INVESTIGATIONS ON ACCELERATED CONSOLIDATION OF COIR REINFORCED LATERITE, LITHOMARGIC CLAY AND BLENDED SOILS WITH VERTICAL SAND DRAINS FOR PAVEMENT FOUNDATIONS

Thesis

Submitted in partial fulfillment of the requirements for the degree of

DOCTOR OF PHILOSOPHY

By

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DECEMBER, 2012

NATIONAL INSTITUTE OF TECHNOLOGY KARNATAKA, SURATHKAL

D E C L A R A T I O N

I hereby *declare* that the Research Thesis titled, **Investigations on Accelerated Consolidation of Coir Reinforced Laterite, Lithomargic Clay and Blended Soils with Vertical Sand Drains for Pavement Foundations**, which is being submitted to the **National Institute of Technology Karnataka, Surathkal** in partial fulfillment of the requirements for the award of the degree of **Doctor of Philosophy** in **Civil Engineering** is a *bonafied report of the research work carried out by me*. The material contained in this research has not been submitted to any University or Institution for the award of any degree.

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C E R T I F I C A T E

This is to *certify* that the Research Thesis titled**, Investigations on Accelerated Consolidation of Coir Reinforced Laterite, Lithomargic Clay and Blended Soils with Vertical Sand Drains for Pavement Foundations**, submitted by **RAMAKRISHNA HEGDE** (REG. NO. 040866CV04P6) as the record of the research work carried out by him is *accepted as the Research Synopsis submission* in partial fulfillment of the requirements for the award of the degree of **Doctor of Philosophy** in the **Department of Civil Engineering**, National Institute of Technology Karnataka, Surathkal.

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ACKNOWLEDGEMENTS

I am grateful to my esteemed guide **Dr. A.U. Ravishankar**, Professor and Head of Department of Civil Engineering, for his help guidance, co-operation and encouragement during this work.

I hereby express my sincere thanks and gratitude to my esteemed guide **Dr.Varghese George**, Professor in the Department of Civil Engineering, for his expert guidance, and continuous encouragement for the progress of the research work.

I also take this opportunity to express my gratitude to **Dr. Sitaram Nayak**, Associate Professor, and **Dr. R. Shivashankar**, Professor, Department of Civil Engineering, for their support during my research work.

I express my sincere thanks to management of **Srinivas Group of Institutions** and Principal of Srinivas School of Engineering, Mukka, **Dr. Shreeprakash B.**, for the constant encouragement and support for the research work. I also express my gratitude to the M.Tech Scholars, **Mr. Dwarika Gotamey** (2009), **Mr. Santhosh G.** (2010), **Mr. Durga Prashanth L.M.** (2010), **Mr. Nirmal Kumar (2011)**, and **Mr. Vishnu Vardhan (2011)**, for their assistance, and involvement in this work.

I also take this opportunity to thank all the **faculty members**, and the **technical** and **non-technical staff** of the Department of Civil Engineering, NITK Surathkal, for their support and assistance rendered at critical moments in the progress of this study.

Finally, I am indebted to my wife, **Smt. Yasha K.S.,** and my daughter, **Deeksha R.,** for their moral support and their encouragements at difficult moments in the progress of this work. My parents, **Shri. Vishnu Ramakrishna Hegde**, and **Smt.Vimala Hegde**, and my parents-in-law **Shri. Satyanarayana K. L**, and **Smt. Girija Sathyanarayana**, were a source of inspiration for the progress in research. I thank them for their unstinted support and assistance.

RAMAKRISHNA HEGDE

ABSTRACT

Pavement layers built over the sub-grades are designed to transmit loads to the soil layers below, keeping the deformations within limits even under adverse climatic and loading conditions. Sub-grades on embankments need to provide structural stability to support traffic-loads. Soil obtained from borrow pits, transported, and laid on embankments for highway construction, have very low CBR values in the range of 1-2%. In the conventional practice of road construction, the consolidation of soil layers take 1-2 years to materialize after the soil is compacted.

In this context, the use of natural fibers such as coir in providing vertical sand drains and in soil-reinforcement is expected to accelerate the process of consolidation by permitting pore-water pressures to be easily dissipated when subjected to overburden pressures, which will prevent further subsidence of lateritic sub-grades. The use of vertical drains, accelerate the radial drainage and enhance consolidation, by reducing the length of the drainage paths. However, it is found that more investigations need to be performed on the use of vertical drains for coir-fiber reinforced lateritic soils of the peninsular regions of India with special reference to the District of Dakshina Kannada. Research in this direction is expected to generate information on improving the efficiency of vertical drains, and will have a profound influence in the field of highway construction especially in tropical regions where coir is abundantly available.

Laterite and Lithomargic (Shedi) soil samples used in this study were collected from a site close to National Institute of Technology Karnataka, located in the district of Dakshina Kannada, India. Tests for basic properties and CBR, were performed as per specifications of Bureau of Indian Standards (BIS).

Investigations on consolidation were performed for samples of laterite soils, shedi soils and laterite blended with shedi (lithomargic) soils, using circular test moulds of ferro-cement (of 70 cm. internal diameter, and 85 cm. internal height). Tests for consolidation were performed on the un-reinforced soil samples with and without the use of vertical sand drains. Similar tests were performed on randomly reinforced soil samples also, to assess the effect of the use of vertical drains. The randomly reinforced soil samples were prepared for optimal coir-fiber content by weight of soil, determined based on CBR studies. Each soil sample was first subjected to a preload of 50 kg ($1.2\text{x}10^{-3}$ N/mm^2) at the top of the cylindrical test mould, and the settlement was studied. When the settlement rate reduced to lesser than 0.02 mm per hour, the next increment of preload was applied. The procedure was repeated until the settlement readings were taken for a final preload of 500kg ($11.6\ \text{x}10^{-3}$ N/mm^2).

In the above studies, locally available river-sand passing through 4.75 mm IS sieve was used for fabricating the vertical drains. The aspect-ratio of coir fiber used was 1: 275. In this study, the use of sand randomly distributed with 1.0 % coir fiber was used. The coir reinforcement imparts lateral stability to the vertical drain, while enhancing the drainage properties. River sand of same characteristics was used in the preparation of the top and bottom layers of the cylindrical test mould to ensure uniform loading and drainage for the tests for settlement and consolidation.

While analyzing the results for *100%L+0%S*, it was seen that the soil attained stability at around the 121st minute after application of the pre-loads for *UR* (un-reinforced soil), *UR-VD* (un-reinforced soil with vertical drains), and *RR-VD* (randomly-reinforced soil with vertical drains). Using the 121st minute as the datum, it was observed that the effect of

providing vertical drains alone was not significant when compared to the rate of settlement, for the entire range of pre-loads from 50 kg ($1.2x10^{-3}N/mm^2$) to 500kg ($11.6x10^{-3}N/mm^2$). Also, in the case of *100%L+0%S UR* soils, it was observed that the maximum value of C_v of 2.0825 cm^2/s was found to occur at a higher pressure range of 3.5 to 4.6 $x10^{-3}$ N/mm^2. In the case of *UR-VD* soil conditions, the maximum value of C_v of 2.2683 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. Also, in the case of *RR-VD* soil conditions, the maximum value of C_v of 2.2882 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. This indicates that the consolidation occurs at a faster rate at lower pressure ranges for reinforced soils.

While analyzing the results for *0%L+100%S*, it was seen that the soil attained stability at around the 225th minute after application of the pre-loads for *UR*, *UR-VD*, and *RR-VD* soils. Using the 225th minute as the datum, it was observed that the effect of providing vertical drains was significant considering the rate of settlement for pre-loads ranging from 50kg ($1.2x10^{-3}N/mm^2$) to 250kg ($5.8x10^{-3}N/mm^2$). However, for higher pressures varying from 300kg ($7x10^{-3}$ N/mm^2) to 500kg ($11.6x10^{-3}N/mm^2$), further settlement was not found to be significant. For *UR-VD* soils, the relative increase in the settlement when compared to that of *UR* soils, ranged between 40.61% and 294.55%, with an average increase of 176.6% for the preload ranging between 50kg ($1.2x10^{-3}$ N/mm^2) and 250kg (5.8 $x10^{-3}$ N/mm^2). This is very significant from the practical point of view. But for pre-loads higher than 250kg, the effect of providing vertical sand drains alone (as in *UR-VD* soils) was not significant as it was found to vary between 4.01% and 13.06% only, with an average increase of 7.67%.

Also the case of *0%L+100%S RR-VD* soils, there was an additional increase of 32.6% in the settlement when compared to that of *UR-VD* soils for pre-load ranging from 50kg ($1.2x10^{-3}N/mm^2$) to 250kg ($5.8x10^{-3}N/mm^2$). For higher pre-loads, an increase of 10.76% was observed. Thus, it can be concluded that for *0%L+100%S RR-VD* soils, there is a significant increase in settlement due to random reinforcement with coir fibres when coupled with the use of vertical drains.

In the case of *0%L+100%S UR* soils, it was observed that the maximum value of C_v of 1.0821 cm^2/s was found to occur at a higher pressure range of 3.5 to 4.6 $x10^{-3}$ N/mm^2. In the case of *UR-VD* soil conditions, the maximum value of C_v of 1.3661 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. Also, in the case of *RR-VD* soil conditions, the maximum value of C_v of 1.8277 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. This indicates that the consolidation occurs at a faster rate at lower pressure ranges for reinforced soils. Similar studies were made on *75%L+25%S* soils, *50%L+50%S* and *25%L+75%S* soils.

In the above study, it was observed that in the case of lithomargic soils there was a significant increase in settlement at lower preload pressures. Reinforced soils of this category also displayed very high settlements, indicating that softer soils can be effectively consolidated using vertical drains and random reinforcements using natural fibers. In the case of pure lateritic soils, the use of vertical drains alone was not found to be effective. However, for other lateritic blends, the use of vertical drains significantly contributed to the settlement.

Key words: Laterite, Lithomargic Clays, Vertical Drains, Sand Drains, Consolidation.

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NOMENCLATURE

w	:	Water content
w_l	:	Liquid limit
w_p	:	Plastic limit
w_s	:	Shrinkage limit
G	:	Specific gravity
I_p	:	Plasticity index
γ_d	:	Dry density
$(\gamma_d)_{max}$	:	Maximum dry density
MDD	:	Maximum dry density
OMC	:	Optimum moisture content
C_u	:	Uniformity coefficient
C_c	:	Coefficient of curvature
CBR	:	California bearing ratio
T_v	:	Time factor
C_v	:	Coefficient of vertical consolidation
k	:	Coefficient of permeability
U	:	Degree of consolidation
U_{vh}	:	Combined radial and vertical consolidation.
UR	:	Un-reinforced
UR-VD	:	Un-reinforced with vertical drain
RR-VD	:	Randomly reinforced with vertical drain.
R_{ct}	:	Relative increase in consolidation
L-S Soils	:	Laterite Shedi Soils

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Maintenance of existing roads and also the construction of new roads in water-logged areas pose challenges to highway engineers. Investigations on failure of roads in many areas reveal that defects in the subgrade, including poor compaction and consolidation, and improper sub soil drainage design play a major role, especially in submersible regions, and in areas that frequently encounter changes in the height of water tables. In this connection, the use of natural fibers in soil stabilization, and in providing vertical sand drains is expected to play a vital role in highway embankment constructions. Additionally, it is observed that vast tracts of areas in coastal regions are either water-logged, or exhibit the prevalence of soft-clay or silty sub-grades. But nevertheless, infrastructure development cannot be denied to such areas, which necessitate the use of ground improvement techniques.

The differential settlement of subgrade soils is found to cause extensive pavement deterioration in many regions. The life of a pavement structure depends largely on the type and characteristics of the subgrade soil. Especially in clayey and silty soils that possess low permeability values, the consolidation and the resultant settlement take longer durations to occur, resulting in further settlements after the construction of pavements. Hence it is of prime importance to perform studies on the consolidation characteristics of the subgrade soil extensively.

Even as early as the fifth millennium B.C, compacted soil reinforced with reed was used in the construction of dwellings in the Iranian Plateau **(Indian Jute Industries Research Associations, 2005)**. The use of clay bricks reinforced with river reed mats was widely practiced in most of the settlements of ancient civilizations. Fascines, or foggots made of well-ordered bundles of willow, alder, or brush wood were used in the construction of levees. Fascine-mattresses were first experimented in the protection of river beds of Mississippi. Roads made of split logs topped with soil, also known as 'Corduroy Roads' were popular even in the third millennium B.C. **(Dewar, 1962)**.

The combination of woven cotton fabric covered with hot bitumen, was experimented initially as a geo-textile, and was used successfully in many road construction works undertaken by the South Carolina Highways since 1926. The results of the field tests performed over a period of nine years were found to be encouraging. However, cotton fabrics never became properly established as geo-textiles, since this failed to satisfy the technical requirements **(Indian Jute Industries Research Association, 2005)**.

Geo-textiles are of two types - natural, and synthetic. Natural geo-textiles are made of natural fibers of coir, jute, and of similar materials, while synthetic geo-textiles are made of polymers and petrochemical derivatives. It is found that natural fibers of coir made from processed husk of coconuts can be used effectively in the improvement of sub grade strength mainly due to enhanced consolidation as a result of accelerated drainage of moisture due to presence of coir fibers.

In the conventional procedures adopted in the installation of pre-fabricated vertical drains (PVDs) for accelerated settlement of embankments, the use of mandrels and anchor plates result in lesser permeability and reduced radial drainage in the soil layer adjacent to the vertical drains due to the smear effect (**Baron 1948; Rowe, 1968; Hansbo 1979, 1981**). This consequently decelerates the settlement and consolidation of soil embankments. On the other hand, the use of built-up vertical sand drains are expected to ensure enhanced accelerated settlement due to better lateral soil drainage since the smear-effect is insignificant.

It is felt that studies on the use of built-up sand drains and the effect of accelerated settlement on soils of lateritic origin will provide a clear understanding on the settlement characteristics in the absence of smear-effects. This study on the use of sand-drains randomly reinforced with coir-fibers, is of special importance to the region of Dakshina Kannada in Southern Peninsular India, where the soil is predominantly of lateritic and lithomargic origin.

1.2 LATERITE SOIL

Francis Buchanan (**1807)** described lateritic soils as "Ferruginous, vesicular, un-stratified, and porous soils with yellow ochre's due to high iron content, occurring in Malabar, India". He had made pioneering studies on laterites for the first time in India at locations close to Angadippuram Railway Station, in Malappuram District of Kerala State. Laterite soil sub-grades possess higher strengths.

Also, when blended laterites are exposed to moisture, the strength of lateritic subgrades reduce. In this context, the use of natural fibers such as coir, in providing vertical sand drains and in providing soil-reinforcement, is expected to accelerate the process of consolidation by permitting pore-water pressures to dissipate easily when subjected to overburden pressures.

Laterite soil can be found to occur above underlying shedi soil (or fine silty soil) in almost all parts of Dakshina Kannada and Udupi districts. Laterite soil is comparatively stronger than shedi soil. In India, laterite soils cover a total area of about 248,000 sq.km. Laterites are predominantly encountered in the hills of the Deccan, Karnataka, Kerala, Madhya Pradesh, the Eastern Ghat regions of Orissa, Maharashtra, Malabar, and some parts of Assam**.**

1.3 SHEDI SOIL

Locally available whitish, pinkish or yellowish *lithomargic* soils are called *Shedi soils.* These soils consist of *Lithomargic clays*, and occur at depths of 1-3 meters below the top lateritic outcrops throughout the Konkan area that extends along the western coast from Cochin to Bombay, and also in the Deccan Plateau of India.

Shedi soil possesses high strength in dry conditions, while the soil-strength reduces considerably under moist conditions. Leaching of shedi soil takes place mainly due to heavy rainfall. Engineers have to be extremely careful in tackling this soil. Slope-failures, land-slides, are liable to occur where such soils are prevalent.

The scarcity of land with strong sub-grades now compel engineers to make the best use of available soil sub-grades for infrastructural developmental. The rapid urbanization, and industrialization, has further worsened the existing situation in the

District of Dakshina Kannada, and the adjoining areas. The strength of weak soils can be improved to a large extent by accelerating the consolidation/ compaction of soil by providing better drainage to the soil subgrade.

1.4 USE OF PRELOADS, VERTICAL SAND DRAINS, AND REINFORCED SAND DRAINS FOR ACCELERATED CONSOLIDATION OF WEAK SOILS

Pavement layers built over the sub-grades are required to transmit the loads to the soil layers below, keeping the deformations within acceptable limits under adverse climatic and loading conditions. Thus, it is essential to impart structural stability to foundations of highways and embankments in weak soil for effectively supporting the traffic loads imposed on them. Preloading for a longer duration is to be performed for soft clays and fine silty soils due the lower permeability values, in order to attain the desired consolidation so that the subgrade develops sufficient shear strength.

The use of vertical sand drains randomly reinforced with fibers is expected to further reduce the resistance to the movement of water when moisture is expelled by preloading. This approach is considered to be beneficial in the stabilization of soft and compressible soils rendering higher load carrying capacities.

Vertical drains are generally employed in improving the soil strength in areas related to the construction of the following:

- Highways and embankments.
- Runways.
- Bridge abutments and approaches.
- Retaining walls.
- Parking lots.
- Consolidation of Landfills.
- Reclaimed of wet-lands.

Sand used in drainage layers must satisfy the permeability and piping requirements as mentioned in **Table 4.3** to ensure higher drainage efficiencies.

1.4.1 Major Benefits in the Use of Sand Drains

The main advantages in the use of sand drains include the following (**Indraratna et al., 1994**):

i) acceleration in the rate of gain of shear strength;
ii) enhancement in the rate of settlement;
iii) reduction in the lateral transmission of excess hydrostatic pressures that causes damage to soil foundations;
iv) improvement in drainage properties; and
v) achievement of significant increase in the stiffness of soft compressible soils.

1.5 STABILIZATION OF SOIL USING NATURAL FIBERS

The use of jute products in civil engineering is confined to the manufacture of sand bags and curing of concrete.

Coir fiber which is abundant in South Asian countries like India, Srilanka, Indonesia and the Philippines is considered to possess good mechanical properties and higher tensile strengths even in the presence of moisture**.** The major advantages in the use of these natural fibers are that they are easily available, cheaper, and biodegradable. The use of natural fibers in geo-engineering ensures the sustainability of the natural environment, preserving of ecological balance, and in assisting eco-friendly disposal. The promotion of use of natural fibers will generate employment opportunities in the rural sector and improve the village economy.

Lekha et al. (2003) and **Vishnudas et al (2006)** present a few case studies on the construction and performance monitoring of coir fiber reinforced side-slopes and bunds, and found that the use of coir is a cost effective eco-hydrological measure compared to stone pitching and other stabilization measures.

Coir is produced by processing the husk of coconut. India is the largest producer of coir accounting for 66% of worldwide production. About 80 tons of fibers can be extracted from the husk of one million coconut husks. See Table 1.1 for details on coconut production from India. Figs 1.1, 1.2, 1.3, and 1.4 show the various stages in the manufacture of coir fibers. 25 % of the refined fibers manufactured are utilized industrially **(Rao et al., 2005)**.

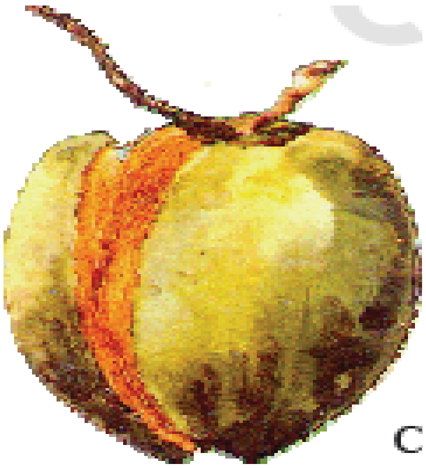

Fig. 1.1 Coconut

Fig. 1.2 Coir Fiber

Fig. 1.3 White Fiber

Fig. 1.4 Brown Fiber

1.6 PROBLEM DEFINITION

Continued focus on expansion of highway and railway infrastructure is an essential prerequisite for maintaining higher and sustained economic growth rates. The constant demand for widening of existing roads, and the need to expand the transportation network on a war-footing has made it imperative for engineers to adopt ground improvement methods for constructing stabilized surfaces over soft soils **(Johnson, 1970; Indraratna et al., 1992)**.

Since compressible soils are usually characterized by very low permeability, the time needed for achieving the desired consolidation can be very long, even with high surcharge loads. Therefore, the application of preloading alone may not be sufficient in order to maintain speedier construction. In this context the use of vertical drains is expected to further accelerate radial drainage and consolidation, by reducing the length of the drainage paths.

Table 1.1 Area Under Coconut Cultivation in India, and Production

States / Union Territories	2005-2006 (Revised)			2006-2007 (Final)		
	Area (‘000 Hectares)	Production (Million nuts)	Productivity (Nuts/ha)	Area (‘000 Hectares)	Production (Million nuts)	Productivity (Nuts/ha)
Andhra Pradesh	104	892	8577	105	1326	12629
Assam	19.1	204.9	10728	19	153	8053
Goa	25.3	125.3	4953	25.5	126.7	4969
Tamil Nadu	370.6	4867.1	13133	374.6	5429.9	14495
Tripura	3.3	7	2121	3.3	7	2121
West Bengal	24.9	323.5	12992	25.1	359.1	14307
Andaman & Nicobar Islands	25.5	87.1	3416	21.4	89	4159
Lakshadweep	2.7	53	19630	2.7	53	19630
Pondicherry	2.1	27.9	13286	2.1	27.9	13286
All India	1946.8	14811.1	7608	1939.9	15840	8165

Source: *Directorate of Economics & Statistics, Ministry of Agriculture, Govt. of India, 2008*

1.7 SCOPE AND OBJECTIVES OF THE PRESENT STUDY

It is observed that consolidation of well-drained soils occurs faster when compared to that of soft soils during the construction stage and the latter must be further strengthened by applying preloads in addition to using suitable vertical drains to improve the bearing capacity of the subgrade.

Pore-water pressure builds up under the overburden pressure, and surcharge loads in the case of weak soils undergoing consolidation. Water particles under pressure moves horizontally along the drainage paths until these are squeezed out of the soil structure. However, the use of vertical drains will shorten the length of the horizontal drainage paths. As the water particles squeezed out, come under the zone of influence of vertical drains, these particles move horizontally through the soil layers rapidly, and then vertically downwards through the vertical sand drains. The speed of dissipation of pore-water pressure is further enhanced as in the case of soil layers randomly reinforced with natural fibers, subjected to higher surcharge loads, provided with reinforced or un-reinforced vertical drains.

It is felt that studies on the use of built-up sand drains and the effect of accelerated settlement on soils of lateritic origin will provide a clear understanding on the settlement characteristics in the absence of smear-effects. This study on the use of sand-drains randomly reinforced with coir-fibers, is of special importance to the region of Dakshina Kannada in Southern Peninsular India, where the soil is predominantly of lateritic and lithomargic origin.

The present work focuses on performing load-settlement studies on accelerated consolidation of various blends of *laterite* and *lithomargic* (*shedi*) soils, with and without reinforcements of randomly dispersed coir-fibers, and with and without the use of vertical sand drains randomly reinforced with coir. It also includes a quantitative assessment of the improvement in the California Bearing Ratio (CBR) values of L-S soil, when randomly mixed with coir for soaked and un-soaked conditions.

- To perform basic laboratory investigations such as grain size analysis, Atterberg's limits, CBR tests, and tests for standard and modified compaction tests for 100%

Laterite (*100%L+0%S*), 100% Shedi (*0%L+ 100%S*), and blended soil samples (50%L+50%S),(75%L+25%S) & (25%L+75%S) as specified by Indian Standard

- Further analysis were performed based on test results for soil samples subjected to accelerated consolidation with and without the use of vertical drains. Additionally, the tests were repeated for soil samples randomly distributed with coir fibers, and the results were then analyzed and compared.

In this investigation, laterite soil was obtained from a site close to NITK campus, Surathkal, Mangalore, in the District of Dakshina Kannada. Shedi soil samples were collected from Vidyanagar, close to Mangalore, Karnataka. Coir fiber used in the study were purchased from the local market. The basic tests on soil samples were then performed as specified by the Indian Standard Institution (ISI). The sequence of activities related to this work is given in **Fig.1.5**. **Fig.1.6** provides details on the load-settlement tests.

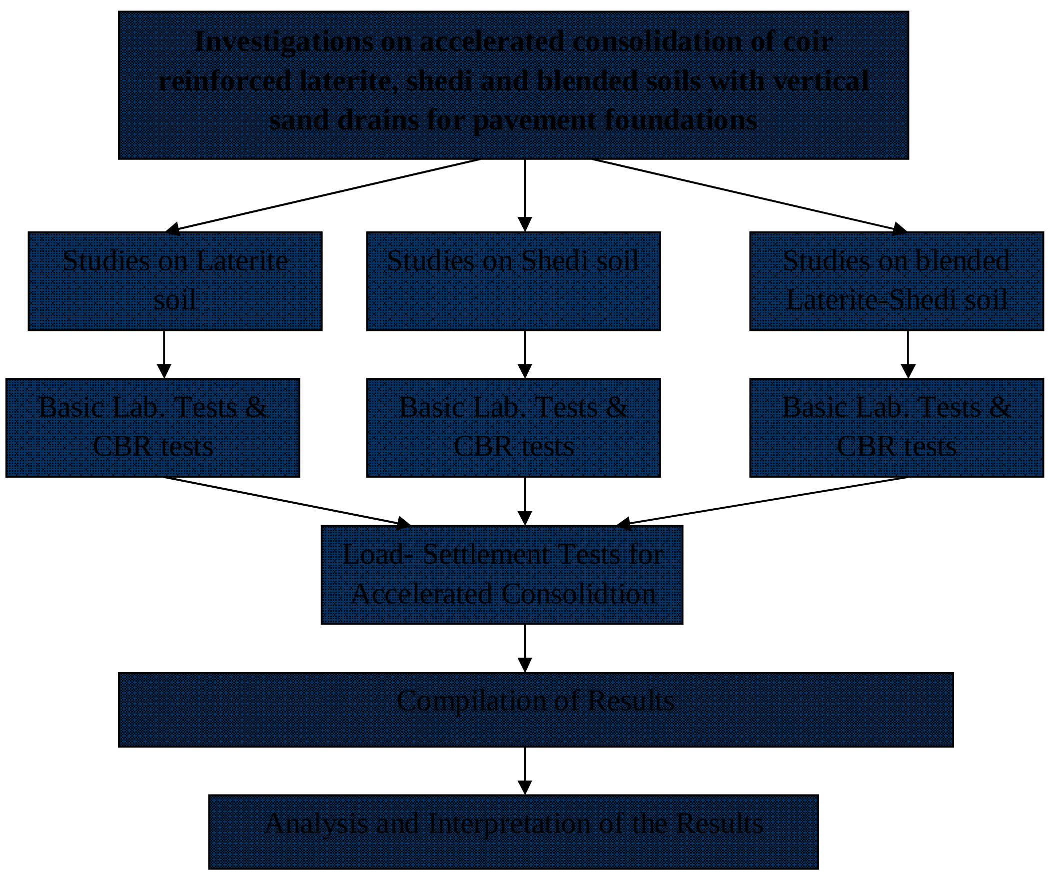

Fig.1.5 Sequence of Activities Related to the Research Work

1.8 ORGANIZATION OF THE THESIS

This work is organized and presented in six chapters. The first chapter provides an introduction to the importance of soil reinforcement using natural fibers, uses of vertical drains and their advantages, and preloading of soils for accelerating the consolidation process. It describes the problem definition, the scope of work, and the objectives of this study.

The second chapter provides a literature review on studies performed on natural fibers for soil reinforcement**,** influence of preloading, consolidation, the use of vertical drains, and the theoretical background for this study.

The third chapter deals with the theoretical foundations for the study, and provides the theoretical background on Terzhagi's theory of one-dimensional consolidation, radial consolidation, and fundamentals on load-settlement measurement. It also gives details on, consolidation of soil samples under preloads when provided with vertical drains, radial consolidation, secondary consolidation, compaction consolidation curve and Taylor's approach to determination of coefficient of vertical consolidation, and the use of prefabricated vertical drains and their limitations.

The fourth chapter deals with the experimental set up and the methodology adopted on this study. It throws light on the detailed description of the test procedures and provides the results of basic soil tests performed on various blends of Laterite-Shedi (L-S) soil.

The fifth chapter provides the results and discussions on the investigations performed on accelerated consolidation of various blends of L-S soil, simulating un-reinforced soils, un-reinforced soils with vertical drains, and randomly reinforced soils with vertical drains.

The sixth chapter provides a summary of the major research findings, along with the conclusions of the study. This chapter also provides recommendations for future research in this field.

CHAPTER 2

LITERATURE REVIEW

2.1 GENERAL

Soil improvement techniques play a vital role in enabling the use of local materials without any risk of failure of pavement structures. This is of special importance in sub-grade soils with intrusions of soft soils that result in large settlements during constructions, and differential settlement at later stages.

The use of natural fiber such as coir and jute for soil reinforcement and strengthening of sub-grades built on embankments is gaining popularity in place of using geo-synthetics, due to environmental and economic concerns. A number of studies have been reported on the use of natural fibers.

This chapter provides details of a literature review on the use of natural fibers in areas related to ground improvement. The following sections also include the studies on the influence of preloading, consolidation, and the use of vertical drains.

2.2 STUDIES ON NATURAL FIBERS FOR SOIL REINFORCEMENT

One of the first systematic study on the stability of loaded footings with reinforced soils was undertaken with the use of 'iko' vegetable fiber **Akinumusura and Akinbolade (1981)**. In this study, the behavior of square footings on deep homogeneous sand beds reinforced with flat strips of 'iko' was analyzed. Model studies were performed in the laboratory using a wooden box (of size 1.0m x 1.0m x 0.7m), filled with uniformly-graded dry-sand reinforced with 'iko' fiber strips (10mm wide and 0.03mm thick), using a steel plate (of size 100mm x 100mm x 13mm) as the footing. The researchers studied the effect of vertical and horizontal spacing, and the effect of various numbers of reinforcement layers. It was observed that there was a sharp increase in the bearing capacity, when up to three layers of reinforcements were used. But the addition of more layers of reinforcement did not result in any significant increase in the bearing capacity. Also with the increase in vertical spacing between the reinforcing layers, the tiebreak was found to be restricted to the top most layers or the second layer. It was found necessary to treat reinforcements with appropriate methods to render them waterproof and free from insect attack.

Laboratory investigations on vertical drains made of natural fiber such as jute and coir fiber for soil improvement were performed by **Lee et al (1989)** on models at simulated field conditions. The axial permeability of a fiber drain surrounded by Singapore marine clay was studied using the triaxial test. From these experiments, it was found that the axial and filter permeability of fiber drains was higher than 10^{-5} m/sec for consolidation pressures of up to 400 kN/m^2.

Maher et al (1994) performed tests for uniform compression, splitting tension, three points bending, and hydraulic conductivity on specimens of Kaolinite clay mixed with various types of fibers of different lengths. In this study, the inclusion of natural fibers was found to increase the compressive strength, ductility, splitting tensile strength, and the flexural strength of Kaolinite clay.

Sastry et al (1994) studied the efficiency of abundantly available natural resources of jute and coir in place of synthetic Geo-textiles in road construction. Un-soaked and soaked laboratory CBR tests were carried out on sub-grade soil reinforced with jute and coir mats of 60mm, 90mm, 120mm and 150mm diameter sized. Mats of these sizes were placed in one, two or three layers at equal vertical spacing in compacted CBR specimens of 125mm thickness and tested. It was concluded that the natural fibers are effective materials for reinforcing weak sub-grades in place of synthetic materials of coir may be used for reinforcing weak sub-grades in the place of synthetic materials affecting a lot of economy. Also, the location and size of geo-fibers also were found to affect the performance of roads considerably.

Gosavi et al (2003) investigated the strength and behavior of locally available black cotton soil reinforced with jute fabrics and coir ropes. It was observed that the swelling pressure of black cotton soil decreased considerably with the addition of 1% of randomly distributed coir/jute rope reinforcements in the soil sample. The reinforcement was noted as a percentage of the weight of the soil sample for a given aspect ratio. The CBR values of the soil samples showed an increase of 19% to 60% in these investigations. It was also found that the inclusion of randomly oriented discrete fibers increased the CBR values of SW and SM soil by about 96%. The optimum quantity of fibers to be mixed with soils was found to be 0.75% and any further addition of fibers beyond this did not have significant increase in the CBR values.

Kavitha and Lekha (2005) performed studies on coir geo-textile reinforced clay dykes for the drainage of low-lying areas**.** This study revealed the scope of coir geo-textiles as an environment friendly solution for strengthening water-front clay dykes. The study was carried out in the Kuttanad region, a low-lying paddy cultivated area in the State of Kerala. Over the past many decades, it was found that a large amount of labor had to be employed every year in the construction and strengthening of bunds, which resulted in causing economic losses to farmers. However, the use of coir geo-textiles in the construction of bunds contributed towards early consolidation, while minimizing the chance of early failure. The use coir geo-textiles was found to be effective in providing soil reinforcement, filtration, and drainage in the wet-lands of Kerala State.

Rao et al (2005) investigated the strength characteristics of sand reinforced with coir fibers and coir geo-textiles. They performed tri-axial compression tests in the laboratory, with varying confining pressures (of 24.5 kPa to 196 kPa), and with randomly distributed coir-fiber contents varying from 0.5% to 1%. Tests were also performed using layers of coir-mats. Specimens of 100 mm diameter and 200 mm height were used in this study. The investigations indicated that the inclusion of coir fibers and coir geo-textiles improved the performance of sand specimens.

2.3 STUDIES ON THE INFLUENCE OF PRELOADING, CONSOLIDATION AND THE USE OF VERTICAL DRAINS

Comparative studies on soft clay soils of Bangkok, improved using compacted granular piles (CGP), and prefabricated vertical drains (PVD) was done by **Bergado et al (1992)**. The studies indicated that in the case of soils improved with PVDs, the settlement rates were higher by 30-35% when compared to soils improved using CGPs.

It was also reported that the equivalent diameters of proposed prefabricated drains could be computed with reasonable accuracy using the following proposed equation:

$$d_w = 0.5\mathrm{b} + 0.7t \qquad \text{(Eq. 2.1)}$$

where, d_w = equivalent diameter of drains; b = width of drains; and t = thickness of drains.

Shen (2005) conducted field and laboratory studies on the consolidation of embankments on soft clay deposits with and without the use of PVDs. It was found that the installation of PVDs with a spacing of 1.5 m increased the vertical bulk hydraulic conductivity of the sub-soil by about 30 times when compared to that of similar soils without PVDs. Additionally, it was observed that excess pore pressure developed in soils provided with PVDs. The field discharge capacity of the soil tested ranged between 79–100 m^3/s. the studies were further performed using finite element method.

Stapelfeldt (2006) studied the effect of preloading and the use of vertical drains. It was observed that preloading, and the use of vertical drains, increased the shear strength of the soil, reduced the soil compressibility, and reduced the permeability of the soil prior to construction. It was also found that this approach prevented large differential settlements that could have caused damage to the structures.

Sinha et al (2007) performed studies predicting the settlement of PVD improved soft clays under embankments using inflection-point method. In this study, it was found that PVDs were effective in achieving 61–78% of consolidation, while further consolidation took place very slowly. Hence, it was suggested that the second stage of preloading could be commenced at this juncture. **Indraratna et al (2007**) observed that smears developed due to the disturbance to the soil while installing vertical drains, could result in reduced soil permeability around the smear zone, restricting the rate of consolidation.

2.4 RELATED STUDIES ON LATERITIC SOILS

A number of studies have been performed on consolidation properties of lateritic soils and stabilization of such soils (**Mahalinga-Iyera and Williams, 1994; Olarewaju, Balogun and Akinlolu, 2011**). Similarly studies on plasticity, compressibility and chemical characteristics were also investigated by number of researchers **(Badmus, 2010; Sastry, 1982)**.

2.5 SUMMARY

The above sections provide details on literature review on investigations performed on natural fibers for soil-reinforcement, studies on the influence of preloading, consolidation and the use of vertical drains, in addition to the use of reinforced vertical drains, and investigations on lateritc soils.

Although a number of studies on the use of vertical sand drains have been reported, it was felt that systematic and focused investigations need to be performed on the use of coir fiber reinforced built-up vertical sand drains, and the use of natural fibers in the reinforcement of partially compacted laterite and silty soils, in order to simulate weak soils underlying embankments, and pavements. It is also felt that further investigations on consolidation of various types of lateritic soils such as lateritic-lithomarge and lithomargic-laterite need to be performed.

Further research in this direction is expected to generate information on improving the efficiency of vertical drains, and will have a profound influence in the field of highway construction especially in tropical regions where coir is abundantly available.

CHAPTER 3

THEORETICAL BACKGROUND

3.1 INTRODUCTION

The properties of the soil, the stress conditions, and the thickness of the soil strata determine the degree of settlement of foundations. A number of factors such as the degree of saturation, the coefficient of permeability of the soil, the properties of the pore-fluid, and the length of the effective drainage-path affect the time taken for the reduction in void-ratio when consolidation occurs when subjected to a preload (**Raymond, 1997**).

A process where a decrease in the water-content of saturated-soil takes place without being replaced by air is termed as consolidation. Later, this theory was extended to three-dimensional situations by **Biot (1941 a, b)** and subsequently extended to include the effects of anisotropy and visco-elasticity **(Biot, 1956 a, b; Small and Booker, 1979)**. The various properties of soil were studied extensively by Terzaghi, the "father of soil mechanics" (**Lambe and Whitman, 1969**). In soil engineering related to consolidation, stress, strain, and time relationships are considered to be important (**Taylor, 1948**). The theory of one-dimensional consolidation for saturated conditions and the strain formulation was published by **Terzaghi (1923).** Details on the same were provided by **Terzaghi (1943), Taylor (1948), Scott (1963)** and **Lambe and Whitman (1969)**.

Assumptions made by **Terzaghi (1923)** have been altered to some extent by the extensive studies done on three dimensional analyses. In spite of that the one-dimensional model is still considered to be the most general and idealized model, while the other models are considered to be mathematically complex (**Tan, 1985**).

The fluid potential may act as an additional force that '*disturbs the equilibrium of an elastic system*' if the fluid-flow through deformable porous media like soil is considered. It may result in expansion or contraction strains in the media. This phenomenon is considered to be '*poro-mechanical*' in nature if the porous medium is fractured due to natural processes. Fluid flow will be affected by porosity between the soil layers. These processes are referred to as *coupled processes*. An explanation was provided by **Terzaghi (1923)** on the *poro-elastic* concept based on which the *effective-stress law* was proposed. Here, the external load or *total stress* was considered to be constant, while the *effective stress* and the *pore-pressure* vary with time. This chapter provides details on the theoretical foundations for studies on consolidation on soil samples.

3.2 PRELOADING OF SOIL SAMPLES WITHOUT VERTICAL DRAINS

Prior to the construction and placement of the final construction load, if the soil is compressed under applied vertical stress, preloading is said to be performed. The two common preloading techniques are, conventional preloading, (using embankment), and vacuum-induced preloading. When the load is placed on soft soil, it is initially carried by the pore water pressure. The pore water pressure gradually decreases, as the water drains slowly in the vertical direction. In order to ensure stability, the load must be placed in preferably two or more stages (**Stapelfeldt, 2006).** This is illustrated in **Fig. 3.1a**.

If the temporary load exceeds the final construction load, the excess load is referred to, as the surcharge load. The temporary surcharge load can be removed when the settlements exceed the predicted final settlement. By increasing the time of temporary overloading, or the size of the overload, the secondary settlement can be reduced or even eliminated.

This is because, by using a surcharge higher than the work load, the soil will always be in an over-consolidated state, and the secondary-compression for over-consolidated soil is much smaller than that of normally consolidated soil. This will benefit greatly the subsequent geotechnical design **(Chu et al, 2004)**.

3.2.1 Terzaghi's Theory of One-Dimensional Consolidation

The basic theory on one-dimensional consolidation was originally developed by Terzaghi in 1923 (**Terzaghi, 1943**). A simple one-dimensional consolidation model consists of rectilinear element of soil subject to vertical changes in loading that result in seepage flow in the vertical direction. See **Fig.3.1b**. The assumptions made in Terzaghi's theory of one-dimensional consolidation are given below:

a. The soil is homogeneous, and fully saturated.

b. The solid particles of soil, and water, are incompressible.

c. Compression and flow occur along the vertical direction (one-dimensional).

d. The total stress on the soil element is assumed to remain constant, and the strains observed are small.

e. Darcy's law is valid at all hydraulic gradients.

f. The coefficient of permeability (k) and the coefficient of volume change (m_v) remain constant throughout the consolidation process.

g. There is a unique relationship, independent of time, between the void-ratio and the effective stress.

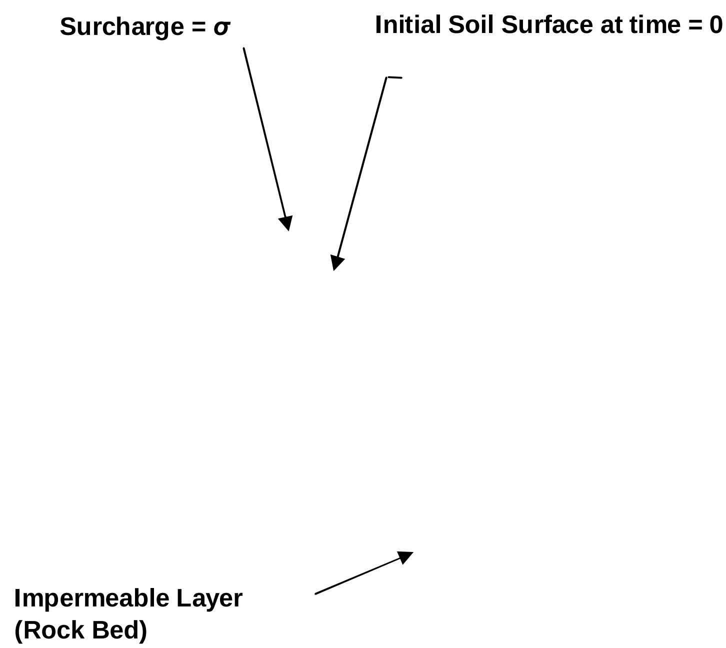

Fig. 3.1b One-Dimensional Consolidation in Field Conditions

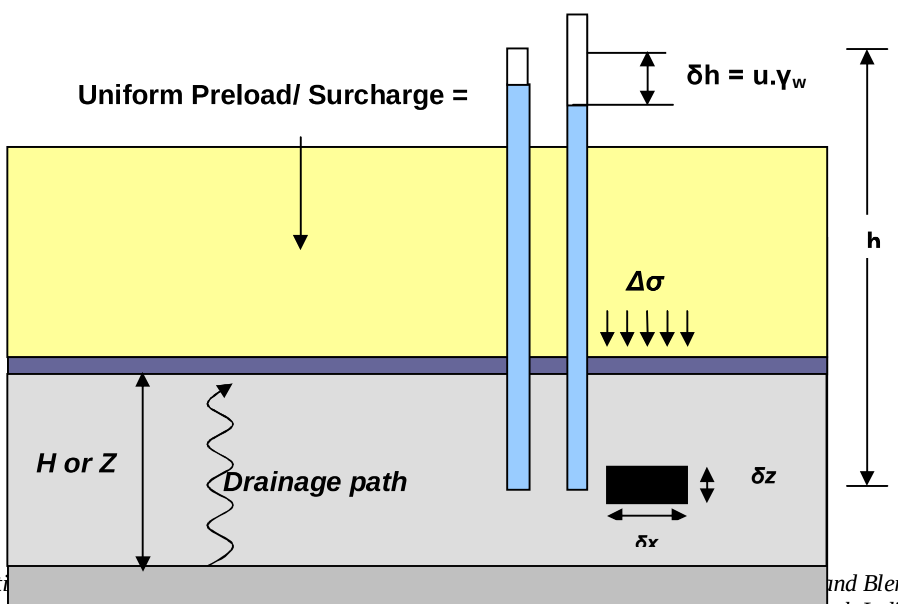

Fig. 3.1c Terzaghi's Model of One-Dimensional Consolidation

The above theory relates the following three quantities:

- the depth (z) below the top of the clay layer subjected to a vertical stress (σ);
- the excess pore pressure (u) developed due to increment in vertical stress ($\Delta\sigma$); and
- The time (t) from the instantaneous application of a total stress increment.

A modified form of the model used by **Terzaghi** (**1923**) is provided in **Fig.3.1c**. Consider an element having dimensions dx, dy and dz within a clay layer of thickness H as shown in this figure. An increment of total vertical stress $\Delta\sigma$ is applied to the element. The flow velocity v_z through the element is given by **Darcy's** (**1856**) law as:

$$v_z = k.\ i_z = k\ .h/z \qquad \text{(Eq 3.1)}$$

where, k is the coefficient of permeability which is assumed to be constant; i_z is the hydraulic gradient; h is the head of water-pressure; and z is the depth of the soil.

Since any change in total head (δh) is due only to a change in pore water pressure, we can express the flow velocity as,

$$v_z = (k / \gamma_w)\ (u / z) \qquad \text{(Eq 3.2)}$$

where, u = excess pore pressure due to water in the same units as that of the stress applied; γ_w = unit weight of water; and z = depth below the top of the clay layer.

In the standard tests for consolidation performed in the laboratory, the soil specimen is sandwiched between the porous stone media placed at the top and at the bottom of the consolidation cell of the consolidometer (or oedometer). In this case, where the soil specimen is drained at the top and at the bottom, the effective drainage path is taken as half the thickness of the soil specimen (or H/2). See **Fig.3.1c**. Also, in the case of the experimental investigations performed, the soil specimens were drained at the top and at the bottom.

The relationship between the final settlement (S_f) and the settlement (S_t) at time t was expressed as (**Terzaghi, 1923**):

$S_t = U_v \,.\, S_f$ (Eq. 3.3)

The expression for the average degree of consolidation at depth z at any instant t was given by **Terzaghi** (**1943**) as,

$U_v = 1 - \sum_{m=0}^{\infty} (2/M^2) \exp(-M^2 T_v)$ (Eq. 3.4)

where Tv = time factor (non dimensional) = $(C_v t)/H^2$; C_v = coefficient of vertical consolidation (m^2/s) = $k/(\gamma_w m_v)$; k = permeability coefficient; m_v = coefficient of volume compressibility = $a_v/(1+e_0)$; e_o = initial void ratio; a_v = coefficient of compressibility = Δ_e/Δ_p; t = time in seconds; H = total distance of drainage path which is equal to the thickness of the layer (in m.) for soil subjected to top drainage, and is equal to half of the thickness of the layer (in m.) for soils drained at the top and the bottom; and $M = \pi (2m+1)/2$ for m = 0, 1, 2…α. Therefore, it is expected that the use of vertical drains will have a profound effect on accelerating the consolidation process.

Derivation of Terzaghi 's expression for U_v, the average degree of consolidation **(Raymond G.P, 1997)**

Consider an element of dimensions *dx, dy* and *dz* within a layer of clay of thickness H as shown in **Fig. 3.1c**. Also consider an increment in the total vertical stress $\Delta\sigma$ applied to the element. The velocity of flow through the element is given by Darcy's law as:

$v_z = k.\, i_z = k\partial h/\partial z$ (Eq 3.5)

where k is the coefficient of permeability which is assumed to be a constant, i is the hydraulic-gradient, h is the pressure-head, and z is the depth of the soil element. Since any change in total head (h) can happen only due to a change in pore-water pressure, we can say that,

$v_z = (k/\gamma_w)(\partial u/\partial z)$ (Eq 3.6)

where, u is the pressure of the water (or pore-water pressure), and γ_w is the unit weight of water. In order to satisfy the condition of continuity, the rate of change in velocity of flow must be equal to the rate of change of the volume (considering soil particles and water as incompressible). Thus we get,

$dV/dt = (k/\gamma_w)(\partial u/\partial z)$ (Eq 3.7)

where, V is the volume of the soil, t is the time elapsed after applying the incremental pressure.

But, the rate of change of volume is also equal to the rate of compression in volume. Expressing this in terms of the modulus of volume change (m_v), which is a constant, according to Terzaghi's theory, we get,

$$dV/dt = m_v(\partial\sigma'/dt)dxdydz \qquad \text{(Eq 3.8)}$$

Combining Equations Eq 3.7 and Eq 3.8, we get,

$$m_v = a_v/(1+eo) \qquad \text{(Eq 3.9)}$$

where, a_v is the coefficient of compressibility of the soil = - $\delta e/\delta\sigma'$, mv is the modulus of volume change, and eo is the initial void-ratio.

As the total stress increment is gradually transferred to the soil skeleton, the pore pressure decreases, and the effective stress increases. Thus, the rate of change in volume can be expressed as,

$$dV/dt = m_v(\partial u/dt)dx\,dy\,dz \qquad \text{(Eq 3.10)}$$

Combining Equations (Eq 3.9) and (Eq 3.10), we get,

$$m_v(\partial u/dt) = (k/\gamma_w)(\partial^2 u/\partial z^2)$$

That is,

$$(\partial u/\partial t) = (k/\gamma_w)(\partial^2 u/\partial z^2) \qquad \text{(Eq 3.11)}$$

$$(\partial u/\partial t) = c_v(\partial^2 u/\partial z^2) \qquad \text{(Eq 3.12)}$$

This is the differential equation of consolidation, in which

$$c_v = k/(m_v\,\gamma_w) \qquad \text{(Eq 3.13)}$$

where c_v is defined as the coefficient of consolidation. Since k and m_v are already assumed to remain constant, c_v too can be considered to be constant.

Boundary Conditions for Terzaghi's differential equations, and Taylor's Solution

In the consolidation test performed in the laboratory, the total stress increment is applied instantaneously, at the commencement of the preload test at each stage. Since the pore water and soil grains are incompressible, the increase in load applied will be instantaneously supported by the pore water. Thus the initial value of excess pore water pressure (u_i) is equal to $\Delta\sigma$. Thus, the boundary conditions at the beginning of each preload is given as,

$$u = u_i\,,\ for\ \ z=0\ \&\ H\,,\ when\ t = 0 \qquad \text{(Eq 3.14)}$$

where, d is the length of the drainage path. For a two-way drainage, where the specimen is drained both at the top and at the bottom, the drainage length is equal to half the height of the specimen ($H/2$). For specimens drained only at the top or bottom (one way drainage), the length of drainage is equal to the height of the specimen (H). In a two-way drainage test, the permeability of the materials (granular soil or porous stone) adjacent to the boundaries at the top and bottom is considered to be very high when compared to that of. Thus the boundary conditions at any time after the application of the increment $\Delta\sigma$, can be expressed as,

$$u = 0 \text{ , for } z = 0 \text{ , when } t > 0$$

$$u = 0 \text{ , for } z = H \text{ , when } t > 0 \qquad \text{(Eq 3.15)}$$

The solution for the excess pore-water pressure at depth z after time t is given by Taylor (1948).

$$u = \sum_{N=1}^{N=\infty} [(2/H) \int_0^H u_i \sin (N\pi z/H)dz] \sin(N\pi z/H) \exp\text{-}(N^2\pi^2 c_v.t/H^2) \qquad \text{(Eq 3.16)}$$

where, u_i is the initial excess pore-water pressure, and is a function of z. Assuming that u_i remains constant throughout the depth of the layer, the equation assumes the form as,

$$u_z = \sum_{N=1}^{N=\infty} (2u_i/M) \sin (2Mz/H) \exp(-M^2T_v) \qquad \text{(Eq 3.17)}$$

where,

$$\text{M}=(\pi /2) (2N + 1) \qquad \text{(Eq 3.18)}$$

$$T_v= C_v T /d^2 \qquad \text{(Eq 3.19)}$$

where T_v is known as the *time factor*.

Eq 3.16 represents a set of parabolic curves when drawn for various time limits. The equation may be integrated to provide the expression for the average degree of consolidation. For a constant load increment (u_i) in the consolidation test performed using an oedometer, and the *degree of consolidation* is given by,

$$U = 1 – [(1/H) \int_0^H u.dz] u_i \qquad \text{(Eq 3.20)}$$

$$U = 1 - \sum_{N=1}^{N=\infty} 2/M^2 \exp(-M^2 T_v) \qquad \text{(Eq 3.21)}$$

3.2.2 Representation of Load v/s Settlement Using the Square Root of Time Method

Fig 3.2 provides a schematic diagram showing one-dimensional consolidation under laboratory conditions. A typical graphical representation of load v/s settlement with the square-root of time v/s the dial-gauge readings of settlements are given in **Fig.3.3**. It is generally observed that the initial portion of the settlement-curve follows approximately an initial straight line OB, since the load-settlement curves are assumed to follow the shape of parabolic *isochrones* (**Terzaghi, 1955**).

The point U_0 representing the percentage of consolidation at the start of the loading exercise (where t = 0 minutes), is first located at the intercept of the straight portion of the settlement-curve with the y-axis. From Terzaghi's analysis (**Terzaghi, 1955**), the straight-line portion is assumed for settlements upto 60%. The general expression for the percentage of settlement at time t is then given as,

$$U_t = [4\ T_v / \pi]^{1/2} \qquad \text{(Eq 3.22)}$$

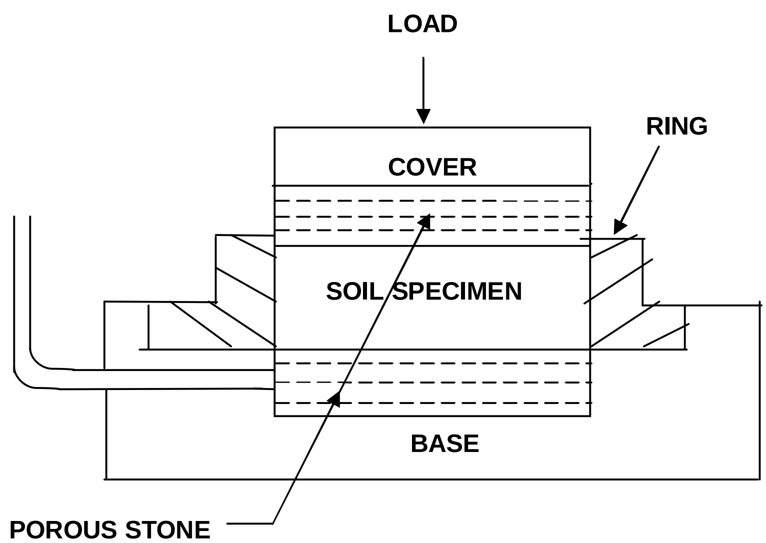

Fig. 3.2 One-Dimensional Consolidation Under Laboratory Conditions

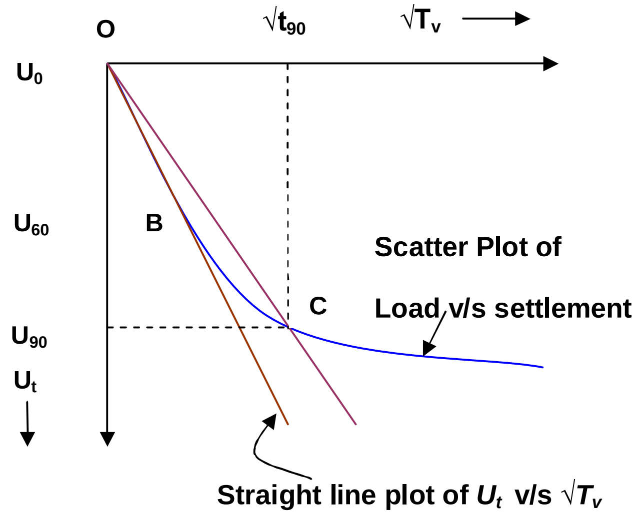

Fig. 3.3 Method of Square-Root of Time for Load-Settlement

Table 3.1 Relationship Between Percentage Consolidation (*U*%) and Time Factor (*T*)

Percentage Consolidation (U%)	Time factor (T)	Percentage Consolidation (U%)	Time factor (T)
0	0.000	55	0.238
10	0.008	60	0.287
15	0.018	65	0.342
20	0.031	70	0.405
25	0.049	75	0.477
30	0.071	80	0.565
35	0.096	85	0.684
40	0.126	90	0.848
45	0.159	95	1.127
50	0.197	100	∞

Source: Terzaghi (1943 & 1955)

Here, it can be observed that U_t v/s $\sqrt{T_v}$ follows a straight line. According to Terzaghi's analysis (**Terzaghi, 1955**), if a line OC is drawn with a slope equal to 1.15 times the slope of line OB, where it intersects the load-settlement curve at C, then the ordinate of point OC, will represent 90% consolidation (U_{90}). The abscissa for the point C then denotes $\sqrt{t_{90}}$, where t_{90} stands for the time taken for attaining 90% consolidation. The coefficient of consolidation (C_v) is then given as,

$$C_v = T_{90} \text{x} d^2 / t_{90} \quad \text{(Eq 3.23)}$$

where, T_{90}= the time-factor; and the length of the drainage path d for one-way drainage = H; and for two-way drainage for soil specimens drained at the top and bottom = $H/2$. See **Table 3.1** for standard values of time-factor (**Terzaghi, 1943, & 1955**).

3.2.3 The Final Settlement of Normally Consolidated Soil

The expression for the final settlement of normally consolidated soil, under a surcharge load was given by **Casagrande (1936)** as follows:

$$S_f = C_c / (1+e_o)\, H \log (\sigma+\Delta\sigma)/ \sigma \qquad \text{(Eq. 3.24)}$$

where, S_f = final settlement due to surcharge (m); H = thickness of the consolidation layer (m);

e_o = initial void-ratio of the representative element of soil = $w.G_s$ (for saturated soil); w = water content; G_s = specific-gravity of soil grains; C_c = compressive index; σ = initial vertical pressure, equal to the constant surcharge load (KPa); and $\Delta\sigma$ = net change in pressure at the middle of each layer when structural load is applied.

3.3 PRELOADING OF SOIL SAMPLE WITH VERTICAL DRAINS

The consolidation settlement of soft soil because of its low permeability, takes a long time to complete. To shorten the consolidation time, vertical drains are installed together with preloading an embankment. Vertical drains are artificially-created drainage paths which are inserted into the subsoil. Thus, the pore water squeezed out during consolidation of the clay due to the hydraulic gradients created by the preloading can flow faster in the horizontal direction towards the vertical drains. Subsequently, pore water can flow freely along the vertical drains, vertically down towards the permeable layers.

Therefore, the use of vertical drains reduces the length of the drainage path and, consequently, accelerates the consolidation process, and allows the soils to gain rapid strength in order to carry the superimposed load. See **Fig. 3.4**. **Fig.3.5** shows graphically, the effect of the use of vertical drains on accelerating the rate of settlement of foundations.

3.3.1 Radial Consolidation

In order to achieve radial consolidation under ideal conditions (without the effect of smear and well resistance), the average degree of consolidation for radial drainage is as (**Barron, 1948**),

U_h (t) = 1- exp [(-8T_h)/μ] (Eq.3.25)

3.3.2 Combined Vertical and Radial Consolidation (Carrillo, 1942)

The expression for combined vertical and radial consolidation is given as (**Carrillo, 1942**),

$$1- U_{vh} = (1-U_V)\ (1-U_h) \qquad \text{(Eq.3.27)}$$

where, U_{vh} = combined radial and vertical consolidation.

3.4 SECONDARY CONSOLIDATION

Secondary settlement or the consolidation that takes place after excess pore water pressure has fully dissipated is generally very small. Vertical drains have no effect on secondary settlement. Secondary settlement is of importance in the settlement of structures built on highly organic soils. The main component of long term settlement of these structures will be of the secondary type. The expression for secondary settlement is given as (**Mesri, 1973**),

$$S_s = C_\alpha\, H \log (t/t_o) \qquad \text{(Eq.3.28)}$$

where, Ss = secondary settlement produced from time 't_o' to 't'; H = thickness of the layer; t_o = initial time; C_α = rate of secondary consolidation.

3.5 COMPACTION CONSOLIDATION CURVE

The co-efficient of consolidation is calculated for every increment of preload on the soil sample. The coefficient of consolidation (C_v) for 90% consolidation is given as (**Taylor, 1948**),

$$C_v = \{T_{90} * [(H_{av}/2)^2]\}/t_{90} \qquad \text{(Eq.3.29)}$$

H= *thickness* of the soil sample (drained at the top and bottom); and (ΔH)= compression of sample after consolidation of load.

The coefficient of consolidation can be determined for various test conditions, such as for un-reinforced soils, un-reinforced soils with vertical drains, and randomly reinforced soils with vertical drains, and the consolidation of the soil can be easily estimated.

3.6 PREFABRICATED VERTICAL DRAINS AND LIMITATIONS

Prefabricated vertical drains (PVDs) consist of a core and a filter sleeve made of polymers. The drains usually are of 100mm width and are 3 to 4 mm thick. The drainage characteristics depend upon the nature of the PVD, the type of soil, and the installation methods (**Bo et al. 2003**). The first prefabricated vertic with cardboard and internal ducts was experimented by the Swedish Geotechnical Institute (**Kjellman, 1948**). This type was later superseded by thin fluted PVC drains. **Table 3.2** provides details of various types of PVDs. Most synthetic drains are made of strips or bands of geotextiles. Some PVDs possess a circular shape with a plastic core.

3.6.1 Effect of Smear and Drain Resistance in PVDs and Preference for Sand Drains

The use of PVDs create disturbances to the adjacent soil, resulting in reduced permeability, which decreases the rate of consolidation. This effect is referred to as the “smear effect”. The *smear effect* increases with the increase in the cross-sectional area of the PVD. It also depends upon the method of installation of the PVD. Dynamic driving of the mandrel (or lance) for installing the PVD results in increased smear effect. The increase in the drainage path within the drain reduces the consolidation and the subsequent increase in soil strength (**Hausmann, 1989**).

In addition to the smear effect, a number of discharge factors also affect the drainage through PVDs. These factors include

- lateral earth pressures;
- large settlements that result in deformation of the drains;
- clogging of drains due to fine particles of soil;
- biological and chemical degradation of drainage material;
- higher hydraulic gradients that result in turbulent flow in drains.

The performance of PVDs is thus affected by the discharge factors, and the smear effect. Additionally, the use of synthetic materials in PVDs, results in environmental pollution. The focus of the study was on performing studies on accelerated consolidation in the absence of smear effect. In view of this, it was resolved to perform investigations on built-up vertical sand drains reinforced with natural geo-synthetic material such as coir fiber. Also, the use of coir-reinforced sand drains is found to be advantageous especially in tropical regions where coir fiber, and natural sand are available in large quantities. This technique can be adopted easily for the construction of embankments for highways, and approach roads to bridges.

Table 3.2 Types of Prefabricated Vertical Drains

Type	Core material	Filter material	Dimensions, † mm
Kjellman	Paper	Paper	100 x 3
PVC	PVC	None	100 x 2
Colbond	PES	PES	100 x 6
Hitek	PE	PP	100 x 6

PE = Polyethylene; PVC = Poly-vinyl-chloride; PP = Poly-propylene;
PES = Polyester
† Dimensions may vary with different models of the same drain.

Source: Koerner (1986), McGown and Hughes (1981), and Gambin (1987).

The above sections provided details on the theoretical aspects of accelerated consolidation in situations where vertical drains were used. With this background, it was considered ideal to focus on the experimental setup for performing tests on various blends of lateritic and Lithomargic soils, and to formulate the methodology for the investigations to be conducted as part of this study.

CHAPTER 4

EXPERIMENTAL SETUP AND METHODOLOGY

4.1 INTRODUCTION

This chapter provides details on the tests conducted to study the long term settlement of various blends of laterite and shedi soils under various test conditions. Investigations were conducted for reinforced and un-reinforced soil specimens, with and without installation of vertical drains. California Bearing Ratio (CBR) tests were conducted for un-reinforced and reinforced soil specimens, both in soaked and un-soaked conditions. These tests were conducted in order to evaluate the improvement in the bearing capacity and CBR values of sub-grade soils. Details on the materials used for the tests and the basic tests conducted on the soil are also described briefly in this section.

4.2 MATERIALS USED IN THIS INVESTIGATION

4.2.1 Laterite Soil

Laterite soil sub-grades possess higher strengths. The subsequent consolidation of the soil layers that takes place 1-2 years after highway construction, affects the stability of embankments, and road sub-grades constructed over them. The proportion of clay fractions is considered to influence the characteristic properties of laterite soils and their permeabilities.

Laterite soil is comparatively stronger than shedi soil. In India, laterite soils cover a total area of about 248,000 sq.km. Laterites are predominantly encountered in the hills of the Deccan, Dakshina Kannada district of Karnataka, Kerala, Madhya Pradesh, the Eastern Ghat regions of Orissa, Maharashtra, Malabar (North Kerala), and some parts of Assam**.** See **Fig.4.1a, Fig.4.1b**, **Fig.4.1c**, and **Fig.4.1d**.

Laterite soil obtained from a site adjoining the newly widened National Highway (NH-17), near NITK, Surathkal, Mangalore in the District of Dakshina Kannada, was used in this study**.** The basic soil properties of laterite soil used in this study are provided in **Table 4.1**.

Fig. 4.1a Soil Profile Showing Laterite Layer at The Top, and Partially Shedi Soil Layers to the Bottom of a Highway Underpass, at NITK Campus, Mangalore

Fig. 4.1b Soil Profile Showing Laterite Layer at The Top, and Partially Shedi Soil Layers to the Bottom of a Highway Underpass, at NITK Campus, Mangalore

4.2.2 Shedi Soil

Shedi soil is the name given to the locally available yellow silty soil (or lithomargic soil). It is found underlying the top lateritic soil layer. The soil possesses good strength in dry condition, and very low strength when exposed to moisture. It is not expansive in nature.

Pure shedi soil used in this investigation was obtained from a road construction site close to Vidyanagara, Kulai, Mangalore, in Dakshina Kannada District. See **Fig.4.2a**, **Fig.4.2.b**, and **Fig.4.2c.** The basic soil properties of shedi soil used in this study are provided in **Table 4.1**.

4.2.3 Preparation of Blended Soil

Along the coastal regions of the districts of Dakshina Kannada and Udupi, engineers frequently come across soil intermixed with laterite and shedi constituents in varying proportions. Lateritic soil in Southern India comprise mainly of lateritic lithomarge (with 25-50% laterite content), and lithomargic laterite (with 50-90% laterite content). Due to this reason, it was resolved to perform investigations on blended soil samples of 75%laterite + 25% shedi soil or lithomargic soil, 50% Laterite + 50% shedi soil, and 25% Laterite + 75% shedi soil, in addition to investigations on soil samples with 100% laterite, and 100% shedi soils.

Laterite, and shedi soil samples, excavated from the respective local sites, were collected in water-proof polythene bags, and transported to the laboratory for further studies. The soil samples brought from the site were first spread on flat trays of size, 1500mm x 900mm. The lumps were broken down using rammers, and the soil was sun-dried for 10 days, until the weight of soil on the trays was found to be constant, indicating that the soil was moisture-free. The laterite and shedi soil fractions were then intermixed 5 times using the technique of quartering. This was done to ensure a uniform mix of the blended soil. Part of the above soil was subjected to drying in an oven for 24 hours at a temperature of 110 degrees Celsius, and then was analyzed for basic soil properties. The basic soil properties of laterite soil blended with shedi soil are provided in **Table 4.1**.

4.2.4 Sand

Locally available river-sand passing through 4.75 mm IS sieve with a coefficient of curvature (C_c) of 0.82, and a uniformity coefficient (C_u) of 1.7, was used in this study. See **Table 4.2**. The above sand was used preparation of vertical drains to enhance the rate of dissipation of pore water from the soil. The sand selected satisfies the general requirements of permeability and piping as suggested by Khanna and Justo (2001). **Khanna and Justo** (**2001**) provide details on the requirements of filter sand for sub-grades, based on the need to prevent piping and to provide the required permeability. The river-sand used in this study also possesses properties satisfying these requirements. See **Table 4.3**. The river-sand used in fabricating the vertical drains for this study satisfy the permeability and piping requirements for ideal filter materials.

Also, randomly distributed coir fibers were used in fabricating the vertical sand drains used in this study, as this will restrict the lateral deformation at the peripheral areas of sand drains, as corroborated by **Rao et al** (**2005**). Generally 0.5-1.0 % of coir fibers by weight is used in such cases and in the present study, 1.0 % of coir fiber by weight of sand was used in preparing the vertical drains **Rao et al** (**2005**).

Table 4.2 Properties of Sand Used in this Investigation

Property	Result
Coefficient of curvature, Cc	0.82
Uniformity coefficient, Cu	1.97
D_{15}, mm	0.35

Table 4.3 Properties of Filter Sand

Sl. No.	Requirements for Sand Filter		Desired ranges	Sand Used in the Vertical Filter
1	Permeability	(D_{15} of sand or filter) / (D_{15} of soil)	> 5	5.5
2	Piping	(D_{15} of sand or filter) / (D_{85} of soil)	< 5	0.035

Source: **Khanna and Justo (2001)**

4.2.5 Coir Fiber Used

The name *coir* originated from the Malayalam word *kayar* or *kayaru*, which refers to a twisted cord made of coconut fibers. The botanical name for the coconut tree is **Cocos nucifera**, with cocos believed to have come from Spanish, meaning "monkey-faced" or "eerie-faced", and nucifera from Latin meaning, “nut-bearing plant”. Coir is the coarse fiber extracted from the husk of coconut. Matured coir fibers are stronger and less flexible compared to immature fibers. The individual fiber cells are narrow and hollow, with thick walls made of cellulose. They are pale when immature but later become hardened and yellowed as a layer of lignin, is deposited on their walls. Mature brown coir fibers contain more lignin and lesser cellulose than fibers such as flax and cotton, and so, are stronger. The coir fiber is relatively waterproof and is the only natural fiber resistant to damage by salt water. Green coconuts, harvested after about six to twelve months on the plant, contain pliable white fibers. Brown fiber is obtained by harvesting fully mature coconuts. The general properties of coir fibers are provided in **Table 4.4**.

Fig.4.3 Coir Fiber for Reinforcement of Sand and Various Soil Blends

In the present study, coir fibers were purchased from the local market. This comprised of brown coir fibers with aspect ratios of about 275 and an average diameter of 0.18 mm. The average length of the fibers used in this study was maintained at 50 mm. These were used in the study for accelerating the consolidation process in soil sample.

4.2.6 Jute Textiles

Woven jute textile fabric of 600 g per sq.m and 1.43 mm thickness was used in fabricating the sleeves for the vertical drains. The properties of jute textiles used in this study are given in **Table 4.5**. Mild steel sleeves of 100 mm average diameter and 600mm height were used to build-up the vertical drains to span the entire depth of laterite soil tested.

4.3 TESTS FOR DETERMINATION OF OPTIMAL FIBER CONTENT USING CBR TESTS

The California Bearing Ratio (CBR) method, one of the traditional approaches to pavement strength evaluation, provides an estimate of the shear strength of soil sub-grades. It is an indirect measure that compares the strength of the sub-grade material to the strength of standard crushed rock. This is a penetration test developed by the California Division of Highways in the 1930s. The ratio is usually determined for penetration of 2.5 mm and 5 mm. Where the ratio at 5 mm is consistently higher than that at 2.5 mm, the ratio at 5 mm was reported as the CBR value.

Oven dried soil particles passing through 20 mm IS sieves were used to determine the optimum moisture content (OMC) and maximum dry density (MDD) based on the IS modified (heavy) Proctor test conducted as per IS 2720: Part VII (1983). The soil was compacted in 5 layers, providing 56 blows on each layer using a proctor rammer (falling weight of 4.5 kg). Subsequently, the CBR test was performed as per IS 2720: Part XVI (1987) for the un-reinforced soil sample, and the CBR values at 2.5 mm and 5 mm penetration were noted for OMC conditions determined using the IS modified Procter test mentioned above.

In the later stages, soil samples were randomly reinforced with 0.25 % of coir fibers by weight of the soil specimen at OMC, and the CBR tests were performed. The coir fibers used had an aspect ratio of 275 (with an average diameter of 0.18 mm, and fiber length of 50 mm). Also, CBR tests were performed on soil samples randomly reinforced with 0.5 %, 0.75 %, 1.0 %, and 1.25 % of coir, and the CBR values at 2.5 mm and 5 mm penetrations were noted for soaked and un-soaked soil samples.

In the tests for soaked specimens, a perforated bottom plate was used and the specimen was kept immersed in water for 4 days as per IS 2720: Part XVI (1987). The assembly and test apparatus for the CBR test is shown in **Fig. 4.4. Table 4.6** provides details on the CBR tests performed on un-reinforced soil specimens for soaked and un-soaked soil conditions. **Fig 4.5a** to **Fig 4.5f** provide details on the load penetration relationships for the soil specimens tested in un-soaked and soaked conditions.

The Optimum Fiber Content (OFC) was found from the CBR tests for various blends of laterite shedi soil which is as shown in the **Table 4.7**. It was observed that the further increase in fiber content is not found to result in a significant increase in the strength of soils. Thus, it was found appropriate to adopt an optimum fiber content found in further tests for randomly reinforced specimens.

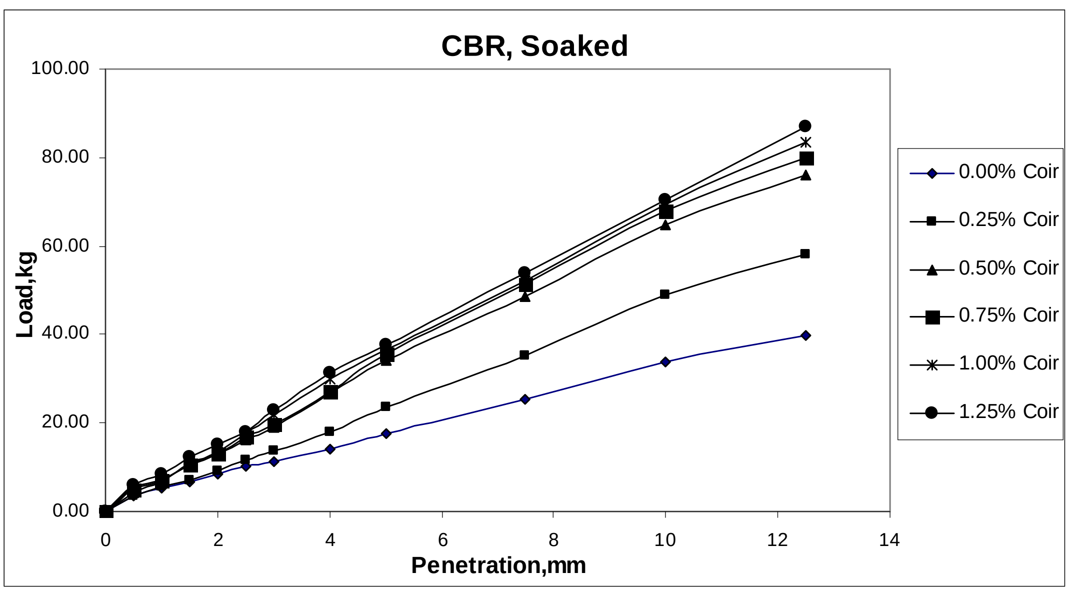

Fig. 4.5b Load-Penetration Characteristics of Un-Reinforced (Soaked Soil), and Soil with Various Percentages of RDNFRS of Coir for 100%L+0%S

Thus, it was found appropriate to adopt an optimum fiber content found in further tests for randomly reinforced specimen.

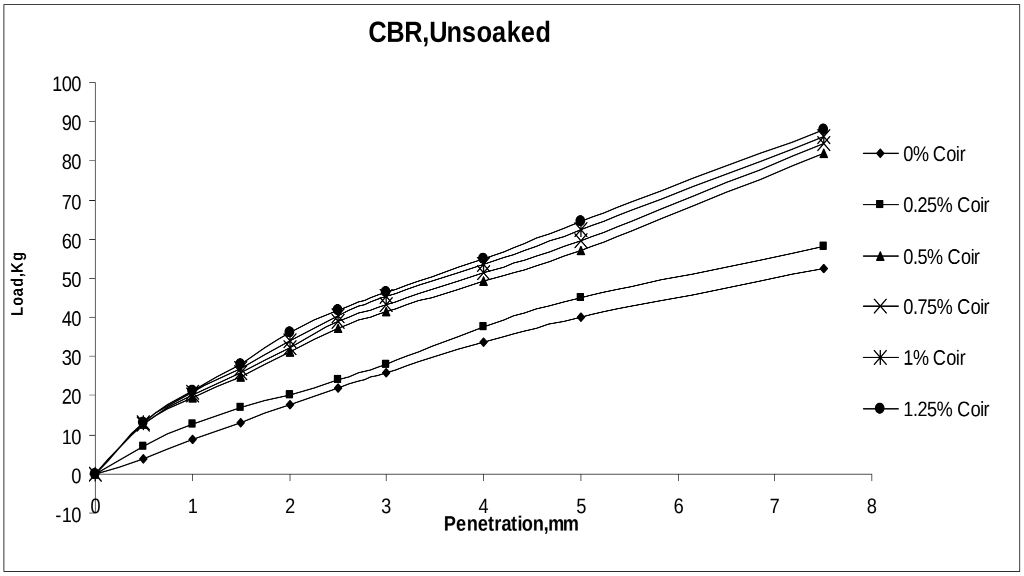

Fig. 4.5c Load-Penetration Characteristics of Un-Reinforced (Un-Soaked Soil), and Soil with Various Percentages of RDNFRS of Coir for 7*5%L+25%S*

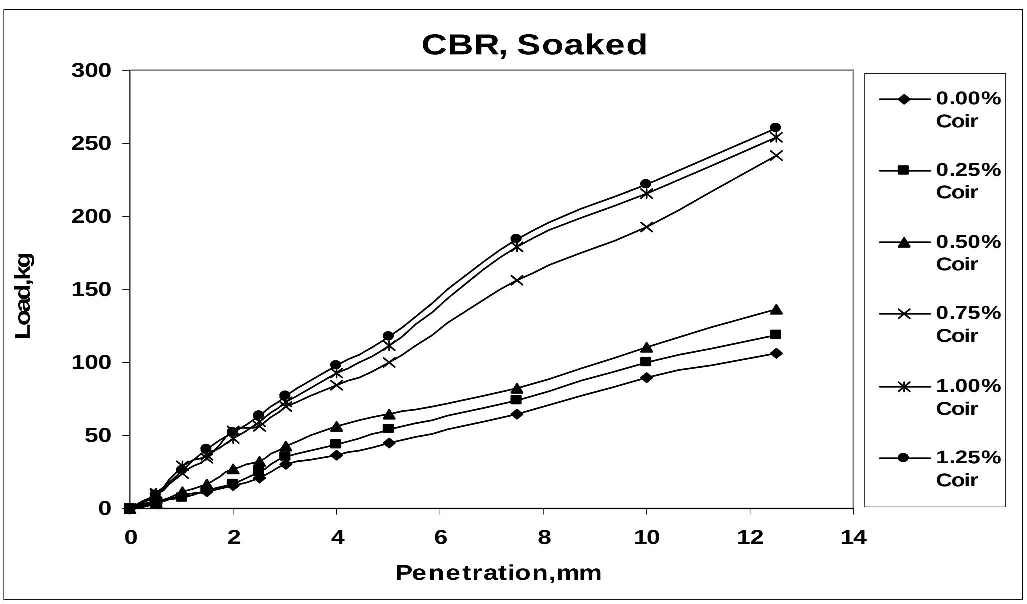

Fig. 4.5d Load-Penetration Characteristics of Un-Reinforced (Soaked Soil), and Soil with Various Percentages of RDNFRS of Coir for 7*5%L+25%S*

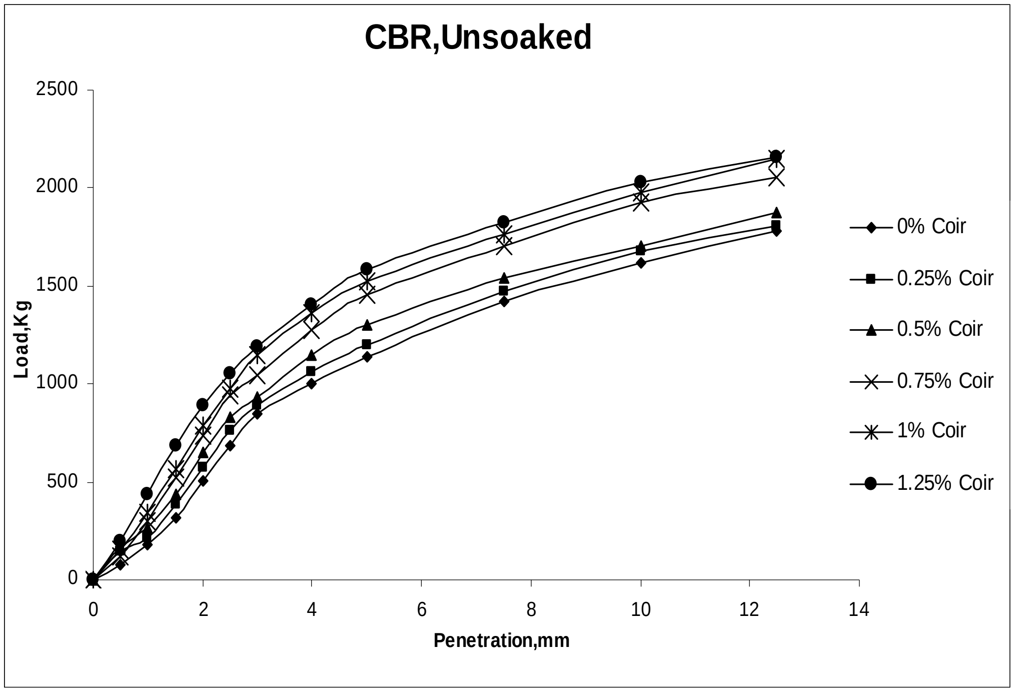

Fig. 4.5e Load-Penetration Characteristics of Un-Reinforced (Un-Soaked Soil), and Soil with Various Percentages of RDNFRS of Coir for 50%L+50%S

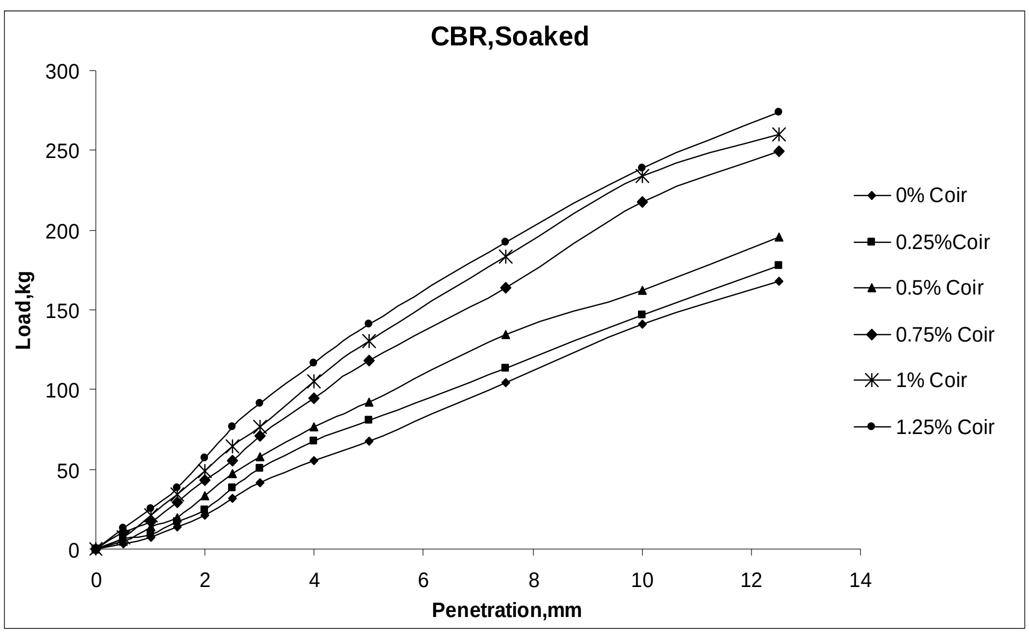

Fig. 4.5f Load-Penetration Characteristics of Un-Reinforced (Soaked Soil), and Soil with Various Percentages of RDNFRS of Coir for 50%L+50%S

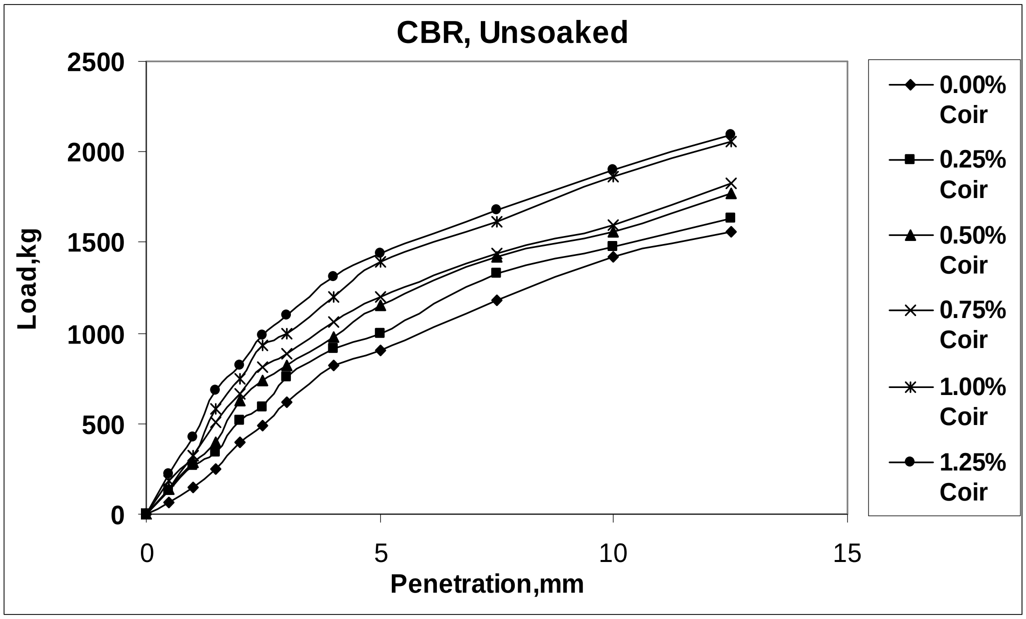

Fig. 4.5g Load-Penetration Characteristics of Un-Reinforced (Un-Soaked Soil), and Soil with Various Percentages of RDNFRS of Coir for 25%L+75%S

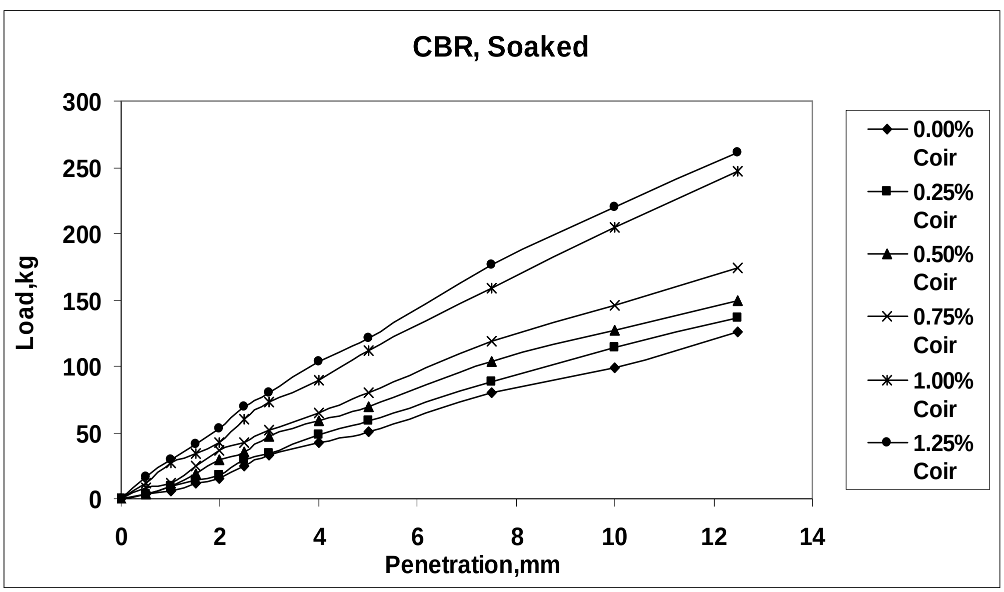

Fig. 4.5h Load-Penetration Characteristics of Un-Reinforced (Soaked Soil), and Soil with Various Percentages of RDNFRS of Coir for 25%L+75%S

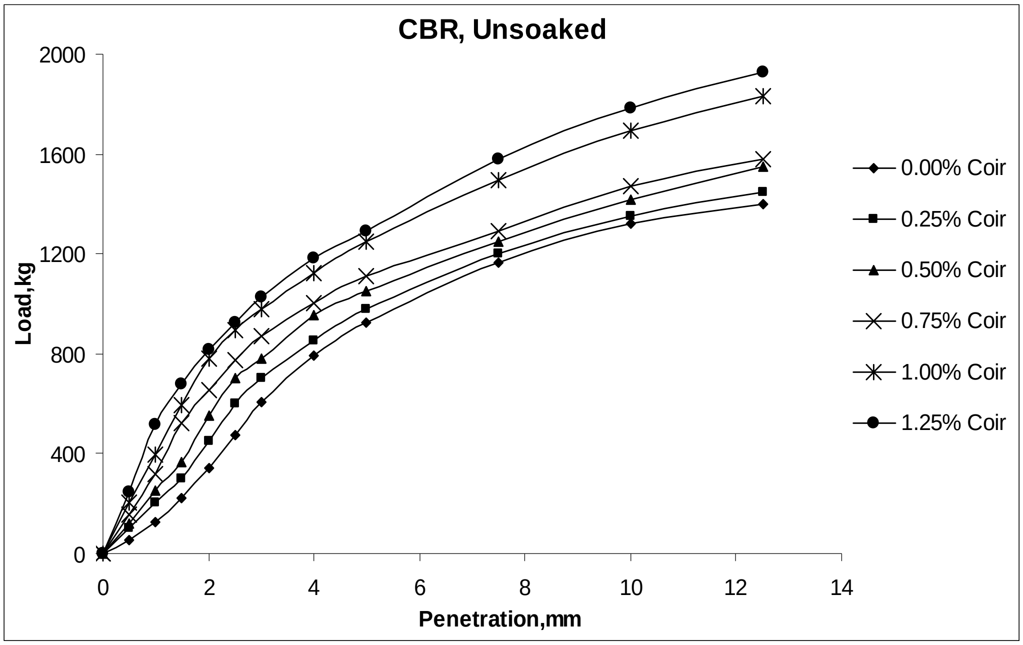

Fig. 4.5i Load-Penetration Characteristics of Un-Reinforced (Un-Soaked Soil), and Soil with Various Percentages of RDNFRS of Coir for 0%L+100% S

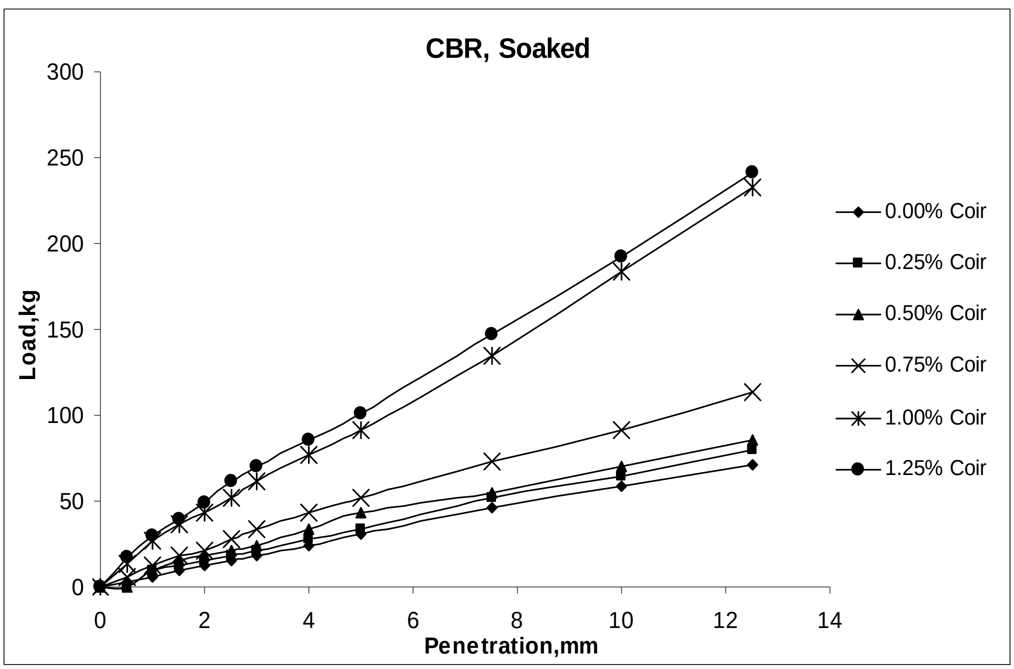

Fig. 4.5j Load-Penetration Characteristics of Un-Reinforced (Soaked Soil), and Soil with Various Percentages of RDNFRS of Coir for 0%L+100% S

4.4 TESTS FOR CONSOLIDATION

The test for accelerated consolidation involves several stages such as, preparation of the soil sample, soaking of specimens, loading, and installation of vertical drains and preloading of soil samples. Soil sample was spread over a level ground was sun-dried for 10 days. The tests for OMC and MDD were performed, and the water contents required to prepare soil beds at 80% MDD were determined using the compaction curves. It was decided to perform tests at moisture content lesser than the OMC in order to study the load-settlement characteristics effectively.

4.4.1 Tests for Consolidation of Un-Reinforced Soil without using Vertical Drains

In order to study the compressibility and consolidation of soil sample, the following test setup was adopted. A Ferro-cement cylindrical test mould of 740 mm internal diameter, 850 mm height, and 30 mm wall thickness was used for conducting the test. The cylindrical test mould was provided with an inlet pipe at the top and an outlet pipe at the bottom to permit soaking of the soil sample, and drainage of water. These pipes were of 20 mm diameter. The cylindrical mould was placed on leveled ground. A schematic diagram of the test set up is shown in **Fig. 4.6**.

A sand layer of 100 mm thickness was provided at the bottom of the Ferro-cement tank, and was compacted to a density of 1.53 g/cc. Above this sand layer, a jute textile layer was provided as a separation layer. Over this, three layers of the soil sample, each of 200 mm thickness, were placed and compacted to 80% of the MDD. The soil and the sand layers were compacted to the respective densities using a steel rammer (of 885 mm height, 140 mm diameter, and 11.5 kg weight) and a wooden rammer (of 870 mm height, 40 mm diameter, and 1.17 kg weight). On top of the three layers of compacted soil sample, a layer of jute textile was placed and a layer of sand of 100mm thickness compacted to a density of 1.53 g/cc was provided. The sand layer at the top was used to provide a level-surface on which the load for consolidation was to be applied.

The sand bedding material used at the top and bottom of the soil sample in the cylindrical test mould also possesses the same characteristics as mentioned in the sections above. A flat surface made of treated perforated plywood (of 730 mm diameter, and 12 mm thickness) was provided above the sand layer. Weighed concrete cubes and standard steel weights for preloading were placed on the plywood sheet. See **Fig.4.7a** to **Fig.4.7h** Steel railings were then fixed to the top portion of the cylindrical mould, and three dial gauges (of 0.01 mm least count) for measuring soil settlements, were attached. The average of the three readings was taken while measuring the settlements.

Components of the test setup:

1. Cylindrical Ferro-cement mould of 740 mm inner diameter, inner clear height of 850 mm, and 30 mm wall thickness.
2. Bottom layer of sand of 100 mm thickness, hand compacted to 1.53 g/cc using a wooden tamping tool of base of 120 mm diameter and 10mm thickness, provided with a wooden handle of 1000 mm height and 25mm square cross-section.
3. Bottom layer of jute-textile for separation of soil sample and sand layer.
4. Three layers of soil sample, each of 200 mm thickness, compacted to a density using mild steel tamping rod of base of 140 mm diameter and 75 mm thickness, provided with steel handle of 885 mm height and 25 mm diameter. The weight of tamping rod was 11.5 kg.

Fig 4.7a Ferro-cement Cylindrical Test Mould

Fig 4.7b Mould Filled with 10cm Thick Sand Layer and Compacted

Fig 4.7e Gunny-bag Fabric Provided on Top of the Soil-bed

Fig 4.7f Sand Layer of 10 cm Thickness above the Soil-bed

4.4.2 Tests on Consolidation of Un-Reinforced Soil using Three Vertical Drains

The test setup and procedure for test on consolidation of un-reinforced soil sample is the same as that explained in **Section 4.4.1**. A Ferro-cement cylindrical test mould with vertical drains is as shown in **Fig. 4.8a**.The vertical drains of 100 mm diameter and 600 mm height is installed in a triangular pattern such that the center to center distance between the adjacent drains is 350 mm. This arrangement is considered to be more effective as it is expected to result in uniform consolidation between the drains due to uniform center to center distances, when compared to vertical drains installed in a square pattern (**Holtz et al., 1991**). The radius of influence (R) of a vertical drain depends upon the spacing (S) between drains. In the case of cylindrical drains installed in triangular pattern the radius of influence (R) can be computed from the empirical formula (**Stapelfeldt, 2006**),

Fig. 4.8a Ferro-cement Test Mould with Accessories for Accelerated Consolidation

The cylindrical test mould was then filled with the next two layers of soil sample up to a total height of 600 mm, and the sampling tubes were gradually withdrawn in various stages while the vertical drains were filled. Finally the sampling tubes were completely lifted and taken out leaving the jute textile sleeves filled with the drainage material held in the required positions. Since the vertical drains were installed in build up procedures, the smear effects are assumed to be negligible. On top of the 3 layers of compacted soil sample, a layer of jute textile was placed and a layer of sand of 100 mm thickness compacted to a density of 1.53 g/cc was provided. A flat surface made of treated perforated plywood as mentioned in **Section 4.4.1** was provided above the sand layer. Weighed concrete cubes and standard steel weights were used in preloading the soil sample. The load settlement characteristics were then observed as mentioned in **Section 4.4.1**.

Components of the test set up:

1. Cylindrical Ferro-cement mould of 740 mm inner diameter, inner clear height of 850 mm, and 30 mm wall thickness.
2. Bottom layer of sand of 100 mm thickness, hand compacted to 1.53 g/cc using a wooden tamping tool of base of 120 mm diameter and 10mm thickness, provided with a wooden handle of 1000 mm height and 25mm square cross-section.
3. Bottom layer of jute-textile for separation of soil sample and sand layer.

   Three layers of soil sample, each of 200 mm thickness, compacted to a density using mild steel tamping rod of base of 140 mm diameter and 75 mm thickness,

4. Three layers of soil sample, each of 200 mm thickness, compacted to a density using mild steel tamping rod of base of 140 mm diameter and 75 mm thickness, provided with steel handle of 885 mm height and 25 mm diameter. The weight of tamping rod was 11.5 kg.
5. Top layer of jute-textile for separation of soil sample and sand layer.
6. Top layer of sand of 100 mm thickness, hand-compacted and leveled using wooden tamping tool with a handle of 1000mm height as mentioned above.

4.4.3 Consolidation of Randomly Reinforced Soil using Three Vertical Drains

In this part of the experiment, the soil sample was randomly reinforced using coir. The optimum fiber content was adopted in the tests for reinforced soil samples. The overall experimental setup remains the same as explained in **Section 4.4.2.**

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1 INTRODUCTION

Tests were performed for soil compacted in ferro-cement cylindrical moulds of 70 cm dia and 85 cm internal height for various blends of laterite and shedi soils in the laboratory. The soil samples for laterite/ laterite-shedi/ shedi were carefully obtained by the method of quartering. Standard soil tests on soils, such as the grain-size analysis, Atterberg limits, hydrometer analyses, tests for specific gravity, tests for standard and modified proctor tests, and CBR tests were then performed. It may be observed here that the following laterite –shedi soil mixes were studied.

- 100% Laterite & *0%* Shedi (*100%L+0%S*),
- 75%Laterite & 25% Shedi(75%L+25%S)
- 50%Laterite & 50% Shedi(50%L+50%S)
- 25%Laterite & 75% Shedi(25%L+75%S)
- 0%Laterite & 100% Shedi(0%L+100%S) as specified by Indian Standard (IS) codes.

The soil sample to be tested was compacted to 80% of the maximum dry density (MDD) in three layers of 20 cm each, for a total compacted soil thickness of 60cm. A preload of 50kg was applied, and the load settlement characteristics were observed. The next higher preload of 100 kg was applied when the settlement rate was lesser than 0.02mm. In a similar manner, tests were performed for preloads of 150kg, 200kg, 250kg, 300kg, 350kg, 400kg, 450kg, and 500kg. The results were tabulated, and the load-settlement characteristics were analyzed using the square-root of time v/s dial-gauge readings approach. The coefficient of consolidation (C_v) was then determined for each preload. The above tests were performed for the following conditions:

- Un-reinforced soil (*UR*)
- Un-reinforced soil with vertical drains (*UR-VD*)

- Randomly reinforced soil with vertical drains (*RR-VD*)

Tests on drained soil samples were performed by providing 3 vertical sand drains reinforced with coir fibers. Studies on reinforced soils were performed by soil samples randomly with coir fibers. This chapter provides details on the results on load-settlement characteristics for the above investigations performed.

5.2 RESULTS ON SETTLEMENT CHARACTERISTICS FOR *UR, UR-VD,* AND *RR-VD* SOILS FOR VARIOUS BLENDS

In the following sections, it was considered ideal to provide details on the results pertaining to the investigations performed in the following sequence, so as to enable making comparative studies and inferences in a systematic manner.

5.2.1 Settlement Characteristics for *UR, UR-VD,* and *RR-VD* for *100%L+ 0%S*

5.2.1.1 Settlement characteristics for *UR (100%L+0%S)*

The soil sample was originally compacted to 80% of the density at OMC before the commencement of the load-settlement test. The preloads of 50 to 500 kg were then applied in various stages in order to study the settlement characteristics for *UR* soil conditions.. **Table 5.1** provides details on the coefficient of consolidation (C_v) for various preloads for the soil sample tested. Also, it can be observed that during the initial stages of loading using preloads of 50kg, 100kg, 150kg, and 200kg, the compaction of soil is considered to have taken place. This can be visualized from the corresponding increase in the rate of compaction in the above table. Thereafter, the rate of compaction for preloads of 250kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.1** provides details on the load-settlement characteristics for un-reinforced soils for 100%L + 0%S.

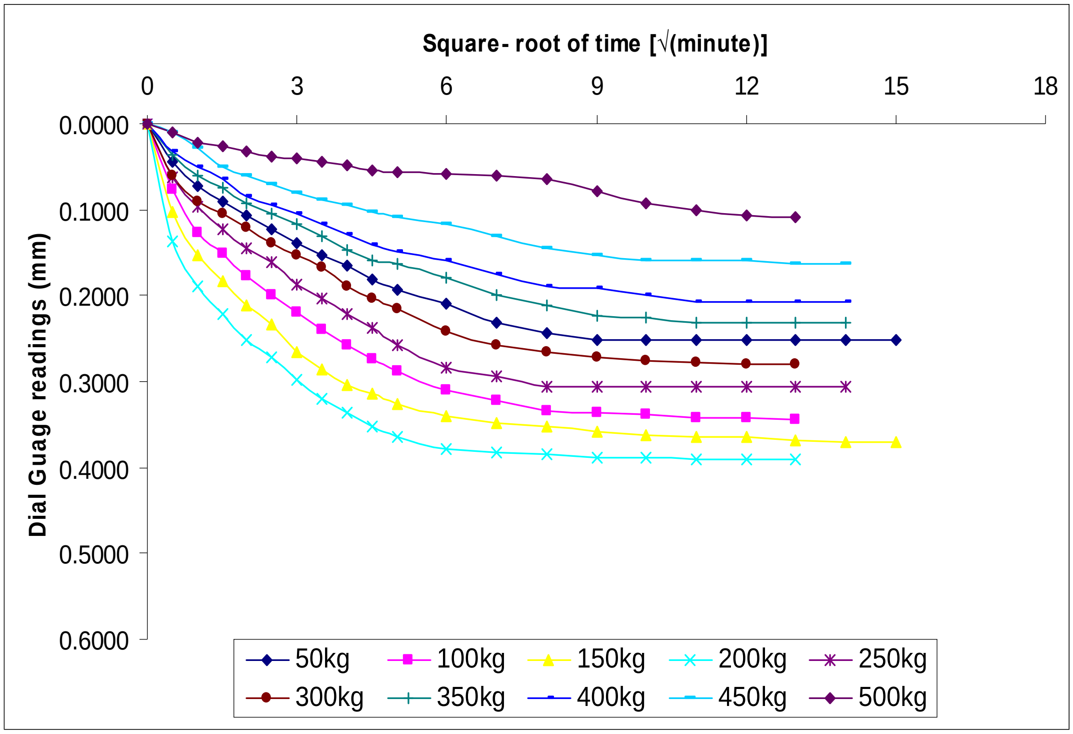

Fig. 5.1 Load-Settlement Curves for *UR (100%L+0%S)* Soils

5.2.1.2 Settlement characteristics for *UR-VD (100%L+0%S)*

As mentioned in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *UR-VD* soil conditions. **Table 5.2** provides details on the coefficient of consolidation (C_v) for various preloads for the soil sample tested. It can be observed that during the initial stages of loading, as in the case of preloads of 50kg, 100kg, and 150kg, the compaction of soil is considered to have taken place. Thereafter, the rate of compaction for preloads of 200kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.2** provides details on the load-settlement characteristics for un-reinforced vertically drained soils comprising 100% Laterite.

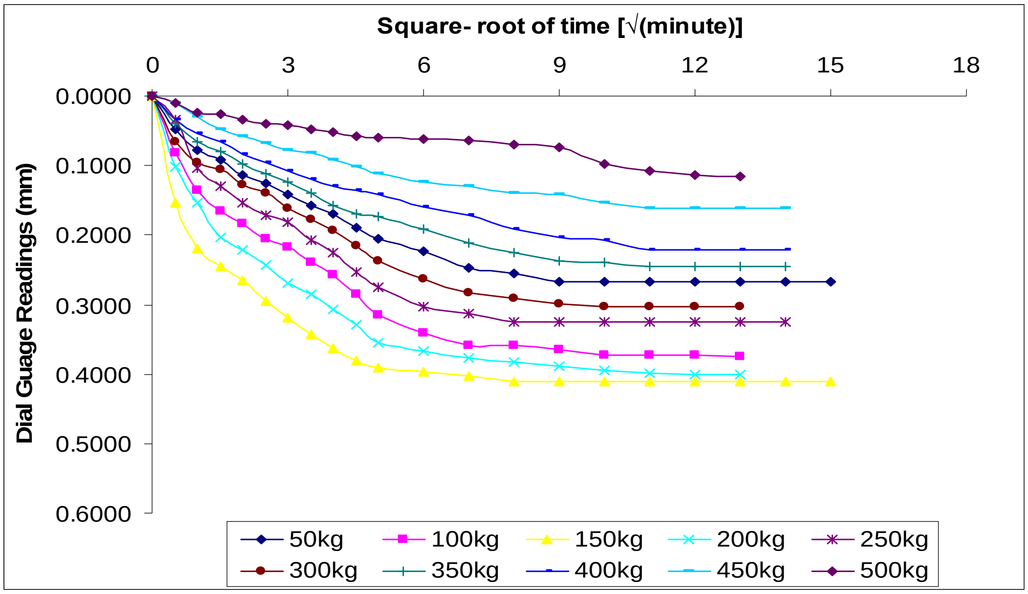

Fig. 5.2 Load-Settlement Curves for *UR-VD (100%L+0%S)* Soils

5.2.1.3 Settlement characteristics for *RR-VD (100%L+0%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *RR-VD* soil conditions. **Table 5.3** gives details on the coefficient of consolidation (C_v) for various preloads. It can be observed that in the initial stages of loading, the behavior of the soil is almost the same as observed in the case of ***UR-VD (100%L+0%S)*** soil. The compaction of the soil is considered to have taken place at the preloads of 50kg, 100kg, and 150kg. Thereafter, the rate of compaction for preloads of 200kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig.5.3** illustrates details on the load-settlement characteristics for randomly-reinforced vertically drained soils comprising 100% Laterite.

5.2.2 Settlement Characteristics for *UR, UR-VD,* and *RR-VD (75%L +25%S)*

5.2.2.1 Settlement characteristics for *UR (75%L+25%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *UR* soil conditions. **Table 5.4** provides details on the coefficient of consolidation (C_v) for various preloads for the soil sample tested. It can be observed that during the initial stages of loading using preloads of 50kg, 100kg, 150kg, and 200kg, the compaction of soil is considered to have taken place. This is evident from the corresponding increase in the rate of compaction in the above table. Thereafter, the rate of compaction for preloads of 300kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.4** provides details on the load-settlement characteristics for un-reinforced soils comprising 75% laterite and 25% shedi soil.

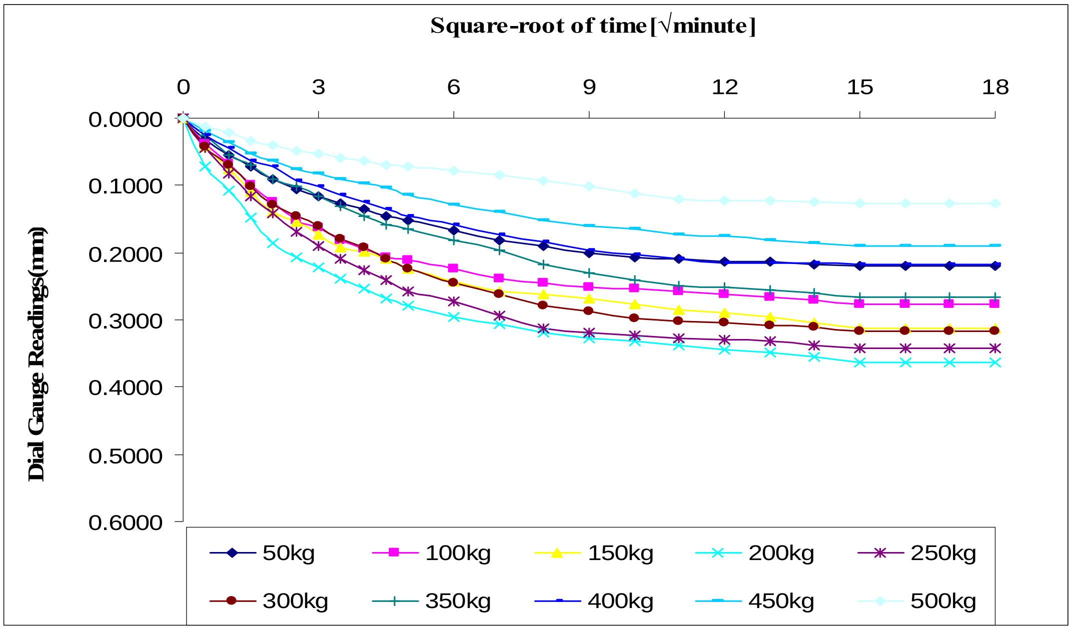

Fig. 5.3 Load-Settlement Curves for *RR-VD (100%L+0%S)* Soils

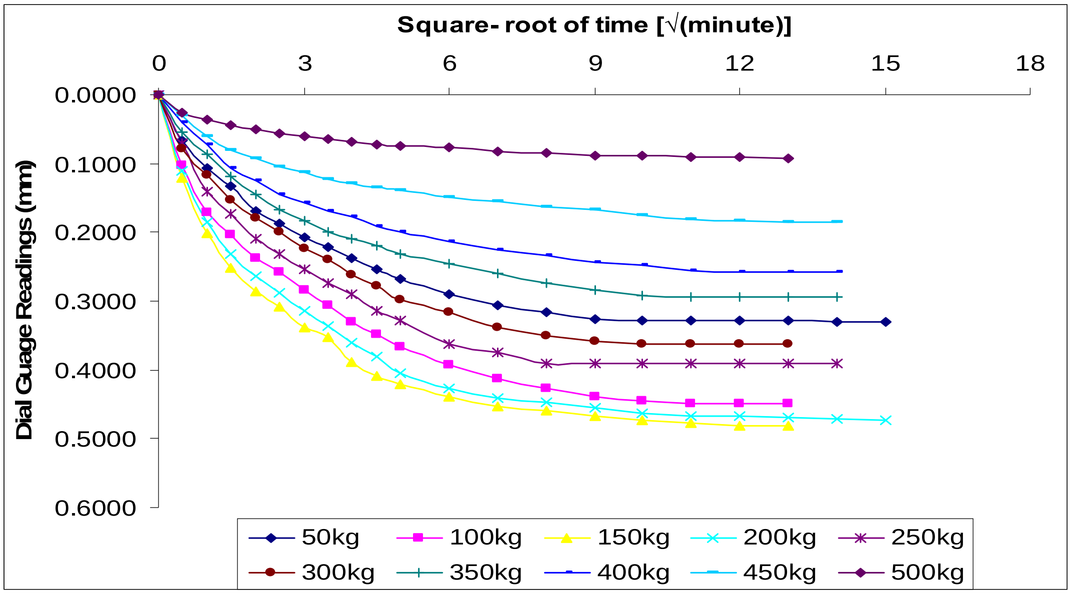

Fig. 5.4 Load-Settlement Curves for *UR (75%L+25%S)* Soils

5.2.2.2 Settlement characteristics for *UR-VD (75%L+25%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *UR-VD* soil conditions. **Table 5.5** provides details on the coefficient of consolidation (C_v) for various preloads. It can be observed that during the initial stages of loading, as observed in the case of 50kg, 100kg, and 150kg, the compaction of soil is considered to have taken place. Thereafter, the rate of compaction for preloads of 200kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.5** provides details on the load-settlement characteristics for un-reinforced vertically drained soils comprising 75% laterite and 25% shedi soil.

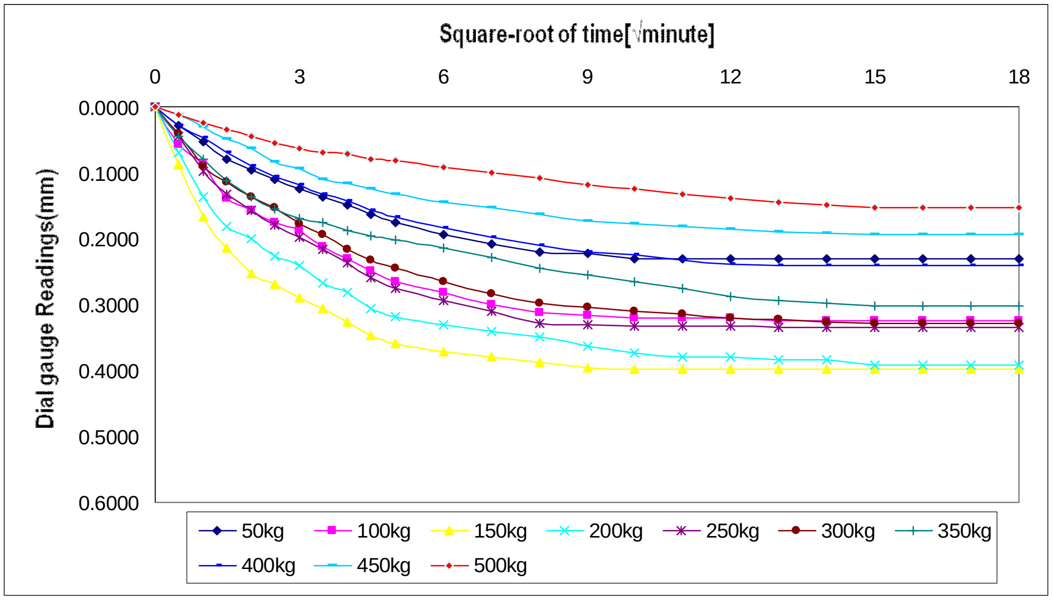

Fig. 5.5 Load-Settlement Curves for *UR-VD (75%L+25%S)* Soils

5.2.2.3 Settlement characteristics for *RR-VD (75%L+25%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *RR-VD* soil conditions. **Table 5.6** provides details on the coefficient of consolidation (C_v) for various preloads. It can be observed that during the initial stages of loading, as observed in the case of preloads of 50kg, 100kg, and 150kg, the compaction of soil is considered to have taken place. Thereafter, the rate of compaction for preloads of 200kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process, as in the case related to *UR-VD (100%L + 0%S)*. **Fig. 5.6** provides details on the load-settlement characteristics for randomly-reinforced vertically drained soils comprising 75% laterite and 25% shedi soil.

5.2.3 Settlement Characteristics for *UR, UR-VD,* and *RR-VD (50%L +50%S)*

5.2.3.1 Settlement characteristics for *UR (50%L+50%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *UR* soil conditions. **Table 5.7** provides details on the coefficient of consolidation (C_v) for various preloads for the soil sample tested. It can be observed that during the initial stages of loading using preloads of 50kg, 100kg, 150kg, 200kg, and 250kg, the compaction of soil is considered to have taken place.

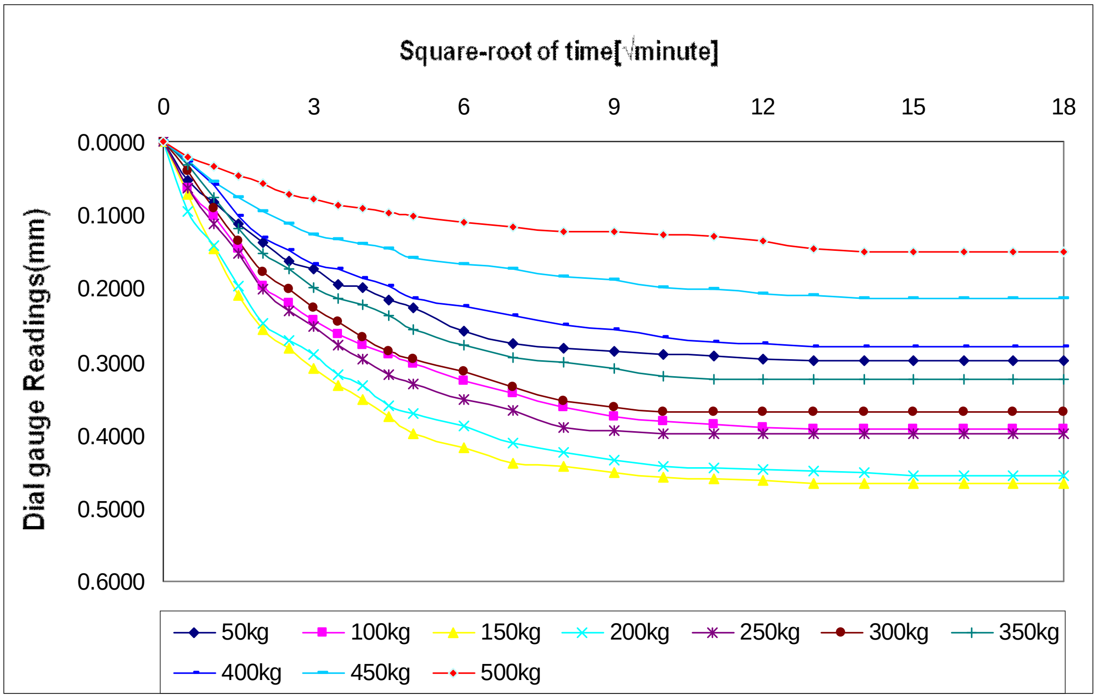

Fig. 5.6 Load-Settlement Curves for *RR-VD (75%L+25%S)* Soils

This can be visualized from the corresponding increase in the rate of compaction in the above table. Thereafter, the rate of compaction for preloads of 300kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.7** provides details on the load-settlement characteristics for un-reinforced soils comprising 50% laterite and 50% shedi soil.

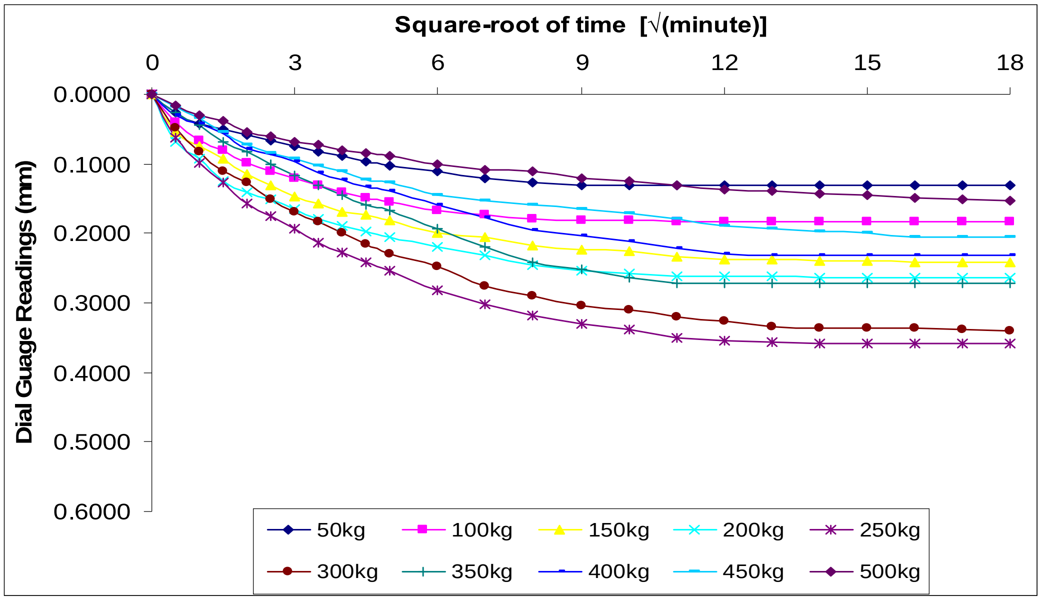

Fig. 5.7 Load-Settlement Curves for *UR (50%L+50%S)* Soils

5.2.3.2 Settlement characteristics for *UR-VD (50%L+50%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *UR-VD* soil conditions. **Table 5.8** provides details on the coefficient of consolidation (C_v) for various preloads. It can be observed that during the initial stages of loading, as observed in the case of 50kg, 100kg, and 150kg, the compaction of soil is considered to have taken place. Thereafter, the rate of compaction for preloads of 200kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.8** provides details on the load-settlement characteristics for un-reinforced vertically drained soils comprising 50% laterite and 50% shedi soil.

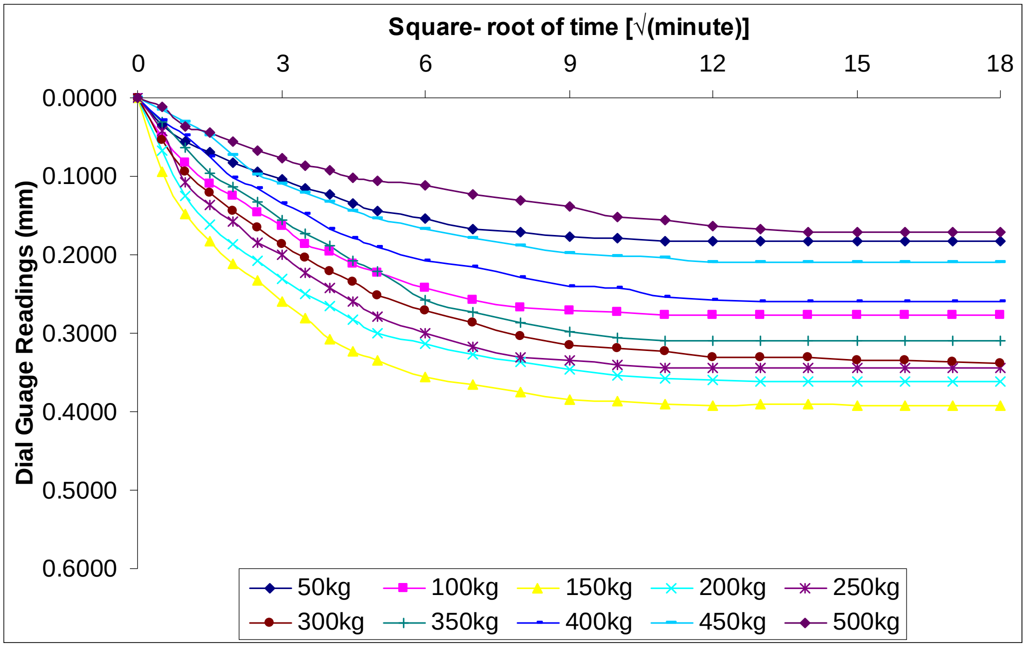

Fig. 5.8 Load-Settlement Curves for *UR-VD (50%L+50%S)* Soils

5.2.3.3 Settlement characteristics for *RR-VD (50%L+50%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *RR-VD* soil conditions. **Table 5.9** provides details on the coefficient of consolidation (C_v) for various preloads. It can be observed that during the initial stages of loading, as observed in the case of preloads of 50kg, 100kg, and 150kg, the compaction of soil is considered to have taken place. Thereafter, the rate of compaction for preloads of 200kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.9** provides details on the load-settlement characteristics for randomly-reinforced vertically drained soils comprising 50% laterite and 50% shedi soil.

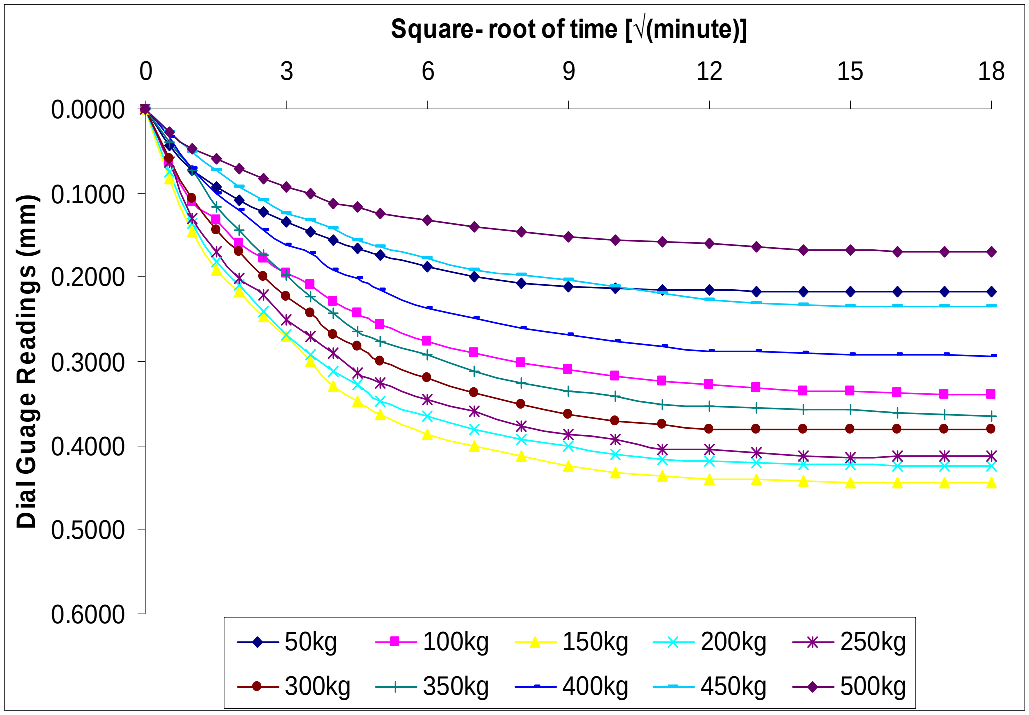

Fig. 5.9 Load-Settlement Curves for *RR-VD (50%L+50%S)* Soils

5.2.4 Settlement Characteristics for *UR, UR-VD,* and *RR-VD (25%L +75%S)*

5.2.4.1 Settlement characteristics for *UR (25%L+75%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *UR* soil conditions. **Table 5.10** provides details on the coefficient of consolidation (C_v) for various preloads for the soil sample tested. It can be observed that during the initial stages of loading using preloads of 50kg, 100kg, 150kg, 200kg, 250kg and 300kg, the compaction of soil is considered to have taken place.

This can be visualized from the corresponding increase in the rate of compaction in the above table. Thereafter, the rate of compaction for preloads of 350kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.10** provides details on the load-settlement characteristics for un-reinforced soils comprising 25% laterite and 75% shedi soil.

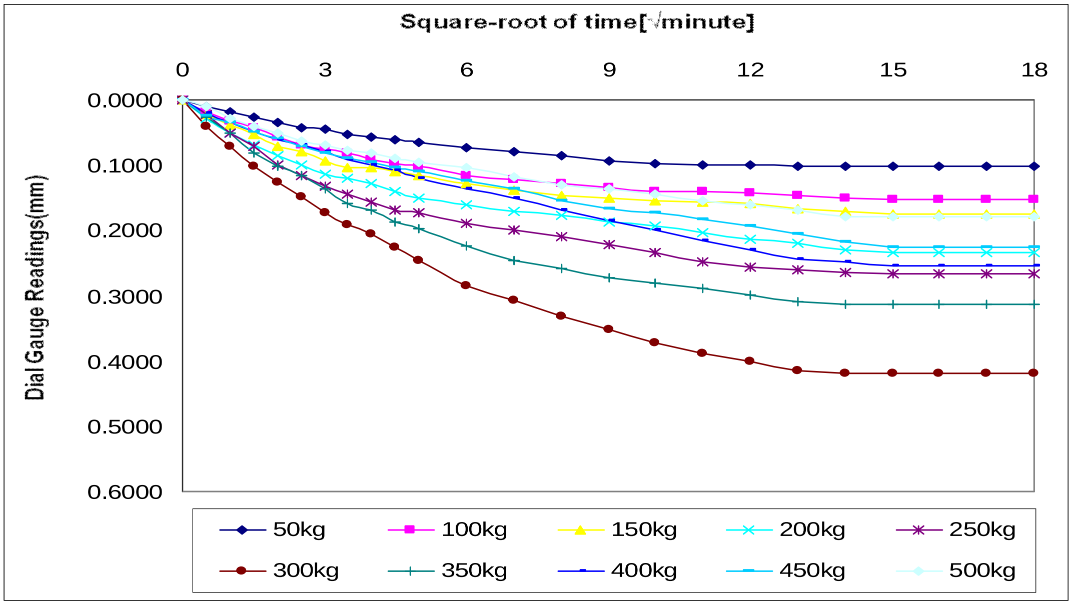

Fig. 5.10 Load-Settlement Curves for *UR (25%L+75%S)* Soils

5.2.4.2 Settlement characteristics for *UR-VD (25%L+75%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *UR-VD* soil conditions. **Table 5.11** provides details on the coefficient of consolidation (C_v) for various preloads. It can be observed that during the initial stages of loading, as observed in the case of 50kg, 100kg, 150kg and 200kg, the compaction of soil is considered to have taken place. Thereafter, the rate of compaction for preloads of 250kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.11** provides details on the load-settlement characteristics for un-reinforced vertically drained soils comprising 50% laterite and 50% shedi soil.

Table 5.11 C_v Values for *UR-VD (25%L+75%S)* Soils

Co-efficient of consolidation, C_v		
Load in kg	**Pressure (10^{-3} N/mm^2)**	C_v (cm^2/s or x10^2 mm^2/s)
0-50	0-1.2	1.0379
50-100	1.2-2.3	1.2812
100-150	2.3-3.5	1.4602
150-200	3-5-4.6	1.6798
200-250	4.6-5.8	1.5103
250-300	5.8-7.0	1.4600
300-350	7.0-8.1	1.3661

350-400	8.1-9.3	1.1326
400-450	9.3-10.5	1.0994
450-500	10.5-11.6	1.0681

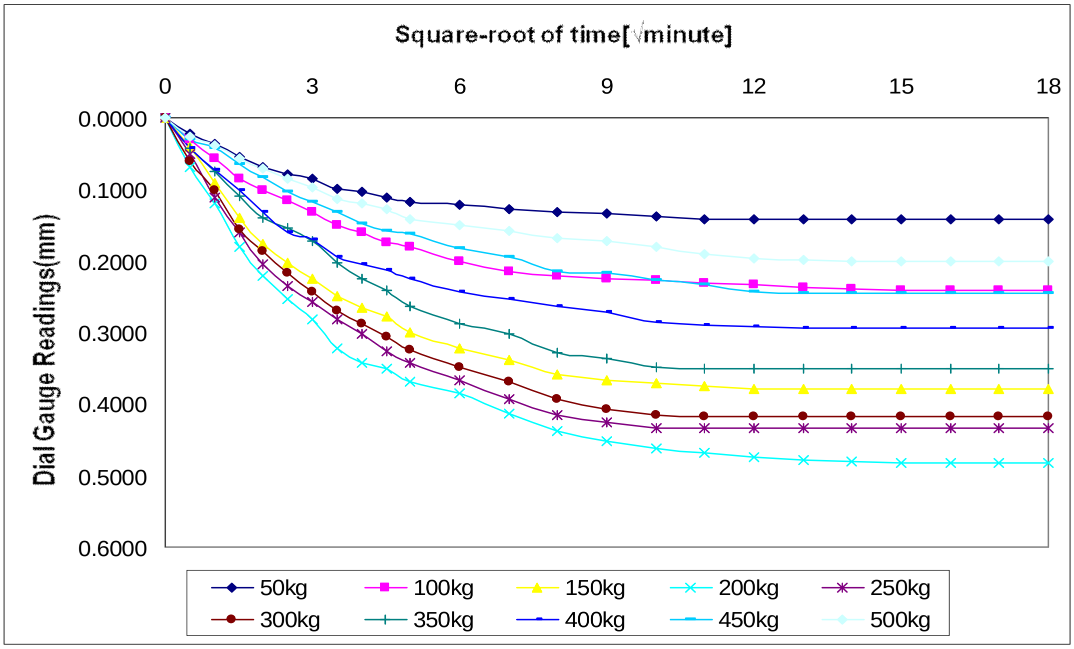

Fig. 5.11 Load-Settlement Curves for *UR-VD (25%L+75%S)* Soils

5.2.4.3 Settlement Characteristics for *RR-VD (25%L+75%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *RR-VD* soil conditions. **Table 5.12** provides details on the coefficient of consolidation (C_v) for various preloads. It can be observed that during the initial stages of loading, as observed in the case of preloads of 50kg, 100kg, 150kg and 200kg, the compaction of soil is considered to have taken place. Thereafter, the rate of compaction for preloads of 250kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.12** provides details on the load-settlement characteristics for randomly-reinforced vertically drained soils comprising 50% laterite and 50% shedi soil.

5.2.5 Settlement Characteristics for *UR, UR-VD,* and *RR-VD* for *0%L+100%S*

5.2.5.1 Settlement characteristics for *UR (0%L+100%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *UR* soil conditions. **Table 5.13** provides details on the coefficient of consolidation (C_v) for various preloads for the soil sample tested.

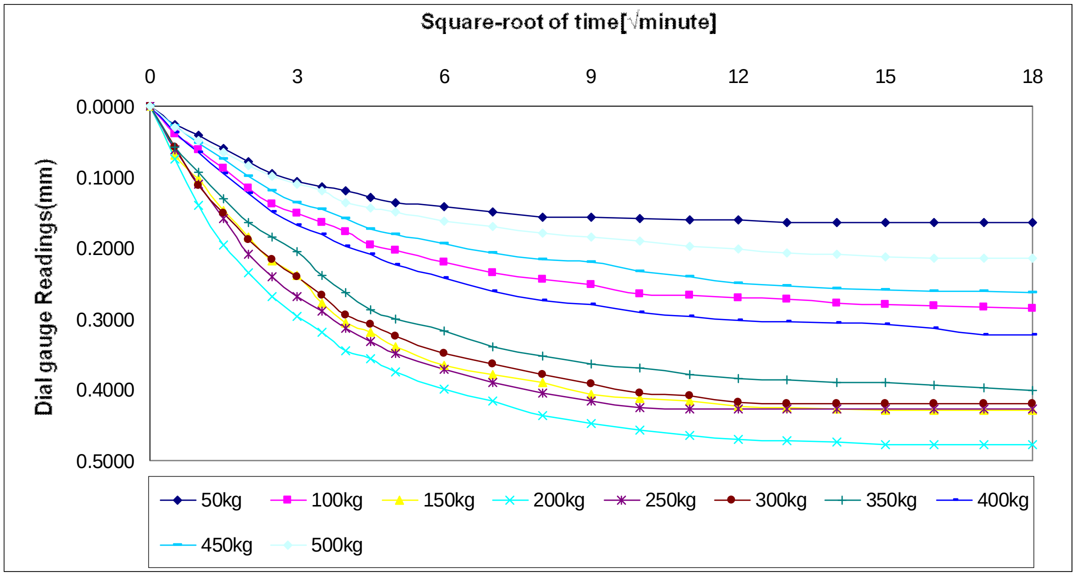

Fig. 5.12 Load-Settlement Curves for *RR-VD (25%L+75%S)* Soils

It can be observed that during the initial stages of loading using preloads of 50kg, 100kg, 150kg, 200kg, 250kg and 300kg, the compaction of soil is considered to have taken place. This can be visualized from the corresponding increase in the rate of compaction in the below table. Thereafter, the rate of compaction for preloads of 350 kg to 500kg shows a decreasing trend, indicating the commencement of consolidation process. **Fig.5.13** provides details on the load-settlement characteristics for un-reinforced shedi soil.

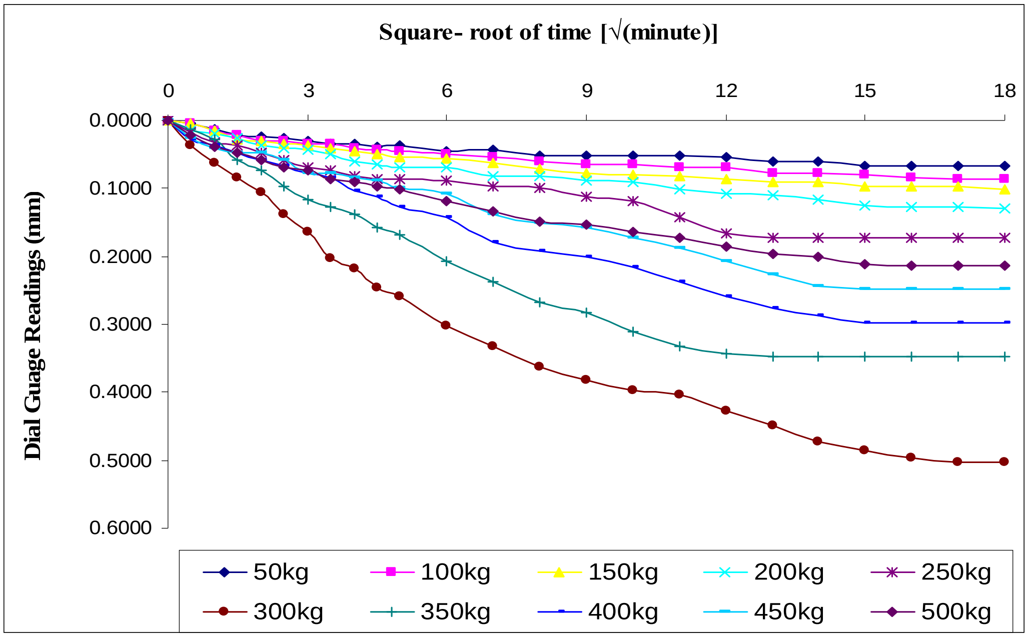

Fig. 5.13 Load-Settlement Curves for *UR (0%L+100%S)* Soils

5.2.5.2 Settlement characteristics for *UR-VD (0%L+100%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *UR-VD* soil conditions. **Table 5.14** provides details on the coefficient of consolidation (C_v) for various preloads for the soil sample tested. It can be observed that during the initial stages of loading, as in the case of preloads of, 150kg 50kg, 100kg and 200kg, the compaction of soil is considered to have taken place. Thereafter, the rate of compaction for preloads of 250kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process. **Fig. 5.14** provides details on the load-settlement characteristics for un-reinforced vertically drained soils comprising 100% Shedi.

Table 5.14 C_v Values for *UR-VD (0%L+100%S)* Soils

Co-efficient of consolidation, C_v		
Load in kg	**Pressure (10^{-3} N/mm^2)**	C_v (cm^2/s or x10^2 mm^2/s)
0-50	0-1.2	1.0214
50-100	1.2-2.3	1.1673
100-150	2.3-3.5	1.3661
150-200	3-5-4.6	1.5951
200-250	4.6-5.8	1.4939
250-300	5.8-7.0	1.4598
300-350	7.0-8.1	1.3233

350-400	8.1-9.3	1.1326
400-450	9.3-10.5	1.0375
450-500	10.5-11.6	1.0088

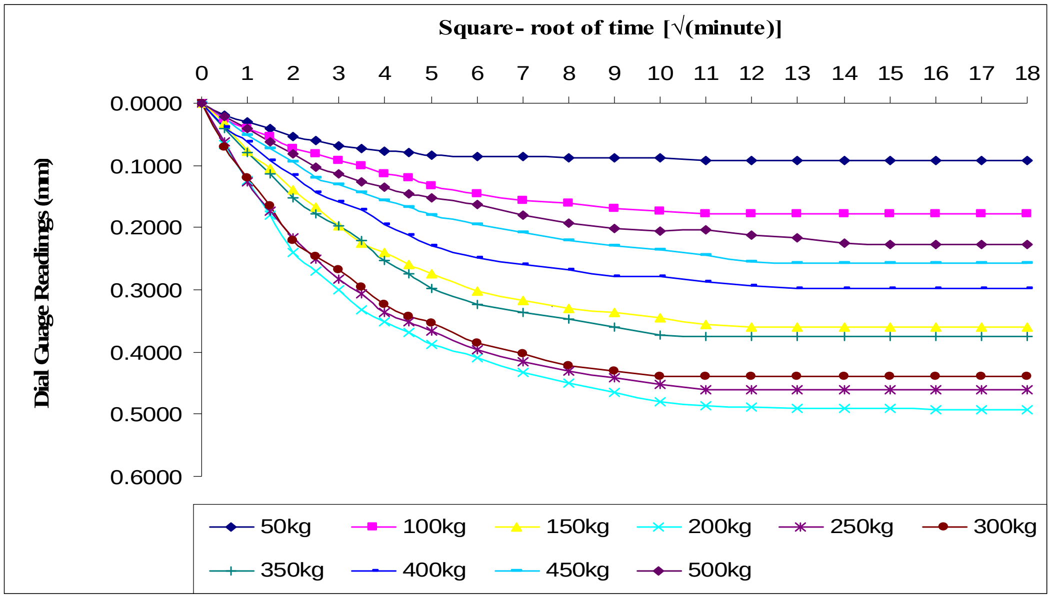

Fig. 5.14 Load-Settlement Curves for *UR-VD (0%L+100%S)* Soils

5.2.5.3 Settlement characteristics for *RR-VD (0%L+100%S)*

As in the above section, the preloads of 50 to 500 kg were applied in various stages in order to study the settlement characteristics for *RR-VD* soil conditions. **Table 5.15** provides details on the coefficient of consolidation (C_v) for various preloads. It can be observed that during the initial stages of loading, as in the case of preloads of 50kg, 100kg, 150kg and 200 kg the compaction of soil is considered to have taken place. Thereafter, the rate of compaction for preloads of 250kg to 500kg shows a decreasing trend, indicating the commencement of the consolidation process, as in the case related to *UR-VD (0%L + 100%S)*. **Fig. 5.15** provides details on the load-settlement characteristics for randomly-reinforced vertically drained soils comprising 100% Shedi.

Table 5.15 C_v Values for *RR-VD (0%L+100%S)* Soils

Co-efficient of consolidation, C_v		
Load in kg	**Pressure (10^{-3} N/mm^2)**	C_v (cm^2/s or x10^2 mm^2/s)
0-50	0-1.2	1.0092
50-100	1.2-2.3	1.3240
100-150	2.3-3.5	1.8277

150-200	3-5-4.6	1.8782
200-250	4.6-5.8	1.5638
250-300	5.8-7.0	1.5101
300-350	7.0-8.1	1.4113
350-400	8.1-9.3	1.2032
400-450	9.3-10.5	1.0993
450-500	10.5-11.6	1.0208

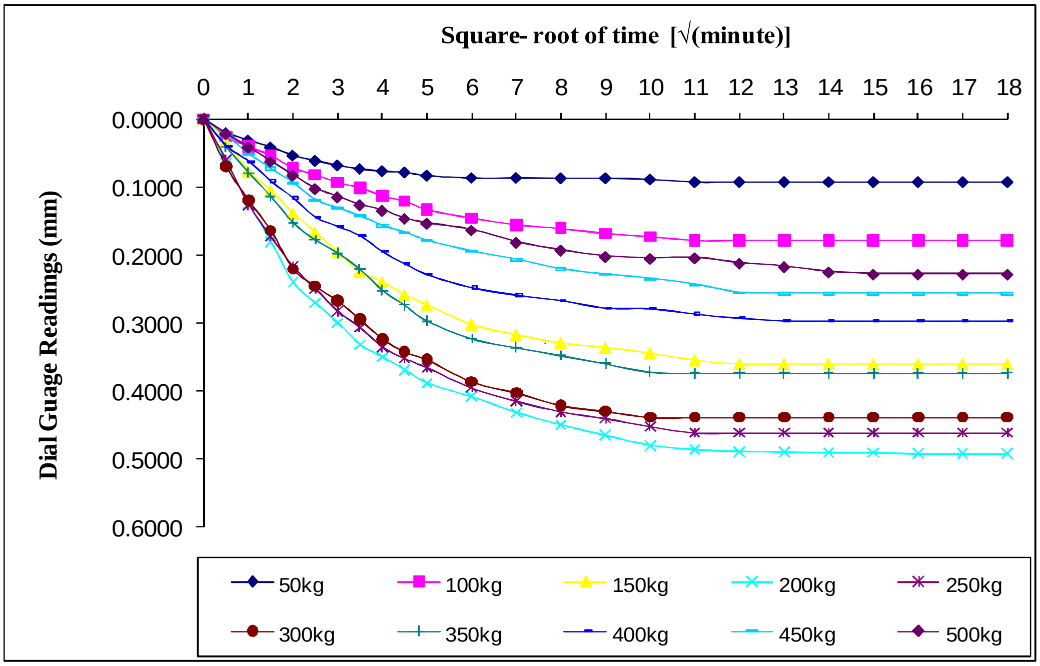

Fig. 5.15 Load-Settlement Curves for *RR-VD (0%L+100%S)* Soils

5.3 COMPARISON OF *Cv* VALUES FOR *UR*, *UR-VD*, AND *RR-VD* SOILS FOR VARIOUS BLENDS

5.3.1 Comparison of *Cv* Values for *UR*, *UR-VD*, and *RR-VD*: *100%L+0%S*

The values for *Cv* were recompiled as shown in **Table 5.16** for *UR*, *UR-VD*, and *RR-VD* cases for *100%L + 0%S* soils. Here, it can be observed that in the case of *UR* soil conditions, the maximum value of C_v of 2.0825 cm^2/s was found to occur at a higher pressure range of 3.5 to 4.6 x10^{-3} N/mm^2.

In the case of *UR-VD* soil conditions, the maximum value of C_v of 2.2683 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 x10^{-3} N/mm^2. Also, in the case of *RR-VD* soil conditions, the maximum value of C_v of 2.2882 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 x10^{-3} N/mm^2.

5.3.2 Comparison of *Cv* Values for *UR, UR-VD,* and *RR-VD*: *75%L+25%S*

The values for *Cv* were recompiled as shown in **Table 5.17** for *UR*, *UR-VD*, and *RR-VD* cases for *75%L + 25%S* soils. Here, it can be observed that in the case of *UR* soil conditions, the maximum value of C_v of 1.9546 cm^2/s was found to occur at a higher pressure range of 3.5 to 4.6 $x10^{-3}$ N/mm^2. In the case of *UR-VD* soil conditions, the maximum value of C_v of 2.1172 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. Also, in the case of *RR-VD* soil conditions, the maximum value of C_v of 2.204 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2.

5.3.3 Comparison of *Cv* Values for *UR, UR-VD,* and *RR-VD*: *50%L+50%S*

The values for *Cv* were recompiled as shown in **Table 5.18** for *UR*, *UR-VD*, and *RR-VD* cases for *50%L + 50%S* soils. Here, it can be observed that in the case of *UR* soil conditions , the maximum value of C_v of 1.4280 cm^2/s was found to occur at a higher pressure range of 3.5 to 4.6 $x10^{-3}$ N/mm^2. In the case of *UR-VD* soil conditions, the maximum value of C_v of 2.0164 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. Also, in the case of *RR-VD* soil conditions, the maximum value of C_v of 2.0322 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2.

Table 5.18 Co-efficient of Consolidation C_v for *50%L+50%S*

Co-efficient of Consolidation C_v (cm^2/s)				
Load in kg	Pressure (10^{-3} N/mm^2)	UR	UR-VD	RR-VD
0-50	0-1.2	1.2109	1.0594	1.0681
50-100	1.2-2.3	1.2712	1.3965	1.3962
100-150	2.3-3.5	1.3103	2.0164	2.0322
150-200	3-5-4.6	1.4280	1.7053	1.9542
200-250	4.6-5.8	1.6500	1.5686	1.6079
250-300	5.8-7.0	1.5126	1.5308	1.3980
300-350	7.0-8.1	1.3377	1.4606	1.3160
350-400	8.1-9.3	1.2723	1.1554	1.1826
400-450	9.3-10.5	1.2462	1.1249	1.0076
450-500	10.5-11.6	1.1827	1.0420	1.0090

5.3.4 Comparison of *Cv* Values for *UR, UR-VD,* and *RR-VD*: *25%L+75%S*

The values for *Cv* were recompiled as shown in **Table 5.19** for *UR*, *UR-VD*, and *RR-VD* cases for *25%L + 75%S* soils. Here, it can be observed that in the case of

UR soil conditions for 100%L + 0%S soils, the maximum value of C_v of 1.2813 cm^2/s was found to occur at a higher pressure range of 3.5 to 4.6 $x10^{-3}$ N/mm^2. In the case of *UR-VD* soil conditions, the maximum value of C_v of 1.4602 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. Also, in the case of RR-VD soil conditions, the maximum value of C_v of 1.8790 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2.

5.3.5 Comparison of *Cv* Values for *UR, UR-VD,* and *RR-VD*: *0%L+100%S*

The values for *Cv* were recompiled as shown in **Table 5.20** for *UR*, *UR-VD*, and *RR-VD* cases for *0%L + 100%S* soils. Here, it can be observed that in the case of *UR* soil conditions for *0%L + 100%S* soils, the maximum value of C_v of 1.0821 cm^2/s was found to occur at a higher pressure range of 3.5 to 4.6 $x10^{-3}$ N/mm^2. In the case of *UR-VD* soil conditions, the maximum value of C_v of 1.3661 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. Also, in the case of *RR-VD* soil conditions, the maximum value of C_v of 1.8277 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2.

5.4 COMPARISON OF RELATIVE CONSOLIDATIONS *UR* vs *UR-VD* and *UR* vs *RR-VD* FOR VARIOUS SOIL BLENDS

5.4.1 *UR* vs *UR-VD,* and *UR* vs *RR-VD*: *100%L+0%S*

Fig.5.16 provides details of the load-settlement trends for a preload of 50kg for *UR*, *UR-VD*, and *RR-VD* test conditions for *100%L+0%S* soils. Similar figures can be obtained and studied for preloads of 100kg, 150kg, 200kg, 250kg, 300kg, 350kg, 400kg, 450kg, and 500kg. See **Appendix I**.

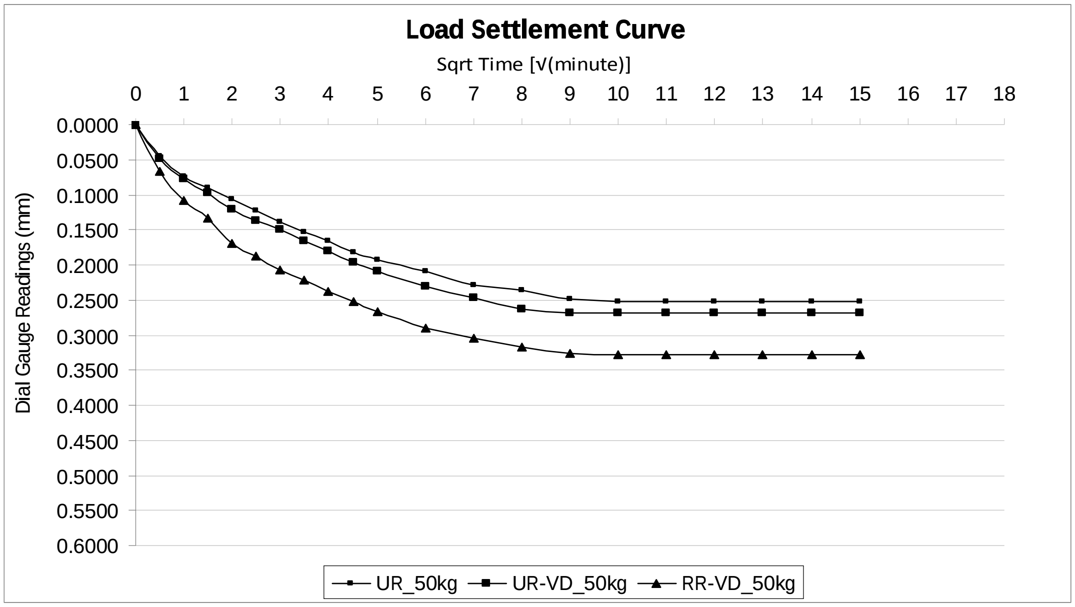

Fig. 5.16 Load-Settlement Trend for Preload of 50kg for *100%L+0%S* Soil

On observation of the load-settlement trends for various pre-loads, it was found that the 121st minute could be taken as a reference for comparison, since the soil was found to attain stability at this instance. In other words, the soil can be assumed to have been consolidated at this point in time. **Table 5.21** provides details on load settlements at the 121st minute after commencement of the tests for each preload for *100%L+0%S* soil mix.

Table 5.21 provides details on the relative increase in consolidation at different preloads at the 121st minute of the load settlements for *UR v/s UR-VD*, and *UR v/s RR-VD* observations. In this table, it is observed that at lower preloads of 50kg, 100kg, 150kg, 200kg, 250kg and 300kg, the R_{ct} *(UR v/s RR-VD)* is significantly higher when compared to R_{ct} *(UR v/s UR-VD)*. This is because of the additional drainage paths created by the randomly reinforced coir fibers. It is seen that within this range of preloads, the R_{ct} *(UR v/s RR-VD)* varies between 29.00% and 28.47%, while the R_{ct} *(UR v/s UR-VD)* varies from 6.38% to 8.47%.

5.4.2 *UR* vs *UR-VD*, and *UR* vs *RR-VD*: *75%L+25%S*

Fig.5.17 provides details of the load-settlement trends for a preload of 100 kg for *UR*, *UR-VD*, *RR-VD* test conditions for 75%L+25%S. From the observations on load settlement trends for different pre-loads, it was found that the soil stained stability at 169th minute after the commencement of the tests. **Table 5.22** provides details on relative increase in consolidation at different preloads at the 169th minute of settlement for UR *v/s* UR-VD, and UR *v/s* RR-VD observations. See **Appendix II**.

In **Table 5.22**, it is observed that at lower preloads of 50kg, 100kg, 150kg and 200kg, the R_{ct} *(UR v/s RR-VD)* is significantly higher when compared to R_{ct} *(UR v/s UR-VD)*. It is seen that within this range of preloads, the R_{ct} *(UR v/s RR-VD)* varies between 28.85% to 58.17%, while the R_{ct} *(UR v/s UR-VD)* varies from 7.14% to 34.94%.

At medium level preloads of 250kg to 400kg the relative advantages of providing vertical drains and random reinforcements to the soil layer are found to be moderate. The R_{ct} *(UR v/s RR-VD)* is found to range between 11.35% and 30.39%, while the R_{ct} *(UR v/s UR-VD)* varies between 5.12% and 17.43%.

Table 5.22 Relative Advantages at Different Preloads at 169th min. of *75%L+25%S*

Load in kg	**Pressure (10^{-3} N/mm^2)**	***Rct (UR vs UR-VD)* (%)**	***Rct (UR vs RR-VD)* (%)**
0-50	0-1.2	07.14	39.85
50-100	1.2-2.3	20.45	45.28
100-150	2.3-3.5	34.94	58.17
150-200	3-5-4.6	10.20	28.85
200-250	4.6-5.8	05.12	11.35
250-300	5.8-7.0	05.17	20.26
300-350	7.0-8.1	15.18	26.64
350-400	8.1-9.3	17.43	30.39
400-450	9.3-10.5	04.58	16.40
450-500	10.5-11.6	18.18	24.83

5.4.3 *UR* vs *UR-VD*, and *UR* vs *RR-VD*: *50%L+50%S*

Fig.5.18 provides details of the load-settlement trends for preloads of 50kg, 100kg, 150kg, 200kg, 250kg, 300kg, 350kg, 400kg, 450kg, and 500kg. **Table 5.23** provides details on load settlements at the 169th minute after commencement of the tests for each preload for *50%L+50%S* soil mix. The 169th minute is taken as a reference since the graphical trends indicate that the soil has attained stability consolidation at this instance of time. **Appendix III**.

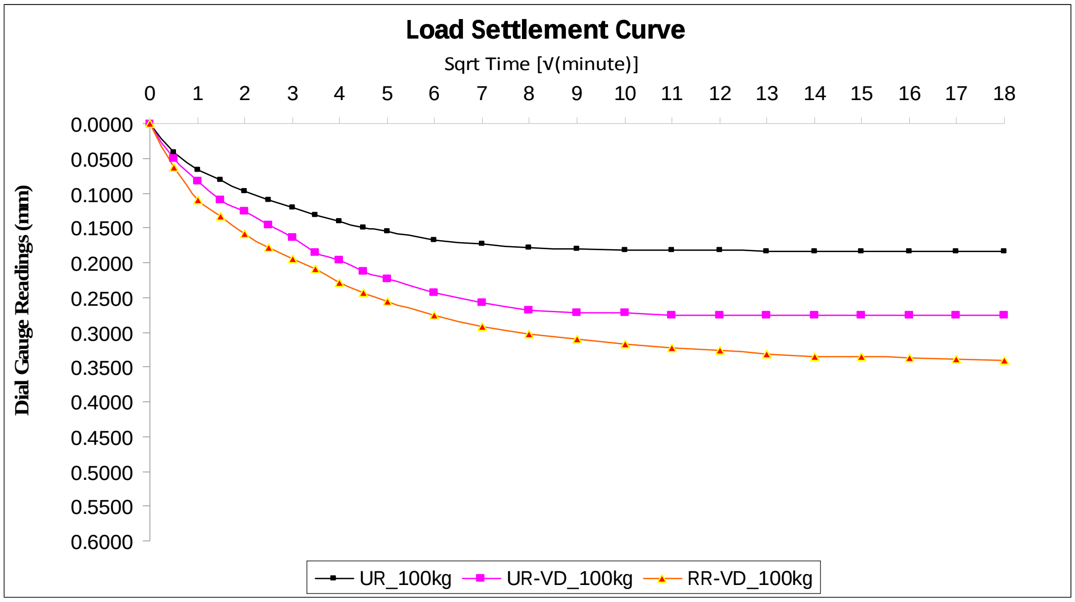

Fig. 5.18 Load-Settlement Trend for Preload of 100 kg for *50%L+50%S* Soil

For pre-loads ranging from 50kg to 200kg, the effect of providing vertical drains was significant when compared to the rate of settlement in the case of *UR* soils. The R_{ct} *(UR v/s UR-VD)* values within this range of pre-loads vary from 37.54% to 65.19%. But for pre-loads higher than 200kg, the effect of providing vertical sand drains was not significant as it was found to vary between 2.11% to 14.35%.

5.4.4 *UR* vs *UR-VD*, and *UR* vs *RR-VD*: *25%L+75%S*

Fig 5.19 provides details of the load-settlement trends for preloads of 50kg, 100kg, 150kg, 200kg, 250kg, 300kg, 350kg, 400kg, 450kg, and 500kg. **Table 5.24** provides details on load settlements at the 196th minute after commencement of the tests for each preload for *25%L+75%S* soil mix. The 196th minute is taken as a reference since the graphical trends indicate that the soil has attained stability/ consolidation at this instance of time. **Appendix IV**.

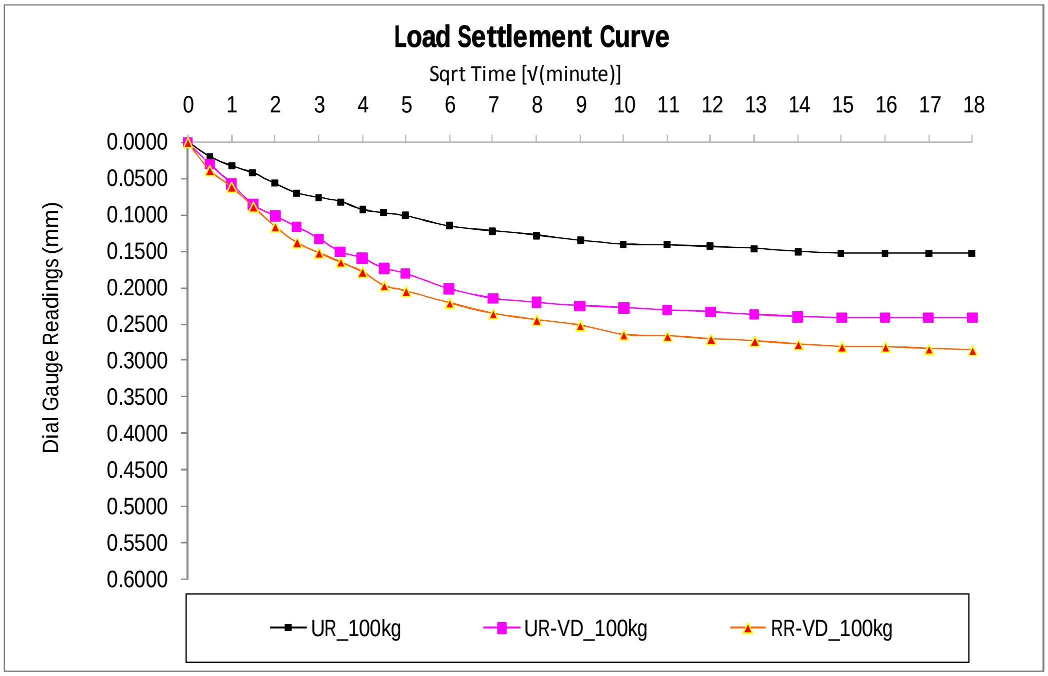

Fig. 5.19 Load-Settlement Trend for Preload of 100 kg for *25%L+75%S* Soil

In **Table 5.24**, it is observed that at lower preloads of 50kg, 100kg, 150kg , 200kg and 250kg, the R_{ct} *(UR v/s RR-VD)* is significantly higher when compared to R_{ct} *(UR v/s UR-VD)*. It is seen that within this range of preloads, the R_{ct} *(UR v/s RR-VD)* varies between 61.36% to 149.15%, while the R_{ct} *(UR v/s UR-VD)* varies from 40.91% to 120.85%.

Table 5.24 Relative Advantages at Different Preloads at 196th min. of *25%L+75%S*

Load in kg	Pressure (10^{-3} N/mm^2)	*Rct (UR vs UR-VD)* (%)	*Rct (UR vs RR-VD)* (%)
0-50	0-1.2	40.91	61.36
50-100	1.2-2.3	59.95	85.15
100-150	2.3-3.5	120.85	149.15
150-200	3-5-4.6	102.52	106.43
200-250	4.6-5.8	58.96	61.68
250-300	5.8-7.0	05.20	09.30
300-350	7.0-8.1	11.96	24.40
350-400	8.1-9.3	18.29	22.83
400-450	9.3-10.5	12.04	18.21
450-500	10.5-11.6	16.01	20.22

Thus it can be concluded that at higher preloads, the effect of reinforcing soils randomly with fibers is not effective when compared to the use of vertical drains in the case of *25%L+75%S* soils.

5.4.5 *UR* vs *UR-VD,* and *UR* vs *RR-VD*: *0%L+100%S*

Fig.5.20 provides details of the load-settlement trends for a preload of 50kg for UR, UR-VD, and RR-VD test conditions for *0%L+100%S* soils. Similar figures can be obtained and studied for preloads of 100kg, 150kg, 200kg, 250kg, 300kg, 350kg, 400kg, 450kg, and 500kg. See **Appendix V**.

In this table, it is observed that at lower preloads of 50kg, 100kg, 150kg, 200kg and 250kg, the R_{ct} *(UR v/s RR-VD)* is significantly higher when compared to R_{ct} *(UR v/s UR-VD)*. It is seen that within this range of preloads, the R_{ct} *(UR v/s RR-VD)* varies between 57.73 % and 325.15%, while the R_{ct} *(UR v/s UR-VD)* varies from 40.61% to 294.55%.

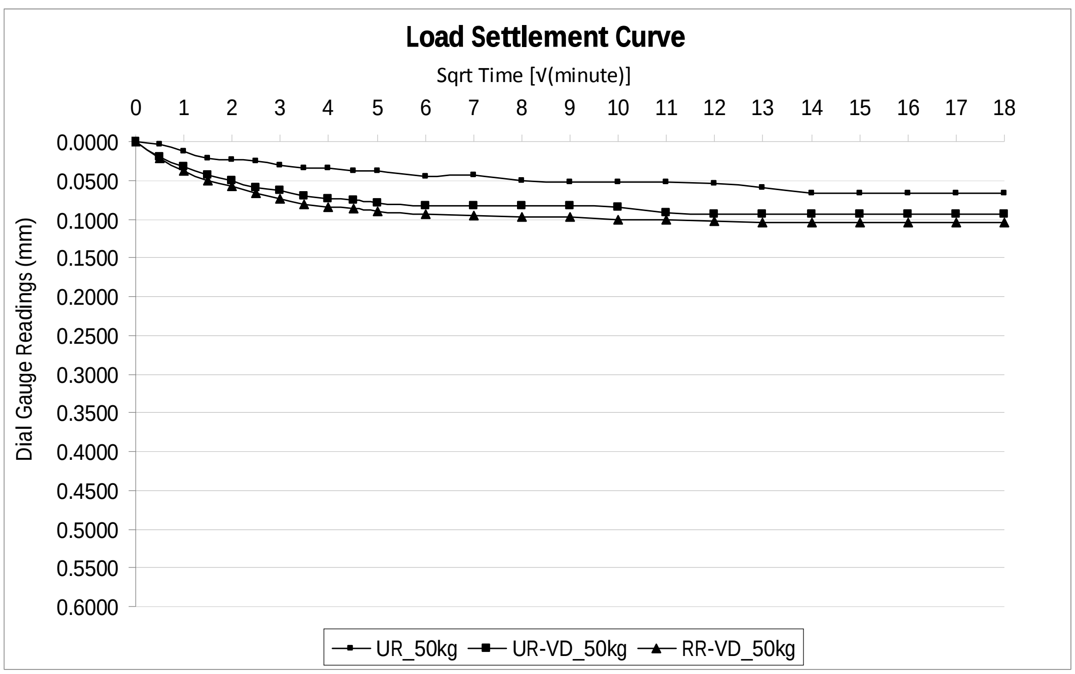

Fig. 5.20 Load-Settlement Trend for Preload of 50kg for *0%L+100%S* Soil

Thus it can be concluded that at higher preloads, the effect of reinforcing soils randomly with fibers is not effective when compared to the use of vertical drains in the case of *0%L+100%S* soils.

5.5 DISCUSSION ON COMPARISONS OF SETTLEMENT ACROSS VARIOUS BLENDS FOR *UR, UR-VD,* AND *RR-VD*

This section provides details on comparisons on the rates of settlement observed across various blends for given preload values under *UR, UR-VD,* and *RR-VD* test conditions under the following sub-sections.

5.5.1 Discussions on Settlements across Various Blends for *UR* Soils

Fig 5.21a provides details of load-settlement trends of *100%L+0%S, 75%L+25%S, 50%L+50%S, 25%L+75%S* and *0%L+100%S* for a load of 50 kg under *UR* conditions.

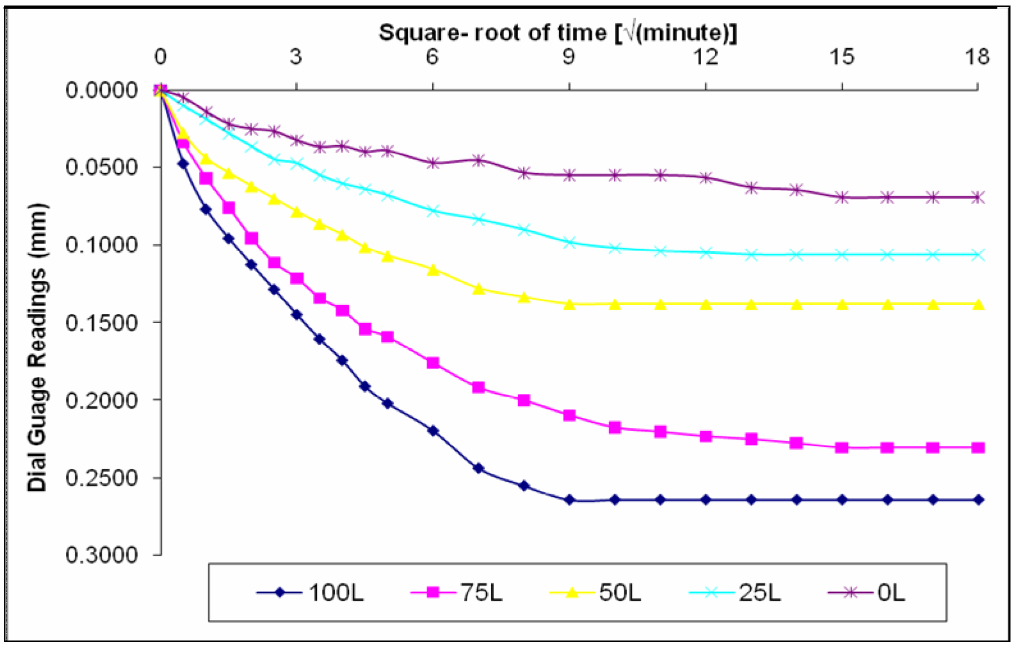

Fig. 5.21a Load-Settlement Trend for Preload of 50 kg for *UR* Soil Conditions

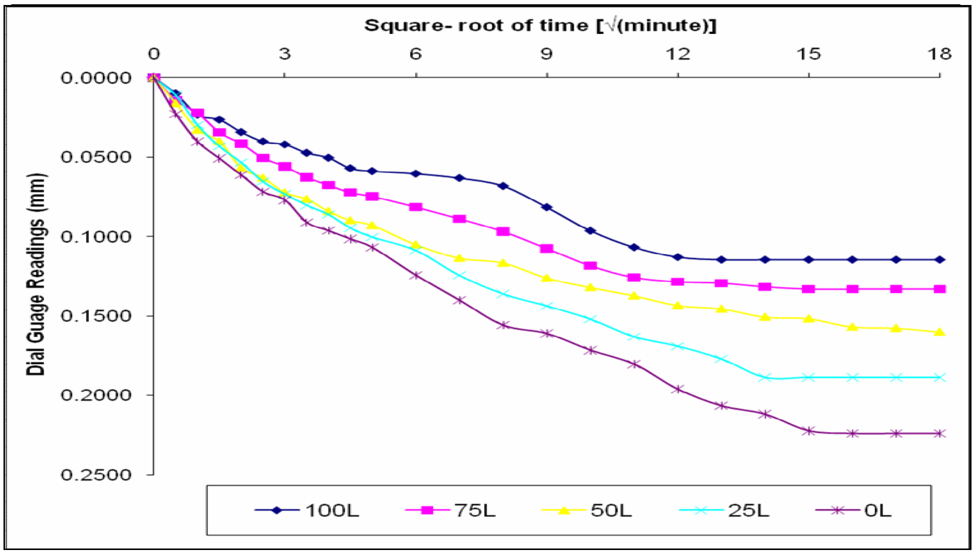

Fig. 5.21b Load-Settlement Trend for Preload of 500 kg for *UR* Soil Conditions

5.5.2 Discussions on Settlements across Various Blends for *UR-VD* Soils

In the case of *UR-VD* soils, at the initial stages of loading, the settlement is higher for the soil mixes with higher laterite content and lower for soil mixes with higher shedi content for initial preloads of 50 kg. This can be mainly attributed to the high permeability and hence the faster drainage of water from lateritic soils compared to shedi soil which takes longer duration for dissipation of pore water pressure due to its low permeability.

Fig. 5.22a provides details of load-settlement trends of *100%L+0%S*, *75%L+25%S* and *50%L+50%S* for a load of 50 kg under *UR-VD* conditions.

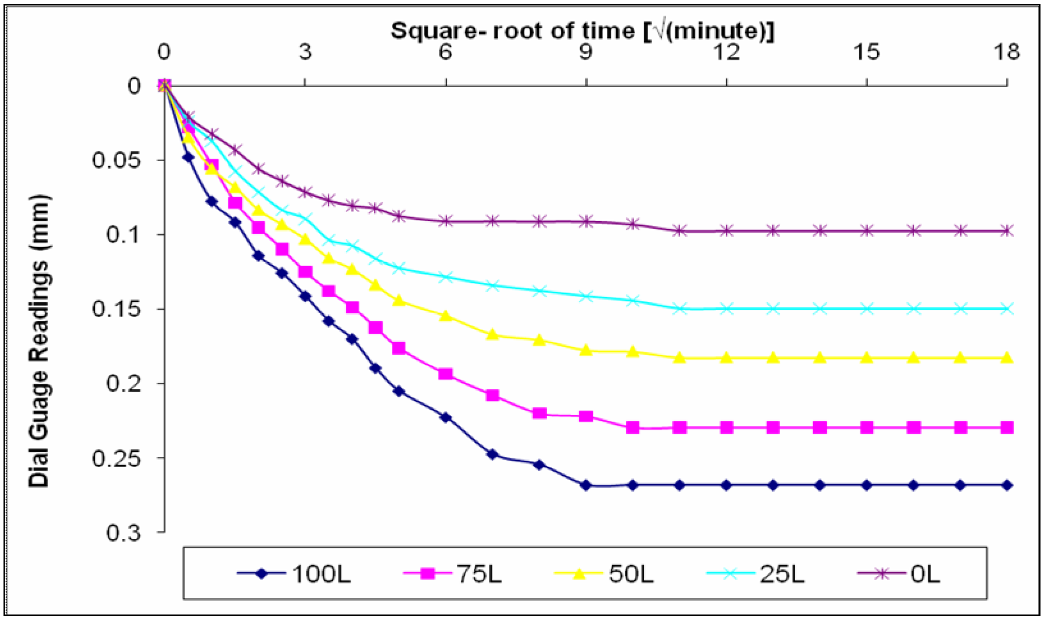

Fig. 5.22 a Load-Settlement Trend for Preload of 50 kg for *UR –VD* Soil Conditions

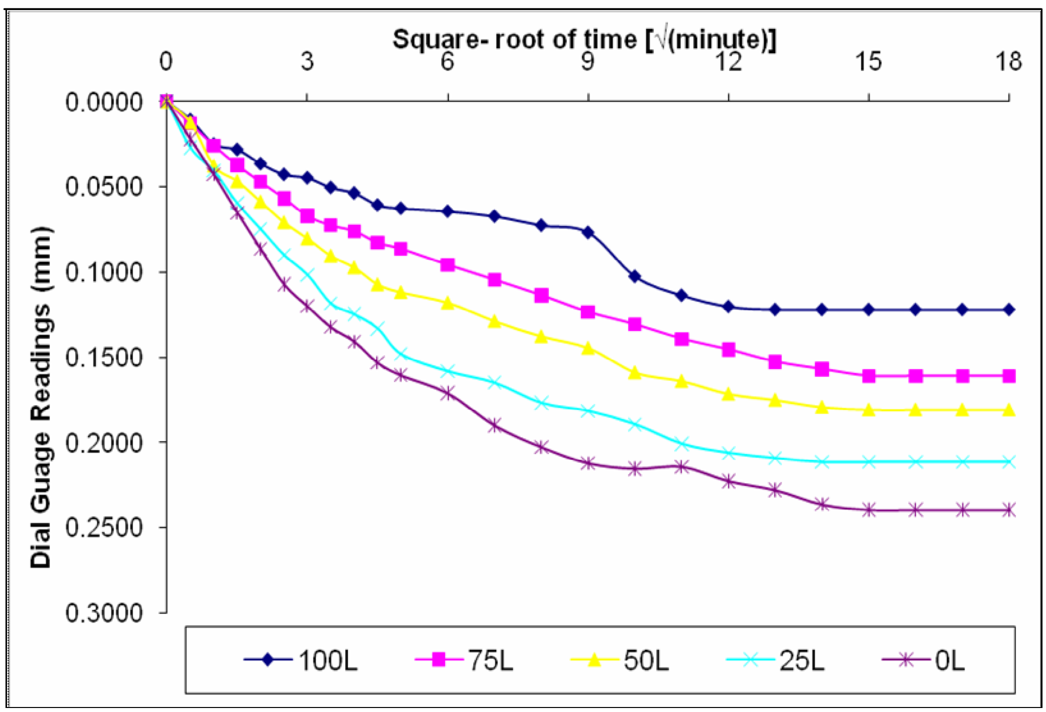

Fig. 5.22b Load-Settlement Trend for Preload of 500 kg for *UR –VD* Soil Conditions

5.5.3 Discussions on Settlements across Various Blends for *RR-VD* Soils

In the case of *RR-VD* soils, at the initial stages of loading, the settlement was found to be higher for the soil mixes with higher laterite content and lower for soil mixes with higher shedi content. A similar trend was observed in the case of *UR-VD* soils also. This can be mainly attributed to the high permeability and hence the faster drainage of water from lateritic soils compared to shedi soil which takes longer duration for dissipation of pore water pressure due to its low permeability. The soils reinforcements also enhance the settlement rates due to improved soil drainage. **Fig. 5.23a** provides details of load-settlement trends of *100%L+0%S*, *75%L+25%S* and *50%L+50%S* for a load of 50 kg under *RR-VD* conditions.

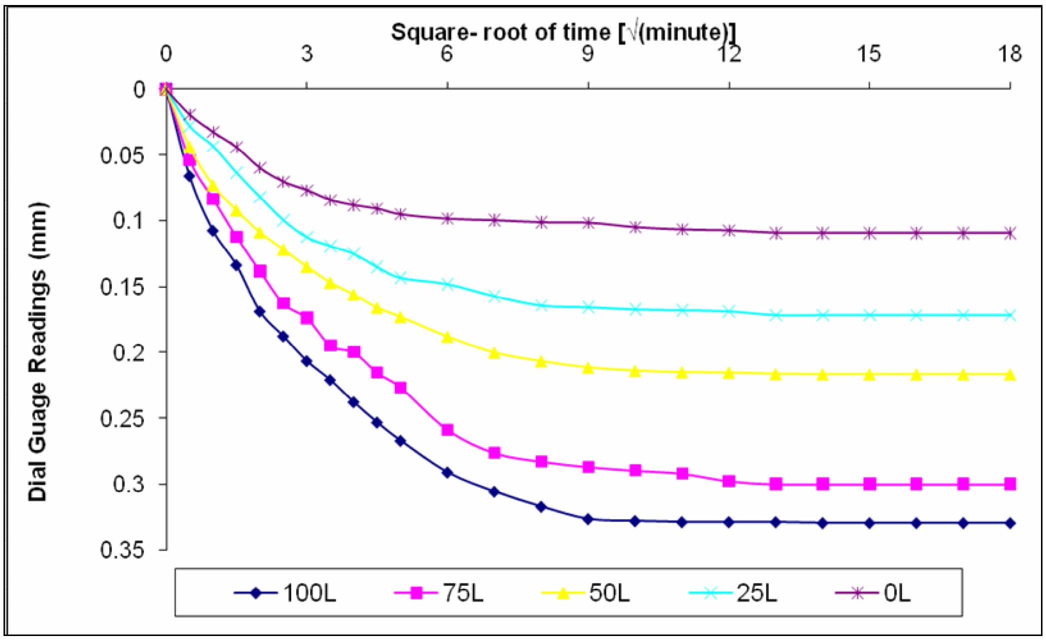

Fig. 5.23a Load-Settlement Trend for Preload of 50 kg for *RR –VD* Soil Conditions

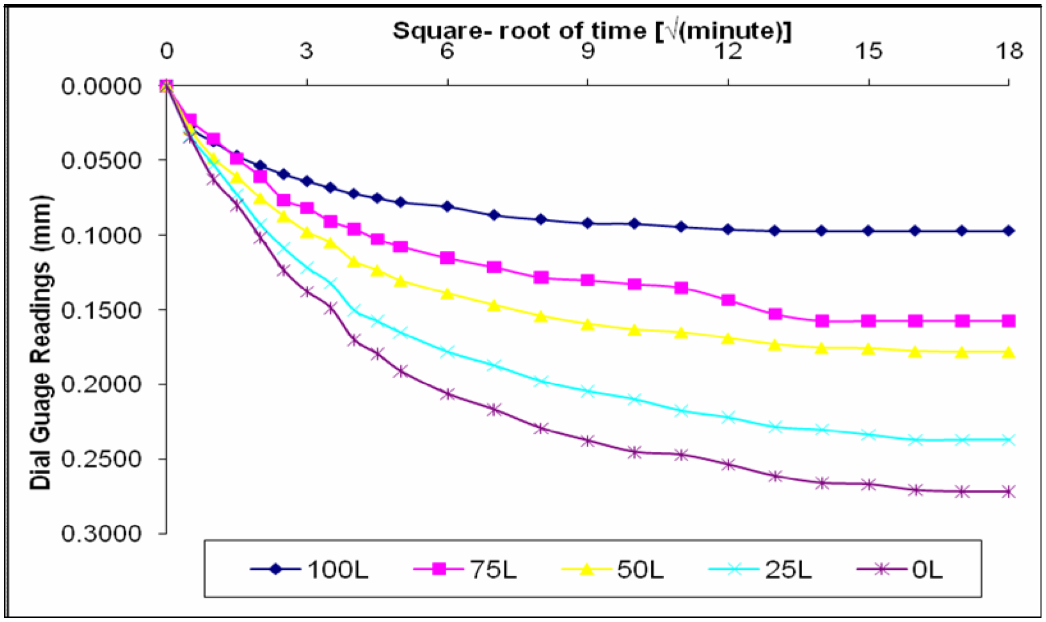

Fig. 5.23b Load-Settlement Trend for Preload of 500kg for *RR –VD* Soil Conditions

CHAPTER 6

CONCLUSIONS

6.1 INTRODUCTION

It has been observed that, in a large number of road-sections, particularly those situated close to areas with weak sub-grades, like in Dakshina Kannada District, have failed due to settlement of the base courses. Sub-grades of this region comprise laterite and shedi soil mixes, that possess higher strengths in dry conditions, and lower strengths with CBR values ranging between 1-2%, when exposed to wet conditions.

The possible reason for the settlement of roads after construction could be the lack of proper consolidation of the sub-grade soils. In the case of sub-grades with clayey and silty soils, the permeability is much lower, and so, the consolidation of soil layers, occur over longer durations. The time taken for consolidation is thus a crucial factor that needs to be considered before undertaking the construction of roads.

A soil blend with 0% laterite and 100% shedi soil (or lithomargic soil) is denoted as, *0%L + 100%S.* In this study, laboratory investigations were performed for various soil blends ranging from *100%L + 0%S* to *0%L + 100%S*, compacted in cylindrical moulds of 70cm diameter and 85cm internal height. Tests on drained soil samples were performed by providing 3 vertical sand drains reinforced with 1% coir fiber. Further tests on randomly reinforced soils were also performed using 0.75 to 1% coir fiber by weight of the soil mass. The load-settlement observations were performed for the following soil conditions:

- Un-reinforced soil (*UR*)
- Un-reinforced soil with vertical drains (*UR-VD*)
- Randomly reinforced soil with vertical drains (*RR-VD*)

Initially, studies on 100% un-reinforced laterite soil samples were performed with and without reinforced vertical drains. Tests were also conducted on soil samples randomly reinforced with coir fiber. Similar tests were performed for various soil blends.

6.2 CONCLUSIONS

In the conventional installation of PVDs the use of mandrels and other equipment reduce the rate of settlement due to the smear effect (**Barron 1948; Rowe, 1968; Hansbo 1979, 1981**) This study on accelerated settlement in lateritic and lithomargic soils using randomly reinforced built-up vertical sand drains provides a sound basis for analyzing the settlement rates in the absence of smear effects. This study on the use of coir reinforced sand-drains, is of special importance to the region of Dakshina Kannada in Southern Peninsular India, where the soil is predominantly of lateritic and lithomargic origin, and where coir-is available in plenty. The use of coir fiber for the soil reinforcement can offer the possible advantage of cheaper construction and lesser damage to the environment as coir is a bio-degradable material.

6.2.1 Settlement Characteristics for *100%L+0%S*

From **Sections 5.2.1** and **5.3.1**, and **Table 5.16**, it is seen that the *100%L+0%S* soil attains stability at around the 121st minute after application of the pre-loads for *UR*, *UR-VD,* and *RR-VD* soils.

However, using the 121st minute as the datum, it can be observed that for pre-loads ranging from 50 kg (1.2×10^{-3} N/mm^2) to 150 kg, (3.5×10^{-3}N/mm^2) for *RR-VD* soil conditions, the effect of reinforcing soils randomly with fibers was found to be significant when compared to the rate of settlement of *UR* soils. The relative increase in settlement of *RR-VD* soils when compared to that of *UR* soils, denoted as R_{ct} *(UR v/s RR-VD)),* was found to be around 29%. But, for pre-loads greater than 150kg, (3.5×10^{-3} N/mm^2), the effect of random reinforcement in the soil sample was found to be moderate with the R_{ct} *(UR v/s RR-VD)* values ranging between 16.80% and 28.47%, with an average increase of 22.63%.

6.2.2 Settlement Characteristics for *75%L+25%S*

From **Sections 5.2.2** and **5.3.2**, and **Table 5.17**, it is seen that the *75%L+25%S* soil type attains stability at around the 169^{th} minute after application of the pre-loads. Using the 169^{th} minute as datum, it can be observed that for *75%L+25%S UR* soils, for pre-loads ranging from 50kg ($1.2x10^{-3}N/mm^2$) to 150kg, ($3.5\ x10^{-3}N/mm^2$) the effect of providing vertical drains was significant when compared to the rate of settlement in the case of *UR* soils.

The relative increase in settlement of *75%L+25%S UR-VD* soils when compared to that of *UR* soils (R_{ct} *(UR v/s UR-VD))* for the above range of preloads varied between 7.14% and 34.94%, with an average increase of 20.84%. But for pre-loads higher than 150kg, ($3.5\ x10^{-3}N/mm^2$) the effect of providing vertical sand drains was not found to be significant as the R_{ct} value was found to vary between 4.58% and 18.18% only, with an average increase of 10.83%.

6.2.3 Settlement Characteristics for *50%L+50%S*

From **Sections 5.2.3** and **5.3.3**, and **Table 5.18**, it is seen that the *50%L+50%S* soil type attains stability at around the 169^{th} minute after application of the pre-loads. Using the 169^{th} minute as datum, it can be observed that for pre-loads ranging from 50kg ($1.2x10^{-3}\ N/mm^2$) to 200kg ($4.6\ x10^{-3}\ N/mm^2$), the effect of providing vertical drains (as in *UR-VD* soils) was significant when compared to the rate of settlement in the case of *UR* soils. The relative increase in settlement of *UR-VD* soils when compared to that of *UR* soils (R_{ct} *(UR v/s UR-VD))* within the above range of pre-loads varied from 37.54% to 65.19% with an average increase of 47.9%. But for pre-loads higher than 200kg, the effect of providing vertical sand drains alone (as in *UR-VD* soils) was not significant as it was found to vary between 2.11% to 14.35%, with an average increase of only about 9.8%.

Therefore, it can be seen that the combined effect of vertical drains and random reinforcement with coir fibers resulted in considerable increase in the rate of consolidation at lower pre-loads. Considering the average values, for pre-loads ranging from 50kg ($1.2x10^{-3}N/mm^2$) to 200kg ($4.6x10^{-3}N/mm^2$), there is an additional increase of 24.8% in the settlement of randomly reinforced soil blends (or *RR-VD* soils) when compared to that of soil blends with vertical drains (or *UR-VD* soils) alone. For higher pre-loads, an increase of 10.63% was observed. Thus, it can be concluded that for *50%L+50%S RR-VD* soils, there is a significant increase in settlement due to random reinforcement with coir fibres in addition to the vertical drains.

6.2.4 Settlement Characteristics for *25%L+75%S*

From **Sections 5.2.4** and **5.3.4**, and **Table 5.19**, it is seen that the *25%L+75%S* soil type attains stability at around the 196th minute after application of pre-loads. Using the 196th minute as datum, it can be observed that for pre-loads ranging from 50kg ($1.2x10^{-3}$ N/mm^2) to 250kg (5.8 $x10^{-3}$ N/mm^2), the effect of providing vertical drains (as in *UR-VD* soils) was significant when compared to the rate of settlement in the case of *UR* soils. The relative increase in settlement of *UR-VD* soils when compared to that of *UR* soils (R_{ct} *(UR v/s UR-VD))* for the above range of preloads varied between 40.91% and 120.85%, with an average increase of 76.6%. But for pre-loads higher than 250kg (5.8 $x10^{-3}$N/mm^2), the effect of providing vertical sand drains was found to be moderate as the R_{ct} value was found to vary between 5.2% and 18.29% only, with an average increase of 11.73%.

6.2.5 Settlement Characteristics for *0%L+100%S*

From **Sections 5.2.5** and **5.3.5**, and **Table 5.20**, it is seen that the *0%L+100%S* soil type attains stability at around the 225th minute after application of the pre-loads. Using the 225th minute as datum, it can be observed that for pre-loads ranging from 50kg to 250kg, the effect of providing vertical drains (as in *UR-VD* soils) was significant when compared to the rate of settlement in the case of *UR* soils. The relative increase in settlement of *UR-VD* soils when compared to that of *UR* soils (R_{ct} *(UR v/s UR-VD))* for the above range of preloads varied between 40.61% and 294.55%, with an average increase of 176.6%. But for pre-loads higher than 250kg, the effect of providing vertical sand drains (as in *UR-VD* soils) was not found to be significant as the R_{ct} value was found to vary between 4.01% and 13.06% only, with an average increase of 7.67%.

6.2.6 Comparison of *Cv* Values for *UR, UR-VD,* and *RR-VD* for Various Blends

From the values for *Cv* for *UR*, *UR-VD*, and *RR-VD* cases for *100%L + 0%S* soils, it can be observed that in the case of *UR* soil conditions, the maximum value of C_v of 2.0825 cm^2/s was found to occur at a higher pressure range of 3.5 to 4.6 $x10^{-3}$ N/mm^2. In the case of *UR-VD* soil conditions, the maximum value of C_v of 2.2683 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. Also, in the case of *RR-VD* soil conditions, the maximum value of C_v of 2.2882 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2.

Based on the values for *UR*, *UR-VD*, and *RR-VD* cases for *75%L + 25%S* soils, it can be observed that in the case of *UR* soil conditions, the maximum value of C_v of 1.9546 cm^2/s was found to occur at a higher pressure range of 3.5 to 4.6 $x10^{-3}$ N/mm^2. In the case of *UR-VD* soil conditions, the maximum value of C_v of 2.1172 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. Also, in the case of *RR-VD* soil conditions, the maximum value of C_v of 2.204 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2.

Considering the values for *Cv* for *UR*, *UR-VD*, and *RR-VD* cases for *50%L + 50%S* soils, it can be observed that in the case of *UR* soil conditions , the maximum value of C_v of 1.4280 cm^2/s was found to occur at a higher pressure range of 3.5 to 4.6 $x10^{-3}$ N/mm^2. In the case of *UR-VD* soil conditions, the maximum value of C_v of 2.0164 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2. Also, in the case of *RR-VD* soil conditions, the maximum value of C_v of 2.0322 cm^2/s was found to occur at a lower pressure range of 2.3 to 3.5 $x10^{-3}$ N/mm^2.

6.2.7 Settlement Characteristics across Five Different Blends (*100%L to 100%S)*

It was observed that soil blends with higher shedi contents exhibit lower settlement rates during the initial phase of consolidation since these soils possess lower permeability when compared to soil blends with higher laterite content.

Comparing the settlements observed during the preload using 50kg ($1.2x10^{-3}$ N/mm^2) to that using 500kg (11.6 $x10^{-3}$ N/mm^2) it is observed that there is a total reversal in the rate of settlement for various soil blends. That is, the soil blend with higher shedi content, which displayed lower settlement rates for initial preloads of 50 kg (1.2×10^{-3} N/mm^2), showed higher settlement rates for higher preloads of 500kg (11.6×10^{-3} N/mm^2). This is due to the reason that soil samples with a predominant mix of shedi soil offer lesser stability at later stages of preloading, where the soil moisture is more effectively drained out at higher preloads, and also due to accelerated drainage. In comparison, the soil samples with a predominant mix of laterite soil undergo consolidation at a higher rate initially due to higher permeability, and exhibit continued consolidation at higher preloads. The variations in the settlement rates for various blends of soil can be visualized through the load-settlement graphs for various blends.

6.3 MAJOR CONTRIBUTIONS OF THIS WORK

- The above investigations provide an insight into the load-settlement characteristics of embankments provided with built-up reinforced vertical sand drains in lateritic-lithomarge and lithomargic-laterites soils. The use of built-up vertical drains enabled the study of settlement and consolidation in the absence of the smear zones normally associated with vertical drains driven into the soil.
- From the behavior of the soil mentioned above, it is observed that the presence of the vertical drains has increased the rate of settlement and amount of settlement, mainly at lower pre-loads.
- The random reinforcement of soil blends in addition to the provision of vertical drains has significantly increased the rate of settlement of lateritic lithomarge soil blends.
- The use of vertical sand drains for drainage improves the drainage of pore-water, and could be a cost effective approach to achieve accelerated consolidation. Coir fibers also have the inherent capability to restrict the lateral deformation at the peripheral areas of sand drains due to their reinforcing effect.
- The use of randomly distributed coir fibers in the soil samples improves the drainage of pore-water, while simultaneously increasing the strength of the soils.

6.4 SCOPE FOR FUTURE WORK

- Studies may also be performed by altering the number of vertical drains, the spacing between them, and their diameters.
- In this study only one aspect-ratio (1:275) of coir fibers has been considered. The study can be carried out for different aspect ratios of the coir fiber.
- In this study, investigations were performed using the cylindrical test mould of 70cm internal diameter, and 80cm internal height. The scale effect in actual site conditions can be further investigated.
- Further investigations may be performed on comparing the same soil blends with the performance of pre-fabricated vertical drains where the smear effect is more pronounced.
- Similar studies may be performed for different types of soils including black-cotton soils, silty clay soils, and other blended soils.

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1. George, Varghese, Santosh G., Hegde, Ramakrishna, Prashanth, Durga, L.A., Gotamey, Dwarika, Ravishankar, A.U. (2012) A Model Study on Accelerated Consolidation of Coir Reinforced Laterite and Blended Shedi Soil with Vertical Sand Drains For Pavement Foundations, Int. Journal of Earth Sciences and Engineering, ISSN 0974-5904, Vol. 05, No. 01, pp. 141-148

2. Hegde, Ramakrishna, George, Varghese, Gotamey, Dwarika, and Ravishankar, A.U. (2011) Effect of Coir Reinforcement and Vertical Drains on the Settlement Properties of Laterite Soil, NITK Research Bulletin, Vol. 20, No. 2, pp. 30-41.

3. Hegde, Ramakrishna, George, Varghese, Gotamey, Dwarika, and Ravishankar, A.U. (2011) "Settlement Studies in Partially Compacted Lateritic Soil with and without Coir Reinforcements and Vertical Drains", Proc. National Conference on Recent Development in Civil Engineering - 2011 (RDCE-2011), 13-14 January, Manipal Institute of Technology, Manipal, Karnataka (Abstract Published) pp.13.

4. Hegde, Ramakrishna, George, Varghese, Kumar, Nirmal A.G., Prashanth, Durga, L.A., and Santosh G. (2012) Model Study on Accelerated Consolidation of Coir Reinforced Lateritic Lithomarge Soil Blends with Vertical Sand Drains for Pavement Foundations). Open Journal of Soil Science, 2012, 2, 320-332 doi:10.4236/ojss.2012.23038. Online September 2012. (http://www.SciRP.org/journal/ojss)

5. George, Varghese, Hegde, Ramakrishna, Vishnuvardhana, M., Santosh G., Gotamey, Dwarika (2012) "Accelerated Consolidation of Coir Reinforced Lithomargic Laterite Soil Blends with Vertical Sand Drains for Pavements." Electronic Journal of Geotechnical Engineering, Paper #12.177, Vol. 17, Bund. O, July, pp. 2115-2133.
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APPNDICES

APPENDIX I

LOAD-SETTLEMENT TRENDS FOR DIFFERENT PRELOADS FOR *100%L+0%S* SOIL

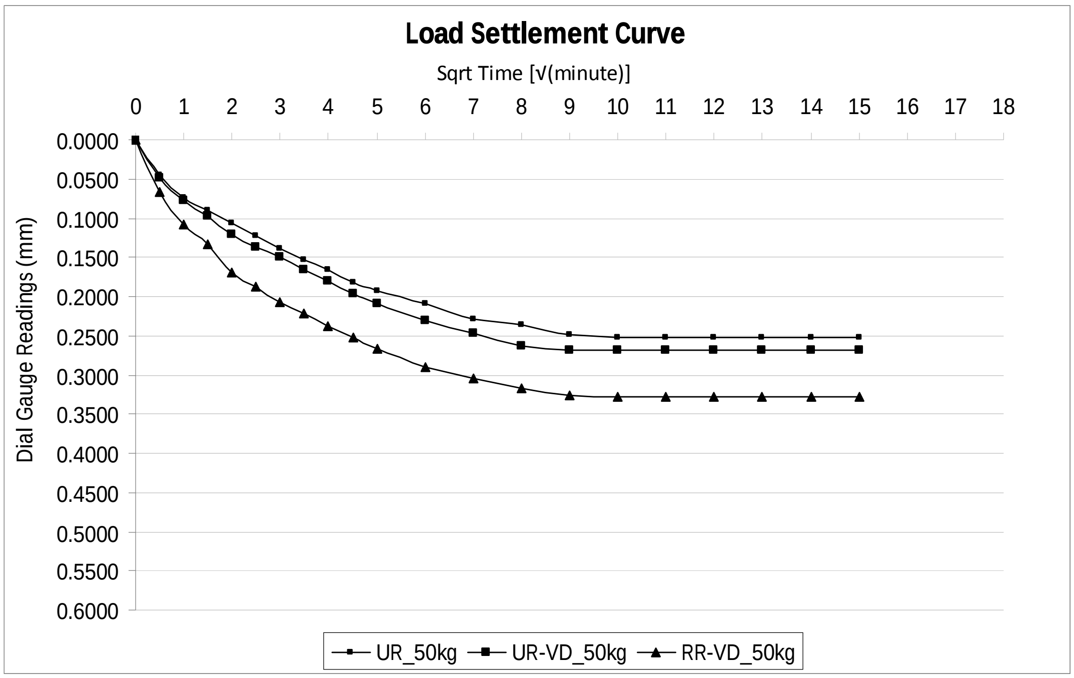

Fig.A.1 Load-Settlement Trend for Preload of 50kg for *100%L+0%S* soil

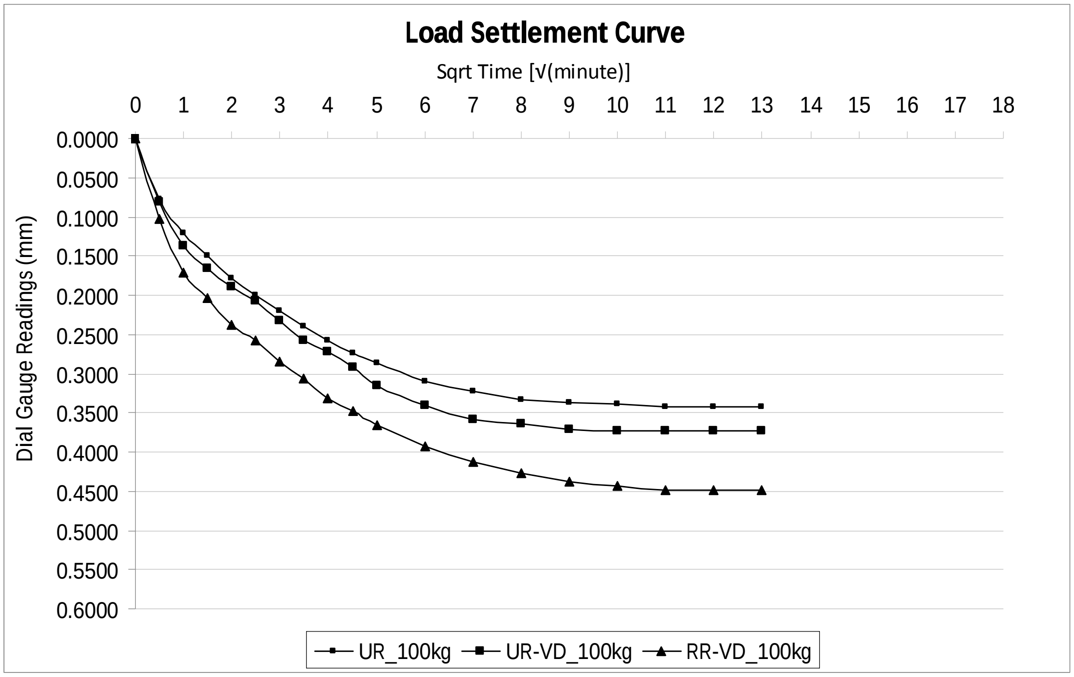

Fig.A.2 Load-Settlement Trend for Preload of 100kg for *100%L+0%S* soil

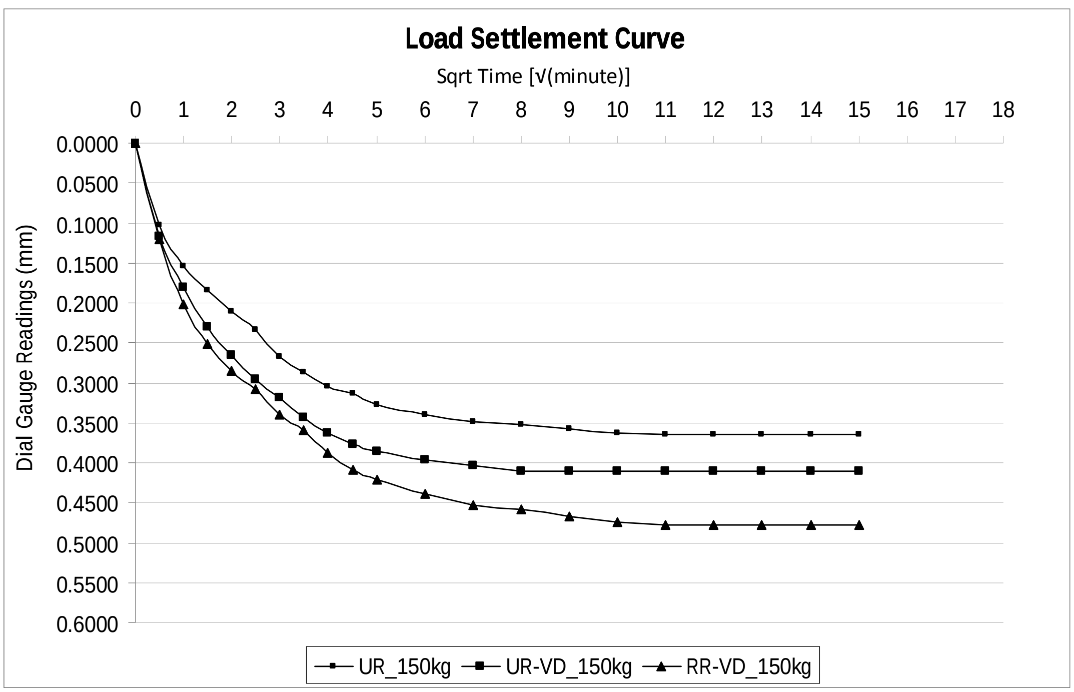

Fig.A.3 Load-Settlement trend for preload of 150kg for *100%L+0%S* soil

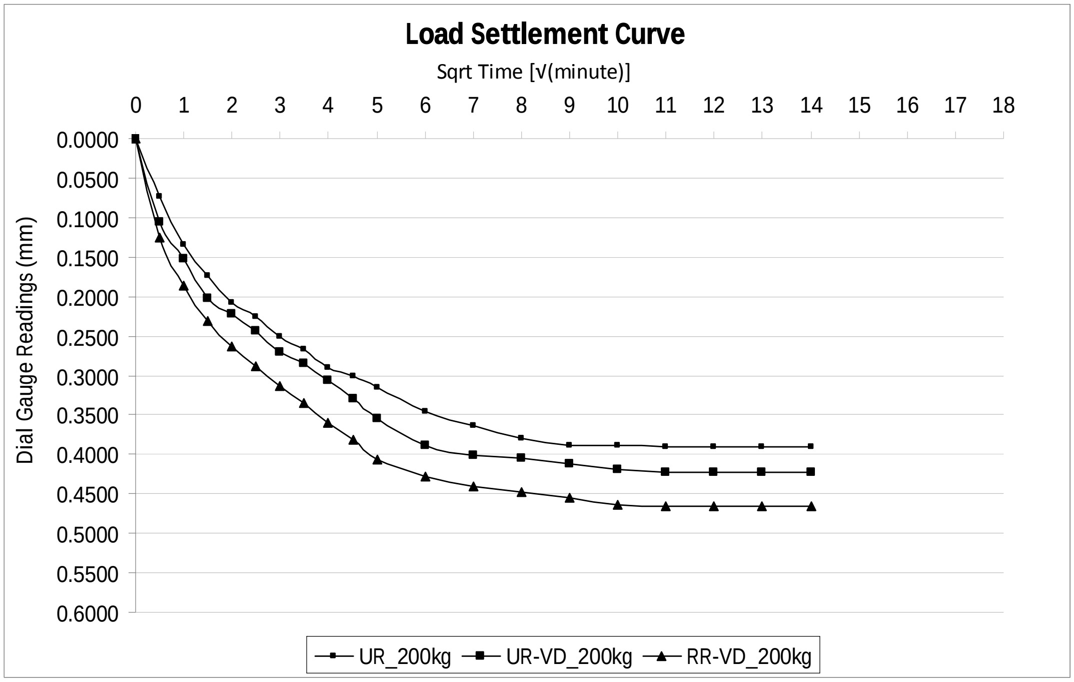

Fig.A.4 Load-Settlement trend for preload of 200kg for *100%L+0%S* soil

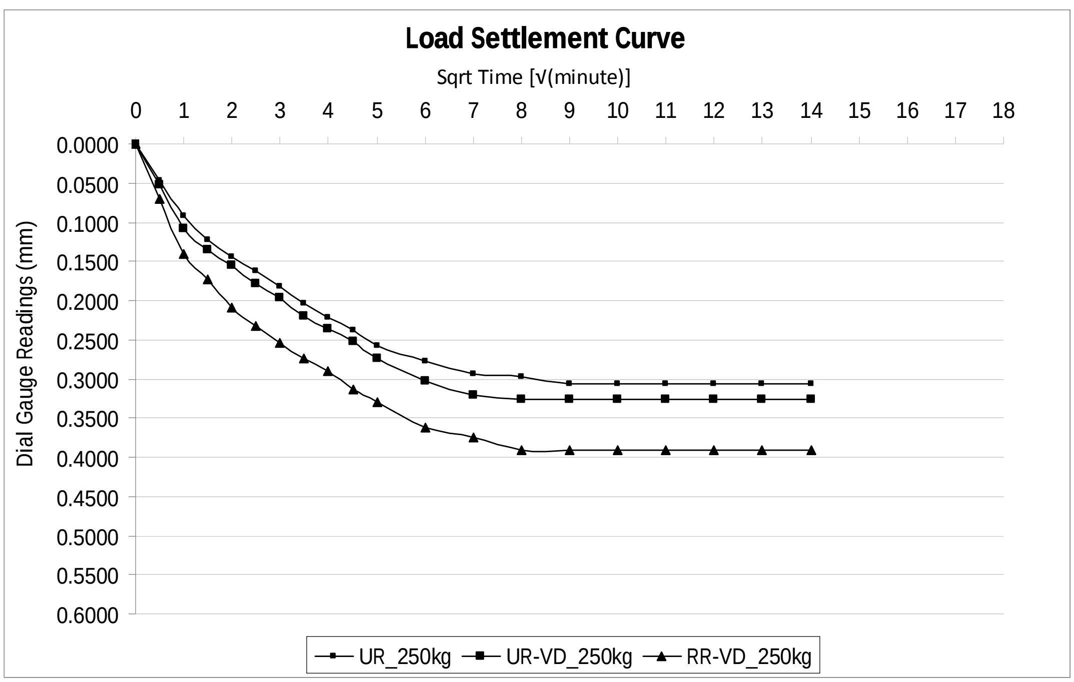

Fig.A.5 Load-Settlement trend for preload of 250kg for *100%L+0%S* soil

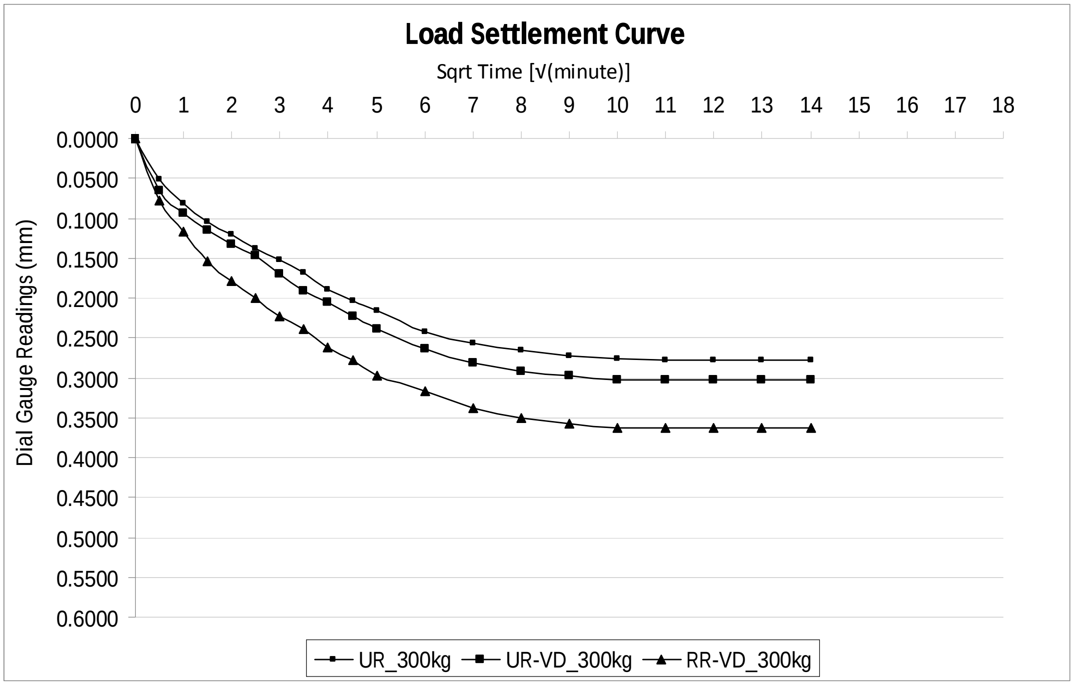

Fig.A.6 Load-Settlement trend for preload of 300kg for *100%L+0%S* soil

APPENDIX II

LOAD-SETTLEMENT TRENDS FOR DIFFERENT PRELOADS FOR *75%L+25%S* SOIL

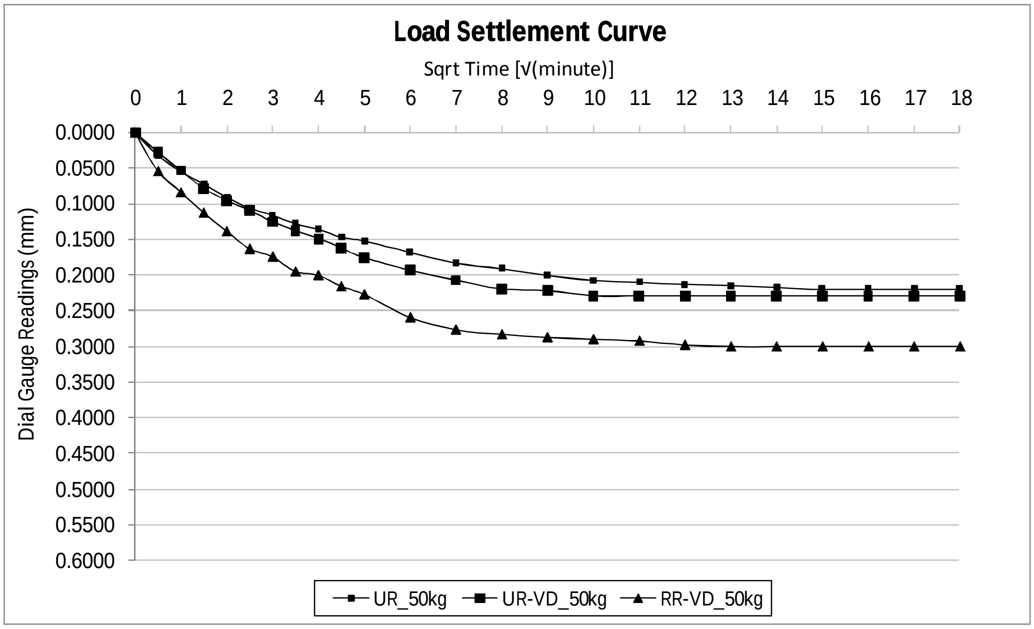

Fig.B.1 Load-Settlement trend for preload of 50kg for *75%L+25%S* soil

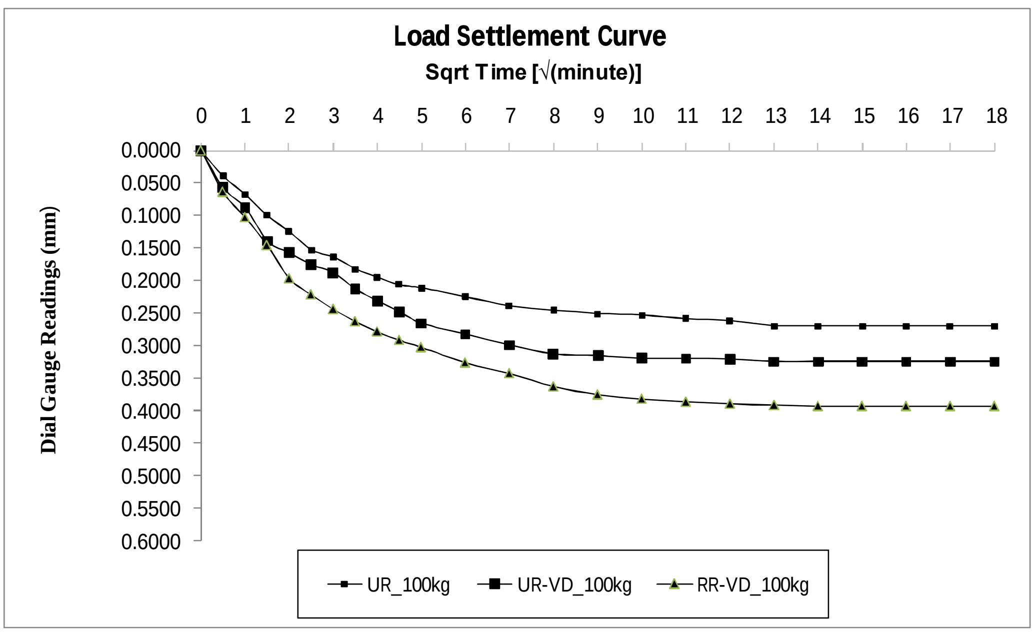

Fig.B.2 Load-Settlement trend for preload of 100kg for *75%L+25%S* soil

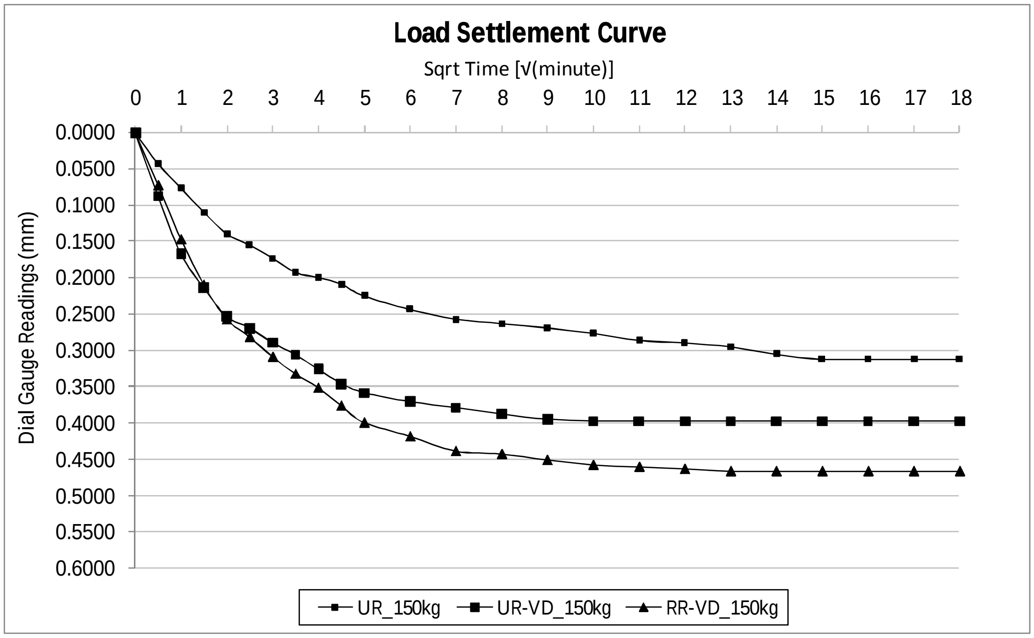

Fig.B.3 Load-Settlement trend for preload of 150kg for *75%L+25%S* soil

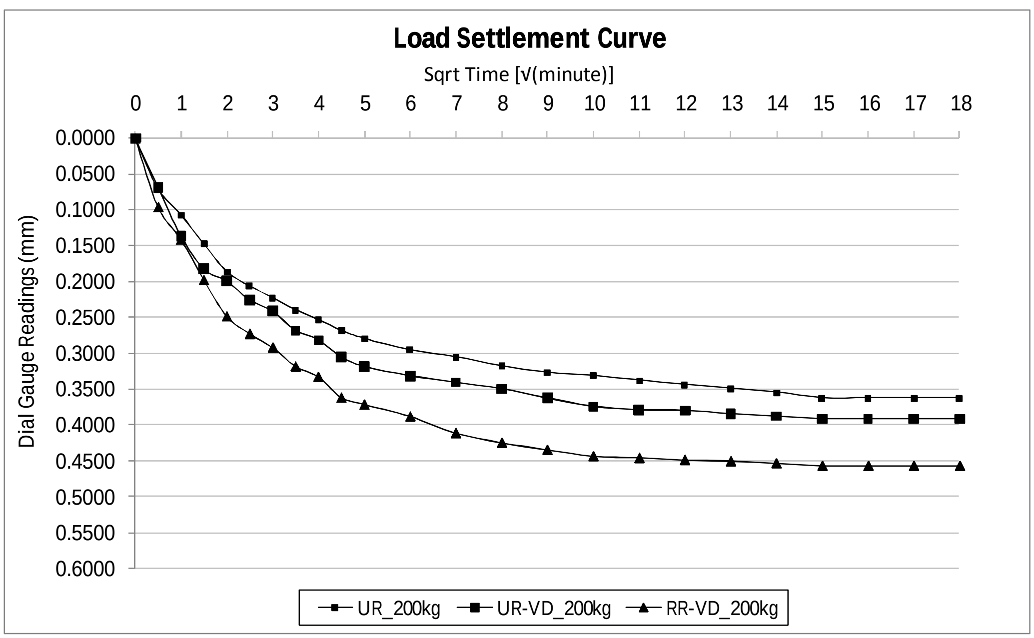

Fig.B.4 Load-Settlement trend for preload of 200kg for *75%L+25%S* soil

APPENDIX III

LOAD-SETTLEMENT TRENDS FOR DIFFERENT PRELOADS FOR *50%L+50%S* SOIL

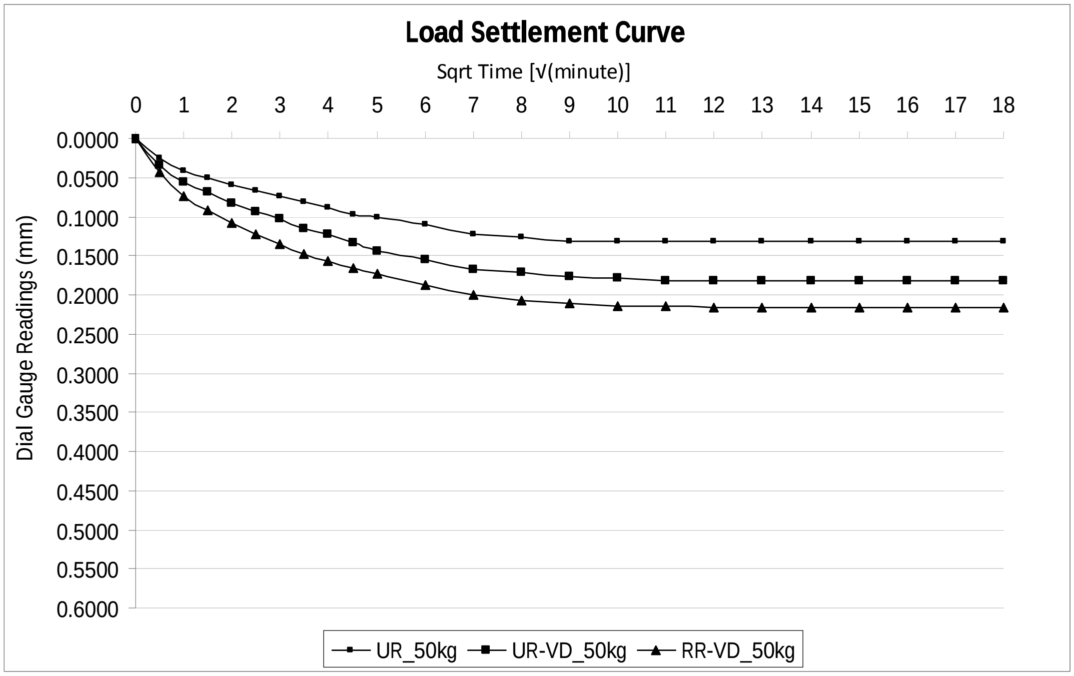

Fig.C.1 Load-Settlement trend for preload of 50kg for *50%L+50%S* soil

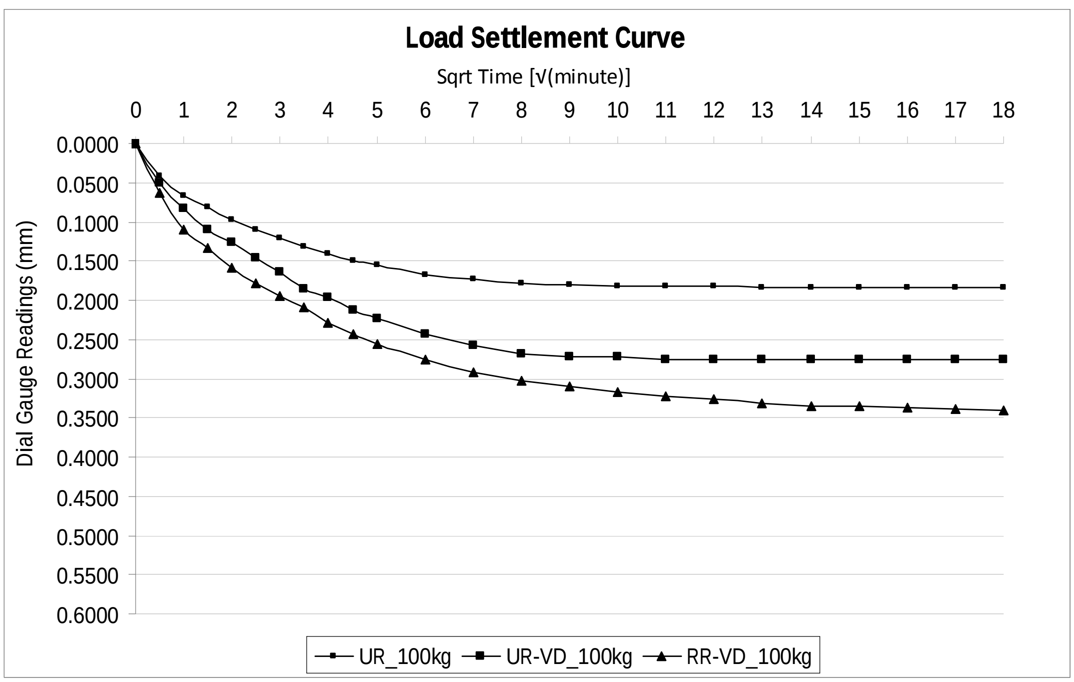

Fig.C.2 Load-Settlement trend for preload of 100kg for *50%L+50%S* soil

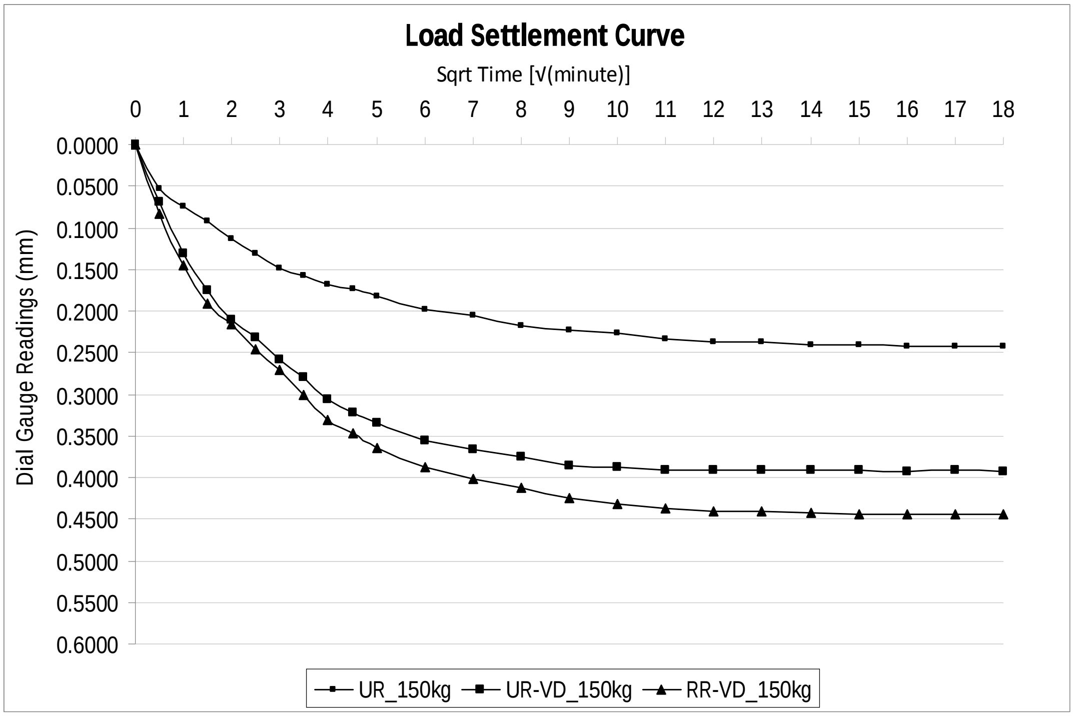

Fig.C.3 Load-Settlement trend for preload of 150kg for *50%L+50%S* soil

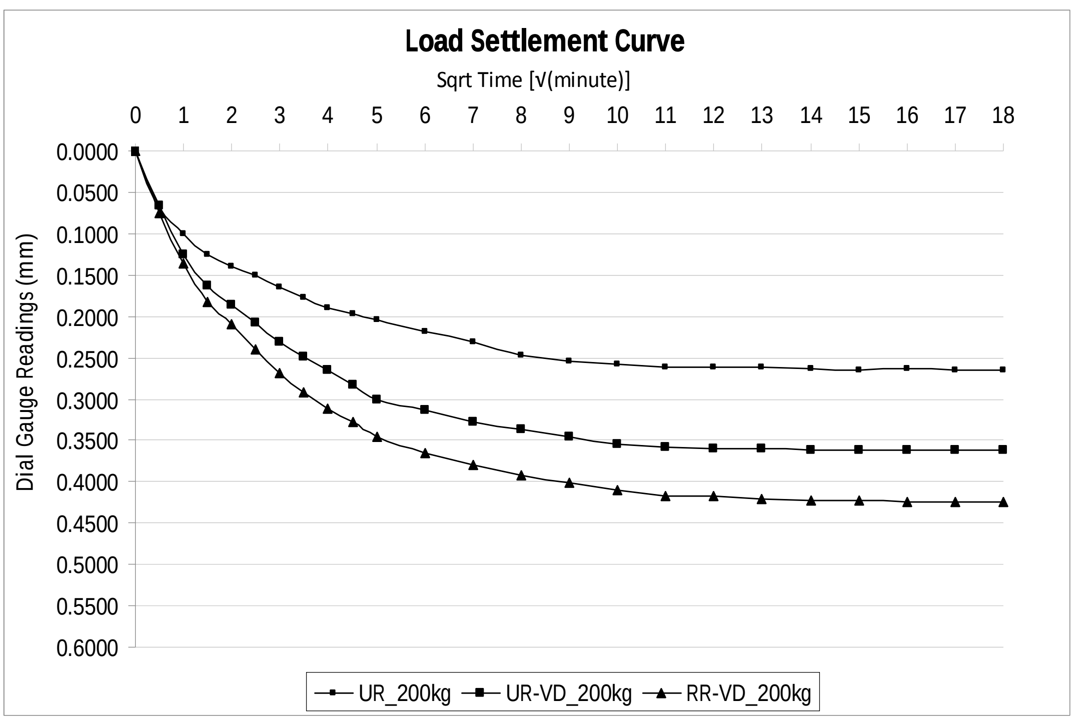

Fig.C.4 Load-Settlement trend for preload of 200kg for *50%L+50%S* soil

APPENDIX IV

LOAD-SETTLEMENT TRENDS FOR DIFFERENT PRELOADS FOR *25%L+75%S* SOIL

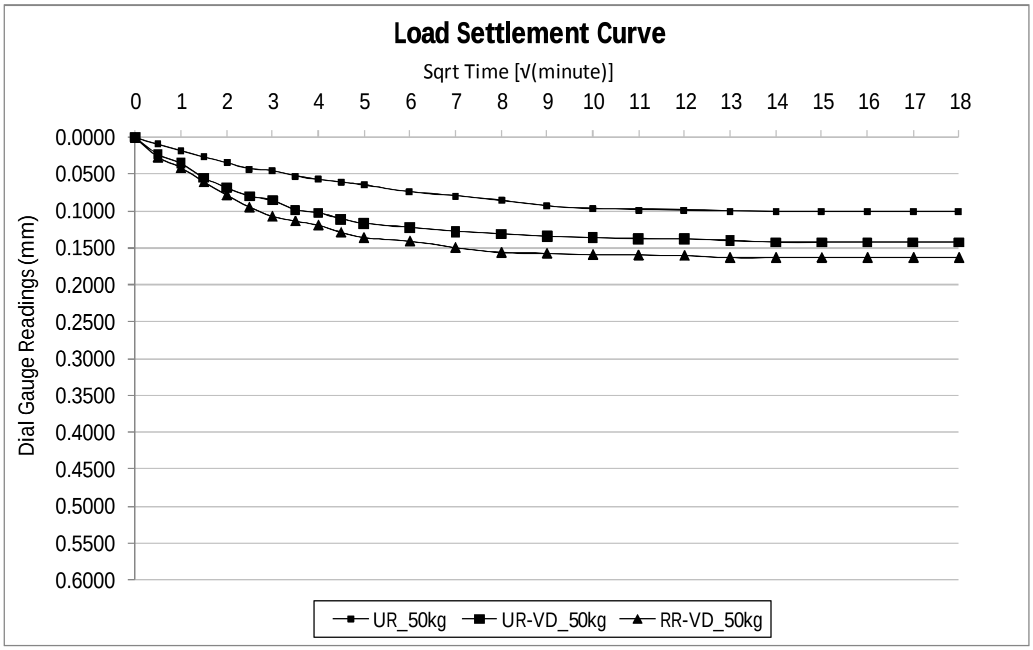

Fig.D.1 Load-Settlement trend for preload of 50kg for *25%L+75%S* soil

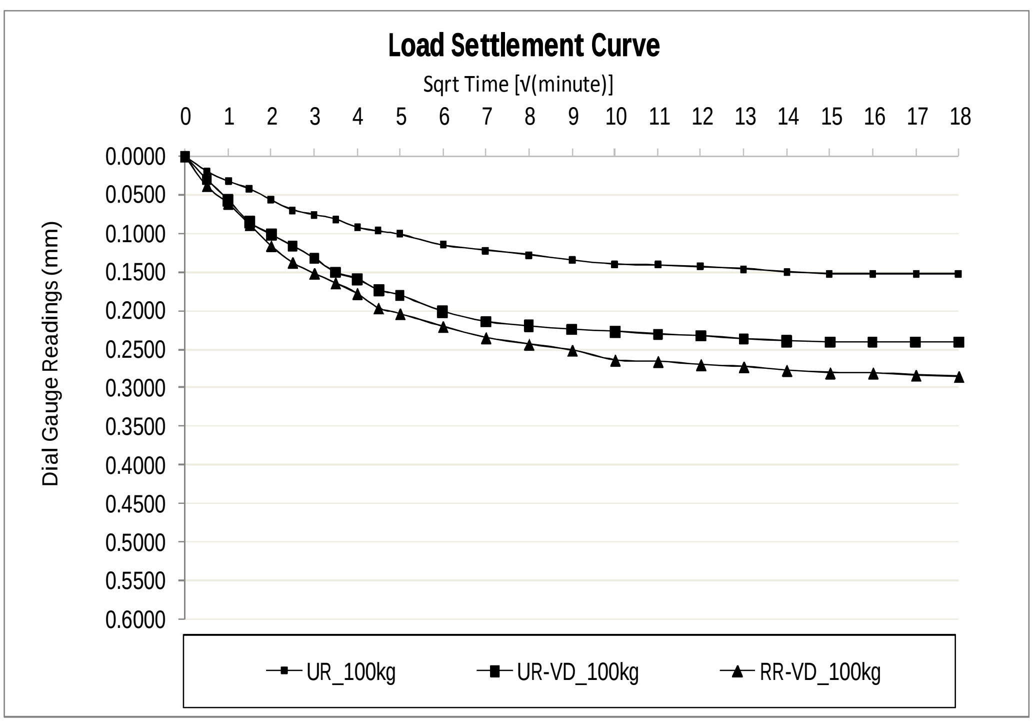

Fig.D.2 Load-Settlement trend for preload of 100kg for *25%L+75%S* soil

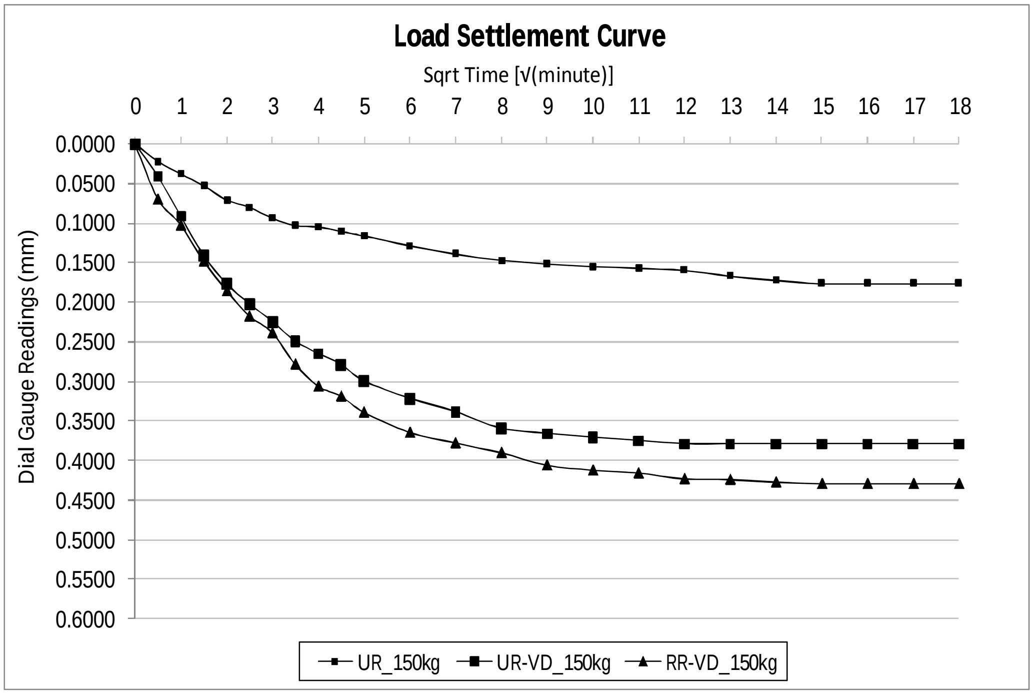

Fig.D.3 Load-Settlement trend for preload of 150kg for *25%L+75%S* soil

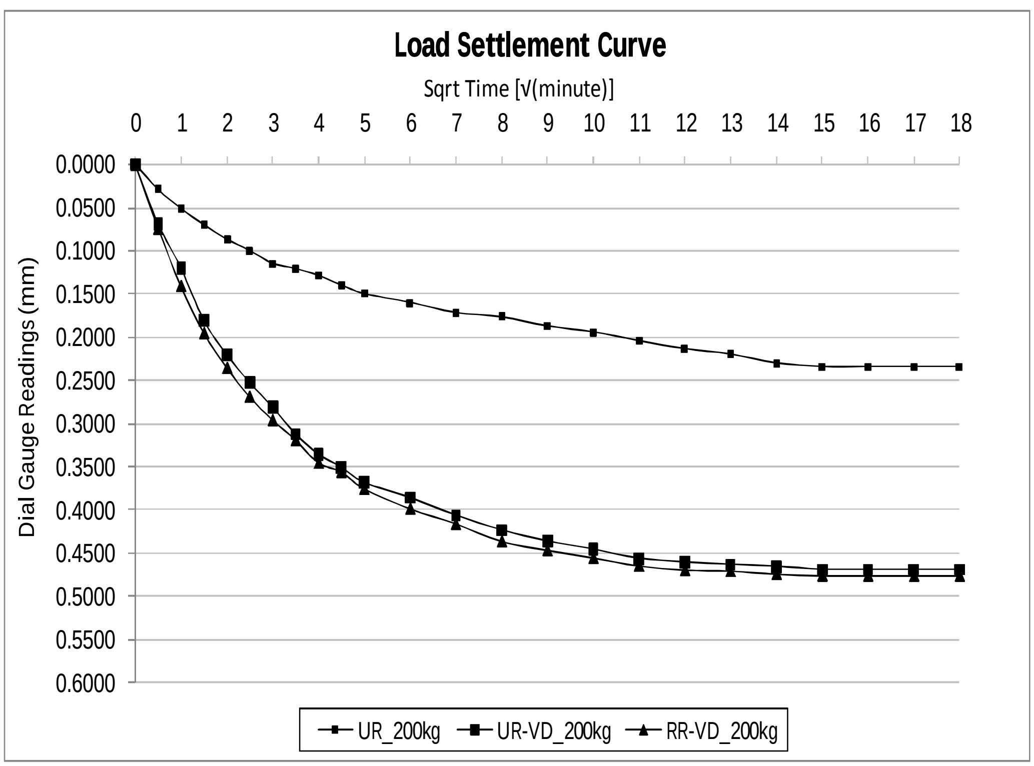

Fig.D.4 Load-Settlement trend for preload of 200kg for *25%L+75%S* soil

APPENDIX V

LOAD-SETTLEMENT TRENDS FOR DIFFERENT PRELOADS FOR *0%L+100%S* SOIL

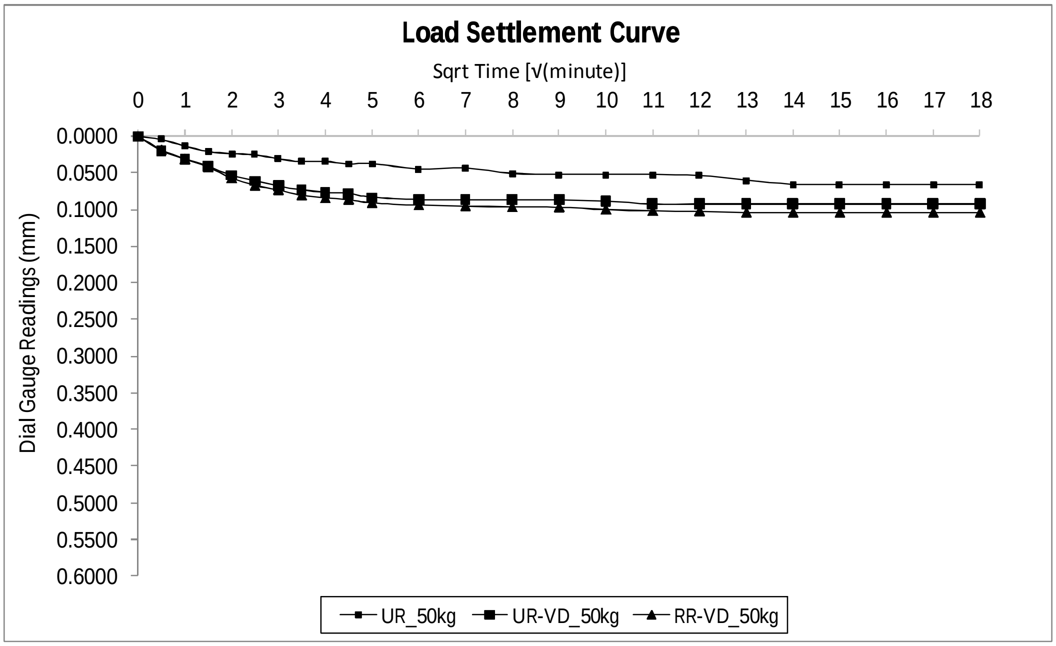

Fig.E.1 Load-Settlement trend for preload of 50kg for *0%L+100%S* soil

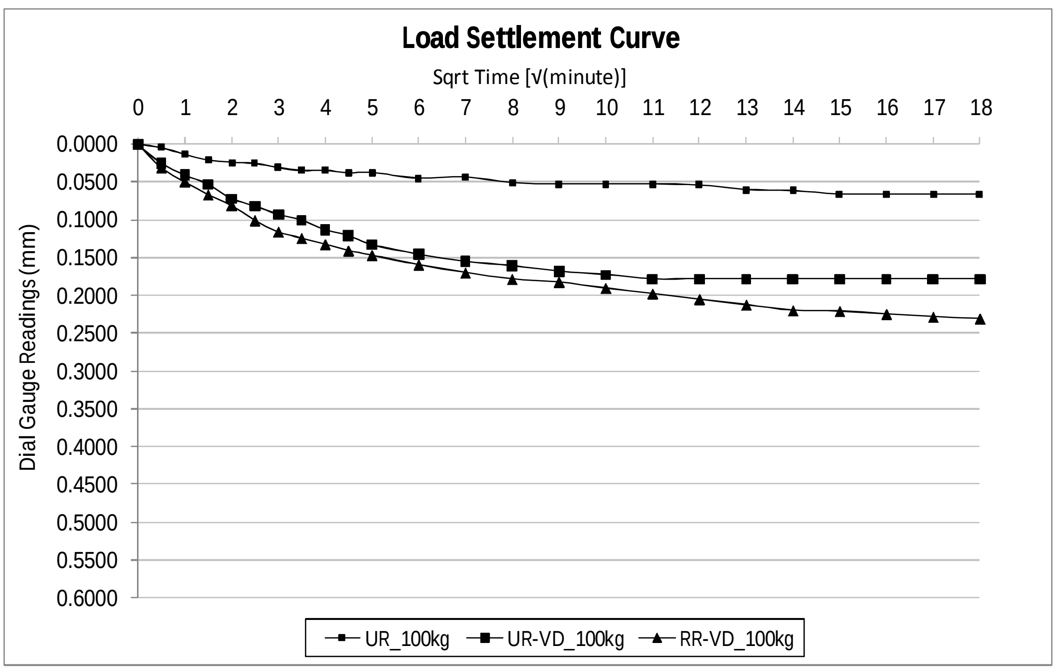

Fig.E.2 Load-Settlement trend for preload of 100kg for *0%L+100%S* soil

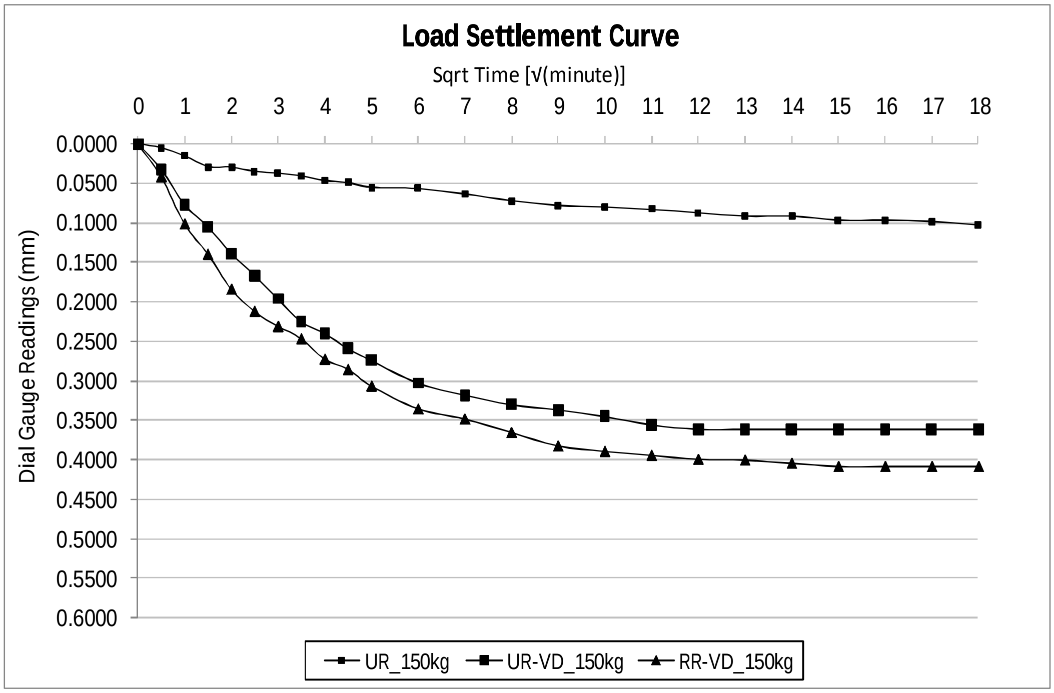

Fig.E.3 Load-Settlement trend for preload of 150kg for *0%L+100%S* soil

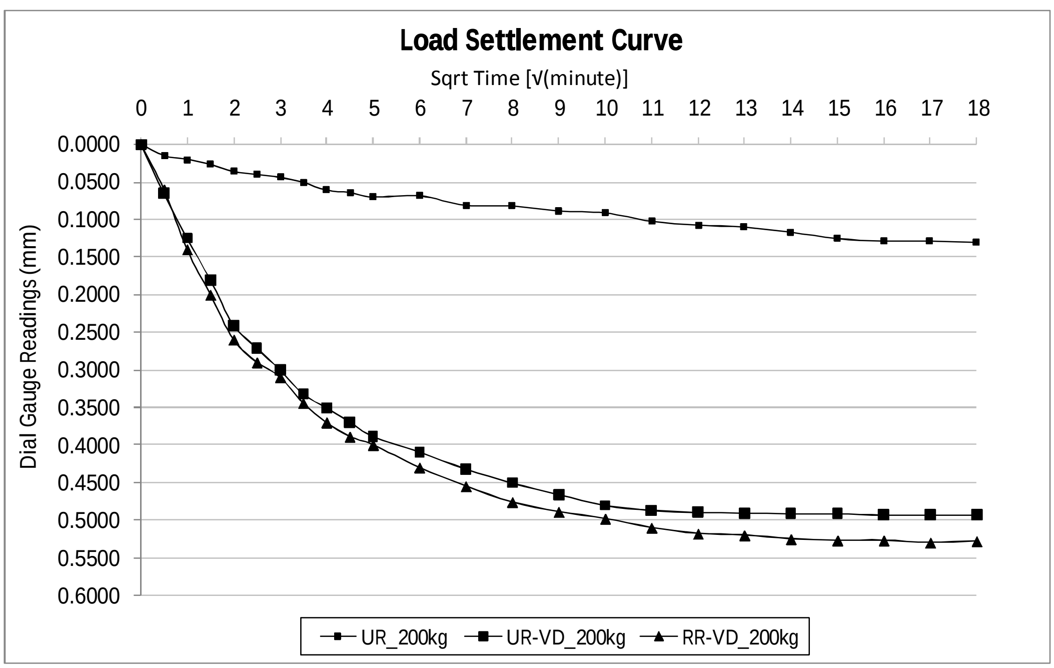

Fig.E.4 Load-Settlement trend for preload of 200kg for *0%L+100%S* soil

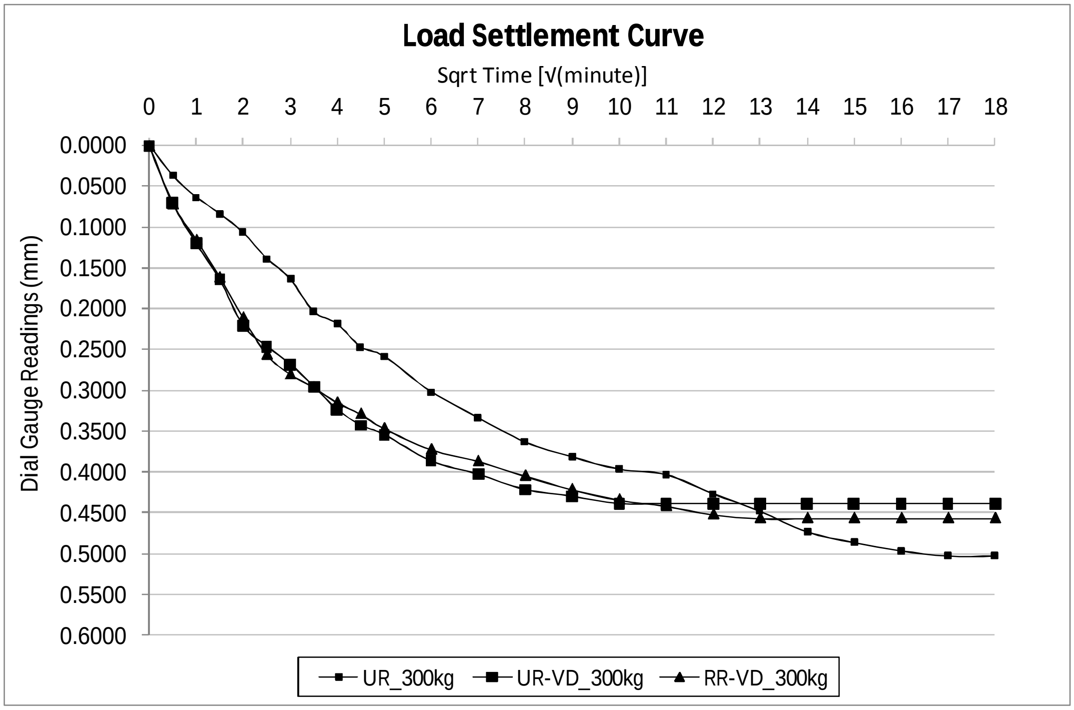

Fig.E.6 Load-Settlement trend for preload of 300kg for *0%L+100%S* soil

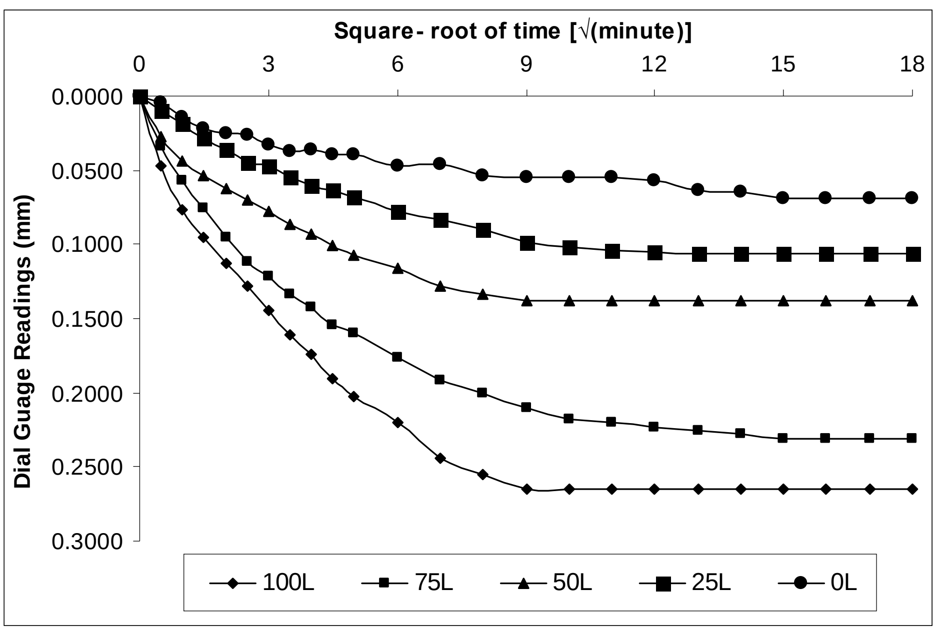

Fig.F.1a Load-Settlement Trend for Preload of 50 kg for *UR* Soil Conditions

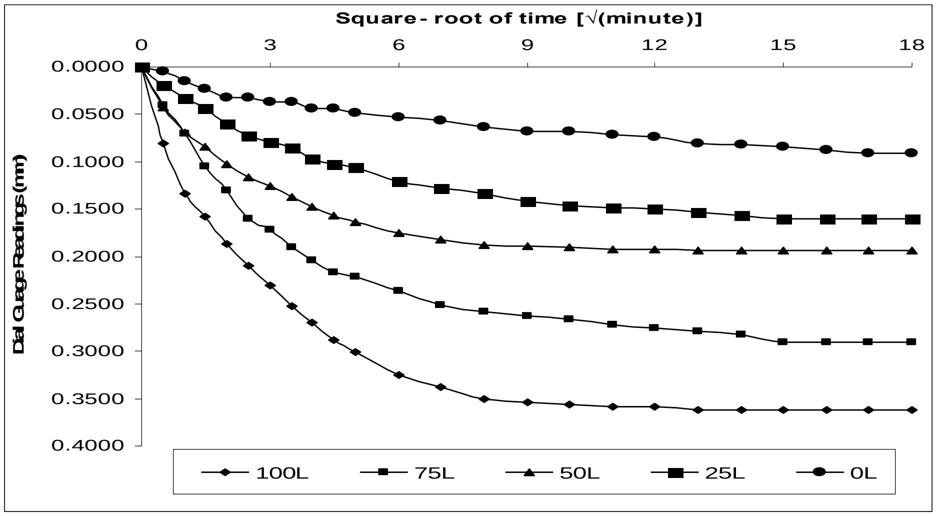

Fig.F.1b Load-Settlement Trend for Preload of 100 kg for *UR* Soil Conditions

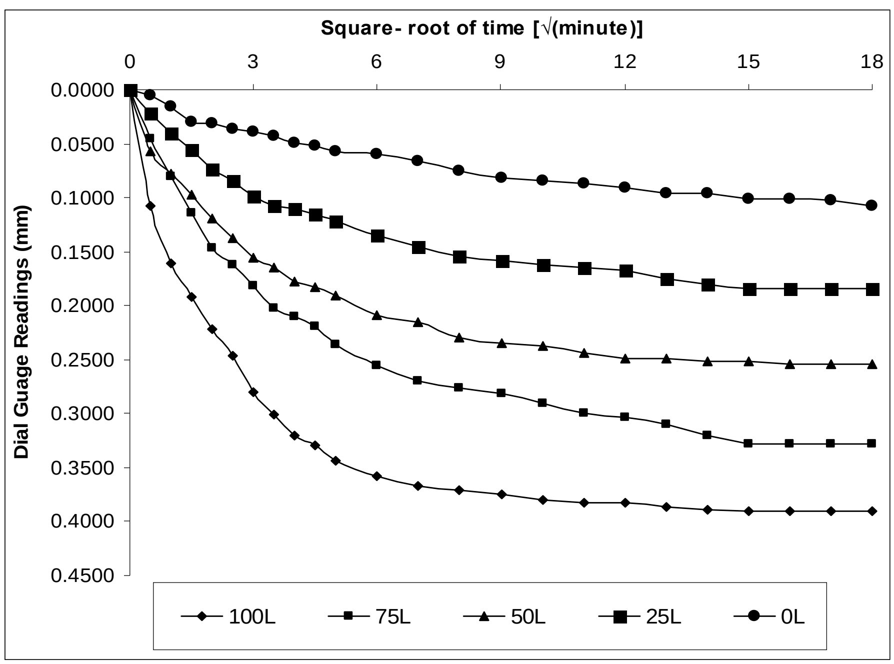

Fig.F.1c Load-Settlement Trend for Preload of 150 kg for *UR* Soil Conditions

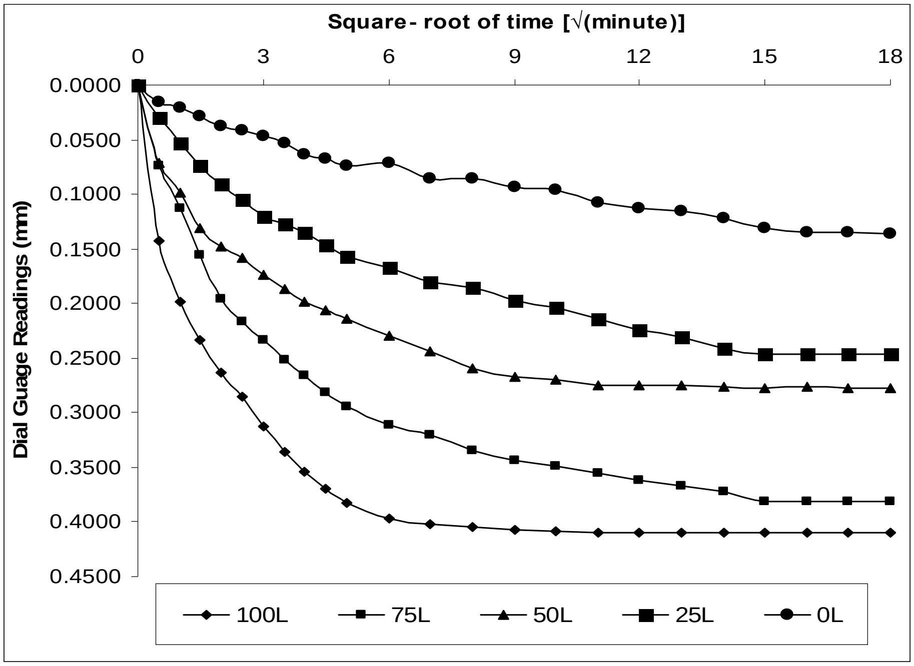

Fig.F.1d Load-Settlement Trend for Preload of 200 kg for *UR* Soil Conditions