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

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MILITARY COLLEGE OF ENGINEERING

NATIONAL UNIVERSITY OF SCIENCES & TECHNOLOGY

RISALPUR CAMPUS, PAKISTAN

(2018)

This to certify that the

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Has been accepted towards the partial fulfilment of the requirements for

BE Civil Engineering Degree

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Dr. Mazhar Iqbal Arshad, PhD

Syndicate Advisor

Dedication

To my parents and Siblings who always supported me in my endeavors and to Mrs Gulfam Naz my third Grade teacher who taught me to question.

-Muhib

To my country

-Modood

To my parents, my wife and my child who are source of motivation for me

-Munam

To my family

-Junaid Imam

To my parents, my wife and my child who are source of motivation for me

-Mahmood

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All Syndicate members

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ABSTRACT

Pile Soil behavior under varying field conditions and soil parameters is essential to understanding of geotechnical design. Most Civil Engineering foundation consist of Piles however piles are huge structures and mostly studied in field. This make the analysis of piles costly. Moreover, most of soils in Pakistan are understudied creating a handicap for geotechnical designer. We have endeavored to remove this handicap by developing a Model Pile Testing facility where varying soils under varying field conditions can be studied for purpose of education and Research. The facility has the capability of simulating varying confining stresses, varying densities, depicting strata and static and dynamic loading. This facility will serve as a pivot for geotechnical research in Piles in Pakistan apart from being a tool for geotechnical learning for student.

This facility affords sensor system to measure different type of loadings and their respective response on base and shaft capacity of Pile. Facility is a miniature of Pile and soil interaction. The facility is equipped with various tools for rapid use and reuse.

Chapter 1

INTRODUCTION

1.1 Background

Foundations are one of the most important part of any civil Engineering structure and yet a complex one since its behavior depends on nature and type of soil, it is interacting with. There are two type of foundations widely used – Shallow and Deep. Shallow foundations includes footings, raft foundations etc and are mostly used in small single to double story buildings. Deep foundations that include piles are mostly used in huge structures. Every mega project of civil Engineering usually consist of deep foundations. Piles design are usually complex as it requires intricate study of soil strata, it is interacting with. The behavior of various type of soil with varying gradations, various mechanical properties and type of mineral composition exhibits diversity in behavior under different type of loading. Usually design process of piles consist of calculation of ultimate vertical load capacity. Capacity is then divided by a suitable factor of safety to account for uncertainties in soil parameters etc. giving allowable vertical loads.

1.2 Problem Statement

Design of Deep foundations requires understanding of different soil strata and their interaction with sides and base of pile. Various theoretical models exist for calculation of these parameters however these models are required to be calibrated/validated as per changing soils. This can be best understood by simulating field conditions in Laboratory and then observe Pile behavior under required loading and comparing results thus obtained with theoretical calculations. Therefore, there is a need to develop a model Testing facility where different loading patterns can be tested on different soil types and conditions

1.3 Scope and Objectives

The project aims at development of a facility to facilitate research in deep foundations and to help student develop better understanding of the concepts related to them. The objective of the project is development of model Pile Testing facility with following capabilities:

a. Simulation of vertical effective stress on the soil in tank.
b. System to observe behavior of pile under loading.
c. Capability of imparting dynamic as well as static loads
d. Measurement of skin friction and End bearing stresses
e. Using soil with different characteristics and densities

1.4 Relevance of Research and Research Questions

Model Pile Testing Facility will help in better understanding of Pile behavior. This will help students to validate their theoretical calculations as per practical behavior of pile. This will also help student understand relation of various parameters such as density, effective stress etc. on behavior of Pile. This model will help conduct research on behavior of Pile on different soils in Pakistan. This will help researchers to develop different correlations for calculation of Pile behavior under varying soil parameters. Research will revolve around following Questions:

a. What are the suitable dimensions for Model Pile Facility?
b. What load and stress arrangement best suites the functioning of facility?
c. What arrangement are to be made for deposition of soil?
d. What arrangement of sensors meet the adequate requirements of the facility?
e. How different field conditions can be simulated in the facility?
f. How varying of different perimeters of soil can be simulated?

Chapter 2

Literature Review

2.1 Introduction

In the last few decades, a wide-ranging research has been performed on the designing and investigation of pile groundwork. However, knowledge gaps still exist in many phases of pile loading behaviour which includes the effects of pile loading and installation on pile mounding response. Because of the complexity of the pile installation and loading problem, estimation of pile capacity (i.e., base and shaft resistances) in practice is based mostly on empirical methods. As a result of the many uncertainties involved, a great degree of conservatism is used in design. Two approaches are being used for the designing of single loading of piles on which loading is implied axially: the approached includes 1- Direct method which is in – situ based method and 2- property based method which is considered as indirect. To imply the indirect designing, the property which should be estimated on priority are soil properties in which we are interested whereas in the direct designing, which includes in – situ based methods, input variables are the test output which is given by in – situ method designing. This chapter considerably focuses on the responses of single piles in sand.

2.2 Estimation of Base Resistance in Sand

2.2.1 Soil property-based methods

q_b defines the "unit base resistance" which interrelates the "in situ vertical effective stress σ_v at the base of the pile and the dimensionless bearing capacity factor N_q," which can be stated as:

In a similar manner, this equation can be expressed conveniently in terms of the "ultimate unit base resistance":

$$q_b = \sigma_v' N_q$$

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

The "ultimate unit base resistance" depends on the criterion used to obtain ultimate loads. The most widely used criterion is the 10% relative settlement criterion which insists 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 10%.

A number of researchers proposed failure mechanisms for the base of the pile and equations for calculation of the bearing "capacity factor N_q", which is directly proportional to the frictional angle with the soil. However, N_{qult} is not constant with σ_v and decreases with an increase in the vertical effective stress σ_v. Salgado (1995) showed that the 'ultimate unit base resistance q_{bult} shows nonlinearity upon increase when the rates are decreasing, with an increase in σ_v, hence with an increase in length of the pile.

According to Fleming and Weltman (1992), the bearing capacity factor N_{qult} for driven piles is given as follows:

$$N_{b.ult} = 0.136 e^{0.182\phi_b}$$

Whereas I_p is the 'peak friction angle of the sand in degrees'. Equation. 2.4 is created on the explanation given by Berezant zev et al. (1961). The peak friction angle I_p can be calculated and deduced for 'tri-axial compression conditions' using the iterative co-relation proposed by Bolton (1986)

$$\phi_b = \phi_c + 3\left[\frac{D_R}{100}\left(Q - ln\phi_{mp}\right) - R_Q\right]$$

where I_c is defined as the critical-state friction angle expressed in degrees, DR is the relative density in percentage, σ_{mp} is defined as the mean peak 'effective confining stress'. Q and RQ are the parameters which are dependent upon the fundamental characteristics of the sand. If no other information is available, for clean silica sand, values of 10 and 1 can be used for Q and RQ, respectively.

The American Petroleum Institute (API, 1993) recommended values for the bearing capacity factor Nq that depend on soil density and particle size (see Table 2.1). The method known as A.P.I design which is used for axially loaded piles, is created on an internationally recognized large databank of 'axial pile load tests' that is continually re-evaluated and updated (Pelletier, Murff, and 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 (1993) also recommends limiting unit base resistance q_{bL} values (see Table 2.1). This method was shown by Chow (1996to Chow and Jardine (1996)be highly conservative when used to predict the capacity of 100-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 (1963) suggested a semi-empirical approach to design driven piles in cohesion-less soils are created on several pile loading tests and provided charts for obtaining design parameters. According to the Nordlund method, the ultimate base resistance q_{bult} is expressed as follows:

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

where D_t is a dimensionless factor that depends on an embedded pile length-pile diameter ratio, and N_q is a bearing capacity factor that is obtained from design charts (see Figure A.1 and A.2 in the Appendix).

Lee et al. (2003) showed via an investigational platform including 36 calibration chamber model pile tests and 2 full-scale field pile loading tests that the has the loading capability of piles of pipes, 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.

Salgado and Prezzi (2007) proposed an equation, in terms of relative density DR and lateral effective stress σ_h for the calculation of the limit unit base resistance q_{bL}:

$$\frac{q_{bL}}{P_A} = 1.64 \exp[0.1041\phi_c + (0.0264 - 0.0002\phi_c)\, D_R] \left(\frac{\sigma'_h}{P_A}\right)^{0.841-0.0047D_R}$$

where I_c is the critical-state friction angle expressed in degrees, DR exhibits relative density given in percentage units, σ_h is the in situ horizontal effective stress and pA is the reference stress (=100kPa). The ultimate unit base resistance q_{bult} of displacement and non-displacement piles in sand can be computed from q_{bL} (Salgado, 2008) using Eq. 2.8 (Foye et al., in press) respectively:

$$q_{b.ult} = (1.020 - 0.0051D_R)q_{bL}$$

$$q_{b.ult} = [0.23 \exp(0.0066D_R)]q_{bL}$$

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'. In situ test-based methods were developed because of the uncertainty and difficulty in characterizing soil properties. Use of CPT data in pile design is considered ideal because the cone penetration process is similar to that of a pile plunging into the ground, 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_{bL}.

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 $q_{b,ult}$ 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, p_A is a 'reference stress (=100kPa)'. Table 2.2 shows design values for the constants cb and nb proposed by several researchers for piles in sand.

Pile type	Value	Source
Driven Piles	C_b= 0.35-0.50	Chow (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% C_b =0.20-0.43 for D =90%	Lee and Salgado (1999) Basu et al. (2005)
	n_b = 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 et al. (1994)
	C_b = 0.23exp(-0.0066D_R)	Salgado (2005, 2006)
	n_b =0.82 for clean sand n_b =0.72 for sand with silt or clay	Lopes and Laprovitera (1988)
	n_b = 0.6	Reese and O'Neill (1989)

Table 2: Design values of the constants c and b n for piles in sand

A number of Multiple design methods were proposed recently that provide relationships between qc and the ultimate unit base resistance of closed-ended pipe piles driven in sand. These methods are known as the Fugro (Kolk et al., 2005), ICP (Imperial College; Jardine et al., 2005), NGI (Norwegian Geotechnical Institute; Clausen et al., 2005), and UWA (University of Western Australia; Lehane et al., 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 al..(2005)
ICP	$q_{b.ult} = \left[1 - 0.5\left(\frac{B_o}{B_{CPT}}\right)\right] q_{cb.avg}$	Jardine et al..(2005)
NGI	$\frac{q_{b,ult}}{q_{cb.avg}} = \frac{0.8}{1 + \left[0.4 \ln\left\{\frac{q_{cb,avg}}{22(\sigma'_{vo} p_A)^{0.5}}\right\}\right]^2}$	Clausen et al..(2005)
NWA	$\frac{q_{b,ult}}{q_{cb.avg}} = 0.6$	Lehane et al..(2005)

Table 3: Multiple Design Methods

2.3 Estimation of Shaft Resistance in Sand

2.3.1 Soil property-based methods

In most cases, the shaft resistance of a pile is completely organized along with the shaft of the pile when it is in an in-service condition. Figure 2.2 illustrates the concept of pile shaft resistance. The appropriate design shaft resistance is the limit shaft resistance q_{sL} 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. Therefore, the limit unit shaft resistance can be expressed as follows:

$$q_{sl} = K\sigma'_v \, tan\delta$$

Whereas K defines the coefficient of lateral earth pressure, σ_v represents the vertical effective stress, and φ 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 (1993) suggested the specific amendments to the original A.P.I method. They demonstrated that, as a result of the erroneous assumption of a constant value of K, the API method inclines to under-predict the capability of

shorter piles and over-predict the capability of longer piles. They suggested the use of a new parameter Kav, an average horizontal earth pressure coefficient used to reduce the discrepancies in the predictions of pile capacities. This parameter is calculated as follows:

$$Kav = 60 / (L/ B + 5)$$

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, Fleming et al. (1992) proposed that K values are approximately 2% of $N_{q,ult}$:

$$K = 0.02Nq,ult$$

2.3.2 In situ test-based methods

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

$$q = c_b*q_c$$

$$q = p_A N_b n_b$$

where c_{si} and n_{si} are constants which are dependent on the type of soil and the type of pile. q_{ci} and N_{si} are the representative cone resistance and blow count number for layer I, and p_A is the reference stress (=100kPa).

Pile type	Value	Source
Drien Piles	c_s =0.008 for open-ended steel pipe piles c_s =0.012 for precast concrete and closed-ended steel pipe piles c_s =0.018 for Franki and timber piles	Schmertmann (1978)
	c_s =0.004-0.006 for D 50% c_s = 0.004-0.007 for 50%<D 70% c_s =0.004-0.009 for 70%<D 90% For closed-ended pipe piles	Lee et al. (2003)
	c_s =0.0040 for clean sand c_s =0.0057 for silty sand c_s = 0.0069 for silty sand with clay c_s =0.0080 for clayey sand with silt c_s =0.0086 for clayey sand	Aoki and Velloso (1975) Aoki et al. (1978)*
	n_s =0.033 for sand n_s =0.038 for silty sand n_s = 0.040 for silty sand with clay n_s =0.033 for clayey sand with silt n_s =0.043 for clayey sand	
Non Displacement Piles	c_s =0.0027 for clean sand c_s =0.0037 for silty sand c_s = 0.0046 for silty sand with clay c_s =0.0054 for clayey sand with silt c_s =0.0058 for clayey sand	Lopes and Laprovitera (1988)
	n_s =0.014 for sand n_s =0.016 for silty sand n_s = 0.020 for silty sand with clay n_s =0.024 for clayey sand with silt n_s =0.026 for clayey sand	

Table 4: provides design values for cs and ns that are available in the literature

2.4 Influence of Pile Installation Method

Piles are classified according to the installation method because pile installation is one of the most important factors affecting the load reaction of piles. Pile foundations have been used for thousands of years, but only in the last few decades there has been significant progress in pile installation technology (Salgado, 2005). In this section, displacement (driven) piles and non-displacement piles will be reviewed considering the influence of the fitting method on the response of piles in sand, since this is the primary concern of this study. When piles are installed by driving or jacking methods, they are referred to as displacement piles because they are installed by pushing and preloading the soil around the pile.

In the case of non-displacement piles, piles are installed by pre-removal of soil from the ground. Because when imposed with non-displacement piles, the condition of soil which is surrounded by the pile, non-displacement piles impose minimal change, they typically have a smaller load capacity than displacement piles. Indeed, the BCP Committee (1971) showed through a series of field pile load tests in dense sands that the load-settlement curves of driven and jacked piles are stiffer as compared to the one subjected to non-displacement piles.

Driven piles are typically made of prestressed concrete, steel, and timber. Depending on the pile and soil conditions, different pile driving systems are used in practice. When a pile is driven into sand by the blows of an impact hammer, soil is displaced to make room for the pile, and, therefore, the density and stress condition of the soil surrounding the pile changes significantly after pile driving. As a result of these changes, the pile load capacity increases accordingly. For this reason, driven piles are considered advantageous when compared with non-displacement piles.

Robinsky and Morrison (1964)) conducted model pile tests in sand to investigate the extent of soil compaction and displacement around driven piles. By using radiography techniques, these authors observed 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 base. In the caseof DR=35%, the soil movement extended to 4.5 to 5.5 pile diameters from the shaft and 3.0 to 4.5 pile diameters below the base.

Meyerhof (1983)summarized a number of empirical data obtained mainly from full-scale pile load tests and proposed that the ultimate unit base resistance of non-displacement piles is roughly one-third of that of driven piles. Meyerhof also suggested that about one-half of the unit shaft resistance of driven piles may be used for preliminary estimates of non-displacement pile shaft capacity.

Paik and Salgado (2004) performed a number of model pile load tests on 14 driven and two jacked pipe piles in sand to explore the outcome of the pile fitting method. Tests were performed under various testing conditions, using different combinations of hammer weights and drop heights but maintaining the driving energy constant; the test results directed that the bearing capability of the pile increases as the hammer weight increases. When the driving energy increased by increasing the hammer weight for the same drop height, the “rate of increase of the shaft resistance” was higher than that of the base resistance because of friction fatigue. For the same conditions, the shaft resistance of the jacked piles was found to be larger 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	
Parkin et al (1980)	25.2 35.7	20-48	760 1,220	1,220 1,500	0.45	0.30	CPT
Champmen & 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 al (1986)	36	21	760	800	0.37	0.25	CPT
Been et al (1987)	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
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 lateral 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 of installation of piles

Table 5: Caliberation chamber used in past

Various Design aspects have also been discussed by various researchers such as Chamber to Probe Ratio or Probe to Particle size Ratio:

2.5.1 Sample preparation

Because of close relation between relative density and preparation of sand samples with the behavior of piles, preparation of samples is utmost importance. Higher dry density, no particle crushing, less segregation of particle sizes, accuracy of density measurements and better repeatability are few of the advantages of Pluviation (raining) method over ASTM method (ASTM D 4253) as highlighted by Presti, Pedroni, and Crippa (1992). Being more economical, this method has also an edge over the vibrating table method. The efficiency and reliability of 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.

2.5.2 Size Effects

Parkin et al. (1980) established effects of cone penetration and chamber size based on relative density of sand. The effects were measured 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 dense and loose sand respectively, however the boundary effects must be considered for conducting penetration tests on dense sand samples.

In order reduce the effect of chamber size Been et al. (1986) remarked that chamber-to-cone ratio must be more than 50 for sands having DR=90%, where as 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.

For limiting the theoretical plastic zone within chamber, 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 et al. ,1980).

For eliminating chamber size effects, a study conducted through numerical and experimental studies, proposed that lower limit of chamber-to-probe diameter ratio should be restricted to 50 in dense sands (Schnaid and Houlsby,1991) where as the same ratio was suggested to be greater than 100 by Salgado et al. (1998) for reducing 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) have suggested pile/probe diameter to soil particle diameter for reducing the internal scale effects. A suggested ratio of pile diameter to particle diameter of 80 and larger was suggested by Peterson (1988) based on lab examination to establish effect of specimen density, grain size, penetrometer diameter, penetration rate and pore water on penetration of fine sands in order to reduce the internal scale effects. For probe-to-particle diameter ratio of 40 and less it was pointed out by Peterson (1988) that penetrometer will sense individual particles as opposed to Vipulanandan et al. (1989) which suggested the ratio to be at least 50 for Soil Dia D_{10}.

2.5.4 Sand Relative Density

Turner and Kulhawy (1987) established that sand drop height and discharge rate is directly related to unit weight of sand deposited. Hence, by altering Pulviator sieve size and sand drop height the relative density of sand being deposited can be varied. Moreover, the density variation was not expected to surpass 1% if prepared by Pulviation Method as indicated by Parkin and Lunne (1982). Since the sand properties and pile capacities are greatly dependent on sand density therefore its verification is of utmost importance. The study established an optimum combination of sieve size and drop height of sand Pulviation was determined.

Chapter 3

Methodology

For purpose of development of this facility, thorough Literature Review was carried out followed by Designing of facility on Solidworks Software. These designs were then given to various fabricators and machinists to ensure acceptable tolerances. Sensors were installed and calibrated. Pluviation was also calibrated for relative Densities. This was followed by different tests conducted on the facility to establish its functionality.

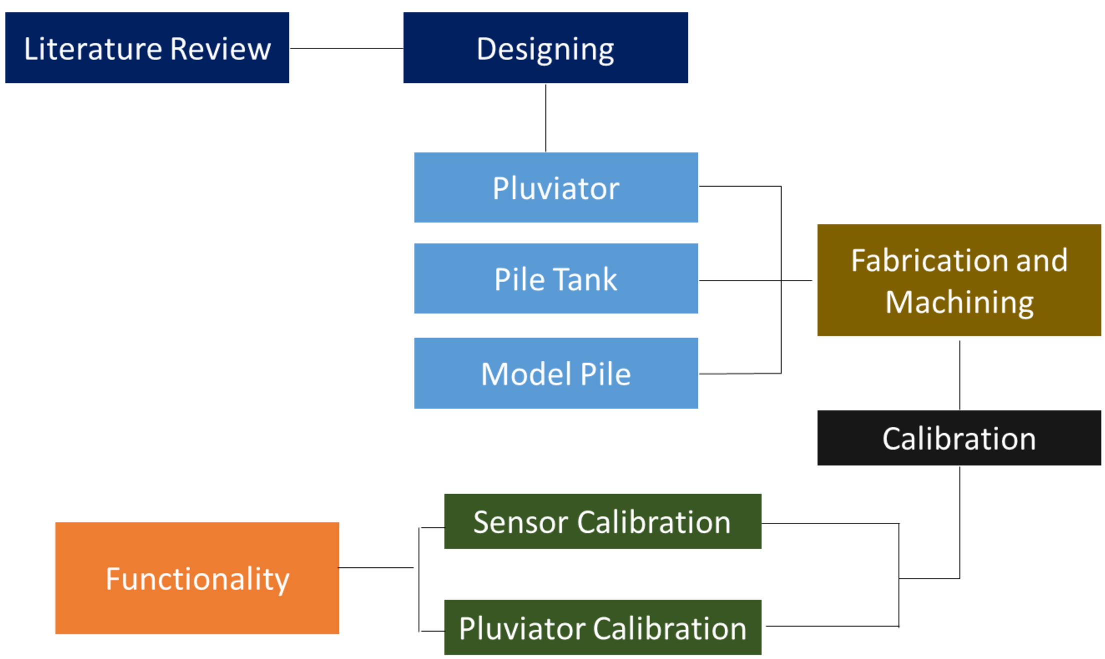


Figure 1: Methodoly

3.1 Literature Review.

Literature Review was conducted to search for past usage such chambers discussed in detail in Chapter 2. Parameters such as Pile to Dia and Pile to particle size ratio were derived to establish Pile and Soil Tank size. 1200mm Diameter and 1000mm height of Soil tank was established. Pile diameter was established to be 32mm and Lawrencepur sand was selected as suitable Sand to carry out further study.

3.2 Designing.

Solidworks® was used to design in detail all the parts. Strength of steel and its distortions under various loadings were considered to establish various thicknesses. Workability in Laboratory environment and Versatility to cater for various simulations of field conditions were the major considerations. Design were modified to cater for rapid use and reuse of the facility.

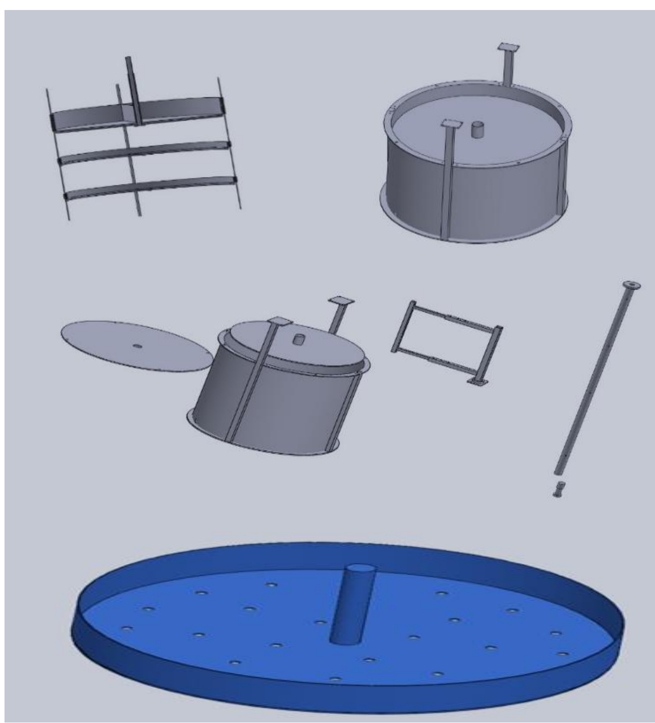


Figure 2: 3D Modeling in Solidworks

3.3 Fabrication and Machining

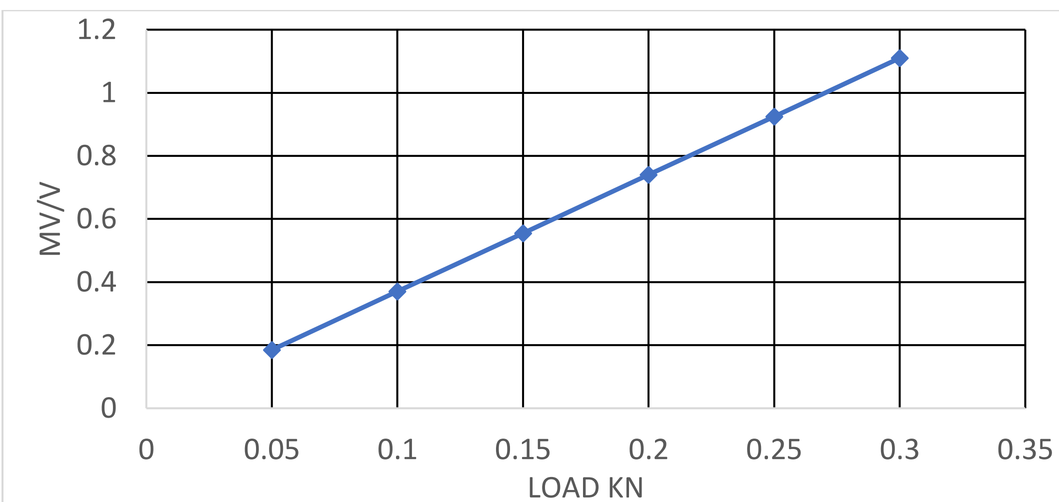
Soil tank alongwith reaction beam and pluviator were manufactured from a workshop in Rawalpindi while Model Pile and Guide mechanism was fabricated/machined at a workshop near Kamra Attock.High tolerances were maintained in the machining of model pile. Pile was designed keeping in view already procured load cells and housings for various sensors were provided in the model pile. Pile was prepared keeping in view both dynamic and static load tests.

3.4 Calibrations

Both Load Cells and pluviation systems were calibrated for further use.

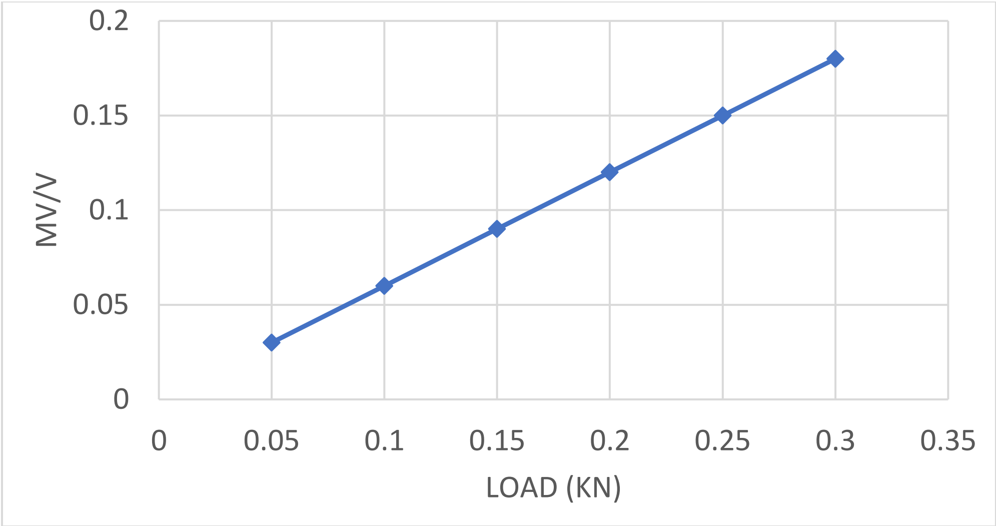
3.4.1 Calibration of load cells

Before embedding them with piles. Both the sensors were calibrated. LCM-307 Omega Load cell was calibrated after being housed in the base of Model pile. Oedometer equipment was used to calibrate both cells and equations were derived for the calculation of load (kN) from signal (mV/V).



S-CELL Calibration

$$\text{Load(kN)} = \text{Output(mV/V)} / 3.7$$

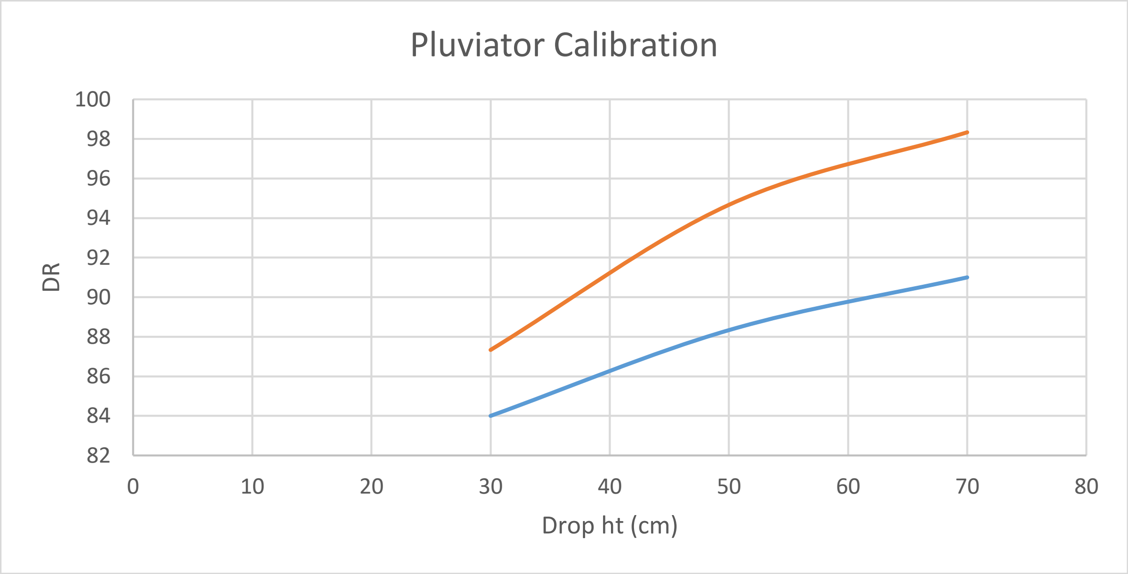


Bottom Load Cell Calibration

$$\text{Load(kN)} \quad = \quad \text{Output(mV/V)} / 0.6$$

3.4.2 Pluviator Calibration

A Series of Experiments were conducted to calibrate pluviator height vs density. Following graph are depicting the result:



3.5 Functionality

Sand was was tested for maximum and minimum densities, specific gravity, max and min void ratios. Direct Shear Test and Grain size Distribution was also conducted. Two samples were prepared – one with loose Sand and one with medium dense Sand. Dynamic and Static Load Tests were conducted on both samples. Capacity was also calculated using theoretical methods.

Chapter 4

Development of Experimental Setup

4.1 Introduction

There has been a lot of studies of Pile Soil Interaction and resultant capacity of Piles from it. However the soils in Pakistan has not been thoroughly studied and empirical design formula along with experience with different soils forms basis of design. The approach is experience based and derives from already conducted projects. Therefore as the experience increases, the design sophisticates. Another handicap with geotechnical Engineering students is no exposure to Piles. Piles are usually driven in Transportation Projects and students' exposure depends upon visit to Project sites. Academic Learning about Piles and its behavior is restricted to theoretical knowledge. Practical demonstration of Pile – soil behavior and resultant capacities is a field activity and Laboratories do not have sufficient equipment to simulate similar conditions. The change in Soil type and varying field conditions leaves the design process to the results of static load test. Prediction of Pile length for static pile load test is a question whose answer is yet to be determined for varying soil types of Pakistan. For this purpose, a model facility to simulate pile-soil interface under various conditions and using various soils needs to be constructed. Endeavors have been made in this regard in this project.

4.2 Soil Tank

A cylindrical Tank with 8mm thickness and 1.2 m diameter has been fabricated from local industrial setup. At the base, a plate, 10mm thick, is welded and mounted on wheels. Height of tank is 1m with a collar welded on top to bolt lid and other accessories. The cylindrical surface of tank is supported from outside using four channel sections, two of whom are extended upward to support reaction beam.

Figure 3: Pile Tank

Reaction beam is an H-Beam with 5 ½" wide flange placed on top of stiffeners and bolted. Mounted on H-Beam is manual jack to apply static load. Following features of Sand tank are worth mentioning.

4.2.1 Water Inlet/ Outlet

Water inlets/Outlets are provided on the cylindrical surface to introduce water saturation and varying head conditions. This water inlet/outlet can be used for both static water as well as flowing water conditions.

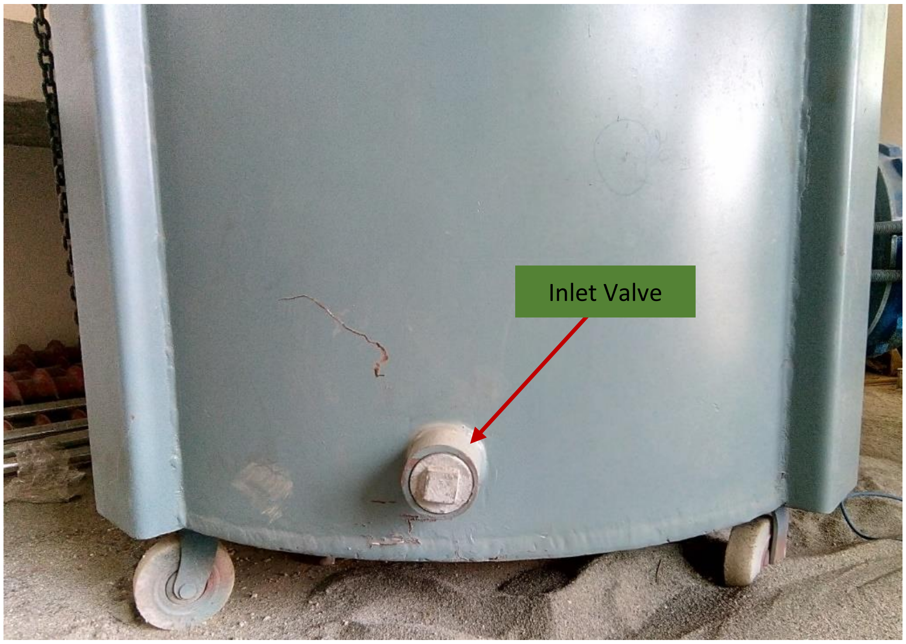


Figure 4: Water Inlet

4.2.2 Sand outlet valve

A 12” butterfly valve has been provided near the bottom of cylindrical surface. The purpose of valve is to extricate Sand for rapid reuse of the facility. The valve is particularly useful in case of saturated sands where extrication using shovels from top is many times difficult and time taking. A simple flowing water can extract the sand by opening the valve. This reduces the Tank reuse time by half.

Figure 5: Butterfly Valve

4.2.3 Bottom plate

A 10mm bottom plate is welded at the bottom and supported on four wheels. This wheels are holed on a specially made platform to avoid bending of the plate.

4.2.4 Tank Collar

A circular collar is welded on top of Pile Tank to Support Top plate and various other accessories such as guide mechanism. Tank collar as well as Top plate are cut to allow columns supporting reaction beams.

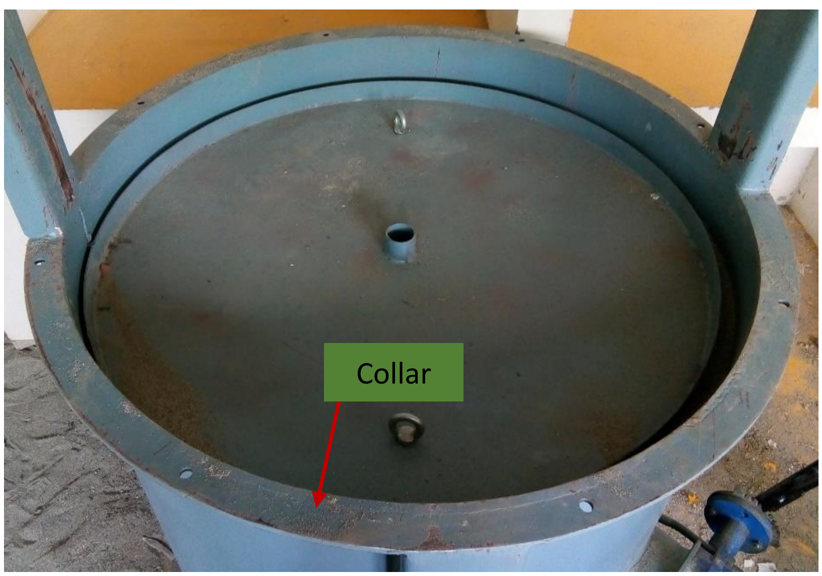


Figure 6: Pile Tank Collar

4.2.5 Stiffener and Columns

Four stiffeners are provided to cater for bending in the cylindrical surface. Two of the stiffeners are extend upward to support reaction beam. On these columns, ½’’ bolts are provided to for attachment of beam.

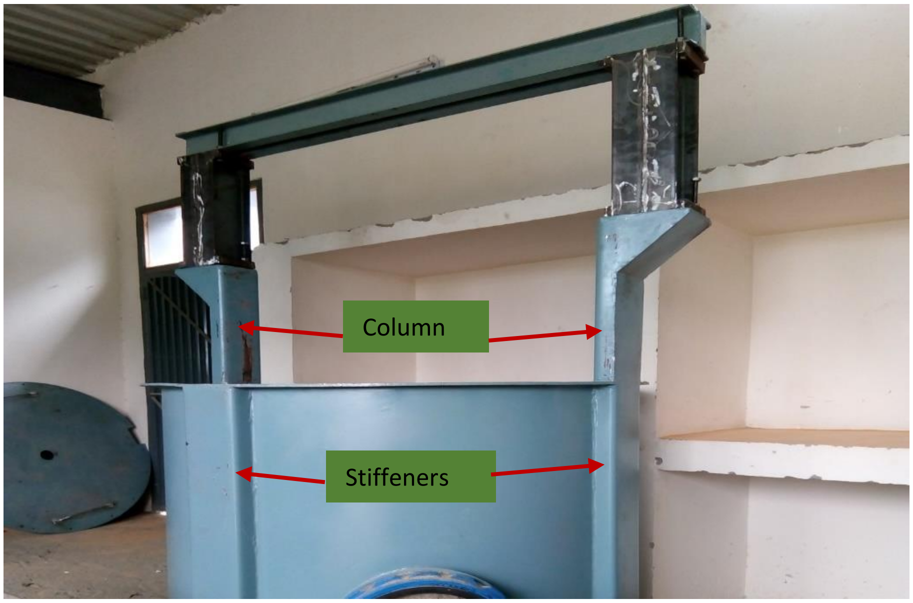


Figure 7: Reaction Beam

4.2.6 Stress distribution plate

A 10 mm thick and 1135 mm diameter circular plate with a 50mm diameter hole in the center is welded with two cylinders – a 10 cm long cylinder with 50 mm internal diameter welded at the central hole and a 10 cm long cylinder with 1135 mm diameter around the collar of the plate. Plate is provided with two threaded holes on center where i-hooks can be attached for further lifting using pulley. Purpose of stress distribution plate is to house pneumatic pressure system which simulate vertical effective stress condition.

Figure 8: Pressure Distribution Plate

4.2.7 Top plate

Top plate is of same diameter as collar and is provided with holes to house bolts for its tightening. Handles are provided on top side of the plate whereas bottom side is plain. Top plate acts as a reaction plate to pneumatic pressure system.

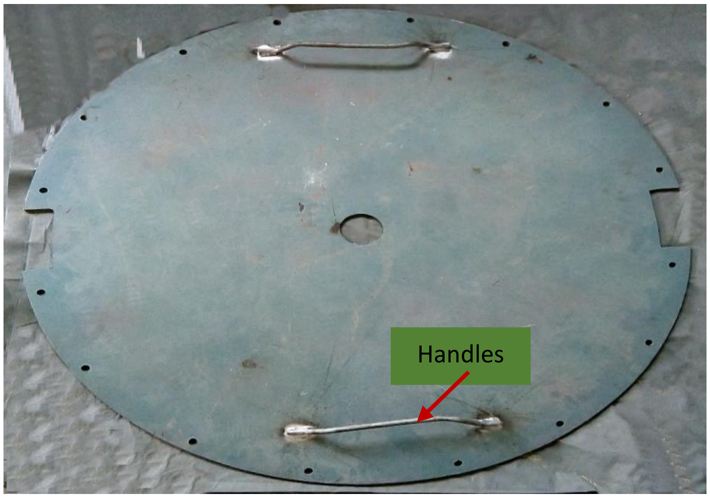


Figure 9: Top Plate

4.2.8 Reaction beam

Reaction beam is a H-Beam with flange width of 5 ½“. At the sides, beam flange is provided with holes in line with holes of the column. At the center, manual jacking mechanism is bolted on the bottom side of Beam.

Figure 10: Reaction Beam

4.3 Pneumatic Pressure system

It consists of pneumatic pump, along with regulator and a gauge. This apparatus is connected to a tyre tube placed in between top plate and pressure distribution plate. Inflation of tube in confinement causes downward push on the pressure distribution tube. Pressure distribution plate uniformly exert 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 connect the compressor, regulator and air tube. Airlines are provided with connectors on both end for easy use.

4.3.3 Pressure Gauge

A pressure gauge is used to measure air pressure in the 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.

Figure 11: Pneumatic System

4.4 Guide mechanism

Guide mechanism is provided and can be bolted to the Collar and top lid of Soil tank. Guide mechanism consist of a hollow rectangular section supporting two telescopic rectangular sections which in turn support the guide rail. Guide rail have clamps attached to it. The purpose of guide rail is to restrict horizontal movement of the model pile while allowing vertical movement during driving.

4.5 Sand Pluviator

As discussed earlier, sand is being rained to attain required density. Therefore a pluviator is designed as shown in figure. The pluviator consist of upper drum with lid bolted on bottom to form pan. The bottom lid consist of pattern of holes in rings at equal distance to each other. The hole diameter is 10mm. Below the lid is a plexi-glass shutter of 5mm thickness joined with a handle to rotate it to on and off position. The shutter consist of same pattern of hole therefore alignment of both the holes allow sand to rain. The drum is attached to two diffuser sieves (No.6 and No.10) which increase spread of the sand and align the bedding plane. Both sieves have mesh aligned at 45 degrees to each other. Arrangement have been made so as distance between the sieves and pan can be varied. Moreover, any individual component can be removed from pluviator. Pluviator is provided with hooks on all four sides to support hoist mechanism. Pluviator can be moved up and down using the hoist mechanism.

Figure 12: Pluviator

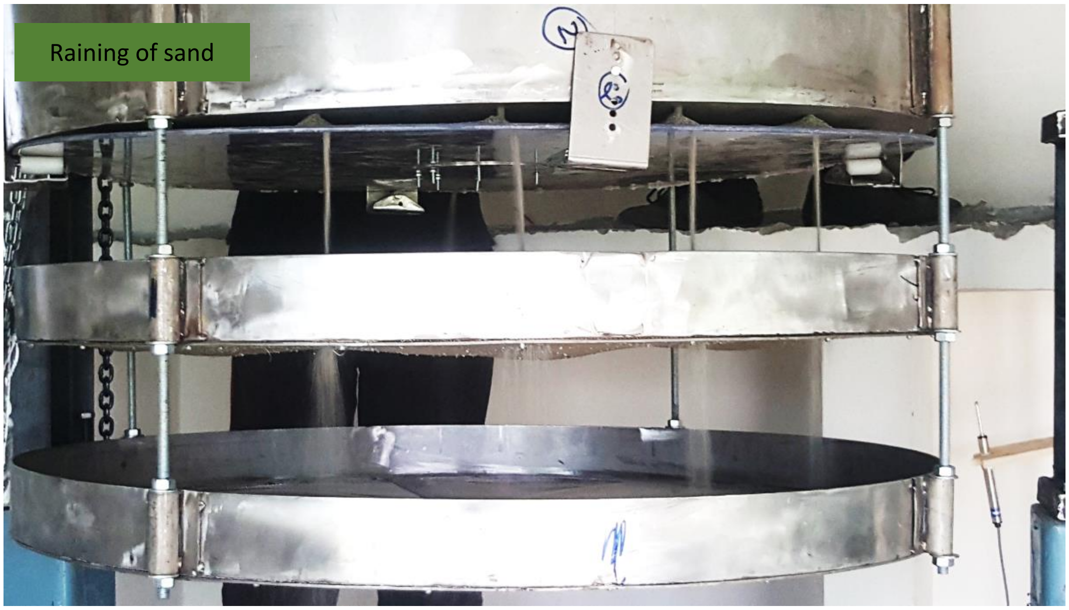


Figure 13: Pluviation

4.6 Hoist Mechanism

Hoist mechanism consist of a pulley block attached with a gantry crane mounted on a Steel Beam. The crane can move to and fro while pulley moves load up and down. The primary purpose of Hoist mechanism is movement of pluviator however other loads such as top plate, pressure distribution plate etc. are also moved using the same mechanism.

Figure 14: Hoist Mechanism

4.7 Instrumented model Pile

This consist of a pipe with 32mm external diameter and 2mm thickness. The pile consist of following accessories: -

4.7.1 32mm machined pipe

A 32 mm pipe was machined for adjustment of pile head on top and base load cell. Various slots for various sensors are provided on the external periphery of the pipe. The pipe is drilled with holes to allow signal wiring to be moved inside the pipe. A hole is provided near the top for exit of these wires. In total 32mm Pipe can support 6 strain gauges, a base load cell, a strain transducer and an accelerometer and a top load cell.

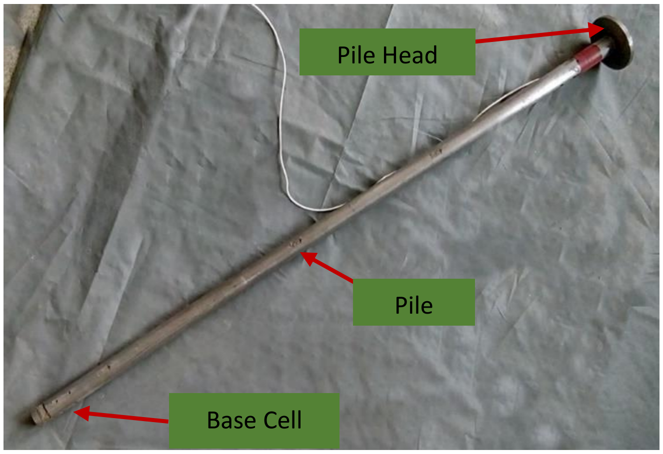


Figure 15: Model Pile

4.7.2 Base Load cell

Two different base load cell can be threaded to the bottom of the pile. One support Omega LCM-203 Load cell and other can be fitted with strain gauges to get strain based reading. A small machined calibrator housing is also provided for easy calibration of cell.

Figure 16: Bottom Load Cell

4.7.3 Pile head

A pile head can be screwed on top of the pipe. This pile head contain a threaded hole in the center to support guide rod for 5kg hammer. This rod can be removed to allow static loading of pile.

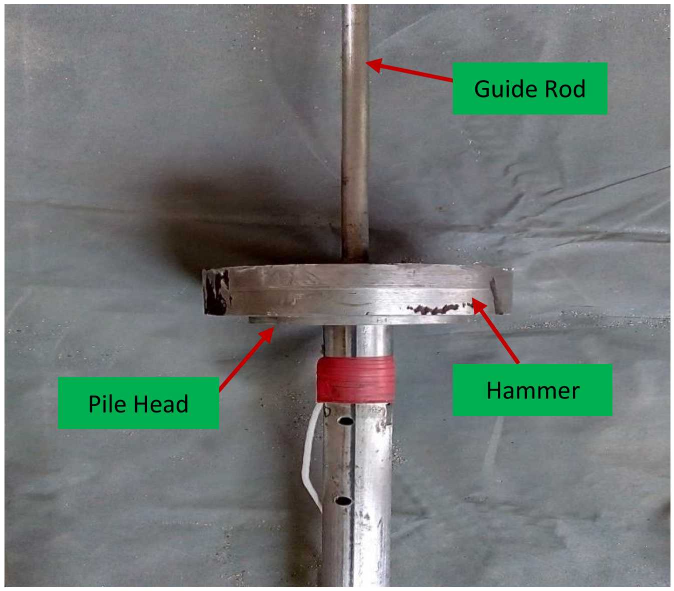


Figure 17: Dynamic Loading Mechanism

4.8 Data Acquisition System

Data Acquisition system is RJ45 based System 8000 Micro measurement Device. For this purpose a small board for pin connectivity is also manufactured. System 8000 Micro measurement device send data directly to CPU from where it can be exported using Micro-Measurement device software.

Figure 18: Data Acquisition

Chapter 5

Commissioning of facility

5.1 Introduction

To test the functionality of the facility, a series of Tests were conducted in the facility. The facility was erected in Soil Mechanics Laboratory. The hoist mechanism was put in place using an existing Steel H-beam in the building. Pneumatic Pressure system was installed and Model pile was sealed using silicone after installation of base load cell. Two samples were prepared using Sand – one with loose sand and one with Medium Dense Sand. Pile was installed using driving mechanism and both static and dynamic load test were conducted on the Sand Samples. These tests were used to calculate ultimate bearing capacity of Sand using Davisson, Chin, Decrout Methods (Static load test) and Case Method (Dynamic Load Test). The capacity was also calculated theoretically using method proposed by SALGADO.

5.2 Sand

Lawrencepur Sand was brought from a predesignated Quarry Site at Attock Road near Kamra. The Sand has earlier been investigated by Engr Amer Ahmad in his MS thesis under supervision of our co-Advisor Dr Kamran Akhtar. Consolidated Drained shear Strength tests were taken from work of Engr Amer Ahmad. Various Tests to determine Max and Minimum Dry Densities, Direct Shear Strength and Grain size distribution were carried out.

5.2.1 Max and Minimum Dry Densities

Test was conducted using vibratory table and Fol Calculations were made. Dried Sand was used for the experiment.

Mass of empty Cylinder	$=m_1$	$= 1055g$
Inner diameter of cylinder	$=d$	$= 15.21$ cm
Ht of cylinder	$=h$	$= 15.57$ cm
Volume	$=V$	$= \pi d^2 h/4 = 2830\ cm^3$

(1) Loose Sand

Mass of Cylinder +Sand	$=m_2$	$= 5134g$
Mass of Sand	$=m_3$	$= m_2 - m_1 = 4079$ g
Dry Density of loose Sand		$=\gamma_{dmin} = m_3 / V = 1.44 g/cm^3$

(2) Dense Sand

Mass of Cylinder +Sand	$=m_4$	$= 6007g$
Mass of Sand	$=m_5$	$= m_4 - m_1 = 4952$ g
Dry Density of loose Sand		$=\gamma_{d\ max} = m_5 / V = 1.75 g/cm^3$

5.2.2 Grain Size Distribution

Standard sieve set was utilized as per ASTM D2487. Test results are as following

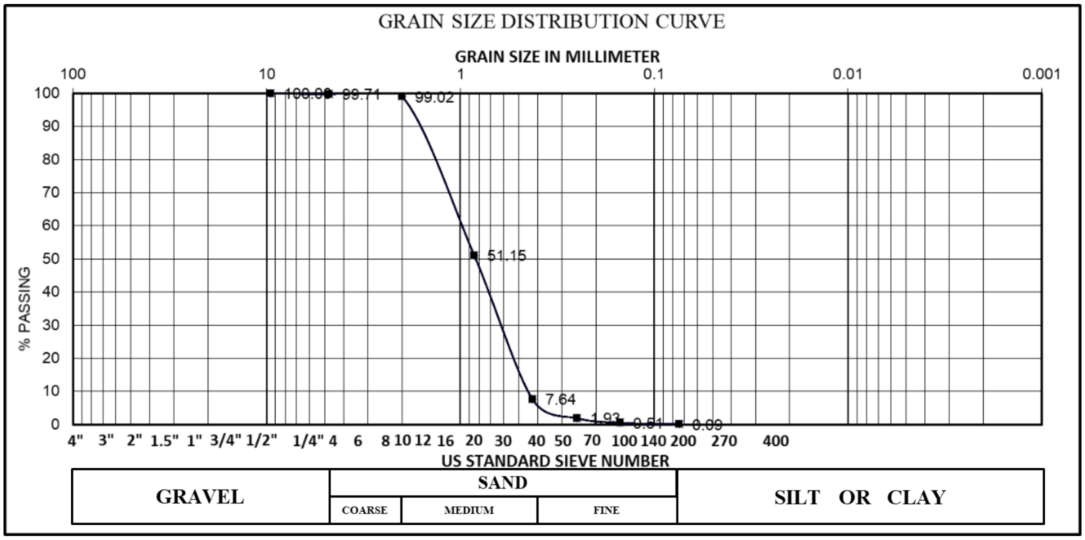


Figure 19: Grain size Distribution

Soil was classified as Poorly Graded Sand (SP) as per USCS classification with following Salient parameters.

Ser	Parameter	Value
1.	C_c	0.87
2.	C_u	2.36
3.	D_{60}	1.06
4.	D_{30}	0.84
5.	D_{10}	0.45

5.2.3 Direct Shear Test

Direct Shear Test was carried out in MCE Geotechnical Laboratory. The resultant critical friction Angle Φ_c came out as 30° under Normal Stress 170kPa.

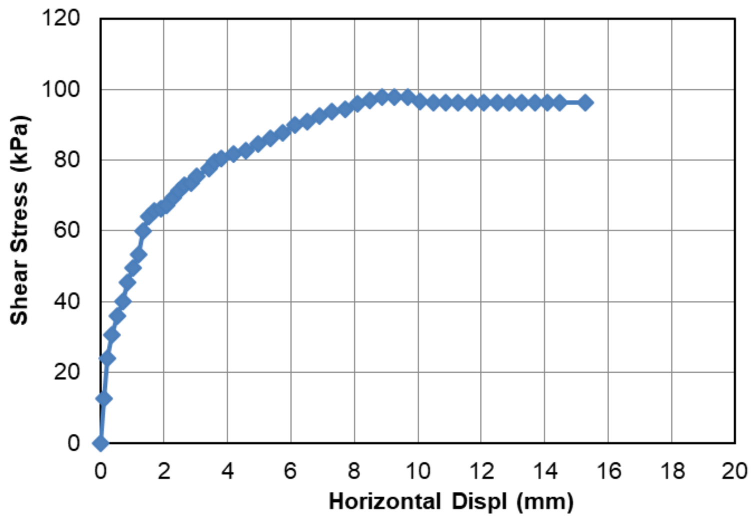


Figure 20: Direct Shear Test

5.2.4 Consolidated Drained Triaxial Tests

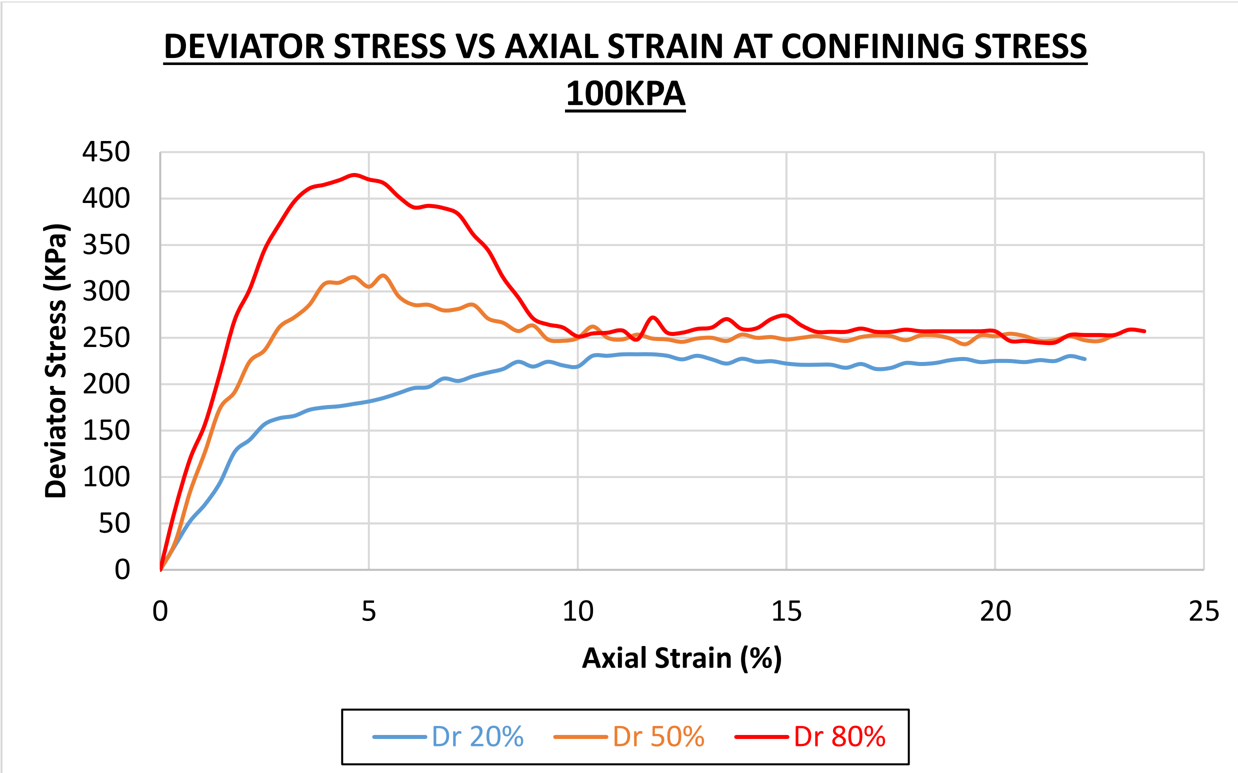
A series of tests were earlier conducted by Engr Amer Ahmad on same Quarry site. Following results were summarized from his work

		Relative Density			Relative Density		
		20	50	80	20	50	80
		Peak Sigma 1			Peak Friction Angle (Peak Phi)		
Confining Stress	100	332.180	416.911	525.333	32.44	37.75	42.79
	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	1870.670	2163.113	2554.419	30.90	34.39	38.22

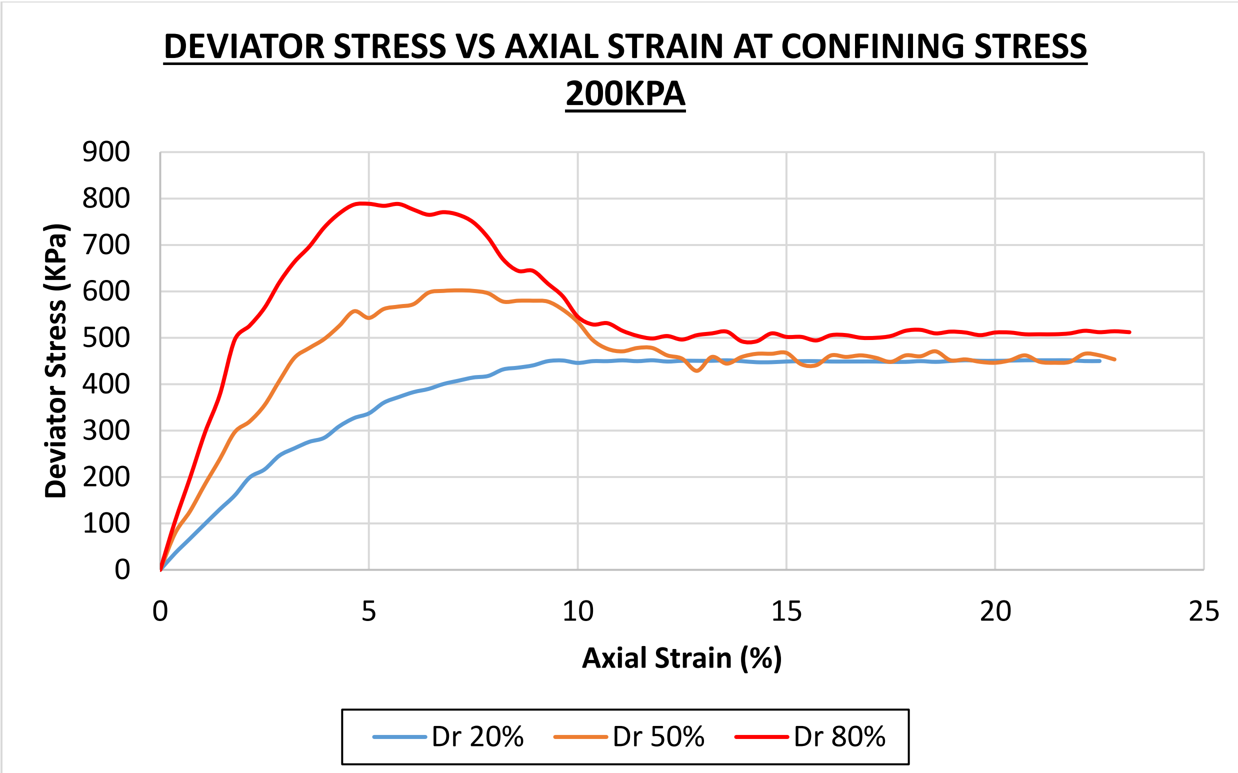
Table 6: Consolidated Drained Test

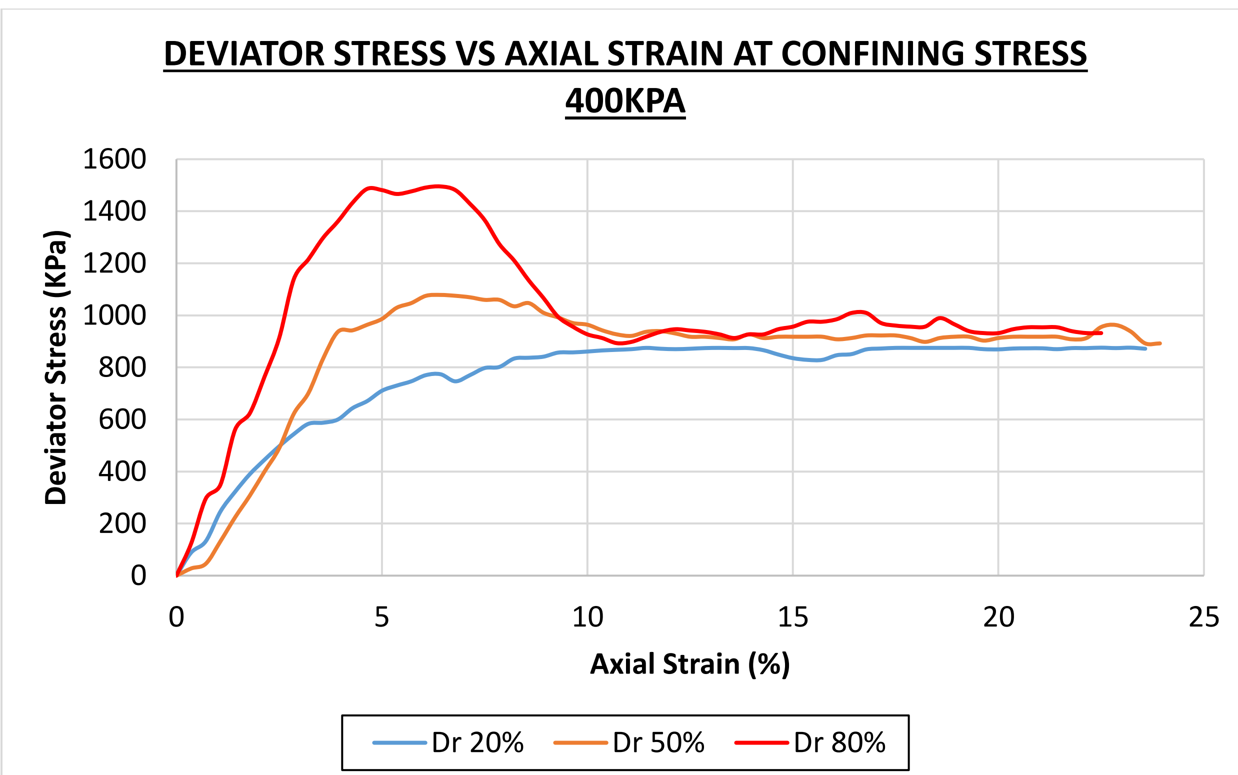
					Relative Density		
		Sigma 1 Critical	P Critical	Critical Phi	20	50	80
					(Peak - Critical)Friction Angle		
Confining Stress	100	338.4	179.486	32.9	0.14	5.45	10.49
	200	671.0	357.025	32.7	-0.33	4.57	9.20
	400	1310.3	703.438	32.1	-0.84	2.68	8.29
	600	1917.8	1039.283	31.5	-1.40	2.10	5.92

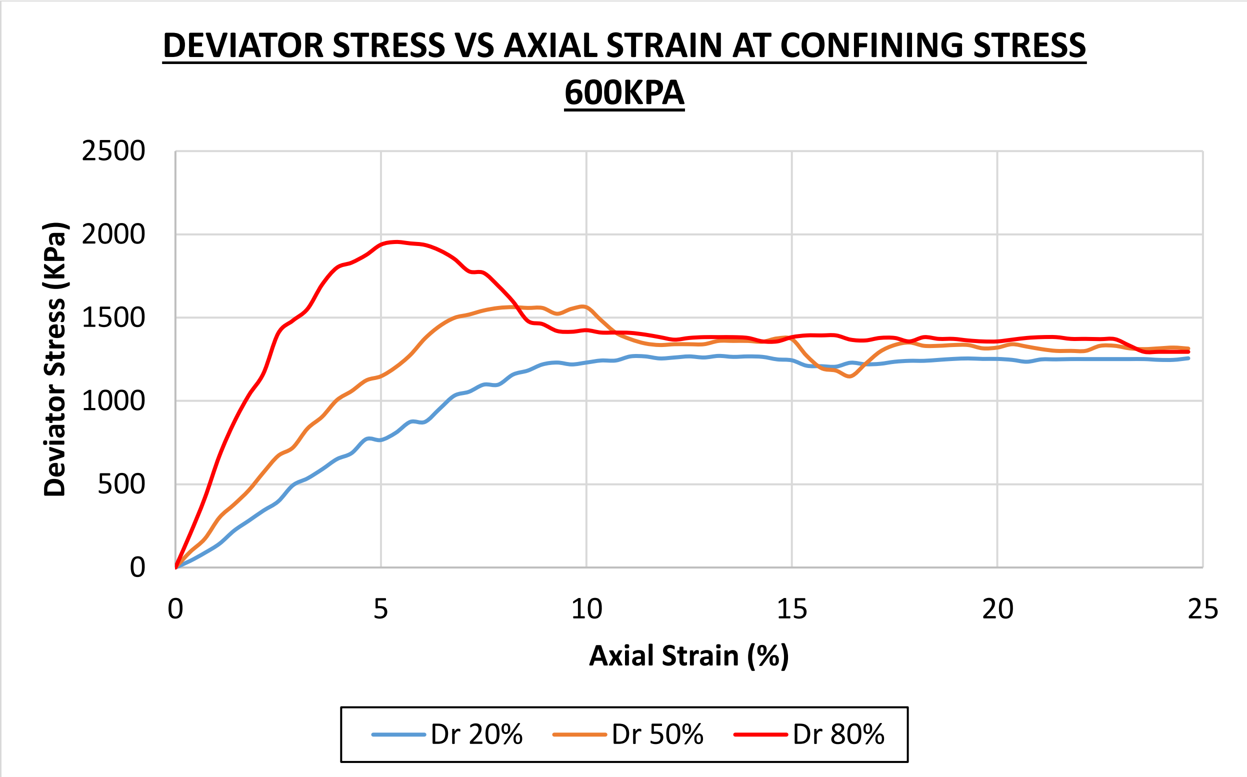
Table 7:Consolidated Drained Test



Triaxial Test


DEVIATOR STRESS VS AXIAL STRAIN AT CONFINING STRESS 200KPA
Deviator Stress (KPa)
900
800
700
600
500
400
300
200
100
0
0
5
10
15
20
25
Axial Strain (%)
Dr 20%
Dr 50%
Dr 80%


DEVIATOR STRESS VS AXIAL STRAIN AT CONFINING STRESS 400KPA
Deviator Stress (KPa)
1600
1400
1200
1000
800
600
400
200
0
0
5
10
15
20
25
Axial Strain (%)
Dr 20%
Dr 50%
Dr 80%



5.3 Functionality Test

Two samples were prepared – one with very loose Sand and one with Medium Dense Sand and Piles were driven into both. Both static and dynamic load test conducted on the sample. Following are the results of the test

5.3.1 Pile Driving Resistance

A graph was plotted for penetration vs Blows. A 5kg hammer was dropped from the height of .4 m, therefore, Energy of each blow comes out to be 19.6J. Following depicts the relation of loose and Medium Dense Sand

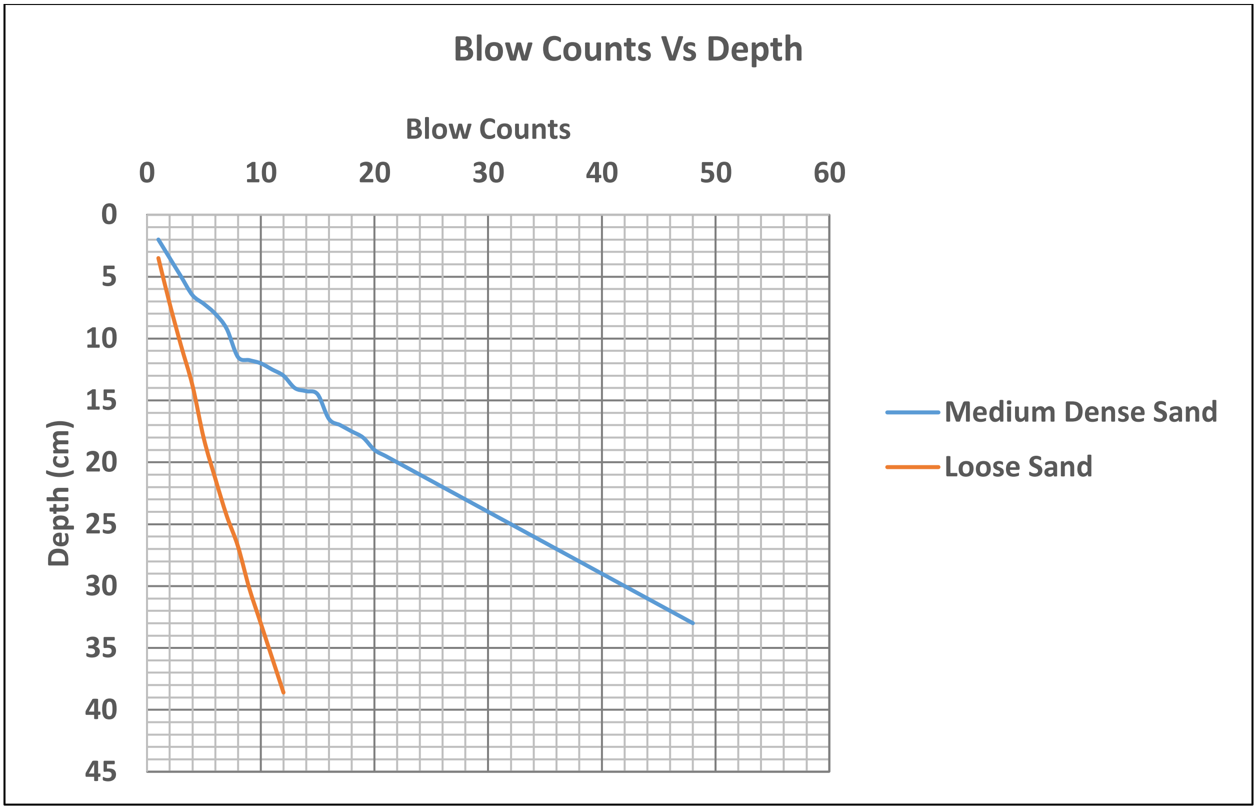


Figure 21: Pile Driving Resistance

5.3.2 Dynamic Load Test

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

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

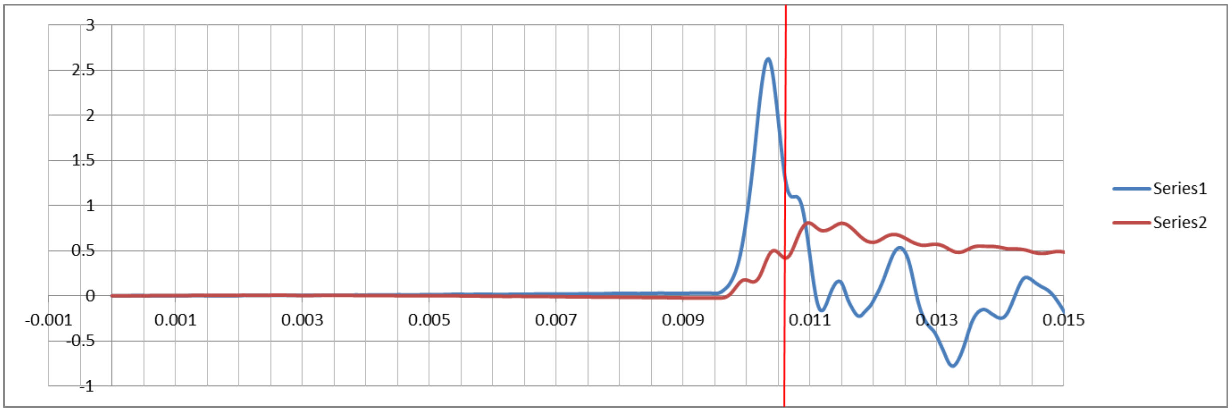


Figure 22: Dynamic Load Test –Dense Sand

$F|_{t_o+2L/c}$ = 8.197kN

$F|_{t_o}$ = 5.95kN

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

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

c = 5123 m/s

Jc = 0.35

Q_{ult} = 1.6kN

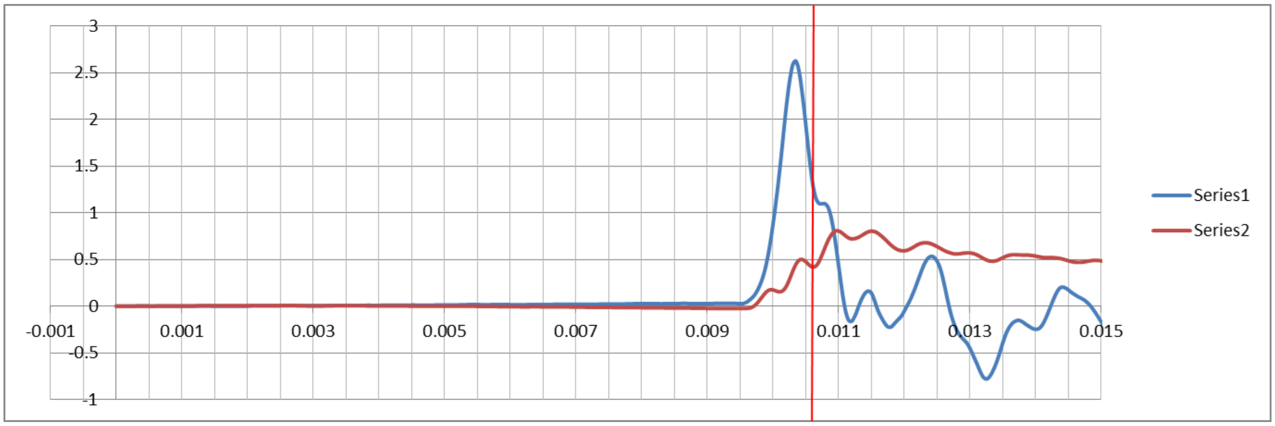


Figure 23: Dynamic Load Tesr- Loose Sand

$F|_{t_o+2L/C}$ = 0.6222 kN

$F|_{t_o}$ = 0.623 kN

$v|_{t_o+2L/C}$ = 0.0196 m/s

$v|_{t_o}$ = 0.021 m/s

c = 5123 m/s

Jc = 0.35

Q_{ult} = 0.14kN

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

Table 8: Static Load Test

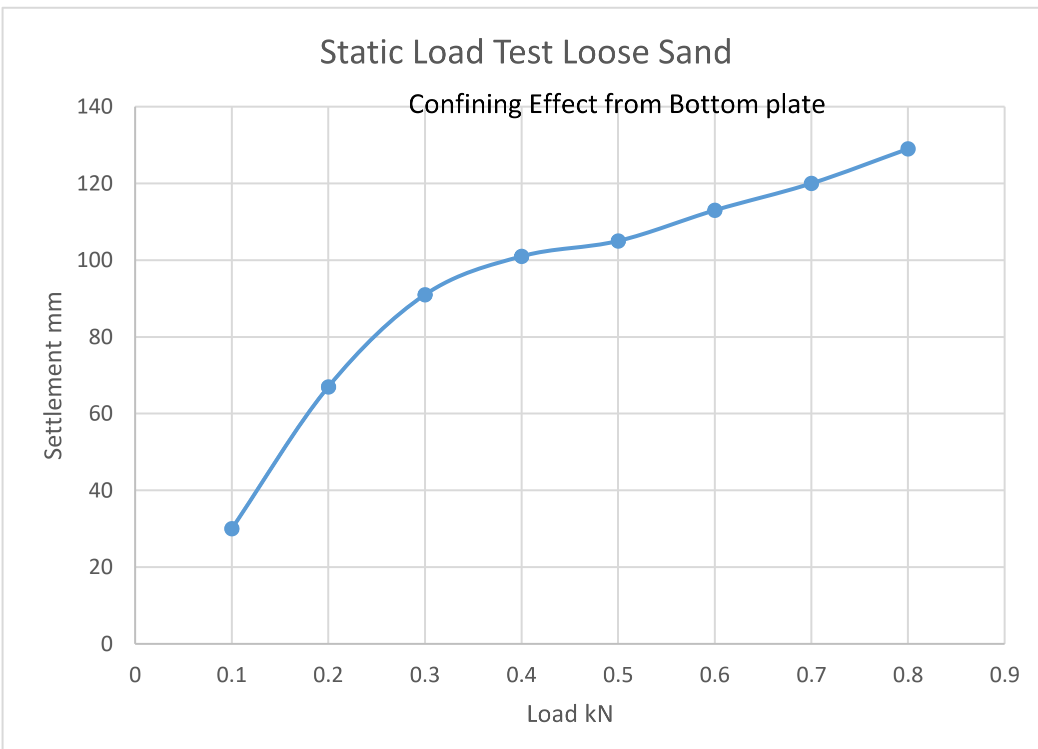


Figure 24: Static Load Test- Loose Sand

Ultimate Capacity was found out using 3 different criteria's namely Davisson, Chin and decrout's Method. The capacity varied between 0.27 to .43 kN. Following graphs depict the analysis.

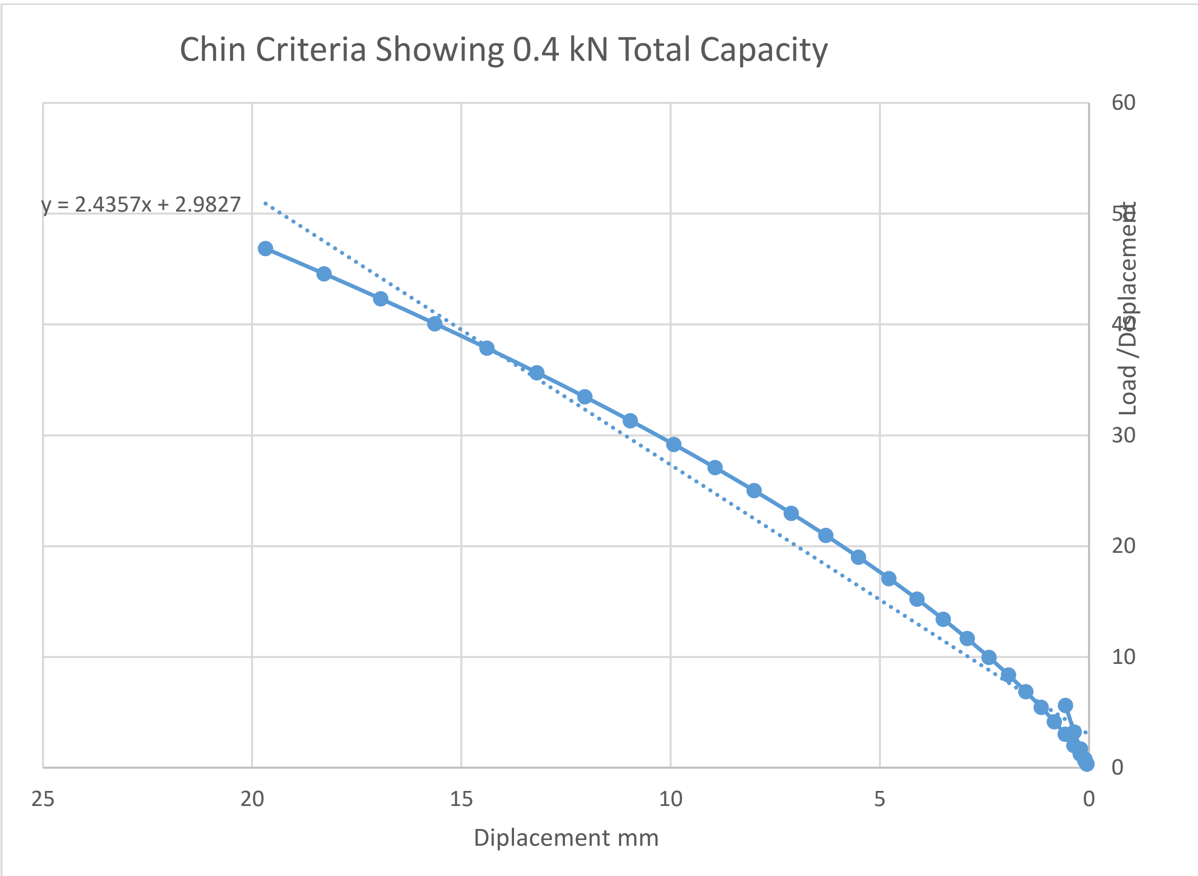


Figure 25: Chin's Method

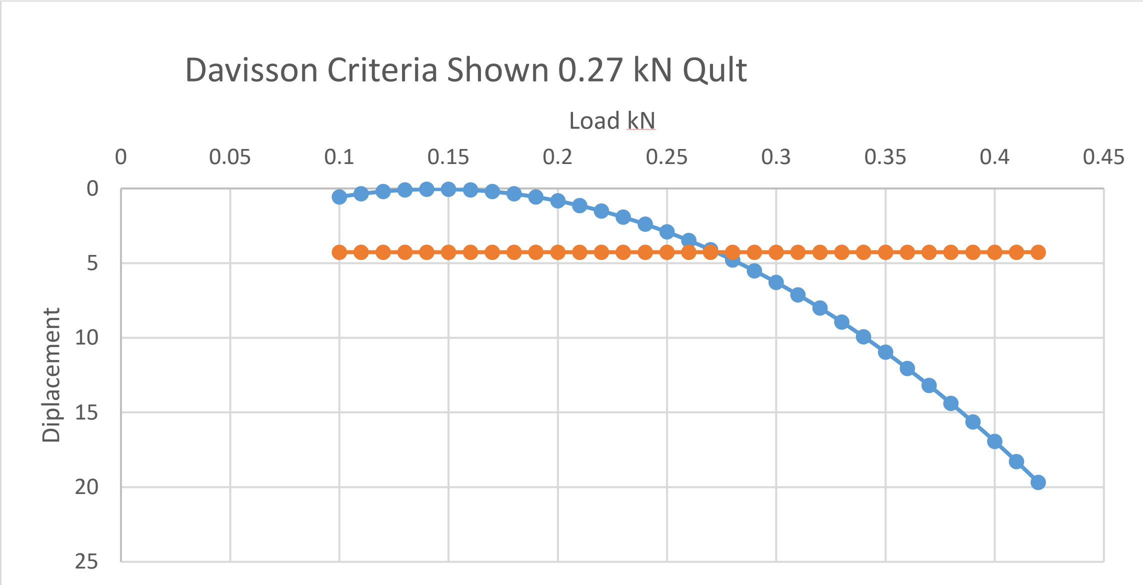


Figure 26: Davisson Method

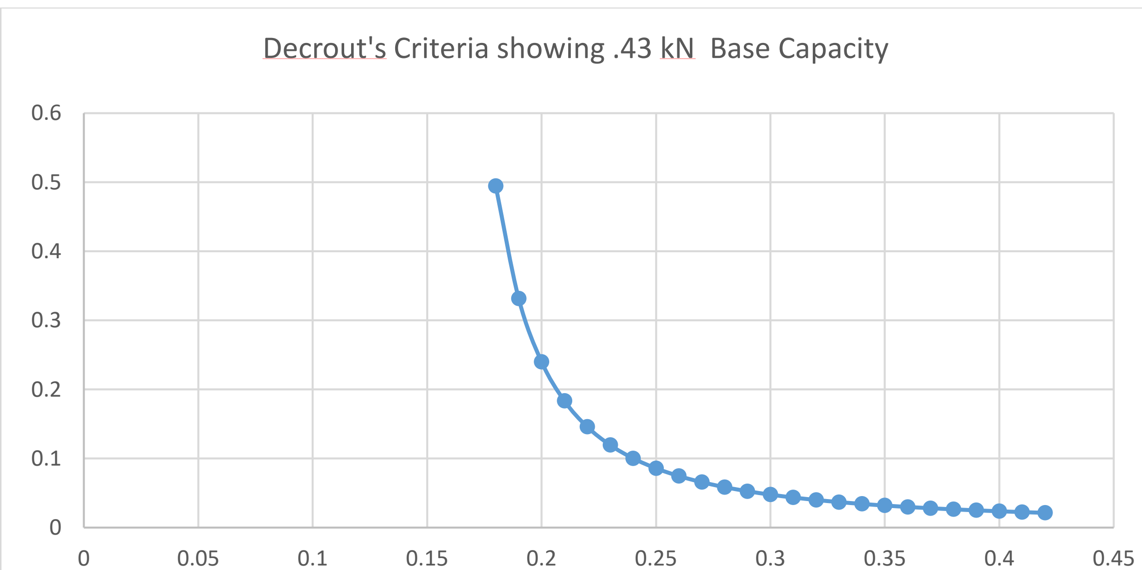


Figure 27: Decrout's Method

5.3.4 Theoretical Calculations

Following equations as suggested by Randolph (2003), Salgado et al.(2004) Foyes et al and Salgado and Prezzi (2006) were used to calculate Ultimate capacity of Piles theoretically

$$q_{sL} = 0.02 \tan\delta\, [1.02 - \ 0.0051\ D_R]\ q_{bL}$$

$$\frac{\delta}{\phi_c} = \ 0.85\ for\ steel\ Pile$$

$$\frac{q_{sL}}{P_A} = 1.64\ exp(0.1041\ \phi_c\ + (0.0264 - 0.0002\ \phi_c)\ D_R)\left(\frac{\sigma'_h}{P_A}\right)^{0.841-0.0047\ D_R}$$

Length	Phi C	δ	DR	Unit wt		Vert Eff Stress	Horz Eff Stress
0.6	30	25.5	15	16.5		4.95	3.7125
qbl	qsL	Qbl	Qsl	Qult	Area		
399.5073	3.595777	0.107101	0.060319	0.16742	0.000804		

Table 9: Loose Sand (15% D_R)

Length	Phi C	δ	DR	Unit wt		Vert Eff Stress	Horz Eff Stress
0.6	30	25.5	55	16.5		4.95	3.7125
qbl	qsL	Qbl	Qsl	Qult	Area		
1681.218	11.8601	0.450705	0.060319	0.511024	0.000804		

Table 10: Medium Dense Sand

The differences in theoretical and practical values are in fact due to the unexperienced handling of pile after being driven.

Chapter 6

Conclusion

Model Pile Testing facility is first of its kind facility in Pakistan. It provides a way forward into scientific investigation in Behavior of soil under loading and an insight into Soil-Foundation interface. It has following capabilities

6.1 Capabilities of Model Pile Testing Facility

(1) It can simulate Model Pile – Soil behavior in the laboratory.

(2) Static and Dynamic Load tests can be carried out in this facility.

(3) Simulation of different confining stress can be carried out in the laboratory

(4) Study of Pile – Soil behavior can be carried out in dry and wet conditions.

(5) Preparation of homogenous samples of varying densities.

(6) Preparation of samples with different soils collected from field.

6.2 Applications of Model Pile Testing Facility

6.2.1 Academic Value

Following are academic benefits of the facilty:

(1) Depiction of Pile behavior in Lab

(2) Understanding of Effect of Different soil strength parameters on behavior of Piles.

(3) Simulating various field conditions in the laboratory and study their effect on Pile capacity

(4) Pile capacity prediction by students in Laboratory.

6.2.2 Research Value

Following researches can be carried out on soils of Pakistan

(1) Study of various understudied soils and effects of their behavior on Piles

(2) Establishment of Correlations of Different parameters of soils with field tests such as dilatometer and CPT

(3) Study of effect of Saturation of soils on capacity of Piles

(4) Correlation between Dynamic and Static Load Test in different soils in Pakistan

(5) Study of Effect of Installation methods on Pile capacity

6.2.3 Proposed future development

Following development are recommended for enhancement of capacilities of the facility

(1) Jacking system to be installed for Jacking of piles

(2) Larger Bladder to be installed in pneumatic system for Effective stress simulation

(3) Pressure actuator to be installed with pneumatic system

(4) Pile Group testing

(5) Lateral and combined loading mechanism for piles to be installed

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

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