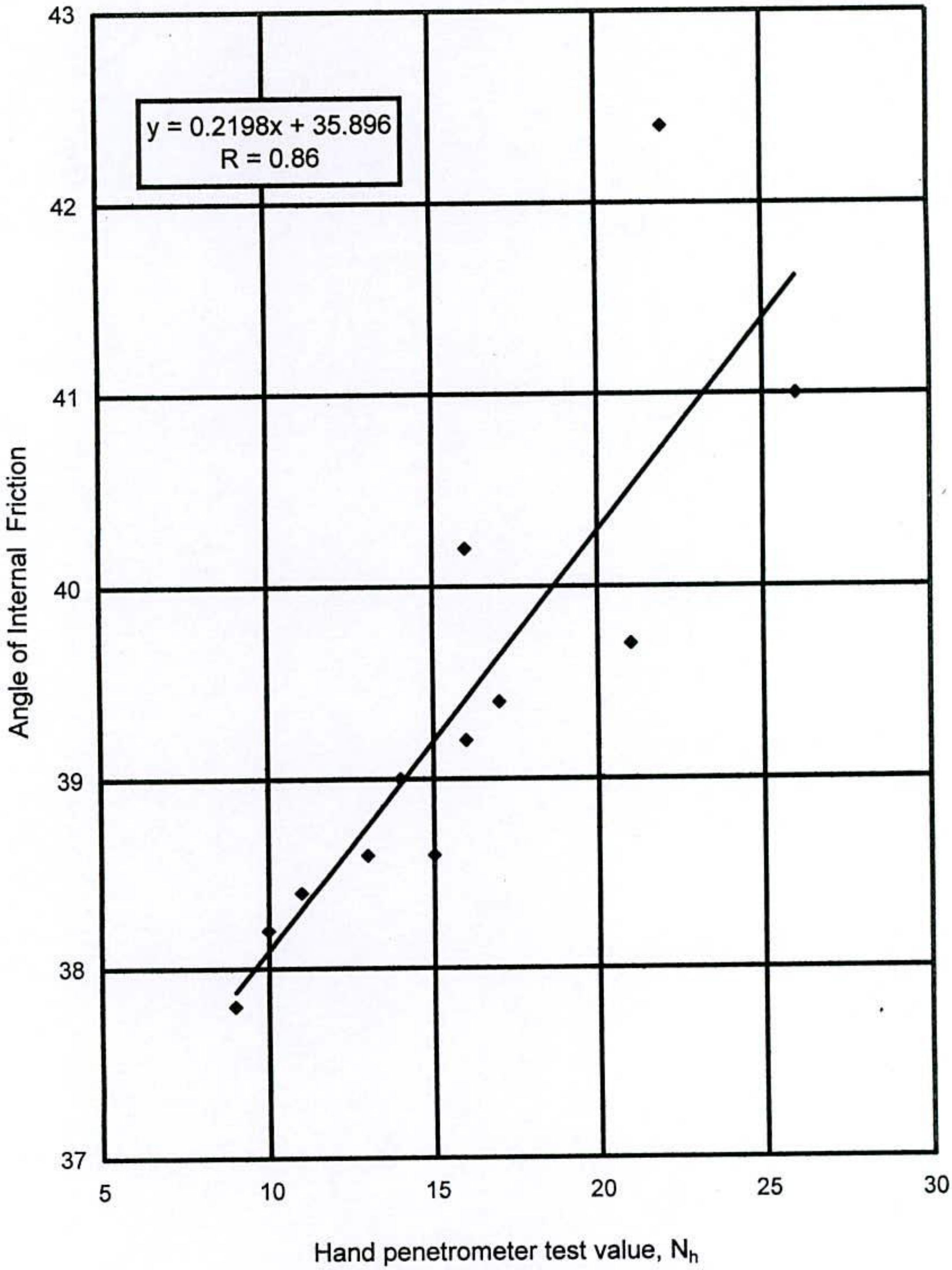

Fig.5.14 Relation Between Hand Penetrometer Test Value and Angle of Internal Friction for Granular Soil (Sand) (For 18.75mm Dia Penetrometer)

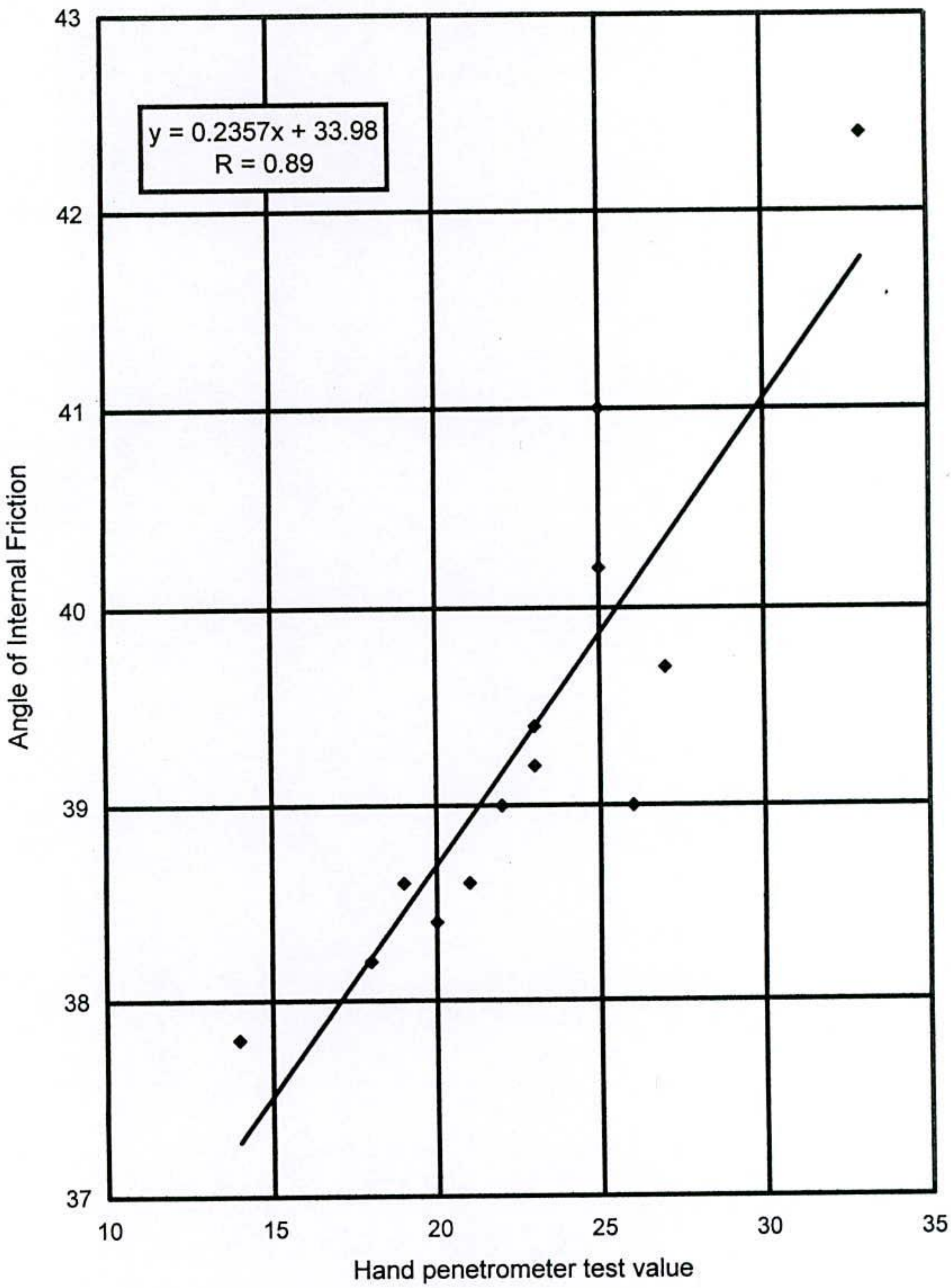

Fig.5.15 Relationship Between Hand Penetrometer test Value and Angle of Internal Friction for Granular Soil (Sand) (For 25 mm Dia Penetrometer)

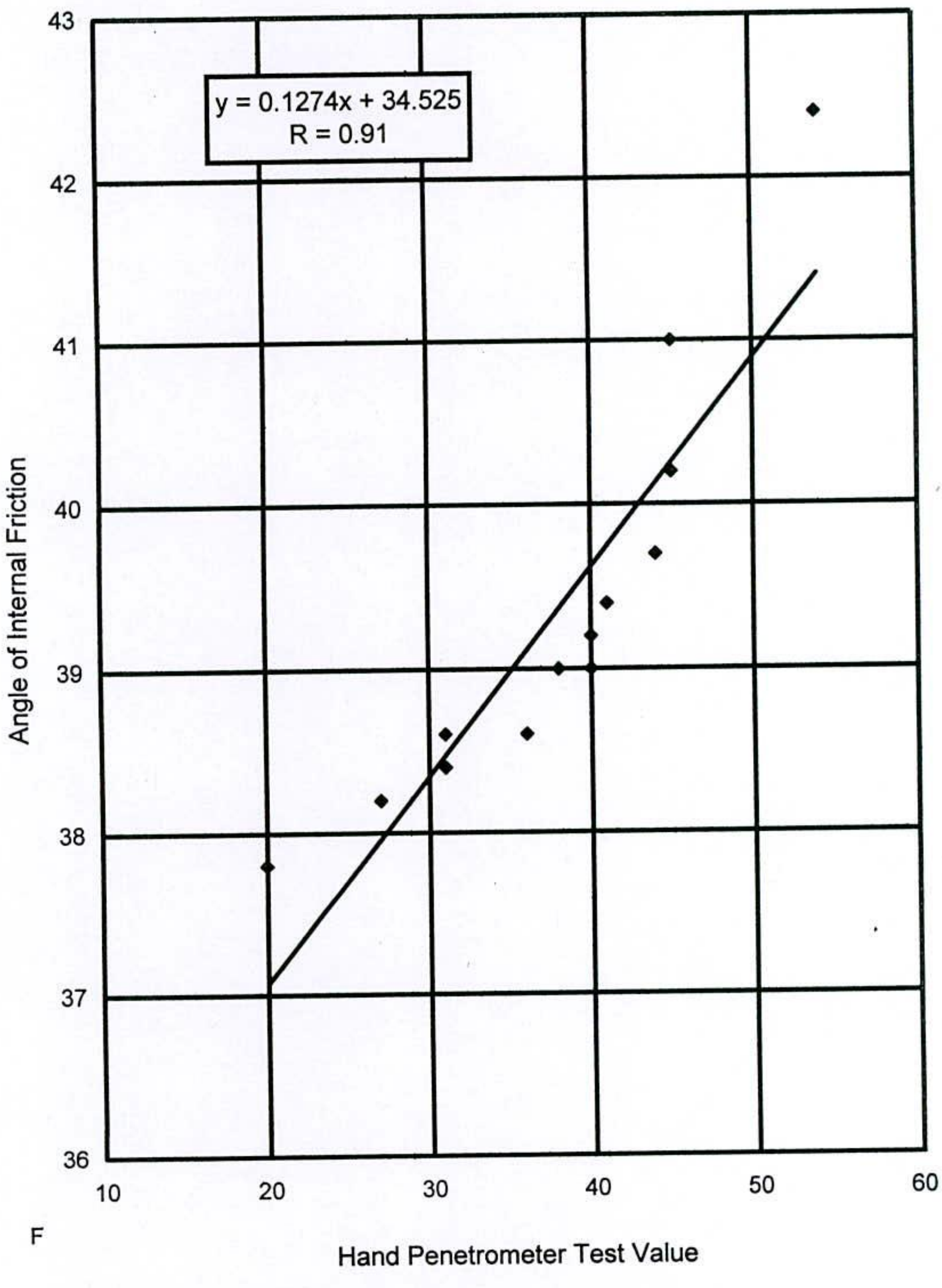

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Fig.5.16 Relationship between Hand Penetrometer Test Value and Angle of Internal Friction for Granular Soil (Sand) (For 31.25 mm Dia. Penetrometer)

Table 5.17 Comparison of Penetrometer Test Values of Three Sizes Penetrometers With the Variation of Friction Angles, ϕ.

Sample No.	F.M.	Friction Angle, ϕ	N_h for 18.75 mm penetrometer	N_h for 25 mm penetrometer	N_h for 31.25 mm penetrometer
S1	2.28	42.4°	22	33	54
S2	2.05	41.0°	16	25	45
S3	1.95	40.2°	16	25	45
S4	1.90	39.7°	21	27	44
S5	1.64	39.4°	17	23	41
S6	1.63	39.2°	16	23	40
S7	1.46	39.0°	14	26	40
S8	1.42	39.0°	14	22	38
S9	1.24	38.6°	15	21	36
S10	1.09	38.6°	13	19	31
S11	1.04	38.4°	11	20	31
S12	0.89	38.2°	10	18	27
S13	0.68	37.8°	9	14	20

Table 5.18 Constitutive Equations for Three Hand Penetrometers

Diameter of Penetrometer (mm)	Constitutive Equations	Coefficient of Correlations, R
18.75	$y = 0.2198x + 35.896$	0.86
25.00	$y = 0.2357x + 33.980$	0.89
31.25	$y = 0.1274x + 34.525$	0.91

where, y = friction angle or angle of internal friction
x = hand penetrometer test value

From the values of coefficient of correlation, it can observed that the equations as mentioned in Table 5.18, might be used to predict friction angle from hand penetrometer test value.

5.6. Correlation Between Hand Penetrometer Test Value and Bearing Capacity of Sand Bed

No. of penetration from hand penetrometer test and bearing capacity from Terzaghi's equation were determined on each sample. Figs. 5.17 to 5.19 show the variation of bearing capacity with the penetrometer test value for three types of penetrometers. This variations are also shown in Table 5.19. It is observed from the Figs. 5.17 to 5.19 that bearing capacity of sand increases with the increase of penetrometer test value in all penetration tests. From these figures three equations have been developed with coefficient of correlations from 0.79 to 0.88 which are shown in Table 5.20.

Table 5.19 Comparison of Penetrometer Test Values of Three Sizes Penetrometer With the Variation of Bearing Capacity (q_{ult}).

Sample No.	F.M.	Ultimate bearing capacity, q_{ult}	N_h for 18.75 mm penetrometer	N_h for 25 mm penetrometer	N_h for 31.25 mm penetrometer
S1	2.28	4617	22	33	54
S2	2.05	3352	16	25	45
S3	1.95	2488	16	25	45
S4	1.90	2532	21	27	44
S5	1.64	2349	17	23	41
S6	1.63	2353	16	23	40
S7	1.46	2258	14	26	40
S8	1.42	2354	14	22	38
S9	1.24	2139	15	21	36
S10	1.09	2205	13	19	31
S11	1.04	1944	11	20	31
S12	0.89	1971	10	18	27
S13	0.68	1114	9	14	20

Table 5.20 Constitutive Equations for Three Hand Penetrometers

Diameter of Penetrometer (mm)	Constitutive Equations	Coefficient of Correlations, R
18.75	$y = 168.65x - 80.233$	0.79 .
25.00	$y = 152.43x - 1034$	0.88
31.25	$y = 79.724x - 580.63$	0.87

where, y = ultimate bearing capacity (kPa)
x = hand penetrometer test value

From Table 5.20 it is observed that the coefficient of correlation of medium and larger penetrometer is much higher than smaller penetrometer of diameter 18.75 mm. So, medium and larger penetrometers of diameters 25 mm and 31.25 mm have to provide more accurate value of ultimate bearing capacity from knowing penetrometer test value.

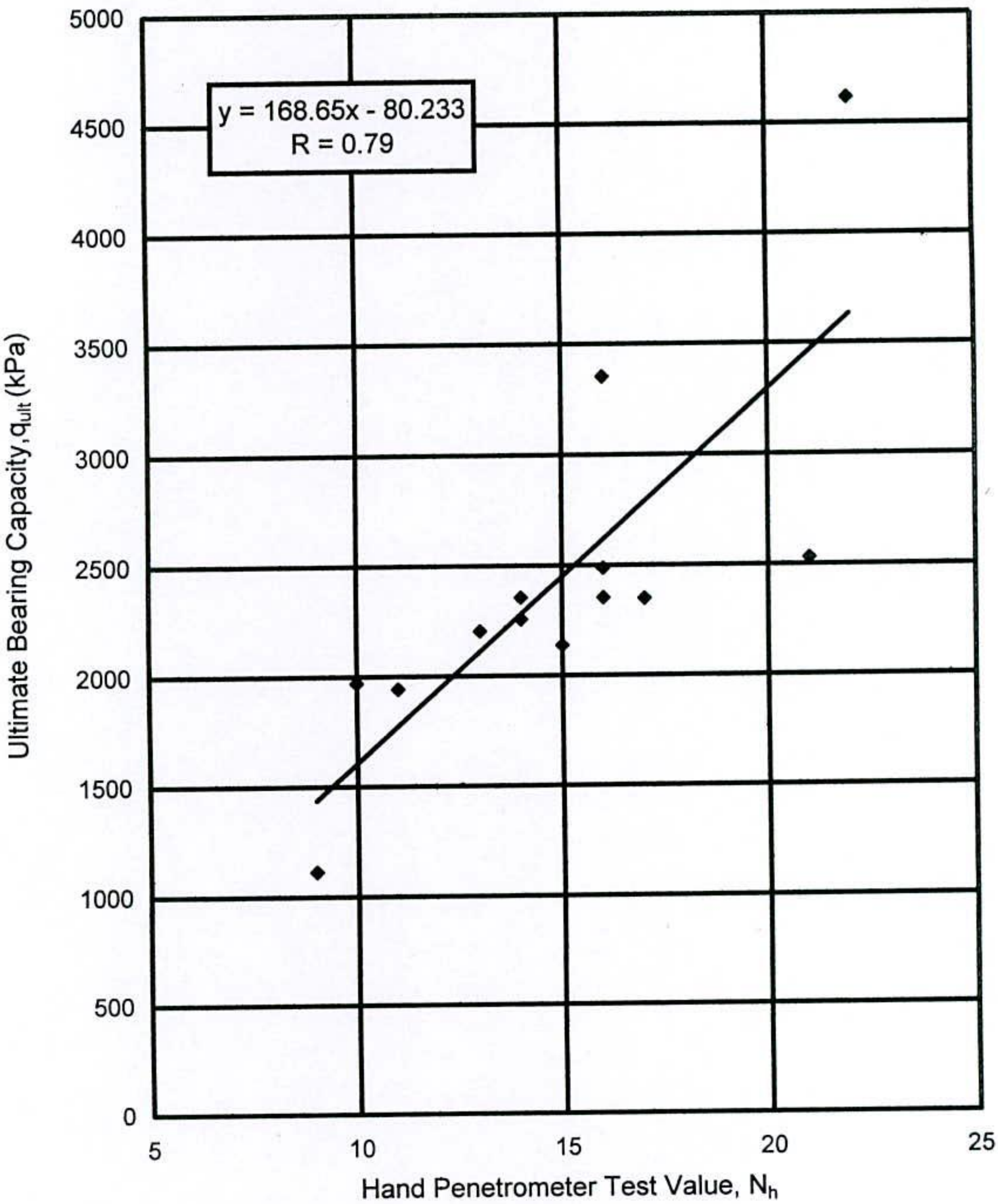

Fig. 5.17 Relation Between Ultimate Bearing Capacity and Hand Penetrometer Test Value for Continuous Footing (For 18.75 mm Dia. Penetrometer, D =120 cm, B=D).

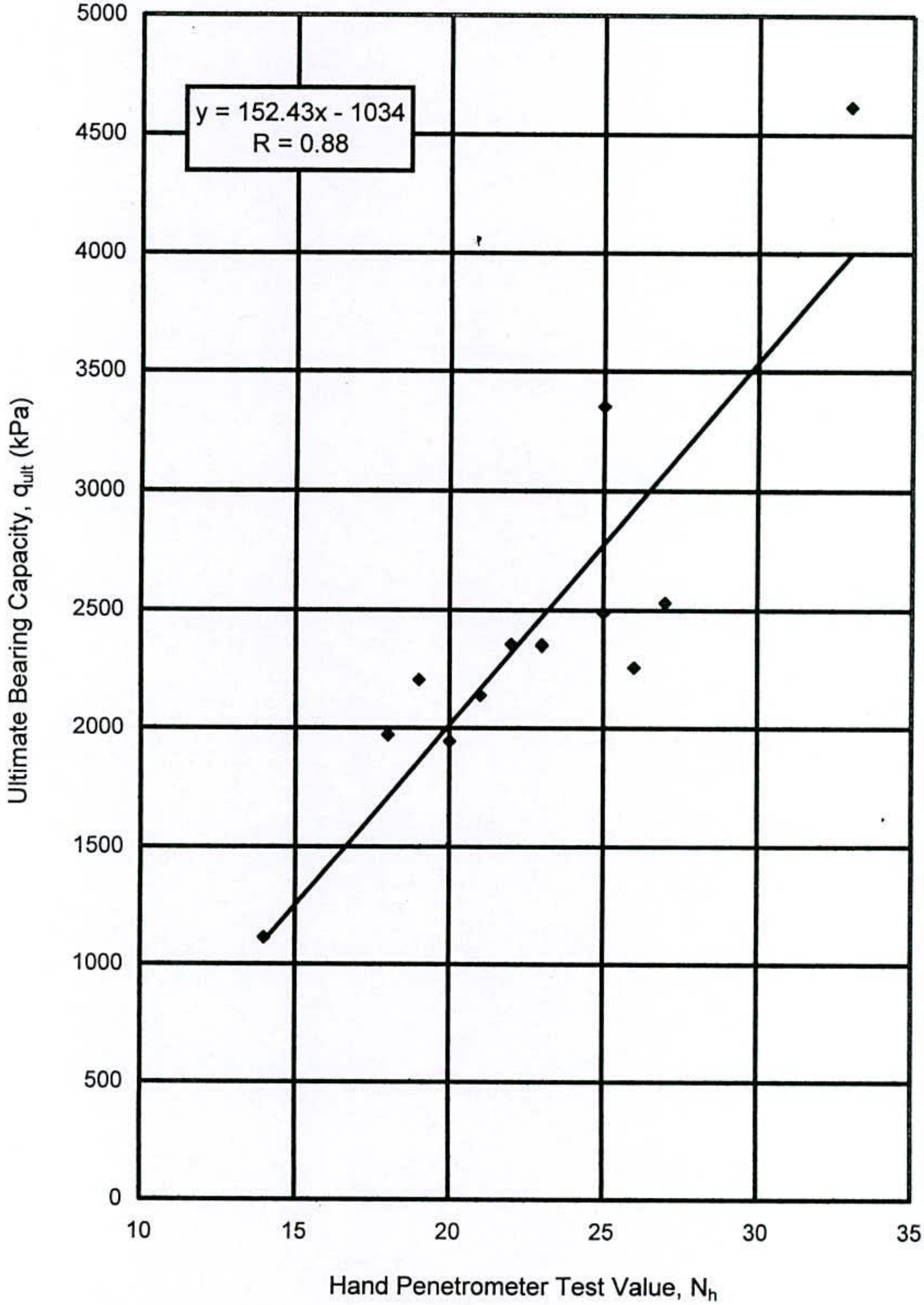

Fig. 5.18 Relation Between Ultimate Bearing Capacity and Hand Penetrometer Test Value for Continuous Footing (For 25mm Dia. Penetrometer, D = 120 cm, B = D)

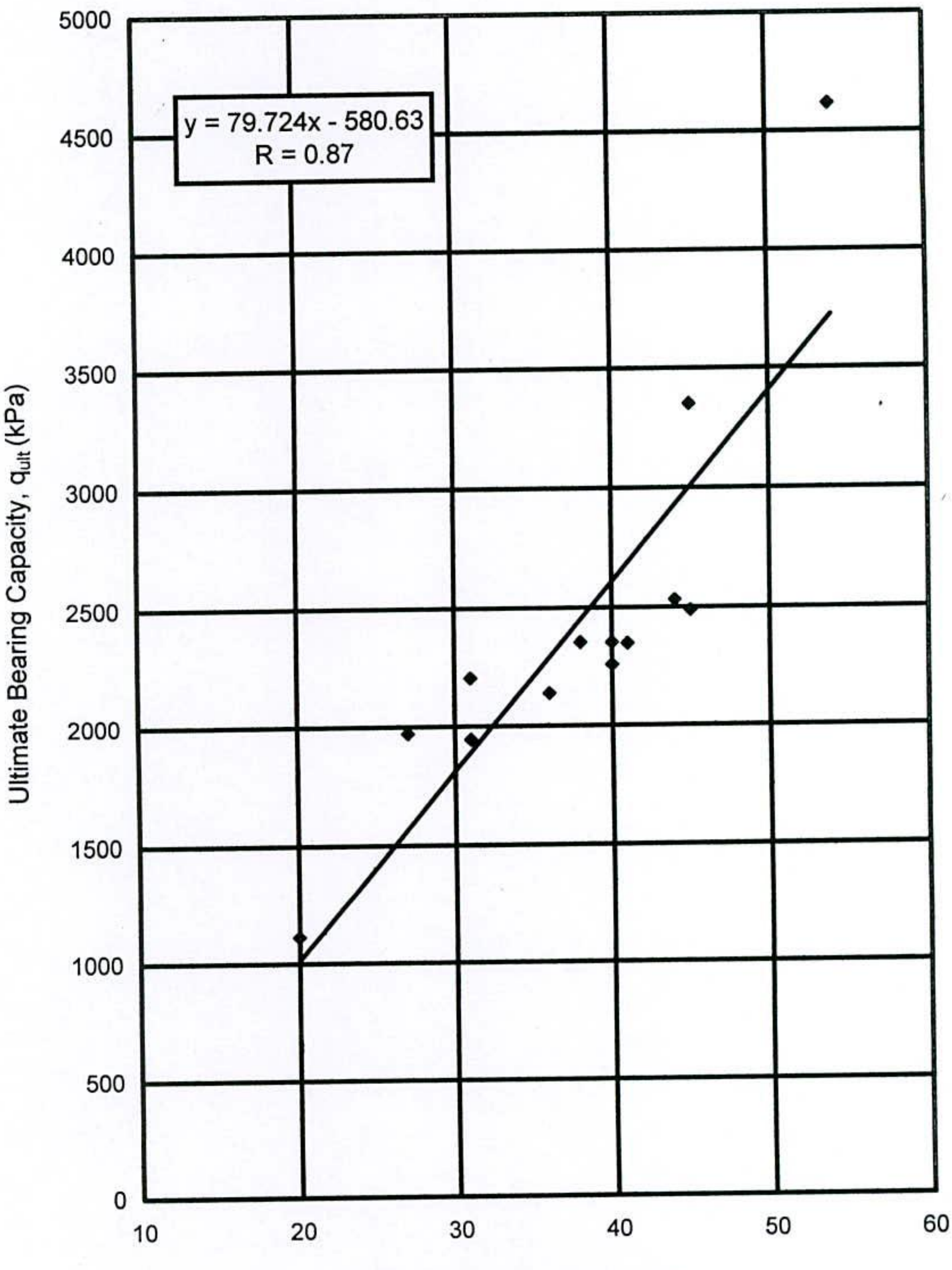

Fig. 5.19 Relation Between Ultimate Bearing Capacity and Hand Penetrometer Test Value for Continuous Footing (for 32.25mm dia. Penetrometer, D = 120 cm, B =D)

5.7 Correlation Between Friction Angles and Bearing Capacities of Sand Beds

Fig. 5.20 shows the variation of bearing capacity with the friction angle of the samples. This variations are also shown in Table 5.21. It is observed from the Fig. 5.20 that bearing capacity of sand increases with the increase of friction angle. From this figure an equation has been developed with coefficient of correlation 0.96 which is shown in Table 5.22.

Table 5.21 Comparison of Friction Angle, ϕ with the variation of Ultimate Bearing Capacity, (q_{ult}).

Sample No.	F.M.	Friction Angle, ϕ	Ultimate Bearing Capacity, (q_{ult}),
S1	2.28	42.4°	4617
S2	2.05	41.0°	3352
S3	1.95	40.2°	2488
S4	1.90	39.7°	2532
S5	1.64	39.4°	2349
S6	1.63	39.2°	2353
S7	1.46	39.0°	2258
S8	1.42	39.0°	2354
S9	1.24	38.6°	2139
S10	1.09	38.6°	2205
S11	1.04	38.4°	1944
S12	0.89	38.2°	1971
S13	0.68	37.8°	1114

Table 5.22 Constitutive Equation for Friction Angle, ϕ and Ultimate Bearing Capacity, (q_{ult})

Serial No.	Constitutive Equations	Coefficient of Correlations, R
1.	$y = 623.94x - 22113$	0.96

where, y = ultimate bearing capacity (kPa)
x = friction angle or angle of internal friction

The value of the coefficient of correlation (R) is very high. From Table 5.22 it can be observed that the equation provide excellent result of ultimate bearing capacity from known value of friction angle.

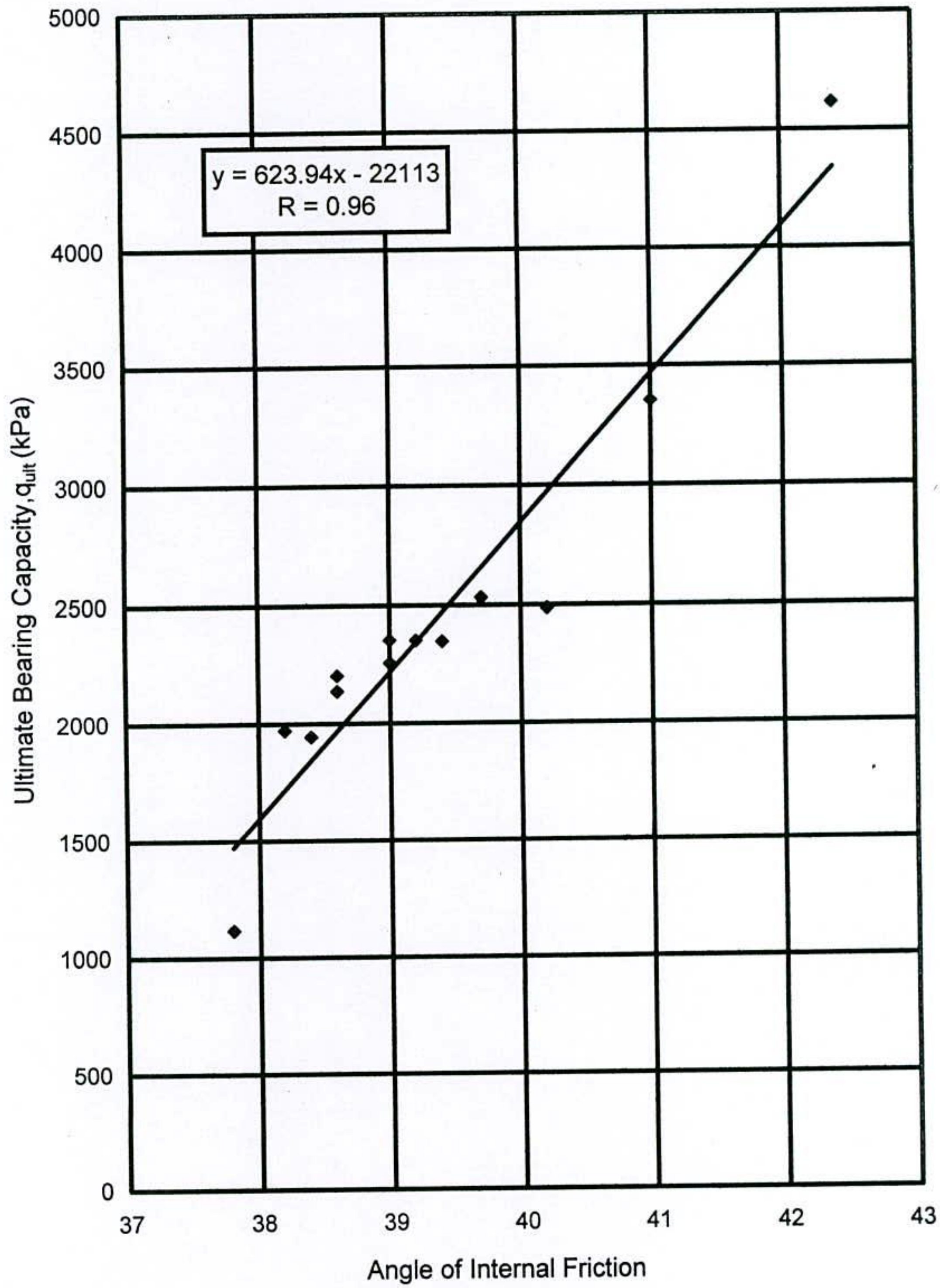

Fig. 5.20 Relationship Between Angle of Internal Friction and ultimate Bearing Capacity of Continuous Footing (for D = 120 cm, B = D)

CHAPTER 6

CONCLUSION AND RECOMMENDATIONS

6.1 Conclusions

From this investigation the main findings and conclusions may be summarized as follows:

(i) Penetrometer of all the diameters showed good correlation between hand penetrometer test value and bearing capacity of granular soil (sand).

(ii) Penetrometer of all the diameters showed good correlation between hand penetrometer test value and friction angle of granular soil (sand).

(iii) From the finding of present investigation, it was established that irrespective of any diameter, ultimate bearing capacity of granular soil can be found out directly from the known value of hand penetrometer test.

6.2 Recommendations for Future Study

The following recommendations can be made to the extend the scope of the present investigation:

(i) To establish the correlation between the hand penetrometer test value and other soil parameters.

(ii) Wide variation of F. M. can be considered to verify the present findings.

(iii) In this study cohesionless sand soil was used, similar study can be carried out on c-ϕ soil.

(iv) Correlation between relative density and hand penetrometer test value can be established.

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APPENDIX-I

A: Determination of Field Density of Granular Soil of Sample S1

Volume of Test Hole

1. Weight of apparatus filled with sand	4906.00 gms
2. Weight of apparatus and remaining sand	1982.40 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	2923.60 gms
4. Weight of sand in cone and plate	1706.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1217.60 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	822.70 cc

Wet Density

8. Weight of moist soil from hole plus tare	1999.50 gms
9. Weight of tare	410.00 gms
10. Weight of moist soil (Item 8 minus Item 9)	1589.50 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.93 gm/cc

Moisture Content and Dry Density

12. Weight of wet sample plus tare	142.50 gms.
13. Weight of dry sample plus tare	132.00 gms.
14. Weight of water in sample	10.50 gms.
15. Tare Number and Weight No.	22.70 gms.
16. Weight of dry soil (Item 13 minus Item 15)	109.30 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	9.60 %
18. Dry density $\frac{\text{Item 11}}{\text{1.0 plus Item 17}}$	1.76 gm/cc

B: Determination of Field Density of Granular Soil of Sample S2

Volume of Test Hole

1. Weight of apparatus filled with sand	4832.30 gms
2. Weight of apparatus and remaining sand	1957.20 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	2875.10 gms
4. Weight of sand in cone and plate	1706.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1169.10 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	789.93 cc

Wet Density

8. Weight of moist soil from hole plus tare	1832.00 gms
9. Weight of tare	410.00 gms
10. Weight of moist soil (Item 8 minus Item 9)	1422.00 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.80 gm/cc

Moisture Content and Dry Density

12. Weight of wet sample plus tare	134.50 gms.
13. Weight of dry sample plus tare	127.30 gms.
14. Weight of water in sample	7.20 gms.
15. Tare Number and Weight No.	22.70 gms.
16. Weight of dry soil (Item 13 minus Item 15)	104.60 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	6.88 %
18. Dry density $\frac{\text{Item}}{\text{1.0 plus Item 17}}$	1.68 gm/cc

C: Determination of Field Density of Granular Soil of Sample S4

Volume of Test Hole

1. Weight of apparatus filled with sand	4857.00 gms
2. Weight of apparatus and remaining sand	2154.00 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	2703.00 gms
4. Weight of sand in cone and plate	1620.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1083.00 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	731.76 cc

Wet Density

8. Weight of moist soil from hole plus tare	1779.50 gms
9. Weight of tare	411.00 gms
10. Weight of moist soil (Item 8 minus Item 9)	1368.50 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.87 gm/cc

MoIsture Content and Dry Density

12. Weight of wet sample plus tare	130.30 gms.
13. Weight of dry sample plus tare	120.20 gms.
14. Weight of water in sample	10.10 gms.
15. Tare Number and Weight No.	22.70 gms.
16. Weight of dry soil (Item 13 minus Item 15)	97.50 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	10.36 %
18. Dry density $\frac{\text{Item}}{\text{1.0 plus Item 17}}$	1.69 gm/cc

D: Determination of Field Density of Granular Soil of Sample S5

Volume of Test Hole

1. Weight of apparatus filled with sand	4934.00 gms
2. Weight of apparatus and remaining sand	2081.50 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	2852.50 gms
4. Weight of sand in cone and plate	1620.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1232.50 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	832.77 cc

Wet Density

8. Weight of moist soil from hole plus tare	1876.70 gms
9. Weight of tare	411.00 gms
10. Weight of moist soil (Item 8 minus Item 9)	1465.70 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.76 gm/cc.

Moisture Content and Dry Density

12. Weight of wet sample plus tare	130.20 gms.
13. Weight of dry sample plus tare	121.60 gms.
14. Weight of water in sample	8.60 gms.
15. Tare Number and Weight No.	22.50 gms.
16. Weight of dry soil (Item 13 minus Item 15)	99.10 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	8.68 %
18. Dry density $\frac{\text{Item 11}}{\text{1.0 plus Item 17}}$	1.62 gm/cc

E: Determination of Field Density of Granular Soil of Sample S6

<u>Volume of Test Hole</u>

1. Weight of apparatus filled with sand	4822.00 gms
2. Weight of apparatus and remaining sand	1721.00 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	3101.00 gms
4. Weight of sand in cone and plate	1620.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1481.00 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	1000.68 cc

<u>Wet Density</u>

8. Weight of moist soil from hole plus tare	2244.00 gms
9. Weight of tare	411.00 gms
10. Weight of moist soil (Item 8 minus Item 9)	1833.00 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.83 gm/cc

<u>Moisture Content and Dry Density</u>

12. Weight of wet sample plus tare	141.00 gms.
13. Weight of dry sample plus tare	130.10 gms.
14. Weight of water in sample	10.90 gms.
15. Tare Number and Weight No.	22.50 gms.
16. Weight of dry soil (Item 13 minus Item 15)	107.60 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	10.13 %
18. Dry density $\frac{\text{Item 11}}{\text{1.0 plus Item 17}}$	1.66 gm/cc

F: Determination of Field Density of Granular Soil of Sample S7

Volume of Test Hole

1. Weight of apparatus filled with sand	4929.00 gms
2. Weight of apparatus and remaining sand	2141.00 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	2788.00 gms
4. Weight of sand in cone and plate	1620.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1168.00 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	789.19 cc

Wet Density

8. Weight of moist soil from hole plus tare	1880.00 gms
9. Weight of tare	411.00 gms
10. Weight of moist soil (Item 8 minus Item 9)	1473.00 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.87 gm/cc

Moisture Content and Dry Density

12. Weight of wet sample plus tare	124.50 gms.
13. Weight of dry sample plus tare	111.70 gms.
14. Weight of water in sample	12.80 gms.
15. Tare Number and Weight No.	22.70 gms.
16. Weight of dry soil (Item 13 minus Item 15)	89.00 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	14.38 %
18. Dry density $\frac{\text{Item 11}}{\text{1.0 plus Item 17}}$	1.63 gm/cc

G: Determination of Field Density of Granular Soil of Sample S8

Volume of Test Hole

1. Weight of apparatus filled with sand	4834.00 gms
2. Weight of apparatus and remaining sand	2116.50 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	2717.50 gms
4. Weight of sand in cone and plate	1620.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1097.50 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	741.55 cc

Wet Density

8. Weight of moist soil from hole plus tare	1864.00 gms
9. Weight of tare	411.00 gms
10. Weight of moist soil (Item 8 minus Item 9)	1453.00 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.96 gm/cc

Moisture Content and Dry Density

12. Weight of wet sample plus tare	120.40 gms.
13. Weight of dry sample plus tare	107.50 gms.
14. Weight of water in sample	12.90 gms.
15. Tare Number and Weight No.	22.60 gms.
16. Weight of dry soil (Item 13 minus Item 15)	84.90 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	15.19 %
18. Dry density $\frac{\text{Item 11}}{\text{1.0 plus Item 17}}$	1.70 gm/cc

H: Determination of Field Density of Granular Soil of Sample S9

Volume of Test Hole

1. Weight of apparatus filled with sand	4867.50 gms
2. Weight of apparatus and remaining sand	1862.40 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	3005.10 gms
4. Weight of sand in cone and plate	1620.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1385.10 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	935.88 cc

Wet Density

8. Weight of moist soil from hole plus tare	2084.40 gms
9. Weight of tare	418.50 gms
10. Weight of moist soil (Item 8 minus Item 9)	1665.90 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.78 gm/cc

Moisture Content and Dry Density

12. Weight of wet sample plus tare	135.80 gms.
13. Weight of dry sample plus tare	125.50 gms.
14. Weight of water in sample	10.30 gms.
15. Tare Number and Weight No.	22.60 gms.
16. Weight of dry soil (Item 13 minus Item 15)	102.90 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	10.01 %
18. Dry density $\frac{\text{Item 11}}{\text{1.0 plus Item 17}}$	1.62 gm/cc

I: Determination of Field Density of Granular Soil of Sample S10

<u>Volume of Test Hole</u>

1. Weight of apparatus filled with sand	4705.00 gms
2. Weight of apparatus and remaining sand	2015.50 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	2689.50 gms
4. Weight of sand in cone and plate	1620.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1069.50 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	722.64 cc

<u>Wet Density</u>

8. Weight of moist soil from hole plus tare	1813.00 gms
9. Weight of tare	411.00 gms
10. Weight of moist soil (Item 8 minus Item 9)	1402.00 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.94 gm/cc

<u>Moisture Content and Dry Density</u>

12. Weight of wet sample plus tare	110.80 gms.
13. Weight of dry sample plus tare	98.60 gms.
14. Weight of water in sample	12.20 gms.
15. Tare Number and Weight No.	22.60 gms.
16. Weight of dry soil (Item 13 minus Item 15)	76.00 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	16.05 %
18. Dry density $\frac{\text{Item 11}}{\text{1.0 plus Item 17}}$	1.67 gm/cc

J: Determination of Field Density of Granular Soil of Sample S11

Volume of Test Hole

1. Weight of apparatus filled with sand	4652.30 gms
2. Weight of apparatus and remaining sand	1596.00 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	3056.30 gms
4. Weight of sand in cone and plate	1620.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1436.30 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	970.47 cc

Wet Density

8. Weight of moist soil from hole plus tare	2014.00 gms
9. Weight of tare	418.50 gms
10. Weight of moist soil (Item 8 minus Item 9)	1595.50 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.64 gm/cc

Moisture Content and Dry Density

12. Weight of wet sample plus tare	129.50 gms.
13. Weight of dry sample plus tare	121.20 gms.
14. Weight of water in sample	8.30 gms.
15. Tare Number and Weight No.	22.60 gms.
16. Weight of dry soil (Item 13 minus Item 15)	98.60 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	8.42 %
18. Dry density $\frac{\text{Item 11}}{\text{1.0 plus Item 17}}$	1.51 gm/cc

K: Determination of Field Density of Granular Soil of Sample S12

Volume of Test Hole

1. Weight of apparatus filled with sand	4239.80 gms
2. Weight of apparatus and remaining sand	1324.80 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	2915.00 gms
4. Weight of sand in cone and plate	1620.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1295.00 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	875.00 cc

Wet Density

8. Weight of moist soil from hole plus tare	1980.50 gms
9. Weight of tare	418.50 gms
10. Weight of moist soil (Item 8 minus Item 9)	1562.00 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.79 gm/cc

Moisture Content and Dry Density

12. Weight of wet sample plus tare	130.00 gms.
13. Weight of dry sample plus tare	116.60 gms.
14. Weight of water in sample	13.40 gms.
15. Tare Number and Weight No.	22.80 gms.
16. Weight of dry soil (Item 13 minus Item 15)	93.80 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	14.29 %
18. Dry density $\frac{\text{Item 11}}{\text{1.0 plus Item 17}}$	1.57 gm/cc

L: Determination of Field Density of Granular Soil of Sample S13

Volume of Test Hole

1. Weight of apparatus filled with sand	4902.50 gms
2. Weight of apparatus and remaining sand	1859.20 gms
3. Weight of sand in hole, plate and cone (Item 1 minus Item 2)	3043.30 gms
4. Weight of sand in cone and plate	1620.00 gms
5. Weight of sand in hole (Item 3 minus Item 4)	1423.30 gms
6. Bulk density of sand	1.48 gms/cc
7. Volume of test hole Item 5 ÷ Item 6	961.69 cc

Wet Density

8. Weight of moist soil from hole plus tare	1997.00 gms
9. Weight of tare	418.50 gms
10. Weight of moist soil (Item 8 minus Item 9)	1578.50 gms
11. Wet Density $\frac{\text{Item 10}}{\text{Item 7}}$	1.64 gm/cc

Moisture Content and Dry Density

12. Weight of wet sample plus tare	124.50 gms.
13. Weight of dry sample plus tare	109.60 gms.
14. Weight of water in sample	14.90 gms.
15. Tare Number and Weight No.	22.80 gms.
16. Weight of dry soil (Item 13 minus Item 15)	86.80 gms.
17. Moisture content $\frac{\text{Item 14}}{\text{Item 16}} \times 100$	17.17 %
18. Dry density $\frac{\text{Item 11}}{\text{1.0 plus Item 17}}$	1.40 gm/cc