NOTICE

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Validity of Small Model Testing
in
Foundation Engineering

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ABSTRACT

Validity of Small Model Testing in Foundation Engineering

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In foundation engineering, the need for laboratory testing arises due to the complexity associated with the theoretical analysis. Laboratory testing results are usually utilized for developing theoretical models. One of the most essential requirements for these laboratory testing for cohesionless material is the development of the placing technique. Few sand placing techniques can be found in literature. Although they may produce the same sand state, the utilized technique affect greatly the in-situ stress level in the sand. The parameters representing the stress level were not incorporated in design theories because they were either ignored or not recognised during the experimental study. The present study is directed to evaluate the effect of sand placing technique by vibratory densification on the produced mechanical characteristics and the stress level in the sand layers.

The testing tank was equipped with pressure transducers located in predetermined locations to measure the induced vertical and lateral stresses due to compaction. Density cans were placed in different locations in each layer to measure the average unit weight in each layer. Based on the measured unit weight, shear box tests were used to determine the angle of the shearing resistance of the tested sand. The measurements of the vertical and horizontal stresses were utilized to calculate the overconsolidation ratio and the corresponding coefficient of lateral earth pressure.

The factors affecting the mechanical characteristics of the sand due to compaction were identified. An energy factor relating the parameters influencing the energy input to the sand layers was established. Theoretical correlations were presented, based on the present experimental results, to relate the energy input to the angle of shearing resistance of the sand, unit weight, coefficient of lateral earth pressure, thickness of compacted layer, and the overconsolidation ratio. Proposals for future research are given.

To

My Fiancee Monica

And

To My Family

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Symbol		Chapter
a	= spacing between particles	4
d	= thickness of layer	4
D	= average particle diameter	4
E	= the energy input per unit volume	4
E_F	= the energy factor	4
f	= coefficient of surface friction	4
H	= total height of the sand in the testing tank	4
H_R	= the relative height	4
$K_{O(NC)}$	= coefficient of lateral earth pressure for normally consolidated soil	4
$K_{O(OC)}$	= coefficient of lateral earth pressure for overconsolidated soil	4
n	= total number of layers in the testing tank	4
n'	= rank of the layer under consideration	4
N	= normal force	4
P	= number of particles in a unit volume	4
t	= time of compaction	4
T	= shear force	4
γ	= the average unit weight	4
ϕ	= the angle of shearing resistance	4
δ_F	= final resultant of the relative tangential displacement	4
δ_r	= tangential displacement during reloading	4
δ_u	= tangential displacement during unloading	4

CHAPTER 1

Introduction

The developing of design theories is usually dependent on small model test results. The results of these laboratory testing are often utilized for the development of the failure mechanisms for bearing capacity theories. Usually, the failure mechanism is altered until the predicted values agree with the experimental data. Bearing capacity theories include some factors which depend solely on the angle of shearing resistance of the tested sand and ignore the effect of the insitu stress level in the sand layers. By ignoring the effect of the stress history in the developed theories, the laboratory results were treated as if it was generated for normally consolidated sand. This constitute the main reason that explain the wide discrepancies observed between the existing theories.

The prediction of in-situ state of stress in a sand stratum is of major importance in a wide variety of geotechnical problems. The techniques for determining the stress level in case of cohesive soils are well developed. In case of cohesionless soils, however, an accurate estimation of the state of stress based on the stress history of the soil is rather difficult due to the difficulty of determining the maximum past vertical consolidation stress. This problem arises due to the fact that it is almost impossible to obtain an undisturbed sample of granular soil in the laboratory or to insert field instruments without causing disturbance to the soil. Laboratory tests, of sufficient quality, were developed to measure these stresses within an acceptable range of accuracy. Few investigators have addressed this problem and however, they have achieved varying degrees of success.

Investigator	Test	Placing Technique	Sand State
Dickin and Leung (1990)	Centrifuge	Depositing sand directly over tested package.	Loose
	Centrifuge	Depositing sand from height of 1.97 m.	Medium
	Centrifuge	Depositing sand from a height of 1.85 m. and compacting every 25 mm. layer by a small mechanical compactor.	Dense
Ghaly, A.M. (1990)	Lab.	Layers of 200 mms. compacted thickness by a hand-held air compactor and for different compaction time.	Loose, Medium or Dense
Murray & Gedds (1987)	Lab.	Compacted layers of 51 mm. thickness.	Very dense
		Pouring from a hooper.	Medium - dense
Sutherland et al. (1983)	Lab.	Air-activated, sand spreader device.	Loose, Medium or Dense
Rowe & Davis (1982)	Lab.	Sprinkling from a height of 25 mm.	Loose
		Dropping from a height of 250 mm.	Medium
Anderson & Hanna (1977)	Lab.	Stirring sand in layers of 75 mm. thickness.	Loose
Saglamer, A. (1975)	Lab.	Static loading of sand layers.	Loose, Medium-dense or Dense

Table 1.1 : Sand Placing Technique adopted by a number of investigators .

The stress history of the tested sand; represented by the overconsolidation ratio (OCR); is an important parameter which should be incorporated in theories developed. The mechanical behaviour of sand depend to a large extent on the level of the insitu stresses within the soil mass. Sand placing technique plays an important role in developing these stresses. It is of interest to know that sand placing induces vertical and lateral stresses higher than those may be found for normally consolidated sand.

The effect of the stress history; represented by OCR; on bearing capacity theories have not been recognised yet. Most of the available theories in the literature are based on the results of small model test where placing technique was chosen for convenience. Accordingly, wide discrepancies of theoretical values of bearing capacity for foundations can be found in literature. This can be attributed to neglecting the effect of the stress level inside the prepared sand, because at the time of developing theories, instrumentation and soil testing were not well documented as todays.

In the laboratory testing, sand placing technique is usually developed before conducting the testing program. The mechanical properties of sand (unit weight and angle of shearing resistance) are greatly affected by the method of placing the sand. Table 1.1 shows a survey of sand placing techniques adopted by a number of investigators and the resulting sand type. It can be noticed that a variety of placing techniques can be used to produce the same sand state. For instant, dense sand can be prepared by mechanical compaction, compacted layers, air-activated sand spreader device, static loading or vibrations. Although these techniques may result in dense sand in terms of the unit weight (γ), the angle of shearing resistance (ϕ) and/or the relative density (D_r); the stress level induced inside the sand was not equal for all techniques. However, it was accounted as an important factor considerably affected by the applied compaction degree or dropping level.

The present research program is directed to study experimentally and theoretically the effect of sand placing technique, by a vibratory densification, on its behaviour in the laboratory. Two different energy inputs were applied to the sand. Sand was placed in layers of constant thickness. Compaction was applied to the sand layer according to a calibrated technique. Stress transducers were placed inside the sand deposit to measure lateral and vertical stresses. Density cans were also placed at different positions in each layer to determine the average unit weight of each layer. A chart of the unit weight versus the angle of shearing resistance was prepared and utilized to determine the angle of shearing resistance of each layer corresponding to a specific unit weight. Based on the stress measurements, the coefficient of lateral earth pressure for overconsolidated sand ($K_{O(OC)}$) and the overconsolidation ratio (OCR) were calculated for all tested layers. Having known the value of the angle of shearing resistance, the theoretical coefficient of lateral earth pressure for normally consolidated soils ($K_{O(NC)}$) was calculated. Semi-empirical relationships were proposed to correlate the energy input to the mechanical properties (represented by γ and ϕ) of the sand and the stress level (represented by OCR).

CHAPTER 2

Review of Previous Work and Scope of Research

2.1 General

The determination of soil characteristics and the calculation of soil structure interaction was one of the earliest problems considered in geotechnical engineering. Since the developed lateral earth pressure theories of Couplet (1726), Coulomb (1776) and Rankine (1857), a number of complex theories and analytical procedures have been developed to calculate the bearing capacity of foundations. Most of the theories developed for bearing capacity of foundations were based on experimental observations deduced from small model test results. Little or no attention was given to sand placing technique. However, as early as 1934, Terzaghi noted that preparation of sands in the laboratory by compaction affects the state of stress within the sand mass. These effects have a direct reflection on the behaviour of the sand under the influence of external agents.

2.2 Review of Previous Work

In 1891, Donath [12] introduced the term "Coefficient of Lateral earth pressure" (K_O) referring to the condition that no horizontal yielding in the soil occurs. The definition of K_O, according to Donath, is the ratio of the horizontal (σ_h) to the vertical (σ_v) pressure acting on a soil element as given in equation (1)

$$K_O = \frac{\sigma_h}{\sigma_v} \qquad \text{..... (1)}$$

In 1920, Terzaghi [34] reported the results of a study on the evaluation of the coefficient of lateral earth pressure at rest (K_O) for various soil types. He found that the value of K_O for coarse sand was in the order of 0.42. Also, he stated that when the sand was compacted in layers with hand compactor, the value of K_O increased and became in the range of 0.6 to 0.7. The values reported by Terzaghi was the first attempt to evaluate K_O numerically, however he did not provide any information with respect to the adopted compaction procedure.

In 1944, Jáky [21, 22] conducted theoretical studies on the evaluation of the coefficient of lateral earth pressure (K_O). He introduced the first semi empirical expression for evaluating K_O for normally consolidated soils in terms of the angle of shearing resistance ϕ (Eq. (2)):

$$K_O = \left(1 + \frac{2}{3}\sin\phi\right)\left[\frac{1 - \sin\phi}{1 + \sin\phi}\right] \qquad \text{..... (2)}$$

In 1948, the expression given by Jáky (Eq. (2)) was further simplified as follows (Eq. (3)):

$$K_O = 1 - \sin\phi \qquad \text{..... (3)}$$

Equation (3) is widely used for the evaluation of K_O because of its practical significance and attractive simplicity.

In 1963, Hendron [19] reported a comprehensive study on the behaviour of sand in one dimensional compression. He indicated that the value of K_O increased as the angle of shearing resistance (ϕ) decreased, up to $\phi = 20^o$, after which the value of K_O for a given overconsolidation ratio (OCR) decreased. He reported that the value of K_O for non-angular sand were always lower than that for angular sand having the same value for the angle of shearing resistance (ϕ). Hendron, suggested that the angle of shearing resistance may not be completely a unique parameter for the value of K_O. Hendron proposed the following expression to calculate the value of K_O in terms of ϕ:

$$K_O = \frac{1}{2}\left[\frac{1 + \frac{\sqrt{5}}{8} - 3\frac{\sqrt{5}}{8}(\sin\phi)}{1 - \frac{\sqrt{5}}{8} + 3\frac{\sqrt{5}}{8}(\sin\phi)}\right] \qquad \text{..... (4)}$$

In 1965, Brooker and Ireland [9] conducted experimental study on the effect of soil stress history on earth pressures. They stated that the value of K_O increased with the increase of the overconsolidation ratio and should eventually became asymptotic with a curve representing the coefficient of passive earth pressure (K_P), where K_P is given by the following equation:

$$K_P = \left[\frac{1 + \sin\phi}{1 - \sin\phi}\right] \qquad \text{..... (5)}$$

They demonstrated that the stress history of the soil governed the value of the coefficient of earth pressure at rest conditions. For the values of the overconsolidation ratio (OCR) greater than about 20, the value of K_O appeared to approach the coefficient of passive earth pressure and probably became equal to K_P.

In 1966, Schmidt [31] presented a series of test results showing the relationship between axial and radial stresses in soils; satisfying the requirement of zero lateral yield. Schmidt derived an empirical mathematical representation of the relation between σ_3 and σ_1 in rebound. This formula is as follows:

$$\frac{\sigma_3}{\sigma_1} = K_{O(R)} = K_{O(NC)}\left(\frac{\sigma_{1\,max}}{\sigma_1}\right)^b \qquad \text{..... (6)}$$

Where b is a non dimensional exponent to be determined experimentally and $\sigma_{1\,max} / \sigma_1$ is the overconsolidation ratio. He also reported that "b" was practically independent of the relative density (D_r) and was uniquely related to the drained angle of shearing resistance (ϕ) of the soil.

In 1967, Alpan [2] analysed the experimental data reported by Wiseman (1962) and Brooker and Ireland (1965) to develop a relation between $K_{O(R)}$, $K_{O(NC)}$ and OCR. He reported the following empirical formula:

$$\frac{K_{O(R)}}{K_{O(NC)}} = R^{\lambda} \qquad \text{..... (7)}$$

Where λ is a factor depends on the mechanical characteristic of the soil. He presented a relationship between the factor λ and the angle of shearing resistance of the soil (ϕ). Alpan found that λ decreased with the increase of the angle of shearing resistance of the soil.

In 1975, Al-Hussaini and Townsend [1] presented results of an experimental investigation on the stress-deformation of sand under K_O conditions; zero lateral strain. A predetermined unit weight of the sand were achieved by depositing the sand in layers and vibrating each layer with hand-held air hummer. They reported that K_O was dependent on the stress history of the sand; represented by the overconsolidation ratio (OCR). They showed that K_O increased with increasing OCR and also with the increase of the applied compacting effort to the sand layers. This may conclude that the sand placing technique plays an important role in the determination of the value of the OCR. They concluded that, for normally consolidated sand, the equation proposed by Jáky (Eq.(3)) provided a good basis to calculate the value of K_O while Hendron's relationship (Eq.(4)) underestimated these values.

In 1975, Bellotti et al. [5] conducted an experimental investigation to study the effect of the overconsolidation on K_O. Tests were conducted on overconsolidated soil samples with OCR of 64 to 96. This ratio was achieved by loading the soil up to maximum vertical stress of 3200 to 4800 kPa, then unloading up to a minimum pressure of 50 kPa. They found that the experimental results were in a good agreement with the relationship proposed by Schmidt [31, 32] (Eq. (6)) and they reported a value of 0.42 for the non-dimensional exponent "b".

In 1975, Saglamer [30] conducted an experimental investigation on the factors affecting the coefficient of lateral earth pressure at rest (K_O) for cohesionless soils. Tests were carried out on loose, medium-dense and dense states. The experiments were performed using three different maximum vertical stress levels of $\sigma_V = 5, 10, 20$ kg/cm^2. He reported that, for normally loaded cohesionless sands, under K_O-conditions, there was a linear relationship between σ_V and σ_H, with $K_{O(NC)}$ as the ratio of the minor principal effective stress to the major principal effective stress. He, indicated that the value of $K_{O(NC)}$ was higher for the loose than for dense one. Saglamer proposed the following equation to calculate the value of K_O for normally consolidated sands:

$$K_{O(NC)} = 0.97 (1 - 0.97 \sin \phi') \qquad \text{......(8)}$$

Where ϕ' is the effective angle of shearing strength. This equation showed an agreement with the empirical expression given by Jáky [21] (Eq. (3)) with a deviation of 1%. He also found that the coefficient of earth pressure at rest in reloading ($K_{O(PC)}$) was lower than that obtained in primary loading. He suggested the following formula for the calculation of the coefficient $K_{O(PC)}$:

$$K_{O(PC)} = 0.75 (1 - 0.80 \sin \phi') \qquad \text{..... (9)}$$

He concluded that the main factor affecting the coefficient of earth pressure at rest is the stress history of the sand represented by OCR which was a major factor influencing the value of K_O. He also concluded that the relationships proposed by Alpan [2] (Eq. (7)) and Schmidt [31] (Eq. (6)) can be used to determine the coefficient of earth pressure at rest during unloading $K_{O(R)}$.

Wroth [35,36] presented an evaluation of the in situ measurements of initial stresses and deformation characteristics. He confirmed the validity of the expression proposed by Jáky [21] (Eq. (3)) to calculate the coefficient of the lateral earth pressure at rest for normally consolidated soils. Wroth proposed the following relationship to calculate the coefficient of lateral earth pressure at rest conditions for overconsolidated soils $K_{o(OC)}$:

$$K_{o(OC)} = K_{o(NC)}\,OCR - \left[\frac{\mu}{1-\mu}\right](OCR - 1) \qquad \text{..... (10)}$$

Where $K_{o(OC)}$ and $K_{o(NC)}$ are the respective values of K_o for overconsolidated and normally consolidated soils, μ is the Poisson's ratio and OCR is the overconsolidation ratio.

In 1976, Meyerhof [25] conducted experimental and theoretical investigations on the bearing capacity and settlement of deep foundations. He proposed a semi-empirical formula for approximate estimation of K_o for overconsolidated soils. This formula is written as follows:

$$K_{o(OC)} = (1 - \sin\phi)\,\sqrt{OCR} \qquad \text{..... (11)}$$

In 1977, Anderson and Hanna [3] performed a series of tests to study the behaviour of anchored retaining walls in overconsolidated sand. They noticed that the stress history of the soil affects the behaviour of the retaining wall. Furthermore, in case of normally consolidated soils K_o obtained from Jáky's formula (Eq. (3)) was in good agreement with the experimental values. In case of overconsolidated soil, the value of K_o increased due to the increase of OCR. They suggested the use of the expression presented by Dayal et al.

(1970) for the determination of $K_{O(OC)}$:

$$K_{O(OC)} = \left(A - B\,e^{-(OCR)}\right) K_O \qquad \text{..... (12)}$$

Where A and B are constants which are dependent on the soil type only. They reported that, the wall movement caused by lateral earth pressure during excavation was greater in the case of wall supporting an overconsolidated sand than in a wall supporting normally consolidated sand.

In 1977, Sherif and Mackey [33] performed an experimental study on the pressure exerted on retaining walls due to compaction of sand deposit. They found that a percentage of the peak lateral earth pressure induced by the process of compaction may remain as residual pressure in the sand. This portion was determined experimentally and found to be in the range of 40 % to 90%. They also reported that previously compacted sands (sand with locked-in compaction stresses) could develop higher rate of increase in the peak earth pressure if subjected to an additional compaction. This pressure may be retained as residual earth pressure which increases upon the completion of compaction. This results in an increase of the coefficient of lateral earth pressure (K_O) and Poisson's ratio (μ).

In 1982, Mayne and Kulhawy [24] investigated the relationship between K_O and OCR for primary loading - unloading - reloading conditions. They reviewed laboratory data from over 170 different soils and evaluated the validity of previous empirical methods for estimating K_O for normally consolidated soils. They reported that the at-rest coefficient remained constant during virgin compression ($K_{O(NC)}$). Any further reduction in the effective overburden stress will result in overconsolidation of the soil. They also presented a common approach to evaluate the value of K_O for overconsolidated soils in terms of the effective angle of shearing resistance (ϕ'), the overconsolidation ratio (OCR) and a new stress history parameter OCR_{max} defined as the ratio between the maximum and the minimum vertical stresses applied to the soil:

$$K_O = (1 - \sin\phi)\left[\left(\frac{OCR}{OCR_{max}^{(1-\sin\phi)}}\right) + \frac{3}{4}\left(1 - \frac{OCR}{OCR_{max}}\right)\right] \quad \text{..... (13)}$$

For normally consolidated soils, OCR_{max} = OCR =1, and equation (13) will reduce to the equation proposed by Jáky [21] (Eq. (3)). Mayne and Kulhawy stated that, at present, there appears to be no known technique for determining OCR_{max} of specific soil deposit. This limits the use of the above formula to calculate the value of K_O. They indicated that good knowledge of local geology and stress history of a given soil deposit may lead to a proper evaluation of OCR_{max}. Mayne and Kulhawy reported that, for overconsolidated soils during swelling or rebounding, the value of OCR_{max} = OCR and the above given equation will reduce to the form:

$$K_{O(OC)} = (1 - \sin\phi)\ OCR^{(\sin\phi)} \quad \text{..... (14)}$$

They concluded that theoretical relationship for $K_{O(NC)}$ of normally consolidated soils introduced by Jáky appears valid for cohesive soils and moderately valid for cohesionless soils. They also stated that the variation of the coefficient of lateral earth pressure during unloading $K_{O(U)}$ with OCR during unloading is approximately dependent on the effective stress friction angle of shearing resistance (ϕ').

In 1983, Bellotti et al [6] presented a comprehensive review of the existing experimental evidence relating K_O to the stress history represented by OCR. They reported that the validity of Jáky's relationship (Eq. (3)) for $K_{O(NC)}$ should in some way be related to the difficulty in determining the relevant value ϕ' due to the nonlinear strength envelope of many of the tested sands they considered.

In 1983, Handy [17] checked the validity of Jáky's theoretical derivation (Eqs. (2) & (3)) and presented an expression for $K_{O(NC)}$ incorporating the angle of sliding friction (ϕ_S). He reported that as the densification of the soil continues, particle interference may change the angle of sliding friction and divert it to a less favourable orientations, adding to ϕ_S and decreasing K_O. Considering the angle (α) is the diversion of the slip angle (ϕ_S), the proposed expression is written as follows:

$$K_O = \left[\frac{1 - \sin\left(\phi_s + \alpha\right)}{1 + \sin\left(\phi_s + \alpha\right)}\right] \qquad \text{..... (15)}$$

In 1983, Schmidt [32] argued that the horizontal pressure could be reduced with time due to aging (secondary compression due to slip occurrence) and relaxation. He suggested an equation for the evaluation of the coefficient of lateral earth pressure similar to Eq. (12) given by Mayne and Kulhawy:

$$K_O = \left[\frac{OCR}{OCR_{max} - 1}\right]\left[OCR_{max} - OCR + (OCR - 1)\, OCR_{max}^{\alpha}\right] \qquad \text{..... (16)}$$

It should be stated here that the problem remains unsolved due to the fact that there is no exact method to determine the value of OCR_{max}.

In 1984, Feda [14] raised two objections against the formula proposed by Jáky [20] (Eq. (3)). First, some experimental results deviate noticeably and cannot be expressed by this formula. Second, the formula claims the dependence of K_O on the strength parameter although K_O is a deformation parameter. For these two reasons, Feda conducted an experimental investigation on dry medium sand, of particle diameter ≤ 2 mm and 90% of grain size in the 0.1 to 1 mm range. Tests were carried out in the standard triaxial apparatus. He found that the maximum $K_{O(OC)}$ did not correspond to the maximum overconsolidation ratio (OCR). He proposed the following expression to estimate K_O for normally consolidated sand:

$$K_{O(NC)} = \frac{1}{4\,K}\left[\frac{1}{\tan^2\left(\frac{\pi}{4}+\frac{\phi_\mu}{2}\right)} + \frac{1}{\tan^2\left(\frac{\pi}{4}+\frac{\phi_{cv}}{2}\right)}\right] \qquad \text{..... (17)}$$

Where ϕ_μ is the angle of intergranular friction for plan strain condition and ϕ_{cv} is constant volume angle of internal friction. K is a constant defined as the ratio between the reversible axial to the total axial strain increments; this constant should be measured experimentally. Feda concluded that the experimental $K_{O(NC)}$ value was dependent on the stress path followed during testing. He also stated that the maximum $K_{O(OC)}$ was close to the Rankine coefficient of passive earth pressure (K_P) and that $K_{O(OC)}$ generally increased with the increase of the overconsolidation ratio (OCR) and reached a maximum, then decreased with further increase of OCR. He confirmed the validity of the formula proposed by Alpan [2] (Eq. (7)) and Schmidt [30] (Eq. (6)) to calculate $K_{O(OC)}$.

In 1986, Duncan and Seed [13] conducted a theoretical study for the evaluation of peak and residual earth pressures resulting from the placement of sand and compaction-induced in earth pressures under K_O conditions. They stated that the strength of the sand was dependent to a large extent , on the stress level within the sand mass, and compaction can significantly increase these stresses. In other words, compaction represents a form of overconsolidation wherein stresses resulting from temporary or transit loading condition are retained to some extent after the removal of the load. They reported also that in previously compacted soils (soil with previously locked-in compaction stresses), additional compaction loading could result in much smaller increase in peak earth pressures during compaction than in uncompacted sand. They concluded that the sand compaction results in an increase of K_O and OCR.

In 1990, Hanna and Ghaly [18] conducted experimental and theoretical investigations on the effect of K_O and overconsolidation ratio (OCR) on the uplift capacity of anchors in sand. They found that sand placing technique affects the stress level within sand layers. They reported that, a mathematical model to include all factors affecting the coefficient of earth pressure at rest (angle of shearing resistance, shape and interlocking of soil particles, porosity, modulus of elasticity, elastic and sliding strain, aging, dilation, compaction effort, stress history and applied stress level) can not be found in literature and difficult to achieve. Hanna and Ghaly concluded that the uplift capacity of screw anchors installed in overconsolidated sand increased with the increase of the overconsolidation ratio (OCR). A mathematical expression was proposed to correlate the uplift capacity of an anchor in overconsolidated sand to that installed in normally consolidated sand.

2.3 Discussion and Scope of Present Research

Theories related to the calculation of bearing and uplift capacities of foundations and lateral earth pressure reported in the literature so far, are based on results of experimental results on small models. These theories were developed based on the assumptions that the soil is homogeneous, isotropic, has constant unit weight and angle of shearing resistance with the depth and the soil is normally consolidated. Accurate laboratory investigations are required to allow better understanding of the variation of sand mechanical parameters and stress level within the utilized soils. Although the value of the overconsolidation and the coefficient of lateral earth pressure at rest conditions varies within the sand with the depth, no attempt has been made to study the variation of the unit weight, angle of shearing resistance, modulus of elasticity and Poisson's ratio with the technique followed for sand placing in the laboratory (Number of layers, thickness of layer, placing technique and compaction effort). It is also believed that the discrepancy between existing theories will vanish, to a great extent, when the above mentioned factors are taken into consideration.

The parameters investigated in the present study are the effect of compaction using a hand-held air compactor on the variation of the unit weight, angle of shearing resistance, coefficient of lateral earth pressure at rest and the overconsolidation ratio of the sand. An energy factor; was established and incorporated in mathematical relationships for the calculation of the coefficient of lateral earth pressure for overconsolidated soil ($K_{o(OC)}$) and the overconsolidation ratio (OCR).

CHAPTER 3

Experimental Investigation and Test Results

3.1 Concept

The sand used in the present experimental investigation was a well graded one in order to achieve a wide range of unit weights and angles of shearing resistance. A hand - held air compactor was used to apply compaction to the sand with various energy input. This was achieved by applying different combinations of compactor frequency and compaction duration.

Vertical and lateral compaction-induced stresses of the sand were required to be measured in order to determine the coefficient of lateral earth pressure of overconsolidated sand and the overconsolidation ratio.

3.2 Experimental Set-Up

Figure 3.1 and Plate 3.1 show an overall view of the experimental set-up used in the present investigation. The testing tank was made of steel plates with dimensions of 1.0 x 1.0 x 1.25 m, length,width, and depth respectively. The thickness of the plates used for the walls and the base is 6.5 mm. The walls were braced with frames of steel angles to increase their stiffness and to prevent their lateral buckling.

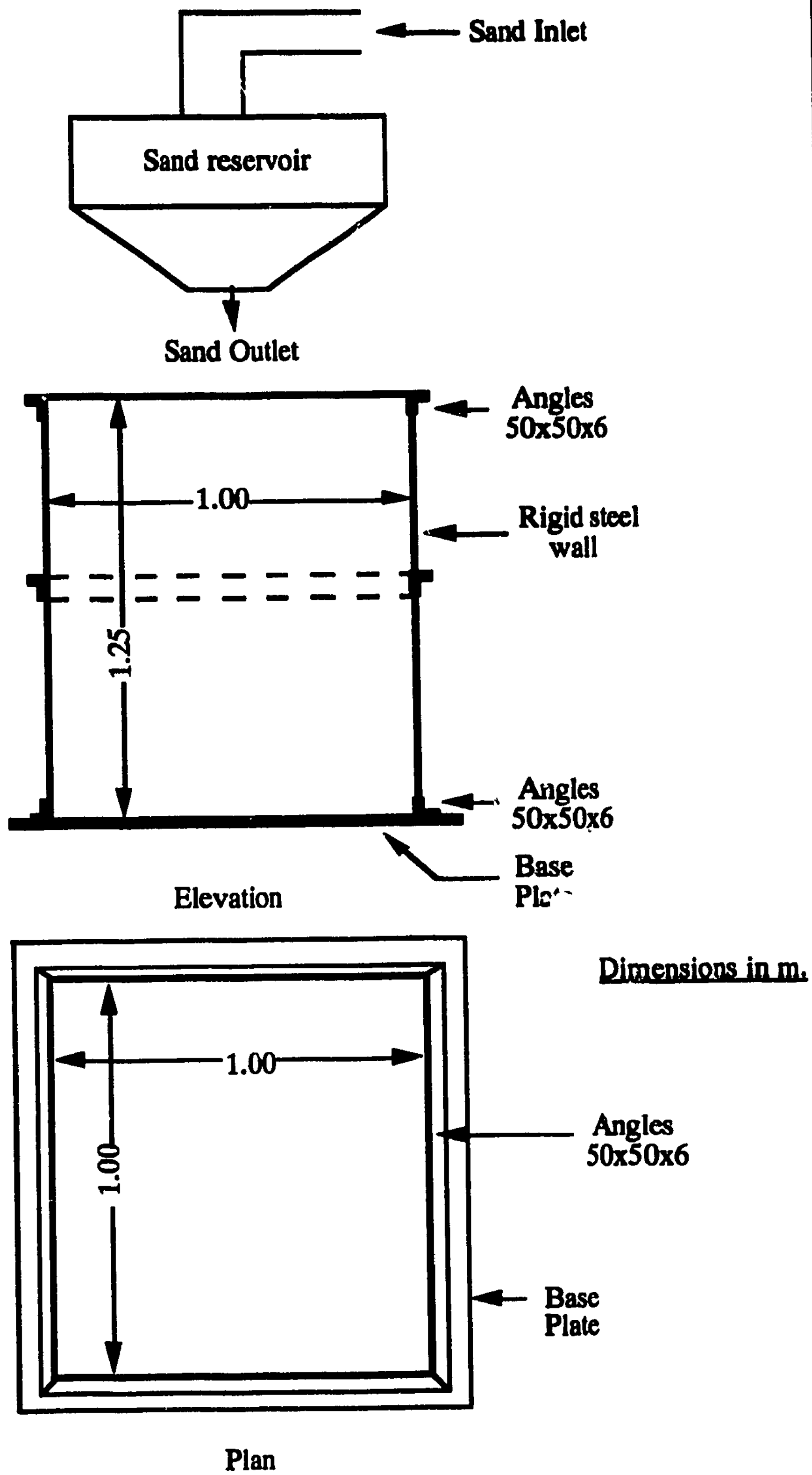


Figure 3.1 : Experimental set-up used in the present investigation

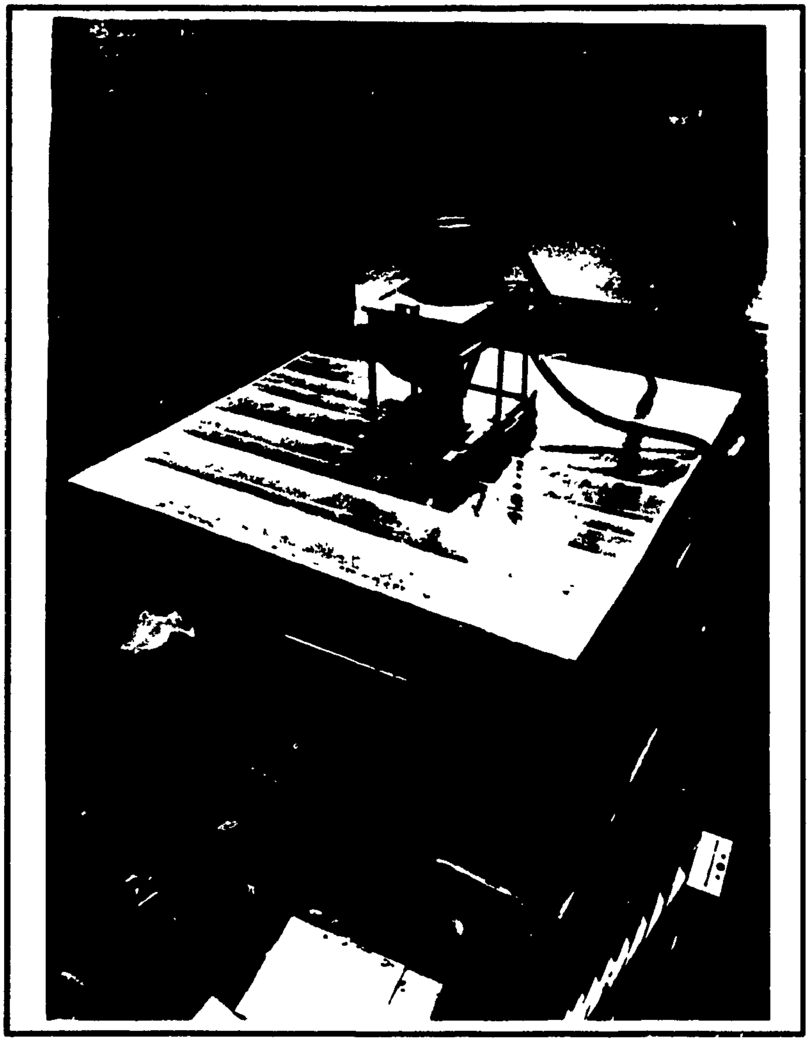


Plate 3.1 : General View for the experimental Set - Up

A sand reservoir of 1.25 m^3 volume, was located above the testing tank where the sand was spread in layer. A hose was used to drop the sand inside the tank from a minimal height to eliminate the effect of dropping distance on the unit weight of the sand.

3.3 Stress Transducer Units

Figure 3.2 and Plate 3.2 show different views of the unit containing the assembly of the stress transducers used in measuring the vertical and lateral stresses. The unit is made of a metal box of dimensions 40 x 40 x 80 mm, height, width and length respectively with three internal machine cut positions for placing the stress transducers inside. Each one of the stress transducers was protected against stress concentration by a surrounding inactive area.

Furthermore, each one of these transducers is functioned to serve the purpose of measuring the stresses acting in the direction perpendicular to its surface. However, two transducers measure the vertical stress after the compaction of each layer, while the third one measures the lateral stress. The design also incorporates a hollow screwed bar attached to the metal box, through which the electrical wires of the stress transducers pass to where they are connected to a Data Acquisition System (DAS) for stress registration. The hollow screwed bar is fixed to the tank wall by a suitable means. The stress transducers were individually calibrated in the laboratory by applying different air pressures to each transducer. Knowing the value of the applied air pressure and the pressure measured by every individual transducer, a calibration factor was calculated for each one. A computer program was developed to calculate the calibration factor utilizing the stresses measured by the Data Acquisition System (DAS) which was automatically dumped into the computer to determine the calibration factors. An appropriate size of O-rings were setted between the

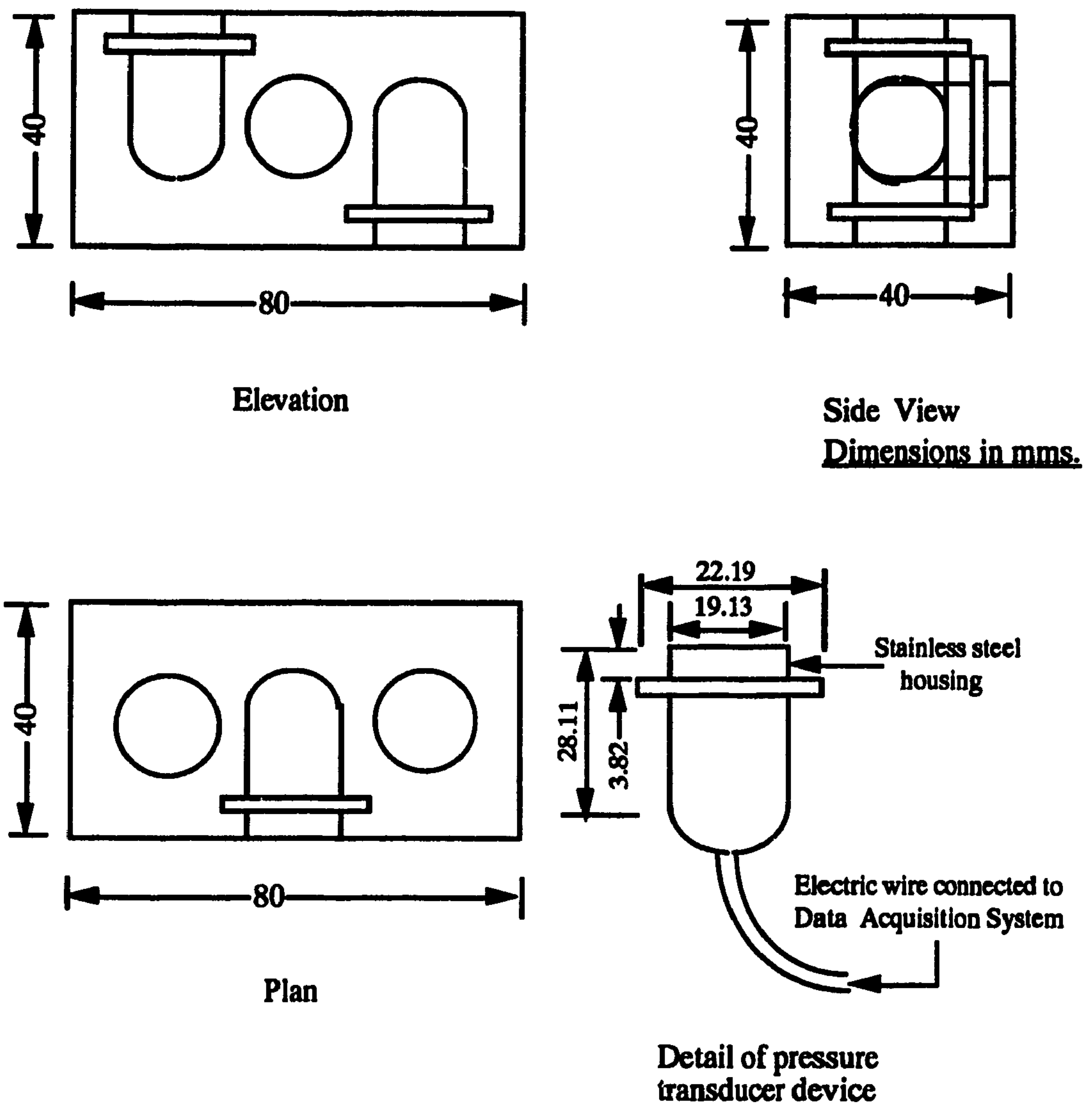


Figure 3.2 : Transducers box unit used to measure lateral and vertical stresses

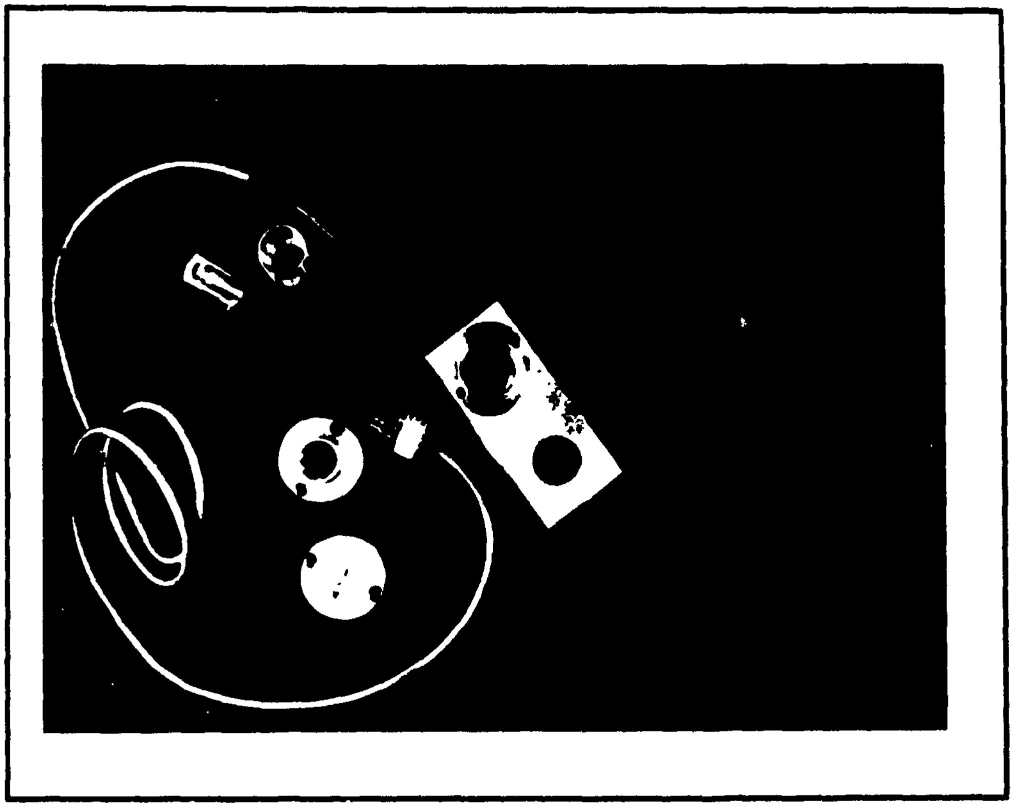


Plate 3.2 : Unit containing the Assembly of the Stress Transducers

area of contact of stress transducers and the metal box to prevent air pressure dissipation through the box.

A typical placement plan of stress transducers units shown in Figure 3.3. Top and corner views of these units are shown in Plate 3.3 and Plate 3.4 respectively. The units are located such that the stress transducers measure the vertical and lateral stresses in different locations of the sand layers.

3.4 Data Reading

The data recorded from the experimental investigation were measured by the stress transducers. A Data Acquisition System (Plate 3.5) was used to measure the stresses in the sand deposit after completing the compaction of each layer. A computer program was developed to record the stresses acting on all stress transducers at the same time. This computer program was set to a manual mode so that it gives the freedom to measure the stresses at any desired time by hitting a special key. This manual mode was used after compacting each layer of the sand, for all the layers below.

3.5 Sand Properties

The present research program was intended to be on loose, medium, and dense sand to duplicate different types of field conditions. Therefore, it was necessary to have a well graded sand in order to obtain the required different relative densities by controlling the degree of compaction applied to the tested sand. For this reason, a sand mixture of the following components was used in the present investigation, Passing Mesh 16: Passing

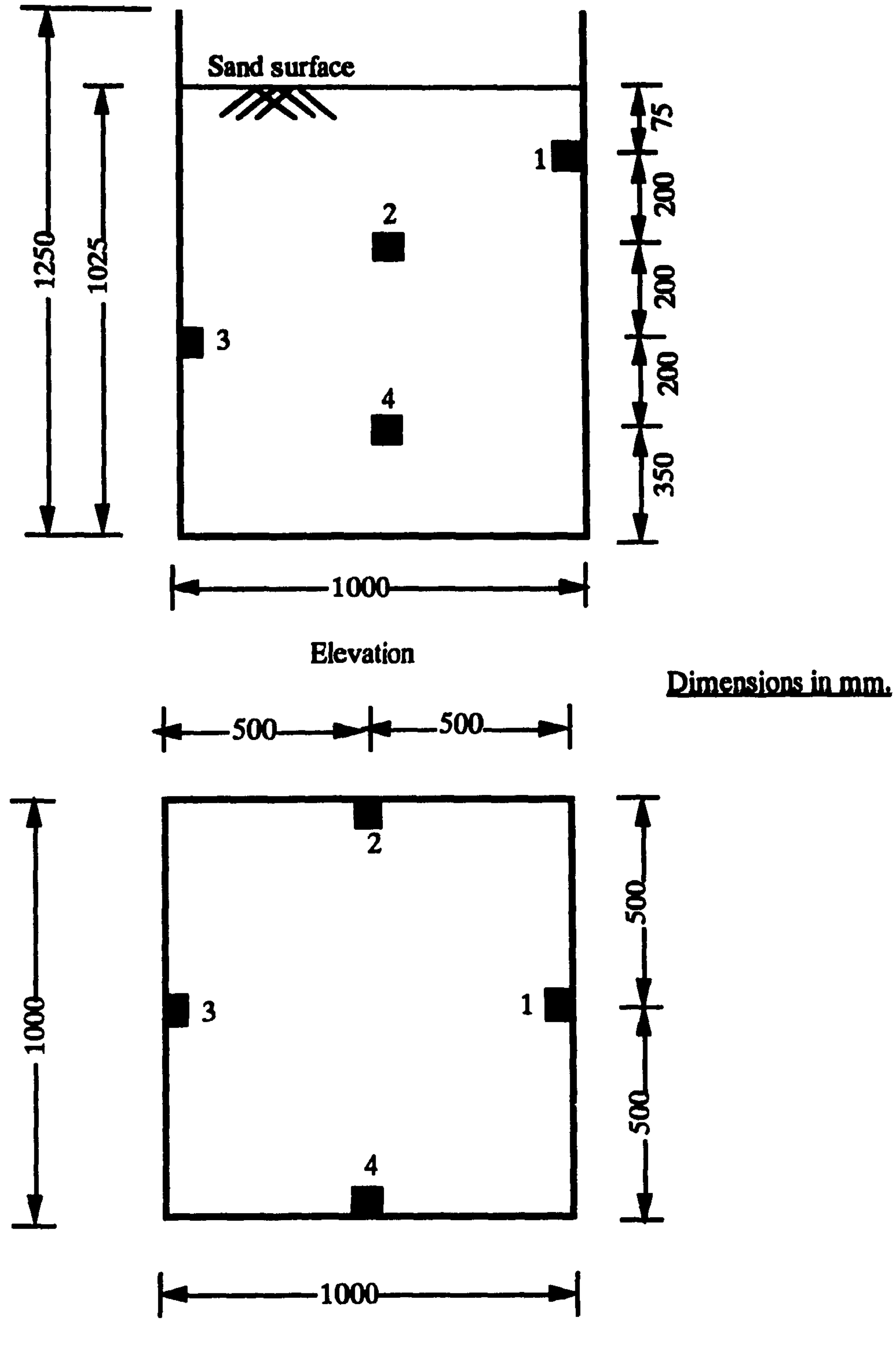


Figure 3.3 : Placement plan of stress transducers box units

Plate 3.3 : Top view showing locations of stress transducers units

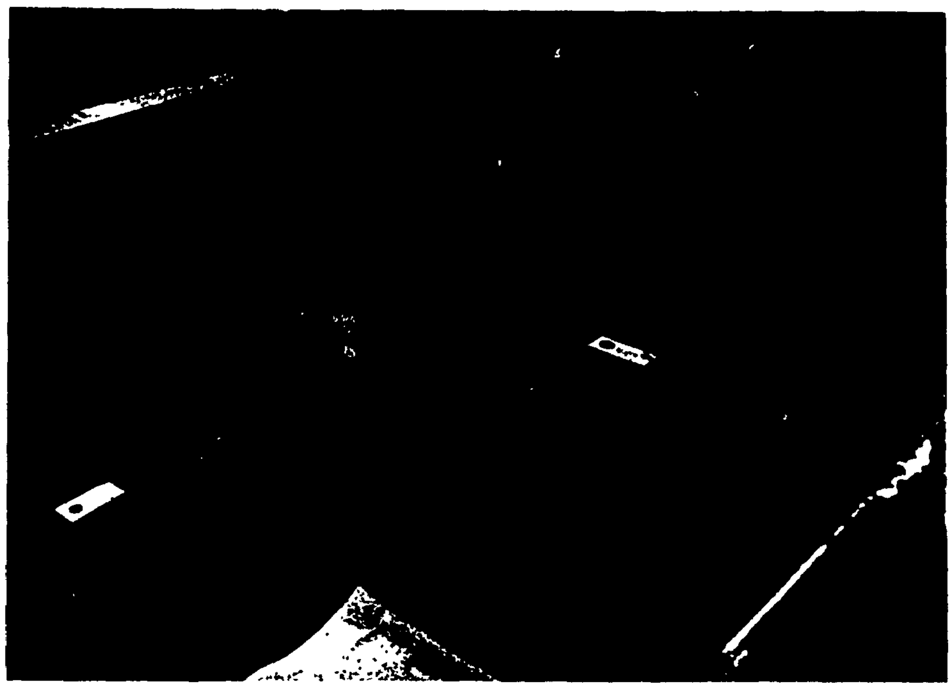


Plate 3.4 : Corner view showing the staggared arrangement of the stress transducers inside the testing tank

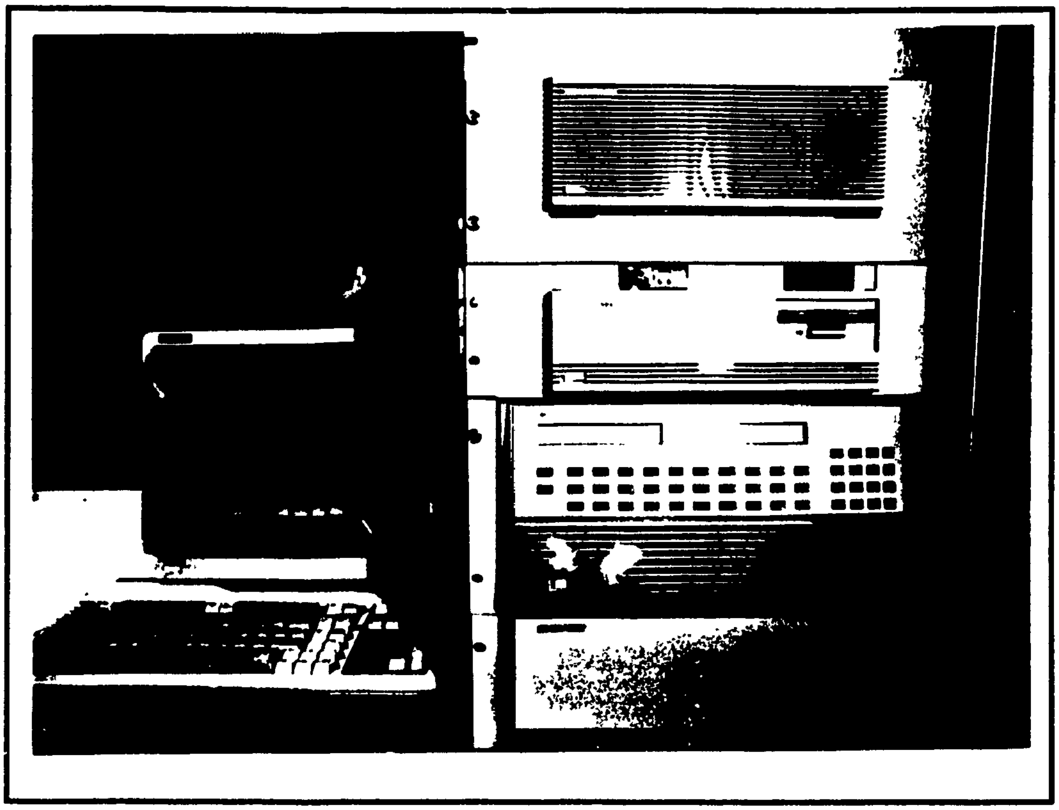


Plate 3.5 : Data Acquisition System and the computer used for measuring and recording the data

Mesh 40: Passing Mesh 100 = 3: 2: 1. The coefficient of uniformity of this sand was C_u= 6.0 and the coefficient of curvature C_c = 2.0.

A grain size distribution (sieve analysis) test was carried out on a sample of the used sand for this experimental investigation. Figure 3.4 shows the mechanical sieve analysis performed on a random sample taken from 2 m^3 of the used mixture. This test together with visual inspection allow the classification of this sand as: Well graded, Clean, Angular, Cohesionless, Homogeneous and Quartz sand.

Table 3.1 summarizes the physical and mechanical characteristics of the of the sand used in the present experimental investigation. Sand was placed in layers of thickness of 150 mm and compaction was applied by means of an air compactor (Plate 3.6). Controlling the compactor acceleration, by adjusting the air pressure and compaction duration, makes it possibie to produce the required energy input. The shear box test was performed to different densities of the sand to determine the angle of shearing resistance (ϕ) corresponding to a given value of unit weight (γ).

3.6 Sand Placing Technique

Placing the sand in the testing tank was carried out by spreading the sand by a hose from a minimal height, a hose-extension were used for the deeper depths (Plate 3.7). This technique was adopted to minimize the sand disturbance caused by the dropping from a great height and to eliminate the effect of the drop distance on the unit weight . Compaction was applied to each layer using an air compactor with an end steel plate of dimensions 125 x 125 x 10 mm length, width, and thickness respectively.

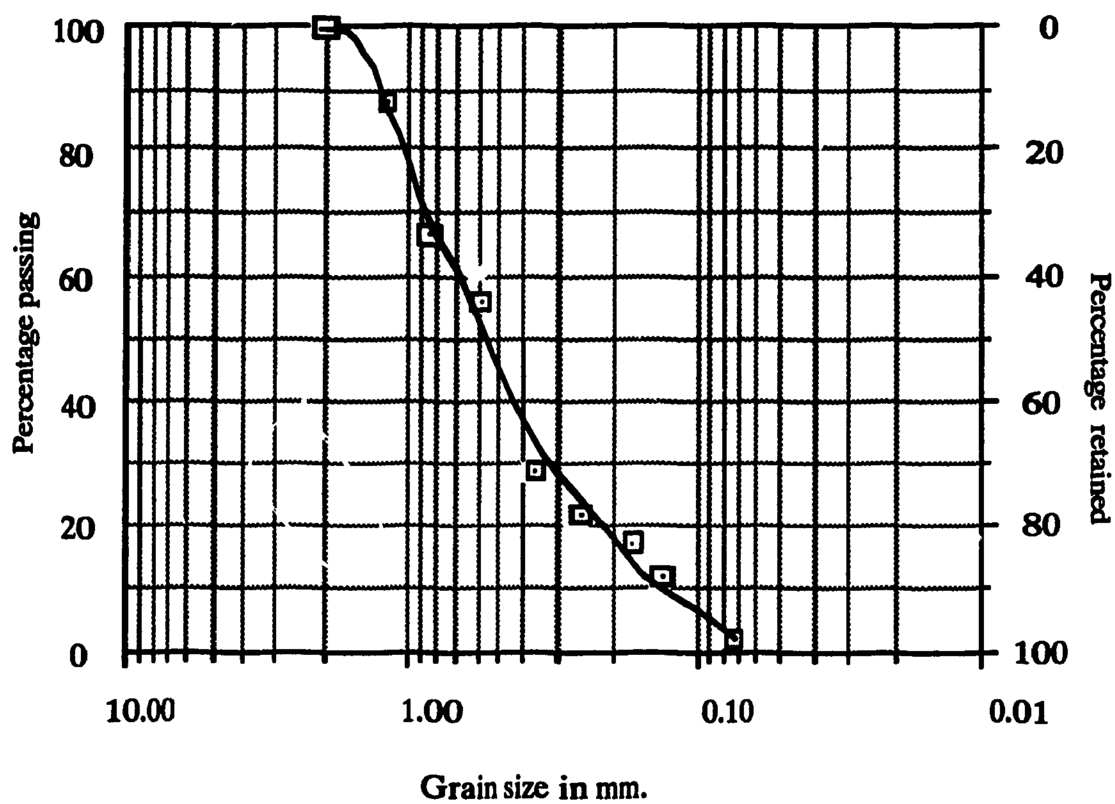


Fig 3.4 : Grain size distribution

Property		Value
Shape of particles		Angular
Classification **		Well graded (SW)
Description		99 % Silica
Coefficient of Uniformity	(C_u)	6.00
Coefficient of Curvature	(C_c)	2.00
Maximum unit weight	(γ_{max})	19.82 KN/m^3
Minimum unit weight	(γ_{min})	17.45 KN/m^3
Maximum voids ratio	(e_{max})	0.52
Minimum voids ratio	(e_{min})	0.33
Specific Gravity	(G_s)	2.70

** As per the Unified Soil Classification System.

Table 3.1 : Mechanical and Physical Characteristics of the tested Sand.

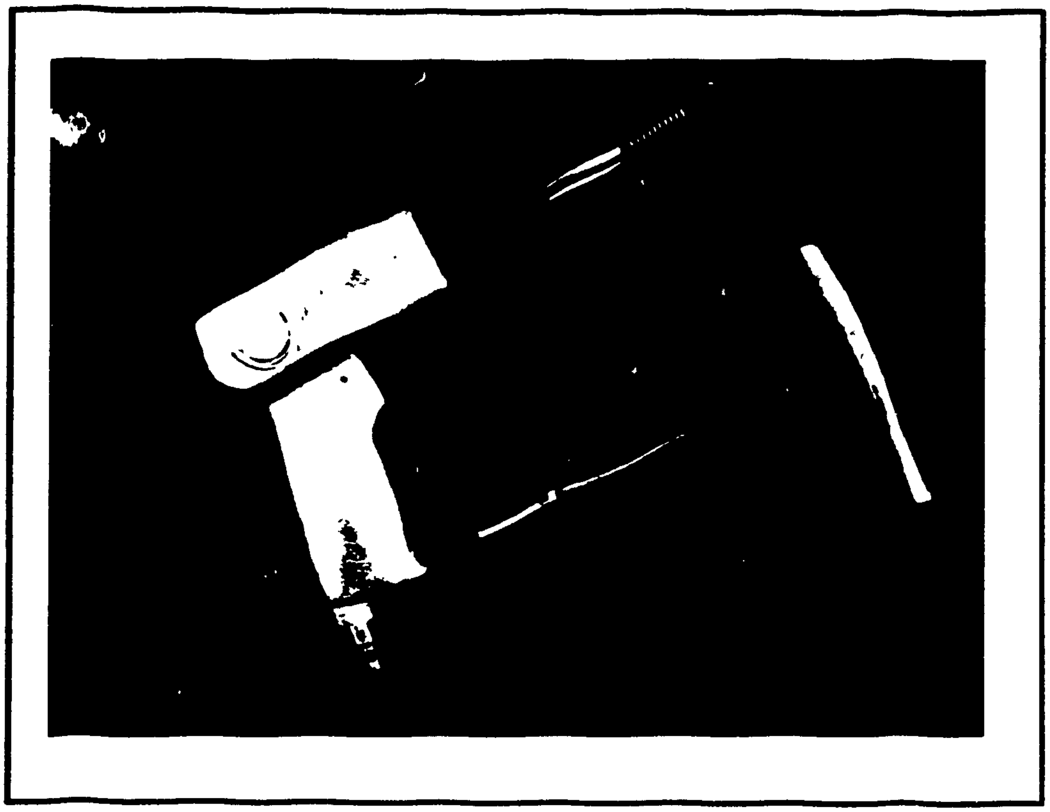


Plate 3.6 : Air compactor used to compact the sand

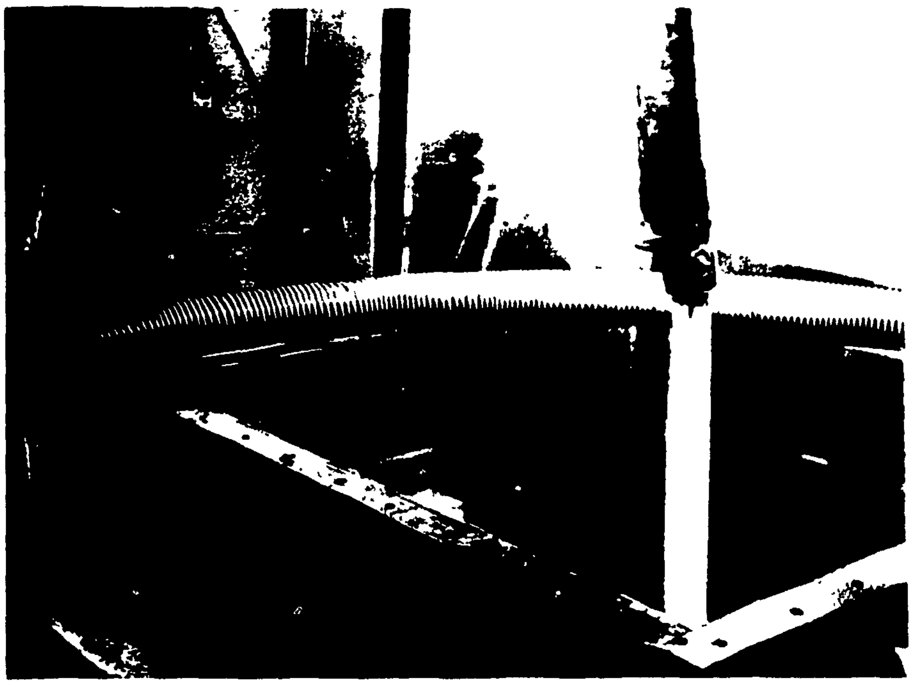


Plate 3.7 : The hose used for spreading the sand

The air pressure used for the air compactor was fixed to a value of 241 KPa, while the range of the compaction duration varied from 3 to 6 seconds, depending on the required unit weight of the sand . Several attempts were made to select the above compaction duration in order to have significant effect of compaction duration on the mechanical properties of the sand. Also, the thickness of the sand layers was chosen so that it gives a considerable respond, to the prespecified energy input. This was fixed to be 150 mm for the entire experimental program. A dead sand layer of thickness of 275 mm was placed at the bottom of the testing tank to reduce the effect of vibration reflection from the rigid base of the tank. Figure 3.5 shows a general layout of the testing tank and the considered layers.

3.7 Test Procedure and Results

3.7.1 Determination of unit weight

A placing technique was employed as described in section 3.6. A sand layer of 150 mm thickness was spread in sub-layers, each of about 25 mm. thickness. The direction of spreading of the sub-layers was perpendicular to each other. The unit weight (γ) of the sand in each layer was taken as the average of three unit weights of samples located in different positions within each layer. The unit weights were obtained by placing three density cans of known weights and volumes in prespecified locations within each layer, then placing and compacting the sand layer. After filling the tank to the required level, cans were taken out from each layer and the unit weights were calculated. An average unit weight of the three cans in each layer was taken as the average unit weight of the layer.

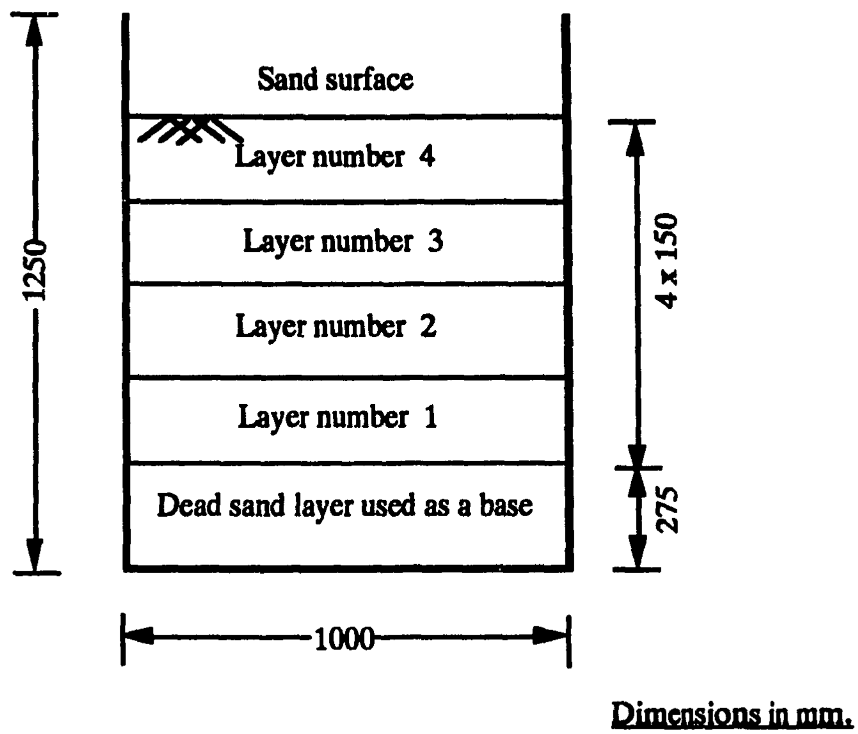


Figure 3.5 : General layout of the testing tank and sand layers

Based on the energy input, a relationship was established between the unit weight (γ) and the angle of shearing resistance (ϕ). The angle of shearing resistance was determined from the direct shear box test under the effect of normal pressure of values similar to those may be found in field projects. Figure 3.6 shows the ϕ-γ relationship where it can be seen that ϕ increases with the increase of the unit weight.

3.7.2 Test Procedure and Discussion

The testing program was divided into two phases:

<u>Phase one: Testing the sand for compaction time of three seconds</u>

The compaction of the sand was carried out by means of a hand-air compactor. The compaction was applied to each layer for a period of three seconds per area spot of the compactor steel plate. When compaction was completed, the lateral earth pressure on the tank walls were measured by the stress transducers and registered by the Data Acquisition System. Then the sand was taken out from the tank by means of vacuum machine to the sand reservoir. The density cans were then taken out, and the unit weight (γ) was calculated. The average of the three unit weights was taken as the average unit weight of the layer.

The above procedure was repeated for two, three, and four layers of sand. A relationship between the unit weight (γ) and the angle of shearing resistance (ϕ) could be

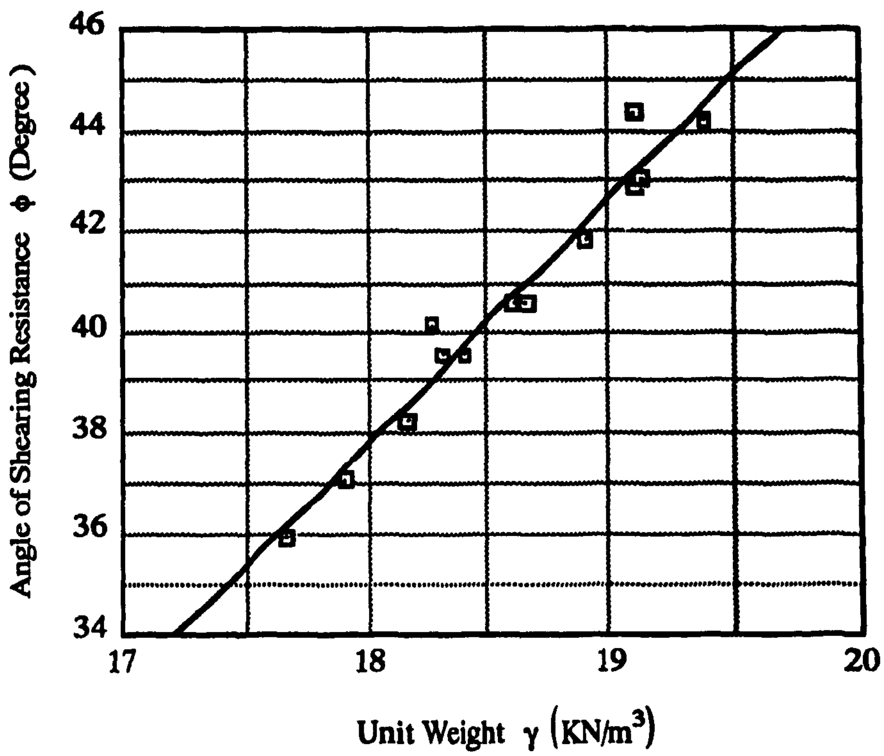


Figure 3.6 : Relation between Unit weight and Angle of shearing resistance

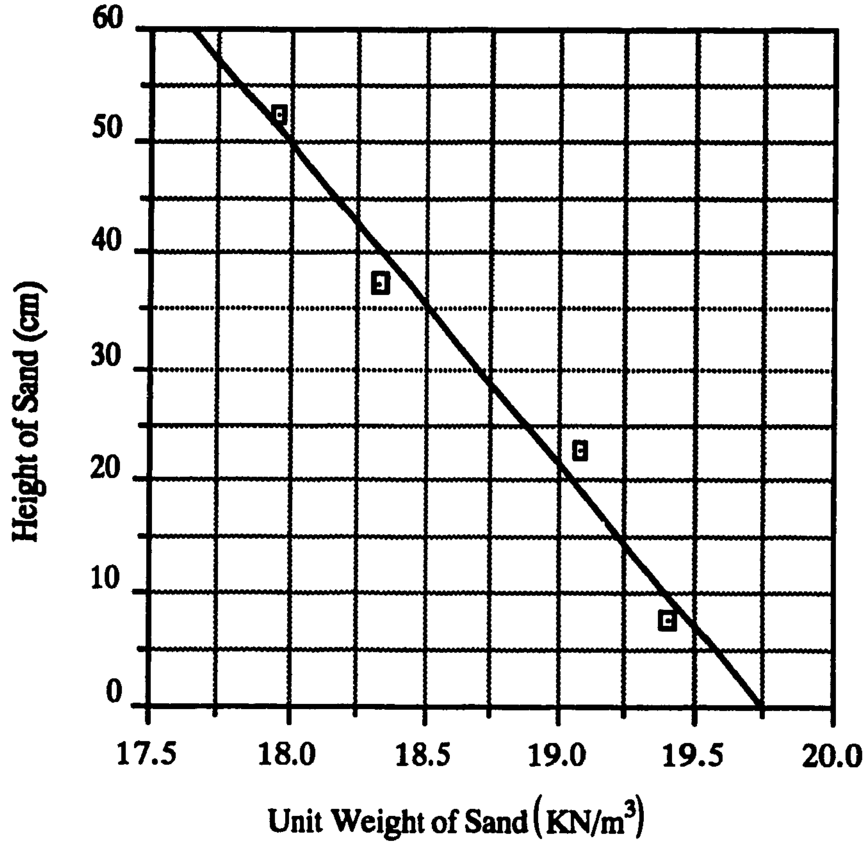


Figure 3.7 : Variation of the Unit Weight with the Height of the Sand for 3 seconds Compaction

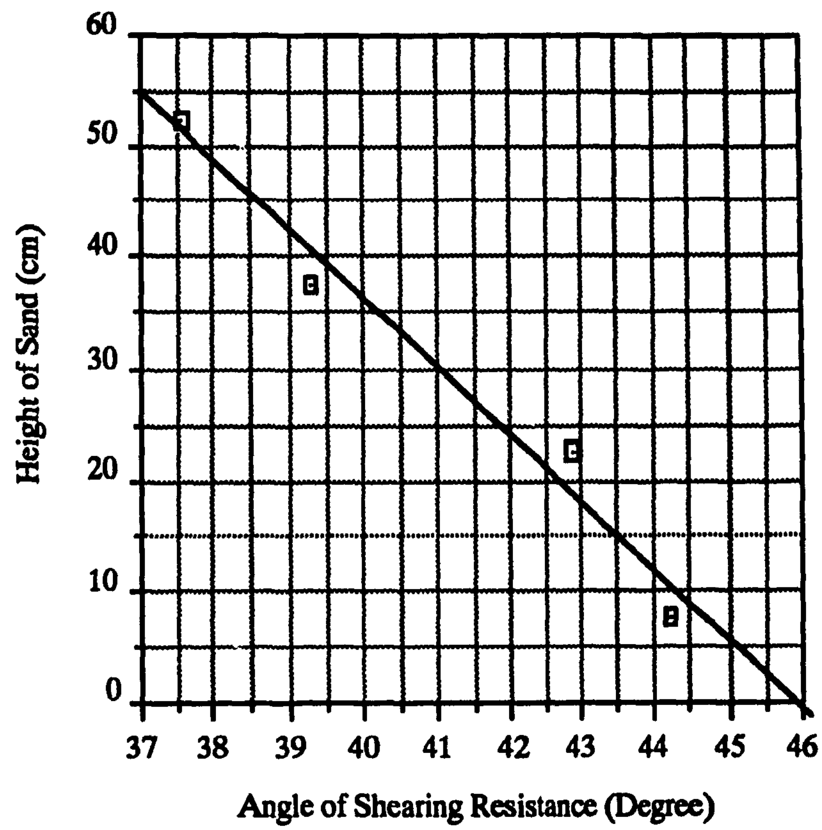


Figure 3.8 : Variation of Angle of Shearing Resistance with the Height of the Sand for 3 seconds compaction .

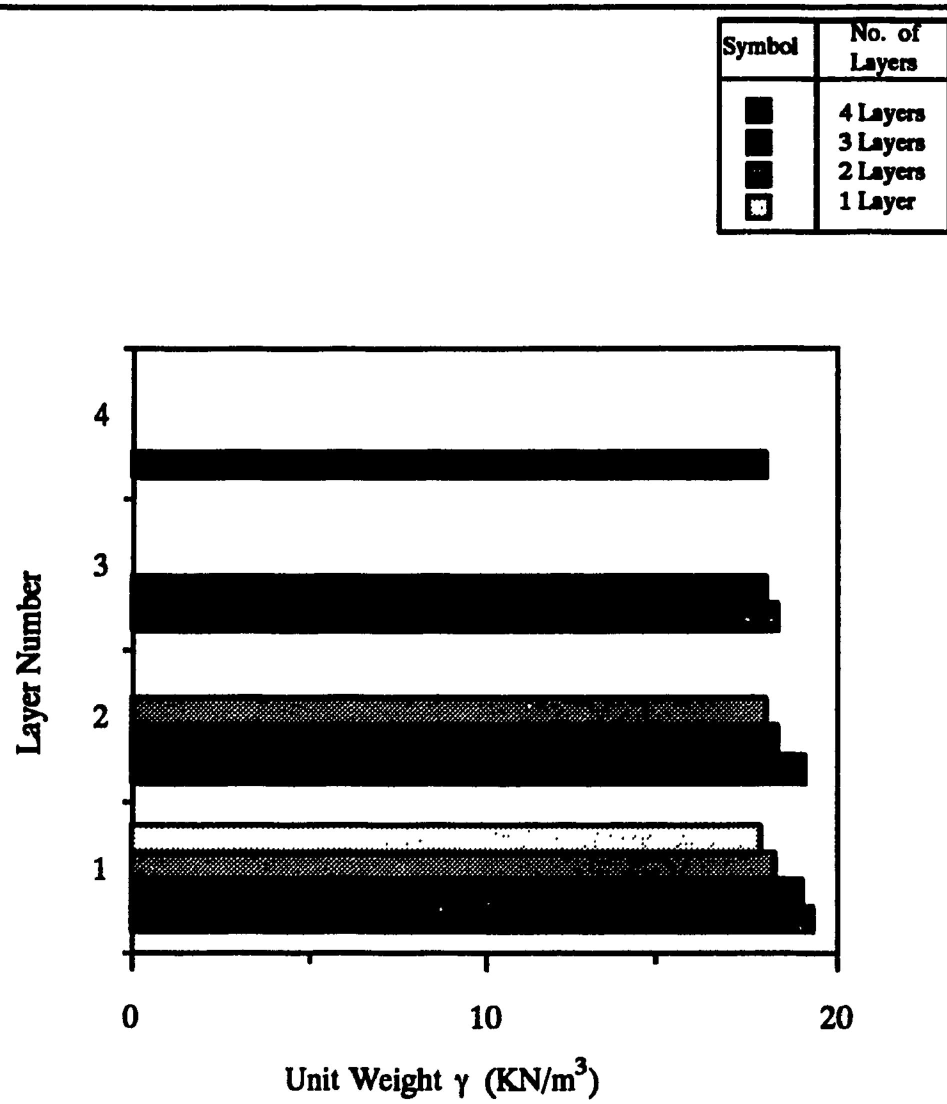


Figure 3.9 : Variation of the Unit Weight with the Number of Layers in the tank for Compaction time 3 seconds .

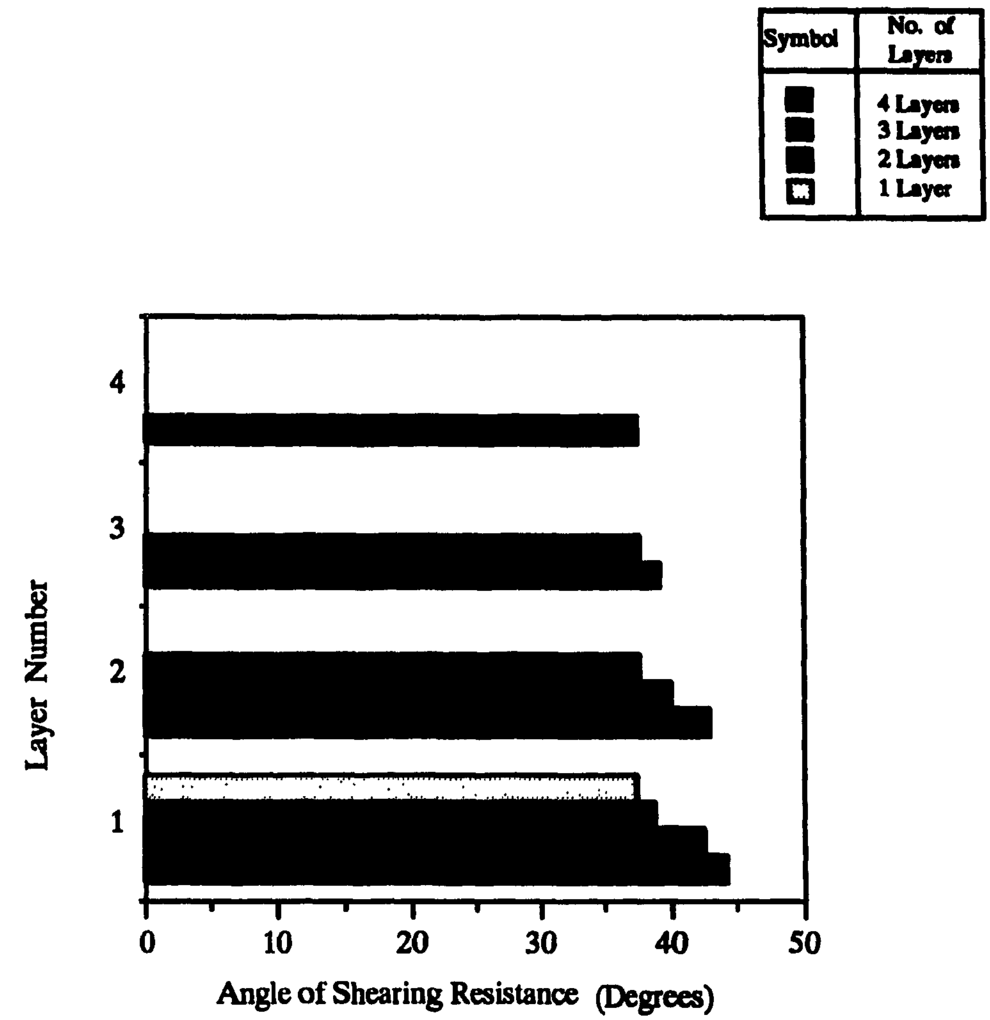


Figure 3.10 : Variation of the Angle of Shearing Resistance with the Number of Layers in the tank for Compaction time 3 seconds .

established, and the corresponding values of ϕ for each unit weight were calculated. Figure 3.7 and 3.8 show the variation of the unit weight and the angle of shearing resistance with the height of the sand in the testing tank, respectively, where it can be seen that both g and f increases with the decrease of the thickness of the compacted layers. Figure 3.9 and 3.10 show a gradual increase in the unit weight and the angle of shearing resistance with the placing of sand layer in the testing tank where it can be noticed that the unit weight and the angle of shearing resistance increase with the decrease of the number of compacted layers. Figure 3.11 shows the variation of the coefficient of lateral earth pressure with the height of the sand for normally consolidated sand (calculated from equation (3)) and overconsolidated sand (measured).

Phase two: Testing the sand for compaction time of six seconds

Figure 3.12 and 3.13 show the variation of the unit weight (γ) and the angle of internal friction (ϕ) with the height of the sand in the testing tank, respectively, where similar observations are reported above could be noticed. Figure 3.14 and 3.15 show a gradual increase in the unit weight and the angle of shearing resistance with the placing of sand layers in the testing tank. Also, Figure 3.16 shows the variation of the coefficient of lateral earth pressure with the height of the sand for normally consolidated sand (calculated from equation (3)) and overconsolidated sand (measured).

Table 3.2 presents the results of the present experimental testing program. The data listed in Table 3.2 are plotted in the following figures where discussion and comments are given.

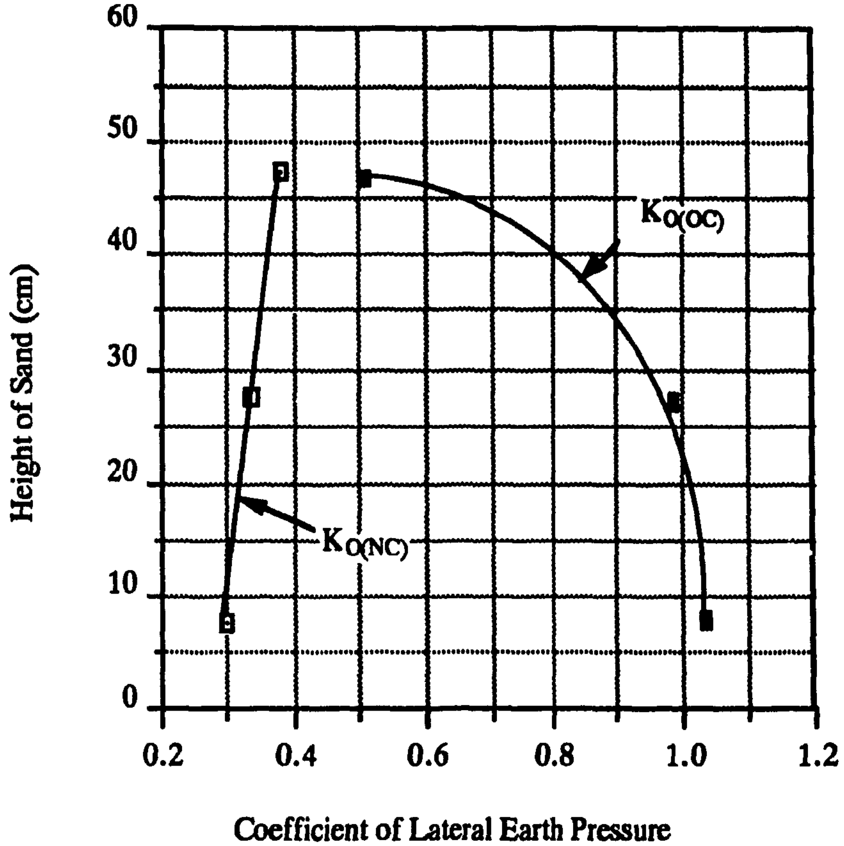


Figure 3.11 : Variation of the Coefficient of Lateral Earth Pressure with the Height of the Sand for 3 seconds Compaction

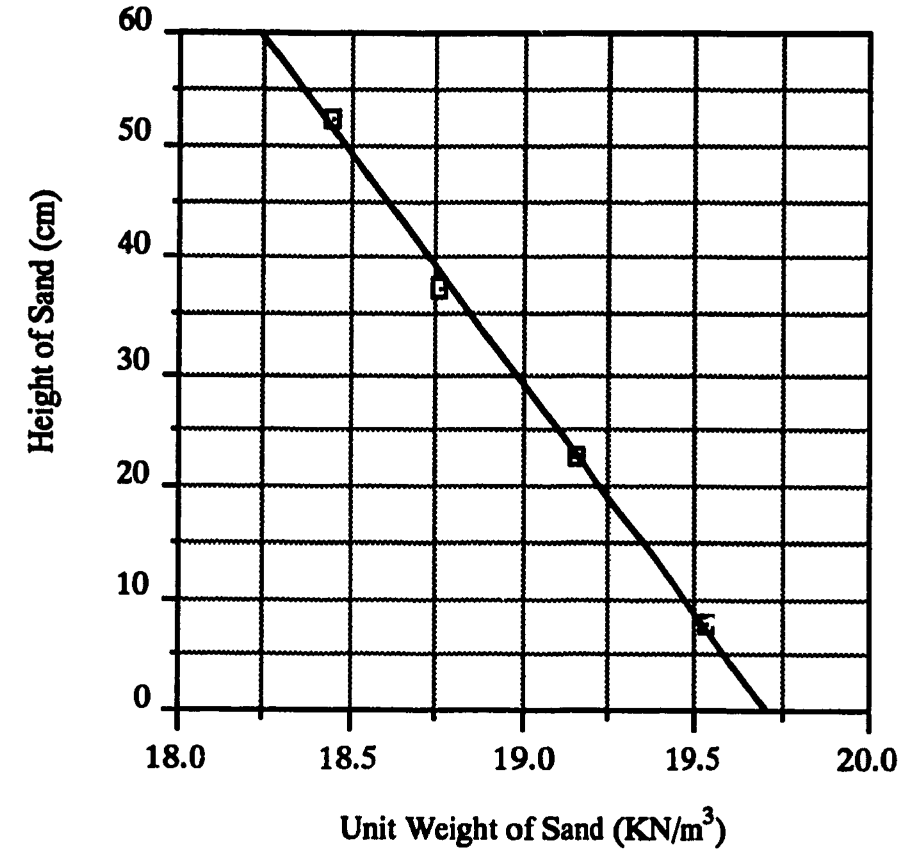


Figure 3.12 : Variation of the Unit Weight with the Height of the Sand for 6 seconds Compaction .

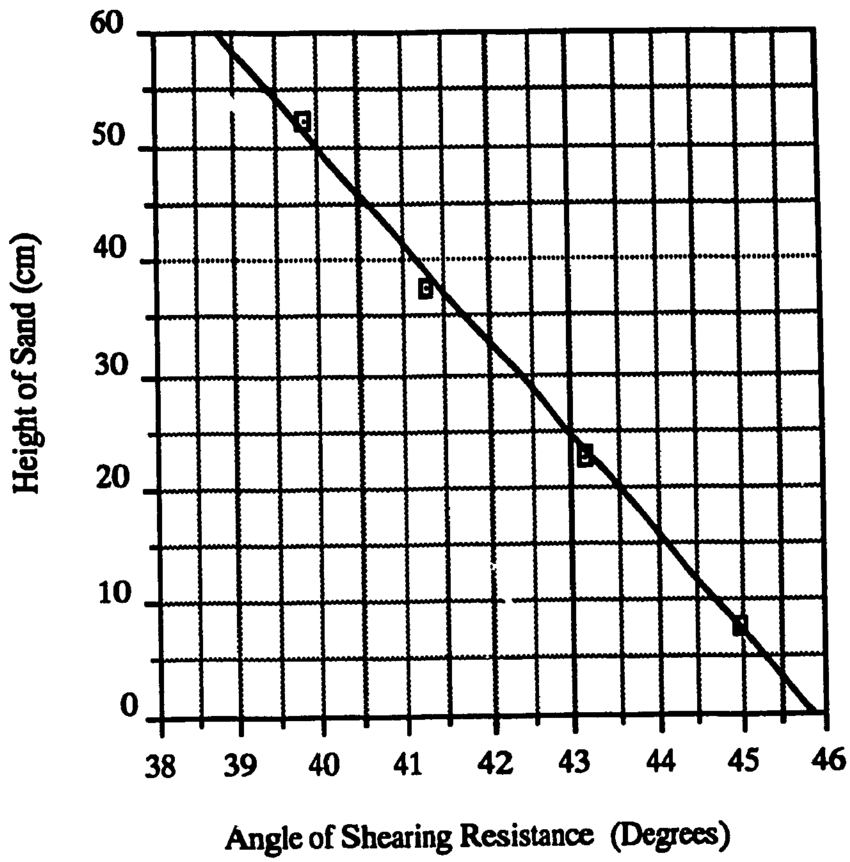


Figure 3.13 : Variation of the Angle of Shearing Resistance with the Height of the Sand for 6 seconds Compaction .

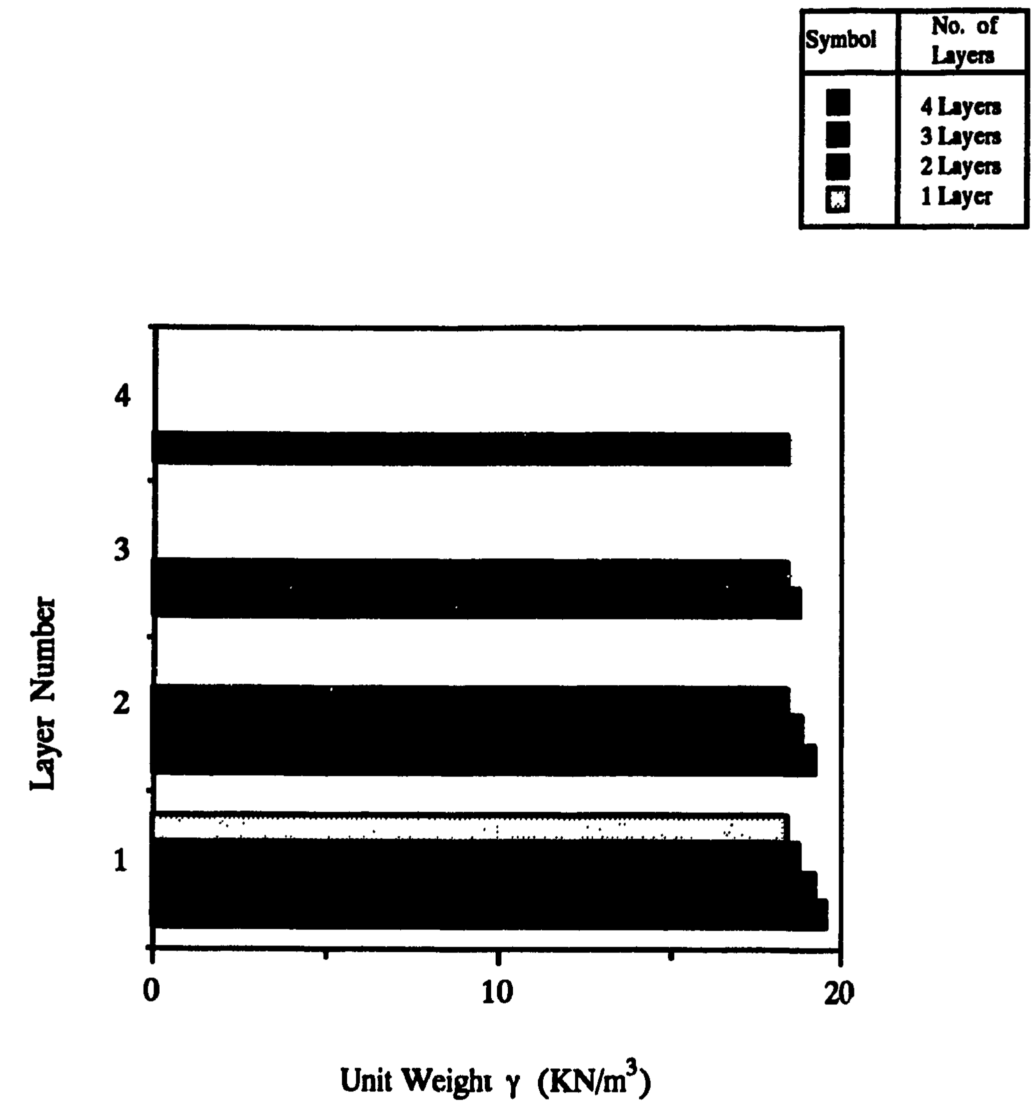


Figure 3.14 : Variation of the Unit Weight with the Number of Layers in the tank for Compaction time 6 seconds .

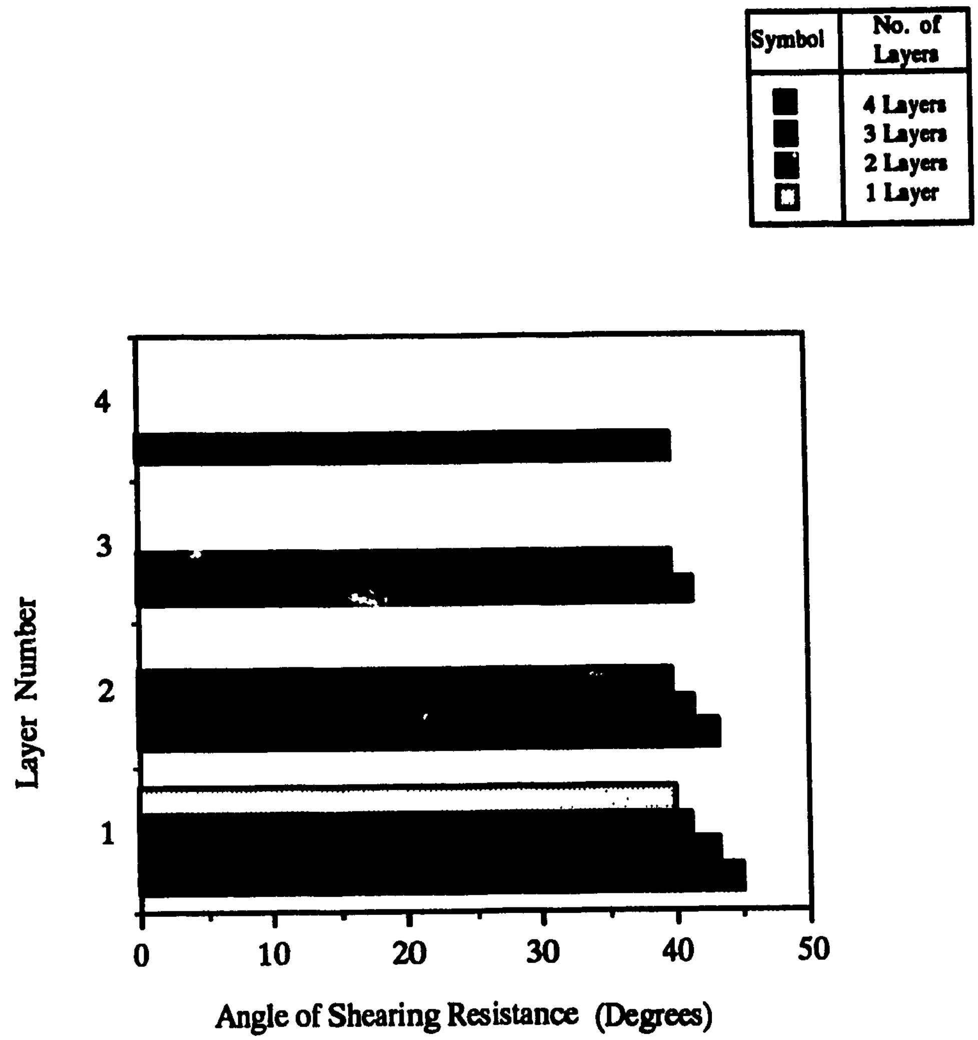


Figure 3.15 : Variation of the Angle of Shearing Resistance with the Number of Layers in the tank for Compaction time 6 seconds .

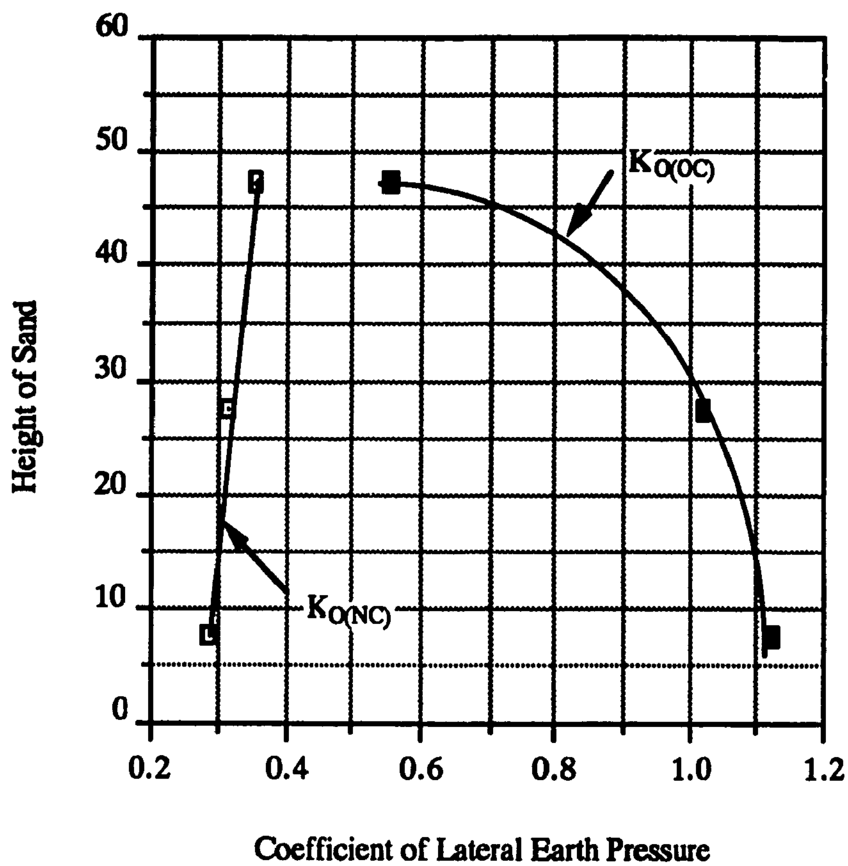


Figure 3.16 : Variation of the Coefficient of Lateral Earth Pressure with the Height of the Sand for 6 seconds Compaction .

Test Number	Compaction Duration (Sec.)	Total Height of Sand, H (m.)	Thickness of Layers , d (m.)	Layer Number , n	Realative Height , H_R	Measured Unit Weight , γ (KN/m^3)	Corresponding Angle of Shearing Resistance , ϕ (Degree)
1	3	0.150	0.150	1	0.500	17.933	37.46
2	3	0.300	0.150	1	0.250	18.247	38.90
				2	0.750	18.001	37.77
3	3	0.450	0.150	1	0.167	19.002	42.44
				2	0.500	18.384	39.94
				3	0.833	17.982	37.68
4	3	0.600	0.150	1	0.125	19.385	44.22
				2	0.375	19.090	42.86
				3	0.625	18.335	39.31
				4	0.875	17.952	37.55
1	6	0.150	0.150	1	0.500	18.462	39.91
2	6	0.300	0.150	1	0.250	18.737	41.20
				2	0.750	18.433	39.77
3	6	0.450	0.150	1	0.167	19.159	43.19
				2	0.500	18.776	41.38
				3	0.833	18.443	39.81
4	6	0.600	0.15	1	0.125	19.532	44.98
				2	0.375	19.149	43.14
				3	0.625	18.757	41.28
				4	0.875	18.443	39.81

Table 3.2 : Results of the present experimental testing program .

Thickness of sand layers placed inside the testing tank was maintained constant. Also, the applied energy (compaction duration and compactor frequency) was similar for each layer within the same test phase. From figure 3.7 and 3.12, it can be seen that the unit weight in the lower layers were higher than those in higher ones. It can be concluded that compaction of higher layers affects the lower ones; and this may be attributed to the effect of the energy transfer. Also, figure 3.9 and 3.14 show the variation of the unit weight (γ) in each layer with the progress of placing sand layers and applying the compaction energy.

For the same type of soil; as the unit weight (γ) increases by applying compaction effort, the angle of shearing resistance (ϕ) increases. Figures 3.8 and 3.13 are plots for the variation of the angle ϕ for full tank and the height of the sand, corresponding to compaction duration of 3 and 6 seconds respectively. It can be noticed from those curves that the process of compaction increases the angle of shearing resistance. Moreover, the compaction of top layers results in higher energy for lower layers than the top ones. This excess in the energy is reflected as an increase in the angle of shearing resistance (ϕ). Figure 3.10 and 3.15 introduce the gradual increase in the angle ϕ within each layer with the progress of placing and compacting the sand layers.

For the previous two phases, theoretical values of the lateral earth pressure at rest condition (σ_O) were calculated based on the theoretical coefficient of lateral earth pressure at rest (K_O) proposed by Jaky [21] for normally consolidated sand:

$$K_{O(NC)} = 1 - \mathrm{Sin}\,\phi$$

horizontal stress (σ_S) and the vertical stress (σ_V) were measured after placing the sand in the tank.

Based on test results, it was found that the lateral earth pressure (σ_S) after placing the sand is higher than the theoretically calculated one for normally consolidated sand ($\sigma_{O(NC)}$). This indicates that high lateral earth pressure induced in the sand layers due to the application of impulsive effort during sand compaction. Similar observations were reported in other studies [2, 3, 4, 8, 12, 14, 23]. Due to the utilized compaction technique (vibratory densification with a hand-held air hammer), the lower layers of the sand were subjected to higher compactive effort. Also, the testing tank may affect the sand layers constituting the soil sample in terms of the following factors:

a. Bottom layers may well be affected by compaction because they overlay the rigid tank base. This factor was minimized by placing a 275 mm as dead height of sand in the bottom of the tank to absorb the wave reflection from the base.
b. Boundaries were rigid steel plates braced with frames of angles which may significantly reflect the influence of vibrations from the boundaries. This results in higher locked-in stresses in the bottom layers.
c. Compaction beside boundaries constitutes considerable degree of distortion and deformation to the particles adjacent to these boundaries. This is a direct reflection of the inability of these particles to move laterally in the direction of the wall.

For the above mentioned factors, it is clear that the application of vibratory densification results in an increase of soil stiffness and consequently in the value of

Poisson's ratio (μ); also, the value of (μ) increases with the increase of the overconsolidation ratio (OCR) [6, 10, 18, 22, 26]. Poisson's ratio (μ) is defined as the ratio of the lateral strain to the vertical strain. Table (3.3) presents a summary of the theoretical and experimental results.

The coefficient of lateral earth pressure after placing the sand ($K_{O(NC)}$) was calculated as the ratio between the measured lateral and vertical stresses at a given depth after placing and compacting the sand. Also, for the determination of the overconsolidation ratio (OCR), the expression proposed by Wroth [34, 35] was used:

$$K_{O(OC)} = K_{O(NC)} \cdot (OCR) - \left(\frac{\mu}{1-\mu}\right)(OCR - 1)$$

Typical values of Poisson's ratio (μ) were taken in the range of 0.12 to 0.16 (Lambe and Whitman [22]) for loose to dense sand respectively. A typical plots for the coefficient of lateral earth pressure for overconsolidated soils ($K_{O(OC)}$), the coefficient of lateral earth pressure for normally consolidated soils ($K_{O(NC)}$) and the height of the sand are shown in Figure 3.11 and 3.16, for three and six seconds compaction time respectively. Figure 3.17 show the variation of $K_{O(OC)}$ and the overconsolidation ratio where it can be seen that the OCR increases with the increase of the coefficient of lateral earth pressure.

Analysis of test results illustrate that the higher the stiffness of the sand, the higher the overconsolidation ratio and the higher the value of Poisson's ratio.

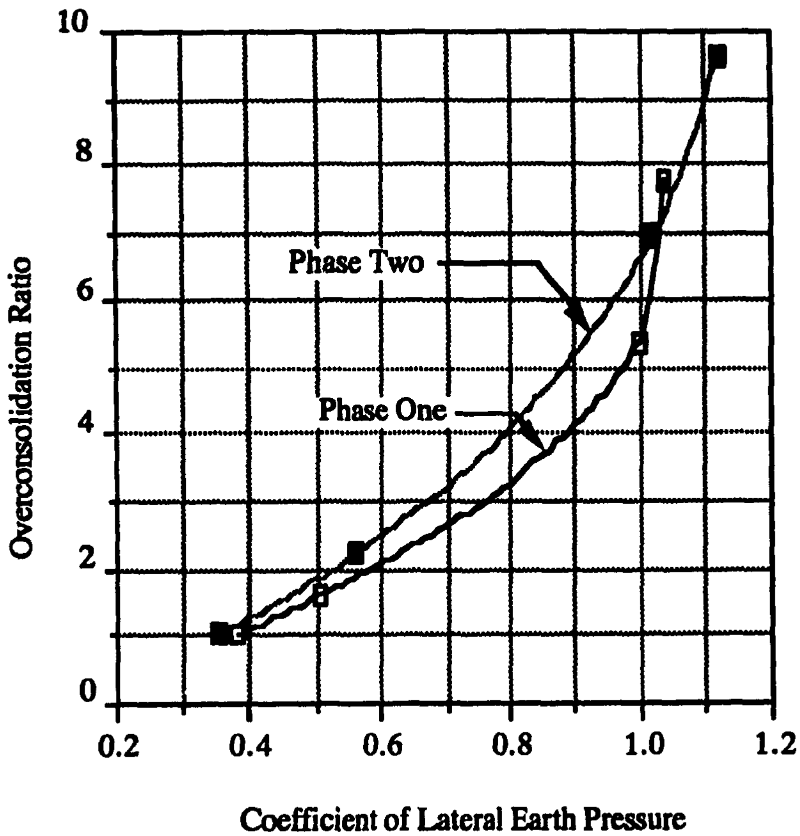


Figure 3.17: Variation of Coefficient of Lateral Earth Pressure with OCR

Test phase	Height	γ	ϕ	$K_{o(NC)} = 1 - \sin\phi$	Theoretical results		Experimental results		$K_{o(OC)} = \frac{\sigma_s}{\sigma_v}$	OCR
					σ_v	σ_h	σ_v	σ_s		
	m	KN/m^3	Degree		KPa	KPa	KPa	KPa		
One	0.475	18.060	38.04	0.384	2.245	0.862	2.684	1.356	0.505	1.57
	0.275	18.796	41.47	0.338	5.920	2.001	11.083	11.051	0.997	5.38
	0.075	19.385	44.27	0.302	10.730	3.240	24.393	25.246	1.035	7.74
Two	0.475	18.541	40.27	0.354	2.305	0.816	3.248	1.826	0.562	2.20
	0.275	19.149	43.14	0.315	6.059	1.908	13.261	13.520	1.020	6.93
	0.075	19.640	45.49	0.287	10.894	3.127	27.535	30.866	1.121	9.64

Table 3.3 : Summary of test results

CHAPTER 4

Analysis

4.1 General

The validity of small model testing results depend to a large extent on the testing conditions. Sand placing technique is an important factor in any laboratory investigations that has a significant effect on the mechanical characteristics and state of stress of the tested sand. The adoption of different techniques may result in sand of the same mechanical characteristics and different stress levels. The application of vibratory densification induces lateral and vertical stresses within the sand mass. These stresses, consequently, result in an overconsolidated sand. The degree of overconsolidation (OCR) depends on the procedure taken for placing the sand.

4.2 Influence of the Placing Technique on the Mechanical Properties of the Sand

As explained earlier in chapter 1, placing techniques can be categorised under three main categories; raining from a height, vibration (shaking), and tamping (vibratory densification). Rad and Tumay (1987) indicated that the relative density of a specimen prepared by raining is a function of the velocity of sand particles immediately before being deposited. This velocity is a function of the drag force and gravitational pull acting on the particles. A sand particle freely leaving the diffuser (raining container) with an initial zero velocity reaches a constant velocity (termed the terminal velocity) after a specific falling height (termed the terminal falling height). Thus, for falling heights above the terminal falling height, relative density (unit weight) is dependent on the actual falling height since the terminal velocity is reached before the particle is deposited.

The energy gained by sand particles during travelling from the diffuser until it reach the deposit level is the main factor controlling the compactness (relative density) of the deposit. The initial velocity of a sand particle leaving the diffuser depends primarily on the sand height in the container and to a lesser extent on particle size, shape of particles (round, angular, subangular, subround, etc.), and size of holes of the container. It can be noticed that the height of sand in the container and the height of sand deposit in the testing tank change with the progress of raining. In order to achieve uniform relative density, the distance between these two levels should be maintained constant, otherwise, the energy gained by sand particles will be variable throughout the height of the sand deposit in the testing tank. Therefore, the deposition intensity (energy input into the particles) is of a strong effect on the relative density of the sand when a raining technique is followed.

The application of vibration to a sand deposit is also an effective method to place the sand according to a desired unit weight (relative density). In this technique, due to the applied vibration, the sand particles adjust their position to take the orientation of lesser spaces between particles (lesser void ratio) and to increase the intensity and number of particles in a unit volume. This technique results in increasing the resistibility of a group of particles since a greater number of these particles acts against the external agents (foundation for example).

The vibration technique is more effective when used to densify angular or subangular sand particles. This is due to the fact that angular particles can easily readjust their position in order to have greater packing per unit volume. It is evident that because of the irregularity of the shape of these particles, a better orientation can be accomplished. This characteristic is not pronounced in the case of round sand particles. This is due to the fact that this type of shape has a limited degree of freedom for orientation into a denser intensity. Therefore, the adoption of the vibration (shaking) technique is constrained to a certain relative density in case of round particles. Higher relative density can be obtained in case of angular and subangular particles. From the above it can be concluded that the reorientation of sand particles (whether for angular or round shape) is mainly dependent on the energy input required for this process. It is of interest to note that the magnitude of energy input is a factor of great importance and should be calculated accurately to obtain the required density. If the maximum unit weight is obtained, an additional input of energy may be of negative effect on the sand deposit.

To overcome the drawbacks of the limited usage of the vibration technique, a tamping technique (vibratory densification) may be used. A tamping technique (sometimes referred to as Impact compaction or vibratory compaction) is an application of vibration with the accompanied effect of a normal force. With the presence of this normal force, higher lateral stresses in the sand deposit (due to confiment) are developed. The normal load, the lateral stresses, together with the falling or oscillating load cause the inter-particle contact and the interlocking between the particles increase. The energy input (compactive effort) greatly influences the above factors and consequently the sand packing. For a given energy input, the relative density or the unit weight in this case is a function of the sand composition, i.e., gradation, grain shape, and mineralogy (chemical adhesion between particles). Due vertical and lateral confiment during the application of a vibratory densification, locked-in stresses remain inside the sand deposit as residual stresses. These stresses are responsible for the increase of the coefficient of earth pressure at rest and the development of an overconsolidation ratio.

It has been argued that an energy input to the sand deposit greater than what the sand can sustain during vibratory compaction may result in grain breakage. Poulous (1988) indicated that grain breakage probably occurs on the top of every compacted layer. Therefore, the effect of breakage is limited. He suggested that a compaction test is preferable to be performed in order to check the effect of grain breakage. Bowles (1989) performed sieve analysis on the sand before and after compaction to study the effect of impact compaction on the excessive soil particle fracture. He concluded that no discernible fracturing took place, and stated that due to pulsating force spreading over the ram (hammer) area, it is not likely to cause much fracturing unless a particle projects in a manner to receive the full impact.

4.3 Factors Affecting Mechanical Properties of Sand

From the above section, it can be concluded that the energy input to a sand deposit is a factor of great importance in the determination of the mechanical properties of the sand. The energy input in case of impact vibration strongly affects the stress system within the sand deposit and reflects its influence on the coefficient of earth pressure at rest and the overconsolidation ratio. The factors affecting the magnitudes of the angle of shearing resistance of the sand (ϕ) and the unit weight (γ) can be summarised in the following:

(a) Factors Affecting the Placing Technique:

1. Energy input per unit volume (E).
2. Period of energy input (t).

3. Thickness of compacted layer (d).

4. Number of layers (n).

5. Rank of the layer under consideration (n').

(b) Factors Related to Sand

1. Shape of particles (angular, round, subangular, subround, etc.).

2. Grain size distribution of the sand.

3. Average particle diameter (D).

4. Spacing between particles (a).

5. Number of particles in a unit volume (P).

6. Mineral composition.

7. Percentage of fines (passing sieve 200).

(c) Factors Related to the Testing Medium

1. Boundary condition of the testing tank.

2. Field conditions (Free boundaries).

The above factors affect the mechanical characteristics of dry sands to different degrees. Some of these factors is of strong effect where the others are of negligible effect. The energy input is, however, the one that reflects strongly on the values of γ and ϕ. The method of calculation of energy input will be explained later, for an impact compaction. This type of compaction has an effective result on packing the sand particles. In the following section, an explanation will be given on the effect of a vibratory compaction on the arrangement of sand particles in a unit volume.

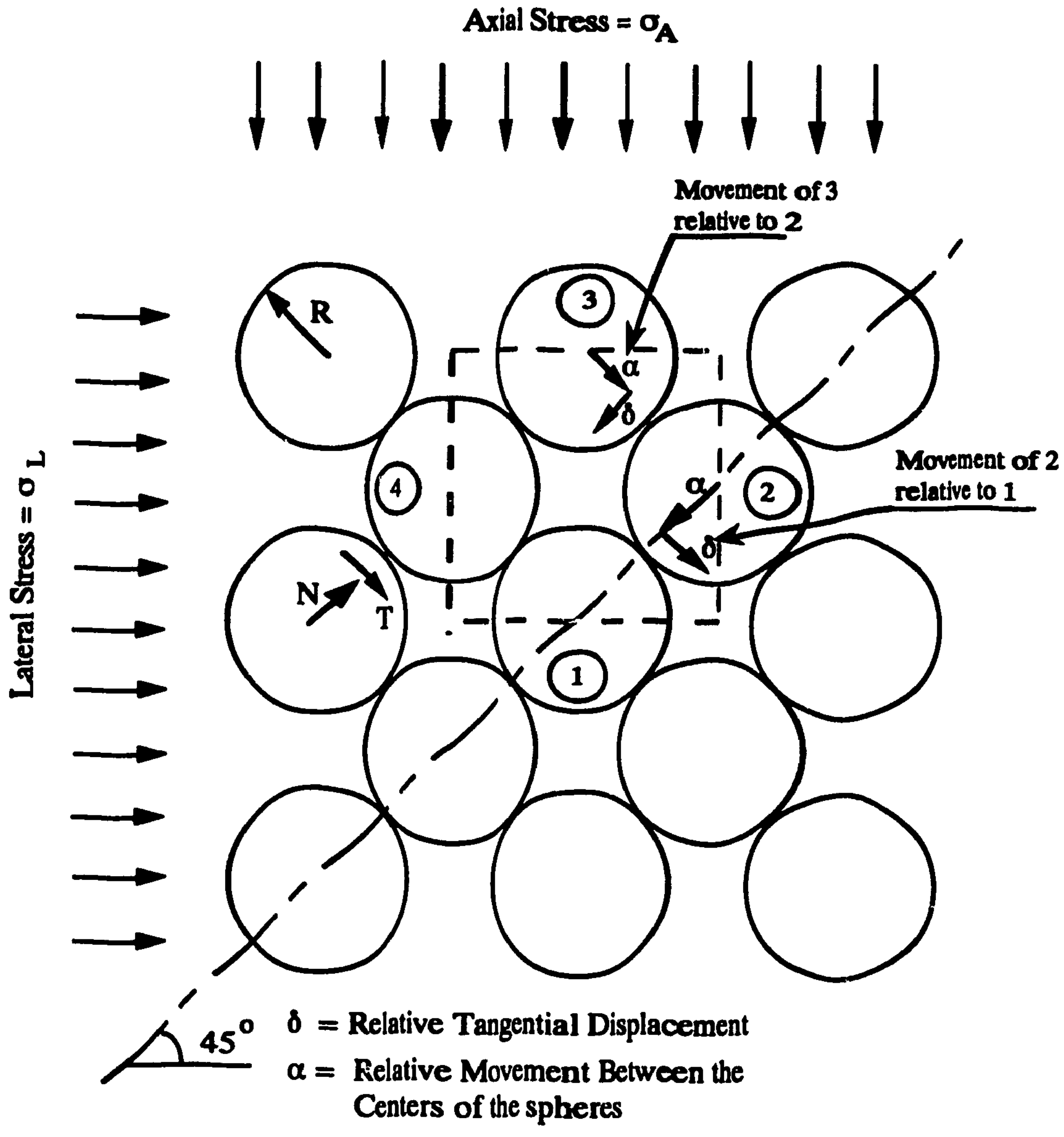


Figure 4.1 : Regular Array of Spheres Showing Unit Element

4.4 Theoretical Calculations for an Idealised Granular System

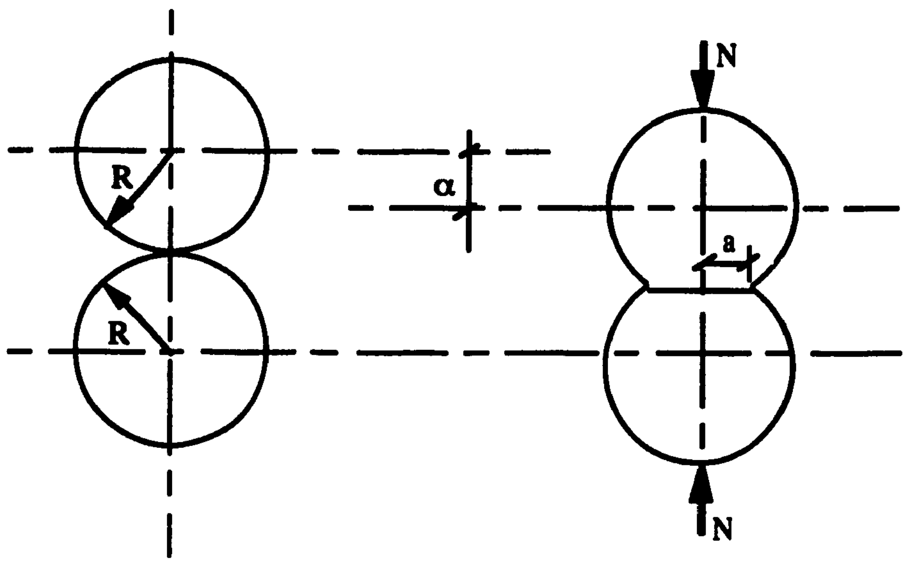
4.4.1 Contact Theory

Mindlin and Deresiewicz (9158) presented a planner array (soil matrix) of elastic spheres as shown in figure 4.1. They assumed that it represents an idealised granular system. The following calculations are based on the contact theories of Hertz (normal forces) and Mindlin Cattaneo (shear forces). These theories are described in detail by Deresiewicz (1958).

When two elastic spheres are pressed together by a normal force N, distortion occur so as to produce a flat contact area (Figure 4.2). When the normal force is removed, the spheres return to their original shape. Thus the normal force versus deformation relationship is nonlinear but reversible (see Figure 4.3). When a shear force T is applied, infinitely large shear stresses are necessary at the edge of the contact area if there is to be no slip on the contact. When T = fN, where (f) is the coefficient of surface friction for the material composing the spheres, relative sliding will occur without further increase of T (see Figure 4.4). Due to load oscillation, during unloading and reloading the behaviour becomes quite complicated. Mindlin and Deresiewicz (1958) presented complex mathematical expression to calculate the relative tangential displacement during unloading (δ_u) and reloading (δ_r) from which the final resultant of the relative tangential displacement (δ_F) due to the oscillating load can be worked out.

4.4.2 Energy Barrier Concept

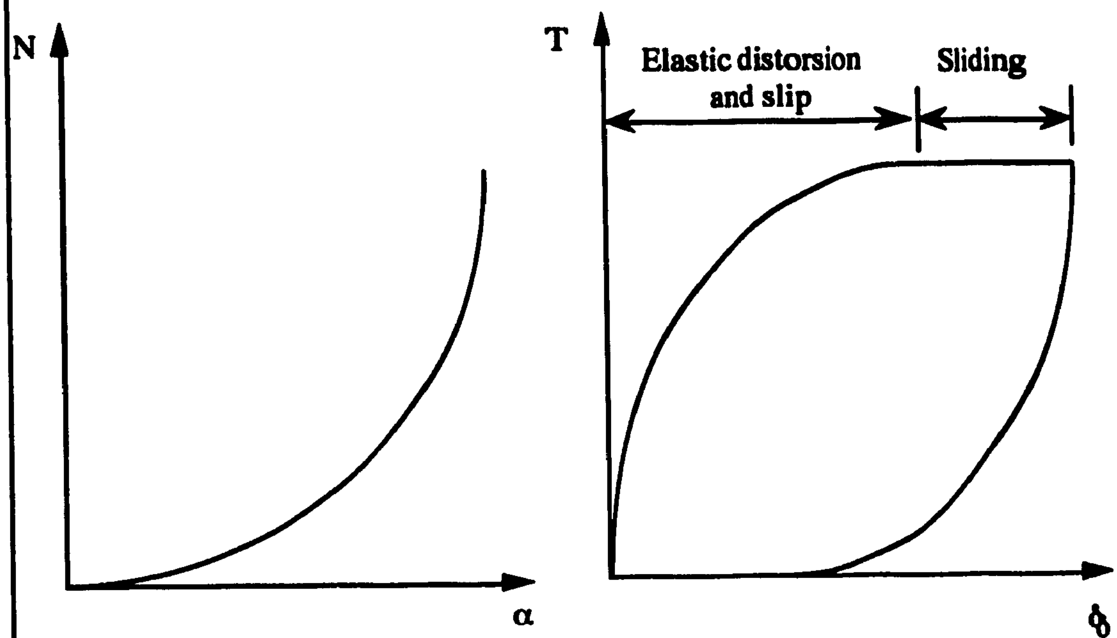
Youd (1970) has introduced the energy barrier concept to illustrate the effect of



Two spheres in contact under zero force

Two spheres in mutual compression under normal force (N)

Figure 4.2 : Deformation of Spheres by Normal Force



(a) Relationship between normal force and relative movement between centers of spheres.

(b) Relationship between shear force and relative tangential displacement

Figure 4.3: Force-Displacement Curves for Two Spheres in Contact

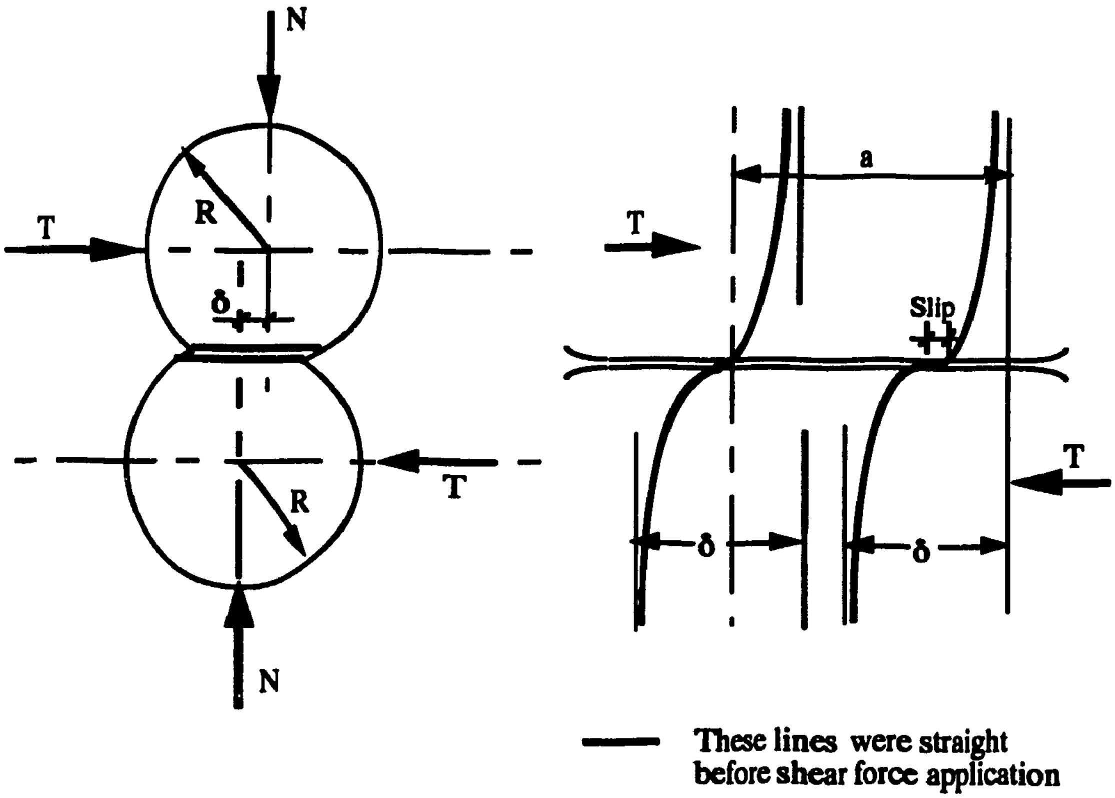


Figure 4.4: Determination of spheres as result of tangential force

densification by vibration on sand. He explained this concept by a mechanical model of semicircular cylinder (see Figure 4.5). A semicircular cylinder T slides or slides and rolls from position A through B to C. Due to friction and interlocking between particles, the movement from A to C passing by B, potential and frictional energy (PE and FE respectively) change within the system. The energy barrier represents the amount of external work (energy) required to cause cylinder (T) to slide from stable position to another. The process shown in Figure 4.5-b simulates densification of sand since densification of a granular material is usually accompanied by a net decrease in potential energy. Youd stated that the magnitudes of both the frictional and the potential energy components of the energy barrier increase with the normal pressure and these components are analogous to the frictional and interlocking components of the coefficient of internal friction.

4.5 Theoretical Correlations

It can be noticed from section 4.4 that the theories available in literature deal with sand particles as perfect elastic spheres (contact theory) or semicircular cylinder (energy barrier concept). This overidealization to the problem results in considerable shifting for the results to an inaccurate direction. However, this idealisation was required since the problem of particles behaviour during compaction is a complex one. The need for a practical approach easy to apply is great. This is not an easy task due to the irregular shape of sand particles and the many possibilities for particles orientation. The use of results of experimental investigation to relate the factors constituting the parameters of the problem of sand compaction can be of great help especially when tests are conducted with great care. Correlations based on experimental data can be an effective tool and valuable guide towards

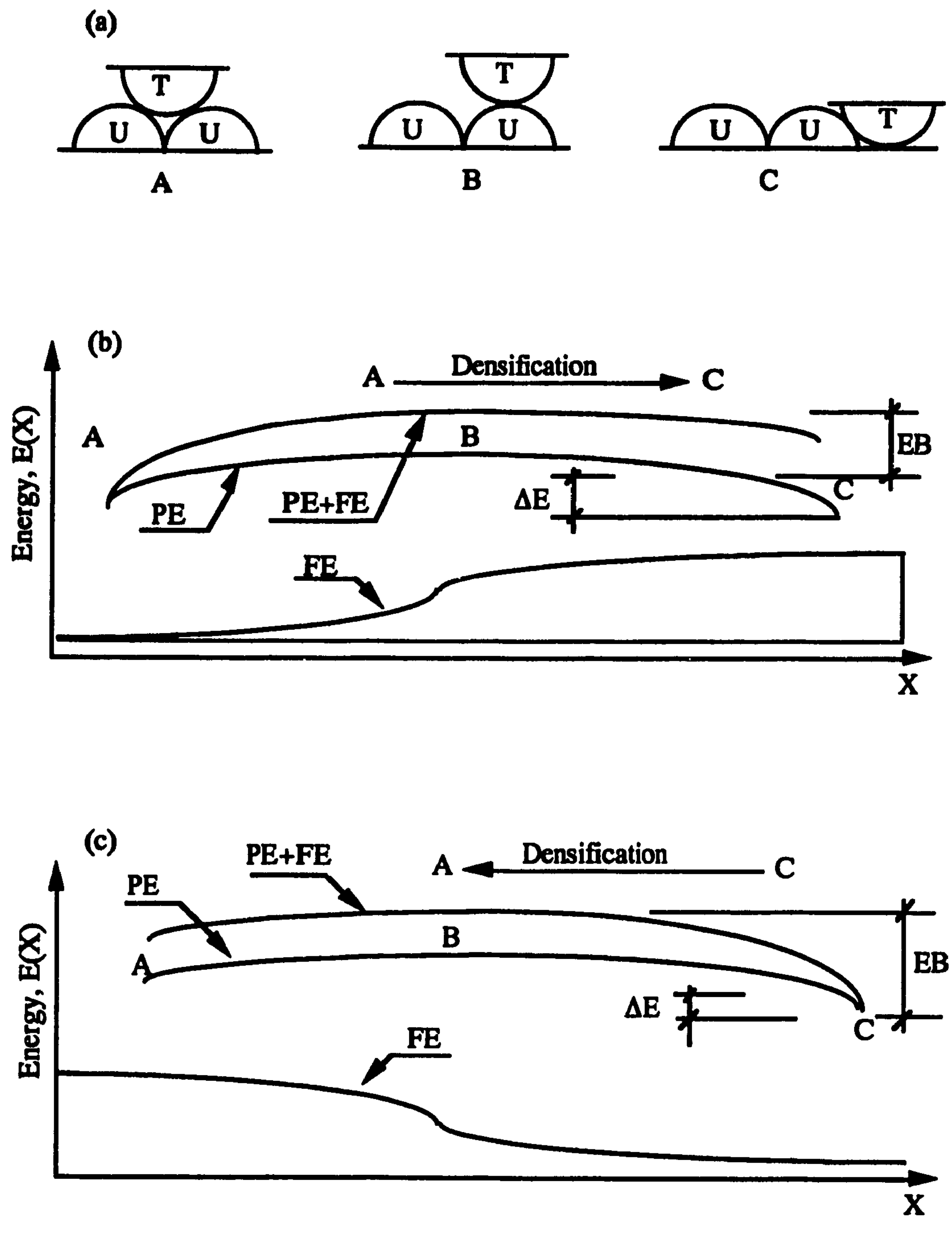


Figure 4.5: Energy Barriers

the determination of the mechanical properties of the sand and predicting the effect of the placing technique on the stress system within the sand deposit. The following section presents an attempt to utilise the results of the present experimental investigation to incorporate the parameters that control the compaction procedure in a semi empirical expressions.

Youd (1972) have presented a diagrammatic illustration of zones under vibratory rollers. This illustration was reproduced for a vibratory compacting rammer as shown in Figure 4.6. It can be noticed from this figure that a vibratory rammer generate stresses with different intensities and different profiles. The extension of these stresses below the vibrating rammer is large vertically and to a lesser distance horizontally. This indicates that for every compacted layer, an effect extends from an upper layer to a lower one due to compaction. For this reason, it was necessary to introduce that effect in the energy input to every individual layer and to account for the relative height of the compacted layer to the total height of the compacted deposit. This can be achieved by considering the rank of the layer under consideration with respect to the total number of layers. An energy factor (E_F) represents all parameters affecting the energy input to the sand layers was established. This factor is defined as follows:

$$E_F = \frac{E/t}{\sum_{n}^{n} \gamma \ d} \qquad .....(1)$$

Where E = The energy input per unit volume at the layer n'.

t = The time for the energy input.

n = Total number of layers in the testing tank.

n'= Rank of the layer under consideration.

γ = The average unit weight of layer n'.

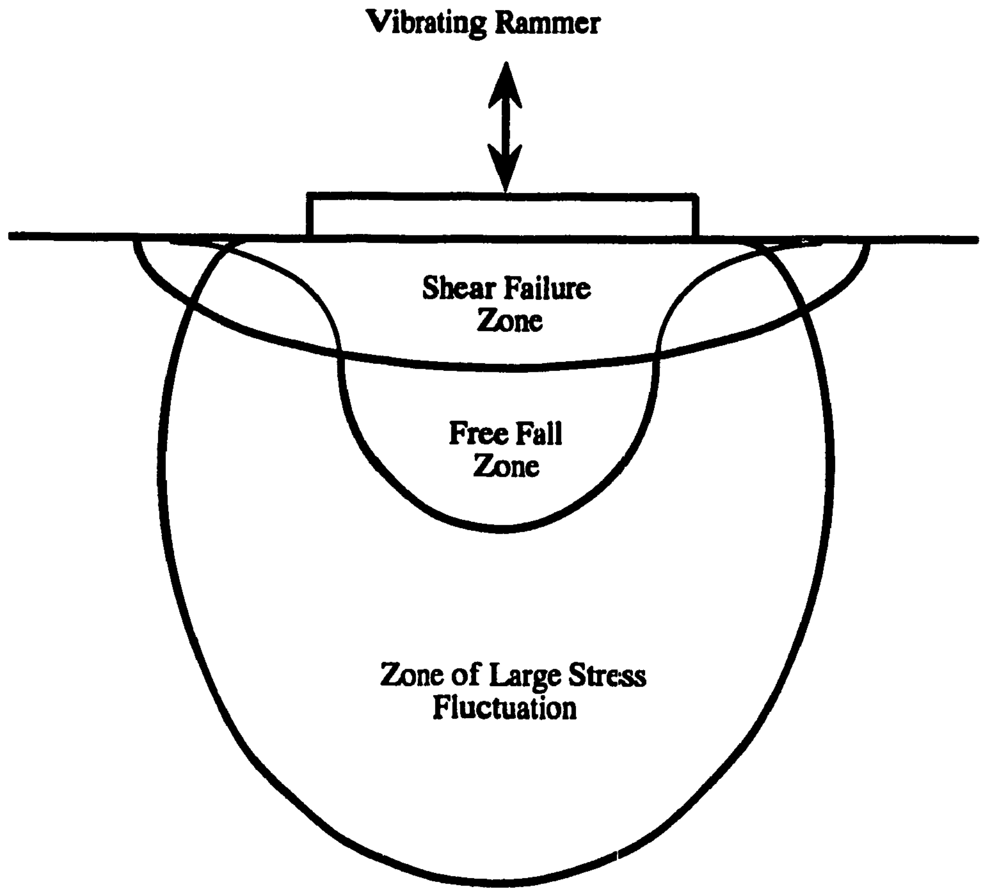


Figure 4.6: Diagramatic Illustration of Zones Under Vibratory Rammer

d = Thickness of layers.

E_F= The energy factor.

This factor (E_F) represents the ratio between the energy given to the soil by compaction at a certain layer per unit time and the calculated theoretical vertical stress at this point.

The energy transfer to lower layers due to the compaction of top ones depend on the thickness of layers and the stress level of the layers underneath. Also, the position of the compacted layers to the bottom ones has a significant effect on the amount of the energy transfer. A dimensionless relative height factor (H_R) was established to incorporate the relative position of the height of any layer to the top compacted one. This factor is defined as follows:

$$H_R = \frac{(n' - 0.5)d}{H} \quad .....(2)$$

where n' = The rank of the analysed layer.

d = Thickness of layers.

H = Total height of the sand in the testing tank.

H_R = The relative height.

This factor represents the ratio of the height of sand layer to the total height of the sand inside the testing tank.

4.5.1 Relationship between the Energy and the Height

According to section D698-70 and D1557-70 of the American Society of Testing and Materials (ASTM 04.08), the energy input due to compaction is calculated as follows:

Energy = Height of drop x Weight of hummer x Number of drops x Number of layers

For the vibratory densification using an air-held hammer, the energy input per layer was calculated as follows:

Energy = Weight of compactor x Height of fall x frequency x time of compaction

Based on the above equation, the energy input corresponding to compaction durations of 3 seconds and 6 seconds was 7.35 and 14.70 KN/m^3. Table 4.1 shows the values of the compaction time, applied energy for a full tank, average unit weight in each layer and the energy per unit volume in each layer. The frequency of the utilized compactor was 1650 revolution / minute according to 241 KPa air pressure and the weight of the compacting device is 3 Kilograms.

As it can be seen from figure 4.7 and 4.8, greater portion of the energy input transfers to lower layers due to the compaction of subsequent layers. This indicates that lower layers are subjected to higher energy input which reflects on the mechanical characteristics of the sand.

As it can be seen from figure 4.7 and 4.8, the energy increases with the decrease of the height in the testing tank. This increase is a function of the sand properties and the technique utilized for its preparation , however, it affects its behaviour in the laboratory.

Layer Number	Compaction Time (Sec)	Number of Layers in the Tank	Average Energy , E_{av} (KN.m/m^3)	Unit Weight (KN/m^3)	Energy per unit volume of sand (KN.m/m^3)
1	3	4	7.35	19.385	7.659
2	3	4	7.35	19.090	7.544
3	3	4	7.35	18.335	7.245
4	3	4	7.35	19.952	7.095
1	6	4	14.70	19.532	15.318
2	6	4	14.70	19.149	15088
3	6	4	14.70	18.757	14.489
4	6	4	14.70	18.443	14.190

* Hummer Weight = 3 Kgs.
* Hummer frequency = 1650 Rev/min
* Height of drop = 0.7 cm.

Table 4.1 : Summery of energy calculation in each layer by the unit weight proportion .

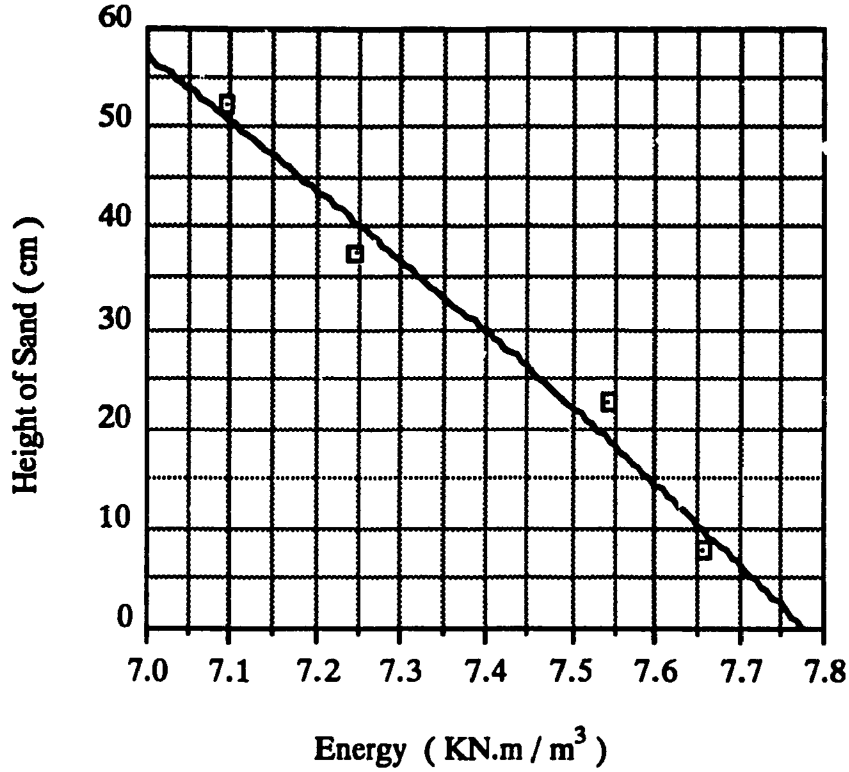


Figure 4.7 : Variation of the Energy Content in the Tested Sand for Compaction time 3 seconds.

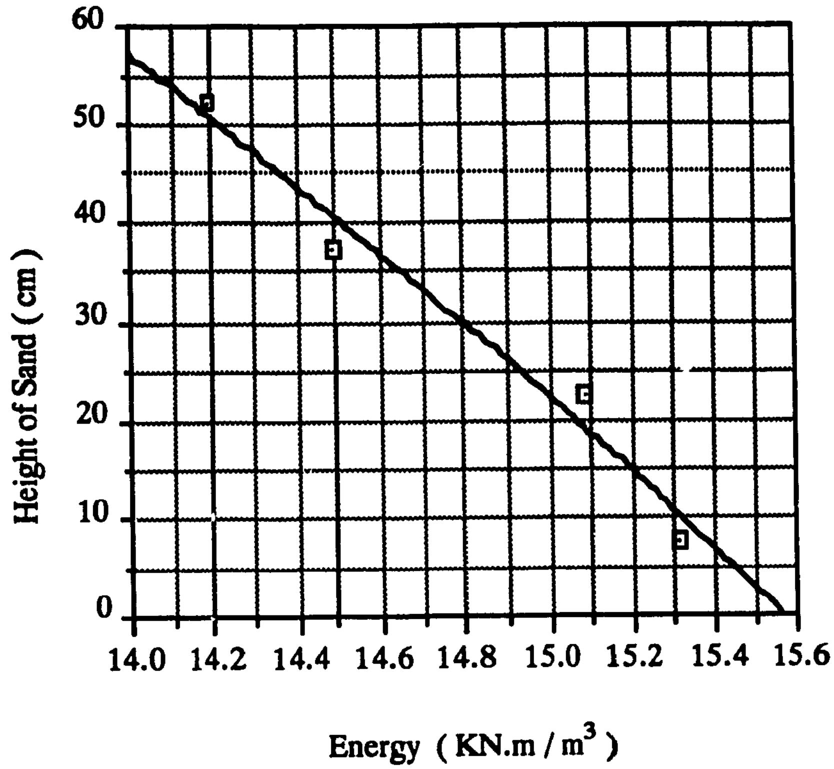


Figure 4.8 : Variation of the Energy Content in the Tested Sand for Compaction time 6 seconds.

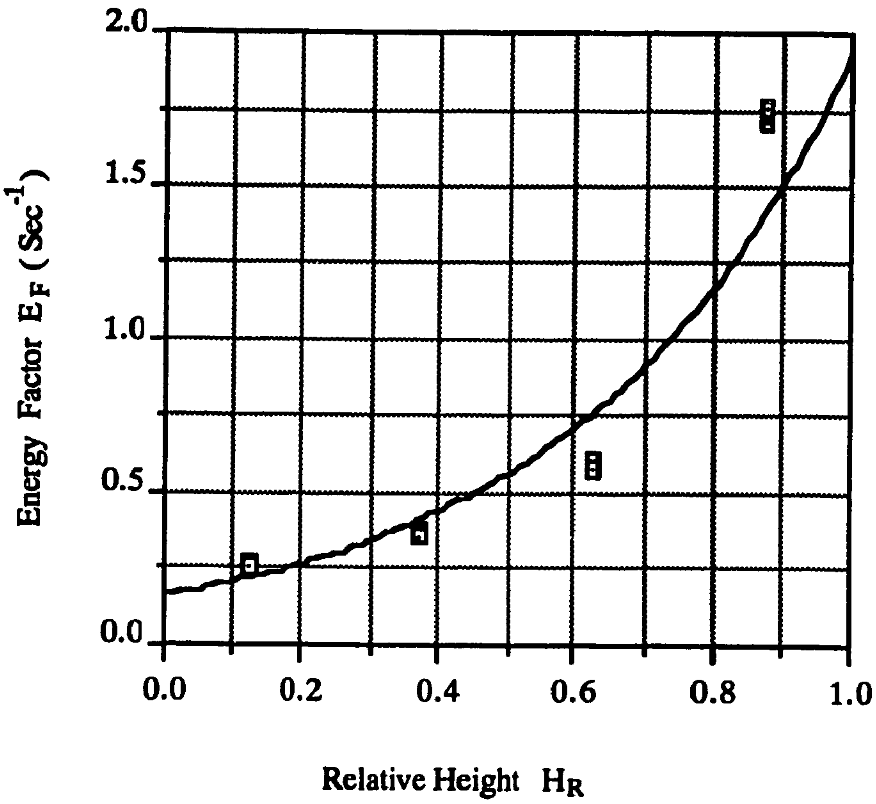


Figure 4.9: Variation of the Energy Factor with the Relative Height.

Figure 4.9 shows the relationship between the energy factor (E_F) and the relative height (H_R). An exponential relationship was found to fit properly with the plot. As it can be seen from the curve, the change in the energy factor at the bottom layer due to compaction of the top one is small (slope of the tangent to the curve). Meanwhile, the change in the energy at the top layer is maximum. This can be related to the original amount of energy in bottom layers when compacting the top one. A semi empirical equation representing the relationship between the energy factor (E_F) and the relative height (H_R) for the utilized technique is proposed as follow:

$$E_F = 0.16 \ (10)^{1.1 \, H_R} \qquad \text{.....(3)}$$

where E_F = The energy factor.

H_R = The relative height.

From equation (3), it can be noticed that it is possible to control the unit weight of the sand (γ) prepared in the laboratory by controlling the energy input. This energy input for the soil is affected by:

a. Total height of the sand (H).

b. Thickness of individual layers (d).

c. Number of layers (n).

By knowing the value of the total height (H), making an appropriate assumptions regarding the layer thickness (d) and the number of layers (n) and specifying the required unit weight in the sand deposit; the energy input required for each layer can be calculated.

This indicates that different compaction effort will be required for each layer to maintain the same energy for all the layers.

Another approach can be deducted from equation (3). A constant energy effort can be applied to layers of variable thickness of the sand in the testing tank. This technique has an advantage of fixing the energy input for all the layers.

4.5.2 Relationship between the Energy, Angle of shearing resistance and Stress Level

The sand mechanical properties are usually represented by the angle of shearing resistance (ϕ) and the stress level is represented by the overconsolidation ratio (OCR).

Having a sand deposit of constant angle of shearing resistance (ϕ) can be achieved if equation (3) is fulfilled. Controlling the stress level in the laboratory requires more understanding of the soil behaviour. A plot between the energy factor (E_F) and the product of OCR. tan ϕ is shown in figure 4.10. A logarithmic curve was found to give the best fit for the plotted points. A relationship proposed as follows:

$$E_F = 1.67 \ (OCR \cdot \tan \phi)^{-0.85} \qquad .....(4)$$

where E_F = The energy factor at a specific layer.

OCR = The overconsolidation ratio at the same layer.

ϕ = The angle of shearing resistance in the layer.

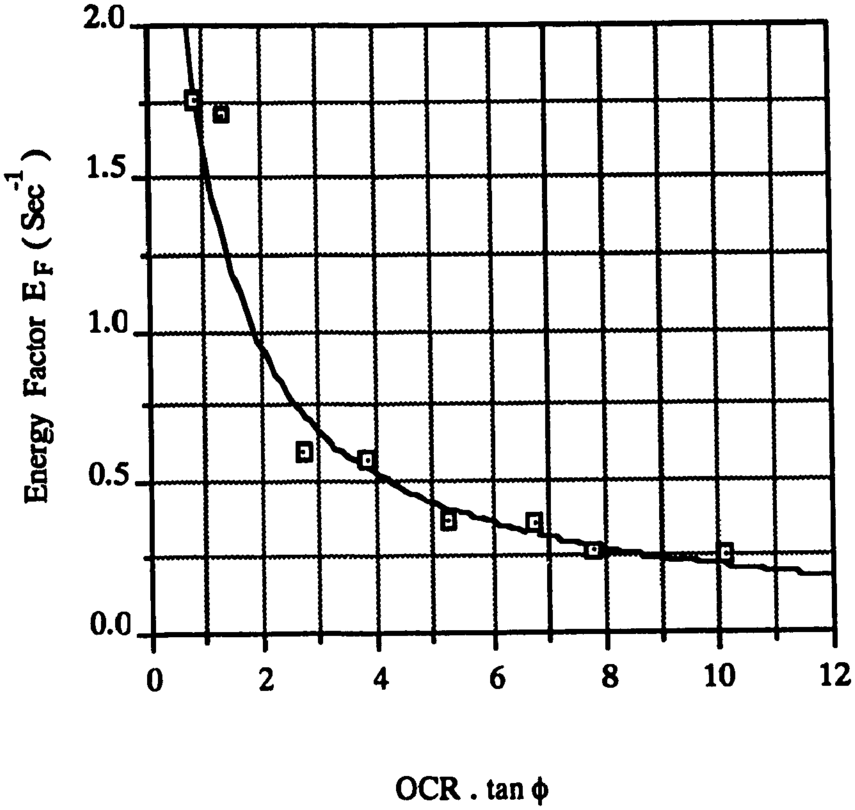


Figure 4.10 Variation of the Energy Factor with the product of OCR.tan ϕ

From equation (4); it can be seen that the stress level can be controlled in the laboratory. The importance of controlling the stress level is to duplicate the field conditions in the laboratory. Fixing the unit weight and the angle of shearing resistance, the overconsolidation ratio can be controlled by adopting the amount of the ... put to each sand layer. Combination between equation (3) and (4) can result in a precise mechanical properties and stress level control in the laboratory.

4.5.3 Relationship between the Energy and the Coefficient of Lateral Earth Pressure

The energy utilised to prepare sand of certain requirements in the laboratory has the largest effect on the sand behaviour. Equation (3) togethe with Equation (4) show that the sand mechanical properties and stress level can be controlled in the laboratory. For a previously compacted sand, additional compaction effort results in much smaller increase in peak lateral earth pressures than in uncompacted sands. This can be seen from the ratio of $K_{O(OC)} / K_{O(NC)}$ in figure 4.11. A plot between the energy factor (E_F) and the ratio of $K_{O(OC)}$ to $K_{O(NC)}$ is presented in figure 4.8. The mathematical relationship of the shown plot is presented on the following form:

$$E_F = 1.6 \quad (K_{O(OC)} / K_{O(NC)})^{-1.30} \qquad .....(5)$$

where E_F = The energy factor.

$K_{O(OC)}$ = Coefficient of lateral earth pressure for overconsolidated sand.

$K_{O(NC)}$ = Coefficient of lateral earth pressure for normally consolidated sand.

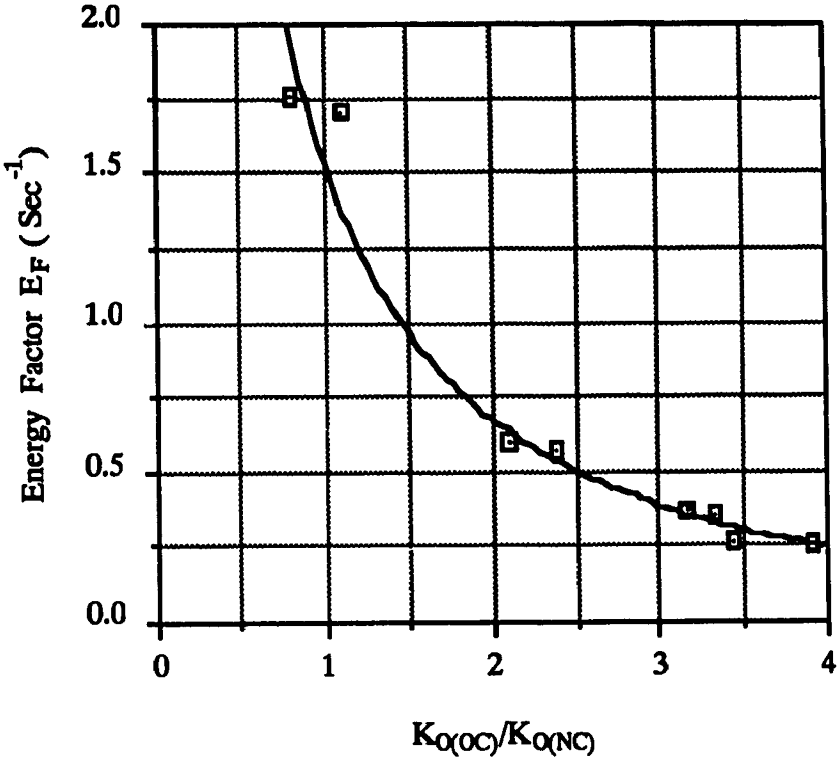


Figure 4.11: Variation of the Energy Factor with the ratio $K_{\alpha(OC)}/K_{\alpha(NC)}$

For an overconsolidated sand, the coefficient of lateral earth pressure at rest $K_{O(NC)}$ is usually altered due to the applied energy input. The coefficient lateral earth pressure for overconsolidated sand $K_{O(OC)}$ is an important one in the design of foundations and its value is required for bearing capacity calculations or to relate the bearing capacity of normally consolidated sand to an overconsolidated one. The above equation presents a simple correlation between the energy input and both $K_{O(NC)}$ and $K_{O(OC)}$. The value of $K_{O(NC)}$ can be determined theoretically from the equation given by Jaky (1948), and by knowing the value of E_F, $K_{O(OC)}$ can be calculated. It is of interest to note that the above equation will yield value of $K_{O(OC)}$ equals to $K_{O(NC)}$ if the energy input was not higher than that required to produce normally consolidated sand. In this case it can be expected that equation (4) will yield an OCR equivalent to 1.

CHAPTER 5

Conclusion and Recommendations

5.1 Conclusion

Experimental and theoretical studies were conducted in order to evaluate the effect of applying vibratory densification on the sand properties. Based on these studies, the following conclusion can be drawn:

1. Sand placing technique has a significant influence on the measured mechanical properties in the laboratory.

2. The application of vibratory densification results in lateral and vertical stresses higher than those of normally consolidated sand.

3. Energy input to the sand layer has a direct reflection on the measured in situ stresses within the layer. The higher the energy input, the greater the stresses.

4. Energy input in top layers affect all the layers underneath due to of the energy transfer through layers.

5. Wave reflection from the rigid boundaries (base and walls) of the rigid steel tank

contribute significantly to stress surplus in lower layers.

6. The energy profile (energy versus depth) indicates that energy conservation is greater at lower layers.

7. For a given depth of sand and energy input, number of layers used during sand placing has a considerable effect on the sand mechanical properties. The higher the number of layers, the greater the unit weight and angle of shearing resistance.

8. An energy factor representing all parameters influencing the energy magnitude was established. A simple relationship between this factor and the relative height of the sand was developed. This relationship is useful to calculate the energy that should be applied to a predetermined thickness of a sand layer in order to obtain the required unit weight.

9. A relationship between the energy factor and the product of (OCR.tan ϕ) was established. This equation can be used to calculate the required energy for the soil to create a predetermined stress level. Also, this equation can be used to calculate the value of the overconsolidation ratio in each layer.

10. A relationship between the energy factor and the ratio of the coefficient of lateral earth pressure for overconsolidated and normally consolidated sands, respectively, was proposed. This relationship is useful to determine the value of the coefficient of lateral earth pressure for overconsolidated sand in terms of the energy input and the coefficient of lateral earth pressure for normally consolidated sand.

5.2 Recommendation

The validity of the proposed expressions based on the present experimental investigation, which should be examined against results of laboratory and field tests. Laboratory and field testing should measure the value of the utilized energy and the stress level applied by the utilised technique, and report it in the future. It is believed, however, that if this data is available, it would constitute a significant contribution to our knowledge on the behaviour of sands with different placing techniques and consequently different stress history. Also, the existing discrepancies between the design theories in foundation engineering shall hopefully be vanished.

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