“Effect of Various density of Sand on the Pullout Capacity Of Model Steel Pile”

A

DISSSERTATION

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In

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SANDEEP RAJPUT

(Roll No. 2K19/GTE/11)

Under the Guidance of

PROF. NARESH KUMAR

Department of Civil Engineering

DELHI TECHNOLOGICAL UNIVERSITY

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Date: 15/7/2021 SANDEEP RAJPUT

Roll No. 2K19/GTE/11

CERTIFICATE

This is to certify that major project report entitled "**Effect of various density of sand on the pullout capacity of model steel pile"** is an authentic record of my own work carried out in fulfillment of the requirements for the award of Master of Engineering (Geotechnical Engineering), department of Civil Engineering, Delhi Technological University, Delhi under the guidance of Prof. NARESH KUMAR

Dated: 15/7/2021

SANDEEP RAJPUT
Roll no: 2K19/GTE/11

It is certified that the above statement made by the student is correct to the best of my knowledge and belief.

Prof. NARESH KUMAR
Civil Engineering Department,
Delhi Technological University, Delhi

ABSTRACT

In today's urban planning and regeneration, tall structures play a critical role. The design and assessment of piling foundations are complicated by the development of high-rise buildings. The lateral loads on the piles because of the eccentricity of wind impact on vertical projections of multistory tall buildings are of particular significance among these combinations of vertical forces.

Because of various elements that influence foundation behavior, engineers face a difficult task in designing and analyzing pile foundations. Such characteristics include the method of loading, soil attributes, pile shape, and construction method. Magnitude of superstructure loads will influence the pile foundation that is chosen to withstand the imposed loads. IS code 2911 is used.

Understanding pile behavior and forecasting pile capacity under uplift loading are critical foundation design subjects. Cohesion-free soil was used in the experimental model tests, which were subjected to pure uplift loading. The experimental test was conducted on solid straight vertical steel pile with diameter of 0.03175m and length of pile is 0.9m. The sand bed is prepared in a variety of density conditions, including medium and dense sand. Singles pile is embedded in sand was tested the results are presented and discussed in this thesis. The influence of density of soil mass, the pullout capacity of steel pile is calculated. The study of revealed that the behavior of single pile under uplift loading depends on the various density of soil mass. The experimental findings given in this thesis are expected to aid professional understanding of the soil-pile-uplift interaction problem.

Therefore, objectives of the present work are:

(1) Mechanism of load applying on the model steel pile.

(2) Experimentally and theoretically to determine the pull out capacity of model steel pile.

(3) Validation of experimental pullout capacity with theoretically pullout capacity.

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CHAPTER 1
INTRODUCTION

1.1 Introduction

The superstructure requires some support in order to transfer loads and moments to the underlying soil or rock; this is where foundations, sometimes known as substructures because they are located below ground level, come into play. As a result, the superstructures are effectively supported by the foundation framework.

When the soil at shallow depths from the ground level is insufficient to sustain shallow foundations such as footings or rafts, deep foundations are required.

1.2 General

Piles are structural foundation elements that are vertical or slightly slanting and have a modest cross-sectional dimension in terms of their length. They are embedded in the soil and transmit loads and stresses from the superstructure to the subsoil. The length, method of placement, and action of heaps can all vary significantly, making them easily adaptable to a variety of conditions. Piling has come a long way, not just over thousands of years of human history, but also during the technology age and even before soil mechanics was established. Giant piles with a length of 50 to 100meter and a diameter of 3 to 5 meter have replaced the 10 to 20 m long and 0.2 to 0.5 m diameter piles of the past. The application of piles has become more diverse and sophisticated than it has ever been **(Fang, 1975).**

Steel is increasingly being utilized for piling because of its ease of manufacture and handling, as well as its capacity to endure heavy driving. Corrosion problems in marine constructions have been solved because to the adoption of long-lasting coatings and catholic protection.

Piles can be used for a variety of purposes, including:

1. Using end bearings piles to transfer loads through water or soft soil to a suitable bearing stratum (point-bearing piles).
2. Using "skin friction" throughout the length of the piles to transmit weights to a depth of relatively weak soil (friction piles).
3. Increase the bearing capacity of granular soils by compacting them (compaction piles).

4. To offer safety in the event that the soil is eroded away, the foundation must be carried into the depths of scour.
5. To secure structures that is prone to uplift owing to hydrostatic pressure or overturning (tension piles or uplift piles).
6. In the face of sheet piling walls or other pulling forces, provide horizontal anchorage (anchor piles).
7. To prevent ships or other floating items from colliding with structures at the water's edge (fender piles and dolphins).
8. Forces that are horizontal or slanted must be resisted (batter piles).

1.3 Classification of pile

1.3.1 Types of Piles Based on Construction Materials.

1. Concrete Piles
2. Steel Piles
3. Timber Piles
4. Composite Piles

Concrete pile

Concrete can be adjusted to fit a variety of pile kinds. It can be used as insertion units in bore piles or as precast units in driven piles Dense, well-compacted, and high-quality concrete can resist moderate driving and is resistant to attack by hostile elements in the soil, as well as in seawater or ground water. Concrete, on the other hand, performs well in difficult driving situations.

Steel pile

Steel piles are more expensive than lumber or concrete, but their ease of handling, ability to endure forceful driving, bending resilience and strength, and ability to hold tremendous weights may offset this disadvantage. Steel piles may be pushed over long distances with little disturbance to the earth. If long steel piles with narrow sections depart from their actual alignment during driving, they may buckle and cause damage.

Timber pile

Untreated timber piles can be used for temporary construction, revetments, fenders, and other similar projects, as well as permanent construction, when the pile's cutoff elevation is below the permanent ground water table and the piles are not exposed to marine borers. Trench construction can also be done with them, however treated piles are recommended. Timber piles are difficult to extend, anchor into the footing for uplift resistance, and are susceptible to damage if not driven carefully. Timber piles have a maximum bearing capacity of 45 tones, although most construction piles are designed to sustain at least 70 tones.

Composite piles

In piles, different materials can be combined, with steel and concrete being the most prominent example. This could be accomplished by using several forms of driven steel casings filled with a concrete structural core, or a steel pile covered from the outside by concrete casing.

1.3.2 Types of pile Based on Construction Method

- Displacement piles.
- Non-displacement piles.

Displacement piles

The earth is shifted downwards and sideways as a pile is pressed into the ground, but no material is actually removed. Displacement piles are piles that are inserted in this manner.

Non-displacement piles

To form a pile, a shaft (or hole) is excavated and the dirt replaced with concrete. A replacement pile, also known as a non-displacement pile, is one of many types of piles.

1.3.3 Classification of pile based on Functional Behavior

- End bearing piles (point bearing piles)
- Friction piles
- Friction and End bearing piles

End bearing piles

The majority of these piles' carrying capacity comes to the soil's penetration resistance at the pile's toe, and they transfer their weight on the hard stratum situated at long distance below the structure's base. Because the pile functions in the same way as a typical column, it should be built in the same way. Even on weak soil, a pile will not buckle and this impact should be considered only when a component of the pile is unsupported, such as in air or water. To transmit load to the earth, friction or cohesion is used. The soil around the pile, on the other hand, may adhere to its surface, causing "Negative Skin Friction." On rare cases, this might have a major impact on the pile's capacity. Groundwater drainage and soil consolidation cause negative skin friction. The results of the site research and soil test have an impact on the pile's foundation depth.

Friction and cohesion piles

Skin friction transfers the majority of the weight to the ground (cohesion piles). The porosity and compressibility of the soil within and surrounding piles is greatly reduced when they are driven close together in groups. As a result, the term "compaction heaps" is sometimes used to refer to piles in this category. When the pile is driven into the ground, the soil is deformed, and some of its strength is lost as a result. As a result, the pile is unable to transfer the exact amount of load that it was designed to transport in a timely manner. The soil normally regains some strength three to five months after being driven.

(Friction piles) Skin friction is also used to transport the weight of these piles to the earth. The procedure of driving such piles does not appreciably compact the earth. A floating pile foundation is the most common type of pile foundation.

1.3.4 Preference of steel piles over other types of piles

(a) Steel piles can be made as long as you want. The maximum length of cast-in-place concrete piles (shells pressed without a ring) is 10 to 25 meters, and the maximum length of cast-in-place concrete piles (shells removed) is 36 meters, whereas timber piles have a maximum length of 35meters.

(b) A steel pile should be between 12 and 50 meters long. The maximum length of cast-in-place concrete piles (shells pressed without a mandrel) is 9 to 25 meters, and The maximum length of cast-in-place concrete piles (shells removed) is 8 to 12 meters, but timber piles have a maximum length of 35 meters.

(c) As a result, the maximum load Steel piles have a substantially larger carrying capacity (maximum permitted stress, cross section) than other types of piles. Steel Pile provides the following advantages over other types of piles: **(Tang, 1969).**

- Splicing is simple.
- It has a high storage capacity.
- Minor displacement
- It has the ability to pass through light barriers.

1.4 Mechanics of Load Transfer through Piles

End-bearing piles and friction piles are the two types of piles. This is the classification that is most relevant to the topic of pile geotechnical design. If the soil near the surface isn't strong enough to maintain the load, a point-bearing pile is used to transfer the load to the firm soil at the point of support. If the soil condition is such that soils capable of supplying some shearing resistance at the interface with the pile are accessible to an appropriate depth, piles are used to transmit the applied load to the surrounding soil via skin resistance mobilized over the entire surface area. To transfer the imposed load to the surrounding soil, skin resistance is mobilized along the entire surface area of piles. Shearing resistance will be in the form of skin friction if the soil is cohesion less. In the event of such a soil, the unit value of skin friction (at depth h) $K.\gamma.h.\mu$ where K is the earth pressure coefficient and μ is the friction coefficient, $\mu = \tan\delta$, where δ called the angle of wall friction.

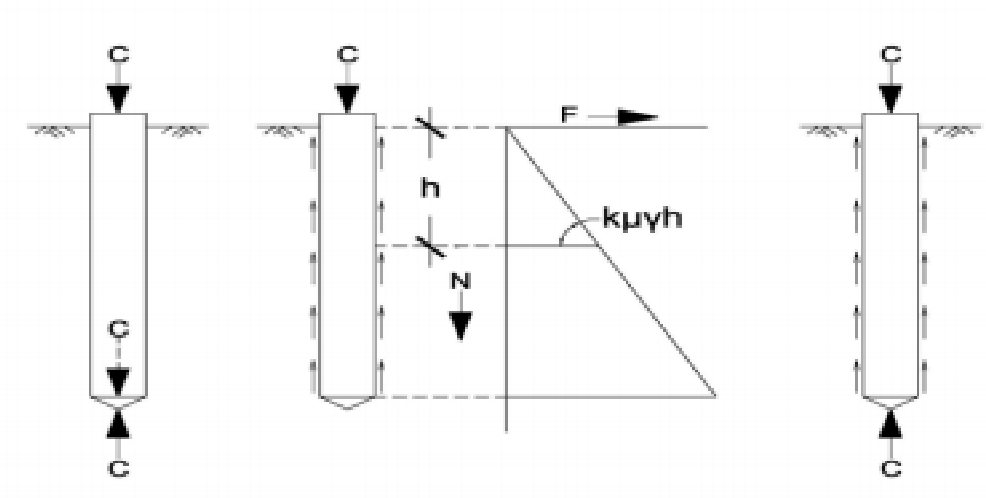


Fig 1.0: pile subjected to compressive load

1.5 Skin friction

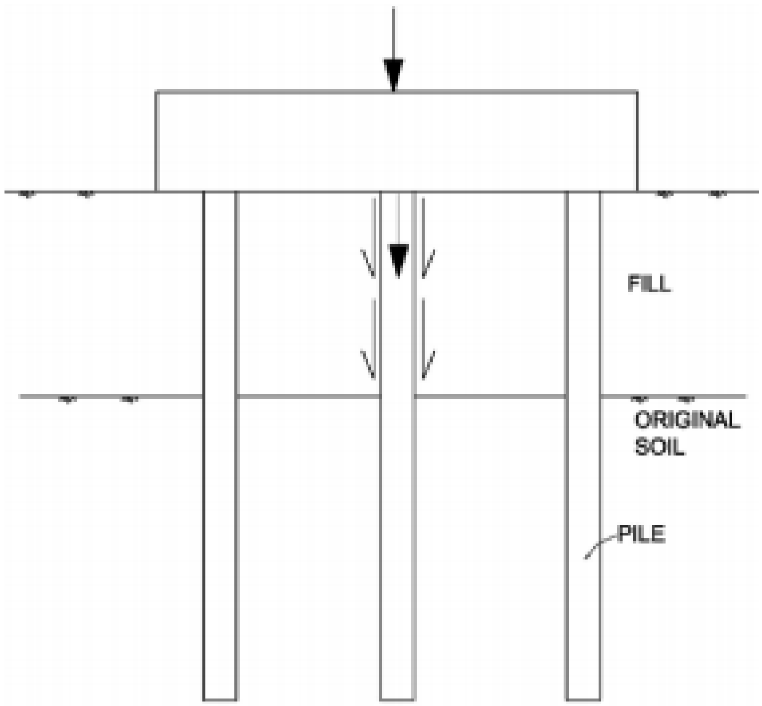
The unit value of ultimate skin friction may be taken as βc, where β is a reduction coefficient' whose value falls around 0.45, and c is the unconfined shear strength, half the unconfined compressive strength, in the situation of a pier when the applied load is totally resisted by the skin friction.

1.6 Negative skin friction

The fill seen below the original soil where the pile group is built will settle under its own weight over time. Because of the friction at the interface between the pile and the soil, this will cause the pile to drag. Since this down drag ads to the load on the pile, instead of resisting it, it is called 'negative skin friction'. Because it is an additional load, it must be factored into the design. **(Fellenius, 1998)**

As a result of the consolidation of the strata after piles are set, piles installed in freshly deposited fills of soft compressible deposits are susceptible to downward pull. When the earth moves down relative to the pile, the downward pull on the pile surface contributes to the structural loads and is referred to as negative skin friction. This is in contrast to the normal shaft friction that occurs when the pile travels down into the soil. As a result, negative skin friction reduces the allowed load on the pile. If the fill material is a loose sand deposit, negative skin friction may arise.

It can also happen as a result of a drop in the ground water table, which increases effective stress and causes soil consolidation, resulting in pile down drag **(Poulos, 1998).**



(Fig1.1: Negative skin friction)

1.7 Piles in sand

1.7.1 Point-bearing piles in sand

Point-bearing piles are utilized in sand to transfer loads to a dense sand layer beneath weak deposits. Depending on the relative density of the sand, such piles must be driven to the depths required for acceptable bearing. The easiest way to predict the weight carrying capacity of such piles is to conduct load testing. In the case of these piles, determining the load bearing capacity using an analysis based on the wave equation is equally reliable.

1.7.2 Friction piles in sand

The maximum frictional resistance available to resist the load on pile in sand can be taken as $(\pi/2)*d*K\gamma L^2 \tan\delta$, where K is the earth pressure coefficient. The expression's uncertainty is limited to the value of K, which is determined by the relative density of the sand surrounding the pile. K ranges from 1 for loose sand to more than 3 for thick sand. The angle of wall friction ∂ should be chosen based on the pile's material and other factors such as whether the pile's driving has been aided by jetting.

1.8 Objective of the study

(1) To determine the ultimate load failure for single pile.

(2) To perform the pullout load test for various density, and determine the ultimate pullout capacity or uplift capacity of single pile.

(3) Drawing the load pullout curves for different loads.

(4) Compare the results of experimental work with theoretical results.

1.9 Organization of Thesis

(1) There are five chapters in all in this thesis. The need/objective of the study pile and pile foundation is discussed in the first chapter, which is titled Introduction. There are also several types of pile load tests, such as dynamic and static equations. The tests were carried out in order to calculate the pile load capacity, and an outline is provided at the end.

(2) The second chapter, titled Literature Evaluation, is a brief but critical review of the work of a few key investigators. The piling foundation system, as well as IS 2911(Part IV) and ASTM codal regulations, have been briefly discussed.

(3) The laboratory tests that are undertaken before the pile load test are covered in the third chapter, experimental Program. It also provides the experimental setup used to conduct the pullout test as well as the results of the pullout test.

(4) The fourth chapter is titled Discussion on Results, and it contains the experimental results of the single pile load test.

(5) The conclusion of this thesis paper is in the fifth chapter.

CHAPTER 2

LITERATURE REVIEW

When the soil at shallow depths from the ground level is insufficient to sustain shallow foundations such as footings or rafts, deep foundations are required. The 'pile,' 'pier,' and 'caisson' are the three most common types of deep foundations. While a precast concrete pile's cross section might be square, octagonal, or circular, cast-in-situ piles and piers always have a circular cross section. A caisson's cross section can be any shape because it is basically prefabricated. The fundamental difference between the three types of deep foundations is the cross sectional dimensions in proportion to depth. Thus, although a pile's cross sectional width is normally around 300 mm, a pier's is around a meter, and a caisson's is several meters. Because of this distinction, although a pile bends under a horizontal force, caissons and piers with enormous diameters basically rotate as rigid bodies under the same load. **(bowles, 1970).**

The growing importance of pile foundations is reflected in the large number of contributions to international conferences and other publications dealing with various elements of pile foundations; monographs and state-of-the-art reports published in recent years provide essential information.

This cutting-edge and quickly evolving foundation system is also one of the oldest. This technology has been utilized in river valleys and floodplains with unpredictable soil conditions since prehistoric times; pile-dwellings can be discovered in the beginnings of many cultures. Well-grown pine trees are a good material for this purpose. From these humble beginnings, today's variety has grown, with a piling system for nearly every type of foundation.

Piling has come a long way, not just over thousands of years of human history, but also during the technology age and even before soil mechanics was established. The application of piles has become more diverse and sophisticated than it has ever been. Construction methods have evolved rapidly; in the past, timber piles and hand-powered rammers were used; now,

incredibly intricate machinery and highly specialized processes are used **(fang, 1975).** The usage of piles to overcome foundation concerns is most common in the following situations:
1. A reliable bearing capacity soil layer can only be obtained at a deeper depth.

2. Scours may occur if the layers directly beneath the structure are washed out.

3. For constructions that are capable of transferring particularly high and/or horizontal loads.
4. For structures that are extremely vulnerable to settlements.

5. for the construction of offshore structures.

In certain circumstances, piles are just used to improve the bearing capacity of the surrounding soil and do not directly carry the structure's load.

Pile foundations are designed to withstand compressive loads from superstructures in general. Transmission towers, tall chimneys, jetty superstructures, and mooring systems for ocean surface or submerged platforms, for example, need pile foundations to withstand massive uplift loads. A plethora of thorough theoretical and experimental investigations are available. The behavior of piles or pile groups subjected to axial, compressive, inclined, or lateral loads was studied for nearly 10 years. Research on the capability of uplift, on the other hand, is ongoing. There is a limit to the amount of pile load that can be carried. The uplift resistance of a straight shafted pile in sand is thought to be completely reliant on skin. Friction is created as the pile shaft grinds against the dirt. The net outcome is often determined using a limiting friction approach. In sand, the uplift capacity Pnu(NET) of a vertical circular pile with a diameter of d and an embedded depth of L, formulated as

$$\mathrm{P_{nu}} = \mathrm{p_{av}} \times \pi \times \mathrm{d} \times \mathrm{L} = (0.5\mathrm{K_s}\tan\delta \times \gamma\mathrm{L}) + \mathrm{W}$$

Where: In which $\mathrm{p_{av}}$= average skin friction =$0.5\mathrm{K_s}\tan\delta \times \gamma\mathrm{L}$; $\mathrm{K_s}$ = earth pressure coefficient; δ = pile friction angle W=weight of pile and γ = unit weight of soil.

Ankush chaudhary, Pankaj goswami and Dr. A.K. sahu(2016) have presented the following conclusions:

(1) Steel pile lifting capacity increases with diameter and embedment depth.

(2) Steel piles' uplift capacity increases with the roughness of the surface in a similar.

(3) The uplift capacity of steel piles increases with the size of the base, both with and without the size of the base.

Surface roughness is a term used to describe the roughness of a surface.

(4) Steel piles with enhanced roughness and a larger base have the highest uplift capacity. In comparison to other pile types

(5)The following proposed formula can be used to calculate the uplift capability of various types of piles with a maximum inaccuracy of 30. **(DR. A.K. Sahu, 2016)**

Ireland has reported the results of six pullout tests in the field for Raymond step taper pile in sand. He made his decision based on the analysis of these findings. Provided a formula for calculating the net ultimate uplift piles' capacity:

$$P_0 = K\sigma_v A_S \tan\emptyset$$

Where

P_0= net ultimate uplift capacity

K=coefficient of lateral earth pressure

σ_v= avg. effective overburden pressure

A_S= surface area of the pile

$\emptyset$= soil angle of friction **(IRELAND, 1975)**

Ultimate uplift capacity can be defined as

$$P_u = P_0 + W$$

Where

P_u = gross ultimate uplift capacity

W=effective self weight of pile

Avg. value of k as recommended by Ireland was 1.75.

Further **Meyerhof** have suggested an equation of rigid piles

$$P_0 = 0.5\gamma K_b L^2 d$$

Where

$$K_b = \text{uplift coefficient}$$

(Meyerhof G. , Uplift Resistance of Inclined anchors And Piles, 1973)

Another theoretically equation for the ultimate uplift capacity of piles has also been proposed by **Meyerhof. According to this equation, ultimate capacity may be expressed as**

$$P_0 = K_u \sigma_v A_s \tan\delta$$

Where $K_u = \text{uplift coefficient}$

$\delta = \text{friction angle between the soil and the pile's surface}$

(Meyerhof G. , The uplift Capacity Of Foundation Under Oblque Loads, 1973)

BRAJA M. DAS and DAVID B.ROZENDAL have investigated that

1. f is the unit of skin friction between the soil particles and the stacks grow in a roughly linear fashion up to a critical depth in a systematic manner. The unit skin friction remains nearly constant beyond the critical depth.

2. According to these measurements, the critical embedment ratio (L/d) increases with relative density of compaction up to roughly 1.5 for $D_r \leq 80\%$.

3. Equations for predicting ultimate uplift capacity have been devised based on the limited test findings. Before they are used to field design challenges, these equations may need to be updated to reflect the findings of additional field and laboratory studies. (Rozendal, 0-4)

Hamed Niroumand, Khairul Anuar Kassim, Amin Ghafooripour and Ramil Nazir have presented that

The load is carried and transferred by the building's substructure, commonly known as pile foundations. The bearing ground is below ground level, and the structure is attached to it. The two most important foundation components are the pile cap and piles. Long and thin pile members help to transfer weight from shallow, low-bearing-capacity soil to deeper, higher-bearing-capacity soil or rock. They can also be utilized to resist uplift forces in typical soil conditions to resist lateral forces, the soil must be in poor condition. Pile caps are typically linked to piles that have been driven, drilled, or jacked into the earth. Many investigators reported the uplift response of piles in cohesion less soil based on a few laboratory model results, according to a survey of related literature. Previous research demonstrates that there hasn't been much done to identify the uplift capability in the problem of cohesion less soil is one that is frequently encountered in the field. **(Hamed Niroumand, 2012)**

H. Suha Aksoy, Mesut Gor and Esen Inal have investigated that Friction forces between the building and the soil Geotechnical engineering is designed with this in mind piles, retaining walls, sheet piles, and other similar structures walls of the diaphragm Despite the fact that numerous studies have been conducted on the subject, In recent years, the soil-structure interaction has gotten a lot of attention. Frictional forces are still computed in pile design using empirical methods from the early twentieth century. Wood has been utilized as friction piles for centuries. This study looked at a variety of low-to-high ratios. Plasticity clays (CL) were added to the sandy soil and compacted to the Proctor density level. Interface shear tests were used to determine the skin friction angles of these soils with steel (st37) and FRP (IST). Engineers can use the data acquired from the test results to create a chart that they can use in pile design. The skin friction angles of soils for which only the internal friction angles are known can be estimated using this chart. The skin friction angles of soils can be determined using steel and FRP materials. Conclusions of this paper is-

(1) On the interfaces between soils and piling materials, IST was applied (steel and FRP). These materials' skin friction angles with various soils were measured.

(2) Soils with internal friction angles ranging from 28 to 43 degrees were used in laboratory tests.

(3) Following a study of the test findings, a chart is provided that allows for the determination of the angle of skin friction between soil and different pile materials.

(4) Many research were located in the literature, and when they were compared to the proposed chart, they were found to be in agreement, it was discovered that over 90% of the (δ) values were in agreement. In today's world, design engineers use formulae that accept the same values for all pile materials (δ=2/3).

This method prevents the creation of more realistic designs. The proposed chart can be used to establish true skin friction angles (δ). By selecting suitable pile diameter, length, and quantity, more cost-effective designs can be created. **(H. Suha Aksoy, 2016)**

Codal Provisions

According to IS: 2911 (Part 4)-1985, there are two types of tests for each type of loading (vertical, lateral, and pullout): initial and routine tests. **Initial test:** This test is required for one or more of the following reasons. In the case of critical and significant projects, this is done, with the number of tests varied depending on the number of piles required.

Routine test: One or more of the following reasons need of this test, the number of tests should, in general, behalf of the total number of piles required. Depending on the nature and type of building, the number of tests may be increased by up to 2% in a single case and strata condition:

(a) One of the parameters used to determine the safe load of a pile.

(b) At working load, verifying the safe load and range of safety for the pile's unique functional need; and

(c) Detection of any unexpected performance that differs from the findings of the first test, if one was performed.

Application of load: Applying a series of vertical downward incremental loads on the pile, each increment being approximately 20% of the safe load, is how the vertical load test should be done. When evaluating raker piles, it's essential to load them along the axis**.**

For the initial test, the safe load on a single pile should be the least of the following:

(1) Unless the nature and type of structure require differently, in which case the safe load must match to the stated total displacement permissible, the safe load should be two-thirds of the final load at which the total displacement reaches a value of 12 mm.

(2) In uniform diameter piles, overall displacement equals 10% of pile diameter; in under-reamed piles, total displacement equals 7.5 percent of bulb diameter. **(I.S. code part 4 , 1985)**

Meyerhof (1973) presented an analysis for determining batter pile axial pullout resistance. The pullout resistance, Pu, for an inclined pile with vertical axis and vertical depth of embedment, D, is given as, neglecting pile weight.

$$P_u = (\rho_0 K_u \tan\delta) A_s$$

Where,

A_s = Embedded pile surface area

P_0 = Avg. effective overburden pressure

K_u = uplift coefficient

δ = pile soil friction angles **(Meyerhof G. a., 1968)**

Awed and Ayoub (1976) investigated vertical and inclined anchors' ultimate lifting capacity in soil with reduced cohesiveness the net ultimate uplift capacity of inclined piles was calculated using an empirical equation.

$$P_{\alpha} = P_0 \frac{\cos \alpha}{\cos \alpha + \tan \alpha}$$

Where:

$P_0 =$ Net ultimate uplift capacity of vertical pile (**Ayoub, 1976**)

Weeraya Chim-Oye and Narin Marumdee has compared that Kulhawy et al (1979); Das (1983); and Chattopadhyay and Pise (1989) provided three estimates of a pile's ultimate lifting capability (1986). The Modified Mazurkiewicz approach was utilized to forecast net ultimate uplift capacity of piles because the field experiment did not reach the failure stage. The ultimate uplift capacity of Kulhawy et al (1979) is shown to be similar to field data, offering the highest safety. The ultimate uplift capacity calculated by Chattopadhyay and Pise (1986) differ the most from the ultimate uplift capacity. **(Narin, 2013)**

Kulhawy et al. (1979) was given an empirical equation:

$$\mathbf{P_{unet}} = \pi \mathbf{d} \frac{\mathbf{L^2}}{\mathbf{2}} \mathbf{K\gamma' \tan\delta}$$

Where:

$\emptyset =$ friction angle

$\gamma' =$ Effective unit weight

$\delta =$ Angle of pile friction

Coefficient of earth pressure (K):

$$K = K_a \text{ for Loose sand}$$

$$K = \sqrt{K_p} \text{ for dense sand}$$

Where:

K_a and K_p are the rankine active and passive earth pressure.

(Kulhawy, 1979)

CHAPTER 3

Experimental Setup

3.1 Construction of Pit

The D.T.U. Campus near O.A.T. was chosen as a suitable location (open air theater). I built the pit by myself.

Fig.3.0 Real Image of Pit

The length, width, and depth of the pit are 1m, 1m, and 1m, respectively. Local building materials such as cement, sand and aggregates were transported to the main site. The site of interest received approximately 1 cubic meter of Yamuna sand. Yamuna sand was transported and put in layers on the dry floor. The sand was then dried in the sun for 15days.

Fig. 3.1Transporting sand bags at site

Fig.3.2 Sand laid for drying

About 1 m3 of Yamuna sand was poured into the pit. It was delivered in polypropylene bags from a building materials shop. I poured three layers of sand in the pit. 4 to 5 bags of sand were carried to the work site on the first day and dried for 7 to 8 days. After drying, it was sieved for particle size uniformity, weighed using a weighing machine, and then poured in the pit with proper hammer tempering. Similarly, when filling 3 to 4 bags of sand, they were first dried for 7 to 8 days and then weighed before being placed in the pit. Finally, the pit was filled with 4 to 5 dry sand bags. In total, 12 to 15 sand bags were filled in the pit, resulting in a sand volume of approximately 1 cubic meter. After that I have calculated the density (density=mass/volume).

This whole process I was repeated 5 to 6 times for getting a pullout capacity of model steel pile at various densities.


Fig. 3.3 Sand bag with Weighing machine (Real Image)

3.2 Placing of girders

The girders were hauled to the site once the sand in the pit had been filled. To hold the Dial gauge magnetic stand, I was used two girders shown in figure.

Fig.3.4 Girders with Magnetic stand

3.3 Manufacturing of Pile

A single pile was manufactured with the help of lathe machine. Mild steel material was used to manufacturing a single pile. Steel single pile is hollow of 31.75mm diameter and 90cm length. Pile is marked at interval of 5cm as shown in figure.

Fig 3.5 Model steel pile

The hollow pile's end is welded with a solid mild steel cone. The cone is supposed to be driven firmly into the sand. A cutter and a lathe machine are used to make a solid cone and pile, as shown in figure 3.6

Fig 3.6 Manufacturing of pile

3.4 Arrangement to apply load on the pile

A circular plate is welded on the top most ends of the model steel pile and another circular plate is welded at the end of wire rope. Mechanism is made in such a manner that when loads are hanged at the other end of wire rope with the help of hanging rod, pile is pulled out from the sand by same load. Both circular plate is holled with the help of drill machine. Four holes are done in each plate for tighting both the plate with the help of nut and bolts.

Fig.3.7 Hole in circular plate

Fig 3.8 welding of circular plate on model pile

Fig.3.9 Petal welding between wire rope and circular plate

Fig.3.10 wire rope welded with circular plate

Fig.3.11 hanging rod for load pans

Fig.3.12 marking of pit at interval of 10cm

Fig.3.13 inside whole view of pit (marked)

Fig. 3.14 Bucket for pouring uniform sand

Fig 3.15 sand fills in bucket

Fig.3.16 Bucket inside the pit at 10cm from bottom

Fig.3.17 side view of pit

Fig.3.18 connection of both circular plates with the help of nut and bolts

(By myself)

Fig.3.19 pouring sand into pit

Fig.3.20 set up of two dial gauge stand at equidistant

Fig.3.21 complete view of mechanism

3.5 Test procedures:

For the experimental programmed, the following technique was used.

- Before using, let the sand air dry for about 2-3 days.
- For each stage, pour sand into the testing pit from a constant height by moving hopper type by hand. We pour the sand into the pit from a stable height of 10cm in this situation. This is done to ensure that the density of the sand in the pit is uniform.
- Model steel pile is vertically held in the pit
- Set up two dial gauges with a sensitivity of 0.01mm to calculate the pullout (in mm) of the model pile.
- Dial gauge is set initially at zero mm.
- Apply the load by dead weights.
- The loads were applied with dead weights, starting with the smallest and steadily increasing in size.
- Dial gauge readings for both dial gauges were observed for each increment of the pile corresponding to the pullout load applied when dial gauge readings became stable.
- The average value of displacement recorded from both dial gauges was used as the axial displacement of the pile corresponding to the pullout load applied.
- Apply the second loading in the same manner, taking note of the pile pullout for this loading.
- The load at which pile is completely comes out from the sand and do not take further load, called ultimate pullout capacity of pile.
- Plot the load versus pullout curves for different densities.
- Compare the results of ultimate pullout capacity of the piles for different densities.
- Compare the experimental results with analytically.

Chapter 4

Results and discussion

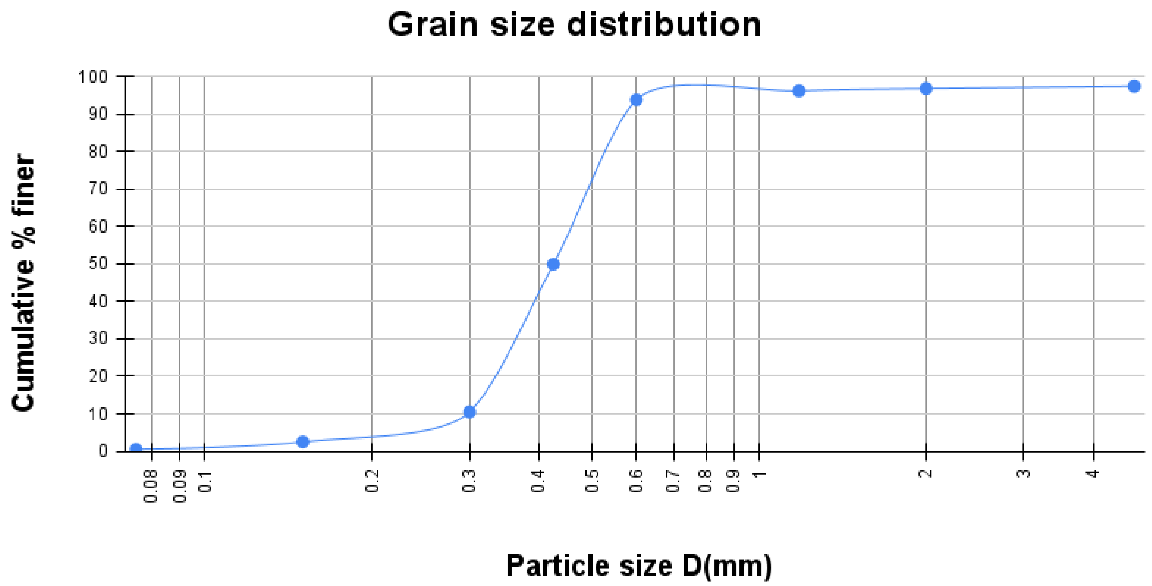
4.1 sieve analysis

The sand sample taken from the place of interest was sieved in the laboratory, and the following observation sheet was prepared:-

Table 4.0 observation sheet for sieve analysis

S.No.	**IS Sieve**	**Particle size D(mm)**	**Mass retained(g)**	**%Retained**	**Cumulative % retained**	**Cumulative % finer(N)**
1	4.75mm	4.75mm	26	2.6	2.6	97.4
2	2.36mm	2.36mm	6	0.6	3.2	96.8
3	1.18mm	1.18mm	6	0.6	3.8	96.2
4	600 μ	0.600mm	24	2.4	6.2	93.8
5	425μ	0.425mm	439	43.9	50.1	49.9
6	300 μ	0.300mm	394	39.4	89.5	10.5
7	150μ	0.150mm	80	8	97.5	2.5
8	75μ	0.075mm	20	2	99.5	0.5
Pan			5	0.5	100	0

Graph.4.0: Grain size distribution curve



$$D_{30} = 0.35$$

$$D_{60} = 0.48$$

Now $\quad C_u = 1.6$, (for uniformly graded sand, $C_u = 1$or less than 2)

$$C_c = 0.85 \quad D_{10} = 0.3$$

- **Hence soil is classified as poorly/uniformly graded sand (SP)**

4.2 specific gravity of sand (G):

The weight in air of a certain volume of soil solids at a given temperature divided by the weight in air of an equal volume of distilled water at that temperature is known as specific gravity (G). Pycnometer method is used to determine the specific gravity of sand.

Fig. 4.0 specific gravity of sand by pycnometer method

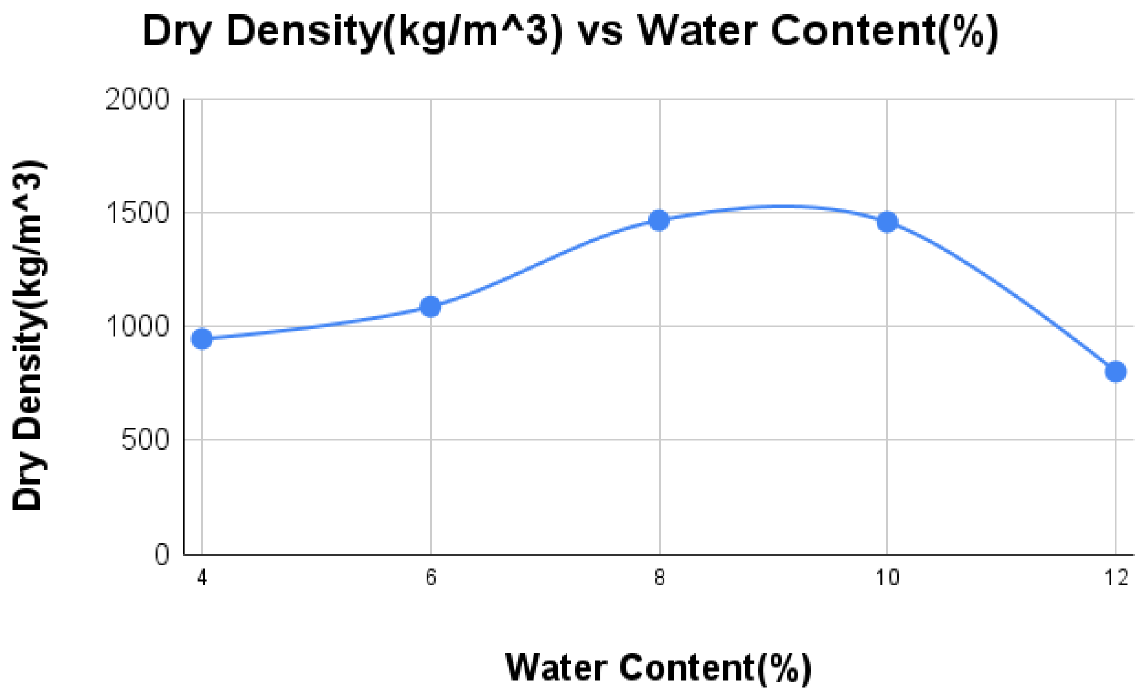
- **Specific gravity of sand (G) = 2.60**

4.3 Standard proctor test:

Table 4.1: Standard proctor test

Water Content (%)	Dry Density(kg/m^3)
4	945.84
6	1087.38
8	1466.05
10	1458.96
12	801.78

Graph.4.1: Determination of maximum dry density



Maximum dry density ($\rho_{max.}$) = $1458.96 kg/m^3$ at O.M.C. (9.53%)

4.4 Direct shear test at max. Dry density:

Fig.4.1 Perform D.S.T. in soil mechanics lab

Table 4.2: Direct shear test at maximum dry density

		Trial 1	Trial 2	Trial 3	
Normal Stress (kg/cm2)		0.65	1.15	1.65	
	Displacement (mm)	Load 1 (N)	Load 2 (N)	Load 3 (N)	Corrected Area (cm2)
0	0	0	0	0	36
0.166666667	0.1	15	31	52	35.94
0.333333333	0.2	43	70	96	35.88
0.5	0.3	71	91	128	35.82
0.666666667	0.4	87	114	154	35.76
0.833333333	0.5	99	129	171	35.7
1	0.6	107	146	189	35.64
1.166666667	0.7	115	158	201	35.58
1.333333333	0.8	122	168	218	35.52
1.5	0.9	128	180	234	35.46
1.666666667	1	133	186	253	35.4
1.833333333	1.1	137	195	267	35.34
2	1.2	142	200	282	35.28
2.166666667	1.3	145	209	289	35.22
2.333333333	1.4	147	213	294	35.16
2.5	1.5	149	220	297	35.1
2.666666667	1.6	150	225	300	35.04
2.833333333	1.7	152	232	302	34.98
3	1.8	155	233	304	34.92
3.166666667	1.9	157	238	306	34.86
3.333333333	2	159	241	307	34.8
3.5	2.1	158	248	308	34.74
3.666666667	2.2	156	250	310	34.68
3.833333333	2.3		254	311	34.62
4	2.4		253	312	34.56
4.166666667	2.5		251	311	34.5
4.333333333	2.6		249	310	34.44
4.5	2.7			310	34.38
4.666666667	2.8			309	34.32
	2.9				34.26
	3				34.2

Shear Load 1 (Kg)	Shear Load 2 (Kg)	Shear Load 3 (Kg)	Shear Stress 1 (Kg/cm2)	Shear Stress 2 (Kg/cm2)	Shear Stress 3 (Kg/cm2)
0	0	0	0	0	0
1.529051988	3.160040775	5.300713558	0.042544574	0.087925453	0.147487856
4.383282365	7.135575943	9.785932722	0.12216506	0.198873354	0.2727406
7.237512742	9.276248726	13.0479103	0.202052282	0.258968418	0.364263269
8.868501529	11.62079511	15.69826707	0.248000602	0.324966306	0.438989571
10.09174312	13.14984709	17.43119266	0.28268188	0.368343056	0.488268702
10.90723751	14.88277268	19.26605505	0.306039212	0.417586214	0.540573935
11.72273191	16.10601427	20.48929664	0.329475321	0.45267044	0.57586556
12.4362895	17.12538226	22.22222222	0.350120763	0.48213351	0.625625626
13.0479103	18.34862385	23.85321101	0.367961373	0.517445681	0.672679385
13.55759429	18.96024465	25.79001019	0.38298289	0.535600131	0.728531361
13.96534149	19.87767584	27.21712538	0.395170953	0.562469605	0.77015069
14.47502548	20.38735984	28.74617737	0.410289838	0.577873011	0.814800946
14.78083588	21.30479103	29.45973496	0.41967166	0.604906049	0.836449034
14.98470948	21.71253823	29.96941896	0.426186276	0.617535217	0.852372553
15.18858308	22.42609582	30.27522936	0.432723165	0.638920109	0.862542147
15.29051988	22.93577982	30.58103976	0.436373284	0.654559926	0.872746568
15.49439348	23.64933741	30.78491335	0.442950071	0.676081687	0.880071851
15.80020387	23.75127421	30.98878695	0.45246861	0.680162492	0.887422307
16.00407747	24.26095821	31.19266055	0.459095739	0.695954051	0.894798065
16.20795107	24.5667686	31.29459735	0.46574572	0.705941627	0.899270039
16.10601427	25.2803262	31.39653415	0.46361584	0.727700812	0.90375746
15.90214067	25.4841998	31.60040775	0.458539235	0.734838518	0.911199762
	25.89194699	31.70234455		0.747889861	0.915723413
	25.79001019	31.80428135		0.746238721	0.92026277
	25.5861366	31.70234455		0.741627148	0.918908538
	25.382263	31.60040775		0.736999506	0.917549586
		31.60040775			0.919150894
		31.49847095			0.917787615
		0			0
		0			0

We know that this equation

$$\boxed{\tau = c + \sigma \tan\emptyset}$$

(Coulomb's, 1991)

Now, $\tau_1 = c + \sigma_1 \tan\emptyset$………………………………… (1)

, τ_2=c + $\sigma_2$tan∅……………………………… (2)

Solve equation 1 and 2 we get internal friction (∅)value.

$$\emptyset = 29.85^{\circ}$$

Table 4.3: Direct shear test at γ_b= 12.21KN/m^3

		Trial 1	Trial 2					
Normal Stress kg/cm2		**0.65**	**1.15**					
	Displacement (mm)	**Load 1 (N)**	**Load 2 (N)**	**Corrected Area (cm2)**	**Shear Load 1 (kg)**	**Shear Load 2 (kg)**	**Shear Stress 1 (kg/cm2)**	**Shear Stress 2 (kg/cm2)**
	0.1	0	0	35.94	0	0	0	0
	0.2	49	65	35.88	4.99490316	6.625891947	0.1392113478	0.1846681145
	0.3	49	76	35.82	4.99490316	7.747196738	0.1394445327	0.216281316
	0.4	51	89	35.76	5.198776758	9.072375127	0.1453796633	0.2537017653
	0.5	57	103	35.7	5.810397554	10.49949032	0.162756234	0.2941033702
	0.6	61	113	35.64	6.21814475	11.51885831	0.1744709526	0.3232002892
	0.7	61	120	35.58	6.21814475	12.2324159	0.17476517	0.3438003345
	0.8	62	124	35.52	6.320081549	12.6401631	0.1779302238	0.3558604476
	0.9	68	132	35.46	6.931702345	13.45565749	0.1954794795	0.3794601662
	1	67	132	35.4	6.829765545	13.45565749	0.1929312301	0.380103319
	1.1	70	140	35.34	7.135575943	14.27115189	0.2019121659	0.4038243318
	1.2	72	144	35.28	7.339449541	14.67889908	0.2080342841	0.4160685681
	1.3	75	148	35.22	7.645259939	15.08664628	0.2170715485	0.4283545224
	1.4	75	151	35.16	7.645259939	15.39245668	0.2174419778	0.4377831819
	1.5	80	151	35.1	8.154943935	15.39245668	0.232334585	0.4385315293
	1.6	82	153	35.04	8.358817533	15.59633028	0.2385507287	0.4451007499
	1.7	84	160	34.98	8.562691131	16.30988787	0.244788197	0.4662632324
	1.8	86	163	34.92	8.76656473	16.61569827	0.2510470999	0.475821829

	1.9	86	169	34.86	8.76656473	17.22731906	0.2514791948	0.4941858595
	2	86	171	34.8	8.76656473	17.43119266	0.2519127796	0.5008963408
	2.1	89	177	34.74	9.072375127	18.04281346	0.2611506945	0.5193671116
	2.2	90	178	34.68	9.174311927	18.14475025	0.2645418664	0.5232050246
	2.3	90	181	34.62	9.174311927	18.45056065	0.2650003445	0.5329451373
	2.4	90	183	34.56	9.174311927	18.65443425	0.2654604145	0.5397695096
	2.5	89	186	34.5	9.072375127	18.96024465	0.262967395	0.5495723086
	2.6	88	190	34.44	8.970438328	19.36799185	0.260465689	0.5623691012
	2.7		190	34.38	0	19.36799185	0	0.5633505481
	2.8		195	34.32	0	19.87767584	0	0.579186359
	2.9		195	34.26	0	19.87767584	0	0.5802006959
	3		195	34.2	0	19.87767584	0	0.5812185918
	3.1		192	34.14	0	19.57186544	0	0.5732825262

We know that this equation

$$\boxed{\tau = c + \sigma \tan\emptyset}$$

(Coulomb's, 1991)

Now, $\tau_1 = c + \sigma_1 \tan\emptyset$………………………………. (1)

, $\tau_2 = c + \sigma_2 \tan\emptyset$………………………………. (2)

Solve equation 1 and 2 we get internal friction ($\emptyset$)value.

$$\emptyset = 32.22^{\circ}$$

Table 4.4: Direct shear test at γ_b= 12.74KN/m^3

		Trial 1	**Trial 2**					
Normal stress kg/cm2		**0.65**	**1.15**					
	Displacement (mm)	**Load 1 (N)**	**Load 2 (N)**	**Corrected Area (cm2)**	**Shear Load 1 (kg)**	**Shear Load 2 (kg)**	**Shear Stress 1 (kg/cm2)**	**Shear Stress 2 (kg/cm2)**
	0.1	0	0	35.94	0	0	0	0
	0.2	26	43	35.88	2.650356779	4.383282365	0.07386724579	0.1221650603
	0.3	43	101	35.82	4.383282365	10.29561672	0.1223696919	0.2874264857
	0.4	54	116	35.76	5.504587156	11.82466871	0.1539314082	0.3306674694
	0.5	63	121	35.7	6.422018349	12.3343527	0.1798884691	0.3455000757
	0.6	67	132	35.64	6.829765545	13.45565749	0.1916320299	0.3775437007
	0.7	70	140	35.58	7.135575943	14.27115189	0.2005501951	0.4011003903
	0.8	76	144	35.52	7.747196738	14.67889908	0.2181080163	0.413257294
	0.9	80	150	35.46	8.154943935	15.29051988	0.2299758583	0.4312047343
	1	80	150	35.4	8.154943935	15.29051988	0.2303656479	0.4319355898
	1.1	80	156	35.34	8.154943935	15.90214067	0.230756761	0.449975684
	1.2	86	165	35.28	8.76656473	16.81957187	0.2484853948	0.4767452343
	1.3	91	168	35.22	9.276248726	17.12538226	0.2633801455	0.4862402687
	1.4	86	172	35.16	8.76656473	17.53312946	0.2493334679	0.4986669357
	1.5	87	174	35.1	8.868501529	17.73700306	0.2526638612	0.5053277225
	1.6	86	177	35.04	8.76656473	18.04281346	0.2501873496	0.5149204753
	1.7	91	181	34.98	9.276248726	18.45056065	0.2651872134	0.5274602817
	1.8	93	183	34.92	9.480122324	18.65443425	0.2714811662	0.5342048755
	1.9	91	186	34.86	9.276248726	18.96024465	0.2661000782	0.5438968631
	2	93	187	34.8	9.480122324	19.06218145	0.2724173082	0.5477638347
	2.1	95	190	34.74	9.683995923	19.36799185	0.2787563593	0.5575127186
	2.2	95	194	34.68	9.683995923	19.77573904	0.2792386368	0.5702346898
	2.3	100	195	34.62	10.19367992	19.87767584	0.2944448272	0.5741674131
	2.4	102	199	34.56	10.39755352	20.28542304	0.3008551365	0.5869624722
	2.5	102	201	34.5	10.39755352	20.48929664	0.3013783628	0.5938926561
	2.6	100	204	34.44	10.19367992	20.79510703	0.2959837375	0.6038068244
	2.7	97	207	34.38	9.887869521	21.10091743	0.2876052798	0.6137555972
	2.8		210	34.32	0	21.40672783	0	0.6237391558
	2.9		214	34.26	0	21.81447503	0	0.6367330714
	3		215	34.2	0	21.91641182	0	0.6408307551
	3.1		219	34.14	0	22.32415902	0	0.6539003814

	3.2		220	34.08	0	22.42609582	0	0.6580427177
	3.3		223	34.02	0	22.73190622	0	0.6681924226
	3.4		225	33.96	0	22.93577982	0	0.6753763197
	3.5		227	33.9	0	23.13965341	0	0.6825856465
	3.6		227	33.84	0	23.13965341	0	0.6837959047
	3.7		226	33.78	0	23.03771662	0	0.681992795
	3.8		225	33.72	0	22.93577982	0	0.6801832686

We know that this equation

$$\boxed{\tau = c + \sigma \tan\emptyset}$$

(Coulomb's, 1991)

Now, τ_1=c + $\sigma_1$tan$\emptyset$.................................. (1)

, τ_2=c + $\sigma_2$tan$\emptyset$.................................... (2)

Solve equation 1 and 2 we get internal friction ($\emptyset$)value.

$$\emptyset = 37.3^{\circ}$$

Table 4.5: Direct shear test at γ_b= 13.24KN/m^3

		Trial 1	Trial 2					
Normal Stress kg/cm2		**0.65**	**1.15**					
	Displacement (mm)	**Load 1 (N)**	**Load 2 (N)**	**Corrected Area (cm2)**	**Shear Load 1 (kg)**	**Shear Load 2 (kg)**	**Shear Stress 1 (kg/cm2)**	**Shear Stress 2 (kg/cm2)**
	0.1	0	0	35.94	0	0	0	0
	0.2	22	49	35.88	2.242609582	4.99490316	0.06250305413	0.1392113478
	0.3	54	83	35.82	5.504587156	8.460754332	0.1536735666	0.2362019635
	0.4	63	116	35.76	6.422018349	11.82466871	0.1795866429	0.3306674694
	0.5	67	121	35.7	6.829765545	12.3343527	0.1913099593	0.3455000757
	0.6	70	132	35.64	7.135575943	13.45565749	0.2002125685	0.3775437007
	0.7	76	140	35.58	7.747196738	14.27115189	0.2177402119	0.4011003903
	0.8	80	144	35.52	8.154943935	14.67889908	0.2295873856	0.413257294
	0.9	80	150	35.46	8.154943935	15.29051988	0.2299758583	0.4312047343
	1	80	150	35.4	8.154943935	15.29051988	0.2303656479	0.4319355898

	1.1	86	156	35.34	8.76656473	15.90214067	0.2480635181	0.449975684
	1.2	91	165	35.28	9.276248726	16.81957187	0.2629322201	0.4767452343
	1.3	86	168	35.22	8.76656473	17.12538226	0.248908709	0.4862402687
	1.4	87	172	35.16	8.868501529	17.53312946	0.2522326942	0.4986669357
	1.5	86	174	35.1	8.76656473	17.73700306	0.2497596789	0.5053277225
	1.6	91	177	35.04	9.276248726	18.04281346	0.2647331257	0.5149204753
	1.7	93	181	34.98	9.480122324	18.45056065	0.2710155038	0.5274602817
	1.8	91	183	34.92	9.276248726	18.65443425	0.2656428616	0.5342048755
	1.9	93	186	34.86	9.480122324	18.96024465	0.2719484316	0.5438968631
	2	95	187	34.8	9.683995923	19.06218145	0.2782757449	0.5477638347
	2.1	95	190	34.74	9.683995923	19.36799185	0.2787563593	0.5575127186
	2.2	100	194	34.68	10.19367992	19.77573904	0.2939354071	0.5702346898
	2.3	102	195	34.62	10.39755352	19.87767584	0.3003337238	0.5741674131
	2.4	102	199	34.56	10.39755352	20.28542304	0.3008551365	0.5869624722
	2.5	100	201	34.5	10.19367992	20.48929664	0.2954689831	0.5938926561
	2.6	100	204	34.44	10.19367992	20.79510703	0.2959837375	0.6038068244
	2.7		207	34.38	0	21.10091743	0	0.6137555972
	2.8		210	34.32	0	21.40672783	0	0.6237391558
	2.9		214	34.26	0	21.81447503	0	0.6367330714
	3		215	34.2	0	21.91641182	0	0.6408307551
	3.1		219	34.14	0	22.32415902	0	0.6539003814
	3.2		220	34.08	0	22.42609582	0	0.6580427177
	3.3		223	34.02	0	22.73190622	0	0.6681924226
	3.4		225	33.96	0	22.93577982	0	0.6753763197
	3.5		227	33.9	0	23.13965341	0	0.6825856465
	3.6		229	33.84	0	23.34352701	0	0.6898205382
	3.7		232	33.78	0	23.64933741	0	0.7000987984
	3.8		237	33.72	0	24.15902141	0	0.7164597096
	3.9		238	33.66	0	24.26095821	0	0.7207652468
	4		238	33.6	0	24.26095821	0	0.7220523276
	4.1		236	33.54	0	24.05708461	0	0.7172654922
	4.2		235	33.48	0	23.95514781	0	0.7155062069

We know that this equation

$$\boxed{\tau = c + \sigma \tan\emptyset}$$

(Coulomb's, 1991)

Now, τ_1=c + $\sigma_1$tan∅.................................... (1)

, τ_2=c + $\sigma_2$tan∅.................................... (2)

Solve equation 1 and 2 we get internal friction (∅)value.

$$\emptyset = 40.01^{\circ}$$

4.5 Experimental study of pile:

- Pile length (h)= 90cm
- Pile diameter (D_p)= 31.75mm
- Weight of pile (W_p)= 1.992kg

4.6 Experimental study of soil:

- Natural moisture content(w_n) present in sand =2.28%
- Soil is poorly graded sand(SP)
- Internal friction (∅) $=$ 29.85° at optimum moisture content (9.53%)

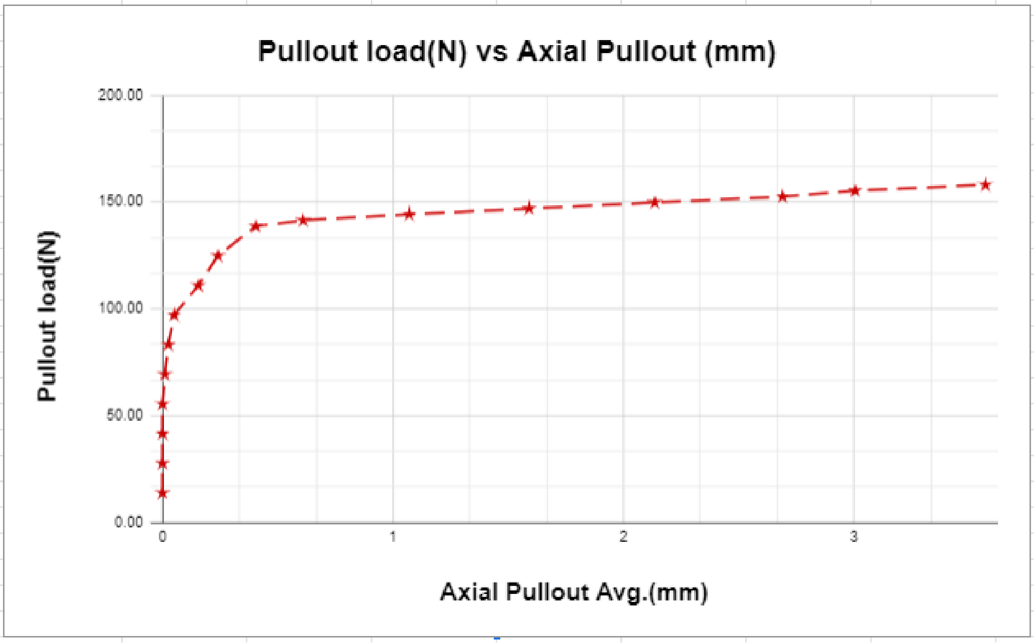
4.7 Experimentally

Determination of Pullout capacity of model steel pile at $\gamma_{(bulk)} = 12.21KN/m^3$

Net ultimate pullout capacity, $Q_{nu} = 138.5N$

Table 4.6: pullout capacity of steel pile

Dial Gauge 1(mm)	Dial Gauge2(mm)	Axial Pullout Avg.(mm)	Pullout load(kg)	Pullout load(N)
0	0	0	1.414	13.87
0	0	0	2.828	27.74
0	0	0	4.242	41.61
0	0	0	5.656	55.49
0.01	0.01	0.01	7.07	69.36
0.02	0.03	0.025	8.484	83.23
0.06	0.04	0.05	9.898	97.10
0.15	0.16	0.155	11.312	110.97
0.25	0.23	0.24	12.726	124.84
0.4	0.41	0.405	14.14	138.71
0.6	0.62	0.61	14.42	141.46
1.08	1.06	1.07	14.7	144.21
1.59	1.59	1.59	14.98	146.95
2.13	2.14	2.135	15.268	149.78
2.69	2.69	2.69	15.55	152.55
3	3.01	3.005	15.83	155.29
3.57	3.57	3.57	16.114	158.08



Graph.4.2: Determination of net ultimate pullout capacity at $\gamma_b = 12.21 KN/m^3$

Fig.4.2 performance of pullout test at $\gamma_{(bulk)} = 12.21KN/m^3$

Fig. 4.3 Ultimate failure of single pile

4.8 Theoretically

Determination of Pullout capacity of model steel pile at $\gamma_{(bulk)} = 12.21KN/m^3$

- Pile length (h)= 90cm
- Pile diameter (D_p)= 31.75mm
- Weight of pile (W_p)= 1.992kg
- Internal friction ($\emptyset$) value at($\gamma_{(bulk)} = 12.21KN/m^3$) = 32.22°
- Soil – pile friction angle (δ) = 23.5° (Huseyin Suha Aksoy, 2016)

4.8.0By IS code 2911 part 1 section 2

$$Q_{nu} = \sum_{i=1}^{n} K_i \,.\, PD_i . \tan\delta \times A_s \qquad \textbf{(I.S. code part 4 , 1985)}$$

Where: $Q_{nu} =$ net ultimate pullout capacity, in KN

$K_i =$ Coefficient of earth pressure

($K_i = 1$ to 1.5, for all $\emptyset = 30^{\circ}$ to 40°)

$PD_i =$ Effective overburden pressure, in KN/m^2

$A_s =$ Surface area of pile, (πdL)

$\delta =$ Angle between soil and pile

$$Q_{nu} = 0.5 \times 12.21 \times 0.90 \times 1 \times \tan 23.5 \times \pi \times 0.03175 \times 0.9$$

$$Q_{nu} = 214.4N$$

<u>% error comparing experimental to theoretical:</u>

$$\% \text{ error} = \frac{|138.5 - 214.4|}{214.4} \times 100 = 35.4\%$$

4.8.1By Meyerhof and Adams (1968)

$$\mathbf{P_{nu} = \frac{1}{3} \pi K_u D \gamma L^2 \tan\delta}$$

where, $P_{nu} =$ net pullout capacity

$$\mathrm{ku = uplift\ coefficient}$$

$$\mathbf{(K_u = 0.9\ to\ 2.5, for\ \emptyset =\ 30^\circ\ to\ 43^\circ\ )}$$

D = pile's diameter

$$\gamma = \mathrm{unit\ weight\ of\ soil}$$

L = pile's length

$$\delta = \mathrm{angle\ of\ soil - pile\ friction}$$

$$P_{nu} = \frac{1}{3} \times \pi \times 0.9 \times 0.03175 \times 12.21 \times 0.9 \times 0.9 \times \tan 23.5$$

$$P_{nu} = 128\mathrm{N}$$

<u>% Error comparing experimental to theoretical:</u>

$$\%\ \text{error} = \frac{|138.5-128|}{128} \times 100 = 8.2\%$$

4.8.2By Kulhawy et al (1979)

$$\mathbf{P_{Unet} = \pi d \frac{L^2}{2} K\gamma' \tan\delta}\ \text{(Kulhawy, 1979)}$$

Where, P_{Unet} = net ultimate pullout/uplift capacity

K = Coefficient of earth pressure

K = K_a to K_0 for loose sand

K = K_0 to 1 for medium sand

K = 1 to K_p for dense sand

Where, K_a and K_p are the active and passive earth pressure coefficient

$$\delta = \mathrm{angle\ of\ pile\ friction}$$

$$\mathbf{P_{Unet} = \pi \times 0.03175 \times \frac{0.9^2}{2} \times 0.47 \times 12.21 \times \tan 23.5}$$

$$\mathbf{P_{Unet} = 100.8N}$$

<u>% Error comparing experimental to theoretical:</u>

$$\% \text{ error} = \frac{|138.5-100.8|}{100.8} \times 100 = 37\%$$

4.9 Experimentally

Determination of Pullout capacity of model steel pile at $\gamma_{(\text{bulk})} = 12.74\text{KN/m}^3$

Net ultimate pullout capacity, $Q_{nu} = 318.8\text{N}$

Table 4.7: Readings of pullout capacity

Dial Gauge1(mm)	Dial Gauge2(mm)	Axial Pullout(mm) Avg.	Pull Out load(kg)	Pullout load(N)
0	0	0	2.828	27.74268
0	0	0	5.656	55.48536
0	0	0	8.484	83.22804
0	0	0	11.312	110.97072
0	0	0	14.14	138.7134
0	0	0	16.968	166.45608
0	0	0	19.796	194.19876
0.03	0.1	0.065	22.624	221.94144
0.07	0.06	0.065	24.038	235.81278
0.12	0.12	0.12	25.452	249.68412
0.16	0.15	0.15	26.866	263.55546
0.23	0.23	0.23	28.28	277.4268
0.32	0.32	0.32	29.694	291.29814
0.43	0.43	0.43	31.108	305.16948
0.6	0.6	0.6	32.522	319.04082
1.02	1.02	1.02	33.936	332.91216
1.25	1.25	1.25	34.501	338.45481
1.49	1.5	1.495	34.501	338.45481
1.75	1.74	1.74	34.501	338.45481
2.05	2.09	2.07	34.501	338.45481
2.44	2.49	2.47	34.501	338.45481
3.58	3.58	3.58	34.501	338.45481
4.02	4.03	4.025	34.501	338.45481

Graph.4.3: Determination of net ultimate pullout capacity at $\gamma_b = 12.74\mathrm{KN/m^3}$

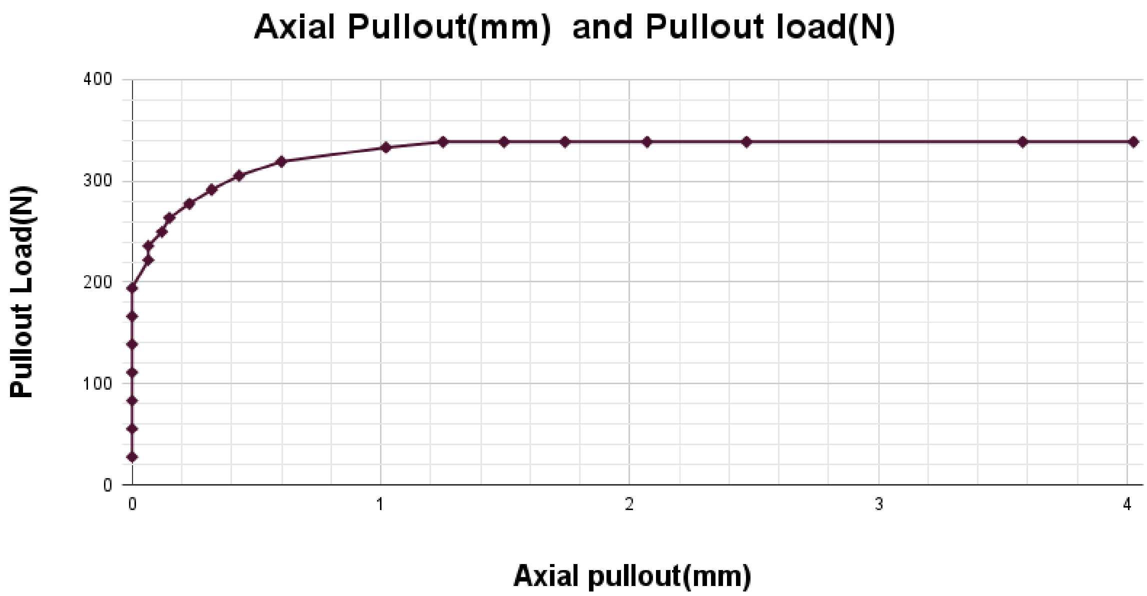


Fig. 4.4 performance of pullout test at $\gamma_{(bulk)} = 12.74KN/m^3$

4.10 Theoretically

Determination of Pullout capacity of model steel pile at $\gamma_{(bulk)} = 12.74KN/m^3$

- Pile length (h)= 90cm
- Pile diameter (D_p)= 31.75mm
- Weight of pile (W_p)= 1.992kg
- Internal friction (∅) value at($\gamma_{(bulk)} = 12.74KN/m^3$) = 37.3°

- Soil – pile friction angle (δ) = 30° (Huseyin Suha Aksoy, 2016)

4.10.0 By IS code 2911 part 1 section 2

$$Q_{nu} = \sum_{i=1}^{n} K_i \, . PD_i . \tan\delta \times A_s$$ (I.S. code part 4 , 1985)

Where: $Q_{nu} =$ Net ultimate pullout capacity, in KN

$K_i =$ Coefficient of earth pressure

($K_i = 1$ to 1.5, for all $\emptyset = 30^\circ$ to 40°)

$PD_i =$ Effective overburden pressure, in KN/m^2

$A_s =$ Surface area of pile, (πdL)

$\delta =$ Angle between soil and pile

$$Q_{nu} = 0.5 \times 12.74 \times 0.90 \times 1 \times \tan 30 \times \pi \times 0.03175 \times 0.9$$

$$Q_{nu} = 297.1N$$

<u>% error comparing experimental to theoretical:</u>

$$\% \text{ error} = \frac{|318.8-297.1|}{297.1} \times 100 = 9.3\%$$

4.10.1By Meyerhof and Adams (1968)

$$\mathbf{P_{nu} = \frac{1}{3}\pi K_u D\gamma L^2 \tan\delta}$$ (Meyerhof G. a., 1968)

where, $P_{nu} =$ net pullout capacity

$K_u =$ uplift coefficient

($K_u = 0.9$ to 2.5, for $\emptyset = 30^\circ$ to 43°)

D = pile's diameter

$\gamma =$ unit weight of soil

L = pile's length

$\delta =$ Angle of soil-pile friction

$$P_{nu} = \frac{1}{3} \times \pi \times 1.8 \times 0.03175 \times 12.74 \times 0.9 \times 0.9 \times \tan 30$$

$$P_{nu} = 357N$$

<u>% Error comparing experimental to theoretical:</u>

$$\% \text{ error} = \frac{|318.8-357|}{357} \times 100 = 10.7\%$$

4.10.2 By Kulhawy et al (1979)

$$\mathbf{P_{Unet} = \pi d \frac{L^2}{2} K\gamma' \tan\delta} \text{ (Kulhawy, 1979)}$$

Where, P_{Unet} = net ultimate pullout/uplift capacity

K = Coefficient of earth pressure

K = K_a to K_0 for loose sand

K = K_0 to 1 for medium sand

K = 1 to K_p for dense sand

Where, K_a and K_p are the active and passive earth pressure coefficient

δ = angle of pile friction

$$\mathbf{P_{Unet} = \pi \times 0.03175 \times \frac{0.9^2}{2} \times 1 \times 12.74 \times \tan 30}$$

$$\mathbf{P_{Unet} = 297N}$$

<u>% Error comparing experimental to theoretical:</u>

$$\% \text{ error} = \frac{|318.8-297|}{297} \times 100 = 7.3\%$$

4.11 Experimentally

Determination of Pullout capacity of model steel pile at $\gamma_{(bulk)} = 13.24 KN/m^3$

Net ultimate pullout capacity, $Q_{nu} = 492N$

Table 4.8: readings of pullout capacity of model steel pile

Dial Gauge1(mm)	Dial Gauge2(mm)	Axial Pullout(mm) Avg.	Pull Out load(kg)	Pull Out load(N)
0	0	0	1.414	13.87134
0	0	0	2.828	27.74268
0	0	0	4.242	41.61402
0.01	0.01	0.01	4.87	47.7747
0.02	0.02	0.02	9.74	95.5494
0.03	0.08	0.055	11.154	109.42074
0.06	0.1	0.08	12.568	123.29208
0.06	0.11	0.085	13.982	137.16342
0.06	0.12	0.09	15.396	151.03476
0.06	0.13	0.095	16.81	164.9061
0.06	0.13	0.095	18.224	178.77744
0.06	0.14	0.1	19.638	192.64878
0.06	0.15	0.105	21.052	206.52012
0.07	0.17	0.12	22.466	220.39146
0.16	0.18	0.17	23.88	234.2628
0.17	0.18	0.175	25.294	248.13414
0.18	0.21	0.195	26.708	262.00548
0.18	0.24	0.21	28.122	275.87682
0.29	0.3	0.295	29.536	289.74816
0.39	0.38	0.381	30.95	303.6195
0.46	0.47	0.465	32.36	317.4516
0.56	0.56	0.56	33.77	331.2837
0.65	0.66	0.655	35.184	345.15504
0.75	0.76	0.755	36.598	359.02638
0.98	0.98	0.98	38.012	372.89772
1.3	1.2	1.25	39.426	386.76906
1.45	1.46	1.455	40.84	400.6404
1.7	1.7	1.7	42.254	414.51174
2	2	2	43.668	428.38308
2.35	2.35	2.35	45.082	442.25442
2.75	2.76	2.755	46.496	456.12576
3.16	3.16	3.16	47.97	470.5857
3.6	3.61	3.61	49.324	483.86844
4.12	4.13	4.125	50.738	497.73978
4.68	4.68	4.68	52.152	511.61112

Graph.4.4: Determination of net ultimate pullout capacity at $\gamma_b = 13.24 \mathrm{KN/m^3}$

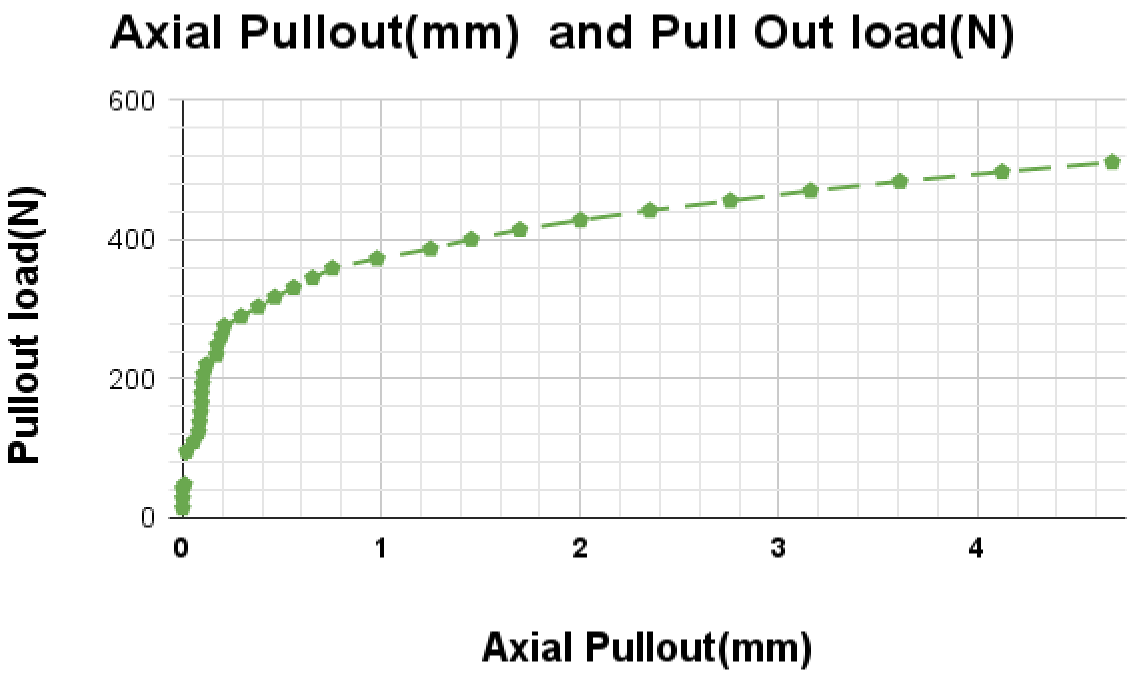


Fig. 4.5 performance of pullout test at $\gamma_{(bulk)} = 13.24KN/m^3$

Fig.4.6: Axial pullout readings (in mm) in digital dial gauge

4.12 Theoretically

Determination of Pullout capacity of model steel pile at $\gamma_{(bulk)} = 13.24KN/m^3$

- Pile length (h)= 90cm
- Pile diameter (D_p)= 31.75mm
- Weight of pile (W_p)= 1.992kg
- Internal friction ($\emptyset$) value at($\gamma_{(bulk)} = 13.24KN/m^3$) = 40.01°
- Soil – pile friction angle (δ) = 31.50° (Huseyin Suha Aksoy, 2016)

4.12.0 By IS code 2911 part 1 section 2

$$Q_{nu} = \sum_{i=1}^{n} K_i \, . PD_i . \tan\delta \times A_s \qquad \text{(I.S. code part 4 , 1985)}$$

Where: $Q_{nu} =$ Net ultimate pullout capacity, in KN

K_i = Coefficient of earth pressure

$$(\mathbf{K_i = 1 \text{ to } 1.5, for\ all\ \emptyset = 30^\circ \text{ to } 40^\circ})$$

PD_i = Effective overburden pressure, in KN/m^2

A_s = Surface area of pile, (πdL)

δ = Angle between soil and pile

$$Q_{nu} = 0.5 \times 13.24 \times 0.90 \times 1 \times \tan 31.5 \times \pi \times 0.03175 \times 0.9$$

$$Q_{nu} = 328N$$

<u>% error comparing experimental to theoretical:</u>

$$\% \text{ error} = \frac{|492-328|}{328} \times 100 = 50\%$$

4.12.1By Meyerhof and Adams (1968)

$$\mathbf{P_{nu} = \frac{1}{3}\pi K_u D\gamma L^2 \tan\delta} \quad \text{(Meyerhof G. a., 1968)}$$

where, P_{nu} = net pullout capacity

K_u = uplift coefficient

$$(\mathbf{K_u = 0.9 \text{ to } 2.5, for\ \emptyset = 30^\circ \text{ to } 43^\circ})$$

D = pile's diameter

γ = unit weight of soil

L = pile's length

δ = angle of soil-pile friction

$$P_{nu} = \frac{1}{3} \times \pi \times 2.3 \times 0.03175 \times 13.24 \times 0.9 \times 0.9 \times \tan 31.5$$

$$P_{nu} = 502N$$

<u>% Error comparing experimental to theoretical:</u>

$$\% \text{ error} = \frac{|492-502|}{502} \times 100 = 1.99\%$$

4.12.2 By Kulhawy et al (1979)

$$\mathbf{P_{Unet} = \pi d \frac{L^2}{2} K\gamma' tan\delta} \text{ (Kulhawy, 1979)}$$

Where, P_{Unet} = net ultimate pullout/uplift capacity

K = Coefficient of earth pressure

$K = K_a$ to K_0 for loose sand

$K = K_0$ to 1 for medium sand

$K = 1$ to K_p for dense sand

Where, K_a and K_p are the active and passive earth pressure coefficient

δ = angle of pile friction

$$\mathbf{P_{Unet} = \pi \times 0.03175 \times \frac{0.9^2}{2} \times 1.3 \times 13.24 \times tan31.5}$$

$$\mathbf{P_{Unet} = 426N}$$

<u>% Error comparing experimental to theoretical:</u>

$$\% \text{ error} = \frac{|492-426|}{426} \times 100 = 15.4\%$$

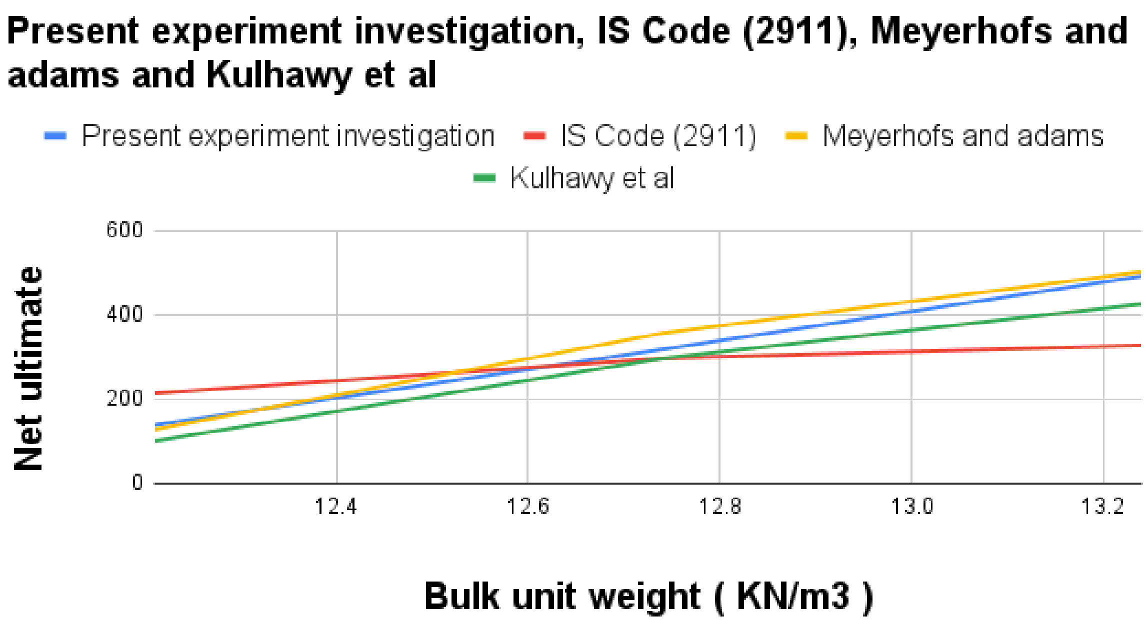
Table 4.9: % Experimental, theoretically pullout/uplift capacity of model pile and respective % error of steel pile

Bulk unit weight (γ_b) KN/m^3	**Experimental results (N)**	**IS code 2911(N)**	**%error**	**Meyerhof (N)**	**%error**	**Kulway et al**	**%error**
12.21	138.5	214.4	35.4	128	8.2	100.8	37
12.74	318.8	291.1	9.5	357	10.7	297	7.3
13.24	492	328	50	502	1.99	426	15.4

Table 4.10: Net ultimate pullout capacity of steel pile

Bulk unit weight (KN/m3)	Net Ultimate Pullout Capacity(N)			
	Present experiment investigation	IS Code (2911)	Meyerhof's and Adams	Kulhawy et al
12.21	138.5	214.4	128	100.8
12.74	318.5	297.1	357	297
13.24	492	328	502	426

Graph.4.5: comparison of net ultimate pullout capacity of model steel pile



CONCLUSION

- Axial pullout load vs. axial pullout (in mm) diagrams for vertical piles are linear at first and non-linear later in the loading process.
- With an increase in soil density, the pullout capacity of a single pile under uplift loading increases.
- With the exception of a few cases, the pullout capacity of a model steel pile was validated using several theories and the percentage error was within 15%.
- Experimental result of pullout capacity is much closer to the Meyerhof's and Adams theory.
- Experimental results of pullout capacity are also slightly closer to the Kulhawy et al. theory.
- Experimental result of pullout capacity is different from IS code 2911 (except some cases).
- For bulk unit weight $(\gamma_{bulk}) = 12.21 KN/m^3$, the net ultimate pullout capacity of single steel pile is 138.5KN, 214.4KN, 128KN and 100.8KN.
- For bulk unit weight $(\gamma_{bulk}) = 12.74 KN/m^3$, the net ultimate pullout capacity of single steel pile is 318.8KN, 297.1KN, 357KN, and 297KN.
- For bulk unit weight $(\gamma_{bulk}) = 13.24 KN/m^3$, the net ultimate pullout capacity of single steel pile is 492KN, 328KN, 502KN, and 426KN.

Future scope of the work

- The test can be carried out on a variety of soils other than sand, such as gravels, silt soil, etc.
- In future, Different pile materials can be used in the test.
- Tests can be carried out on piles with various surface coatings, such as bitumen, anticorrosive paint, etc.
- The effect of vibrations can be investigated in order to learn how piles behave under wind load and earthquake situations.

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