BE CIVIL ENGINEERING

PROJECT REPORT

DEVELOPMENT OF MODEL PILE TESTING FACILITY

Project submitted in partial fulfillment of the requirements for the degree of

BE Civil Engineering

SUBMITTED BY

Name	**Registration No:**
CQMS M. Asad Ullah	NUST20l235
Muhammad Hamza	NUST201236
M.Qasim Shahid	NUST201253
SGT. Osama Abid	NUST201229
Saad-bin-Waheed	NUST201284

MILITARY COLLEGE OF ENGINEERING

NATIONAL UNIVERSITY OF SCIENCES & TECHNOLOGY

RISALPUR CAMPUS, PAKISTAN

(2020)

This to certify that the

BE Civil Engineering Project entitled

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Has been accepted towards the partial fulfilment of the requirements for

BE Civil Engineering Degree

Dr. Kamran Akhtar, PhD

Syndicate Advisor

Dedication

To my parents and mentors who have a role in teaching me to never give up

-Asad

To my parents and my country

-Hamza

To my parents and my teachers who have taught me to learn from failure

-Qasim

To my family

-Osama

To my parents and my country

-Saad

ACKNOWLEDGMENT

We are thankful to our Advisor Brig Dr Kamran Akhtar for providing footsteps that lead to the completion of the project and letting us access his resources and experience. We are also thankful to Mr. Sher Zaman of Geo-tech lab for guiding us in the lab testings and equipment management . Gratitude is extended to lab engr. Zafay and Huzaifa who provided technical assistance in connection of gauges. Efforts of our project advisor in administrative support and of all other faculty mambers who extended support are worth mentioning here. Lastly efforts of Brig Dr Irafn, the Dean are appreciated for creating a research friendly environment in the college.

All Syndicate members

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ABSTRACT

Profound establishments play essential part in development of Megastructures and bridges etc. Temperate and secure plan of piles could be a major parcel of geotechnical plan. To successfully attempt this errand, a geotechnical design must be recognizable with pile soil behavior beneath changing soil parameters and different field conditions, pile types and their establishment strategies. Tragically pile behavior can for the most part be watched in Field where such development is being embraced. The introduction of a geotechnical designing understudy is in this way limited to field visits. Another cripple for geotechnical Engineers in Pakistan is the need of think about on existing soils where such structures were limited to existing streets arrange, undertaking a profound establishment plan in remotely found region render the analyst daze curiously expanding the calculate of security and hugely influencing the economy of plan. Besides, any inquire about into piles is hampered due to overwhelming taken a toll of experimentation within the field.

Chapter 1

INTRODUCTION

1.1 Background

Establishments are one of the foremost imperative portion of any gracious Building structure and however a complex one since its behavior depends on nature and sort of soil, it is association with. There are two sort of establishments broadly used— Shallow and Profound. Shallow establishments incorporate footings, flatboat establishments etc. and are for the most part utilized in little single to twofold story buildings. Profound establishments that incorporate piles are for the most part utilized in colossal structures. Each mega venture of respectful Building more often than not comprises of profound establishments. Piles design are ordinarily complex because it requires perplexing consider of soil strata, it is collaboration with. The behavior of different sort of soil with changing degrees, different mechanical properties and sort of mineral composition shows differing qualities in behavior beneath distinctive sort of stacking. As a rule plan prepare of piles comprise of calculation of extreme vertical stack capacity. Capacity is at that point isolated by a reasonable figure of security to account for instabilities insitu.

1.2 Problem Statement

Plan of Profound establishments requires understanding of distinctive soil strata and their interaction with sides and base of pile. Different hypothetical models exist for calculation of these parameters in any case these models are required to be calibrated/validated as per changing soils. This could be best caught on by simulating field conditions in Research facility and after that watch Pile behavior beneath required loading and comparing results thus gotten with hypothetical calculations. Subsequently, there should be a lab equipment arrangement where distinctive stacking designs can be tried on distinctive soil sorts and conditions

1.3 Scope and Objectives

The extend points at improvement of an office to encourage investigate in profound establishments and to assist understudy create way better understanding of the concepts related to them. The objective of the extend is advancement of show Pile Testing office with taking after capabilities:

a. Simulation of vertical compelling push on the soil in tank.
b. system to watch behavior of pile beneath stacking.
c. Capability of conferring energetic as well as inactive loads
d. Measurement of skin contact and Conclusion bearing stresses
e. Using soil with distinctive characteristics and densities

1.4 Relevance of Research and Research Questions

Demonstrate Pile Facility will offer assistance in superior understanding of Pile behavior. This will offer assistance understudies to approve their hypothetical calculations as per commonsense behavior of pile. This will moreover offer assistance understudy get it connection of different parameters such as thickness, successful push etc. on behavior of Pile. This demonstrate will offer assistance conduct inquire about on behavior of Pile on distinctive soils in Pakistan. This will offer assistance analysts to create diverse relationships for calculation of pile behavior beneath changing soil parameters. Investigate will rotate around taking after Questions:

a. What are the reasonable measurements for Show Pile Facility?
b. What stack and push course of action best suites the working of facility?
c. What course of action are to be made for statement of soil?
d. What course of action of sensors meet the satisfactory prerequisites of the facility?
e. How distinctive field conditions can be reenacted within the facility?
f. How shifting of distinctive edges of soil can be simulated?

Chapter2

Literature Review

2.1 Introduction

Pile groundwork has a vital role in the field of construction and civil engineering ,so to enhance the working and increasing the work life of pile groundwork engineers have worked a lot and there is visible improvement in pile groundwork. But still there is room for improvement of pile loading reactions to effects of pile loading and installation on pile mounding response. Due to the difficulties faced during the pile installation and loading issues, estimation of pile capacity (i.e., base and shaft resistances) while doing experiments and studies. With all these studies we face many ambiguities many old techniques are used. Two methods are being used for the designing of single loading of piles on which load is applied

Axially the approached includes I-Direct method which is in– situbasedmethodand2-property based method this method is known as indirect method. To use such methods for designing the field which should be idealize on the dominancy of soil properties ,whereas in the direct designing,whichincludesin–situbasedmethods,inputvariablesarethetestoutputwhichis given by in-situ method designing. This chapter considerably focuses on the responses of single piles in sand.

2.2 SAND BASE-RESISTANCE ESTIMATION

2.2.1 Soil property-based methods

qb defines the “unit base resistance” which interrelates the “in situ vertical effective stress *a*, at the base of the pile and the dimensionless bearing capacity fact of Nqt” termed as

In the same way same equations can also be termed as in terms of the “ultimate unit base resistance”:

$$q_b = \sigma_v' N_q$$

$$q_{b.ult} = \sigma_v' N_{q.ult}$$

“ultimate unit base resistance” is dependent on the criterion used for the ultimate loads. The most widely used criterion is the l1% relative settlement criterion insisting on the “ultimate unit base resistance” corresponds to that for which the settlement of the head of the pile Organizes on the pile diameter which is 11%.

Dozens of proposed failures mechanisms for the base of the pile and equations came from researches, for calculation of the bearing “capacity factor Nt”, which is directly proportional to the frictional angle with the soil. However, Nq_{ult} is not constant with *a*, and decreases with an increase in the vertical effective stress o Salgado (l995) which shows that the ‘ultimate unit base resistance Qb_{ult} Shows nonlinearity upon increase when the rates are decreasing, with an increase in *a* , hence with an increase in length of the pile.

Fleming and Weltman in 1992 states that the bearing capacity factor Nqult for driven piles is given as follows:

$$N_b q_{tt} = 0.13\ 6e^{0'\ 82\wedge}$$

It is the ‘peak friction angle of the sand in degrees’. Equation. 2.4 is generated by the explanation given by Berezantzev, Khristoforov, and Golubkov in 1961. The peak friction angle can be given for ‘tri-axial compression conditions’ using the iterative co-relation presented by Bolton in 1986

$$\$, = \$,+3$$

It is defined as the critical-state friction angle, DR is the relative density in percentage, $a+z$ is defined as the mean peak ‘effective confining stress’. Q and R_q are the variables which depends on properties of the sand, for clean silica sand, values of 10 and l can be used for Q and R_q.

The American Petroleum Institute (API, 1993) recommended values for the bearing capacity factor Nq that depend on soil density and particle size(seeTable2.l).A.P.I design method, which is commonly used for axially loaded piles, these piles are produced on an international scale of ‘axial pile load tests’ that is simultaneously refurnished (Pelletier, Murff, & Young, 1993). In this empirical design method Eq. 2.2 is used to compute the ultimate unit base resistance of driven piles in sandy soils (recommended values of Nq are given in Table 2.1). API method in 1993 also states limiting unit base resistance qbc values (see Table 2. l). Chow and Jardine (1996) proposed this method to be highly conservative when used to estimate the capacity of l 00-mm-diameter model piles.

Density	Soil Description	G (deg.)	Limiting Unit Shaft Resistance (kPa)	Nq	Limiting Unit Base Resistance (kPa)
Very Loose Loose Medium	Sand Sand-Silt Silt	15	47.8	8	1,900
Loose Medium Dense	Sand Sand-Silt Silt	20	67.0	12	2,900
Medium Dense	Sand Sand-Silt	25	81.3	20	4,800
Dense Very Dense	Sand Sand-Silt	30	95.7	40	9,600
Dense Very Dense	Gravel Sand	35	114.8	50	12,00

Table 1: Design parameters for cohesionless siliceous soil (API, 1993)

Nordlund in 1963 proposed a semi-empirical approach to design piles in cohesion- less soils are created on several pile loading tests and provided tables for obtaining design parameters. According to the Nordlund method, the ultimate base resistance Qb_{ult} is given as follows:

$$q_{b.ult} = \alpha_t N_q \sigma_v'$$

where Dt is a dimensionless that depends on an embedded pile length.

diameter ratio, and Nq iS a bearing capacity factor that is obtained from design charts (see Figure A. l and A.2 in the Appendix).

An investigational platform including 36 calibration chamber model pile tests and two full-scale field pile loading tests (that has the loading capability of piles of pipes) had been conducted by Junhwan Lee, Salgado, and Paik in 2003, that had both open-ended and close-ended which is associated to the cone penetration resistance q_c. They proposed the values for pile unit base resistance and unit shaft resistance in terms of q_c.

An equation in terms of relative density 'DR' and lateral effective stress 'α_h' for the calculation of the limit unit base resistance 'q_{bL}', was proposed by Salgado and Prezzi (2007). The equation is as follows:

$$\frac{q_{bL}}{P_A} = \mathbf{1.64\ exp[0.1041a} + (0.0264 - 0.0002\ Q\ )\ \boldsymbol{D_R}\ ]\ \left(\boldsymbol{p}^{\circ}\right)_{0}\ \mathbf{8_4\ 1 - 0.0047Dp}$$

where critical-state friction angle is expressed in degrees as 'I', relative density given in percentage units as 'DR', '*ay'* is the in situ horizontal effective stress and 'pA' is the reference stress (it is equal to l00kPa). 'qbc' (Salgado,2008) Eq (Foye,Abou-Jaoude, Prezzi, and Salgado, 2009) is used to compute the ultimate unit base resistance 9buIt of displacement and non-displacement piles in sand:

$$q_{b.ult} = (1.020 - 0.0051Dp)\ Q_{bl}$$

$$q_{b.ult} —— [0.23\ exp(0.0066Dp)]q_{bl}$$

2.2.2 In situ Test-Based Methods

In situ Test-Based Methods directly correlate with the results of In situ tests [cone resistance qc from cone penetration tests (CPT) or blow count number N from standard penetration tests (SPT)] with pile resistances. Because of the uncertainty and difficulty in characterizing soil properties In situ test-based methods were developed. Because the cone penetration process is similar to that of a pile plunging into the ground, use of CPT data in pile design is considered ideal, although the cone is much smaller in diameter than a pile. For this reason, the cone penetration resistance qc is almost the same as the limit unit base resistance q_{br}.

General CPT-based equation for estimating "ultimate unit base resistance $q_{b,ult}$" is:

$$q_{b.ult} = c_b q_{cb}$$

Whereas ‘cb’ is a constant which depends on the type of soil and type of pile, and ‘qcb’ is the representative cone resistance at pile base. The SPT blow count N is affected by the common factors as of the cone resistance ‘qc’. The general SPT-based equation for estimating ultimate unit base resistance ‘qb’ is:

$$\frac{q_{b.ult}}{p_A} = n_b N_b$$

where ‘N_b’ is a constant similar to ‘c_b’ which is dependent on the type of the soil and the type of the soil, ‘N_b’ is the representative blow count value at the pile base, Pa is a ‘reference stress (which is equal to l00kPa)’. Design values for the constants ‘c_b’ and ‘Nb’ proposed by several researchers for piles in sand is shown in Table 2.2.

Pile type	Value		Source
Driven Piles	C_b' 0.35-0.50		Chow and Jardine (1996)
	C_b	0.4	Randolph (2003)
	C_b '0.32-0.70 for D=30% C_b '0.27-0.57 for D =50% C_b '0.24-0.50 for D =70'o C_b =0.20-0.43 for D =9O° o		JH Lee and Salgado (1999) Basu, Salgado, Prezzi, Lee, and Paik (2005)
	nb	4	Meyerhof (1983)
	n_b =4.8 for clean sand n_b =3.8 for silty sand n_b 3.3 for silty sand with clay n_b =2.4 for clayey sand with silt n_b =2.9 for clayey sand		Aoki and Velloso (1975)
Drilled Piles	C_b	0.2	Franke (1989)
	C_b	0.13 *- 0.02	Ghionna, Jamiolkowski, Pedroni,and Salgado (1994)
	Cb - 0.23exp(-0.0066DR)		Salgado (2006)
	nb -0.82 for clean sand nb =0.72 for sand with silt or clay		Lopes and Laprovitera (1988)
	nb 0.6		Reese and O'Neill (1989)

Table 2.2: Design values of the constants c and b n for piles in sand

Table 2.2: Design values of the constants c and b n for piles in sand

A number of Multiple design methods were recently provided that proposed relationships between 'qc' and the ultimate unit base resistance of closed-ended pipe piles driven in sand. The methods are known as the Fugro ((Kolk, Baaijens, & Senders, (2005), ICP (Imperial College; Jardine, Chow, Overy, and Standing (2005)), NGI (Norwegian Geotechnical Institute;Clausen, Aas, and Karlsrud (2005)), and UWA (University of Western Australia; Lehane, Schneider, and Xu (2005)).

Method	Design Equation	Source
Furgo	$\frac{q_{b,ult}}{q_{cb.avg}} = 8.5\left(\frac{p_A}{q_{cb.avg}}\right)^{0.5}$	Kolk et a1. (2005)
ICP	$q_{b.ult} = \left[1 - 0.5\left(\frac{B_o}{B_{CPT}}\right)\right] q_{cb.avg}$	Jardine et al. (2005)

Table 3: Multiple Design Methods

2.3 Shaft Resistance Estimation in Sand

2.3.1 Property-Based Methods of Sand

When the shaft resistance of a pile is in an in-service condition, it is completely organized along with the shaft of the pile. Figure 2.2 illustrates the concept of pile shaft resistance. The appropriate design shaft resistance is the limit shaft resistance ‘qr’ since for complete mobilization of the resistance of the shaft, displacements of the small pile head (the order of 1% of the diameter of pile) are essential. This resistance results in the product of the normal stress applied on the shaft of pile by the interface friction-coefficient. The limit unit shaft resistance can be expressed as follows:

q, — *Kc[* tnn6

Whereas ‘K’ defines the coefficient of lateral earth pressure, the vertical effective stress is represented by ‘*a’*, and ‘p’ is the interface friction angle between the pile and the soil surrounding it.

According to API (1993), the coefficient of lateral earth pressure K maybe presumed to be 1.0 for full displacement closed-ended piles of pipe compelled in cohesion-less soils. For open-ended steel pipe piles, a value of 0.8 is suggested for K. API also recommended G values and limiting unit shaft resistance values (see Table 2.1). Although Eq. 2.12 implies that the resistance of the shaft of a pile is proportional to vertical effective stress, the shaft resistance of long piles does not increase indefinitely with the vertical effective stress.

Hossain and Briaud suggested the specific amendments to the original A.P.I method in 1993. They concluded that “as a result of the erroneous assumption of a constant value of K, the API method is inclined to under-predict the capability of shorter piles and over-predict the capability of longer piles”. To reduce the discrepancies in the predictions of pile capacities, they suggested the use of a new parameter ‘Kay’, an average horizontal earth pressure coefficient. This parameter is calculated as follows:

$$Kay = 60 / (L/ B * 5)$$

whereas ‘L’ is the ‘embedded pile length’ and ‘B’ is the ‘pile diameter’.

K values for use in Eq. 2.12 depend on pile type and pile installation method. For driven piles, Flemingetel (1992) proposed that K values are approximately 2% of $N_{q, ult}$

$$K = 0.02 N_{q, ult}$$

2.3.2 In situ Test-Based Methods

The following CPT - based and SPT-based general equations are used to "estimate the shaft resistance qs Li" of each soil layer i:

$$q = c_b * Qc$$

$$q - pAN_b n_b$$

where 'cci' and 'n' are constants which depend on the type of soil and the type of pile. q and N are the representative cone resistance and blow count number for layer I, and pA is the reference stress (that is equal to 100kPa).

Pile type	Value	Source	
	c, -0 008 for open-ended steel		
	pipe piles		
	c, =0.012 for precast concrete and	Schmertmann	
	closed-ended steel pipe piles	(1978)	
	c, -0.018 for Franki and timber		
	piles		
	c, =0.004-0.006 for D 50%		
	c, = 0.004-0.007 for 50%°D 70%	Lee et	al.
	c, -0.004-0.009 for 70%°D 90%	(2003)	
	For closed-ended pipe piles		
Drien Piles	c, =0.0040 for clean sand c, -0.0057 for silty sand		
	c, 0.0069 for silty sandwith		
	clay		
	c, -0.0080 for clayey sand with	Aoki and Velloso	
	silt	(1975)	
	c, .0.0086 for clayey sand	Aoki et a1.	
	n, =0.033 for sand	(1978)*	
	n, =0.038 for silty sand		
	n, = 0.040 for silty sand with clay		
	n, =0.033 for clayey sand with silt		
	n, =0.043 for clayey sand		

Non Displacement Piles	c, .0.0027 for clean sand c, -0.0037 for silty sand c, 0.0046 for silty sand with	Lopes Laprovitera (1988)	and

2.4 Influence of Pile Installation Method

Being an important factor in piles' load reaction, they are classified according to the installation method. Until the last few years (with progress in pile installation technology) pile foundation were used since ancient times (Salgado, 2005). Being the primary concern of this study, in this section we will review displacement (driven) piles and non- displacement piles noting the influence of fitting method on the response of piles in sand. Because they are installed by pushing and preloading the soil around the pile, they are called displacement piles

For non-displacement piles, piles which are installed by pre-removal of soil from the ground are called non-displacement piles. This is due to the fact that when imposed with non-displacement piles, on soil condition surrounding the pile, non-displacement piles impose little change. Usually they have a smaller load capacity than displacement piles. Through a series of FPL tests in impenetrable sands, the BCP Committee (BCP (1971)) showed that the load-settlement curves of driven and jacked piles were harder in comparison with the one put through non-displacement piles.

Pre stressed concrete, steel and timbers typically made piles.. Depending on the soil and pile conditions, many pile driving systems are used in daily life practice. When the sand is being drove by a pile by the blows of an impact hammer, usually soil displaces to make room for the pile, and, therefore, after pile driving, it changes the stress condition and density of the soil surrounding pile notably. So, the load capacity of pile increases correspondingly. Thatswhy, driven piles are beneficial while comparing with non- displacement piles.

Robinsky and Morrison (1964)) did model pile tests in sand for examining the extent of soil compaction and displacement around driven piles. Adopting radiography techniques, these authors noted that, in very loose sand (DR=17%), the extent of soil movement was three to four pile diameters from the pile shaft and 2.5 to 3.5 pile diameters below the pile. In the caseof DR=35%, extension of soil movement was from 3.0 to 4.5 pile diameters below the base and 4.5 to 5.5 pile diameters from the shaft.

Meyerhof (1983) did the pile load test and summed up a number of empirical data and claimed that the ultimate unit base resistance of non-displacement piles is roughly one-third of that of driven piles. Meyerhof also proposed that about one-half of the unit shaft resistance of driven piles may be used for preliminary estimates of non-displacement pile shaft capacity.

On two jacked and 14 driven pipe piles in sand Paik and Salgado (2003) performed numerous model pile load test. He explored the consequence of the pile fitting method.Various testing conditions were set during test, using different combinations of hammer weights and drop heights but driving energy was maintained constant; the result of the test was that when the hammer weight increases the bearing capability of the pile increases. The “rate of increase of the shaft resistance” was higher than that of the base resistance because of friction fatigue and when the driving energy increased by increasing the hammer weight for the same drop height. Considering same conditions, the jacked piles shaft resistance was observed to be greater than that of the driven piles.

2.5 Past Chamber Calibration test performed

Various researchers in the past have conducted experiments for establishing correlations between different parameters of Soil. Few are given below:

Researcher	Pile Dia Chamber to Pile Dia		Chamber size (mm)		Particle size (mm)		Purpose
			Dia	Height	D50	D10	
A. Parkin, Holden, Aamot, Last, and Lunne (1980)	25.2 35.7	20-48	760 1,220	1,220 1,500	0.45	0.30	CPT
Chapman and Donald (1981)	35.7	34	1,220	1,820	0.31	0.18	CPT
Smiths (1982)	36	53	1,900	1,150	0.17	0.10	CPT
Hunstman et a1 (1986)	36	21	760	800	0.37	0.25	CPT

Been eta1 (1987) Been, Crooks, Becker, and Jefferies (1986)	36	39	1,400	1,000	0.35	0.18	CPT
Sweeny (1987)	23.2 35.7	65 42	1,500	1,700	0.45	0.35	CPT
Chong (1988) Houlsby & Hitchman (1988) Iwaski et al (1988)	36 36 36	34 25 22	1,200 900 790	1,200 1,000 925	0.39 0.85 0.16	0.26 0.70 0.13	CPT
O Niell & Raines (1991)	102	7	760	2,540	N/A	0.21	Highly Pressured sand
A.K. Parkin (1991)	100	12 8	1,200	1,800	0.17	0.10	Calcareous sand
Iskander (1995)	89	10	884	1,067	0.17	0.12	Steel pile
Ghandi & Salvam (1997)	18.2	40	730	630	0.43	0.22	Group piles in ateral loadings

Alawneh, Malkawi, and Al-Deeky (1999)	41 61	27 18	1,00	1,300	0.27	0.13	Tension test in pile
Paik&Salgado(2004)	60.5	13	775	1270	0.59	0.43	Method ol nstallation o piles

Table 5: Caliberation chamber used in past

Different Design aspects have are also suggested by researchers such as Chamber to Probe Ratio or Probe to Particle size Ratio.

2.5.1 Sample preparation

Because of direct proportion between relative density and preparation of sand samples with the behavior of piles, preparation of samples is extremely important. Greater dry density, no particle crushing, minimum separation of particle sizes, accuracy of density measurements and better repeatability are the merits of Pluviation (raining) method over ASTM method (ASTM D 4253) as highlighted by Presti, Pedroni, and Crippa (1992). This method has edge over vibrating methodbeing economical. Pluviation method used for uniform sand sample preparations was acknowledged by Brandon and Clough (1991) as the technique allows preparation of reproducible soil samples having same density and gradation, Moreover, they also noted that the method is widely used because of its simplicity and resemblance to natural process of sand deposition. It is more efficient and reliable.

2.5.2 Size Effects

A. Parkin et a1. (1980) established on relative density of sand, the effects of cone penetration and chamber size. The effects were determined using four diameter ratios, produced by two penetrometers and two calibration samples. Study concluded that chamber —to-penetrometer diameter ratio of 50 and 20 were adequate for loose and dense sand respectively, however the boundary effects must be taken into consideration for conducting penetration tests on dense sand samples.

In order to reduce the effect of chamber size Been et a1. (1986) remarked that chamber-to-cone ratio must be greater than 50 for sands having DR=90%, whereas chamber size effects were not substantial for loose sand with DR^30%. The significance of lateral boundary conditions was also less for diameter ratios larger than 50 based on interpretation of CPT in sand.

To limit the theoretical plastic zone within the chamber, the chamber diameter should be minimum. 7.5 times the model pile diameter and that the model pile penetration should be restricted to about four times its own diameter above the base of the chamber (Vipulanandan, Wong, Ochoa, and OWeill (1989).

A study was conducted through numerical and experimental studies to eliminate the chamber size effects , it proposed that the lower limit of chamber-to-probe diameter ratio should be restricted to 50 in dense sands (Schnaid & Houlsby, 1991) and the same ratio was suggested to be greater than 100 by Salgado, Mitchell, and Jamiolkowski (1998)to reduce the chamber size effects based on penetration analysis coupled with experimental results.

2.5.3 Internal Scale Effects

Peterson (1988) and Vipulanandan et al. (1989) to reduce the internal scale effects have suggested pile/probe diameter to soil particle diameter. A suggested ratio of pile diameter to particle diameter of 80 and larger was suggested by Peterson (1988) based on lab examination to establish

To reduce the internal scale effects a ratio of pile diameter to particle diameter of 80 and larger was suggested by Peterson (1988). This suggestion was based on lab examination to check effect of specimen density, grain size, penetrometer diameter penetration rate upon pouring water on fine sand. it was brought up by Peterson (1988) that penetrometer will detect individual particles for probe-to-particle diameter ratio of 40 and less as opposed to Vipulanandan et al. (1989) which recommended the proportion to be at any rate 50 for Soil Dia.

2.5.4 Sand Relative Density

Turner and Kulhawy (1987) built up that sand drop height and discharge rate is related directly with unit weight of sand deposited. Consequently, by changing Pulviator sifter size and sand drop stature the overall density of sand deposited saved can be differed. Besides, the thickness variety was not expected to outperform 1% whenever arranged by Pulviation Method as demonstrated by A. Parkin and Lunne (1982). Since the sand properties and heap limits are significantly reliant on sand density in this manner its confirmation is of most extreme significance. The examination built up an ideal mix of sieve size and drop hieght of sand Pulviation was resolved.

Chapter 3

Methodology

Fabrication of this facility was done previously by Sir Salman Muhibullah and group using the Solidworks software. Various research papers were studied before the fabrication to ensure the relativity of the scale. The sensors were ordered and the assembly for the installation of the sensors was prepared by the local fabricator of Nowshera.

There were certain modifications that were to be done in the pluviator that were carried out by the direction of project supervisor Dr Kamran Akhtar from Lahore.

Calibration of sensors and pluviator was done manually by the help of lab attendants.

Pluviator was calibrated using a circular cylinder having smaller dia than the tank to avoid extra labor in the structures lab using automated pully system that kept the height of the pluviator constant while it deposited the sand .

Sensors were calibrated using oedometer in the geotechnical lab.

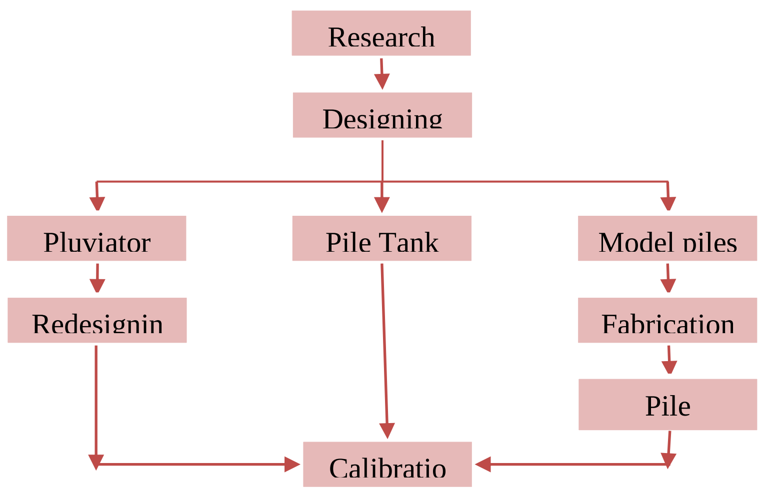


Figure 1: Methodology

3.1 Designing

Steel strength and its deflection under different loadings was kept in mind to determine thickness and size of the parts of the machine. The measurements of the machine were done as such to simulate the field conditions up to much extent. As the machine is to be reused so that factor was also kept in mind.

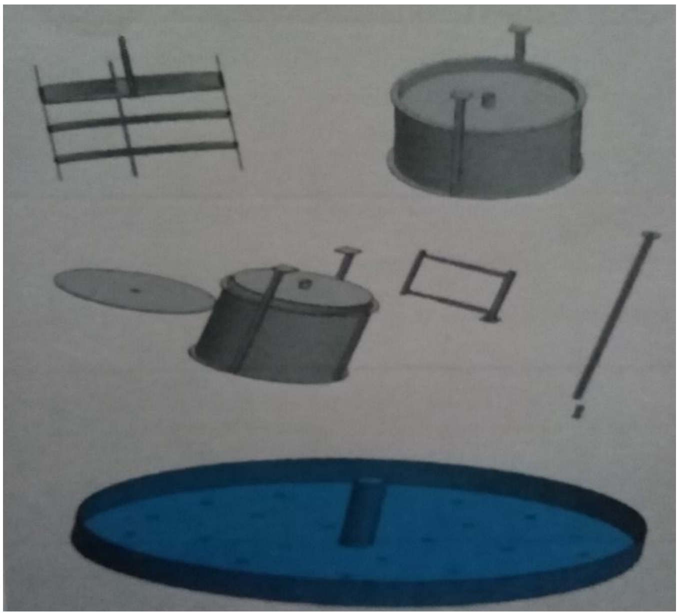


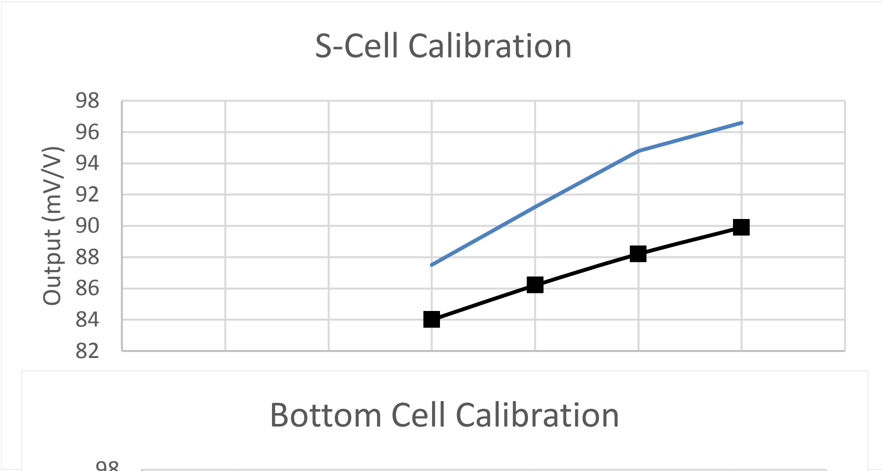
Figure 2 : 3D Modelling in Solid Works

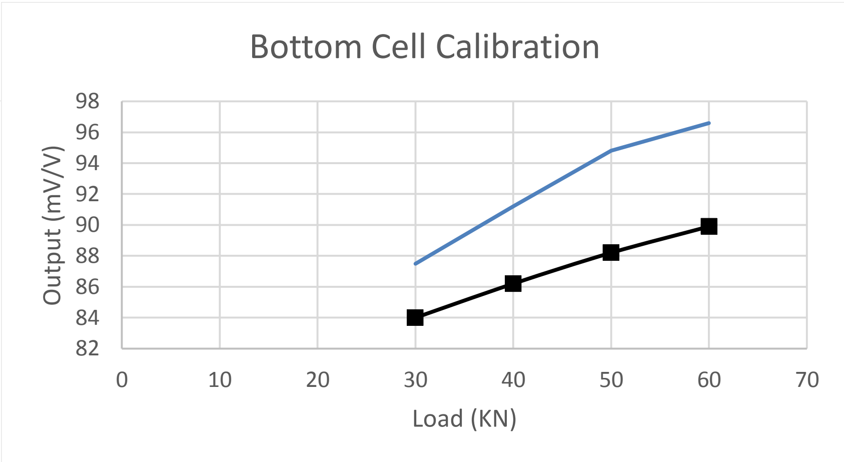
3.2 Fabrication and machining

The modifications that were to be made in the machine were made by the “Universal Engineers” in Lahore. Pluviator was redesigned by us as told by co-advisor Dr. Mazhar. Four new piles of two different and increased thickness, a pair of each, than before were also fabricated. Thickness of the piles was increased keeping in view the deflection of the previous piles with less thickness. The load cell assembly was prepared from Nowshera by a local machinist.

3.3 Calibration of load cell

The assembly according to the size of the pile was prepared beforehand and after fixing the sensor(LCM-307 Omega) in it, it was calibrated by using oedometer equipment and the general equations were derived for the relation of signal(mV/V) varying with the load(KN). The results are shown in the following graph.

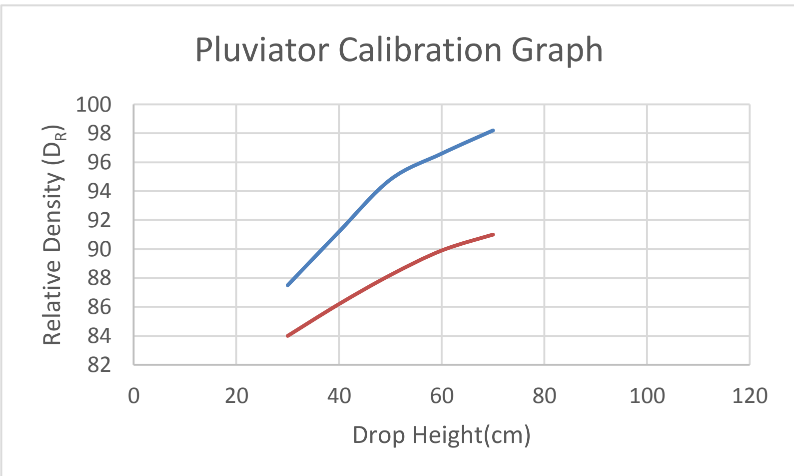




Load(KN) = Output(mV/V)/0.6

3.4 Calibration of pluviator

Multiple experiments were carried out for the relation of height and density using a smaller circular container for the calibration of pluviator. The results are shown in the following graph.



3.5 Functionality

A series of experiments was carried out in order to prepare the sample.

Sample sand was tested for its density, maximum and minimum void ratios, grain size distribution was also done and direct shear test was also conducted. Keeping in view the nature of tests to be conducted o samples were prepared, medium dense sand and loose sand. Static and dynamic both tests were conducted on these samples due to which theoretical capacity was also calculated.

Chapter 4

Development of Experimental Setup

4.1 Introduction

A lot of research work has been done on Pile Soil Interaction and resulting capacity of Piles from it. However, the soils in Pakistan have not been comprehensively studied. Empirical design formula along with experience with different soils forms foundations of design. The approach is experience based and results from already conducted projects. Therefore, as the experience increases, the design complicates. Another obstruction to geotechnical Engineering students is no exposure to Piles. Piles are usually driven in Transportation Projects and students' undersatnding depends upon visit to Project sites.Learning about Piles in university and its behavior is restricted to theoretical knowledge. Since most of the work has to be conducted on field, such as demonstration of Pile soil behavior and resultant capacities, unfortunately our Laboratories lack enough equipment to simulate similar conditions. The variation in Soil type and changing field conditions leaves the the design process to depend mainly upon the results of static load test. Estimation of Pile length for static pile load test is a problem whose solution is yet to be determined for diverse soils of Pakistan. So in order to achieve this objective a simulation for pile-soil interface needs to be developed in a model facility which shall cover various conditions involving various soils needs. Efforts have been made on this aspect in this project.

4.2 Soil Container

A cylindrical Tank has been designed with the help from local manufacturers, with dimensions as follows:

Thickness: 8mm **Diameter: 1.2m** **Height: 1m**

A l0mm thick plate is fixed at the bottom, is welded and placed on wheels. The tank is collar welded on top to bolt lid and other attachments. Four channel sections hold the cylindrical surface of tank externally, two of which are continued upward to support reaction beam.

Figure 3: Pile Tank

Reaction beam is an H-Beam with 5 U” wide flange placed on top of clamps and bolted. Attached on top of H-Beam is manual jack for application of static load. Some of the main components of Sand tank are described as follow:

4.2.1 Water Inlet/ Outlet

Water inlets/Outlets are provided on the cylindrical surface to establish varying head conditions as well as water saturation .These openings can serve dual purposes for both steady water and running water conditions.

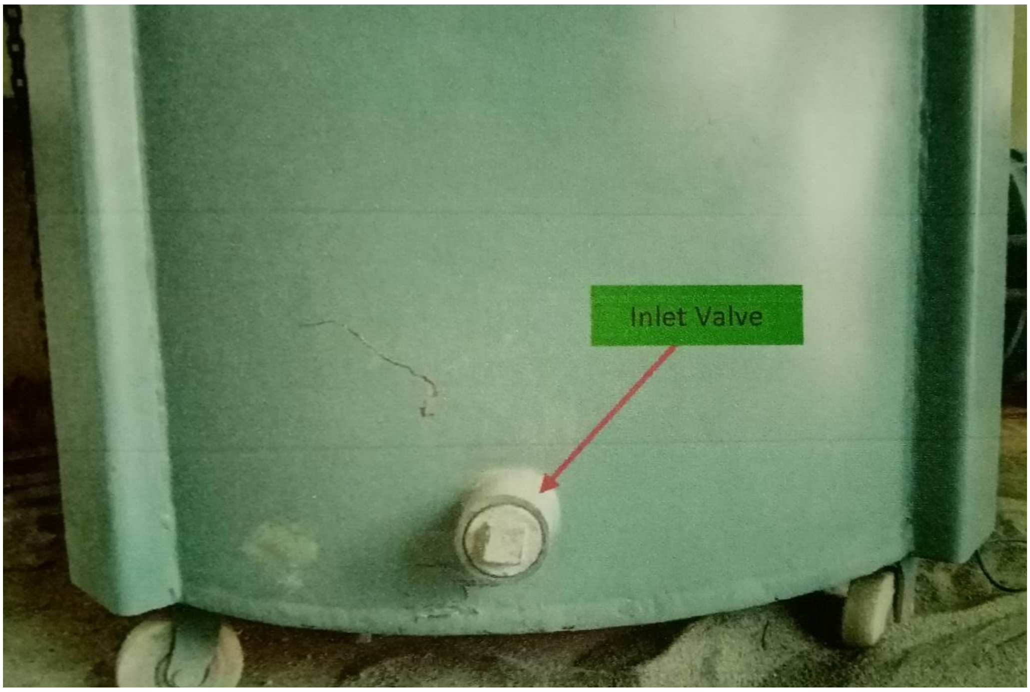


Figure 4: Water Inlet

4.2.2 Sand Exit valve

A 12” valve, also known as Butterfly valve has been provided near the bottom of cylindrical surface. The purpose of valve is to extract Sand for swift reuse. The valve is especially useful in case of saturated sands where extraction using shovels from top is much difficult and time consuming. A simple flowing water can extract the sand by opening the valve. 50 percent of reuse time for tank is saved by this.


Figure 5: Butterfly Valve

4.2.3 Bottom plate

A bottom plate having a thickness of 10mm is welded at the bottom and supported on four wheels. In order to prevent bowing of the plate, the wheels are holed on a specially made platform

4.2.4 Tank Collar

A rounded collar is attached on top of Pile Container to hold up Top plate and other equipment such as guide apparatus. A sliced tank collar and Top plate allow columns supporting reaction beams.

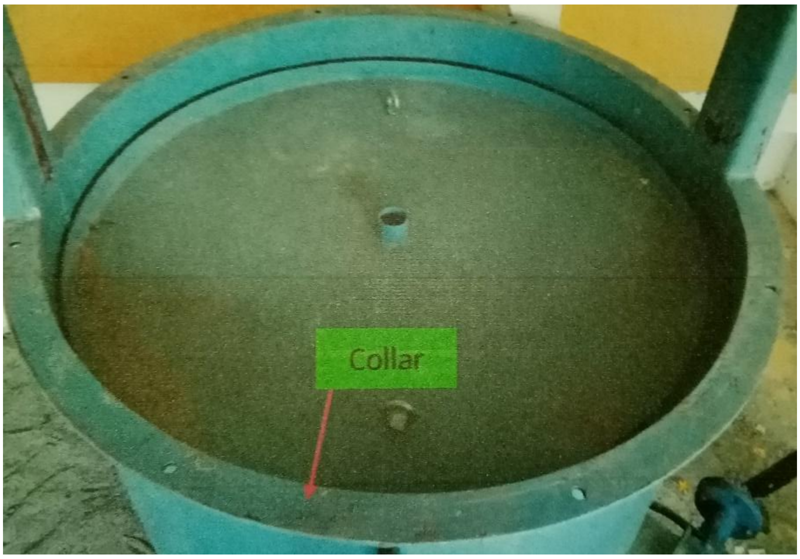


Figure 6: Pile Tank Collar

4.2.5 Stiffener and Columns

Four stiffeners are supplied to prevent bending in the cylindrical surface. Two of them are prolonged upwards for lifting reaction beam. On such columns, bolts are used for joining of beam.

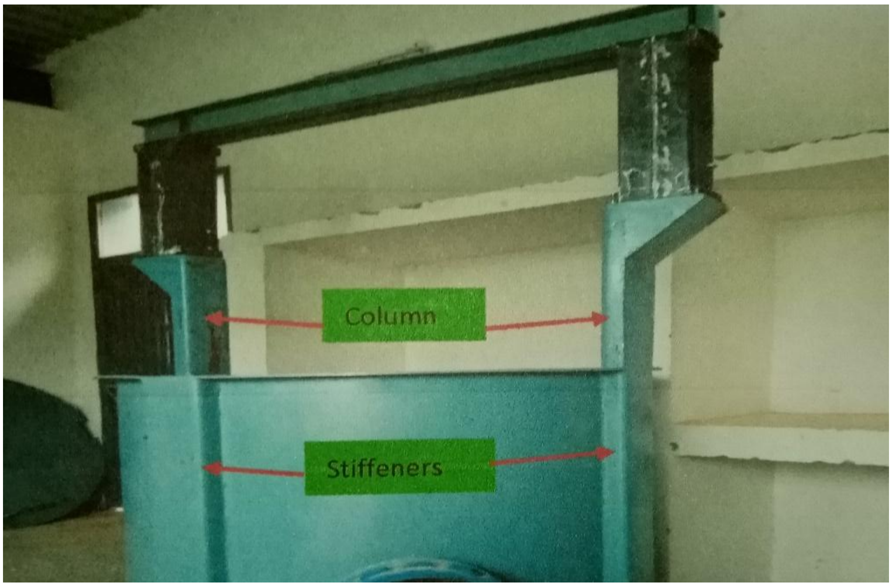


Figure 7: Reaction Beam

4.2.6 Stress distribution plate

A 10 mm thick and 1135 mm diameter A circular plate is welded with two cylinders around the collar of plate with following dimensions: Diameter: 1135mm Thickness: 10mm

Length of each cylinder: 10cm **Diameters: 50mm&1135mm**

Plate is provided with two threaded holes on center where I-hooks can be used for further uplift with the aid of pulley. Objective of stress disposing plate is to contain pneumatic pressure mechanism which simulate vertical effective stress conditions.

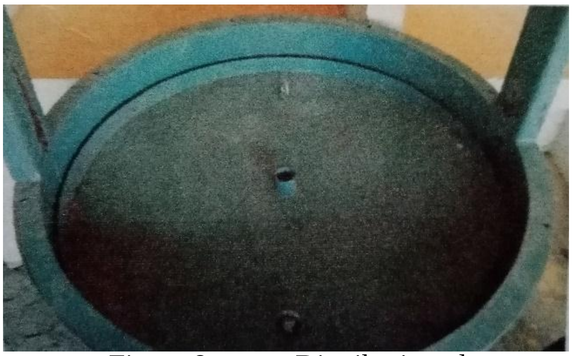


Figure 8: stress Distribution plate

4.2.7 Top plate

Top plate is of equal diameter as collar and it contains holes to place bolts for its tightening. Top side of the plate is equipped with handle whereas bottom surface is flat. Top plate serve as a counter plate to pneumatic pressure system.

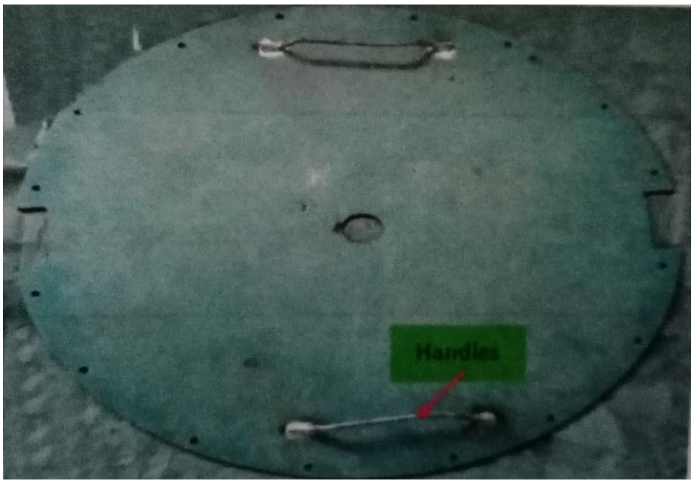


Figure 9: Top Plate

4.2.8 Reaction beam

Reaction beam is a H-Beam with flange width of 5 /z“. At the sides, beam flange is configured with holes aligned with holes of the column. At the midpoint, manual jacking mechanism is connected below the Beam.

Figure 10: Reaction Beam

4.3 Pneumatic Pressure system

It comprises of pneumatic pump, in addition with regulator and a gauge. This mechanism is connected to a tyre tube put in middle of top plate and pressure distribution plate. Expansion of tube in compartment causes downward force on the pressure distribution tube. Pressure distribution plate evenly disperse pressure on soil.

4.3.1 Compressor

An air compressor was installed as a source of air pressure. The compressor operates electrically and have the apparatus to control air pressure of outlet air.

4.3.2 Air lines

Various airlines are used to link the compressor, regulator and air tube. Airlines are provided with connectors on both end for convenience.

4.3.3 Pressure Gauge

A pressure gauge is measures air pressure inside tyre tube.

4.3.4 Pressure Regulator

A pressure regulator is installed with the gauge to maintain tube pressure thus constant confining stress is provided to the soil.

4.4 Guide mechanism

Guide mechanism is provided and can be fastened to the Collar and top cap of Soil container. Guide mechanism contains a hollow rectangular section carrying two telescopic rectangular sections which in succession support the guide rail. Guide rail have in turn fasteners fixed to it. The motive of guide rail is to stop the horizontal movement of the model pile while permitting vertical movement while driving.

4.5 Sand Pluviator

As formerly mentioned, sand is being rained to achieve required density. Therefore a pluviator is set up as shown in figure. The pluviator is composed of upper drum with lid bolted on bottom to form pan. The bottom covering consist of arrangements of holes in rings at equidistant to each other. The diameter of hole is 10mm. At the bottom of the lid is a plexi-glass screen of 5mm thickness attached with a handle to switch it to on and off position. The shutter consist of same pattern of hole therefore positioning of both the holes allow sand to pour. The drum is attached to two diffuser sieves (No#6 and No#10) which expands spread of the sand and adjusts the bedding plane. Both sieves have mesh aligned at 45 degrees to each other. The complete setup is adjusted so that the gap between the sieves and pan can be changed. Furthermore, any single component can be taken out from pluviator.. Pluviator is provided with hooks on all four sides to support hoist /uplifting Structure. The Hoist mechanism enables the pluviator to be moved up and down.

Figure 12: Pluviator

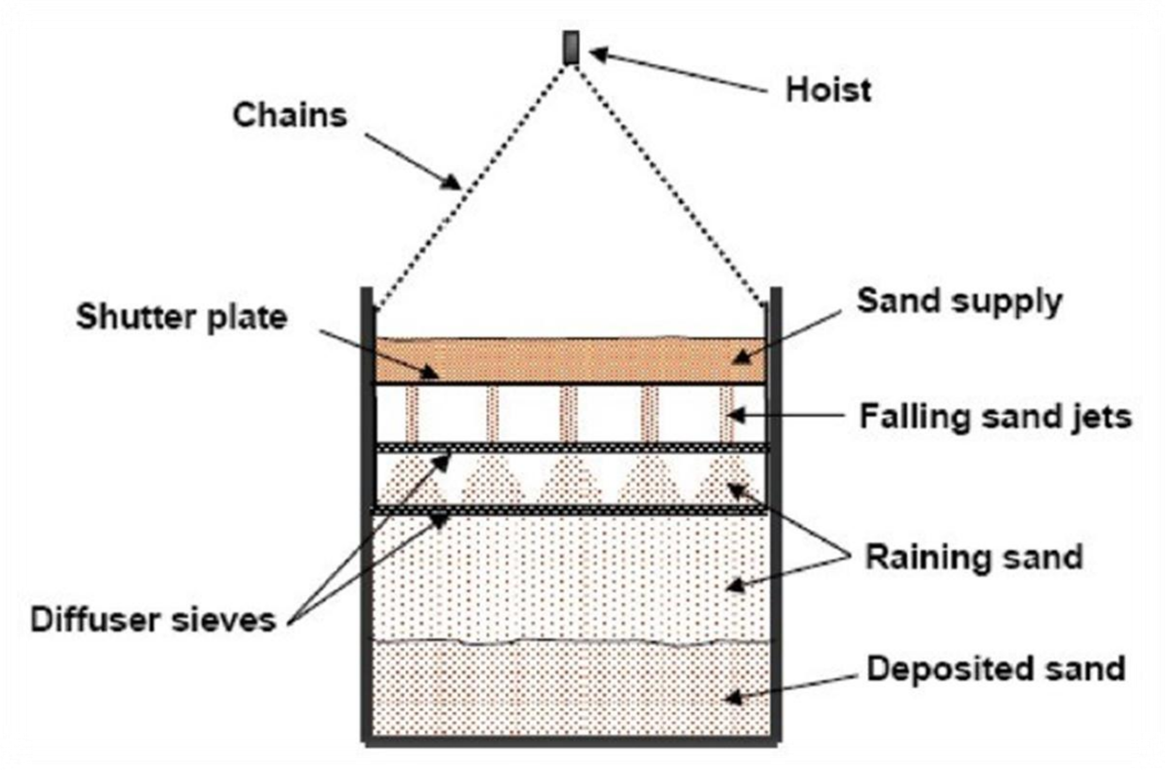


Figure 13: Pluviation

4.6 Uplifting Mechanism

Hoist or uplifting mechanism comprises of a pulley block attached with a gantry crane mounted on a Steel girder. The crane can move backward and forward while pulley moves load vertically. The main purpose of hoist mechanism is movement of pluviator, but it also plays role in moving other burdens such as top plate, pressure distribution plate etc.

Figure 14: Uplifting Mechanism

4.7 Instrumented model Pile

This constitutes pipes with outer diameter of 32mm and 34 and thickness of 3mm. The pile consist of following parts: -

4.7.1 25 mm and 32 mm machined pipe

25 mm and 32 mm pipes were machined for accommodating pile head on top and base load cell. Many slots for various sensors are provided on the external boundary of the pipe. The pipe is drilled with holes for free movement of signal wiring inside the pipe. A hole is provided near the top as an outlet for these power lines. Collectively a 32mm Pipe can support 6 strain gauges, a base load cell, a strain transducer and an accelerometer and a top load cell.

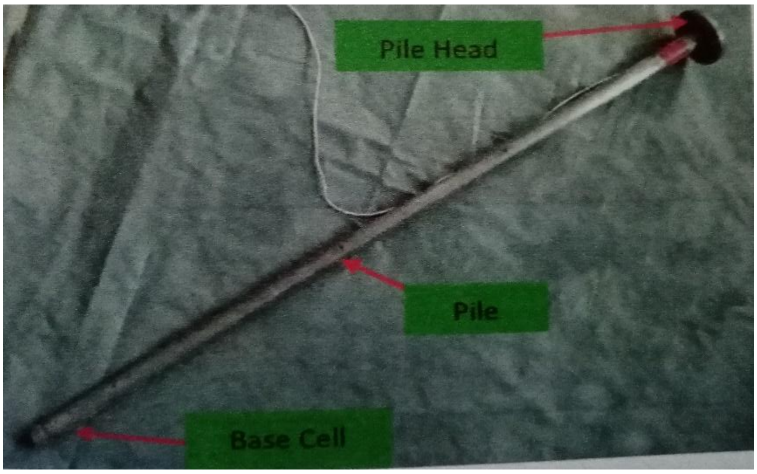


Figure 15: Model Pile

4.7.2 Base Load cell

Two separate base load cell can be threaded to the foundation of the pile. One for lifting Omega LCM- 203 Load cell and other can be coupled with strain gauges to get observation based on strain. A small machined calibrator casing is also provided for convenient calibration of compartment.

4.7.3 Pile head

A pile head can be rested firmly on top of the pipe. This pile head consists of a threaded hole in the middle to support guide rod for 5kg hammer. This rod can be withdrawn to permit static loading of pile.

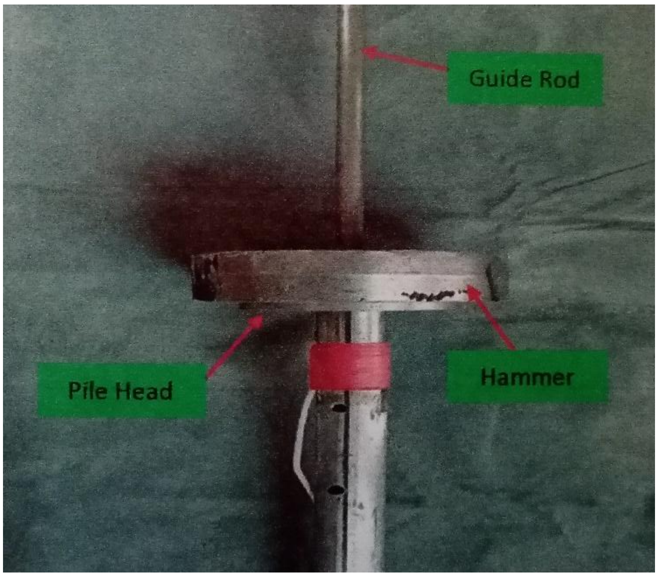


Figure 17: Dynamic Loading Mechanism

4.8 Data Acquisition System

Data Acquisition system is RJ45 based System 8000 Micro measurement machine. For this purpose a small board for pin connectivity is also manufactured. System 8000 Micro measurement device sends data directly to Central Processing Unit from where it can be transmitted using Micro-Measurement computer programs.

Figure 18: Data Acquisition

Chapter 5

Commissioning offacility

5.1 Introduction

After everything was available and put to place the equipment was shifted to Structures lab from Soil Lab as it has mobile crane and machinery for dynamic as well as application of static load. Base load cells were installed in the piles and were sealed using silicone. After that medium dense and loose dense sand samples were prepared. After that piles were put in the sand using driven pile method using both static and dynamic loading. There were also tests for calculating bearing capacity of sand, static load test and dynamic load test.

5.2 Sand

In this experiment Lawrencepur sand was used previously by Capt. Mohibullah, first we tested local Nowshera sand but it was not suitable for the experiment we tested Nowshera sand but it was not fine enough. Ultimately Lawrencepur sand was used. This sand has previously been used by Capt. Mohibullah and also studied by Engr Amer Ahmad in his MS thesis. Some of their test results were also used in our Project.

5.2.1 Max and Minimum Dry Densities

Mass of empty Cylinder = m = l055g

Inner diameter of cylinder = d = 15.2 lcm

Ht of cylinder = h = 15.57cm

Volume =V $= \pi d^2 h/4 = 2830\ cm^3$

(1) Loose Sand

Mass of Cylinder +Sand - 2 = 5l34g

Mass of Sand $= m_3 = m_2 - m_1 = 4079$ g

Dry Density of loose Sand W3 /V =1.44g/cm’

(2) Dense Sand

Mass of Cylinder +Sand $= m_4 = 6007$g

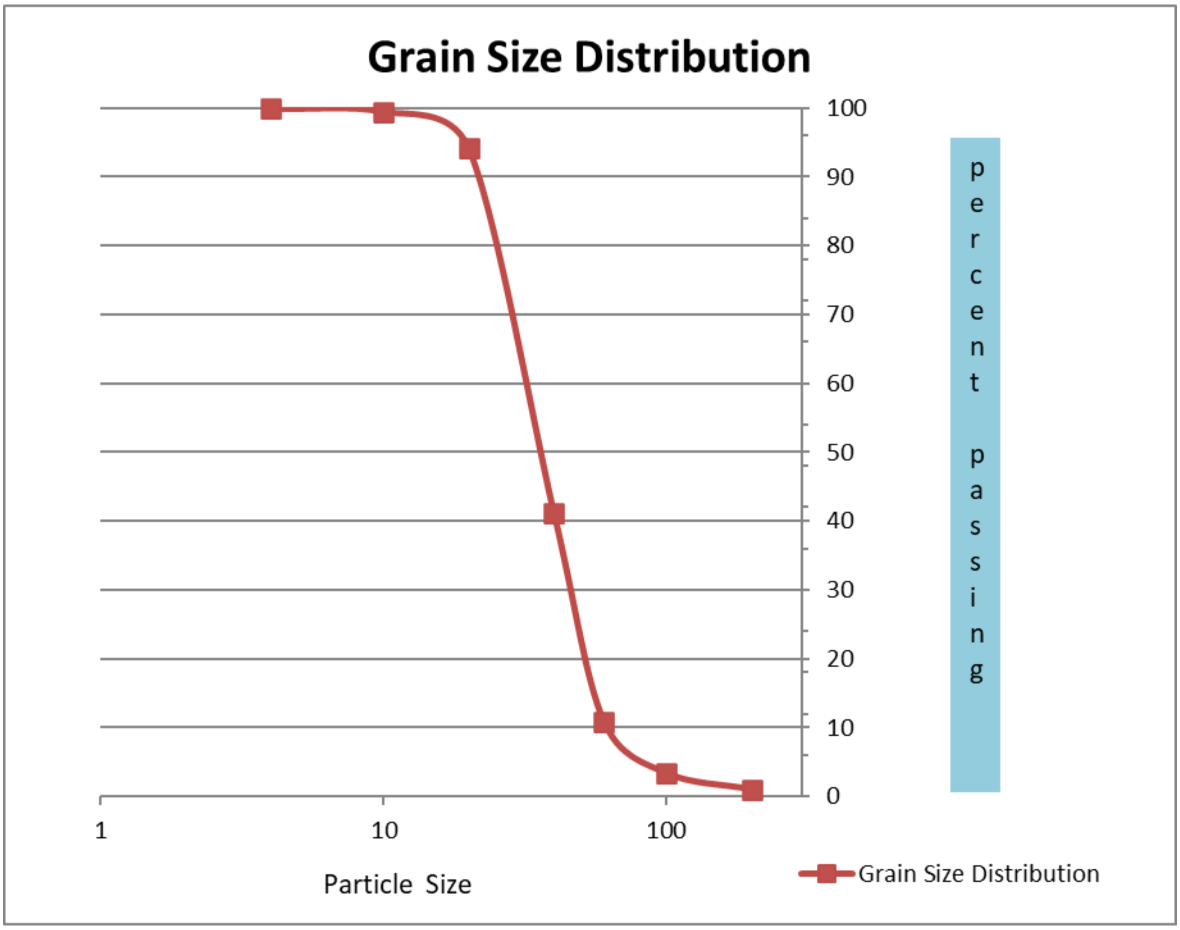
Mass of Sand $= m_5 = m_4 - m_1 = 4952$ g

Dry Density of loose Sand 'Jdm- ms/V 1.75g/cm°

5.2.2 Grain Size Distribution

Table 6

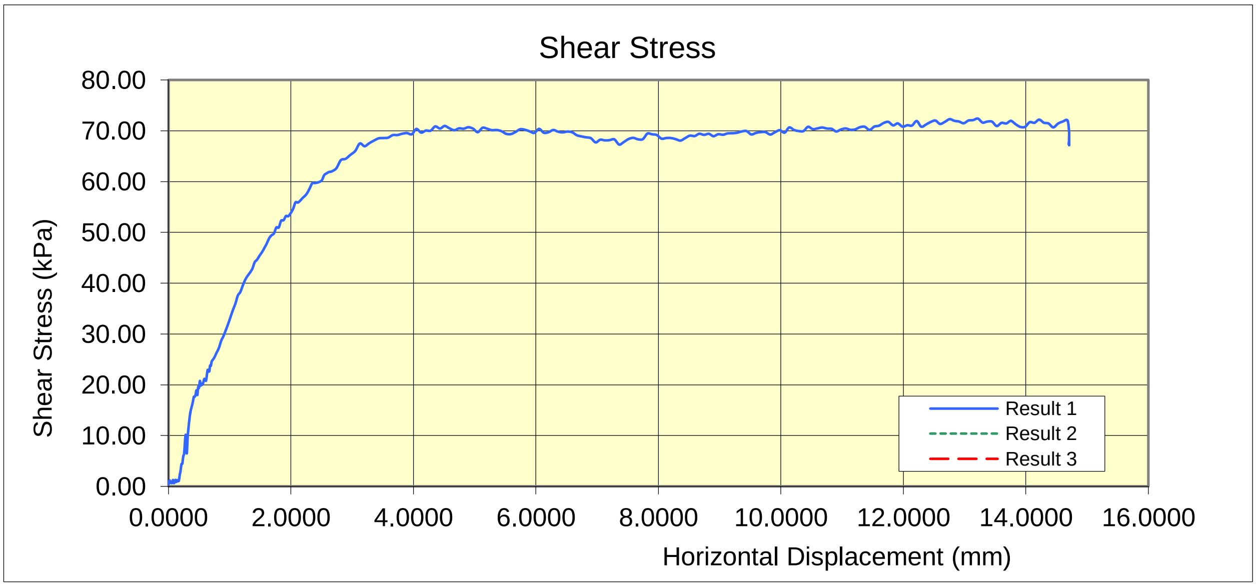
Particle Dia		% Passing
4		99.9
10		99.45
20		94.1
40		41.2
60		10.9
100		3.34
200		0.97
pan		0



The soil is graded as poorly graded soil.

5.2.3 Direct Shear Test

This test was carried out in geo tech lab. They were carried under 100 KPA and critical angle came out to be 34

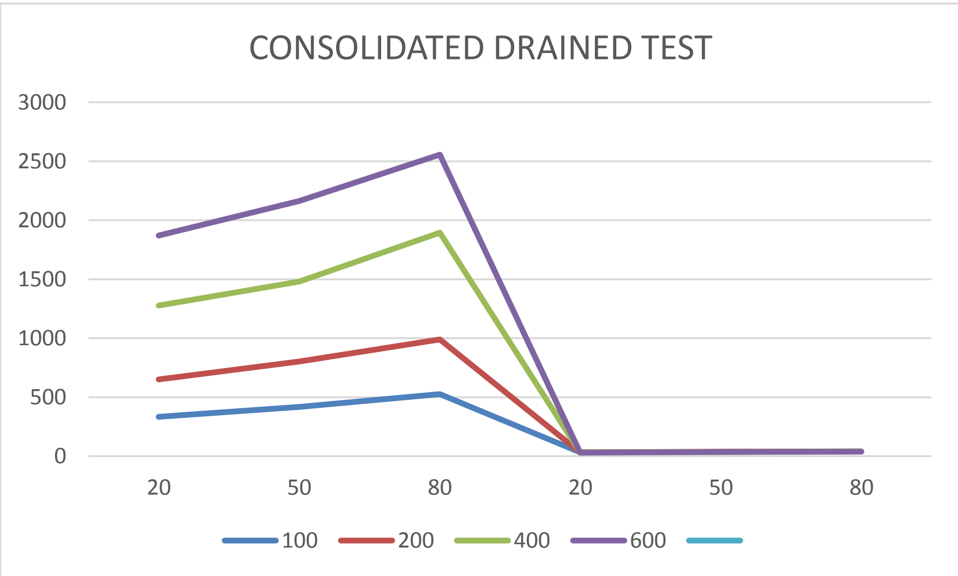


5.2.4 Consolidated Drained Triaxial Tests

Some tests were previously carried out on the same site. Following results were summarized from the work

Table 7: Consolidated Drained Test

			RELATIVE DENSITY				
			PEAK SIGMA 1			PEAK PHI	
		20	50	80	20	50	80
	100	332.18	416.911	525.333	32.44		42.79
CONFINING STRESS	200	651.492	802.141	988.551	31.97	36.87	41.49
	400	1275.954	1478.583	1895.391	31.46	34.98	40.58
	600	1870.67	2163.113	2554.419	30.9	34.39	38.22



5.3FunctionalityTest

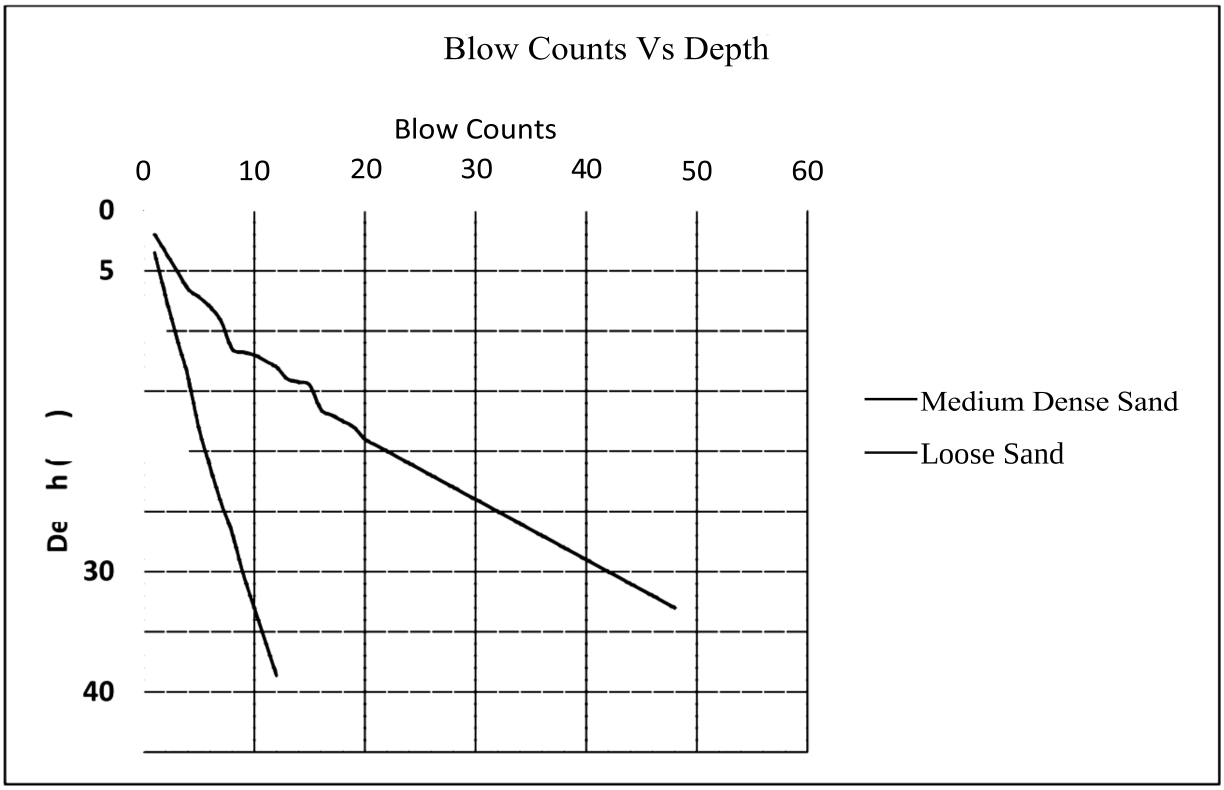
Two samples were prepared i.e loose Sand and medium Dense Sand Piles were driven in them.

Dynamic and static load tests were performed.

That are given below:

5.3.1 Pile DrivingResistance

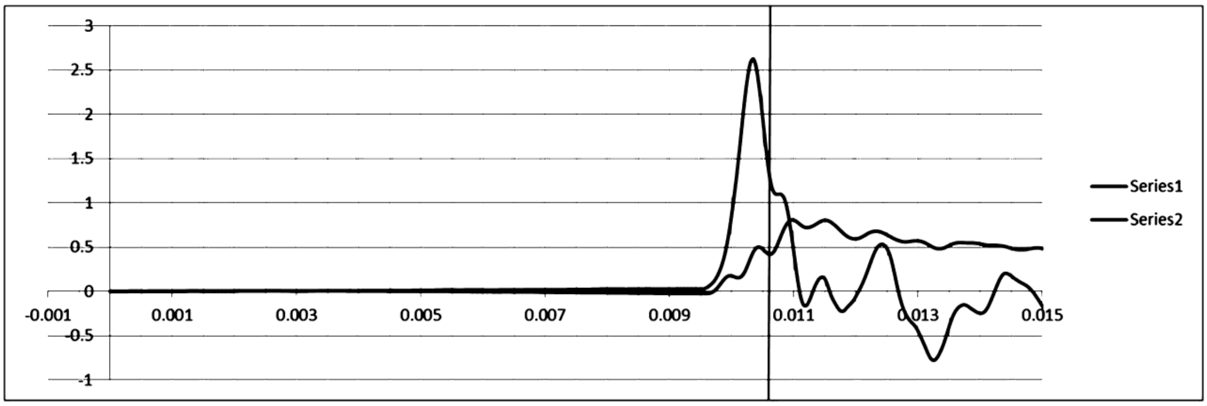
Penetration and blows were recorded and graph was plotted.5kghammerwasdroppedfrom.4 m vertically, Energy of each blow comes out to be 19.6J. Following is relation of loose and Medium DenseSand



5.3.2 Dynamic Load Test

Case Method was used to find the Ultimate Capacity of the pile from dynamic test.

$$Q_{ult} = \frac{1}{2}\left[(F - Zv)(1 + j_c)|_{t_o + 2L/C} \quad + \quad (F - Zv)(1 - j_c)|_{t_o}\right]$$



$F|$¿øzz/C —8.197kN

$F|_{t_o}$=5.95kN

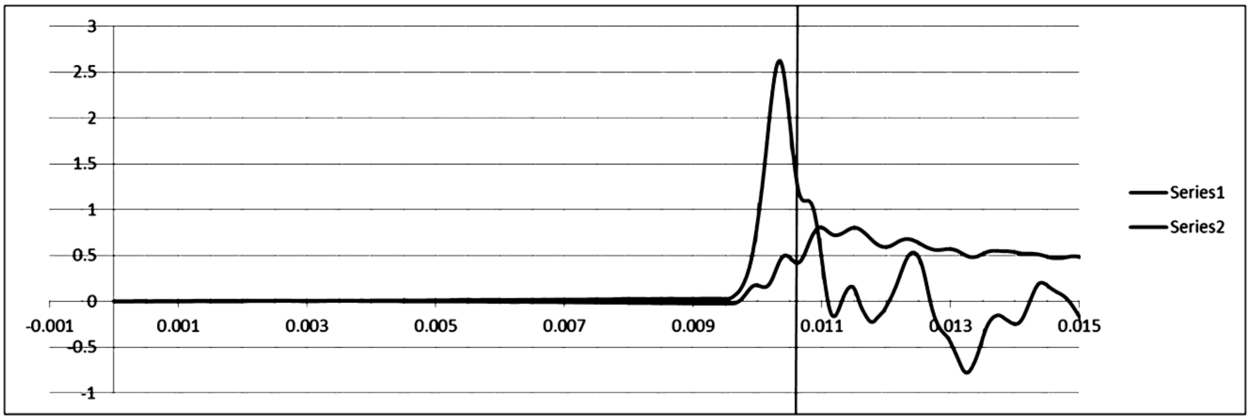
$v|_{t_o+2L/C}$ = 0.45 m/s

$v|_{t_o}$= 0.805m/s

c = 5123 m/s

Jc =0.35

Q_{ult} = l.6kN



Dynamic Load Test- Loose Sand

$F|$qzz/ C — 0.6222kN

$F|_{t_o}$ = 0.623 kN

v| +zr/ = 0.0196mls

$v|_{t_o}$= 0.021mls

c = 5123 mls

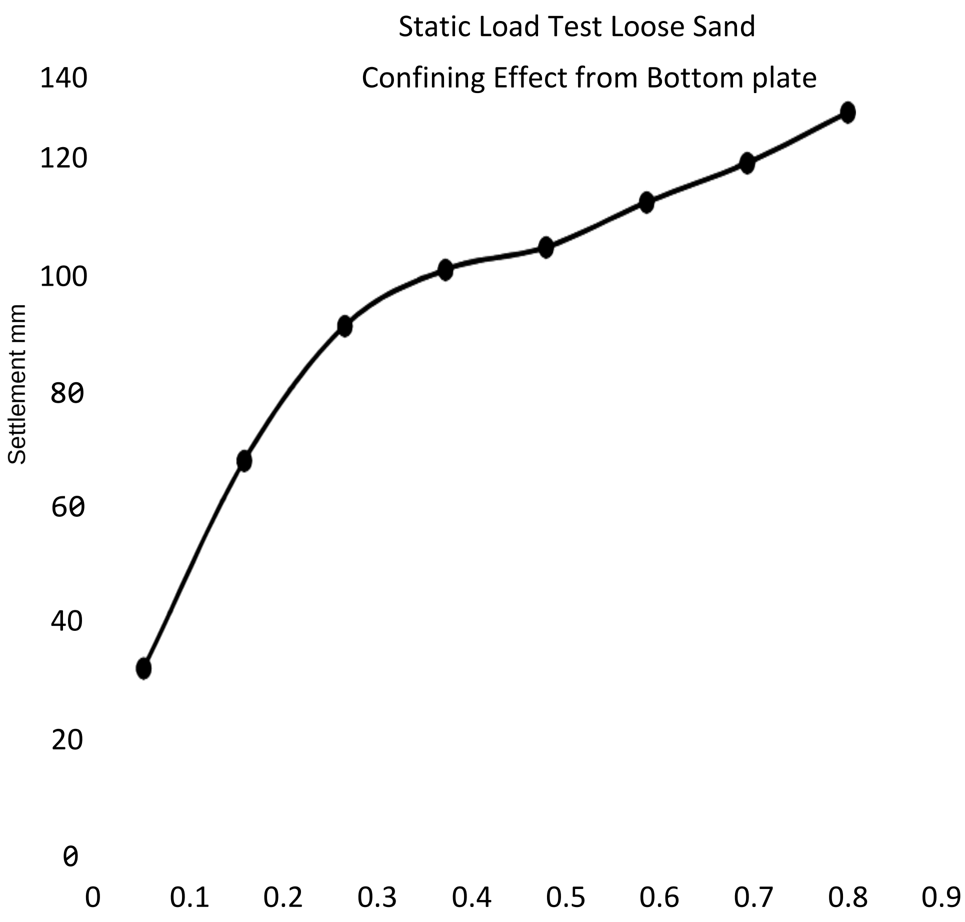
Jc =0.35

Q_{ult} = 0.l4kN

5.3.3 Static Load Test

	STATIC LOAD TEST				
An Equation B(KN)=E(mV/V) /0.6 is derived from calibration of bottom load cell					
S.No	Weight	Bottom Gauge Values	Settlement	Base load	Shaft friction
	KN	E (mV/v)	*mm*	B (KN)	KN
1	0.1	0.01	30	0.02	0.08
2	0.2	0.02	67	0.03	0.17

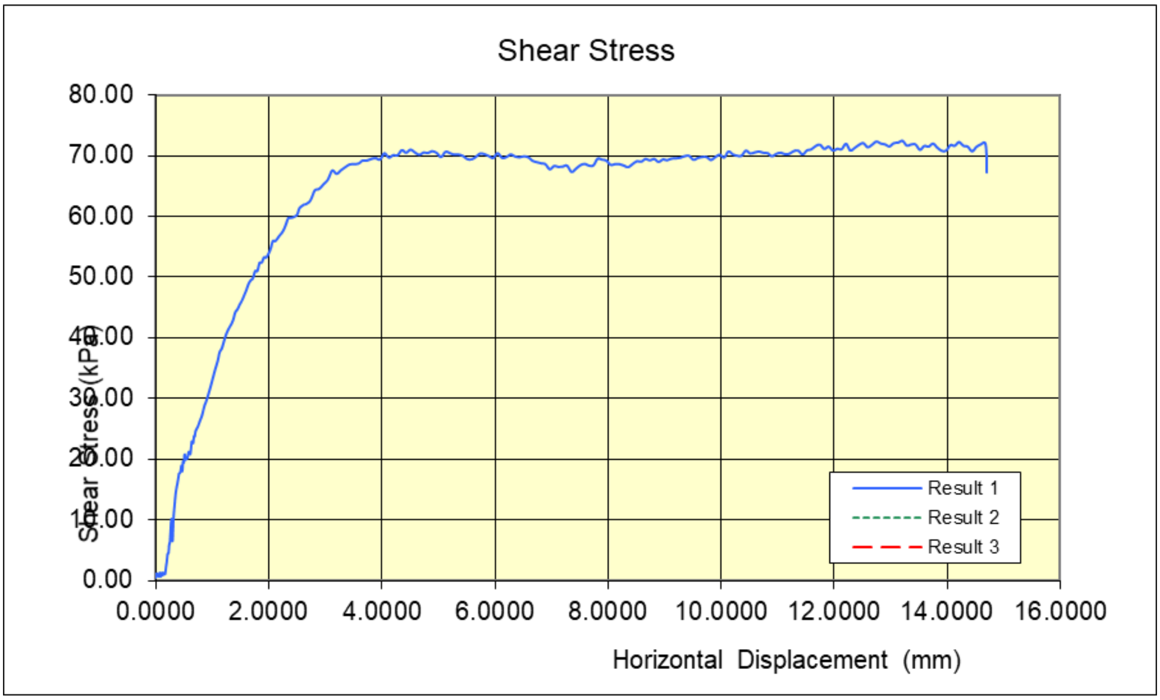
3	0.3	0.03	91	0.05	0.25
4	0.4	0.04	101	0.07	0.33
5	0.5	0.06	105	0.10	0.40
6	0.6	0.07	113	0.12	0.48
7	0.7	0.08	120	0.13	0.57
8	0.8	0.09	129	0.15	0.65

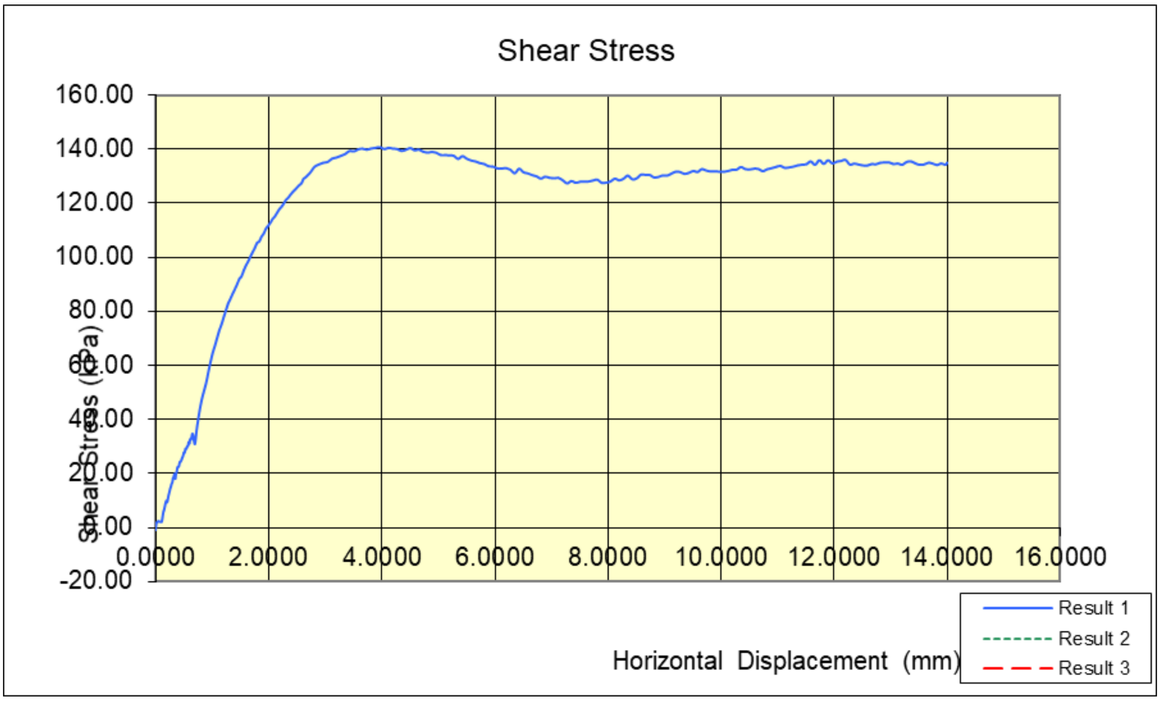


5.3.4 SHEAR TEST:

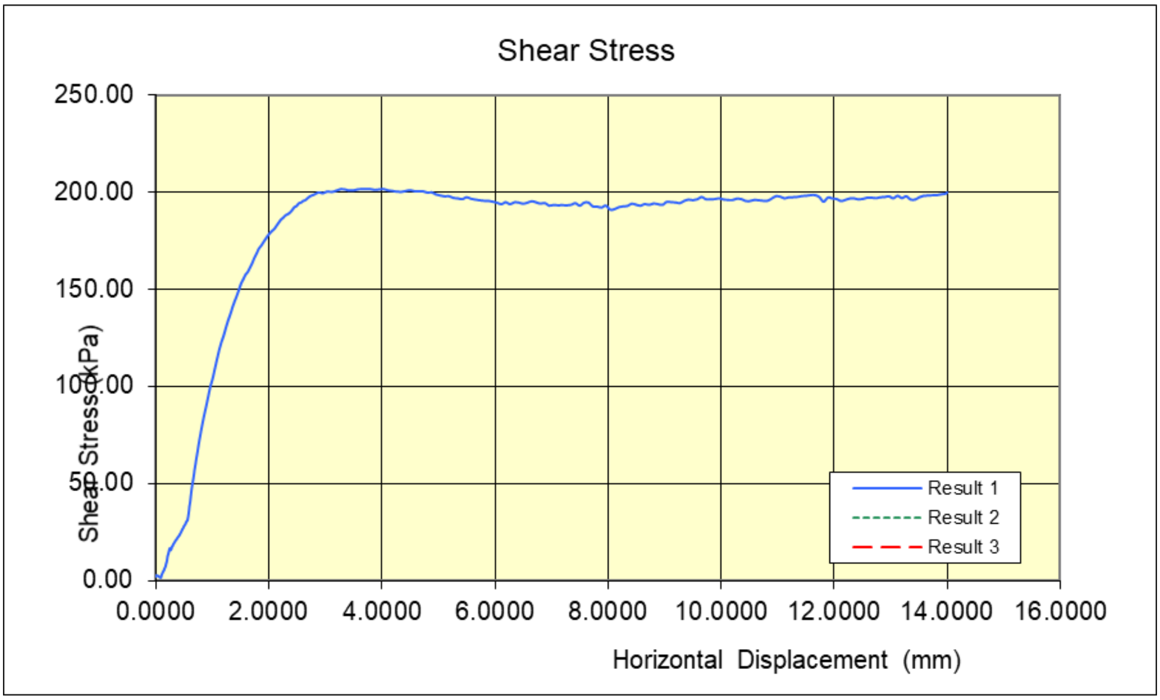
Shear test was conducted on the soil sample and results were collected under 100KPA, 200 KPA and 300KPA.results are as under:

100 KPA





200 KPa


Shear Stress
250.00
200.00
150.00
100.00
50.00
0.00
Shear Stress (kPa)
0.0000
2.0000
4.0000
6.0000
8.0000
10.0000
12.0000
14.0000
16.0000
Horizontal Displacement (mm)
Result 1
Result 2
Result 3

300 KPa

Chapter 6

Conclusion

Model Pile Testing facility is first of its kind lab based equipment in Pakistan. It has enabled the pile testing to be done in a room decreasing the cost and man work . It has following capabilities

6.1 Capabilities of Model Pile Testing Facility

(1) Model pile-soil behavior could be tested in a laboratory.

(2) Different tests including static, dynamic and cyclic loadings could be done by this facility.

(3) Samples of different confining stresses could be prepared in the lab.

(4) Factor of moisture could be taken care of in this equipment.

(5) Different properties soil samples with varying densities could be prepared.

(6) If the roughness ratio is considered somehow it could simulate concrete pile behavior as well.

6.2 Applications of Model Pile Testing Facility

6.2.1 Academic Value

Following are academic benefits of the facilty:

(l) Lab based pile testing equipment.

(2) Factors that affect soil-pile behavior explained in a lab.

(3) Independent of natural factors affecting soil conditions.

(4) Pile capacity prediction by students in Laboratory.

6.2.2 Research Value

Following researches can be carried out on soils of Pakistan

(1) Gap in study of soils of remote areas covered.

(2) Correlation of different parameters of soil with piles.

(3) Effect of moisture on pile capacity.

(4) Dynamic and static loadings research with minimal cost.

(5) Relation of pile capacity and installation of pile.

6.2.3 Proposed Future Development

Following development are recommended for enhancement of capabilities of the facility

(1) Automated jacking system for installation of piles.

(2) Larger capacity tube for large values of stress to be tested.

(3) Pressure actuator with the pneumatic system to control the stress distribution.

(4) Pile Group testing assembly and separate sensors.

(5) Lateral and cyclic loading mechanism for better testing.

(6) Permanent hoisting system for pluviator.

(7) An equipment box for maintaining and storing complete equipment of the facility

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Bibliographv

Alawneh, A. S., Malkawi, A. I. H., & Al-Deeky, H. (1999). Tension tests on smooth and rough model piles in dry sand. *Canadian Geotechnical Journal, 36(4),* 746-753.

Aoki, N., & Velloso, D. d. A. (1975). *An approximate method to estimate the bearing capacity of piles.* Paper presented at the Proc., 5th Pan-American Conf. of Soil Mechanics and Foundation Engineering.

Basu, D., Salgado, R., Prezzi, M., Lee, J., & Park, K. (2005). Recent advances in the design of axially-loaded piles in sandy soils *Advances in Deep Foundations* (pp. l- 13).

BCP, *C.* (l 97 l). Field tests on piles in sand. *Soils and Foundations,* II(2), 29-49.

Been, K., Crooks, J., Becker, D., & Jefferies, M. (1986). The cone penetration test in sands: part I, state parameter interpretation. *Creotechnique, 36(2),* 239-249.

Berezantzev, V., Khristoforov, V., & Golubkov, V. (196 l). *Load bearing capacity and deformation of piled foundations.* Paper presented at the Proceedings of the 5th International Conference on Soil Mechanics and Foundation Engineering, Paris.

Bolton, M. (l 986). The strength and dilatancy of sands. *Geotechnique, 36(* l), 65-78.

Brandon, T. L., & Clough, G. W. (1991). ***Methods*** *of sample fabrication in the Virginia tech calibration chamber.* Paper presented at the Proceedings of the first international symposium on calibration chamber testing. Potsdam. New york.

Chapman, G., & Donald, I. (1981). *Interpretation of static penetration tests in sand.* Paper presented at the International Conference on Soil Mechanics and Foundation Engineering, l0th, 1981, Stockholm, Sweden.

Chow, F., & Jardine, R. (l 996). *Investigations into the behaviour ofdisplacement piles for offshore foundations.* Paper presented at the International Journal of Rock Mechanics and Mining Sciences and Geomechanics Abstracts.

Clausen, *C.,* Aas, P., & Karlsrud, K. (2005). *Bearing capacity of driven piles in sand, the NGI approach.* Paper presented at the Proceedings of Proceedings of International Symposium. on Frontiers in Offshore Geotechnics, Perth.

Fleming, W., & Weltman, A. *1.*, Randolph, MF & Elson, WK (1992). *Piling Engineering.*

Foye, K., Abou-Jaoude, G., Prezzi, M., & Salgado, R. (2009). Resistance factors for use in load and resistance factor design of driven pipe piles in sands. *Journal of Creotechnical and Geoenvironmental Engineering, 135(I),* l- 13.

Franke, E. (1989). Co-report to discussion, session 13: large-diameter piles. *Proc., 12th ICSMGE.*

Ghionna, V. N., Jamiolkowski, M., Pedroni, S., & Salgado, R. (1994). *The tip displacement of drilled shafts in sands.* Paper presented at the Vertical and Horizontal Deformations of Foundations and Embankments.

Jardine, R., Chow, F., Overy, R., & Standing, *I.* (2005). *ICP design methods for driven piles in sands and clays:* Thomas Telford London.

Kolk, H., Baaijens, A., & Senders, M. (2005). *Design criteria for pipe piles in silica sands.* Paper presented at the Proc., 1st Int. Symp. on Frontiers in Offshore Geotechnics.

Lee, J., & Salgado, R. (1999). Determination of pile base resistance in sands. *Journal of Geotechnical and* ***Geoen*** *vironmental Engineering,* 125(8), 673-683.

Lee, J., Salgado, R., & Paik, K. (2003). Estimation of load capacity of pipe piles in sand based on cone penetration test results. *Journal of Geotechnical and Geoenvironmental Engineering,* /29(5), 391-403.

Lehane, B., Schneider, J., & Xu, X. (2005). A review of design methods for offshore driven piles in siliceous sand. *UWA Report GEO, 5358.*

Lopes, F., & Laprovitera, H. (1988). On the prediction of the bearing capacity of bored piles from dynamic penetration tests. *Deep Foundations on Bored and Auger Piles,* 537-540.

Meyerhof, G. G. (1983). Scale effects of ultimate pile capacity. *Journal of Geotechnical Engineering, I09(6)*, 797-806.

Nordlund, R. (1963). Bearing capacity of piles in cohesionless soils. *Journal of the Soil Mechanics and Foundations Division, 89(3),* l -36.

Paik, K., & Salgado, R. (2003). Effect of pile installation method on pipe pile behavior in sands. *Creotechnical Testing Journal, 2* 7(l), 78-88.

Parkin, A., Holden, J., Aamot, K., Last, N., & Lunne, T. (1980). Laboratory investigation of CPT's in sand. *Report No. 52108, 9.*

Parkin, A., & Lunne, T. (1982). Boundary effects in the laboratory calibration of a cone penetrometer for sand. *Norwegian Geotechnical institute publication(* 138).

Parkin, A. K. (1991). Chamber testing of piles in calcareous sand and silt. *Calibration Chamber Testing*, 289-302.

Pelletier, J., Murff, *1.*, & Young, A. (1993). *Historical development and assessment of the current API design methods for axially loaded* ***pipes.*** Paper presented at the Offshore Technology Conference.

Peterson, R. (1988). *Laboratory investigation of the penetration resistance of fine* ***cohesion****less materials.* Paper presented at the Proceedings of the 1st International Symposium on Penetration Testing (ISOPT- l).

Presti, D. L., Pedroni, S., & Crippa, V. (1992). Maximum dry density of cohesionless soils by pluviation and by ASTM D 4253-83: A comparative study. *Geotechnical Testing Journal,* 15(2), 180-189.

Randolph, M. (2003). Science and empiricism in pile foundation design. *Geotechnique, 53(* 10), 847-876.

Reese, L., & O'Neill, M. (1989). *New design method for drilled shafts from common soil and rock tests.* Paper presented at the Foundation Engineering: Current principles and practices.

Robinsky, E., & Morrison, C. (1964). Sand displacement and compaction around model friction piles. *Canadian Geotechnical Journal,* 1(2), 81-93.

Salgado, R. (1995). Analysis of the Axial Response of Non-Displacement Piles in Sand ***Geomechanics*** *II.*

Salgado, R. (2006). Analysis of the axial response of non-displacement piles in sand ***Geomechanics*** *II. Testing, Modeling, and Simulation* (pp. 427-439).

Salgado, R. (2008). *The engineering of foundations* (Vol. 888): McGraw-Hill New York.

Salgado, R., Mitchell, J., & Jamiolkowski, M. (1998). Calibration chamber size effects on penetration resistance in sand. *Journal of Geotechnical and Geoenvironmental Engineering, I* 2#(9), 878-888.

Salgado, R., & Prezzi, M. (2007). Computation of cavity expansion pressure and penetration resistance in sands. *International Journal of Geomechanics,* 7(4), 25 l -265.

Schnaid, F., & Houlsby, G. (1991). Assessment of chamber size effects in the calibration of in situ tests in sand. *Geotechnique, 41(3),* 437-445.

Turner, J., & Kulhawy, F. (1987). Experimental analysis of drilled foundations subjected to repeated axial loads under drained conditions, Report EL-5325. *Electric Power Research Institute, Palo Alto, California.*

Vipulanandan, C., Wong, D., Ochoa, M., & O'Neill, M. (1989). *Modelling of displacement piles in sand using a pressure chamber.* Paper presented at the Foundation engineering: Current principles and practices.