STRENGTH AND DEFORMATION BEHAVIOUR OF A SOIL CONSOLIDATED BY USING RICE HUSK FILLED VERTICAL DRAINS

A Project

by

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Submitted to the Department of Civil Engineering, Bangladesh University of Engineering & Technology, Dhaka in partial fulfilment of the requirements for the degree

of

MASTER OF ENGINEERING IN CIVIL ENGINEERING

May, 1990

BANGLADESH UNIVERSITY OF ENGINEERING AND TECHNOLOGY, DHAKA

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ABSTRACT

An experimental investigation into the prospect of using rice husk in vertical drains for the improvement of a very soft soil of Dhaka city is carried out in this study. The investigation includes a series of vertical and radial flow consolidation test using specially prepared consolidation molds. Three different sizes of molds were used and samples were consolidated using three different dimensions of drains. Drains were filled with rice husk and sand independently. Triaxial tests were performed to measure the undrained shear strength of the consolidated samples. Results were examined and compared interms of gain of undrained shear strength of the soil and its rate. They show a higher and enhanced gain for the instance of rice husk filled drains. Statistical analyses of the observed data indicate good correlations among soil and drain parameters.

ACKNOWLEDGEMENTS

The author wishes to express his sincere gratitude to Dr. Md. Zoynul Abedin, Associate Professor, Department Of Civil Engineering, BUET, for his constant supervision, continuous guidance, helpful suggestions and encouragement given throughout the course of this research.

The author is grateful to Dr. A. M. Haque, Professor and Head, Department of Civil Engineering, BUET, for his Cooperation.

Profound gratude is expressed to Dr. M. Hossain Ali, Professor, Department of Civil Engineering, BUET and Dr. M. Humayun Kabir, Professor, Department of Civil Engineering, BUET, for their support and encouragement.

Thanks are also due to Mr. Habibur Rahman and Mr. Alimuddin for their help during the laboratory work required for this research; and to Mr. M. A. Malek and Mr. Shahiduddin for their neat typing and drafting of the figures respectively.

Finally, the author would like to express his gratitude and appreciation to his parents, relatives and many friends for their constant inspiration and encouragements.

CONTENTS

NOTATION

ASTM	-	American Society for Testing Materials
c	-	cohesion of soil
C_r	-	coefficient of consolidation for radial flow
C_v	-	coefficient of consolidation for vertical flow
D	-	loading/sample diameter
d_e	-	diameter of equivalent cylinder of drained soil
d_s	-	diameter of smeared zone
d_w	-	diameter of drain well
H	-	height of test soil specimen
I_{ss}	-	shear strength improvement ratio for sand
I_{sr}	-	shear strength improvement ratio for rice husk
I_{sw}	-	shear strength improvement ratio with no drain
K_s	-	cofficient of permeability for smeared zone
K_v	-	coefficient of permeability in vertical direction
m	-	a factor (which depends on n, S, k_r, k_s)
N	-	dimensionless governing equation parameter
n	-	a ratio = d_e/d_w
P	-	consolidation pressure
q_w	-	drainage capacity of the drain well
r	-	a coordinate of polar system
r^2	-	coefficient of correlation
R_o	-	theoretical zero compression

r_e - radius of the equivalent cylinder of drained soil

r_s - radius of smeared zone

r_w - radius of drain well

s - a ratio = r_s/r_w

S - spacing of drain wells

S_t - triaxial shear strength

t - duration of loading

T - time factor

t_{50} - time for 50% consolidation

T_r - time factor for radial flow

T_v - time factor for vertical flow

u_o - initial pore water pressure

u - excess pore water pressure

u_{rv} - excess pore water pressure due to both radial and vertical flow

U - average degree of consolidation

U_r - average degree of consolidation due to radial flow

U_{rv} - average degree of consolidation due to both radial and vertical flow

U_v - average degree of consolidation due to vertical flow

w - water content of soil

w_L - liquid limit of soil

w_p - plastic limit of soil

z - a coordinate in both the rectangular and cylindrical systems

P - pressure increment

H - change in sample thickness after consolidation

φ - angle of internal friction

φ_e - true (effective) angle of internal friction

γ - unit weight of soil

γ_w - unit weight of water

CHAPTER 1

INTRODUCTION

1.1 GENERAL

In Bangladesh , there has been a growing demand for construction on sites underlain by a thick layer of soft soil. Circumstances require that, the soft soil must be properly treated before constructing a structure on such a soil. Otherwise the structure may damage due to shear failure and/or excessive total and differential settlement of the underlying soil. Of the several methods of soil improvement being practiced in Bangladesh, vertical sand filled drains and preloading method appeared to be very common. This may be due to its simplicity in construction and its relative lower cost.

When vertical drains are installed, in addition to vertical flow, pore water also escapes in the lateral direction toward the drains and flows freely around the drains vertically to a drainage blanket placed on the soil surface or to other permeable layer deeper down the soil. Thus, the installation of vertical drains in homogeneous clay layer will reduce the length of flow paths, thereby reducing the time of consolidation.

According to Jumikis (1962) the efficiency of a vertical drain depends on smear and well resistances interms of their permeabilities. The permeability of a vertical drain, in turn, is a function of properties of the materials involved and/or gradation of the material filling the drains (Mofiz,1989).

An expedite rate and higher amount of consolidation require that the diameter of a drain be larger. However, the process of consolidation, in such a case, is hindranced due to other reasons. McGown and Hughes (1982) reports that large diameter sand drains may tend to act as strengthening columns in soft soils and modify the settlement behaviour of the structure. Hence, the design of a granular vertical drain calls for the optimization of its dimension considering permeability, drainage path and drain column strength aspects. It is commonly desirable that the permeability of drainage material is higher, the drain diameter is larger and the drain column strength is lower.

The literature reveals that professionals and designers in this field paid enormous attention to the use of wick type of drainage material in vertical drains. However, the attention towards the application of granular materials in vertical drains, other than sand, is surprisingly very low.

Mofiz (1989) initiated the investigation in to the prospect of using rice husk in vertical drains as an alternative to sand. He found that a rice husk filled drain yields a quicker and higher gain in shear strength of the consolidated soil than that of a sand filled drains of comparable dimension. His investigation was limited to a single load intensity and suggests to verify the statement by performing tests for wide range of loading conditions.

In view of the above information, research on the prospect of using rice husk in vertical drain is worthwhile.

1.2 OBJECTIVE OF RESEARCH

The present study is concerned with the use of sand and rice husk in vertical drains for the improvement of soft soils. The main objectives of the present study are:

i) to investigate the effect of load intensity on the gain of shear strength of soft soil.

ii) to compare the gain of shear strength and deformation behaviour of soil for the cases of sand and rice husk filled drains.

1.3 ORGANIZATION OF THE REPORT

The investigation is presented in six chapters, the first one is introduction. Chapter 2 presents literature review and briefly

outlines the aim and the scope of the present study.
Chapter 3 describes the experimental setup and materials used in the investigation. Chapter 4 presents the test programmes. Chapter 5 is concerned with the test results and analysis. Chapter 6 outlines the conclusion of the present study and recommendations for future work.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Porter* (1936) reports that the use of vertical drains to promote rapid consolidation in relatively thick deposits of soft fine grained soils dates back to 1934 in California. Since then, extensive research and practice are called for into its developments which result in the use of mainly two types of vertical drains:(1) Drains made by filling a cylindrical hole with granular materials, mainly, sand and (2) Prefabricated drains. Until the early 1970s the majority of the vertical drains used large diameter sand drains(Mofiz,1989). The principle alternative to large diameter sand drains was the much smaller band shaped cardboard wicks first employed by Kjellman(1948).

Until about 1950 most installed vertical drains were sand drains because the materials that were available for the prefabricated variety(cardboard) did not always provide the strength required for installation and durability required for exposure. In 1950s the availability of reasonably priced plastics gave prefabricated drains a big boosting, first in Europe and Japan and later in

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* Cited by Brand(1981)

North America. Since 1980 prefabricated drains have gained most of the market because they are cheaper and faster to install (Morrison,1982).

The benefits of vertical drains used in conjunction with preloading are related almost entirely to the acceleration of the primary consolidation of the clay, since they bring about the rapid dissipation of excess pore water pressures. Vertical drains have no direct effect on the rate of secondary compression, but early completion of primary consolidation brings about the earlier occurrence of significant secondary compressions(Brand, 1981).

The effectiveness of vertical drains depends to a great extent on the properties of the soil which, in turn, depend largely on geological factors. Of primary importance are the soil permeability and coefficient of consolidation and their variations in space and time. Deposits of recent clays often exhibit horizontal stratification, with frequent thin seams of sand or silt of relatively high permeability. The importance to be ascribed to these macro-structural features has been emphasized by Rowe (1968).

2.2 VERTICAL DRAINS

The decision as to whether or not vertical drains are employed to aid the rate of settlement of a fill depends on a number of factors, as well the selection of the particular type of drain. The types of drain available are: (1) Drains of granular material (2) Cardboard drains, (3) Sandwicks, and (4) Plastic drains. Although the comparative costs of drain installation are of paramount importance in the selection process, there are other factors that relates to drain performance which must be considered (Rowe, 1968). They are (i) Smear and distortion of the drain walls, which reduce drain permeability, and (ii) disturbances and lateral deformations of the soft ground resulting from drain installation, which reduce the permeability, coefficient of consolidation and drained strength of the soil while increasing the pore pressures. In the following sections the principles features of different vertical drains and the theory behind their performance is briefly outlined.

2.2.1 Drains of Granular Material

Sand drains have been widely used for over forty years. They range in diameter from 180 to 450 mm and they have been installed to great depth by a great variety of procedures, the most common of which are the closed mandrel and open mandrel methods.

Closed mandrels, which consists of steel tubes closed at the lower end by a loose cap, are driven by percussion or vibration or sometimes are fitted into place before being filled with sand and the tubes extracted. This is a simple, inexpensive method which is very popular. Its major drawbacks, however, is the disturbance caused to the soil by the displacement during installation, which results in decreased strength and permeability and increased settlement.

Open mandrels are installed by the same means as closed mandrels, the soil in the tube being removed by jetting or by means of an auger before sand is placed in the hole as the tube is removed. Significantly less disturbance results from the installation process than with the closed mandrel method, but problem of smear still exists.

Very recently Mofiz (1989) used rice husk in vertical drains and reports that the rice husk filled drain yields a higher and enhanced gain of undrained shear strength.

2.2.2 Cardboard Drains:

The first to suggest the use of driven cardboard drains to replace sand drains was Kjellman (1948), who had carried out tests over several years. Those first cardboard drains were 100mm wide and 3 mm thick, but other sizes have been used subsequently. They are inserted into ground by means of a purpose made mandrel which is then removed. Channels in cardboard facilitate the passage of water from the clay to the ground surface. A 100 mm x 3 mm cardboard drains is roughly equivalent to a 50 mm diameter sand drain.

Cardboard drains have the advantage of ease of installation, which also causes relatively little soil disturbances. They can be very closely spaced if required. The specially processed cardboard has long life, but there is some question about the ability of this material to withstand the large deformations which often result in soft soils.

2.2.3 Sandwicks

Sandwicks are ready made small diameter sand drains which are contained in long canvas bags. The most common diameter is about 100 mm. They are usually installed by the closed mandrel technique. Sand wicks were first used in India by Dastidar et al (1969) and their satisfactory use in that country has been further described by Subbaraju et al (1973). This type of vertical drain is economical where labour costs are low, since the making and filling of the canvass bags are generally done by hand. Installation in soft clay can be accomplished by hand.

2.2.4 Plastic Drains

Plastic drains have recently been introduced to replace cardboard drains as the most popular manufactured vertical drains. Particularly well known is the Geodrain, which consists of a 100 mm wide x 3 mm thick paper covered polythene strip which contains channels along both sides (Boman*, 1973, Terrafigo*, 1976). The efficiency of Geodrains has been established at various test sites (Hansbo & Torstensson, 1977) including the well known Ska-Edeby Site. Comparison with sand drain showed that relatively little disturbance in the soft soil occurs with the plastic drain, and that large settlements do not destroy drain continuity.

2.3 CONSOLIDATION DUE TO VERTICAL DRAINS

The consolidation of soil due to the installation of vertical drains and subjected to preloading is achieved by mainly two directional flows: (i) vertical flow and (ii) radial flow.

2.3.1 Consolidation due to Vertical Flow

Terzaghi (1925) obtained the solution of basic differential

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* Cited by Mitchel and Katti(1981)

equation for vertical flow and expressed, interms of degree of consolidation, as

$$U(\%) = 1 - \frac{8}{\pi^2} \sum_{N=0}^{N=\infty} \frac{1}{(2N+1)^2} e^{-(2N+1)/4 \; \pi T_v} \times 100 \quad (2.1)$$

where, U is the degree of consolidation

T_v is the time factor

For details reference can be made to Punmia (1977).

2.3.2 Consolidation due to Radial Flow

The treatment of consolidation due to radial flow only is an extension of the Terzaghi's consolidation theory. Drains are installed in triangular or rectangular grid pattern. The problem is approximated by assuming a cylindrical drain placed at the centre of a cylinder of soil to be consolidated. A solution of this truly axisymmetric case was presented by Rendulic* (1935) expressing t and the degree of consolidation U_r by the characteristic equation.

$$U_r = F(T_r) \quad (2.2)$$

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* Cited by Das(1985)

where T_r = time factor for radial flow

$T_r = C_r t/D^2_E$

where C_r = coefficient of consolidation in horizontal i.e. radial direction.

t = time

D_E = length of drainage path but in the case of radial flow effective diameter of soil cylinder from which water will flow into the sand drain.

Solution of equation(2.2) is was given by Rendulic* (1935) in the form

$$U_r = 1 - \exp(-8C_r t/D^2_E\ f(n)) \qquad (2.3)$$

where $f(n) = (n^2/(n^2-1))\ln(n) - (3n^2-1)/4n^2$

Carillo (1942) has shown that the solution of equation(2.2) is given by a combination of the solution as follows:

$$(1 - U_{rv}) = (1-U_v)(1-U_r) \qquad (2.4)$$

where U_{rv} = degree of consolidation for both vertical and radial flow of water through soil.

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* Cited by Das(1985)

Combination of equations (2.2), (2.3) and (2.4) enables the total average consolidation to be calculated. Kjellman (1948) proposed the spacing of vertical drains by considering only radial drainage.

Barron (1948) argued that the governing three dimensional consolidation equation for vertical and radial flow is constituted of two differential equations. For radial flow the equation being

$$\frac{\partial u}{\partial t} = C_r \left(\frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right) \qquad (2.5)$$

Barron (1948) gave the solution for radial flow and expresses the results in the form of curves as shown in Fig. 2.1. The values of 'n' mentioned in this solution is the ratio between diameter of equivalent cylinder of drained soil and diameter of drain.

2.4 CONSOLIDATION BY SAND DRAINS

In order to accelerate the process of consolidation settlement for the construction of some structures, the useful technique of building sand drains can be used.

The basic theory of sand drains was presented by Rendulic* (1935) and Barron (1948) and later summarized by Richart (1959). In the

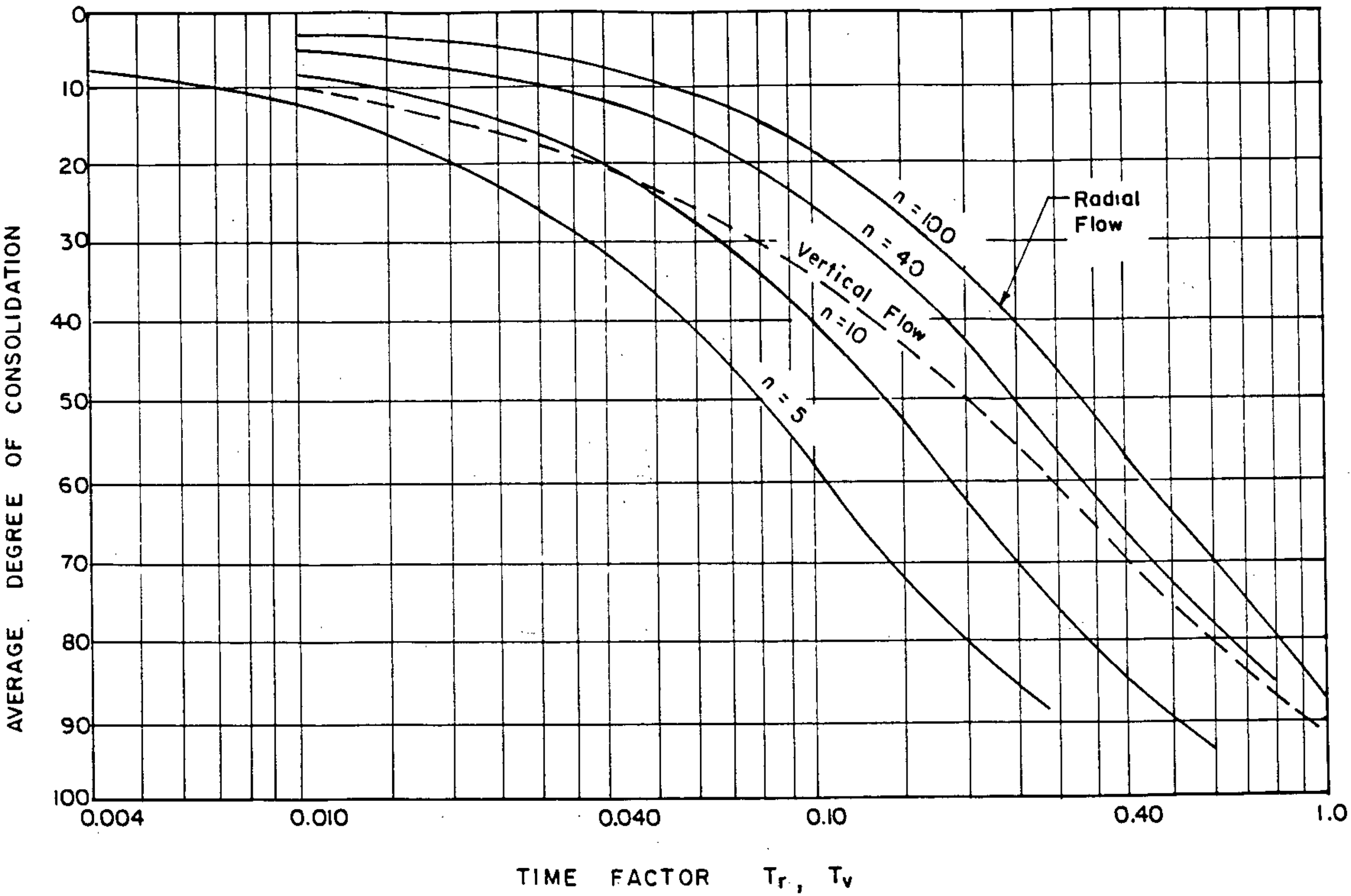

Fig. 2.1 Average rate of consolidation for vertical and radial flow (Barron, 1948).

study of sand drains, two fundamental cases arise:

1. Free strain case: When the surcharge applied at the ground surface is of a flexible nature, there will be equal distribution of surface load. This will result in an uneven settlement at the surface.

2. Equal strain case: When the surcharge applied at the ground surface is rigid, the surface settlement will be the same all over. However, this will result in an unequal distribution of stress.

Another factor that must be taken into consideration is the effect of "smear". A smear zone in a sand drain is created by the remolding of clay during the drilling operation for building it. This remolding of the clay results in a decrease of the coefficient of permeability in the horizontal direction.

In the development of theory of free strain and equal strain, it is assumed that drainage takes place only in radial direction.

Free Strain Consolidation with no Smear:

The initial solution for ideal drain well is made on the assumptions that the load is uniform over the zone of influence for each well and that differential settlements occurring over such a zone, as consolidation progresses, have no effect on the redistribution of stresses. The solution for free strain conditions were obtained by Glover* (1930) and Rendulic* (1935) considering certain boundary conditions, interms of degree of consolidation U_r and and time factor T_r for radial flow. Richart (1959) reported the solution in graphical form (Fig. 2.2) for various ratios of drain spacing and drain diameter 'n'. Details of theoretical solution can be found in Das (1985).

Equal strain consolidation with no Smear:

The problem of equal strain consolidation with no smear ($r_s = r_w$) was solved by Barron (1948). Richart (1959) reported the solution in graphical form, Fig. 2.2. He pointed out that for $n(=r_e/r_w)>5$ the free strain and equal strain solution give approximately similar results for the average degree of consolidation.

2.4.1 Effect of Smear Zone on Radial Consolidation

Barron (1948) extended the analysis of equal strain consolidation by sand drain to account for the smear zone. The analysis is

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* Cited by Das(1985)

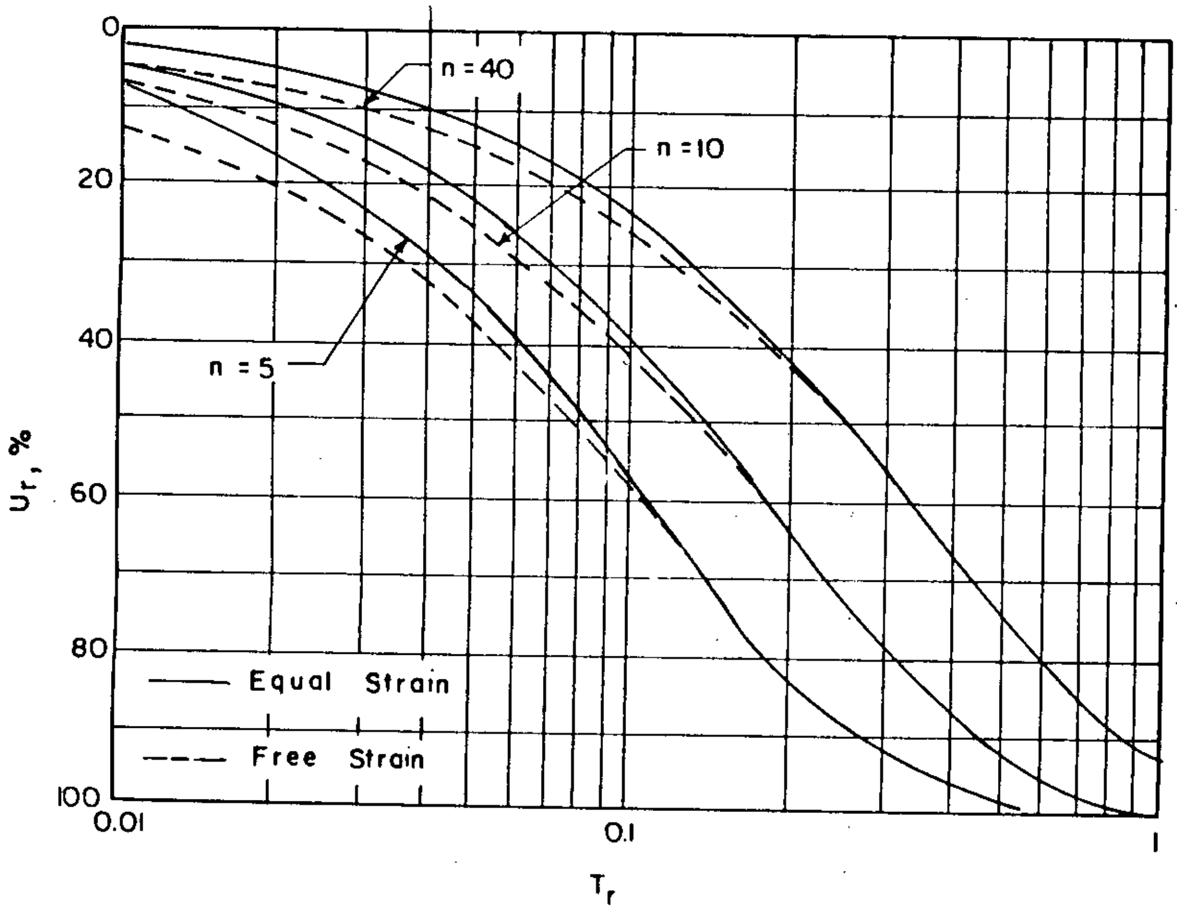

Fig. 2.2 Variation of degree of consolidation U_r with time factor T_r (After F.E. Richart, 1959).

based on the assumption that the clay in the smear zone will have one boundary with zero excess porewater pressure and the other boundary with an excess porewater pressure which will be time dependent. Based on this assumption,

$$u = \frac{1}{m} u_{av} \left[\ln\left(\frac{r}{r_e}\right) - \frac{r^2 - r_s^2}{2r_e^2} + \frac{k_h}{k_s}\left(\frac{n^2 - S^2}{n^2}\right) \ln S \right] \qquad (2.6)$$

where k_s = coefficient of permeability of smeared zone.

$S = r_s / r_w$

$$m = \frac{n^2}{n^2 - S^2} \ln\left(\frac{n}{S}\right) - \frac{3}{4} + \frac{S^2}{4n^2} + \frac{k_h}{k_s}\left(\frac{n^2 - S^2}{n^2}\right) \ln S \qquad (2.7)$$

$$u_{av} = u_i \exp\left(\frac{-8T_r}{m}\right)$$

The average degree of consolidation is given by

$$U_r = 1 - \frac{u_{av}}{u_i} = 1 - \exp\left(\frac{-8T_r}{m}\right) \qquad (2.8)$$

values of m for various values of k_h/k_s and S is given by Richart (1959) and shown in Fig. 2.3.

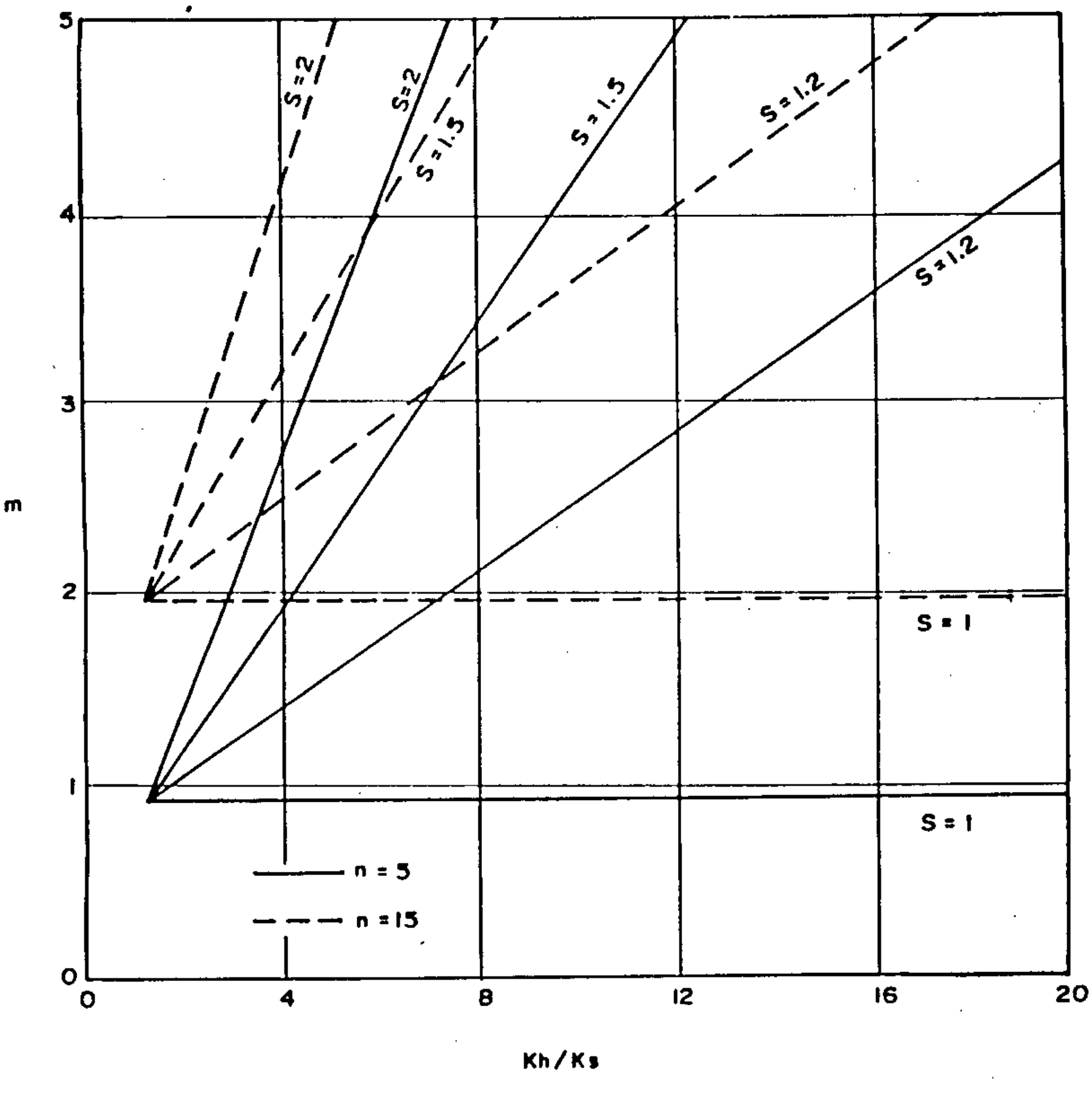

Fig. 2.3 Values of m for various values of k_h/k_s and S (Richart, 1959).

2.4.2 Effect of well resistance on sand drains

Head Losses in sand drains will occur due to flow resistance of the well backfill material (Barron, 1948). The magnitude of head losses will depend upon the rate of flow, the size of well and permeability of the material filling the well.

Barron (1948) has developed a solution for the case of equal vertical strain, with or without smear, where the coefficient of permeability in vertical direction, k_v is equal to zero. When n is in between 7 to 15 and $d_e/H<1$, the effect of consolidation beheaviour due to resistance of the drain well is not significant. However, to the knowledge of the author, solution to the effect of well resistance on the rate of consolidation for free strain case has not been yet developed.

A simple solution to the problem of smear and well resistance was presented by Hansbo (1979), giving results almost identical with those presented by Barron (1948) and Yoshinkuni and Nakanodo (1974). For a saturated soil, the average degree of consolidation U_r at a depth z due to the effect of radial drainage (Fig. 2.4) can be expressed as

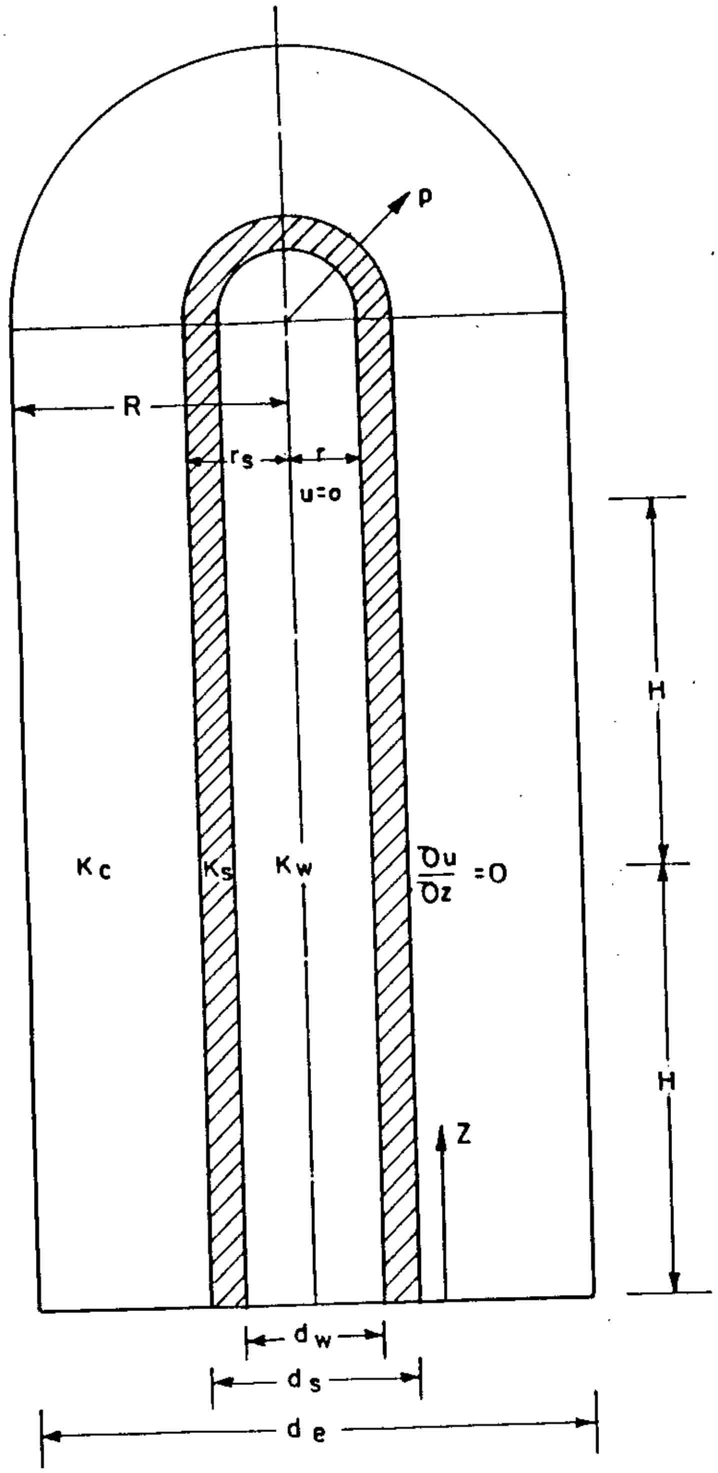

Fig. 2.4 Schematic diagram of soil cylinder dewatered by vertical drain.

$$U_r = 1 - \exp\left(-\frac{8T_r}{\mu_s}\right) \tag{2.9}$$

$$\text{where } \mu_s = \ln\left(\frac{n}{s}\right) + \frac{k_c}{k_s}\ln(s) - \frac{3}{4} + \pi z(2H-z)\frac{k_c}{q_w}$$

$$n = \frac{d_e}{d_w}$$

$$q_w = k_w\ A_w = k_w \frac{\pi\, d_w^2}{4}$$

= drainage capacity of a drain well

The effect of internal resistance of a vertical drain to the flow of the collected water has been considered by a number of authors, notably Barron (1948), Richart (1957) and Bhide* (1979).

In general, consideration has been given to the case where the drain spacing is comparable to the half depth of the drained soil layer. However, in the majority of the cases examined by Casagrande & Poulos (1969), the half depth of the drained stratum was considerably excess of the drain spacing and the internal resistance of the drains may well have made a significant

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* Cited by Atkinson and Eldred(1981)

contribution to the lack of acceleration of the consolidation process (Atkinson and Eldred, 1981).

2.4.3 Consolidation Parameters of Soft Clay due to Vertical Drains

Garassino et al (1979) presented the consolidation characteristics of a thick layer of young normally consolidated silty clay using vertical drains. They used different types of drains namely, Geodrains, sand wicks and soil drains and concluded that there were no appreciable differences between the radial coefficient of consolidation values for different types of drains used. However, a slightly lower values were observed for geodrains. They explained the causes as due either to higher load intensity or to local soil conditions or to a slight reduction of drain efficiency because of partial deterioration of the filter paper. They suggested the use of field measured coefficient of permeability with laboratory measured compressibility to get a reliable value of consolidation coefficient.

A study of the consolidation parameters of soft clay layers improved by means of vertical drains was made by Wolski et al (1979). According to them the radial and vertical coefficient of consolidation, C_h and C_v, used in settlement analysis should be selected with reference to the effective stresses acting at a given moment of the process. They recorded a decrease of coefficient of consolidation of 2 to 30 times due to an increase of effective stresses from 50 to 300 kPa. They claimed that the coefficient of consolidation as calculated by using logarithmic curve were more reliable in settlement analysis.

Beng et al (1982) reported the consolidation characteristics of soft soil which was treated with preloading and sand drains of 400 mm diameter arranged in a triangular grid with spacings ranging from 2.5 m to 3.0 m. The average depth of installation was about 18 metres. According to them the coefficient of consolidation decreased with increasing effective pressure. Their field data indicated that the ratio of coefficients of radial and vertical consolidation varied from 2 to 5.

2.4.4 Consolidation by Rice Husk Filled Drains

Mofiz (1989) investigated the prospect of using rice husk in vertical drains for the improvement of a very soft soil of Bangladesh. He carried out experimental investigation on model consolidation molds of diameters varying from 152 mm to 254 mm. Different dimensions of vertical drains were used. However, his investigation was limited only to a single intensity of preloading of 220-440 kN/m^2. He concluded that the consolidating soil using rice husk filled drains yield a higher shear strength and quicker consolidation than that using a sand filled drain of comparable size. Typical results of his investigation are shown in Figs. 2.5 and 2.6. He introduced the term shear strength improvement ratio as the ratio of undrained shear strength after consolidation to the undrained shear strength at the workable consistency.

2.5 SHEAR STRENGTH OF SOIL

Early tests to determine c and ϕ parameter of soil were made by Navier* (1833), Leygue* (1885) and Frontard* (1914). The first systematic study on the shear strength of clays was that by Bell in 1915. He found that the angle of internal friction was small for clays with a relatively low strength. The concept of pore pressure was introduced by Terzaghi (1920). The first pore pressure measurements inside a clay specimen were made by Rendulic in 1933 (Skempton, 1960). Terzaghi (1920, 1921, 1925) investigated the nature of friction and the physical properties of clays. Terzaghi (1936) was the first to explain why the apparent angle of internal friction of saturated clays under undrained conditions is equal to zero. Many other researchers worked on different aspects of shear strength of soil. A review of the subject can be found in Brand (1981).

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* Cited by Brand(1981)

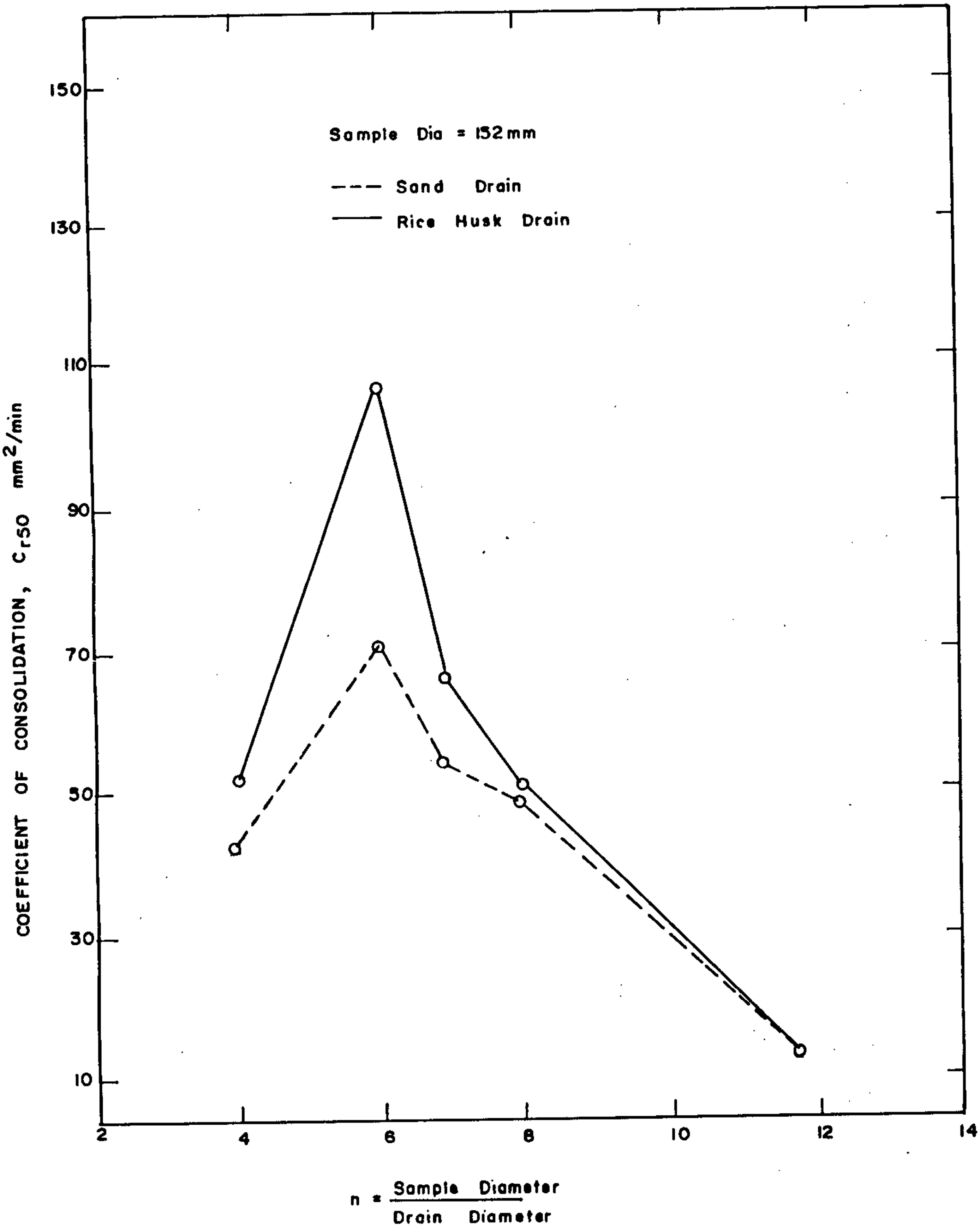

Fig. 2.5 Coefficient fo consolidation (C_r) against n
(After Mofiz, 1989).

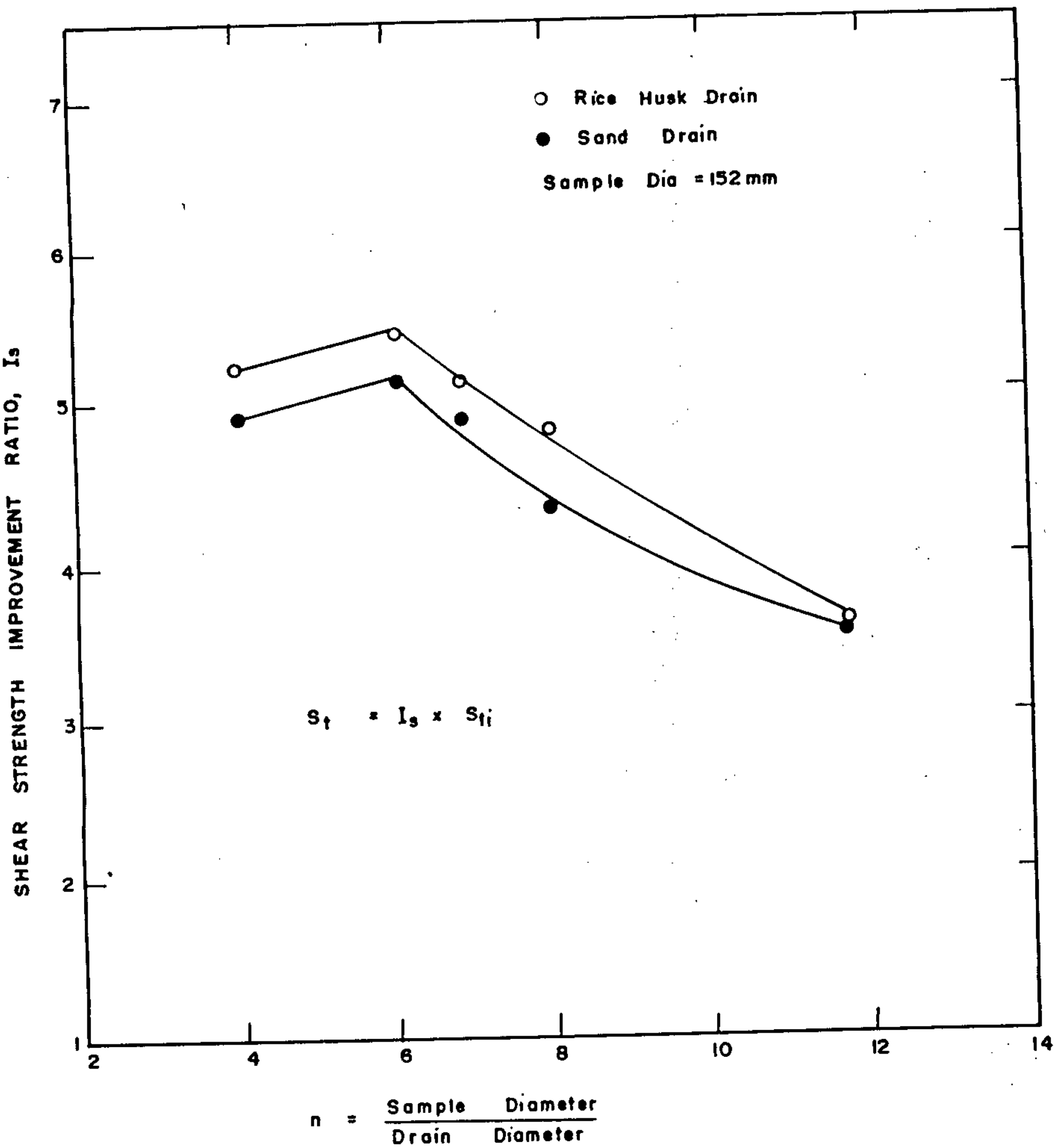

Fig. 2.6 Variation of shear strength improvement ratio with n. (After Mofiz, 1989).

2.6 IMPROVEMENT OF SHEAR STRENGTH DUE TO CONSOLIDATION USING VERTICAL DRAIN

A very limited amount of work are reported in the literature on the improvement of shear strength of soil when vertical drains are used for improvement purposes. Wolski et al (1979) worked on the improvement of soft clay layers and reported that the parameters mainly responsible for the improvement of shear strength of soil were the coefficients of consolidation of soil in the radial and vertical directions. Though these parameters are dependent on the spacing and the dimensions of drains, they did not, however, present any correlations between shear strength and drain dimensions.

Tan et al (1982) reported their work on the effectiveness of non displacement sand drains at Changi airport and concluded that non displaced sand drains were effective in improving the shear strength of soft clay. According to them the increase in shear strength was constant with depth, with values ranging from 30 kN/m^2 to 40 kN/m^2.

In his experimental investigation with rice husk filled drains, Mofiz (1989) claimed that the remolded soil of soft consistency from a selected location of Dhaka city can be improved approximately 10% higher while compared to a sand filled drains of comparable dimension, Fig. 2.6.

2.7 DRAINAGE MATERIALS

To be consistent with economy and availability of drainage materials, no specially prepared graded sand as applied in the construction of vertical drains. The only requirement for drainage materials to be used in vertical drains is that it should carry away the pore water unobstructed, and not to permit the fine grains of the soil to be washed in. The permeability requirements are that the drainage materials should be approximately 1000 times more permeable than that of the consolidating soil (Jumikis, 1962). Jumikis (1962) suggested that not more than 5% of the drainage materials which passes through No. 100 sieve (ASTM) should be allowed.

However, the California Department of Highways, according to Stanton (1948), has used the following typical specimens on several of their vertical sand drain projects.

Sieve size	Percent passing
12.70 mm	90 - 100
2.40 mm	25 - 100
0.60 mm	5 - 50
0.30 mm	0 - 20
0.15 mm	0 - 3

2.8 SOIL IMPROVEMENT PRACTICE IN BANGLADESH

The reported literature on the soil improvement practice in Bangladesh is very few. Shahidullah (1983) reports that soil improvement by using sand drains in Bangladesh is practiced in around 1967. Sand drains were also used under the raft foundation of ten storied Krishi Bhavan at Dilkhusha Commercial area, Dhaka (Mofiz, 1989). However, no comments were reported about the performance of those soil improvement practices.

Islam (1989) reports the use of sand drains under the foundation of two office building at the head quarters of Sunamganj district. He concluded that the increase in shear strength of soil decreases with depth. He also claimed that the settlement of soil layer as calculated using Terzaghi's (1943) formula and laboratory consolidation test data is conservative.

As mentioned earlier, Mofiz (1989) used rice husk drain in his laboratory investigation on models and claimed a higher and quicker gain in shear strength when compared to a sand filled drains. He found important statistical relation for the gain in shear strength interms of drain and sample dimensions which could be used in design. However, he recommends to use it in the field only after the extensive investigation using various conditions of consolidation pressure and soil consistency. Abedin et al (1990) reports the further work on this subject and concluded with the similar remarks of Mofiz (1989). Dastidar (1990) is using sand wicks in vertical drain for the improvement of foundation soil of a housing project in Goran area of Dhaka city. The results of his investigation is yet to be reported.

2.9 CONCLUSIONS

The foregoing literature review on the improvement of soft soil using vertical drain and preloading points to the conclusion that:

i) Sand drain is the most primitive and relatively effective method of soil improvement technique.

ii) An extensive theoretical and experimental work had been performed on this soil improvement method. However, a very limited amount of work in this area is reported in the context of Bangladesh soil.

iii) The use of alternative granular drainage material, other than sand in vertical drain is very few.

It is understood that the extensive investigation on the use of alternative granular material in vertical drainage could suggest a more economical and effective foundation design on soft soil of Bangladesh. And it can be considered worthwhile to study the prospect of using rice husk in vertical drains. Particular research emphasis is required on the effect of load intensity on shear strength improvement.

CHAPTER 3

EXPERIMENTAL SETUP

3.1 GENERAL

The present investigation is an experimental study on shear strength and deformation behaviour of a soil consolidated by using vertical drains. The test scheme includes construction/modification of existing experimental setup as used by Mofiz (1989), collection and preparation of soil samples and drainage materials. In the following articles the experimental setup and materials used in the investigation are briefly described.

3.2 THE EXPERIMENTAL SETUP

The experimental setup consists of consolidation molds, loading frames, mixing apparatus, conventional triaxial machine and deformation measuring gauge. Brief description of the apparatus is given in the following section.

3.2.1 The Molds

The consolidation molds used in the present investigation were modified forms of Rowe cell and similar to that used by Bashar

(1984) and Mofiz (1989). Three consolidation molds of diameter 254 mm, 203 mm and 152 mm, Fig. 3.1, were used. The cylindrical body of mold is made of mild steel having a height of 220 mm and thickness of 7 mm. The base was also from mild steel and bolted to the flanges on the mold at seven position. A rubber gasket was used to provide the seal at the base. Porous plates, also made of mild steel, having pore diameters of 1.58 mm and thickness 7 mm, and filter papers were used at the top and bottom, Fig. 3.1, to facilitate drainage.

3.2.2 The Loading Frame

Three strain control loading frames having different capacities, were used for the application of load. The frames were designed by California state of Highway Department. The loading frames are manually operated. The loading frames 1 and 2, were provided with proving ring and strain dial gauge to record the load and deformation respectively. The loading frame 3, however, was the conventional ASTM consolidation loading frame in which load can be measured from the scale reading of the lever arm. The schematic diagram of a typical loading frame is shown in Fig. 3.2.

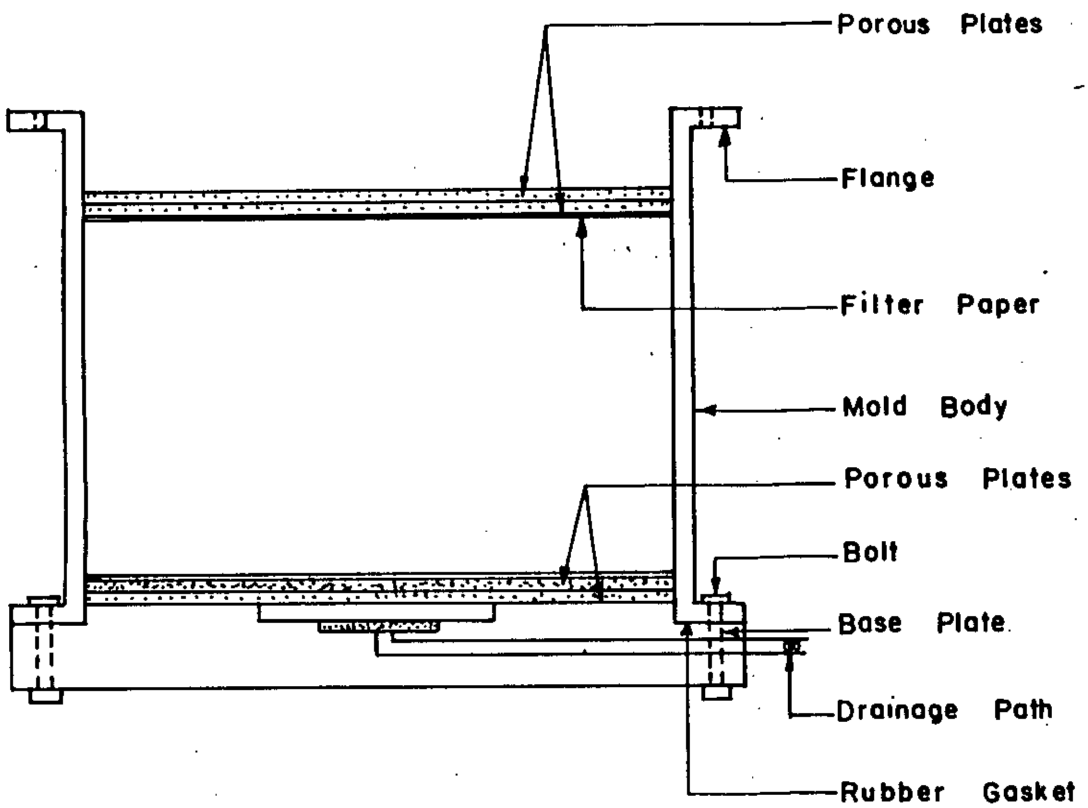

Fig. 3.1 The consolidation mold.

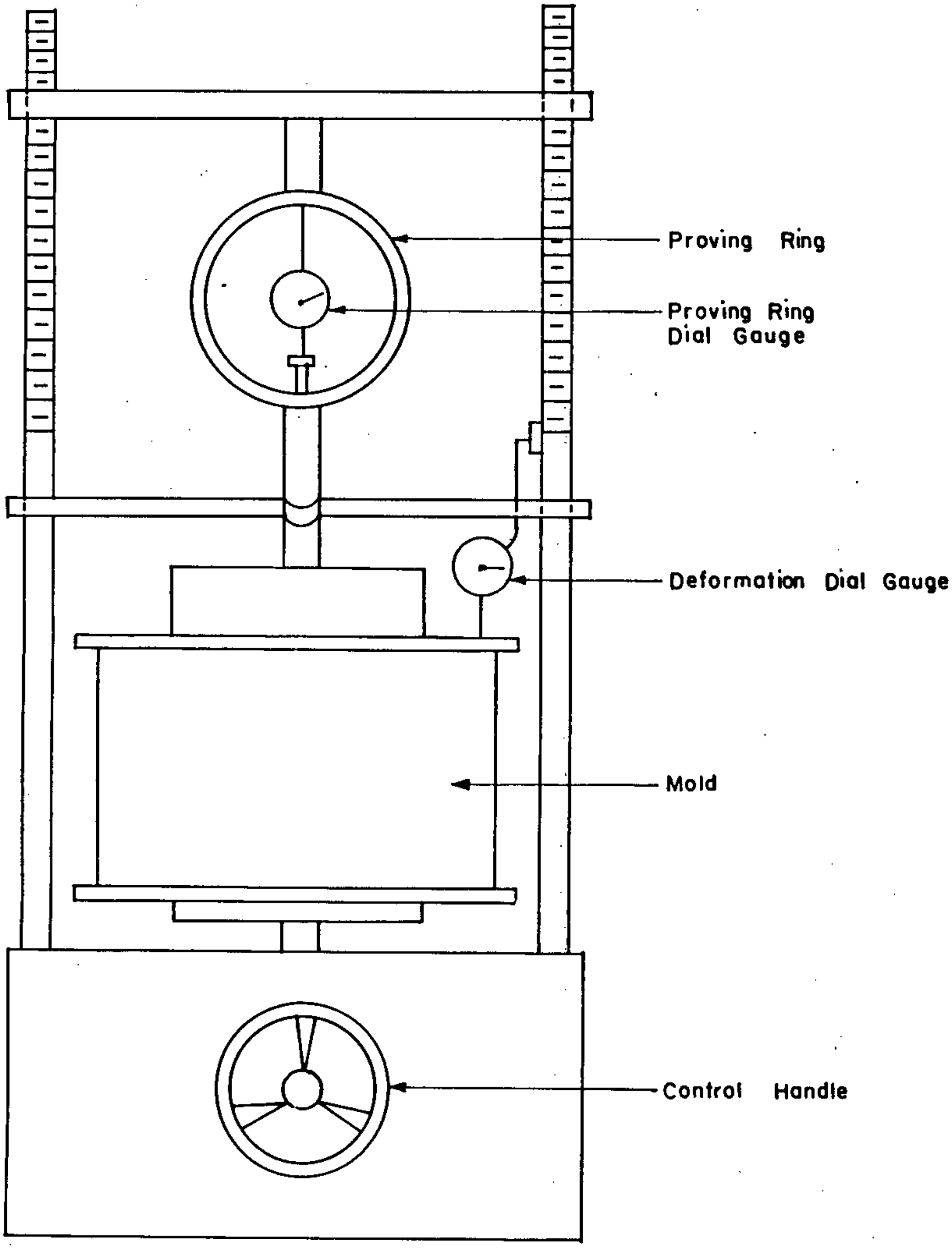

Fig. 3.2 The loading frame.

3.2.3 Drain Installing Apparatus

The loading frames were also used for installation of vertical drains using brass pipes.

3.2.4 Sample Preparation Apparatus

Soil preparation apparatus consists of grinding and the mixing apparatus, supplied by ELE (Engineering Laboratory Equipments).

The grinding apparatus

The grinding apparatus consists of a tank of dimension 17.3 cm x 31 cm (height x diameter), two grinding wheels of diameter 15.2 cm. The apparatus is operated by a motor of rating 230 V, 1/3 H.P., 2.5 ampre, 50 c/s. The apparatus has an opening at the lower end of the tank which can be controlled by a lever arm. The grinding capacity of the machine is 0.01 m^3/operation.

The mixing apparatus

The mixer machine is designed to ensure thorough mixing of the ingredients over a short period of time at the required temperature and/or moisture content.The mixer machine, of dimension 738 mm x 406 mm x 489 mm, has a three-speed gear box driven by a fully enclosed and ventilated motor. The shift handle is mechanically interlocked with the switch, giving a definite gear location and making it necessary to switch off the motor before changing gear. A shock absorber is provided in the main drive gear and the beater shaft is carried on ball bearings. The bowl locks at the top and bottom of the lift travel, which is controlled by convenient hand lever.

3.3 THE SOIL AND THE DRAINAGE MATERIALS

The properties of soil and the drainage materials used in the present investigation are briefly described in the following articles.

3.3.1 The Soil

The disturbed soft clay sample was collected from Kakrail of Dhaka City. The physical and index properties as determined by ASTM standard tests are furnished in Table 3.1.

3.3.2 The Drainage Materials

Two types of drainage materials were used in the present investigation, rice husk and sand.

The sand

The sand used in the present investigation for drainage purposes is a local sand. It was graded by using sieve analysis in order to fulfill the requirements, of drainage material, as suggested by Stanton (1948) and Jumikis (1948). The grading and grain size distribution of the sand used are presented in Table 3.2 and Fig. 3.3 respectively. The index and other properties were determined using ASTM (1979) standard test methods for soils and the results are presented in Table 3.1.

Table 3.1: Properties of Soil and Drainage Materials

Materials	Sp.Gr.	Unit Weight kN/m^3	Water Content (%)	Liquid Limit (%)	Pl. Limit	Permeability (cm/sec.)
Soil	2.64	13.12	35	46	22.80	-
Sand	2.68	15.30	-	-	-	3.98×10^{-2}
Rice Husk	1.65	5.30	-	-	-	5.58×10^{-2}

The rice husk

The rice husk used in the vertical drains was obtained from the locally available paddy 'Roghusal'. The grading and the method followed to obtain the desired gradation were similar to those as used for the sand. The grain size distribution diagram is shown in Fig. 3.3. The physical, index and other related properties were reported by Mofiz (1989). Chemical tests on rice husk by Mehta (1979) indicate that it is constituted of approximately 20% of opaline silica.

Table 3.2 Grading of drainage materials (sand and rice husk)

Sieve No.	Grain size mm	Percent finer
4	4.76	100
8	2.40	100
16	1.20	95
30	0.60	50
40	0.42	30
50	0.30	10
100	0.15	.05

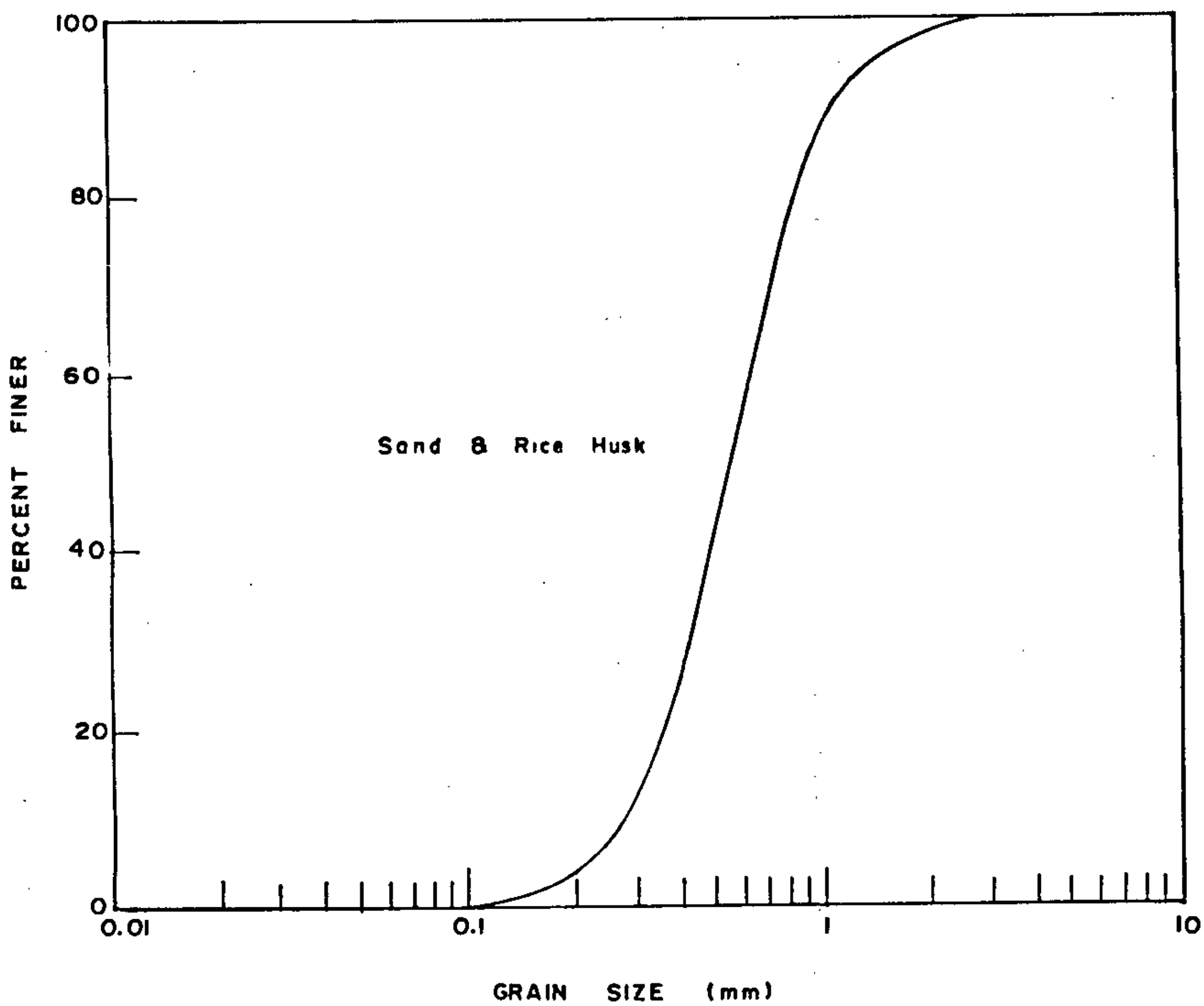

Fig. 3.3 Grain size distribution of rice husk and sand.

CHAPTER 4

TEST PROGRAMME AND PROCEDURES

4.1 TEST PROGRAMME

The present study is carried out in two categories of test conditions ; rice husk filled drains and sand filled drains using three different sample diameters. A total of 48 numbers of consolidation tests using different sizes of mold and 48 undrained triaxial tests (4 specimen of each) were performed. Test programmes are outlined in Table 4.1. Of these consolidation tests 6 preparatory tests were performed at a small pressure increment of 0 to 55 kN/m^2, as suggested by Singh & Hattab (1979) to have the workable consistency of the soil. 36 tests were performed using vertical drain with two pressure increments of 55 - 110 kN/m^2 and 110 - 220 kN/m^2. The rest 6 tests were performed without drains using two pressure increments for different sample dimension. Six triaxial tests were done on initial condition and 21 triaxial tests were carried out for pressure increments of 55-110 kN/m^2 and 21 tests for pressure increment of 110-220 kN/m^2. Tests results for load intensity 220-440 kN/m^2 were taken from Mofiz(1989).

Table 4.1 Consolidation test programme

Drain diameter mm	Consolidation pressure increment kN/m²	Consolidation test number(s)		
		Sample diameter 152 mm	Sample diameter 203 mm	Sample diameter 254 mm
No drain	0 - 55	1,2	3,4	5,6
No drain	55 - 110	7	8	9
No drain	110 - 220	10	11	12
Rice husk filled drain				
38	55 - 110	13	14	15
	110 - 220	16	17	18
25	55 - 110	19	20	21
	110 - 220	22	23	24
19	55 - 110	25	26	27
	28 - 220	28	29	30
Sand filled drain				
38	55 - 110	31	32	33
	110 - 220	34	35	36
25	55 - 110	37	38	39
	110 - 220	40	41	42
19	55 - 110	43	44	45
	110 - 220	46	47	48

4.2 TEST PROCEDURE

The test procedure consists of sample preparation, installation of vertical drains, loading the sample for consolidation, and triaxial tests. The procedure followed was similar to that of Mofiz (1989). In the following sections the test procedure is described.

4.2.1 Sample Preparation

In order to determine the shear strength and deformation characteristics due to of the consolidationinstallation of vertical drains, the experimental programme required a number of identical samples. This was achieved by using artificially prepared specimens in the saturated remolded state. Shield and Rowe (1965) suggested that the identical specimen should have uniform density and moisture content. According to them for the preparation of remolded samples, soil should be mixed with water content near to liquid limit. Berry (1977), however, suggested this value to be twice the liquid limit.

About 25 kg of soil passing through ASTM No. 200 sieve were taken for the preparation of sample. In order to ensure full saturation the soil was taken in a large container, mixed with water at approximately 1.5 times the liquid limit and kneaded by hand to form a slurry. The product was then uniformly mixed by using rotary laboratory mixer for about 30 minutes.

The consolidation mold was bolted to its base. Two porous discs were placed at the bottom of the mold and a brass wire net was placed on them. The inner side of the mold was coated with a thin layer of silicon grease to minimize side friction during consolidation (Bashar, 1984). Two filter papers were placed upon the wire net and the soil slurry was poured into the mold. The slurry was then leveled off. Filter papers and a brass wire net were placed at the top of the slurry. To minimize tilting of the porous disc, another disc of 7 mm thick followed by a spacer block was placed on the previous discs. A clearance of 1.58 mm in between the porous discs and mold body was provided to eliminate side friction (Bashar, 1984).

The soil sample thus prepared was then subjected to a pressure of 14 kN/m^2, using loading frame, which was gradually increased at a slow rate to 55 kN/m^2 for consolidation. After consolidation the slurry was found to form a workable consistency and the thickness of the sample was maintained at approximately 125 mm. Three to four days were required for this consolidation. The sample was then found ready for drain installation.

It should be mentioned that, for each of the three sample diameters, four samples thus prepared were used in order ascertain the uniformity in density and water content throughout the samples. The soil samples were extruded from the mold by using hydraulic jack and specimens were collected from at least three locations of each sample to determine water content and density. It was found that the variations in unit weight and water content were negligible and within the limit of 0.35 to 0.6 percent and 0.85 to 1.20 percent respectively as reported by Mofiz (1989). It is worth mentioning that these samples were also used to determine the initial triaxial strength (12.1 kPa).

4.2.2 Installation of Vertical Drain

The vertical drain was installed at the centre of prepared soil sample in the mold. For the installation of drain, the mold was placed on the platen of loading frame, Fig 3.2. An open circular mandrel of 250 mm length, was pushed into the soil sample to its bottom by fixing the upper end of the mandrel to the bottom of the proving ring and advancing up the platen by revolving the control handle. The mandrel was withdrawn from the sample by revolving the control handle in the reverse direction. A drain hole was thus formed. The hole was then filled with sand or rice husk of desired density. The density was achieved by introducing a known weight of filler material (sand or rice husk) into the hole in four layers and tamping it with a brass rod. It should be pointed out that only a small effort was applied during tamping so that the wall of the hole did not show any significant distortion. The sample thus obtained was ready for consolidation. In the present investigation drains of different diameters of 38 mm, 25 mm, 19 mm and were installed at different stages.

4.2.3 Consolidation of the Sample

As the top drainage platens (two filter papers, brass wire net, porous disc) of the sample were removed during drain installation, fresh platens were placed on the sample to facilitate consolidation. However, this maneuver was not necessary for the sample without drains.

The soil sample with the mold was then placed on the platen of loading frame and necessary spacer block(s) was placed on the top of drainage platen so as to touch the bottom of the proving ring. A dial gauge was also placed on the top of the mold to measure deformations. A pressure of 55 kN/m^2 was applied to reconsolidate the sample to its prior condition of drain installations.

A load of 110 kN/m^2 was applied to two identical samples and deformation dial readings were taken at different intervals until the deformation was found approximately ceased. At end of consolidation the triaxial test was done on one of the samples. The other sample was further consolidated for the load increments of 110 to 220 kN/m^2. Triaxial tests were done also on the later sample.

4.2.4 Triaxial Test

For the determination of initial triaxial shear strength of the soil sample, specimens were collected by using a specially prepared cylindrical brass made soil sampler. The sampler had an inner diameter of 40 mm, tapered on the outside and a length of 77 mm. the average thickness of the wall of the sampler was 2 mm. As the area ratio of the sampler was maintained approximately within 10 percent, the disturbance during sampling should be negligible (Peck, Hanson & Thornburn, 1974). The surfaces of the sampler were smoothen by using fine graded Emery paper. Thin layer of silicon grease was applied on both inner and outer surfaces of the sampler to reduce friction. The sampler was then pushed steadily by hand into the soil in order to trap the soil in the sampler. The soil on the outside the sampler was carefully removed by using a knife and a spatualla. The sampler was withdrawn and both ends were leveled off. A rubber membrane was placed inside a membrane stretcher of 88 mm length and of 44 mm internal diameter and was turned back over the ends. Suction was applied to the membrane stretcher so as to expand the membrane to a convenient size to slip over the sample without touching it. A porous disc and a filter paper were placed at the bottom of the membrane stretcher. The soil sampler was placed on the membrane stretcher and the sample was ejected into the stretcher by using a sample ejector. Suction was released so that the membrane contracts on the sample, and its ends were slipped off the membrane stretcher. A filter paper and a porous disc were placed on the top and the sample was placed on the triaxial machine for testing (unconsolidated undrained test). Four such samples (specimens) were used to determine the triaxial shear strength. Similar test procedures were followed to test the samples for all the load increments. For details of triaxial test, reference can be made to Bishop & Hankel (1962).

CHAPTER 5

TEST RESULTS AND ANALYSIS

5.1 GENERAL

In the present investigation a series of vertical flow and radial flow consolidation tests were performed on clay samples of different diameters and load intensities. Two drainage materials; rice husk and sand were used in vertical drains with different drain dimensions. The study was mainly concerned with the strength and deformation behaviour of the consolidated clay.

The test results are summarized in Tables 5.1 through 5.13. Tables 5.1 to 5.3 show the results of inward flow radial consolidation tests, while Table 5.4 shows the results of vertical flow consolidation tests. In Tables 5.5 through 5.10 results of undrained triaxial tests are presented. Finally the statistical results are presented in Tables 5.11 through 5.13.

Results are presented graphically in Figs. 5.1 through 5.14. The graphical relations include coefficient of consolidation against n, triaxial shear strength against sample and drain dimensions, shear strength improvement ratio against n and load increment.

5.2 CONSOLIDATION CHARACTERISTICS

The coefficient of consolidation (C_{v50}, C_{r50}) were calculated by logarithm of time method as suggested by Wolski et al(1979) using the time and deformation data. The results show that the coefficient of consolidation increases with the increase of pressure. It is apparent from the results that the trend favours the increase of coefficient of consolidation C_r with load increments. This agrees with Singh and Hattab (1979). The results which show the variation of coefficient of consolidation with sample diameter/drain diameter (n) are shown in Fig. 5.1 to 5.3. It indicates that the coefficient of consolidation, in general, has a maximum value at an optimum value of n, n_{opt}. The n_{opt} is found to have a linear variation with sample diameter, D expressed in mm as shown in Fig. 5.4. The variation can be represented statistically as

$$n_{opt} = 0.04D \qquad (5.1)$$

This is found to be independent of load intensity and agrees with the findings of Mofiz(1989). The coefficient of consolidation of consolidating soil is found to have a higher value for the case of a rice husk filled drain comparing to a sand filled drain. The values of coefficient of consolidation of soil for both sand and rice husk filled drains are presented in Tables 5.1 to 5.4.

5.3 EFFECT OF SAMPLE DIAMETERS SHEAR STRENGTH

Figure 5.5 shows the variation of shear strength (at 100% consolidation) with sample diameter when consolidated without drains. Initial condition mentioned on the figures represents the preparatory consolidated sample (consolidated at 55 kN/m^2). It is seen that, the shear strengths at different load intensity remain almost constant with sample diameter.

Figs. 5.6 and 5.7 shows the variation of shear strength with sample diameters for different load intensity using different drainage materials. The experimental results indicate that for a particular drain diameter the shear strength decreases linearly with increasing sample diameter. However, the variation is found to be very small. Test results are also shown in Tables 5.5 and 5.6.

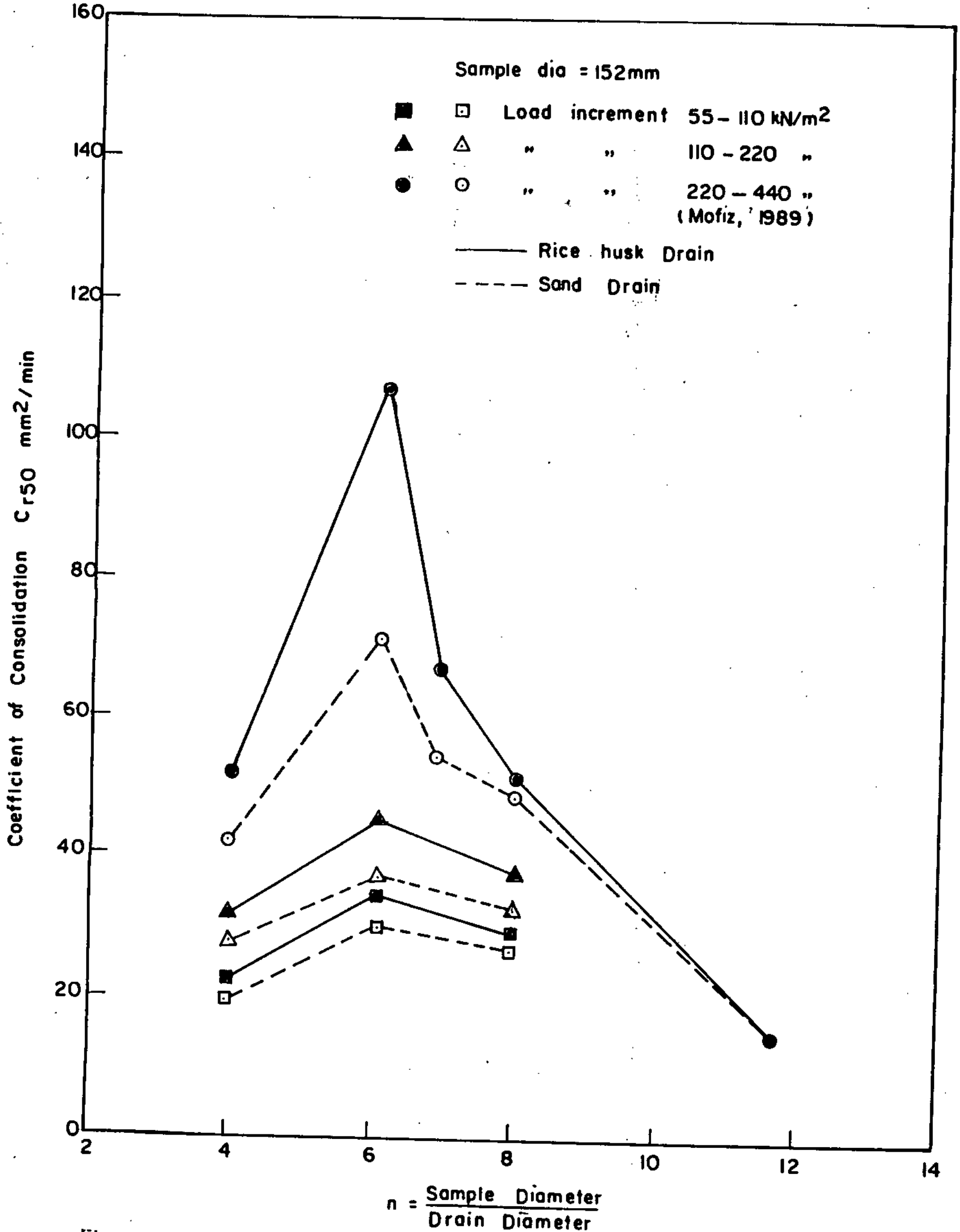

Fig. 5.1 Variation of coefficient of consolidation (C_r) with drain influence factor (n).

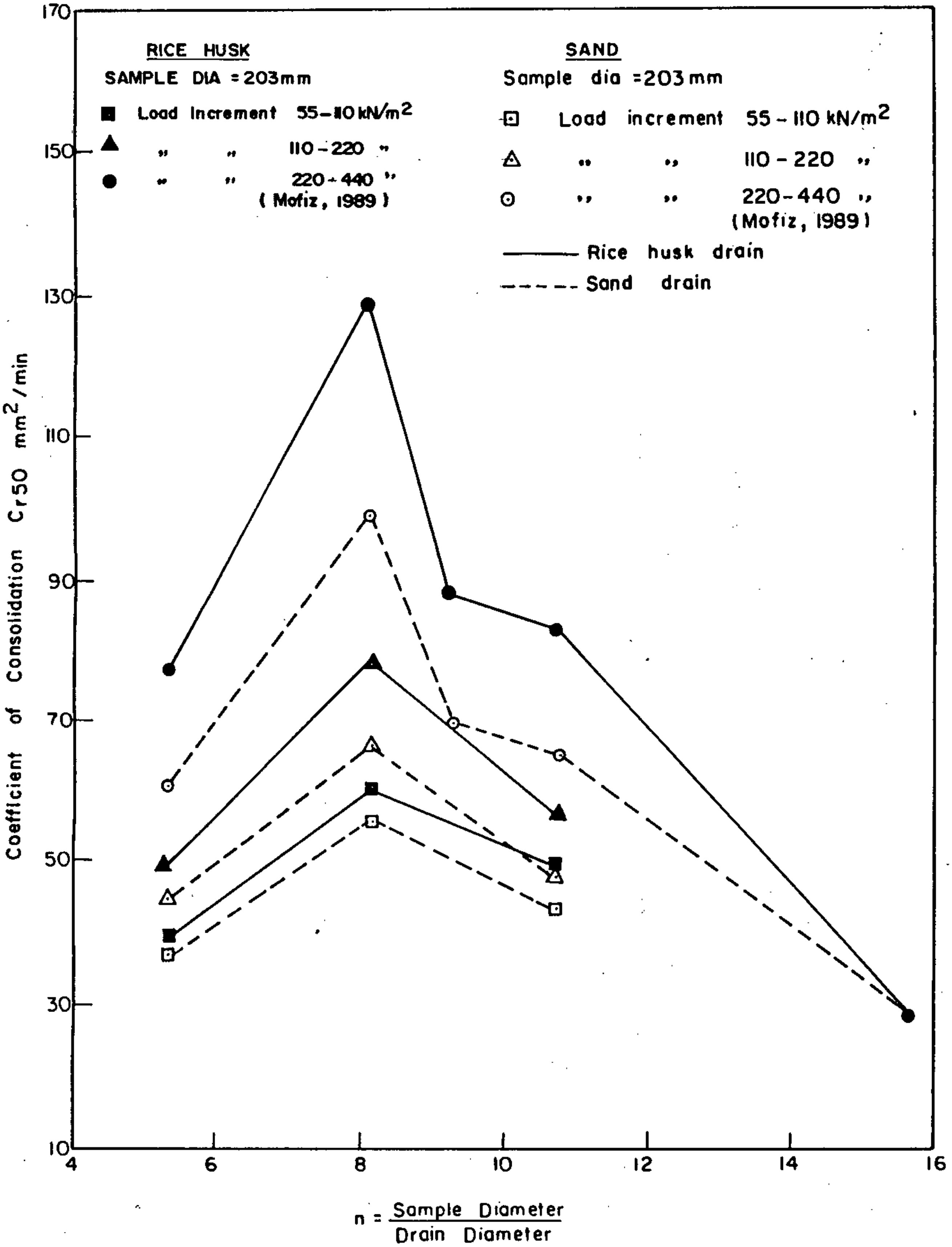

Fig. 5.2 Variation of coefficient of consolidation (C_r) with drain influence Factor (n).

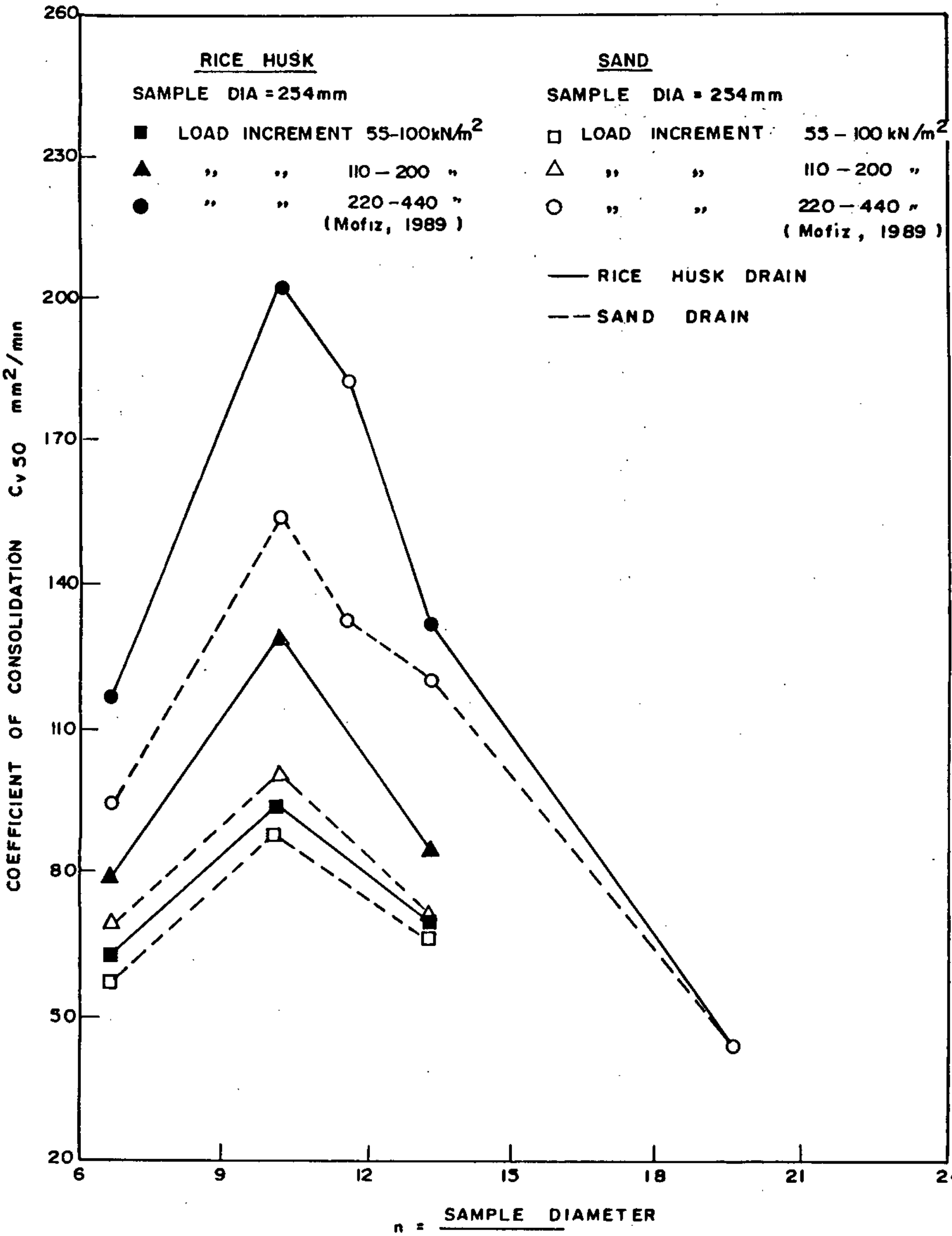

Fig. 5.3 Variation of coefficient of consolidation (C_r) with drain influence factor (n).

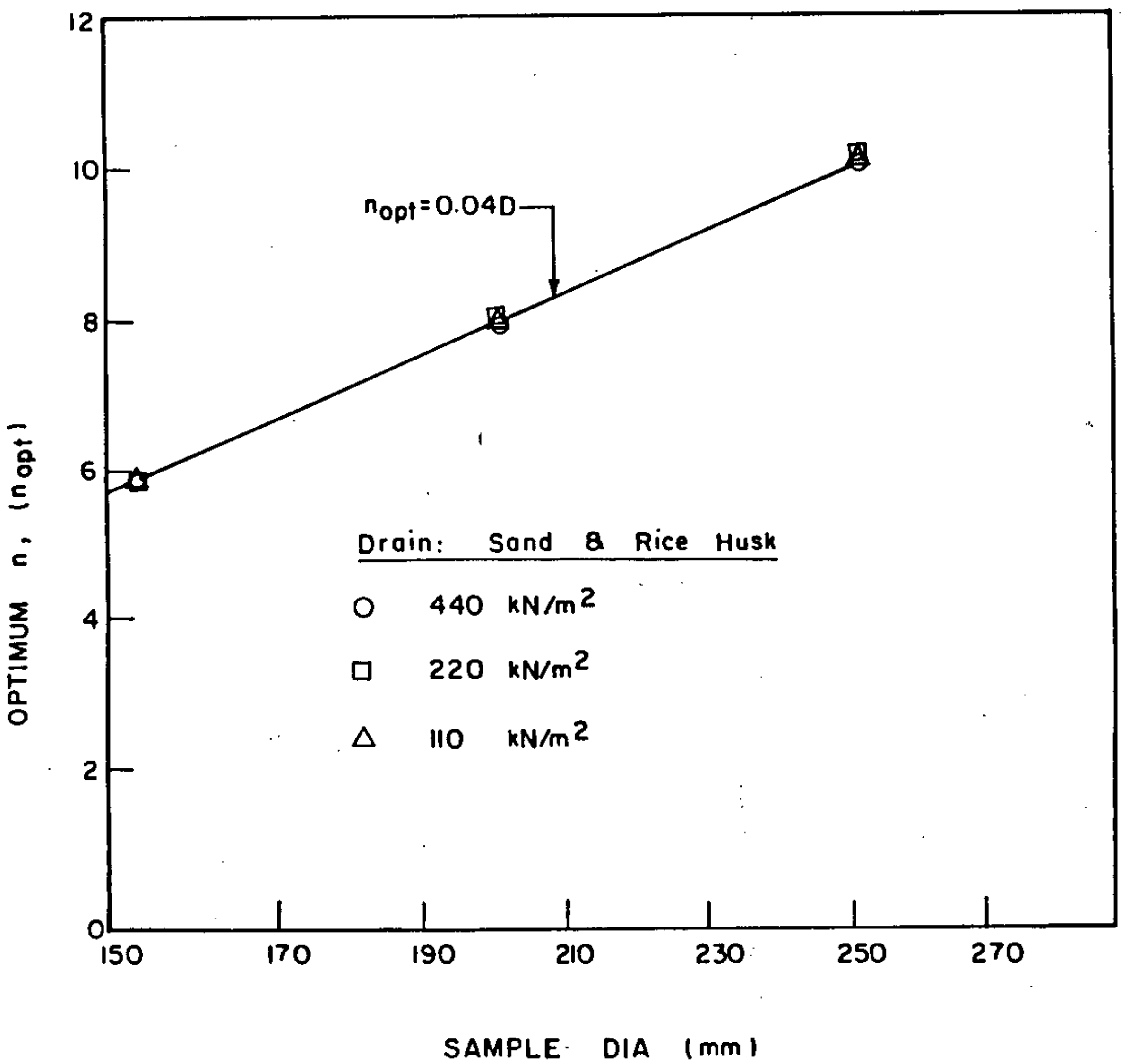

Fig. 5.4 Variation of n_{opt} with sample diameter (D).

Table 5.1 Inward radial flow consolidation tests, 152 mm diameter sample

Drain dia. d_w	n= d_e/d_w	$\Delta p/p$	Initial sample thickness	Final sample thickness	Load increment	Time to 50% consolidation from settlement measurement	Coefficient of consolidation C_{r50}
mm			mm	mm	kN/m²	min	mm²/min
Rice husk filled drain							
38	4	1	127.00		55-110	75	22.49
			117.89	107.28	110-220	53	31.82
25	6.08	1	126.75		55-110	62	34.66
			116.79	106.06	110-220	47	45.72
19	8	1	128.00		55-110	92	28.88
			119.07	109.09	110-220	70	37.96
Sand filled drain							
38	4	1	127.50		55-110	87	19.39
			118.77	108.35	110-220	62	27.20
25	6.08	1	127.25		55-110	72	29.84
			118.02	107.38	110-220	58	37.05
19	8	1	127.50		55-110	100	26.57
			118.67	108.95	110-220	82	32.40

Table 5.2 Inward radial flow consolidation tests, 203 mm diameter sample

Drain dia. d_w mm	$n = d_e/d_w$	$\Delta p/p$	Initial sample thickness mm	Final sample thickness mm	Load increment kN/m^2	Time to 50% consolidation from settlement measurement min	Coefficient of consolidation C_{r50} mm^2/min
Rice husk filled drain							
38	5.34	1	126.95		55-110	90	38.92
			118.46	108.25	110-220	71	49.33
25	8.12	1	127.80		55-110	79	60.51
			118.91	108.34	110-220	61	78.36
19	10.68	1	127.20		55-110	119	48.83
			119.34	110.37	110-220	101	57.53
Sand filled drain							
38	5.34	1	128.00		55-110	97	36.11
			119.69	109.89	110-220	79	44.34
25	8.12	1	127.50		55-110	85	56.24
			118.72	108.55	110-220	72	66.39
19	10.68	1	127.90		55-110	135	43.04
			120.26	111.40	110-220	121	48.02

Table 5.3 Inward radial flow consolidation tests, 254 mm diameter sample

Drain dia. d_w mm	n= d_e/d_w	$\Delta p/p$	Initial sample thickness mm	Final sample thickness mm	Load increment kN/m^2	Time to 50% consolidation from settlement measurement min	Coefficient of consolidation C_{r50} mm^2/min
				Rice husk filled drain			
38	6.68	1	128.25		55-110	104	62.03
			120.01	110.30	110-220	82	78.68
25	10.16	1	127.10		55-110	95	93.72
			118.65	108.63	110-220	69	129.03
19	13.36	1	128.20		55-110	146	70.26
			120.68	112.09	110-220	123	83.40
				Sand filled drain			
38	6.68	1	128.00		55-110	115	56.10
			119.91	110.60	110-220	93	69.37
25	10.16	1	127.50		55-110	103	86.44
			119.21	109.23	110-220	88	101.17
19	13.36	1	127.90		55-110	161	63.71
			120.44	112.02	110-220	142	72.24

Table 5.4 Vertical flow consolidation tests

Sample dia.	$\Delta p/p$	Initial sample thickness mm	Final sample thickness mm	Load increment kN/m²	Time to 50% consolidation from settlement measurement min	Coefficient of consolidation C_{v50} mm²/min
254	1	127.75		55-110	625	5.14
		120.54	112.63	110-220	420	6.32
203	1	128.00		55-110	590	5.47
		120.88	112.67	110-220	440	6.93
152	1	127.50		55-110	450	7.12
		119.39	110.42	110-220	395	7.60

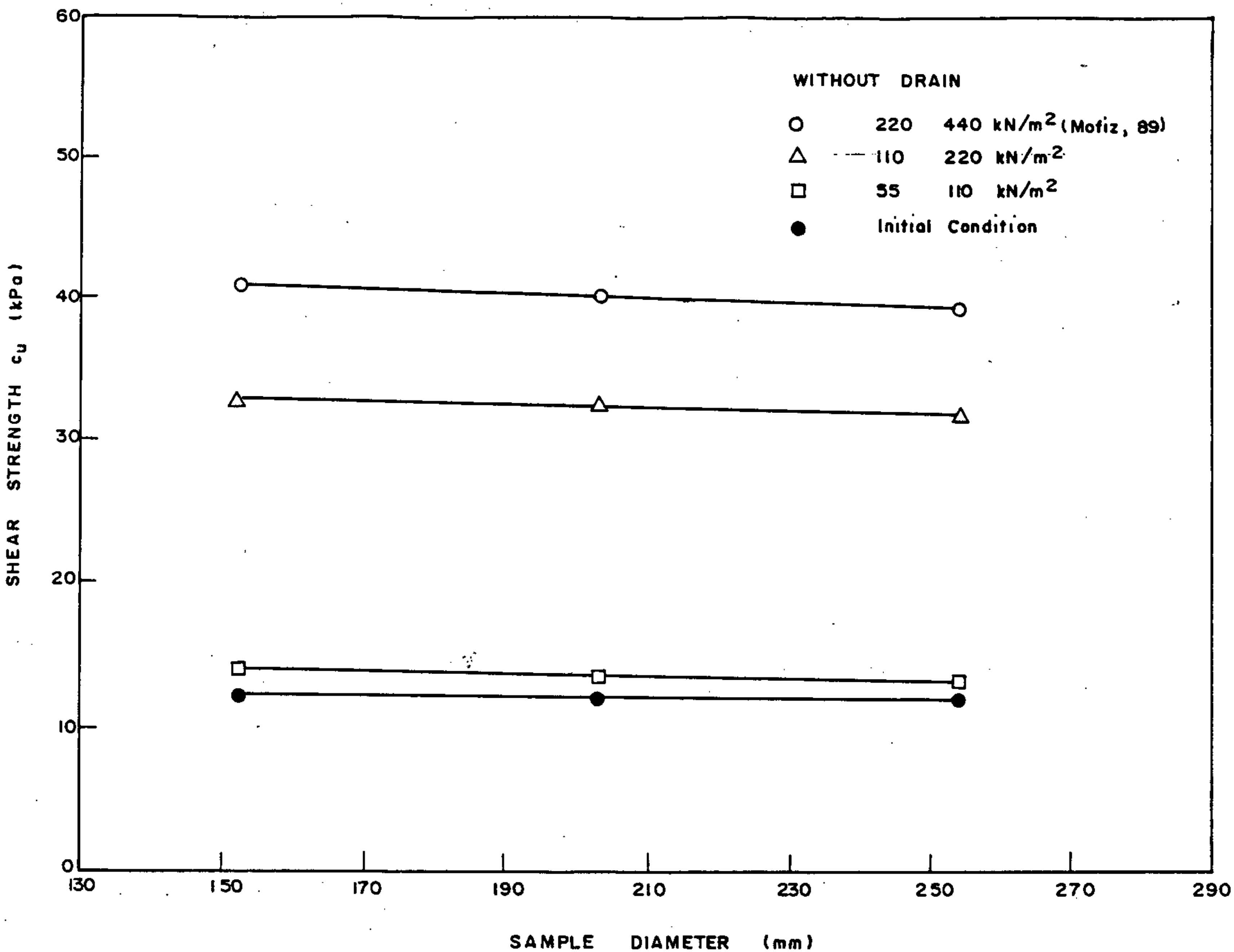

Fig. 5.5 Variation of shear strength with sample diameter.

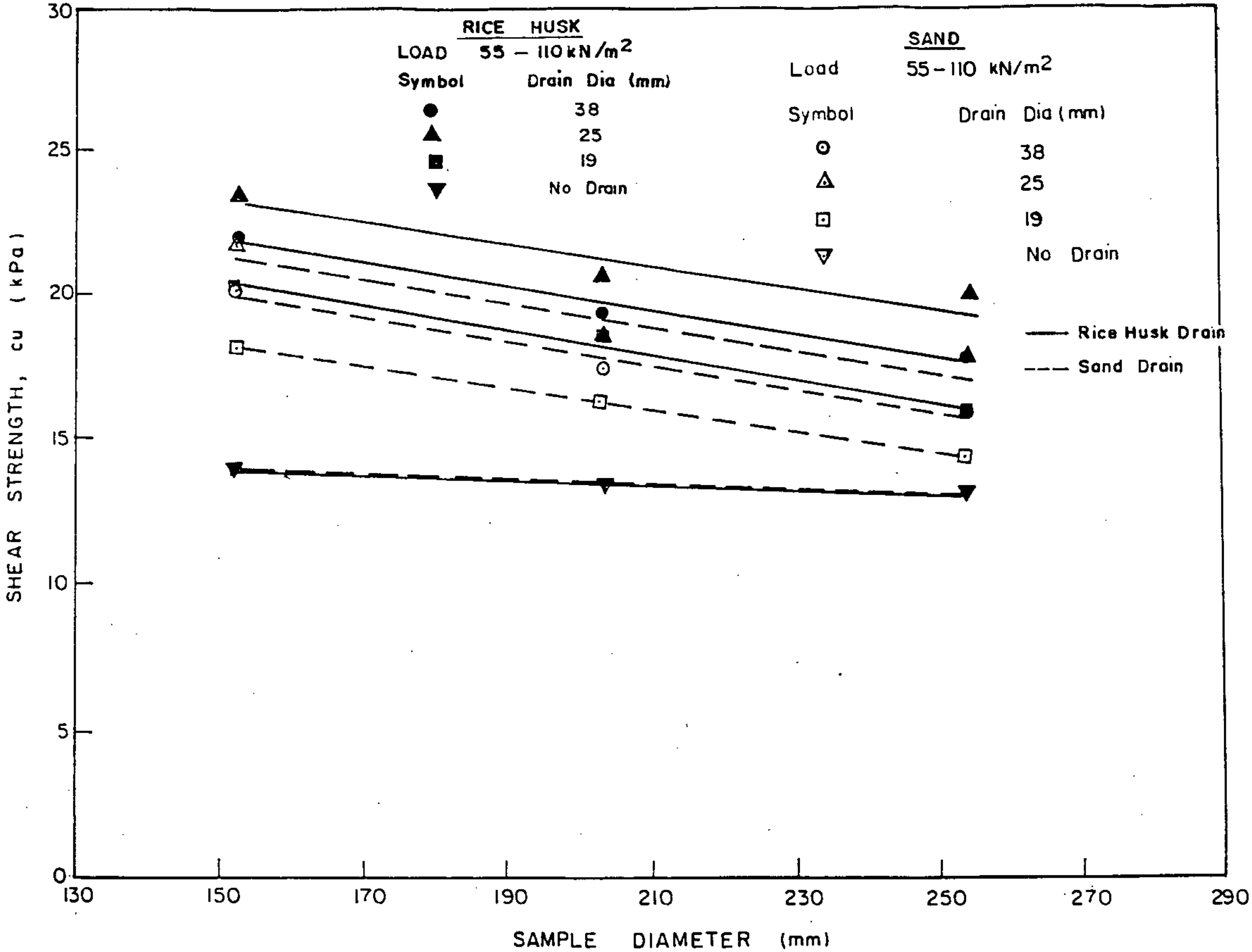

Fig. 5.6 Variation of shear strength with sample diameter.

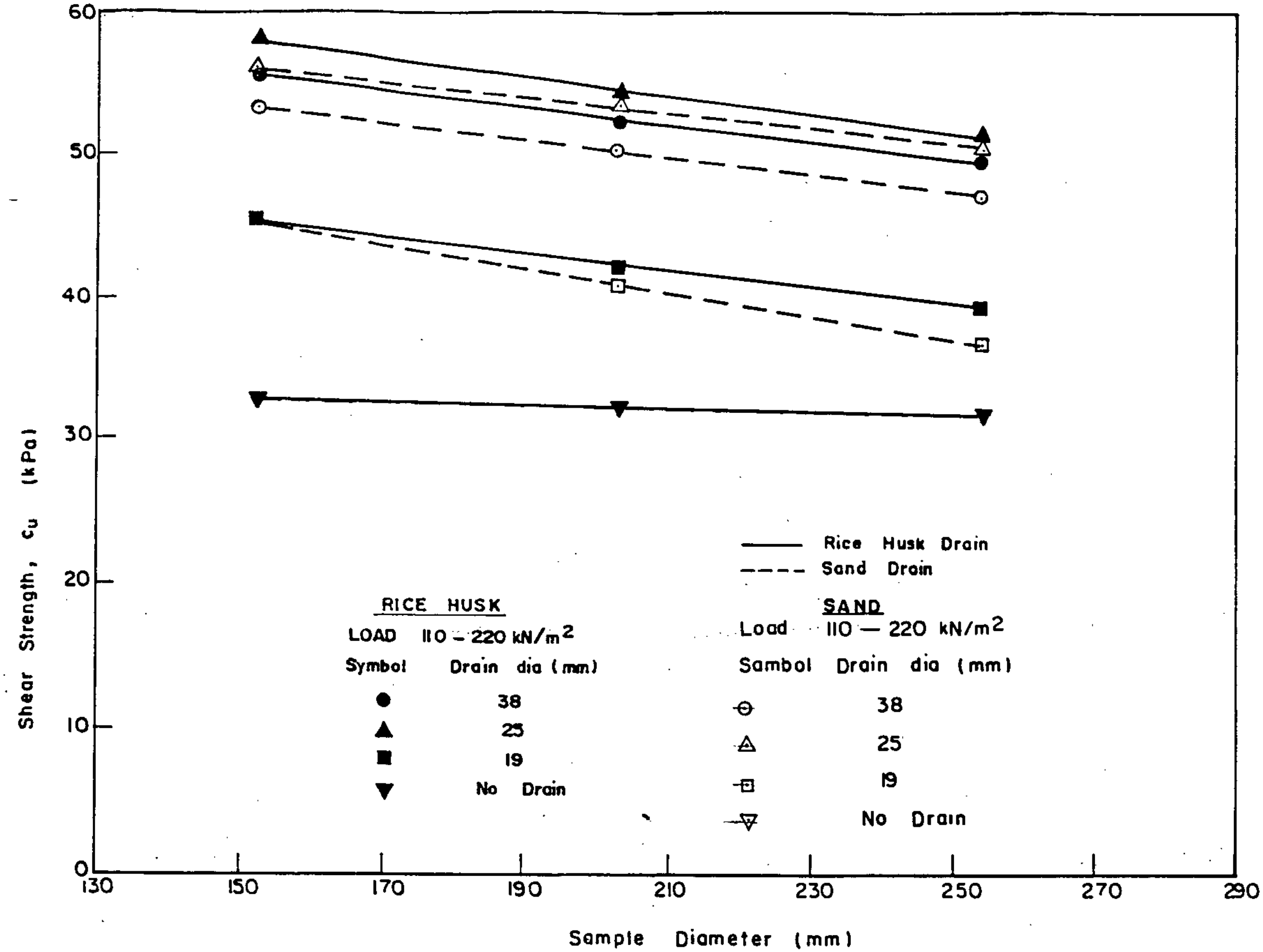

Fig. 5.7 Variation of shear strength with sample diameter.

Table 5.5 Shear strength after consolidation
Load increment: 110-220 kN/m²

Drain dia. d_w	Shear strength after consolidation (using drains) kPa			Shear strength after consolidation (with no drain) kPa			Shear strength at initial condition, kPa		
Rice husk filled drain									
mm	S_{tr254} *	S_{tr203}	S_{tr152}	S_{tw254}	S_{tw203}	S_{tw152}	S_{ti254}	S_{ti203}	S_{ti152}
38	49.64	52.41	55.58						
25	51.23	54.05	58.02	31.46	32.12	32.85	12.00	12.02	12.28
19	39.42	42.08	45.58						
Sand filled drain									
	S_{ts254}	S_{ts203}	S_{ts152}	S_{tw254}	S_{tw203}	S_{tw152}	S_{ti254}	S_{ti203}	S_{ti152}
38	47.37	50.17	53.10						
25	50.49	53.19	55.97	31.46	32.12	32.85	12.03	12.06	12.30
19	36.89	40.80	45.03						

* Note: S_{tr254} = Shear strength of soil using rice husk drain for a sample diameter of 254 mm.

Table 5.6 Shear strength after consolidation
Load increment: 55-110 kN/m²

Drain dia. d_w	Shear strength after consolidation (using drains) kPa			Shear strength after consolidation (with no drain) kPa			Shear strength at initial condition, kPa		
	Rice husk filled drain								
mm	S_{tr254}	S_{tr203}	S_{tr152}	S_{tw254}	S_{tw203}	S_{tw152}	S_{ti254}	S_{ti203}	S_{ti152}
38	17.98	19.51	22.07						
25	20.30	20.79	23.75	13.21	13.54	14.01	12.00	12.02	12.28
19	16.08	18.25	20.48						
	Sand filled drain								
	S_{ts254}	S_{ts203}	S_{ts152}	S_{tw254}	S_{tw203}	S_{tw152}	S_{ti254}	S_{ti203}	S_{ti152}
38	15.93	17.38	20.18						
25	17.90	18.68	21.68	13.21	13.54	14.01	12.03	12.06	12.30
19	14.47	16.38	18.13						

5.4 EFFECT OF DRAIN DIMENSION ON SHEAR STRENGTH

The experimental results, Figs. 5.8 to 5.11, show variation of shear strength with drain diameters. The shear strength of soil sample increases with increasing drain diameter, reaches its peak at particular diameter (optimum diameter) and then decreases with increasing drain diameter. The reasoning may be that upto the optimum drain diameter the soil gets sufficient drainage area in the radial direction and at the same time the deformation behaviour of clay and the granular drain column may be consistent. This may result in a higher consolidation and hence the shear strength. Whereas, beyond optimum drain diameter the granular drain column might exhibit a higher stiffness thus obstructing the consolidation process and resulting no improvement in shear strength (Mofiz, 1989).

5.5 EFFECT OF LOAD INTENSITY ON SHEAR STRENGTH

The variation of shear strength interms of shear improvement ratio is shown against preloading at n_{opt} in Fig. 5.12. It is observed that, the shear strength varies sharply with increasing load upto a preloading of 220 kN/m². The further increase of preloading reduces the rate of gain of shear strength. At lower intensity of loading(upto 220 kN/m²), the

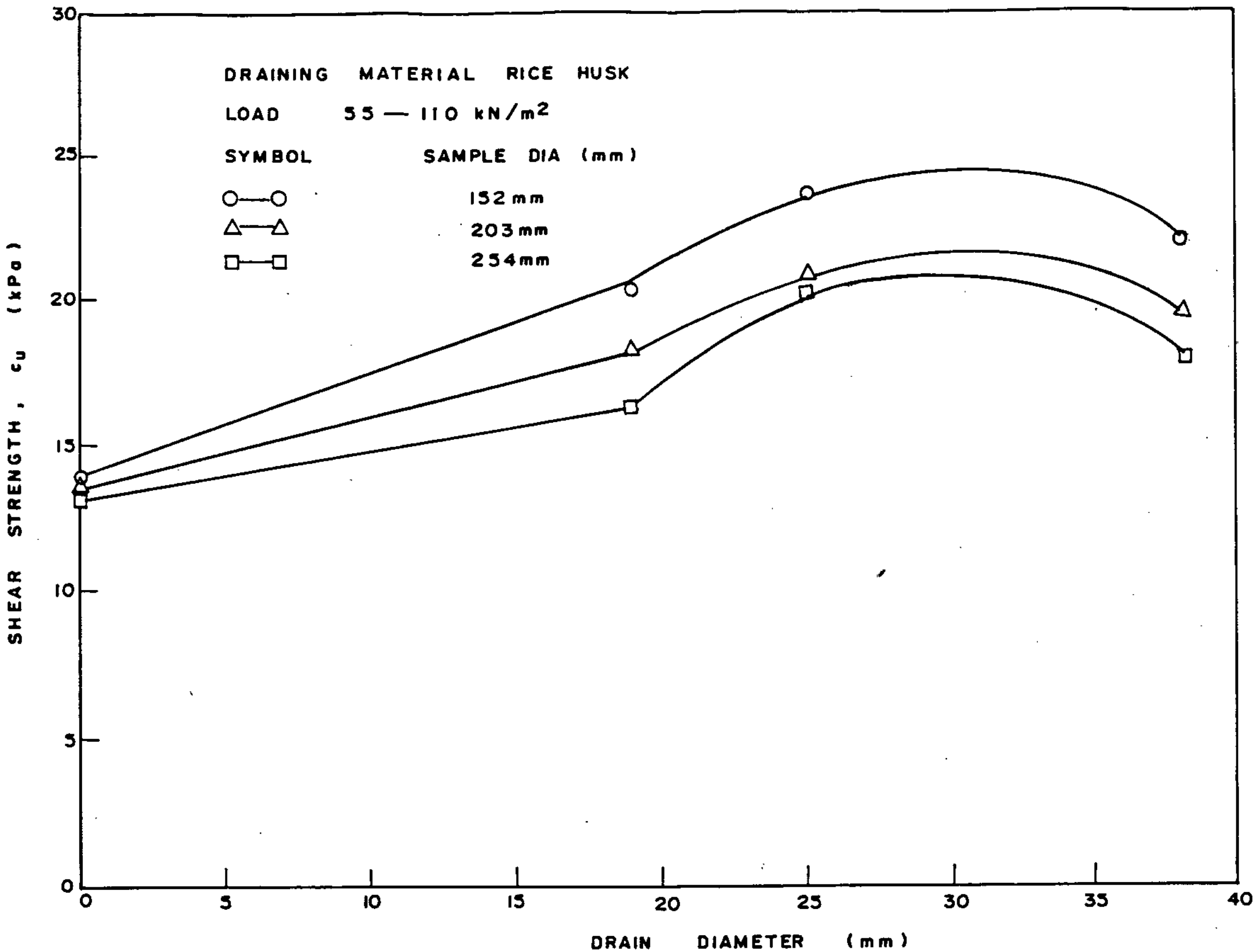

Fig. 5.8 Variation of shear strength with drain diameter.

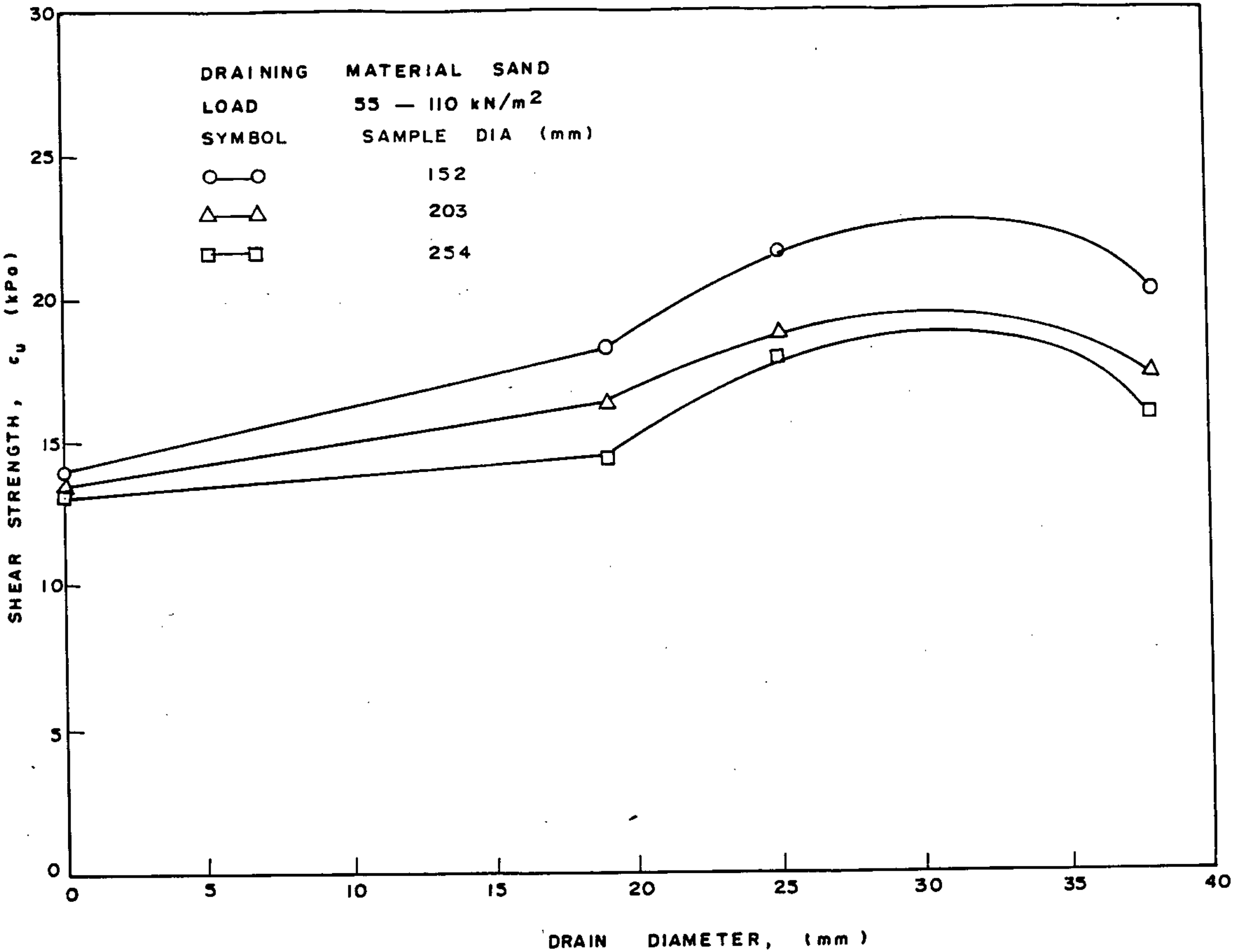

Fig. 5.9 Variation of shear strength with drain diameter.

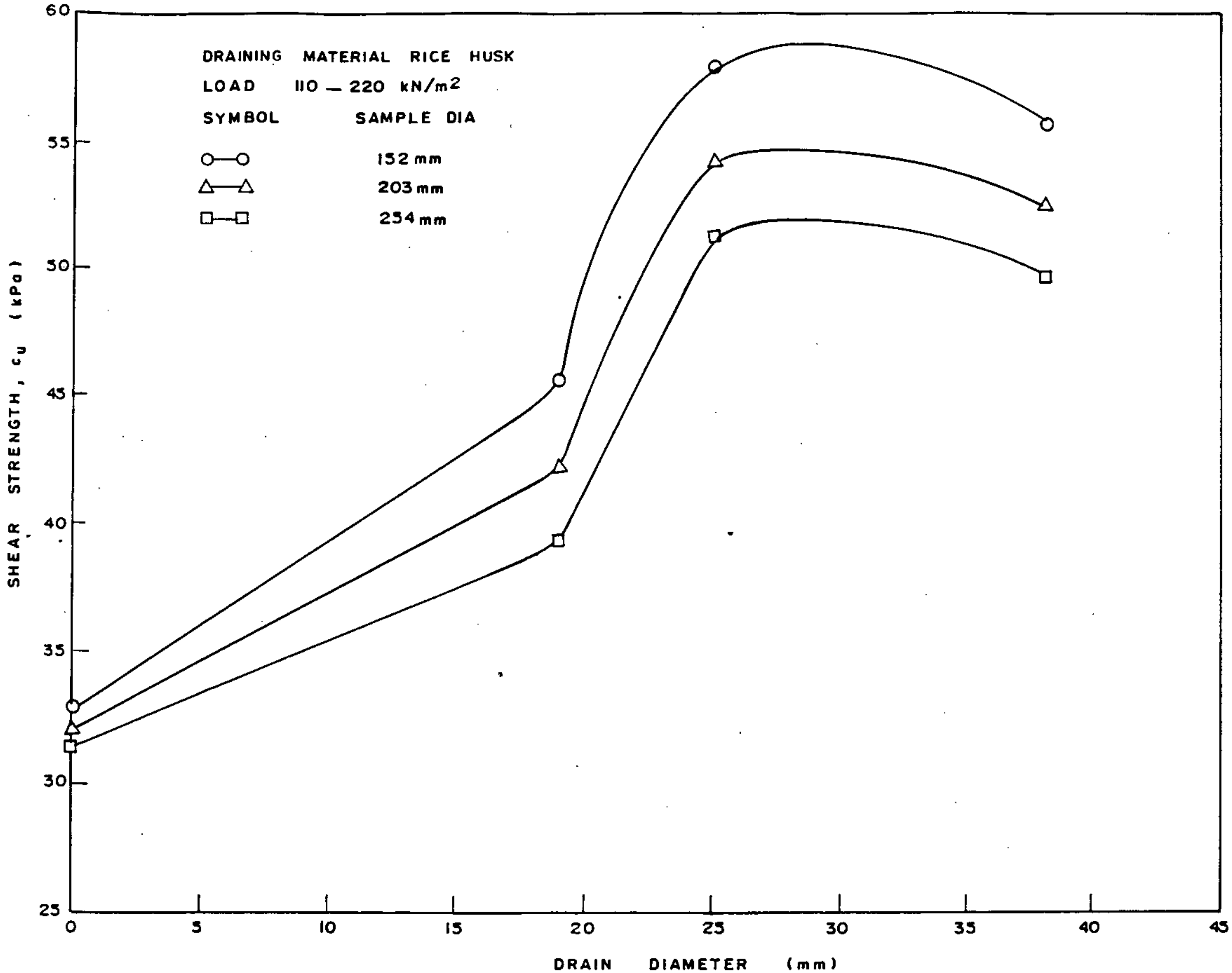

Fig. 5.10 Variation of shear strength with drain diameter.

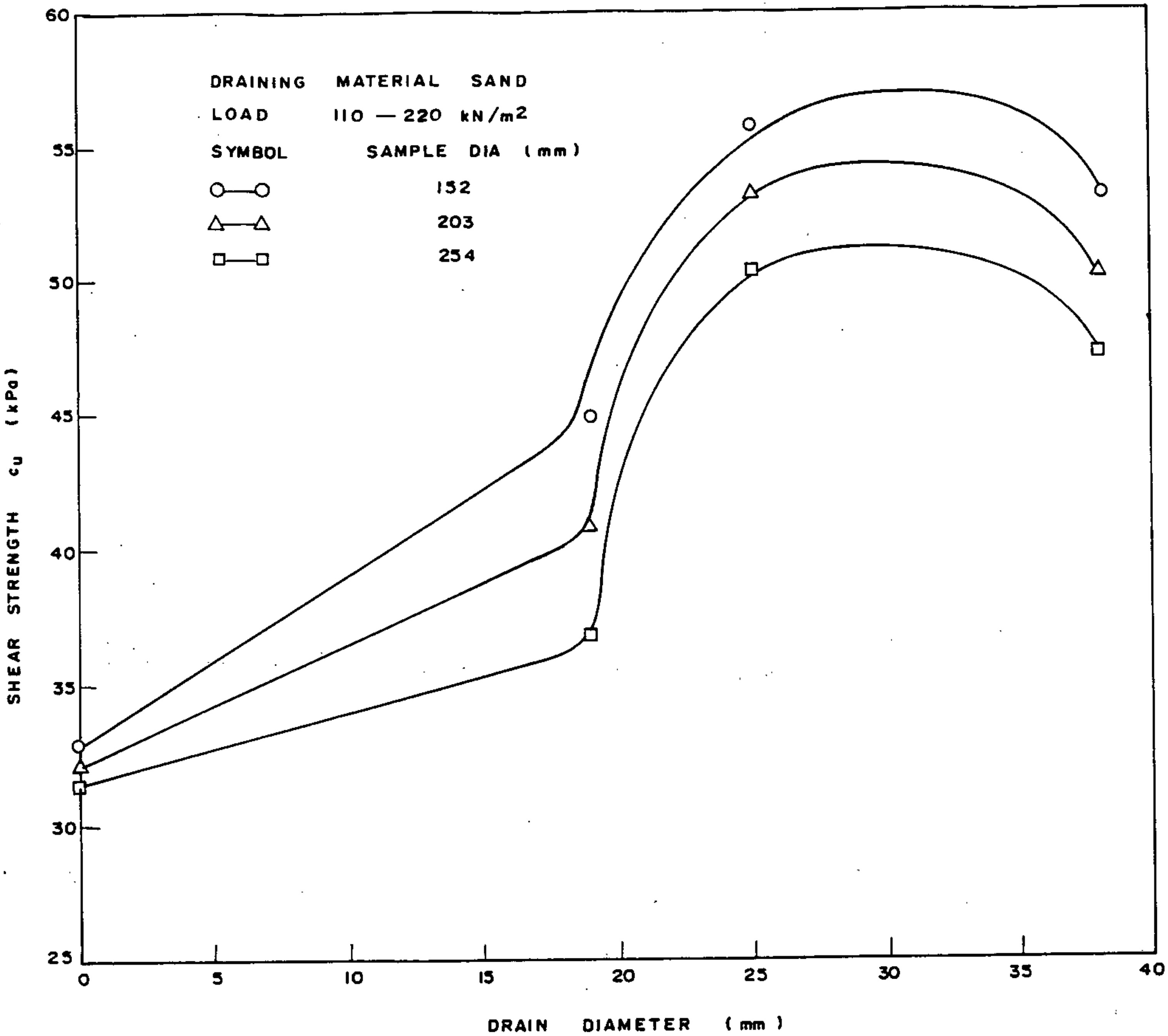

Fig. 5.11 Variation of shear strength with drain diameter.

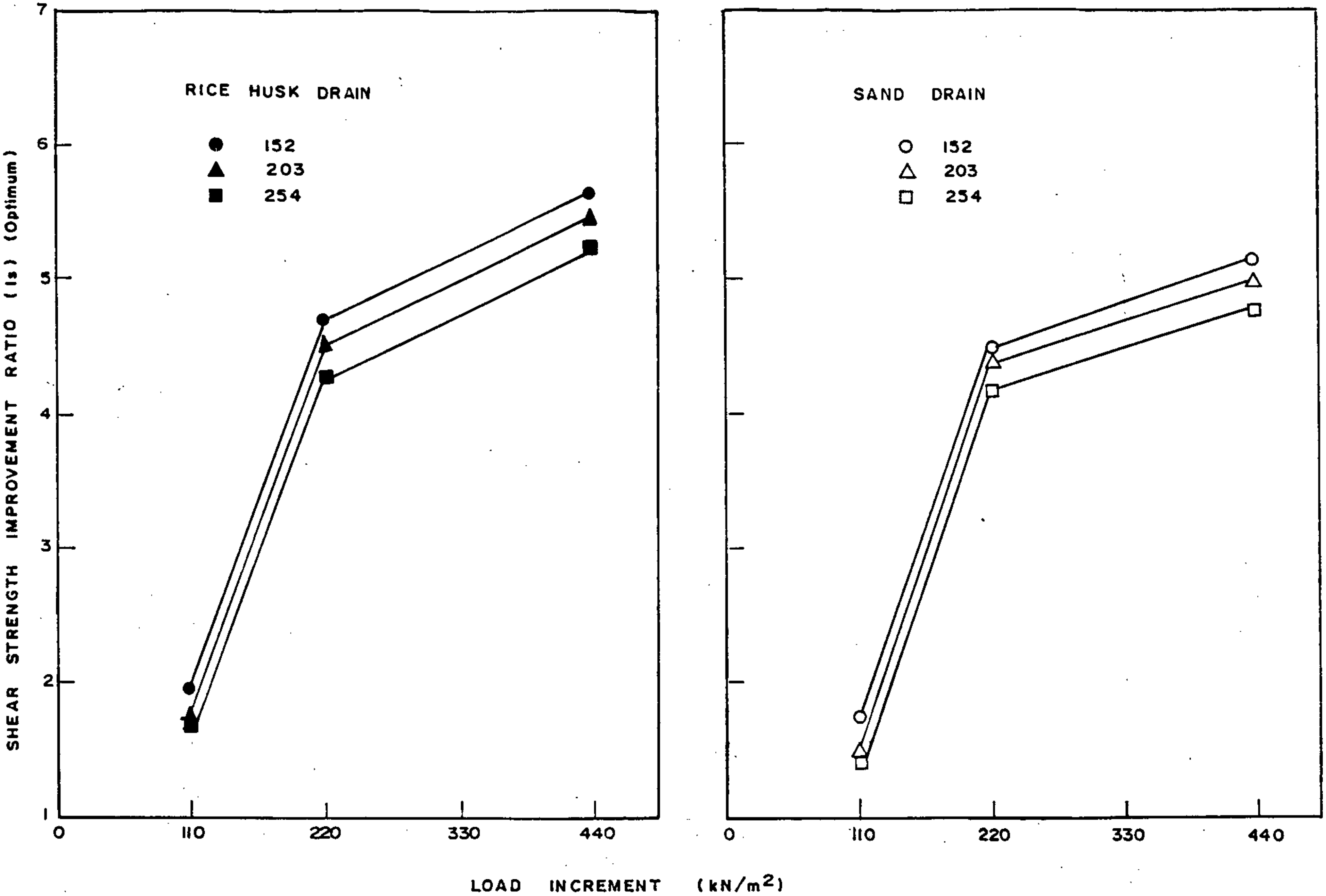

Fig. 5.12 Variation of shear strength improvement ratio (I_s) with load increment.

sample diameter has no remarkable influence on the gain of shear strength. For higher intensity of loading the sample diameter has a significant influence on the gain of shear strength. The experimental results suggest that, there is an optimum gain in shear strength for the design of vertical drains. This should be taken into consideration. For the soft soil of Dhaka city, the optimum loading is found to be approximately 220 kN/m^2.

5.6 SHEAR STRENGTH IMPROVEMENT

The shear strength parameters, c, was investigated by performing undrained triaxial tests on the consolidated sample. Tables 5.5 to 5.6 show the shear strength of clay before and after consolidation for different drain diameters and drainage materials. The term shear strength improvement ratio is introduced which may be defined as the ratio of shear strengths after and before the consolidation of soil of workable consistency. It is already mentioned elsewhere that workable consistency of the soil was achieved by applying, gradually, an ultimate consolidation pressure of 55 kN/m^2 (Mofiz, 1989). Tables 5.7 through 5.10 present the shear strength improvement ratio.

The experimental results show that shear strength increases

Table 5.7 Shear strength improvement ratio (Rice husk drain)

Load increment: 110-220 kN/m²

Drain dia.	Shear strength improvement ratio after consolidation			Shear strength improvement ratio after consolidation		
d_w	(using drains)			(with no drain)		
mm	I_{sr254}	I_{sr203}	I_{sr152}	I_{sw254}	I_{sw203}	I_{sw152}
38	4.14	4.36	4.52			
25	4.27	4.50	4.72	2.62	2.67	2.68
19	3.28	3.50	3.71			

Table 5.8 Shear strength improvement ratio (Rice husk drain)

Load increment: 55-110 kN/m²

Drain dia.	Shear strength improvement ratio after consolidation			Shear strength improvement ratio after consolidation		
d_w	(using drains)			(with no drain)		
mm	I_{sr254}	I_{sr203}	I_{sr152}	I_{sw254}	I_{sw203}	I_{sw152}
38	1.50	1.62	1.80			
25	1.69	1.73	1.93	1.10	1.13	1.14
19	1.34	1.52	1.67			

Table 5.9 Shear strength improvement ratio (Sand drain)

Load increment: 110-220 kN/m²

Drain dia. d_w mm	Shear strength improvement ratio after consolidation (using drains)			Shear strength improvement ratio after consolidation (with no drain)		
	I_{ss254}	I_{ss203}	I_{ss152}	I_{sw254}	I_{sw203}	I_{sw152}
38	3.95	4.17	4.32			
25	4.21	4.42	4.56	2.61	2.66	2.67
19	3.07	3.39	3.67			

Table 5.10 Shear strength improvement ratio (Sand drain)

Load increment: 55-110 kN/m²

Drain dia. d_w mm	Shear strength improvement ratio after consolidation (using drains)			Shear strength improvement ratio after consolidation (with no drain)		
	I_{ss254}	I_{ss203}	I_{ss152}	I_{sw254}	I_{sw203}	I_{sw152}
38	1.32	1.44	1.64			
25	1.49	1.55	1.76	1.10	1.12	1.14
19	1.20	1.36	1.47			

with increasing n upto n_{opt}, and then decreases with increasing n, Fig 5.13. This observation supports the statement made by McGown and Hughes (1982) and Mofiz (1989) that the larger diameter drain my act as a strengthening column in soft soils thus reducing the amount of consolidation.

It is observed that the gain in shear strength is always higher for the case of a rice husk drain compared to a sand drain. At its peak value the shear strength improvement ratio is approximately 10 percent higher, Fig. 5.13, regardless of the sample diameter and load intensity.

Statistical analysis of the data of shear strength improvement ratio, I_s, and n shows that there is a good correlation between them. The relations are similar for all the load intensities.

For n smaller than n_{opt}, the shear strength has a linear correlation with n in the form

$$I_s = a_o + b_o n \qquad (5.2)$$

For larger values of n, there exists exponential relations in the form

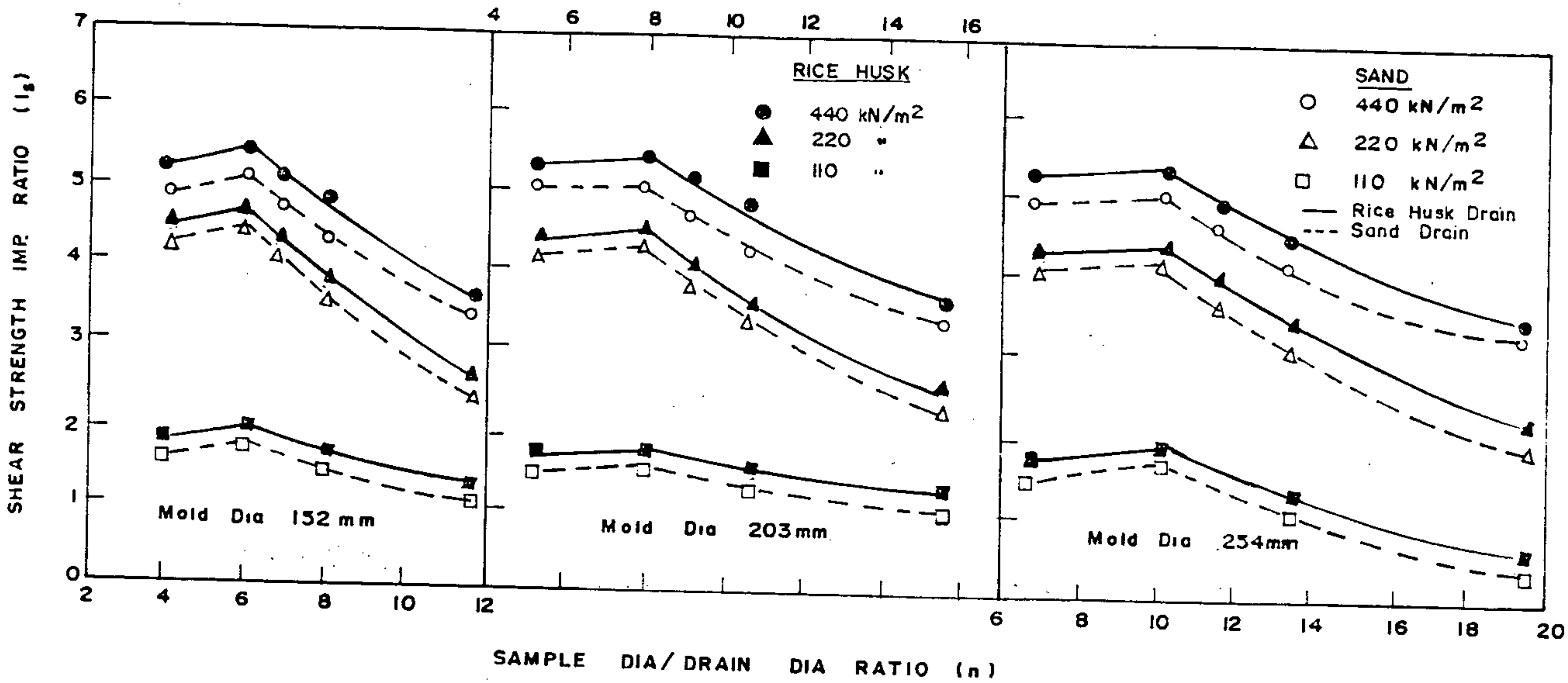

Fig. 5.13 Variation of shear strength improvement ratio (I_s) with drain influence factor (n).

$$I_s = ae^{bn} \qquad (5.3)$$

However, these relations are found to be dependent on sample diameter, load intensity and material properties of drains.

The parameters a_o, b_o, a, b and the coefficient of correlations (r^2) for various conditions are presented in Tables 5.11 and 5.12

It is, however, observed that the maximum shear strength improvement ratio at different load intensity varies approximately linearly with n_{opt}. Their statistical relations are presented in Table 5.13 and Fig. 5.14.

Table 5.11 Statistical Parameters for I_s and n Relations (Drainage Material: Rice Husk, Ref: Eqs.5.2 & 5.3)

Sample Dia(mm)	Load (kN/m^2)	Statistical Parameters				
		a_o	b_o	a	b	r^2
152	55-110	1.534	0.0659	2.971	-0.0697	0.996
	110-220	4.143	0.0957	8.736	-0.1023	0.999
	220-440	4.832	0.1038	8.475	-0.0724	0.999
203	55-110	1.417	0.0385	2.422	-0.0426	0.996
	110-220	4.091	0.0504	8.320	-0.0773	0.998
	220-440	5.167	0.0184	7.884	-0.0485	0.999
254	55-110	1.215	0.0575	6.158	-0.1190	0.998
	110-220	3.883	0.0380	8.918	-0.0735	0.999
	220-440	4.974	0.0184	7.938	-0.0439	0.994

Table 5.12 Statistical Parameters for I_s and n Relations (Drainage Material: Sand, Ref: Eqs.5.2 & 5.3)

Sample Dia(mm)	Load (kN/m^2)	Statistical Parameters				
		a_o	b_o	a	b	r^2
152	55-110	1.303	0.0817	2.956	-0.0852	0.993
	110-220	3.874	0.1125	9.271	-0.1175	0.999
	220-440	4.469	0.1125	7.982	-0.0733	0.998
203	55-110	1.278	0.0396	23482	-0.0526	0.961
	110-220	3.932	0.0454	8.640	-0.0876	0.998
	220-440	4.554	0.0648	7.997	-0.0569	0.999
254	55-110	0.997	0.0629	7.630	-0.1504	0.998
	110-220	3.653	0.0440	9.722	-0.0865	0.999
	220-440	4.538	0.0298	7.245	-0.0418	0.982

Table 5.13 Relation between Maximum I_s amd m_{opt}

Load	Relation and Correlation Coeff. (r^2)			
	Rice Husk Drain		Sand Drain	
	Relation	r^2	Relation	r^2
110	$I_s = 2.09-0.033n_{opt}$	0.43	$I_s = 2.03-0.044n_{opt}$	0.67
220	$I_s = 5.45-0.112n_{opt}$	0.99	$I_s = 5.23-0.112n_{opt}$	0.99
440	$I_s = 5.91-0.074n_{opt}$	0.99	$I_s = 5.65-0.077n_{opt}$	0.91

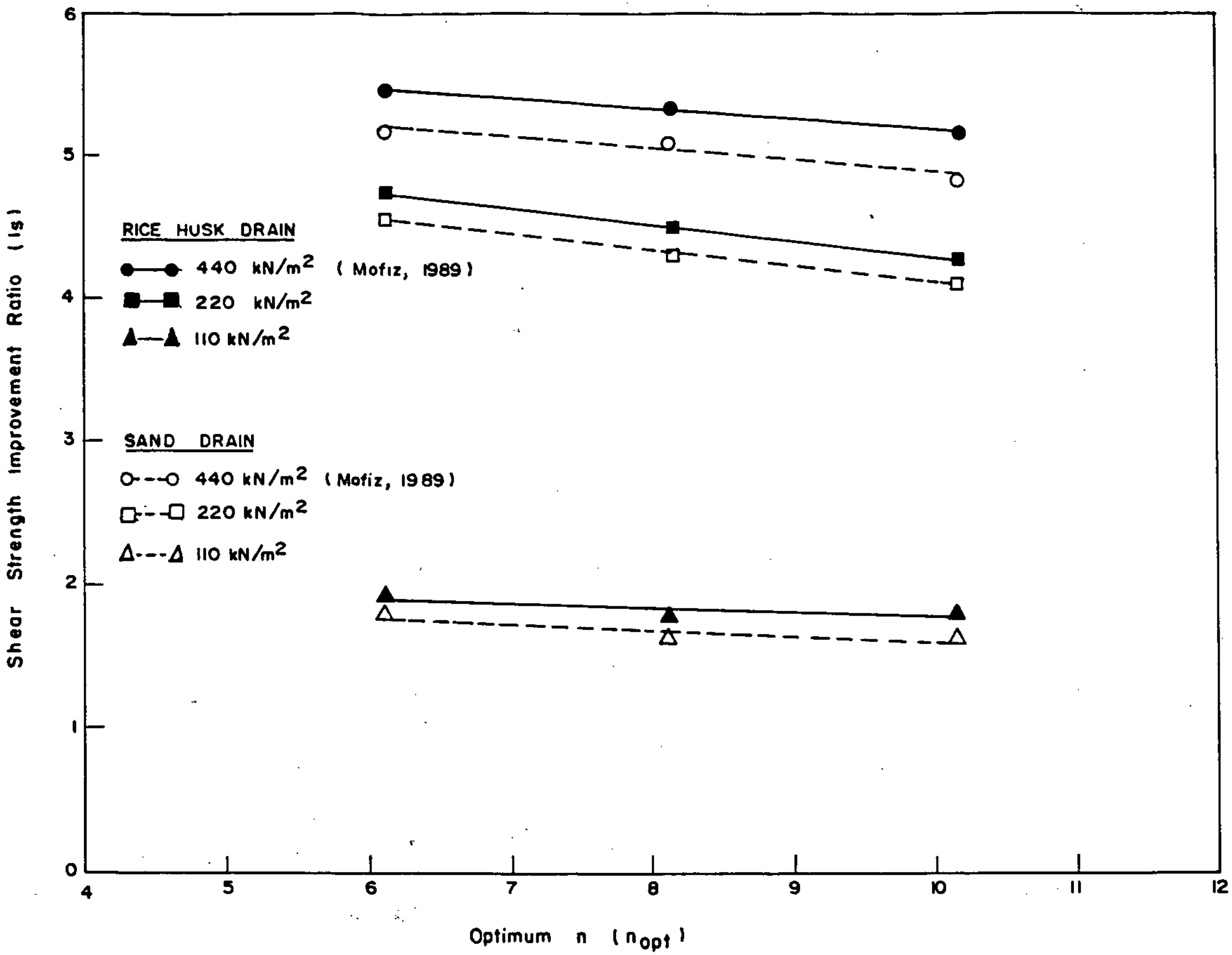

Fig. 5.14 Variation of shear strength improvement ratio (I_s) with optimum drain influence ratio (n_{opt}).

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS FOR FUTURE STUDY

6.1 CONCLUSIONS

The present study is concerned with an experimental investigation of consolidation and shear strength behaviour of a remolded soft soil from a selected location of Dhaka city. The soil was treated for improvement with vertical drains and preloading method. Sand and rice husk were used to fill the vertical drains of different dimensions. Consolidation molds of three different diameter were used in the investigation. The soil was subjected to three different load intensities of 110, 220 and 440 kN/m^2.

Based on the experimental results, the following conclusions can be drawn.

(1) A rice husk filled drain can reduce the consolidation time by up to one third the time required for a sand filled drain of comparable size and dimension.

(2) For a given preloading pressure a rice husk filled drain yields a higher undrained shear strength in the order of approximately 10 percent as compared to a sand filled drain.

(3) The gain in shear strength and drain influence factor has

strong statistical correlations. The relations obtained may be proposed to predict the shear strength gain due to installation of vertical drains and preloading.

(4) Due to installation of vertical drains and imposed loading the shear strength of soil reaches its maximum, at an optimum value of $n(n_{ot})$. Other than n_{ot}, the shear strength has a lower

value. The n_{ot} has a linear relation with the diameter of loaded area. The proposed relation is

$$n_{ot} = 0.04D$$

where, D is expressed in me.

5. When a vertical drain acts for consolidation purposes, the developed shear strength for a smaller value of n ($< n_{ot}$) has a linear relation with n in the form

$$I_s = a_o + b_o n$$

For higher values of n, the relation can be proposed in the form

$$I_s = a\, e^{bn}$$

The constants (parameters a, b) of the relation has a strong correlation with the diameter of the loaded area. The constants were found to be dependent on the materials used in the drains.

6.2 RECOMMENDATIONS FOR FUTURE STUDY

The present study has covered different aspects of strength and deformation behaviour of soft clay consolidated by using rice husk and sand filled vertical drains. However the study was limited to loading, drainage and sample dimensions. It is necessary to extend this study, to generalize the observations, on the following aspects:

1. To study the chemical and time dependent behaviour of draining materials, particularly rice husk.

2. To investigate the above mentioned aspects by using a mixture of rice husk and lime as a draining material.

3. To perform field tests in order to compare the laboratory test results.

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