Analysis of Micro Pile as Alternative Foundation System for Light Weight Structure on Expansive Soil around Addis Ababa City

By

Gebisa Raya Chala

A Thesis Submitted to

Civil Engineering Department

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Award of Degree of Masters in Civil Engineering (Geotechnical Engineering)

Office of Graduate Studies

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Approval Sheet of the Boards of Examiners

We, the undersigned, members of the boards of examiners of the final open defense by Gebisa Raya Chala have read and evaluated his thesis entitled "Analysis of Micro Pile as Alternative Foundation System for Light Weight Structure on Expansive Soil around Addis Ababa City" and examined the candidate. This, is therefore, to certify that the thesis has been accepted in partial fulfillment of the requirement of the degree of masters in civil engineering (geotechnical engineering).

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Declaration

I hereby declare that, this MSc Thesis is my original work and has not been presented for a degree in any other University, and all sources of material used for this thesis have been duly acknowledged

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DEDICATION

Dedicated to my mother Getu Zawude Ketema

Acknowledgement

First, I would like to thank the 'Almighty God' who made things possible and accessible to begin and finish this work successfully. Secondly, I would like to forward my deepest gratitude to my main advisor Dr. Addisallem Zeleke for his critical observations, suggestions, comments and endless support at various stage of this thesis work. I also express my sincere, and thanks to my co-advisor, Mr. Abda Bariso for his continues comments, encouragements and many useful contributions in my study.

I am grateful to Addis geosystem pl.c from where I received soil laboratory result and to Adama Science and Technology University for give me sponsorship. Words cannot express my feelings, which, I have for, Zerihun Lemesa, Bedane Hailu, Tadese Tufa, Estifanos Birhanu, Gadisa Birke, Bekelu Chala, Kechine Chala, Tamiru worku, Balcha Kebede, Dame Bekele and many others for their constant encouragement and help.

Abstract

Most part of Addis Ababa is covered by expansive soil and constructions of structures become increasing due to rapid urbanization. The lightweight structures constructed in this soil show cracking, tilting of window and doors, differential settlement caused by swelling and shrinkages of the soil. Therefore, to minimize such defects, there are different mitigations or remedial measures. The purpose of this study is to analyze and comparison of micro pile foundation system with isolated footing for lightweight structures on expansive soil.

The analysis done by considering foundation construction cost and period, and environmental disturbance. Using allowable bearing capacity of soil and ETABS 2016 software to determine point load coming to footing and then both type foundation are designed using the point load.

The result showed that construction of isolated footing needs large volume of excavation and takes more construction period in comparison with micro pile. The cost required for construction of micro pile is minimized by 7.79% as compared with isolated foundation. This study definitely answers the question concerning economy and feasibility of micro pile foundation for lightweight structure on expansive soil.

Key words:-Expansive Soil, Light Weight Structure, isolated footing and Micro Pile.

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List of symbol and abbreviations

AASHTO - American Association of Highway and Transportation Officials;

ASTM - American Society for Testing and Materials;

D_f - Depth of foundation

D-thickness of footing

d- effective thickness of footing

B - Width of foundation

L - Length of foundation

F_{ctd}- characteristic tensile strength of concrete

f_{ctm} - mean value of axial tensile strength of concrete

f_{ck} - characteristic compressive strength of concrete

f_{yk} - characteristic yield strength of reinforcement

f_y - yield strength of reinforcement

f_d- design strength

f_k - characteristic strength

$f_{0.2}$ - yield strength of reinforcement at 0.2% offset

f_{bd} - design bond strength

l_b - basic anchorage length

l_{bnet} - required anchorage length

l_{bmin} - minimum anchorage length

γ_m - partial safety factor for material

γ_c- partial safety factor for concrete

γ_s - partial safety factor for steel

A_{sef}- Area of reinforcement actually provided

A_{scal} - Theoretical area of reinforcement required by the design

M_{sd}-design value of internal bending moment about tension steel

q_{all} - allowable bearing capacity of soil

q_{ult} - ultimate bearing capacity

M_y- bending moment along y- axis

M_x - bending moment along x- axis

L_x - length of footing along x-axis

l_y - length of footing along y-axis

e_x - eccentricity of load along x-axis

e_y - eccentricity of load along y-axis

A -Area of footing

P_d- design axial compression load

σ -Contact pressure

z - Lever arm between compressive strain in outer most Concrete fiber and tensileforce developed in the tension reinforcement.

Qu - unconfined compressive strength

N_{design} -specific weighted average N-value for footing influence zone

SPT-standard penetration test

N- Standard penetration test value

BH- bore hole

Su'-average undrained shear strength along the depth of penetration

Su-undrained shear strength at the base of micro pile group

Q_g –ultimate capacity of group

B_g-micro pile group width

L_g-micro pile group length

C- Cohesion

Nc-bearing capacity factor

I_f, F_1, F_2-influence factor

I_s-composite Steinbrenner influence factor

E_s-elastic modules of soil

B'-least lateral dimension of contributing base area

q_o- intensity of contact pressure

α- number of corners contributing to settlement

μs- poisons ratio of soil

ɳ-efficiency of group

ρ -Geometric ratio of reinforcement

ρ_{min} - minimum geometric ratio of reinforcement

v_{up} - shear resistance of a section

v_c - shear carried by concrete

S- Spacing

v_{sd} - shear acting along the periphery of the critical section

V -Punching shear

k_1, k_2- margin of strength

ϕ - Diameter of reinforcement

OD_{casing} - Pile casing outside diameter

ID_{casing} - Pile casing inside diameter

A_{casing} - Pile casing steel area

$F_{ycasing}$ - Casing yield strength

A_{bar} -Bar area

$F_{y\,bar}$-Bar yield strength

A_{grout} - Grout area

F_{ysteel} -steel yield stress

Pt_{all} - Allowable tension load

Pc_{all} -Allowable compression load

fc_{grout} - compressive strength of concrete

D_g - grout body diameter

D_b - drill hole diameter

P_{G-all} - allowable geotechnical bond load

L_b - bond length

Fs - Factor of safety

α_{bond} - bond strength between grout and soil

E_{steel} - Steel modulus of elasticity

E_{grout} - Grout modulus of elasticity

L_{casing}- cased length of micro pile

$L_{uncased}$- uncased length of micropile

S -total settlement

Se - elastic Settlement

Si - primary consolidation settlement

Sc - Secondary settlement

H_o - thickness of soil layer

e_o- initial void ratio

Z -Middle depth of soil layer Ho thickness

Cc - coefficient of compressibility

1 INTRODUCTION

Expansive soil is a clay soil that changes its volume when the water content of it changes. Usually, the soil will shrink when water (or moisture) content is reduced and will swell when the water content increases. The degree of expansiveness depends on the content of the active clay mineral called Montmorillonite. Expansive Soils are known to occur in many part of the world. Many countries in the world like Argentina, Australia, Burma, Canada, Cuba, Ethiopia, Ghana, India, Iran, Israel, Mexico, Morocco, Poland, South Africa, Spain, Turkey, U.S.A and Venezuela have been identified. In Ethiopia, widely spread in the southern, south-east and south-west part of the city of Addis Ababa areas, where most of the recent construction are being carried out and central part of Ethiopia following the major trunk roads like Addis-Ambo, Addis-Woliso, Addis-Debre Berhan, Addis- Modjo are some of the areas covered by expansive soils. Some part of Mekele, Gondor, Bahirdar, Debre-berihan and Gambela are also areas partly covered by expansive soils (11).

Structural distress is common due to differential movement of building foundations caused by heave of expansive soils. There are different types of foundation that are used in expansive soils. Slab-on-grade, Slab-on-grade with piles, Pile-and-beam, Basement with wall footings and slab floor, Basement with wall piles and slab floor, structurally suspended floor slab on piles are foundation types used. For foundations constructed on soils consisting of highly expansive clay, underpinning of the foundation is the most reliable method of remediation. Various underpinning methods that have been used include drilled piers, helical piles, push pins, and micro piles. The small size and versatility of the drilling equipment make micro piles well suited for installation in places where access and mobility are limited. The drilling equipment is easily attached to existing foundations utilizing the weight of the structure for reaction. This makes the use of micro piles advantageous in places such as crawl spaces, garages, basements, and other confined areas. Moreover, possible to use shallow foundation by taking one or more of the following method of mitigation or remedial measure but it is not economical.

- Treatment or replacement of the upper 1-3m with non-expansive soil
- Excavation with or without replacement of the soil by a compacted non-expansive soil.
- Flooding in place to achieve swelling prior to construction.

- Control of compaction, water content and density.
- Mixing soil with lime or cement before compaction.
- Using a bearing pressure that is high enough to balance the swell pressure.
- Use of procedure to minimize change in soil water content, including adequate drainage system and water proofing of adjacent surface areas.

1.1 Statement of the problem

The stress caused by alternate heaving and shrinkage of the foundation soil creates stress on the structures. Usually, structures are not designed to withstand this stress or it is difficult to account quantitatively. As a result, the structures are damaged due to the additional stress. Damage due to expansive soils could occur on any type of infrastructure, buildings, road and other civil Engineering constructions that are not properly designed and/or constructed. However light structures like one or two story buildings, warehouses, retaining walls and buried facilities are more vulnerable to damages because these structures could not exert a heavy pressure to counteract the uplift pressure from the expansive soils.

These damages are manifested through crack of floors and walls, stacked windows and doors, bulged floors and tilted walls and structures. The magnitude of the damages can be extended even to the extent of failure of one or the all structure by decreasing the structural safety of the building. Cracks do not only affect the structural safety and beauty of the building but also creates financial burden to the owners for maintenance and repair cost can exceed the original cost of the foundation. Generally, the damage will create economic loss for building owners. Therefore, in this study more emphasis is given to lightweight buildings founded on expansive soil, to avoid the possibility of confusing damages caused by structural faulty design and settlement, which encounter in lightweight buildings.

1.2 Objective of the Study

1.2.1 General Objective

This study concentrates on the design and construction of micro piles in expansive soil as alternative foundation system.

1.2.2 Specific Objectives

- Comparison of micro pile and isolated footing in terms of feasibility, cost, and construction period.
- To check micro pile foundation system feasibility and economy in expansive soil.

1.3 Significance of the Study

There were different researches done on the properties of expansive soil of Ethiopia. However, there is no research done on micro pile, as remedial measure for lightweight structure on expansive soil. The result of this study will be used as a base for findings, remedial measures for new buildings and helps in establishing good design and construction procedure for new construction on expansive soil using micro pile as alternative foundation system.

1.4 Scope and Limitation of the Study

The research addresses the general objectives and specific objective and investigate better foundation system for lightweight structure based on the existing theories and principles. This study not includes effect of ground water table and the cost effectiveness of micropiles will typically be limited to projects with one or more significant geological, environmental, access or performance challenges.

1.5 Organization of the Thesis

The presentation of this thesis work is organized in five chapters. The first Chapter gives a brief description of the thesis background, objectives, scope and limitation. Chapter two presents conceptual background on expansive soils, isolated footing and micro pile. Chapter three presents the characterization of materials and methods used. The fourth chapter briefly describe discussion on the results, and finally conclusions and recommendations drawn from the research are presented in Chapter five.

2 LITERATURE REVIEW

2.1 Introduction

There are a number of issues related to the structures constructed on an expansive soil. It is prone to damage due to swell-shrink behavior of the soil, usually lightweight structures that are not as efficient as high-rise structures to resist the variation of soil strata underneath them. The swell-shrink behavior of a soil is a slow process and cannot be noticed by naked eye but it has a devastating effect on the structure. Therefore, immediate attention should be given to such issues, as the cost of introducing a remedial measure is much less, than the cost incurred due to uplift or settlement of a structure. It has been mentioned that the annual loss to structures due to expansive soils is about £400 million a year in the United Kingdom (10). According to the hazard mitigation plan (36), United States of America faces a total loss of $3.2 billion per annum due to expansive nature of the soils whereas China's experience $1 billion annually (38).

The lowest part of a structure is generally referred to as the foundation. Its function is to transfer the load of the structure to the soil on which it is resting. A properly designed foundation is one that transfers the load throughout the soil without over-stressing the soil. Over-stressing the soil can result in either excessive settlement or shear failure of the soil, both of which cause damage to the structure. Thus, geotechnical and structural engineers who design foundations must evaluate the bearing capacity of soils. Depending on the structure and soil encountered, various types of foundations are used. Spread footings and mat foundations are generally referred to as shallow foundations, and pile and drilled shaft foundations are classified as deep foundations.

A new type of foundation known as Micro piles is mainly used as elements for foundation supports, to resist both static and seismic loading conditions and less frequently as in-situ reinforcements for slope and excavation stability (5). It can withstand axial and/or lateral loads and may be considered as substitute for conventional piles (Fig.2.1) or as one component in a composite soil/pile mass, depending upon the design concept employed (18). Micro piles can be installed at any angle below the horizontal using the same type of equipment used for the installation of ground anchors and for grouting projects.

Figure 2-1: Micro piles withstand axial and/or lateral loads. (Iran, Mahmoudabad, Narenjestan-Southern site) (34).

2.2 Types of Foundation

Foundation is a supporting portion of a structure located below the structure and it is supported only by soil or rock. It is mainly used to support the structure and then distribute the weight of that structure, so it settles to the ground evenly rather than unevenly. Each individual foundation must be sized so that the maximum soil bearing pressure does not exceed the allowable soil bearing capacity of the under lying soil mass. The type of footing for particular structure is influenced by many factors that may control type of footing to be used, these main factor are the strength and compressibility of various soil strata at the site, the magnitude of the column pad, the position of water table and the depth of footing of adjacent buildings.

In addition, footing settlement must not exceed tolerable limits established for differential and total settlement. There are two main types of foundations:

a. Shallow Foundations.

b. Deep Foundations.

a. Shallow foundations

To perform satisfactory, shallow foundations must have two main characteristics:

1. They have to be safe against overall shear failure in the soil that supports them.
2. They cannot undergo excessive displacement, or settlement. (The term excessive is relative, because the degree of settlement allowed for a structure depends on several considerations).

The most common structural foundation in today's construction industry is the shallow foundation. Shallow foundation are those founded near to the finished ground surface; generally where the foundation depth (D_f) is less than or equal to (4) of the foundation width. These are not strict rules, but merely guide lines; if surface loading or other surface conditions will affect the bearing capacity of a foundation it is then shallow. Shallow foundation include spread foundation (carrying a single column), combined foundation, continuous foundation and mat (raft) foundation. The advantage of using Shallow foundations are; follow Simple construction procedure, Affordable cost, Available material (mostly concrete) and does not need experts (labors).

The disadvantage of using Shallow foundation are: Differential Settlement can occur, foundation is subjected to pull off out, torsion, and moment, irregular ground surface (slope), it require large mass of concrete, it require large areas and mass excavation, tying the rebar's is time consuming, limits underground utility access, if weight of structure is high and load of the structure is distributed unequally, the bearing capacity of top surface soil is less not used, if sub-soil water level is high and it is uneconomical to pump out the water from the hole or canal, if there is a chance of scouring as the structure is near sea or river shallow foundation cannot be used.

b. deep foundation

In some cases shallow foundations are inadequate to support the structural loads and deep foundations are required i.e. pile foundation. A pile is a slender structure member installed in the ground to transfer the structural loads to soil at some significant depth below the base of the structure (29). Structural loads include axial loads, lateral loads, and moment.

Pile foundation are used when:

- The soil near the surface does not have sufficient bearing capacity to support the structural loads.
- The estimated settlement of the soil exceeds tolerable limits (i.e. settlement greater than the serviceability limit state).
- Differential settlement due to soil variability or non-uniform structure loads is excessive
- The structural loads consist of lateral loads and /or uplift forces.
- Excavation to construct a shallow foundation on a firm soil are difficult or expensive.

Piles can consist of wood (timber), steel H-sections, precast concrete, cast-in-place concrete, pressure injected concrete, concrete filled steel pipe piles, and composite type piles (32). Piles are either driven into-place or installed in predrilled holes. Piles that are driven into place are generally considered low displacement or high displacement, depending on the amount of soil that must be pushed out of the way as the pile is driven. Examples of low displacement piles are steel H-sections and open-ended steel pipe piles that do not form a soil plug at the end. Examples of high-displacement piles are solid section piles, such as round timber piles or square precast concrete piles, and steel pipe piles with a closed end. A cast-in-place pile is formed by making a hole in the ground and then filling the hole with concrete. In its simplest form, the cast-in place pile consists of an uncased hole that is filled with concrete. If the soil tends to cave into the hole, then a shell-type pile can be installed. This consists of driving a steel shell or casing into the ground. The casing may be driven with a mandrel, which is then removed, and the casing is filled with concrete. In other cases, the casing can be driven into place and then slowly removed as the hole is filled with concrete (32).There are various type of pile in terms of their support capacity,

i. End-bearing pile:-a pile the support capacity of which is derived principally from the resistance of the foundation material on which the pile tip rests. It is used when dense or hard strata underlie a soft upper layer.
ii. Friction pile: - a pile the support capacity of which is derived principally from the resistance of the soil friction and/or adhesion mobilized along the side of the pile. Friction pile are often used in soft clays where the end-bearing resistance is small because of punching shear at the pile tip. It can also resist upward loads.
iii. Combined end bearing and friction pile:-a pile derives it support capacity from combined end-bearing resistance developed at the pile tip and frictional resistance on the pile perimeter.
iv. Batter pile: - a pile driven in at an angle inclined to the vertical to provide high resistance to lateral loads.

2.3 Micro pile Definition and Description

Micro piles were conceived in Italy in the early 1950s, in response to the demand for innovative techniques for underpinning historic buildings and monuments that had sustained damage with time, and especially during World War II. A reliable underpinning system was required to support structural loads with minimal movement and for installation in access restrictive environments with minimal disturbance to the existing structure. An Italian specialty contractor called Fondedile, for whom Dr. Fernando Lizzi was the technical director, developed the palo radice, or root pile, for underpinning applications. The palo radice is a small-diameter, drilled, cast-in-place, lightly reinforced, grouted pile. The use of root piles grew in Italy throughout the 1950s. Fondedile introduced the technology in the United Kingdom in 1962 for the underpinning of several historic structures, and by 1965, it was being used in Germany on underground urban transportation projects. For proprietary reasons, the term "micro pile" replaced "root pile" at that time and Portions of Asia soon became a major market. Fondedile introduced the use of micro piles in North America in 1973 through a number of underpinning applications in the New York and Boston areas.

In general, a micro pile is a small-diameter (typically less than 300 mm), drilled and grouted replacement pile (19) that is typically reinforced. It can be constructed by drilling a borehole, placing the reinforcement, and grouting the hole (18). The construction method can be changed into steps including driving a casing, fixing the casing and grouting the hole, placing the reinforcement and installing a flange for better connection with the cap as illustrated in Fig.2.2.

Figure 2-2: The recommended method for the micro pile construction: a) driving a casing, b) fixing the casing and grouting, c) placing the reinforcement and d) installing the flange (34).

There are various types of micro piles according to application, but all of them have similar parts in their bodies including: 1) the inner part e.g. reinforcements and the grout, 2) the casing e.g. a pipe, a hollow bar (Fig. 2.3), 3) the neat cement grout, 4) the soil/grout mix and 5) the densified ground.

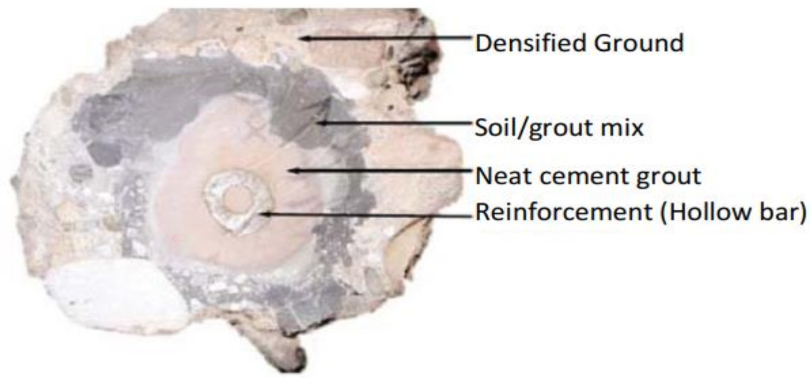

Figure 2-3: A sample of Micro Pile section view (1).

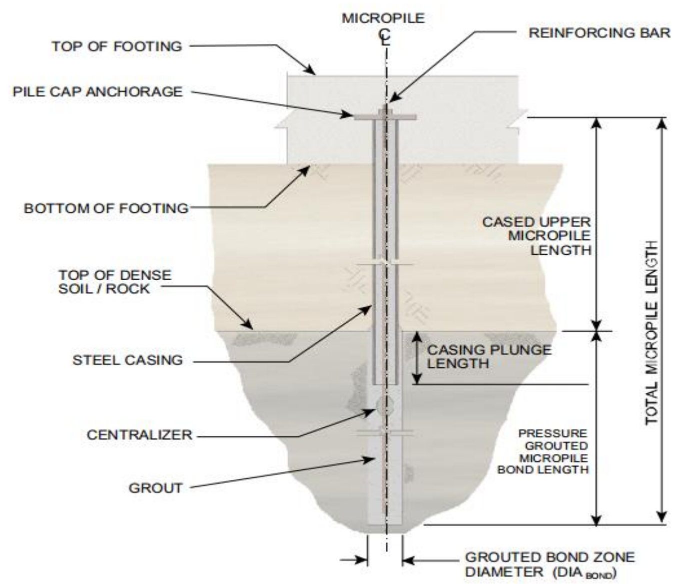

Figure 2-4: Detail Section View of Micro Pile (17).

In the micro pile design procedure there are three principal steps including: 1) Internal–structural, 2) External–geotechnical and 3) Connection of pile to structure (8). To have a comprehensive study in micro pile design, it is recommended to follow the following steps (17):

1. Review available project information (e.g. requirements of the job, pile-loading requirements, and pile layout constraints. Special conditions such as, available access and overhead clearance, presence of hazardous materials, environmental constraints, contractual requirements).

2. Review geotechnical data (e.g. obtain geotechnical/geological subsurface profile, estimate geotechnical design parameters, and obtain soil properties that determine corrosion protection requirements, identify problem areas if any).
3. Complete initial geotechnical pile design (e.g. estimate load transfer parameters (grout-to-ground bond) for the different subsurface layers and determine the pile bond length required to support the loading, evaluate pile spacing for impact to geotechnical capacity from group effects).
4. Complete pile structural design for the various component (e.g. pile cased length structural capacity (bar and/or pipe reinforcement with grout), pile uncased length structural capacity (bar reinforcement with grout), grout to steel bond capacity, transition between reinforcement types (cased to uncased section, strain compatibility between structural components ductility, reinforcement splice connections (bar and/or pipe reinforcement), pile to footing connection).
5. Complete combined geotechnical and structural design considerations (e.g. anticipated settlement/required stiffness analysis, lateral load capacity/anticipated lateral displacement and combined stresses (axial + bending) due to lateral loading conditions, buckling of the pile/soil lateral support considerations).
6. Complete additional micro pile system considerations (e.g. corrosion protection requirements, determine construction load testing and quality control program requirements, examine constructability and cost effectiveness of the design) (17).

Most of the applied load on conventional cast-in-place drilled or non-displacement piles is structurally resisted by the reinforced concrete; increased structural capacity is achieved by increased cross-sectional and surface areas. Micro pile structural capacities, by comparison rely on high-capacity steel elements to resist most or the entire applied load. These steel elements may occupy as much as one-half of the drill hole cross section. The special drilling and grouting methods used in micro pile installation allow for high grout/ground bond values along the grout/ground interface. The grout transfers the load through friction from the reinforcement to the ground in the Micro pile bond zone in a manner similar to that of ground anchors. Due to the small pile diameter, any end-bearing contribution in micro piles is generally neglected. The grout/ground bond strength achieved is influenced primarily by the ground type and grouting method used, i.e., pressure grouting or gravity feed.

2.3.1 Micro Pile Classification System

In 1997, the FHWA published a 4-volume report summarizing the state-of-the-practice for micro piles (FHWA-RD-96-016, –017, -018, and –019; 1997). In that report, a micro pile classification system was developed. This system is based on two criteria: (1) philosophy of behavior (design); and (2) method of grouting (construction). The philosophy of behavior dictates the method employed in designing the micro pile. The method of grouting defines the grout/ground bond strength (or side resistance), which generally controls micro pile capacity.

2.3.1.1 Design Application Classification

The design of an individual micro pile or a group of micro piles differs greatly from that of a network of closely spaced reticulated micro piles. This difference led to the definition of CASE 1 micro pile elements which are loaded directly and where the micro pile reinforcement resists the majority of the applied load (Figure 2.5).

CASE 2, micro pile elements circumscribe and internally reinforce the soil theoretically to make a reinforced soil composite that resists applied loads (Figure2.6). This is referred to as a reticulated micro pile network. CASE 1 micro pile can be used as an alternative to more conventional types of piles since they are used to transfer structural loads to a deeper, more competent or stable stratum. Such directly loaded piles, whether for axial (compression or tension) or lateral loading conditions, are referred to as CASE 1 elements. The load is primarily resisted structurally by the steel reinforcement and geotechnical by side resistance developed over a “bond zone” of the individual micro piles. The remaining micro pile applications involve networks of reticulated micro piles as components of a reinforced soil mass that is used to provide stabilization and support. These micro piles are referred to as CASE 2 elements. Structural loads are applied to the entire reinforced soil mass, as compared to individual micro piles (as for CASE 1 micro piles). CASE two micro piles are lightly reinforced because they are not individually loaded.

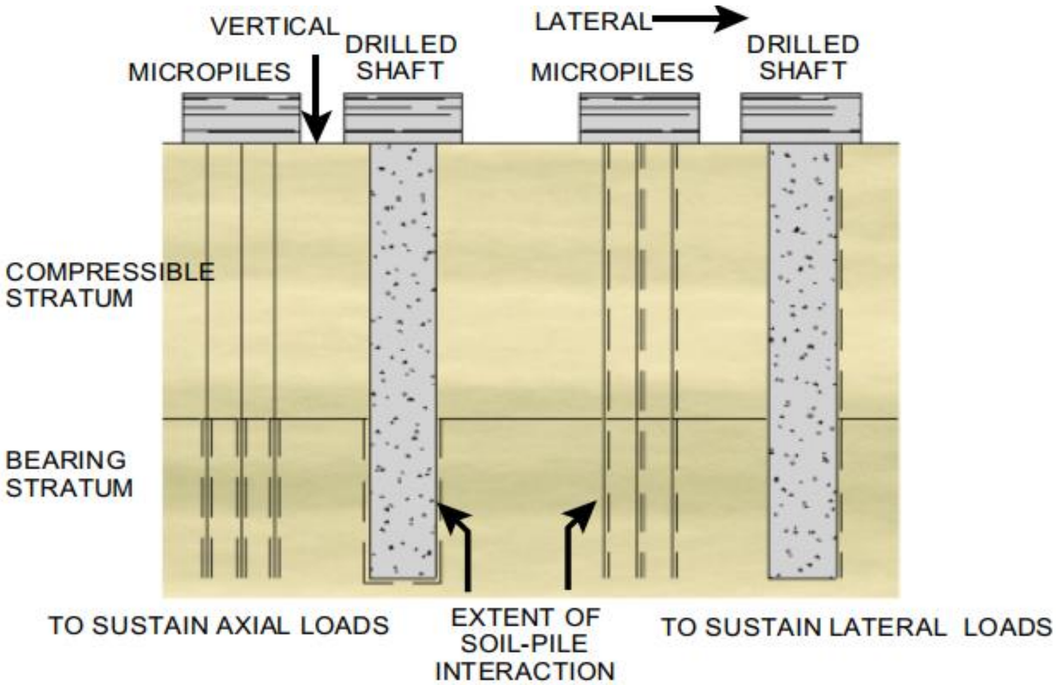

Figure 2-5: CASE 1 Micro piles (18).

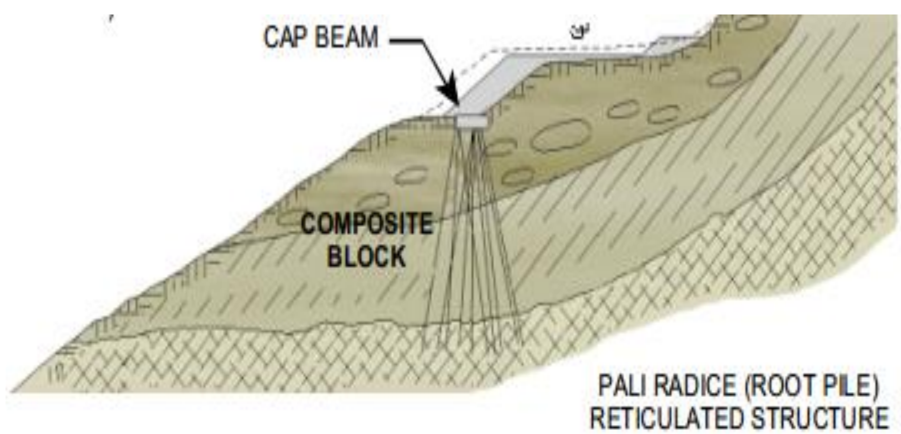

Figure 2-6: CASE 2 Micro piles – Reticulated Micro pile Network (18).

2.3.1.2 Construction Type Classification

The method of grouting is typically the most sensitive construction process influencing grout/ground bond capacity. Grout/ground bond capacity varies directly with the grouting method. The second part of the micro pile classification consists of a letter designation (A through D) based primarily on the method of placement and pressure under which grouting is performed during construction. The use of drill casing and reinforcement define sub classifications.

Type A**:** For this type micro pile, grout is placed under gravity head only. Sand-cement mortars or neat cement grouts can be used. The micro pile excavation may be under reamed to increase tensile capacity, although this technique is not common or used with any other micro pile type.

Type B**:** Type B indicates that neat cement grout is placed into the hole under pressure as the temporary drill casing is withdrawn. Injection pressures typically range from 0.5 to 1MPa to avoid hydro fracturing the surrounding ground or causing excessive grout takes, and to maintain a seal around the casing during its withdrawal, where possible.

Type C**:** Indicates a two-step process of grouting including: (1) neat cement grout is placed under gravity head as with Type A; and (2) prior to hardening of the primary grout (after approximately 15 to 25 minutes). Similar grout is injected one time via a sleeved grout pipe without the use of a packer (at the bond zone interface) at a pressure of at least 1MPa. This pile type appears to be used only in France, and is referred to as IGU (Injection Globale Unitaire).

Type D**:** Type D indicates a two-step process of grouting similar to Type C. With this method, neat cement grout is placed under gravity head (as with Types A and C) and may be pressurized (as for Type B). After hardening of the initially placed grout, additional grout is injected via a sleeved grout pipe at a pressure of 2 to 8MPa. A packer may be used inside the sleeved pipe so that specific horizons can be treated several times, if required. This pile type is used commonly worldwide, and is referred to in France as the IRS (Injection Répétitive et Sélective).

Table 2-1: Provides Additional Information on Type A, B, C, and D Micro piles.

Micro pile type and grouting method	Sub-type	Drill casing	Reinforcement	Grout
Type A Grout gravity only	A1	Open hole or auger	None ,single bar, cage, tube or structural section	Sand/cement mortar or neat cement grout trimmed to base of hole, no excess pressure applied
	A2	Permanent , full length	Drill casing itself	
	A3	Permanent, upper shaft only	Drill casing in upper shaft, tube bar(s) in lower shaft or may extend full length	
Type B Pressure grouted through the casing or auger during withdrawal	B1	Temporary or unlined (open hole or auger)	Mono bars or tube (cages rare due to lower structural capacity)	Neat cement grout is first tremied into drill casing/ auger excess pressure up to 1Mpa is applied to additional grout injected during withdrawal of casing/auger
	B2	Permanent or partial length	Drill casing itself	
	B3	Permanent upper shaft only	Drill casing in upper shaft, tube bar(s) in lower shaft or may extend full length	
Type C Primary grout placed under gravity head, then one phase of second “global” pressure grouting	C1	Temporary or unlined (open hole or auger)	Single bars or tube (cages rare due to lower structural capacity	Neat cement grout is first tremied into hole (or casing/ auger). Between 15 to 25 minutes later, similar grout injected through tubes or reinforcing pipe from head, once pressure is greater than 1Mpa.
	C2	Not conducted	-	
	C3	Not conducted	-	
Type D Primary grout placed under gravity head, (type A) or under pressure (type B). Then one or more phases of secondary “global” pressure grouting	D1	Temporary or unlined (open hole or auger	Single bars or tube (cages rare due to lower structural capacity)	Neat cement grout is first tremied (type A) and /or pressurized (type B) into hole or casing/ auger. several hours later, similar grout injected through sleeved pipe (or sleeved reinforcement) via packers, as many times as necessary to achieve bond
	D2	Possible only if regrout tubes placed full length outside casing.	Drill casing itself	
	D3	Permanent upper shaft only	Drill casing in upper shaft, tube bar(s) in lower shaft or may extend full length	

2.3.2 Micro pile Applications

Micro piles are currently used in two general application areas: (1) structural support; and (2) in situ reinforcement (Figure 2.7). Structural support includes new foundations, underpinning of existing foundations, seismic retrofitting applications and foundation support for earth retaining structures. In-situ reinforcement is used for slope stabilization and earth retention; ground strengthening; settlement reduction; and structural stability.

Table 2-2: Summary of the Typical Design Behavior and Micro Pile Construction Type for Each Application.

	STRUCTURAL SUPPORT	IN-SITU EARTH REINFORCEMENT			
Application	Underpinning of existing foundation, new foundation , and seismic retrofitting	Slope stabilization and earth retention	Ground strengthening	Settlement reduction	Structural stability
Design behavior	CASE 1	CASE 1 and CASE 2	CASE 2 with minor CASE 1	CASE 2	CASE 2
Construction type	Type A (Bond zone in rock or stiff clays)	Type A and type B in soil	Type A and type B in soil	Type A in soil	Type A in soil
Frequency of use	Probably 95% total world application	0 to 5%			

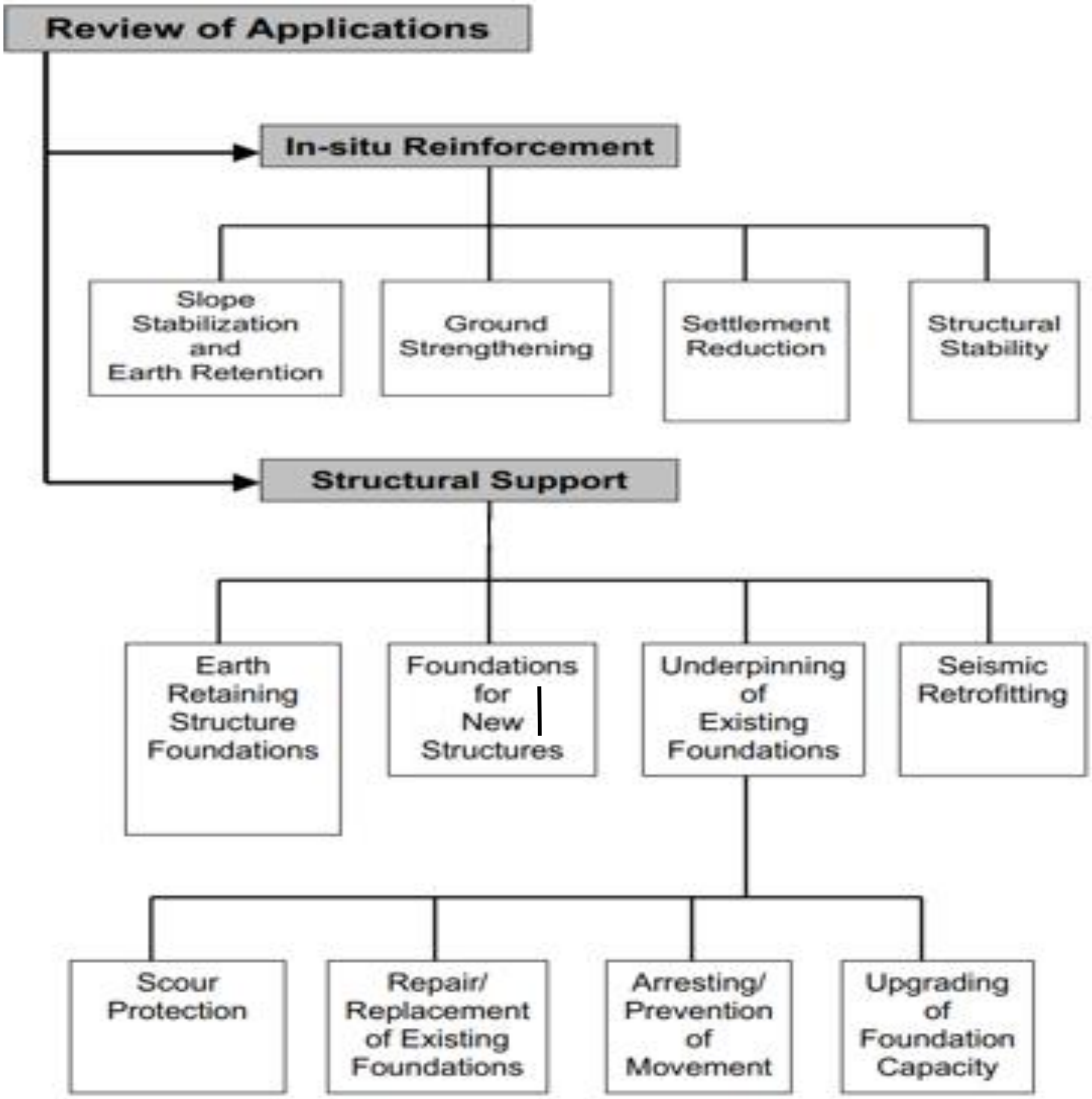

Figure 2-7: Classification of Micro pile Application (18).

2.3.3 Feasibility of Micro piles

Micro piles have specific advantages compared to more support systems that are conventional. In general, micro piles may be feasible under the following project-specific constraints:

- project has restricted access or is located in a remote area;
- required support system needs to be in close pile proximity to existing structures;
- Ground and drilling conditions are difficult (e.g., karstic areas, uncontrolled fills, boulders);
- pile driving would result in soil liquefaction;

- vibration or noise needs to be minimized;
- hazardous or contaminated spoil material will be generated during construction; and
- Adaptation of support system to existing structure is required.

2.3.3.1 Physical Considerations

The drilling and grouting equipment used for micro pile installation is relatively small and can be mobilized in restrictive areas that would prohibit the entry of conventional pile installation equipment. Micro piles can be installed in close proximity to existing walls or foundations, if there is space above for the drill-head and safe work zone or the micro piles are battered to provide this space. Micro pile installation is not as affected by overhead power lines or other obstructions as are conventional pile installation systems. The equipment can be mobilized up steep slopes and in remote locations. In addition, drilling and grouting procedures associated with micro pile installations do not cause damage to adjacent existing structures or affect adjacent ground conditions when proper drilling and grouting procedures are utilized.

2.3.3.2 Subsurface Conditions

Micro piles can be installed in areas of particularly difficult, variable, or unpredictable geologic conditions such as ground with cobbles and boulders, fills with buried utilities and miscellaneous debris, and irregular lenses of competent and weak materials. Soft clays, running sands, and high groundwater not conducive to conventional drilled shaft systems cause minimal impacts to micro pile installations. Micro piles are commonly used in karst limestone formations.

2.3.3.3 Environmental Conditions

Micro piles can be installed in hazardous and contaminated soils. Because of their small diameter, drilling results in less spoil than would be produced by conventional drilled piles. In addition, the flush effluent can be controlled easily at the ground surface through containerization or the use of lined surface pits. These factors greatly reduce the potential for surface contamination and handling costs.

Grout mixes can be designed to withstand chemically aggressive groundwater and soils. Special admixtures can be included in the grout mix design to reduce and avoid deterioration from acidic and corrosive environments.

Micro piles can be installed in environmentally sensitive areas, including areas with fragile natural settings. The installation equipment is not as large or as heavy as conventional pile driving or shaft drilling equipment and can be used in swampy areas or other areas of wet or soft surface soils with minimal impacts to the environment. Portable drilling equipment is frequently used in areas of restricted access.

2.3.4 Micro pile Limitations

Vertical micro piles may be limited in lateral capacity and cost effectiveness. The ability of micro piles to be installed on an incline, however, significantly enhances their lateral capacity. Because of their high slenderness ratio (length/diameter), micro piles may not be acceptable for conventional seismic retrofitting applications in areas where liquefaction may occur due to concerns of buckling resulting from loss of lateral support. The use of micro piles for slope stabilization continues to increase. However, it is recommended that performance data be collected on such projects as experience and detailed design procedures continue to evolve.

2.3.5 Economics of Micro piles

Cost effectiveness of micro piles depends on many factors. It is important to assess the cost of using micro piles based on the physical, environmental, and subsurface factors previously described. For example, for an open site with soft, clean, uniform soils and unrestricted access, micro piles will likely not be a competitive solution. However, for the delicate underpinning of an existing bridge pier in a heavily trafficked old industrial or residential area, micro piles can provide the most cost-effective solution.

2.3.6 Advantage and disadvantage of Micro pile

The advantages of micro piles are:

- The equipment needed for installation is smaller than other foundation systems allowing the use of micro piles in limited access and low headroom condition.
- The installation method for micro piles creates relatively little disturbance to the surrounding area making them ideal for use in environmentally sensitive areas, or indoors for remedial support.

- Due to the higher cross section area of reinforcing steel relative to other foundation systems, micro piles provide a significant increase in lateral or shear capacity.
- Micro piles are not end bearing and as such are not limited to locations with competent rock to develop their load bearing capacity.
- Micro piles do not have to be directly loaded by the structure, but can be used as a passive reinforcement of the surrounding soil mass to improve the foundation materials.
- As micro piles are a drilled replacement foundation, they can be installed in areas of soil with floating or pendant boulders that would make driven piles impossible to install.
- Only small volume of earth to be excavated due to small diameter
- High flexibility during seismic condition

Disadvantages of micro pile are:

- Load can only be applied after grout curing.
- Micro piles can be more costly than other pile systems in certain cases.
- Difficult to design for large lateral loads.

Advantages over conventional piles**:**

- Works in compression and tension
- Does not require temporary casing
- Improved mechanical ground/grout interaction reduces overall depth
- Dramatically increased production rates
- Lightweight rotary percussive drilling equipment
- Installed in confined spaces
- Permits top down jet grouting in saturated clay sand silts complete with re-bar
- Perfect as pali-radice for invisible structural repairs
- Bayonet fixing eliminates pile trimming and facilitates remote de-coupling
- Self-drilling micro piles provided for large working loads
- Minimal noise

2.3.7 Construction Techniques and Materials

There are a large number of drilling systems available for both overburden and rock, and many are used for micro pile construction. The typical construction sequence for simple Type A and B micro piles (Figure 2-9) includes drilling the pile shaft to the required tip elevation, placing the steel reinforcement, placing the initial grout by tremie, and placing additional grout under pressure (for Type B). In general, the drilling and grouting equipment and techniques used for the micro pile construction are similar to those used for the installation of soil nails, ground anchors, and grout holes.

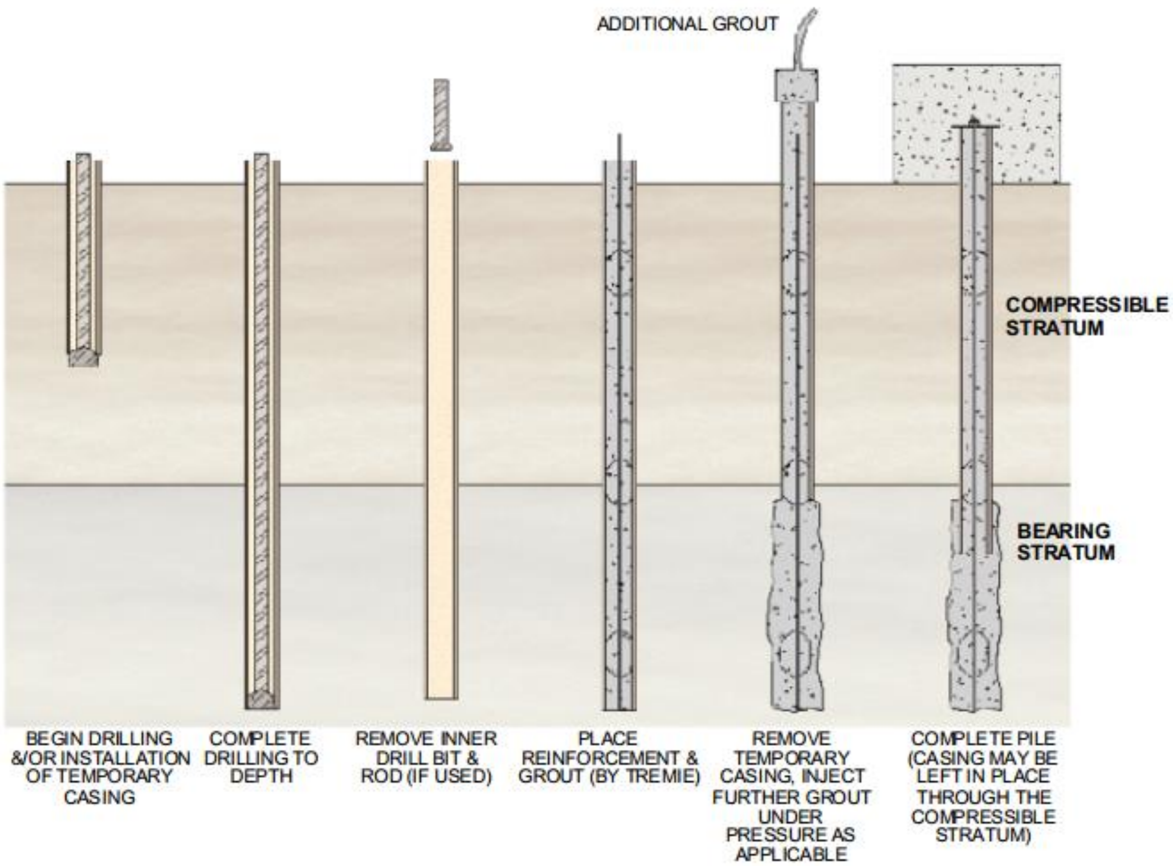

Figure 2-8: Typical micro pile construction sequence using casing (17).

2.3.8 Drilling

The act of drilling and forming the pile excavation may disturb the surrounding ground for a certain time and over a certain distance. The drilling method selected by the contractor should avoid causing an unacceptable level of disturbance to the site and its facilities, while providing for installation of a micro pile that support the required capacities in the most cost effective manner. Drilling, installation of the reinforcement, and grouting of a particular micro pile should be completed in a series of continuous processes and executed as expeditiously as possible. Longer durations between completion of drilling and placing of the reinforcement and grout can be detrimental to the integrity of the surrounding soil. Some materials such as over consolidated clays and clay shales, can deteriorate, relax, or soften because of exposure, resulting in a loss of interfacial bond capacity. In these cases, installation of a micro pile bond zone should be completed within one day to avoid a pile hole remaining open overnight.

2.3.8.1 Drill Rigs

Drill rigs typically used for micro pile installation are hydraulic rotary (electric or diesel) power units. They can be track mounted allowing for maneuverability on difficult and sloped terrain. The size of the track-mounted drills can vary greatly, as seen in Figure 2-9 with the larger drill allowing use of long sections of drill rods and casing in areas without overhead restrictions and the smaller drill allowing work in lower overhead and harder-to-reach locations. The drill mast can be mounted on a frame allowing work in limited-access and low-overhead areas, such as building basements.

Figure 2-9: Large Track-Mounted Rotary Hydraulic Drill Rig (4)

2.3.8.2 Drilling Techniques

2.3.8.2.1 Open Hole Drilling Techniques

When a micro pile can be formed in stable and free-standing conditions, the advancement of casing may be suspended and the hole continued to final depth by open-hole drilling techniques. There is a balance in cost between the time lost in changing to a less-expensive open-hole system and continuing with a more expensive overburden drilling system for the full hole depth. Open-hole drilling techniques may be classified as follows:

2.3.8.2.2 Rotary Percussive Drilling

Particularly for rocks of high compressive strength, rotary percussive techniques using either top drive or down-the-hole hammers are utilized. For the small whole diameters used for micro piles, down-the-hole techniques are the most economical and common. Air, air/water moist, or foam is used as the flush. Top-drive systems can also use air, water, or other flushing systems, but have limited diameter and depth capacities, are relatively noisy, and may cause damage to the structure or foundation through excessive vibration.

2.3.8.2.3 Solid Core Continuous Flight Auger

In stiff to hard clays without boulders and in some weak rocks, drilling may be conducted with a continuous flight auger. Such drilling techniques are rapid, quiet, and do not require the introduction of a flushing medium to remove the spoil. There may be the risk of lateral decompression or wall remolding/interface smear, of which may adversely affect grout-to-ground bond. Such augers may be used in conditions where the careful collection and disposal of drill spoils are particularly important.

2.3.8.2.4 Under reaming

Various devices have been developed to enlarge or under ream open holes in cohesive soils or soft sediments, especially when the micro piles are to act intension. These tools can be mechanically or hydraulically activated and will cut or abrade single or multiple under reams or “bells.” However, this is a time-consuming process, and it is rarely possible or convenient to verify its effectiveness. In addition, the cleaning of the under reams is often difficult; water is the best cleaning medium, but may cause softening of the ground. For all these reasons, it is rare to find under reaming used in typical micro pile practice.

2.3.8.3 Grouting

- Grouts are designed to provide high strength and stability (i.e., bleed), but must also be pumpable. As shown in Figure 2-10, this implies typical water/cement (w/c) ratios in the range of 0.40 to 0.50 by weight for micro pile grout. In Figure 2-10, bleed capacity is defined as amount of free water that develops at final set. For example, for a w/c ratio of1.0, 100 liters of grout would develop 20 liters of free water.
- Grouts are produced with potable water, to reduce the danger of reinforcement corrosion.
- Type I/II cement conforming to ASTM C150/AASHTO M85 is most commonly used, supplied either in bagged or bulk form depending on site condition, job size, local availability, and cost.
- Neat cement-water grout mixes are most commonly used, although sand is a common additive in certain countries (e.g., Italy, Britain). Bentonite (which reduces grout strength) is used in primary mixes only with extreme caution, while additives are allowed only for cases where improved pumpability is required as for pumping over long distances and/or in hot conditions (e.g., high-range water reducers).
- Design compressive strengths of 28 to 35 MPa can be attained with properly produced neat cement grouts.
- If admixtures and/or additives are used, it is essential that they are chemically compatible. This is best achieved by using only one chemical supplier, and not by "mixing and matching". The placed grout is required to serve a number of purposes, as described below.
- It transfers the imposed loads between the reinforcement and the surrounding ground.
- It may form part of the load-bearing cross section of the micro pile.
- It serves to protect the steel reinforcement from corrosion
- Its effects may extend beyond the confines of the drill hole by permeation, densification, and/or fissuring.

The grout, therefore, needs to have adequate properties of fluidity, strength, stability, and durability. The need for grout fluidity can mistakenly lead to an increase in water content , which has a negative impact on the other three properties. Of all the factors that influence grout fluidity and set properties, the water/cement ratio is the most important. Again, Figure

2-10 illustrates why this ratio is limited to a range of 0.40 to 0.50, although even then, additives may be necessary to ensure adequate pump ability for ratios less than 0.40.

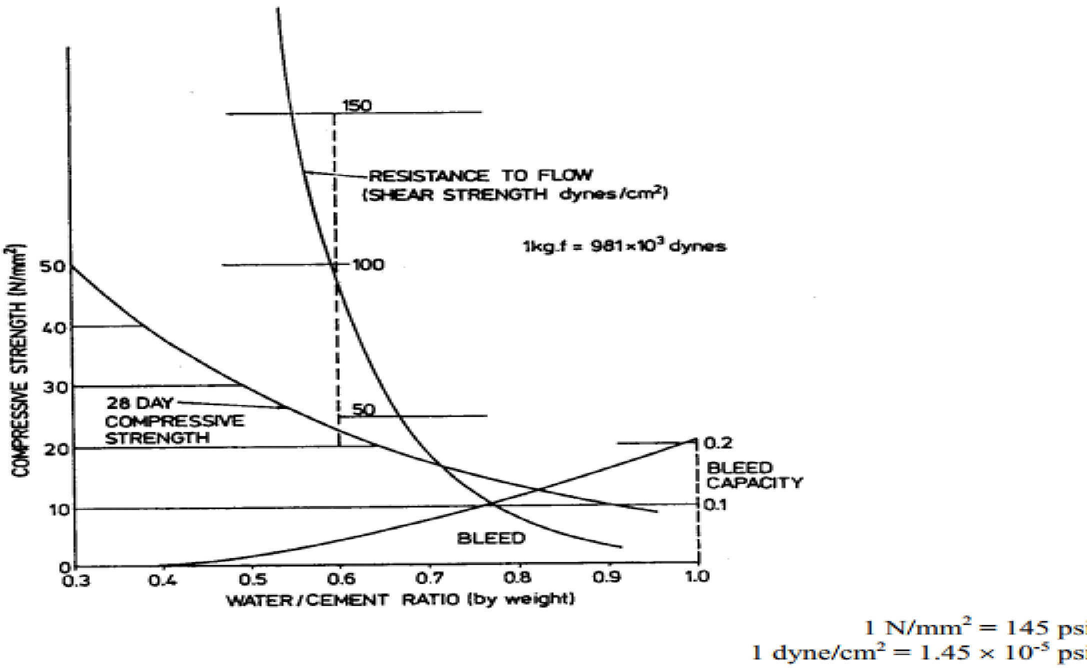

Figure 2-10: Effect of Water Content on Grout Compressive Strength and Flow Properties (25).

2.3.8.4 Grout Equipment

In general, any plant suitable for the mixing and pumping of fluid cementitious grouts may be used for the grouting of micro piles. Mixing equipment can be driven by air, diesel, or electricity, and is available in a wide range of capacities and sizes from many manufacturers.

2.3.8.5 Grout Mixing

During mixing, a measured volume of water is usually added to the mixer first, followed by cement and then aggregate or filler (if applicable). It is generally recommended that grout be mixed for a minimum of two minutes and that thereafter the grout is kept in continuous slow agitation in a holding tank prior to being pumped to the micro pile. The “safe workability” time should be determined on the basis of on-site tests, but is typically not in excess of one hour.

2.4 Reinforcing Steel

The amount of steel reinforcement placed in a micro pile is determined by the loading it supports and the stiffness required limiting elastic displacements. Reinforcement may consist of a single reinforcing bar, a group of reinforcing bars, a steel pipe casing or rolled structural steel. In American practice, the use of a single reinforcing bar and/or a high-strength steel casing is commonly used.

2.4.1 Reinforcement Types

A. Concrete Reinforcing Steel Bars (rebar)

Standard reinforcing steel (Table 2.3), conforming to ASTM A615/AASHTO M31and ASTM A706, with yield strengths of 420 and 520MPa, is typically used. Bar sizes range in diameter from25 to 63mm. Single bars is typically used, but bar groups have been used. For a bar group, the individual bars can be separated by the use of spacers or tied to the helical reinforcement, to provide area for grout to flow between the bars and ensure adequate bonding between the bars and grout.

B. Continuous-Thread Steel Bars

Steel reinforcing bars may be fabricated with a continuous full-length thread, such as the Dywidag Systems International (DSI) Thread bar or the Williams All-Thread Bar. The bar has a coarse pitch, continuous ribbed thread rolled on during production. It is available in diameters ranging from 19 mm to 63 mm in steel conforming to ASTM A615/AASHTO M 31, with yield strengths of 420,520, and 550MPa. The size range of 32 mm to 63 mm is most commonly used.

C. Continuous-Thread Hollow-Core Steel Bars

Steel reinforcing bars that have a hollow core and a continuous full-length thread include the Dwyidag, Ischebeck Titan, MAI International, and Chance IBO Injection Boring Rods. Different sizes are available. The continuous thread allows the bar to be cut to length and coupled, and allows the use of a hex nut for the pile top connection. The main drawback of this type of reinforcement is the higher cost.

D. Steel Pipe Casing

With the trend towards micro piles that can support higher loads at low displacements and for the requirement to sustain lateral loads, steel-pipe reinforcement has become more common. Pipe reinforcement can provide significant steel area for support of high loading and contribution to the micro pile stiffness, while providing high shear and reasonable bending capacity to resist the lateral loads. Pipe reinforcement is placed by either using the drill casing as permanent reinforcement, or by placing a smaller diameter permanent pipe inside the drill casing. Use of the drill casing for full-length reinforcement is typical only for micro piles founded in rock, where extraction of the casing for pressure grouting is not necessary.

E. Composite Reinforcement

For micro piles with partial length permanent drill casing (Type C), the use of a steel bar for reinforcement of the bottom portion of the pile is common, resulting in a composite reinforced pile. The reinforcing bar may be extended to the top of the micro pile for support of tension loading.

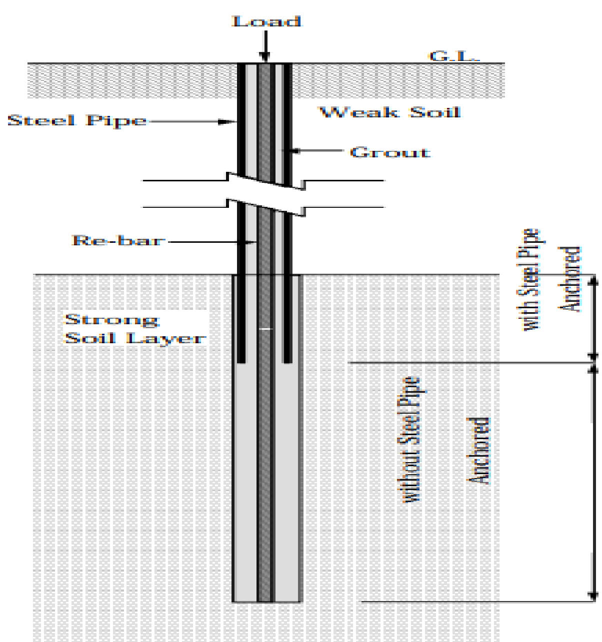

Figure 2-11: Micropile system (17)

Table 2-3: Dimensions, Yield, and Ultimate Strengths for Standard Reinforcing Bars

Steel grade	Rebar size (mm)	Area (mm^2)	Yield strength (kN)
Grade 420(1)	19	284	117
	22	387	160
	25	510	211
Grade 520[(2)]	19	284	147
	22	387	200
	25	510	264
	29	645	334
	32	819	424
	36	1006	520
	43	1452	751
	57	2581	1335
Grade 550[(3)]	63	3168	1747

Note [(1)] grade 420 has yield stress of f_y = 420Mpa and tensile strength of f_u = 620Mpa

[(2)] grade 520 has yield stress of f_y = 520Mpa and tensile strength of f_u = 690Mpa

3 MATERIALS AND METHODS

3.1 General Description of Study Area

The proposed site is located within the existing Eastern Industry zone compound toward side of the main asphalt road. The site characterized by flat topographic feature and approximately rectangular. At the time of fieldwork, the proposed site was fenced, empty and covered with grass. There are no any visible geomorphologic features such as gullies and river cuts.

3.1.1 Site Geology and Subsurface Condition

The proposed G+1 residential building site and its surroundings are covered with CLAY (black cotton), sandy SILT and silty CLAY soil.

3.1.2 Geotechnical Characteristics of Subsurface Layers

Geotechnical investigation carried out for a G+1 building site involved drilling of eight boreholes to a maximum depth of 15.00m, SPT testing, and collection of soil samples for laboratory testing. Ground water measurement made during the fieldwork did not show presence of ground water with in drilled depth. Description of subsurface layers at the building site and relevant engineering parameters are summarized in Table below.

Table 3-1: Summary of Geotechnical Characterization of sub-surface layer

Soil type	Occurrence thickness (m)	Unit weight (kN/m^3)	Normal moisture content (%)	Cohesion (kPa)	specific gravity	Coefficient of compressibility (Cc)
Clay (black cotton)	0-2	16	-	-	-	-
Silty clay with some sand	2-7.5	18	39	55.4,69.54, 60.11,61.29	2.65	0.27
sandy silt	7.5-8.5	17	34	-	-	-
Silty clay	8.5-15	18		-	-	-

Source:-Addis geo-systems plc

3.1.2.1 Ground Water Observation

Measurement of ground water level was made on the days following completion of each borehole drilling operation and ground water was not recorded at any of boreholes.

3.2 Research Methodology

In order to achieve the above objectives of the study the following methodologies were adopted:

I. Literature review: different types of literatures; such as text books, academic journals, seminars and research papers pertaining to expansive soil, isolated footing and micro pile design techniques were reviewed.
II. Obtain soil data from required for design of isolated footing and micro pile from any legal office.
III. Design isolated footing and micro-pile foundation system for the same structure on expansive soil using classical formula.
IV. Analysis and discussion of results: based on the theories, cost comparison and construction period, the results obtained have been analyzed and discussed thoroughly.
V. Formulation of conclusions and recommendations based on the results obtained.
VI. Finally compiling and writing of the thesis work.

3.3 Bearing Capacity Analysis

Determination of bearing capacity of foundation layer for G+1 Building was made by using empirical method based on the results of in-situ test namely Standard Penetration Test (SPT).

Skempton (1951) proposed equation for bearing capacity of footings found on cohesive soils based on extensive investigations, which can be modified to establish a relationship among net allowable bearing capacity, SPT N- value, and bearing capacity factor Nc (27). The bearing capacity factor Nc is a function of depth and shape of the foundation. The allowable bearing capacity using the following equation is 262.5kPa, the calculation is shown in appendix A.

Q_{ultnet}=c*Nc, c = $\frac{q_u}{2}$, qu=12.5*N_{design} 3-1

$$Q_{ult}=\frac{12.5N_{design}}{2}*N_c=6.25N_c*N_{design}$$

$$Nc = 6(1+0.2\frac{Df}{B}) \leq 9 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3\text{-}2$$

The net allowable bearing capacity obtained by dividing of net ultimate bearing capacity to factor of safety FS.

$$Qall = \frac{6.25N_{design*}N_c}{FS} = \frac{6.25N_{design*}N_c}{2.5} = 2.5Nc*N_{design}$$

$$N_{design}=\sum\frac{\frac{N_i}{i*i}}{\frac{1}{i*i}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad 3\text{-}3$$

Table 3-2 determination of corrected and design SPT-value

Field procedure correction				BH 1	BH 2	BH 3	BH 4			weighted		
correction factor	symbol	Value	DEPTH (m)	SPT 1	SPT 2	SPT 3	SPT 4	SPT Average	corrected N value	$\frac{N_i}{i*i}$	$\frac{1}{i*i}$	***N design***
Hammer efficiency	EH	0.6	2.45	15	14	18	20	16.75	12.56	12.56	1	
Bore hole dia factor	CB	1	3.45	25	20	19	17	20.25	15.19	3.797	0.25	
Sampler correction	CS	1	4.45	21	18	22	19	20	15.00	1.667	0.111	
Rod length correction	CR	0.75	5.45	26	23	24	50	30.75	23.06	1.441	0.063	
N55=EH*CB*CS*CR*N/0.55		16.36	6.45	15	34	30	24	25.75	19.31	0.773	0.04	
			7.45	34	25	24	21	26	19.50	0.542	0.028	
Take N55=16								20.65	15.49	20.78	1.49	***13.93***

take N design = 14

Considering sub-surface condition, laboratory test results and bearing capacity determined based on the above method, Isolated Footing foundation system recommended and Place the foundation at average depth of 2.5m below natural ground level (NGL).

3.4 Case One: - Design of Square Isolated Footing

A footing carrying a single column is called a spread footing, since its function is to "spread" the column load laterally to the soil so that the stress intensity is reduced to a value that the soil can safely carry. Assume that the horizontal loads and torsional moments transferred from columns must be very small.

i. Proportioning of footing

The required area of the footing and subsequently the proportions will be determined using presumptive allowable soil pressure and/or the soil strength parameters ϕ and C. In this case it is determined from allowable soil pressure using the following equation.

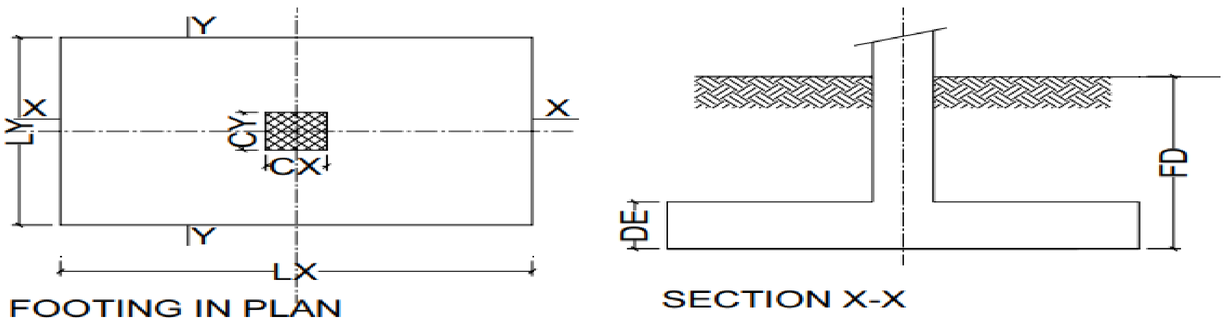

Figure 3-1: Footing plan and section view

$$qall = \frac{P}{B*B} * (\frac{6ex}{B} + \frac{6ey}{B}) \text{ ………………………………………………………………………3.4}$$

Point loads from ETABS output is used to design foundation .Since the forces in the X and Y direction are insignificant as compared with the force in the vertical direction so in this practical project only the vertical load is considered for the design purpose.

Table 3-3: Representative base joint reactions used for footing design

Footing designation	F1			F2										
Loads (kN)	353	314	237	182	173.5	170.3	152.7	121.5	114.9	104.4	97.7	93.5	69.5	60.6

ii. Determination Footing Thickness

Once the required footing area has been determined, the footing must then be designed to develop the necessary strength to resist all moments, shears, and other internal actions caused by the applied loads.

Shear stresses usually control the footing thickness *D*. In addition, there are two modes of shear failure.

1. One-way shear also known as wide beam shear or diagonal compression shear.
2. Two-way shear also known as punching shear or diagonal tension shear.

It should be noted that footing thicknesses are determined for factored service loads and soil pressures.

Calculation of effective depth

$d=\frac{D-0.05-\emptyset}{1000}$ …………………………………………………………………………………...3-4

a) Calculation of Punching Shear Capacity.

It is the normal practice to provide adequate depth to sustain the shear stress developed without reinforcement. The punching shear which is maximum at a distance of 0.5d, where d is the depth of the footing, from the face of the column is compared with the punching shear capacity on the critical section, which is given by:-

P_{RESI}=0.25fcd*k1*k2*U*d…………...…………………………...………………………… 3-5

K1= min (1+50ρ, 2)……….….…….………………………………………………..… 3-6

K2=max (1.6-d, 1)………………………..………………………………………………… 3-7

U =4*(d+a') where a' is column size, U is perimeter……………………………..….….……. 3-8

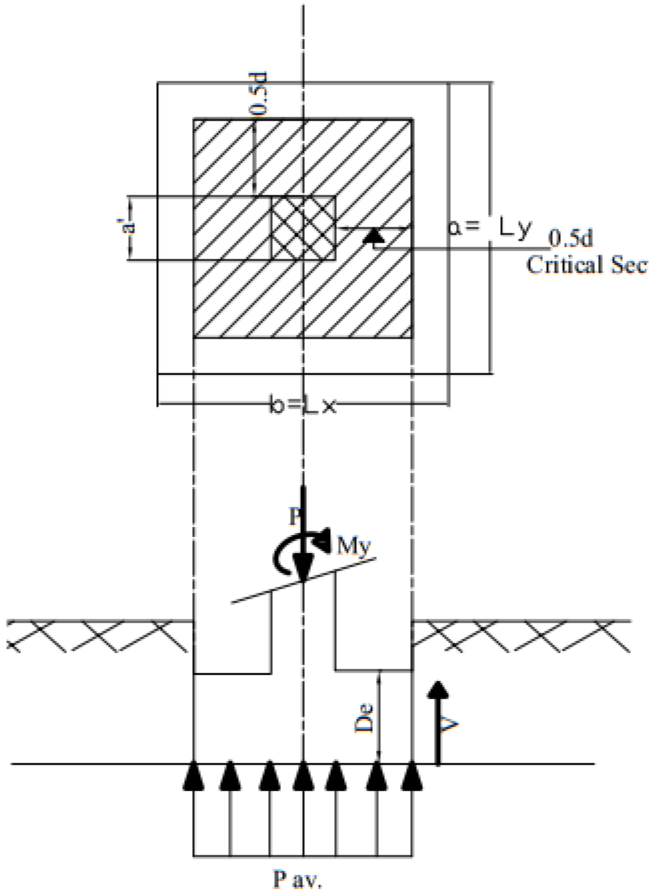

Figure 3-2: Critical section for punching

Calculation of punching shear stress acting on the critical are

$\sigma = \frac{\sigma_1+\sigma_2}{2} * b$…………………………………………………………………………….... 3-9.

Calculation of Average pressure acting on the critical area

$P_{ave} = \frac{\sigma_1+\sigma_2+\sigma_3+\sigma_4}{4}$…………………..……………………………………………..………………… 3-10

$V = P - P_{ave} \leq P_{res}$……………………………………………………..………………… 3-11

b) Calculation of wide beam shear resistance

The selected depth using the punching shear criterion may not be adequate to withstand the diagonal tension developed. Hence, one should also check the safety against diagonal tension and the critical section is located at distance “d” from face of column.

The wide beam shear, which is maximum at a distance of d from the face of the column, is compared with the wide beam shear capacity, which is given by the formula,

P_{RESI}=0.25fcdk_1*k_2*U*d*………………………………………………………………………………. *3-12*

K1= min (1+50ρ, 2)……………………………………………………………………3.12(a)

K2=max (1.6-d, 1)………………………………………………………………………...3.12 (b)

U =4*(d+a') where a' is column size……………………………………………….....3.12 (c)

c) Calculation of Wide Beam Shear Force on the Critical Surface

➢ Calculation of average pressure acting on the critical area

The section must be safe against these two parameters, the design moment is maximum at face of the column, and design is made with this moment.

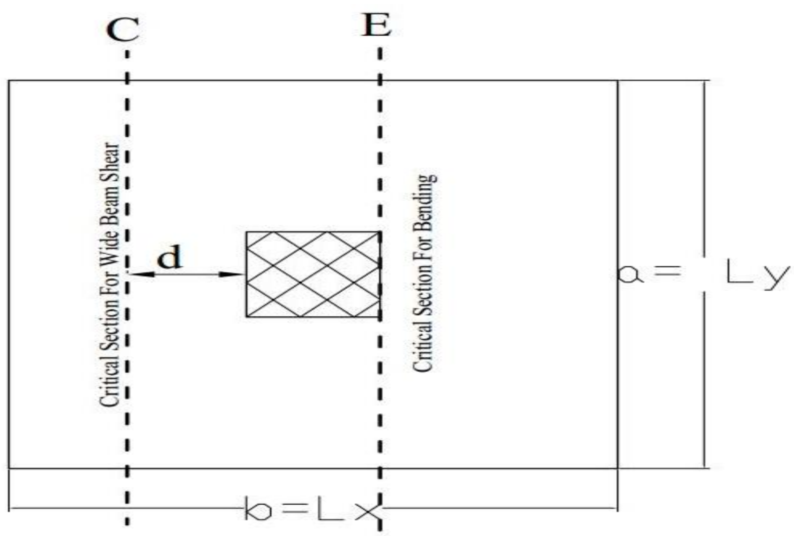

Figure 3-3: Critical section for wide beam shear

iii. The footing is designed as reinforced concrete beam and critical section for bending are at the face of columns. There will only be negative bending steel in the bottom of the footing running in both directions.

Flexural reinforcement for both directions in bottom

$$\rho = \frac{\sqrt{\frac{1-2M_{ox}}{f_{cd}*1000*d^2}}}{f_{cd}*f_{yd}} \qquad 3\text{-}13$$

$$\rho_{min}\,\frac{0.6}{f_{yk}} \qquad 3.13(a)$$

$$As = max(\rho, \rho_{min}) * 1000 * d \qquad 3\text{-}14$$

Development Length Calculation

For good bond conditions the design bond strength of plain bars, f_{bd}, may be obtained from the equation, $f_{bd} = f_{ctd}$

For deformed bars twice, the value for plain bars may be used.

$f_{bd} = 2f_{ctd}$,

The basic anchorage length l_b for a bar of diameter ϕ is:

$$l_b\ needed = \frac{\emptyset * f_{yd}}{4 * f_{bd}} \quad \text{3-15}$$

The required anchorage length l_{bnet} depends on the type of anchorage and on the stress in the reinforcement and can be calculated as:

$As_{cal}/As_{provided}=1$ at the face of column

$l_{b\ net}=a*lb$ where a=1 for straight bar anchorage in tension or compression

=0.7 for anchorage in tension with standard hooks

$l_{b\ min}$=max (0.3lb,10ϕ ,200mm)

$$l_{bavailable}=\frac{B-b}{2} - d' \quad \text{3-16}$$

- Settlement Analysis

The increase of stress in soil layers due to the load imposed by various structures at the foundation level will always be accompanied by some strain, which will result in the settlement of the structure. In general, the total settlement S of foundation can be given as S=Se +Sc +Ss

i. settlement Immediate settlement calculation

The elastic settlement of shallow foundation can be estimated by using the theory of elasticity. Theoretically, if the foundation is perfectly flexible (24), the settlement may be expressed

$$Si = q * \alpha * B\left(\frac{1-\mu s}{Es}\right) * Is * If \quad \text{3-17}$$

$$I_S = F_1 + \left(\frac{1-2\mu s}{1-\mu s}\right) F_2 \quad \text{3-18}$$

F_1 and F_2 are read from table by combining of m' and n' where $m'=\frac{L}{B}$, $n'=\frac{H}{B'}$, $B'=\frac{B}{2}$ α =4 at center

The elastic settlement of a rigid foundation can be estimated from flexible foundation as:

$Se_{rigid}=0.93*Se_{flexible}$ ……………………………………………………………….....3-19

ii. The primary consolidation settlement calculated using the following formula,

$$Se = \frac{C_c}{1+e_0} H_0 \log\left(\frac{\sigma'_c + \Delta\sigma}{\sigma'_c}\right) \quad \text{……………………………………………………………..3-20}$$

$$\Delta б = \frac{q}{(B+z)(L+z)} \quad \text{……………………………………………………………….. 3-21}$$

$\sigma o = \Upsilon_1 h_1 + \Upsilon_2 h_2$ ……………………………………………….……………….................…. *3-22*

The depth considering for calculating the settlement may normally taken as the depth at which the effective vertical stress due to the foundation is 20% of the effective over burden sterss (15).

Manually solved sample calculation shown in apppendix B.

3.5 Case Two: - Micro Pile Design

The cost effectiveness of micropiles will typically be limited to projects with one or more significant geological, structural, logistical, environmental, access or performance challenges. They are anespecially favorable option where, the subsurface conditions are “difficult”, there is restricted access and/or limited overhead clearance, there are subsurface voids (e.g.,karstic limestone)Vibration sand noise must be limited,Structural settlement must be minimized,relatively high unit loads (e.g.,500 to 5000 kN) are required.Following the FHWA State of Practice review in 1997, there have been major efforts made in the quest for a “unified” design approach. Such efforts include the FHWA Implementation Manual (2000), the efforts of the Micropile committees of both the ADSC (International Association of Foundation Drilling), and DFI (The Deep Foundations Institute), and the work products of the IWM (International Workshop on Micropiles) meetings in Seattle (1997), Ube (1998), Turku (2000), Lille (2001), Venice (2002), and Seattle again (2003).

First of all review all available project information and geotechnical data; Such data are well known to all foundation engineers and represent the same scope as for micropiles.

i. Design for Geotechnical Strength Limit States:-Bond length design is most usually based on the assumptions that load is transferred by skin friction only and that this distribution is uniform. The first assumption is nearly always accurate; the second assumption is nearly always false. Nevertheless, it is conservative, although it does tend to confuse inexperienced analyzers of detailed load from 2 to 2.5. End bearing is only a realistic consideration for moderately loaded micro piles founded on competent rock (i.e., act as simple struts).

ii. Design for Structural Strength Limit States:- Although the structural design of micro piles is sufficiently different from more conventional drilled shafts or driven piles, local construction regulations and/or building codes may indirectly address micro pile design and therefore need to be considered by the design engineer.

a. Axial Compression of Cased Length:-the allowable compression load for the cased (free) length is given as ,

$Pc_{all}=0.4fc_{grout}*A_{grout}+ 0.47*Fy_{steel}(A_{bar} +A_{casing})$………………….….……..…………. 3-23

Allowable Tension of Cased Length: the allowable tension load for the cased length is given as

$Pt_{all}=0.55Fy_{steel} (A_{bar}+ A_{casing})$ ………………………………………………………. 3-24

b. structural capacity of uncased length

Soil condition and the method of installations can affect the resulting diameter of the pile bond length. Assume a drill hole diameter of 50mm greater than the casing outside diameter and to improving soil condition and pore-water pressure effect grout diameter is given by Dg=1.4*Db, but D_b=50mm+cassing dia.

Bond length grout area, $A_{grout}== \frac{\pi}{4} *D_g^2-A_{bar}$ ……………………………………………….. 3-25

Allowable tension load: - $Pt_{all}=0.55*Fy_{bar}*A_{bar}$ …………………………………………..…..3-26

Allowable compression load:-

$Pc_{all}=0.4fc'_{grout}*A_{grout}+0.47Fy_{bar}*A_{bar}$ …………………………………………...……3-27

Allowable geotechnical capacity: An allowable geotechnical bond load $PG_{all} \geq$ applied compression load must be provided to support the structural loading.

The bond length L_b required to provide PG_{all} is calculated as

$L_b= \frac{P_{Gall}*FS}{B\alpha_{bond}*\pi*D_g}$……………………..………………………………..…………..….. 3-28

The group capacity to be used for design is evaluated using the following steps (18) and FS=2

Step1. $Q_g = \alpha_{bond} * \pi * D_b * L_b$* number of pile*$\eta$

Step2. $Qg = (2B_g + 2L_g) D*Su' + B_g*L_g*N_c*Su$

Step3. Use lower value of step1 and step2 value for design.

- Pile cap design

  Pile cap is used to distribute load and moment acting on the column to the entire pile group and the design step is more or less similar to footing design.

c. Estimation of micro pile settlement

i. Elastic settlement.

- Casing length elastic stiffness

The upper 2m depth of soil is soft clay, so weak in bond and provide pile casing of 3m length.

Pile compression stiffness value is $EA_{casinglength} = A_{grout}*E_{grout} + A_{steel}*E_{steel}$ ………………….. 3-29

Uncased length elastic stiffness

Pile compression stiffness value is $EA_{uncasedlength} = A_{grout}*E_{grout} + A_{steel}*E_s$ …………………… 3-30

Elastic displacement of cased length = $\frac{Pc * L_{casing}}{E * A_{casing}}$ ……………………………………..… 3-31

Elastic displacement of uncased length = $\frac{Pc * L_{uncased}}{E * A_{uncased}}$ ……………..…………………....… 3-32

Total elastic settlements are= Pile compression stiffness + Elastic displacement of cased length

ii. Micro pile group settlement

Micro pile in a group can undergo vertical displacement because of consolidation of cohesive soil layer below the bottom of the micro pile group where a single pile will transfer its load to the soil in the immediate vicinity of the pile; a pile group can distribute its load to the soil layer below the group. For the purpose settlement calculation in the soil, the loads primarily transferred through side resistance are assumed to act on an equivalent footing located at two- thirds of the depth of embedment of the micro pile into the layer provides support i.e two-third of the depth of the micro pile bond zone.

The depth at which the pressure increase from the equivalent footing is less than 10% of existing effective over burden pressure will provides the total thickness of cohesive soil layer or layers to be used in performing settlement computation (18). The formula used to calculate settlement is similar to that used in isolated footing. Manually solved sample calculations are shown in appendix C.

4 RESULTS AND DISCUSSION

4.1 Results

Damage due to expansive soil could be occur on any type of infrastructure , building, road and other civil engineering construction that are not properly designed and /or constructed. However light weight structures like one or two story building are vulnerable to damage. The design load of foundation for G+1 residential building was determined by ETABS-2014 soft-ware, using soil allowable bearing capacity 262.5kPa which was determined by manual calculation.

- Case 1 design of isolated footing

Table 4-1: Restraint reaction at base

Designation	Fz	Mx	My	Remark
241	69.5	-0.52	2	
246	152.7	-0.91	1.446	
251	182	0.53	0.45	
256	170.3	-0.91	0.6	
261	104.4	0.4	2.3	
266	60.6	-0.25	0.18	
271	236.86	-0.18	1.65	
276	353	0.54	-2.32	
281	313.81	0.36	-2.96	
286	173.51	0.58	1.81	
296	97.67	-0.581	0.381	
301	121.5	0.644	-1.4	
306	114.94	-0.17	-0.66	
311	93.53	0.72	0.962	

For those loads, the following footing dimension determined from which the layout is shown in the appendix D.

Table 4-2: footing dimension

Footing name	Depth (m)	Size (m)	Thickness (m)
F_1	2.5	2*2	0.35
F_2	2.5	1.5*1.5	0.3

For those footing the following bill of quantity was prepared

Table 4-3: Bill of quantity for isolated footing

S .No	Description	Unit	Rate	Quantity	Total price (ETB)
1	Excavation	m^3	170.65	550.67	93,971.84
2	Cart away up to 3km	m^3	75	550.67	41,300.25
3	Select material or non-expansive	m^3	283	371.36	105,094.88
4	Concrete C-25	m^3	2888	16.88	48,749.44
5	Reinforcing bar	Kg	63.29	784.68	49,662.4
6	Form work	m^2	420	41.85	17,577
7	Masonry	m^3	1558	38	59,204
				Total	415,559.81
				Vat (15%)	62,333.97
				Total with vat	477,893.77

Source of unit rate contractor

Construction period of those foundation are shown below

Table 4-4: Construction period of isolated footing

32s.No	Job description	Quantity (m^3)	Duration (in days)
1	Site clearance and bulk excavation	311.3	1
2	pit excavation	239.38	10
3	Cart away	550.67	1
4	Concrete work	16.88	6.5
5	Curing		3
6	Backfill	371.36	9.5
7	Masonry work	38	4
			35

- Case 2 design of micro pile

For the above group of loads the following dimensions of micro pile are determined. The layout showing micro pile is shown in appendix D of this document.

Table 4-5: micro pile dimension

Dimension	For group 1 (M1)	For group 2 (M2)
Diameter (m)	0.2	0.2
Length (m)	4	3

Table 4-6 : Summary of allowable load for controlling axial design load of 353kN

Description	Capacity
Structural upper cased length	569.95kN
Structural lower uncased length	476.54kN
Geotechnical capacity of bond length	370.54kN

The following bill of quantity prepared for those dimension.

Table 4-7: bill of quantity for micro-pile

S.No	Description	Unit	Quantity	Rate	Total price (ETB)
1	Site clearance	m^2	179.38	45	8072.1
2	Drilling and installation of micro pile	ml	180	450	81,000
3	Casing	ml	168	600	100,800
4	Reinforcing bar	Kg	1134.56	63.29	71,805.98
5	Concrete C-30	m^3	24.88	3000	74,640
6	Pile head connection	No	56	300	16,800
7	C-25 for pile cap	m^3	6.86	2888	19,811.68
8	Form work	m^2	24.36	420	10,231.2
				Total	383,160.96
				Vat (15%)	57,474.14
				Total with vat	440,635.1

Source of unit rate is contractor

Table 4-8: Construction period of micro pile

S.NO	Job description	Duration (in days)
1	Site clearance to depth of 20cm	1
2	Drilling, casing and grouting	14
		15

4.2 Discussion

As shown in table 4.3 and 4.7, the total cost of isolated and micro pile foundation is 477,893.77 and 440,635.1 respectively. The cost required for construction of micro pile is decreased or minimized by 7.79% as compared to cost of isolated foundation. This indicates that micro pile is economical than isolated footing on expansive soil.

In table, 4.4 and 4.8 the construction period of isolated foundation and micro pile is 35 and 15 days correspondingly. The construction period of micro pile is decreased by 20 days or 57% as compared to construction period of isolated footing. This indicates the feasibility of micro pile.

In addition, during construction of isolated footing, the environmental disturbance is high due to large quantity of excavation, cart away of excavated material and import of select material from quires is done by large truck. The repetitive movement of this truck creates dust, which cause environmental air pollution, and creates traffic congestion. Most of the time foundation work is take place in dry season due to the problem of rain, moisture fluctuation, mud etc. However, to construct micro pile foundation it is not time or season dependent and we construct at any time.

Therefore, use of micro pile foundation for lightweight structure on weak soil is better, because it minimize different failure on structure due to swelling pressure of soil, maintenance and initial cost of foundation, construction period and environmental disturbance.

5 CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

This study was concerned with the analysis of micro pile as alternative foundation system for lightweight structure on expansive soil. For residential building G+1 structure, isolated footing and micro pile are designed using allowable bearing capacity of soil to support super structural load and transfer to under laying strata.

From the analysis of those foundation system the following conclusion have been made.

1. When use isolated footing amount of excavated material increased by 93.48% as compared to micro pile foundation.
2. The amount of cost required to construct isolated footing is increased by 7.79% relative to micro pile.
3. The construction period required for isolated footing increased by 20 days as compared to micro pile foundation.
4. The environmental impact or disturbance of isolated footing construction is higher than micro pile foundation.

5.2 Recommendations

For a soil with low bearing capacity like expansive or black cotton soil, there are different methods of mitigation to minimize structural failure and to increase bearing capacity. These mitigation measures are either structural or non-structural. From structural system, micro-pile is economical and it takes less construction period to construct. In our country this type of foundation is not practiced well before because, no more research done. It is better to extend this study by considering ground water effect, and medium weight structure.

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APPENDICES

5.3 Appendix A: Bearing capacity determination

The allowable bearing capacity can be expressed as follows:

Q_{ultnet}=c*Nc , $c=\frac{q_u}{2}$, qu=12.5*N_{design}

$Q_{ult}=\frac{12.5N_{design}}{2} * N_c$=6.25$N_c$*$N_{design}$

$Nc=6(1+0.2\frac{Df}{B}) \leq 9$

The net allowable bearing capacity obtained by dividing of net ultimate bearing capacity to factor of safety FS.

$Qall=\frac{6.25N_{design*}N_c}{FS} = \frac{6.25N_{design*}N_c}{2.5}$ =2.5Nc*N_{design}

$N_{design}=\sum\frac{\frac{N_i}{i*i}}{\frac{1}{i*i}}$

Nc= $6(1+0.2\frac{Df}{B})$,<=9, Df=2.5m, B=2

Nc=$6(1+0.2*\frac{2.5}{2})$=7.5 <9

Nc=7.5

Q_{ult}=.6.25N_c*N_{design}

Qult=6.25*7.5*14=656.25kN

$Q_{all}=\frac{Q_{ult}}{FS}= \frac{656.25}{2.5}$=262.5kN

5.4 Appendix B: Design of Isolated Footing

Sample Calculation of Isolated footing (for F_2)

Material:Compresive streangth of Concrete -30 or C-30

fcu=	30	Mpa
fck=	24	Mpa
fctd=	1.16485	Mpa

Material: Steel (S-400)

Fyk	400	Mpa
fyd=	347.83	Mpa
Pmin	0.0015	

Class I work

γ_c = 1.5

γ_s = 1.15

f_{ctd} = 1.16485Mpa = $\frac{0.21}{\gamma c}$*(0.8*$f_{ck}^{2/3}$)

f_{yk} =400 Mpa

$f_{yd} = \frac{fyk}{1.15}$ =347.8Mpa

$\rho_{min} = \frac{0.6}{fyk}$ =0.0015

q_{all} = 262.5Kpa

Result from ETABS

M_y = -2.3189

M_x = 0.5445

F_z = 353KN = P

Lx = $l_{y=}$B (for square footing)

$$ex = \frac{My}{Pd} = \frac{-2.3189}{353} = -0.0066$$

6e_x =-0.0066*6u

= 0.0394

$$ey = \frac{Mx}{Pd} = \frac{0.5445}{353} = 0.00154$$

6e_y = 6*0.00154

= 0.00925

$$\sigma_{\text{allow}} = \frac{P}{A} * (1 \pm \frac{6e_x}{l_x} \pm \frac{6e_y}{l_y})$$

$$qall = \frac{P}{B*B} * (\frac{6ex}{B} + \frac{6ey}{B})$$

$$qall = \frac{353}{B*B} * (1 - \frac{-0.0394}{B} + \frac{0.00925}{B})$$

$$qall = \frac{353}{B*B} * (\frac{B - 0.03015}{B})$$

$$262.5 = \frac{353 * B}{B * B * B} + \frac{17.178}{B * B * B}$$

$$B = \frac{\sqrt[3]{353B + 17.178}}{262.5}$$

By trial and error: B = 1.19, Take, B= 1.2m

Check settlement is within permissible value with this footing width.

Table settlement with different footing width

width (m)	1.2	1.5	2	2.5	3
load (kN)	353	353	353	353	353
Si (mm)	9.17	7.34	5.5	4.4	3.668
Se (mm)	79	69.3	56.6	47.1	39.7
Si + Se	88.2	76.6	62.1	51.5	43.36

The calculation shows that total settlement at width 1.2m is greater than permissible value of settlement of footing on clay i.e 75mm. so increase the width to minimize the settlement and take B=2m

Depth determination: the depth for spread footing is determination from punching (two-way) shear and wide beam shear (one-way considerations the two depth are compared and the greatest will be adopted. Other Information used in this Design:

EBCS-1, 2, 7 &8 Provisions.

C-30 Concrete and S-400 Steel Grading.

FS=2.5 for Isolated footings

Concrete cover =50mm for Isolated footings

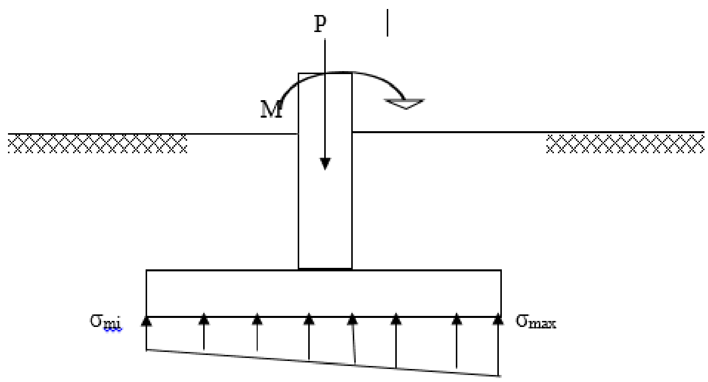


Figure Vertical section of typical isolated footing

A. Punching shear or two way shear

The critical section for punching is at a distance of 0.5d from the face of column. Where d is effective depth of the footing.

$$\sigma_{\text{allow}} = \frac{P}{A} * (1 \pm \frac{6e_x}{l_x} \pm \frac{6e_y}{l_y})$$

Where p is load from envelope

The contact pressures are compared with the ultimate bearing capacity of the soil.

Taking F.S= 2.5

Qall=262.5kPa

The actual contact pressure calculated as follows:-

$$\sigma_1 = \frac{353}{2*2} * \left(1 + \frac{0.0394}{2} - \frac{0.0093}{2}\right) = 90.34kN/\text{m}2 < \sigma_{all}$$

$$\sigma_2 = \frac{353}{2*2} * \left(1 + \frac{-0.0394}{2} + \frac{0.0093}{2}\right) = 86.92kNm2 < \sigma_{all}$$

$$\sigma_3 = \frac{353}{2*2} * \left(1 + \frac{0.0394}{2} - \frac{0.0093}{2}\right) = 89.58kN/m2 < \sigma_{all}$$

$$\sigma_4 = \frac{353}{2*2} * \left(1 - \frac{0.0394}{2} - \frac{0.0093}{2}\right) = 85.58kN/m2 < \sigma_{all}$$

$$\sigma_{ave} = \frac{\sigma_1 + \sigma_2 + \sigma_3 + \sigma_4}{4}$$

$$= \frac{90.39+86.92+89.58+85.58}{4} = 88.1 kN/m^2$$

σ_{avg} = 88.1kN /m^2

σ_{max} = 90.39kN/m^2

σ_{min} = 85.58kN/m^2

σ_{max}=90.39kN/m^2 < Qult-------ok

σ_{min}=85.58 kN/m^2> 0------ No separation will occur

The Punching shear resistance according to EBCS-2 is given by

$V_{up} = 0.25 f_{ctd}\, k_1 k_2 u d$ (MN)

Take d= 0.250m and $\rho = \rho_{min} = 0.0015$

k1 = (1+50ρ) = (1 +50*0.0015) =1.075

k_2 = 1.6 – d =1.6 -0.25= 1.35

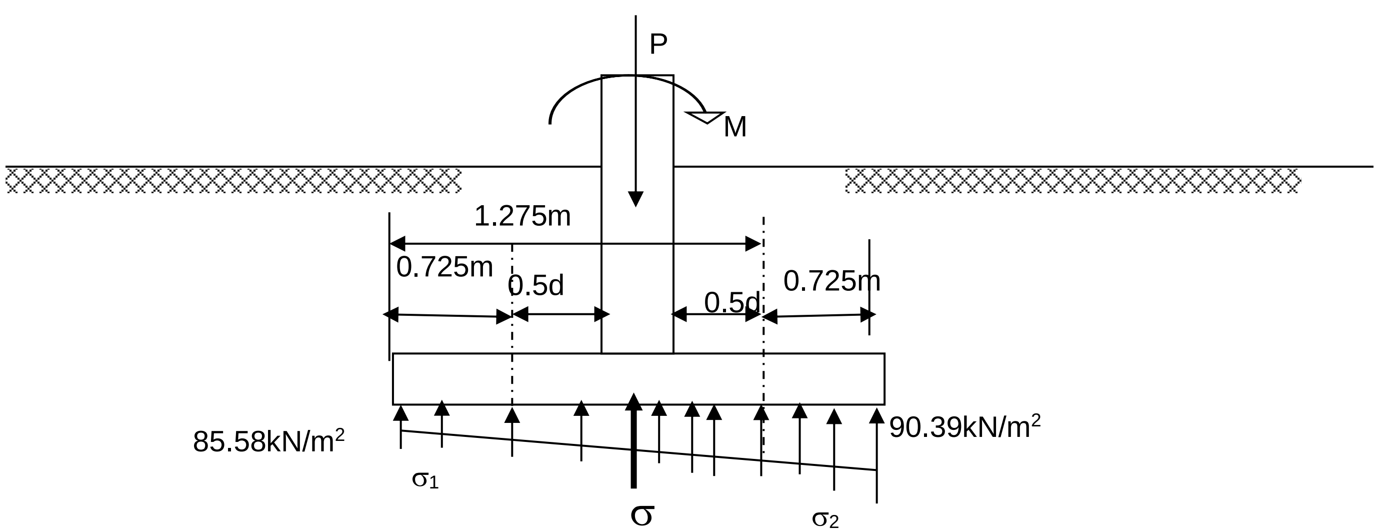
u =4*(d+a') where d is depth of footing

a' is column size

u =4*(0.25+0.3) =2.2

Then

V_{up} = 0.25*1.1648*1.075 *1.35 *2.2*0.25=0.232432MN = 232.432kN



$$\sigma_1 = 85.58 + \frac{0.725 * (90.39 - 85.58)}{2} = 87.32 kN/m^2$$

$$\sigma_2 = 85.58 + \frac{1.275 * (90.39 - 85.58)}{2} = 88.64 kN/m^2$$

$$\sigma = \frac{\sigma_1}{2}\frac{+\sigma_2}{2} = 88.64 kN/m^2$$

V=87.98*2*1=175.96

Net shear force developed = 353 – 175.96=177.1kN < Vup… Ok!. The depth satisfies the punching shear requirement for the assumed ρ_{min}. Then the value of d=0.25m=250mm

ii. Wide beam shear or one-way shear

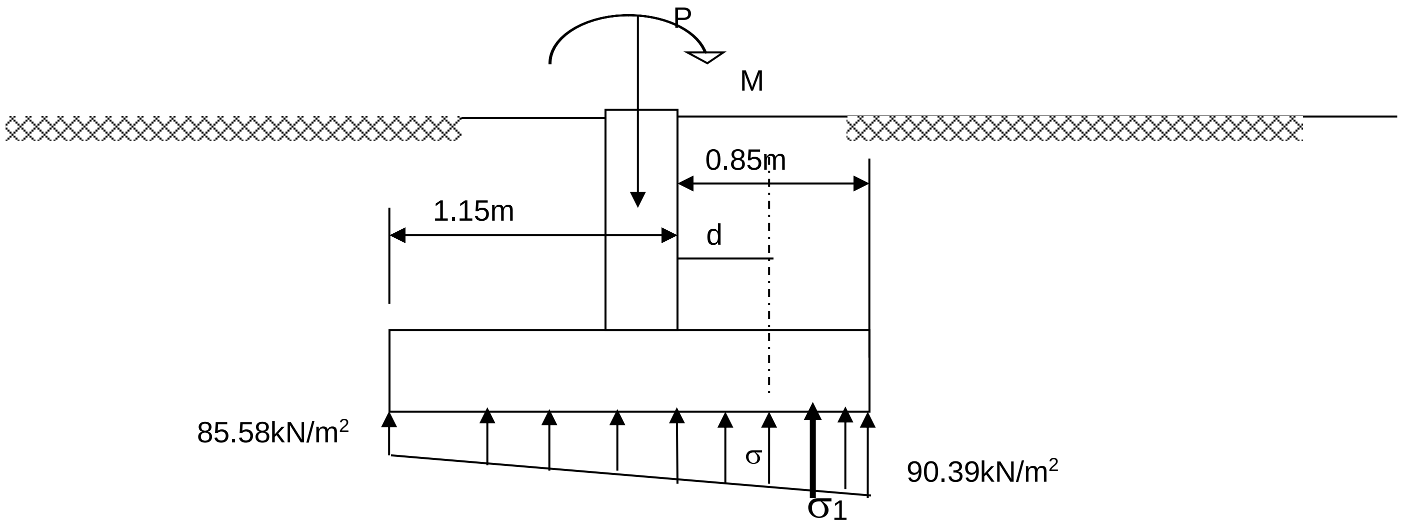


Figure4. 1Vertical section for wide beam shear

Contact stress at distance d from the face of the column, σ

$$\sigma = 85.58 + \frac{(90.39 - 85.58)(0.25 + 1.15)}{2}$$

σ =88.95kN/m²

$$\sigma 1 = \frac{(\sigma \max + \sigma)(0.85 - d)}{2}$$

Developed wide beam shear

V_d = 88.95*2 =177.9kN

Wide beam shear resistance

$V_{ud} = 0.25 f_{ctd}\, k_1 k_2\, b_w d$ (MN)

= 0.25*1.1648*1.075*1.35*2*0.25 =0.211302MN =211.3kN >Vd ok !.

Therefore the total depth of footing is D=d+8+16+50=324, take D=350mm

Reinforcement calculations

The footing is designed as reinforced concrete beam and critical section for bending are at the face of columns. There will only be negative bending steel in the bottom of the footing running in both directions.

Calculation of Average pressures***:*** $M_{x.x}$ and $M_{y.y}$ are equal for square footing with square column.

Bending Moment

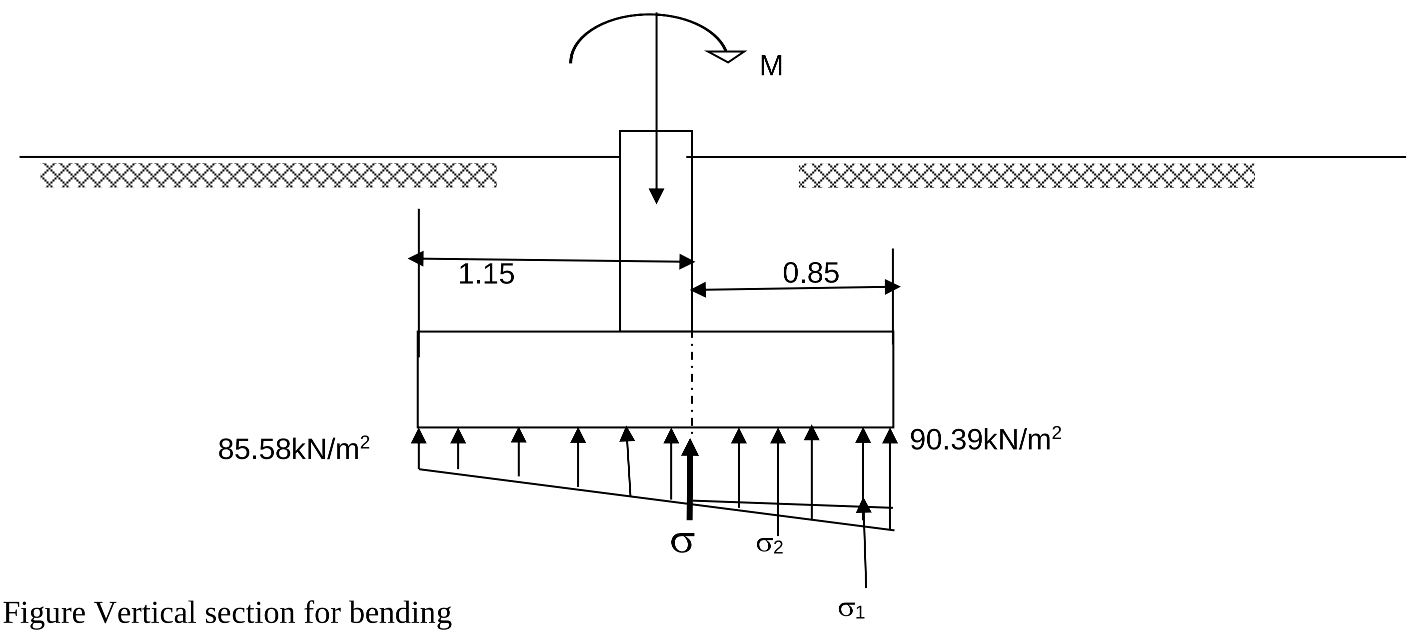


Figure Vertical section for bending

$$\sigma = 85.58 + \frac{(90.39 - 85.58)(1.15)}{2} = 88.34kN/m2$$

$$\sigma_1 = \frac{(90.39 - 88.34)(0.85)}{2} = 1.7425kN/m$$

$$\sigma_1 = 0.85 * 88.34 = 75.1kN/m$$

$$M=(\frac{2}{3}\sigma_1(0.85) + \sigma_2 * \frac{0.85}{2})$$

$$=\left(\frac{2}{3} * 1.7425 * 0.85 + 75.1 * \frac{0.85}{2}\right) 1$$

$$M = 32.9\text{kN} - \frac{m}{m}$$

$M_{x\text{-}x}$ = 32.9*2=65.8kN-m and since square footing,

M_{y-y} = 65.8kN-m

Moment capacity of concrete

$M=0.32*f_{cd}*bd^2$

$=0.32*13.6*10^3*1*0.25^2$

=272kN-m/m

$$\rho=\frac{f_{cd}}{f_{yd}}(1-\sqrt{1-\frac{2M}{f_{cd}bd^2}})$$

$$\rho=\frac{13.6}{347.83}(1-\sqrt{1-\frac{2*272}{13.6*1000*0.25^2}})$$

ρ=0.003155

As= ρbd=0.003155*1000*250=788.75mm^2

	∅	mm^2
Use	16	201

$$spacing=\frac{as*1000}{AS}$$

$$spacing=\frac{201*1000}{788.75}$$

=509.66mm

Provide a reinforcement with Φ16c/c 500mmin both x and y directions

Development length

$$l_b \text{ needed} = \frac{\phi f yd}{4*fbd}$$

f_{yd} = 347.83Mpa,

f_{bd} =0.7*2* f_{ctd} =0.7*2* 1.164 MPa =1.6296&

ϕ = 16

$$lb_{needed}=\frac{16*347.83}{4*1.6296}=853.78mm$$

$$As_{provided}=\frac{201*2000}{500}=804\text{mm2}$$

$$\frac{\text{As}_{\text{cal}}}{\text{As}_{\text{provided}}}=1 \text{ at the face of column}$$

$l_{b\ net}$=a*lb where a=1 for straight bar anchorage in tension or compression

= 0.7 for anchorage in tension with standard hooks

$l_{b\ net}$=0.7*853.78=597.65mm

$l_{b\ min}$=max (0.3lb,10ǿ ,200mm)

= (256, 160, 200,)

$l_{b\ min}$=256mm

$$lb_{available} = \frac{\mathrm{B - a'}}{2} - \mathrm{d}$$

$$lb_{available} = \frac{2000 - 300}{2} - 50 = 800\mathrm{mm}$$

- Settlement analysis

  i. Immediate settlement calculation

The elastic settlement of shallow foundation can be estimated by using:

$$S_e = \mathrm{qo} * \mathrm{m} * \mathrm{B}\left(\frac{1 - \mu\mathrm{s}}{\mathrm{Es}}\right) * \mathrm{Is} * \mathrm{If}$$

$$I_\mathrm{s} = \mathrm{F}_1 + \left(\frac{1 - 2\mu\mathrm{s}}{1 - \mu\mathrm{s}}\right)\mathrm{F}_2$$

F1 and F2 are read from table by combining of m' and n' where m'= $\frac{L}{B} = \frac{2}{2}$ =1, n'= $\frac{H}{B}$ ', B'= $\frac{B}{2} = \frac{2}{2}$ =1 and n'= $\frac{H}{B'} = \frac{2}{1}$=2 then F_1=0.125, F_2=0.109

$I_\mathrm{s} = 0.125 + \left(\frac{1-2*0.3}{1-0.3}\right)0.109 = 0.187$ and Es=300(N_{55}+6) =300(16+6)=6600

$\frac{Df}{B} = \frac{2.5}{2}$=1.125, < =1 take $\frac{Df}{B}$ =1 and $\frac{B}{L}$ =1, μs=0.3, then If=0.65, table 5.10 (24)

qo= $\frac{Q}{B*B} = \frac{353}{2*2}$ =*288.25

$$S_e = 88.25 * 4 * 1\left(\frac{1-0.3}{6600}\right) * 0.187 * 0.65 = 0.05915\mathrm{m} = 5.92\mathrm{mm}$$

Se_{rigid}=0.93$Se_{flexible}$=0.93*5.92=5.5mm

ii. Primary consolidation settlement are calculated using the following formula.

$$Se = \frac{C_c}{1+e_0} H_0 \log\left(\frac{\sigma'_c + \Delta\sigma}{\sigma'_c}\right)$$

C_c = 0.27, Ho = 5.5m

σ_o = 2*16+2.75*18 = 81.5kPa

$$\Delta\sigma = \frac{q}{(B+Z)(B+Z)}$$

$$\Delta\sigma = \frac{353}{(2+2.75)(2+2.75)} = 15.64kPa$$

$\Upsilon = \frac{Gs\Upsilon w(1+w)}{1+e_o}$ => $18 = \frac{2.65*9.81(1+0.39)}{1+e_o}$

18+18e_o=36.14 =>18e_o=18.14 =>e_o=1.00075 take, e_o=1

$$Sc = \frac{0.27*5.5}{1+1}\log\left(\frac{81.5+15.64}{81.5}\right) = 0.05662 = 56.62mm$$

The total settlement is $S_i + S_e$=5.5+56.62=62.12mm<75mm ok

Appendix C: Micro pile design

Sample calculation for compression axial load 353kN

The group capacity to be used for design is evaluated using the following steps (18) and FS=2

Step1. $Q_g = \alpha_{bond} * \pi * D_b * L_b *$ number of pile$*\eta$

Take α_{bond}=150, D_b=0.21, Lb=2m, numb. Pile=4 and for s=4.5Db, η=0.95

Qg=150*3.14*0.21*2*4*0.95=751.72kN

Qgall= $\frac{Q_g}{FS} = \frac{751.72}{2}$ =375.86kN

Step2. Qg = $(2B_g + 2L_g)$ D*Su' + $B_g * L_g * N_c *$Su

Su''= (55.4+69.54+60.11)/3=61.68kPa, su=60.11, $L_g = B_g$ =1.1 and D=2m

$N_c = 5(1+0.1*B_g/L_g)(1+0.2D/B_g)$ for $D/B_g \le 2.5$

$N_c = 7.5(1+0.2*B_g/L_g)$ for $D/B_g > 2.5$

N_c =5(1+0.1*1.1/1.1) (1+0.2*2/1.1))

=5*1.2*1.364

Nc = 8.18

Q_g = (2*1.1+2*1.1)*2*61.68+1.1*1.1*8.18*60.11

=1137.37kN

$Q_{all} = \frac{Qg}{2} = \frac{1137.37}{2}$ =840.26kN

And

Step3. Use lower value of step1 and step2 value for design, so Q_{allg} = 375.86kN < 840.26kN

1. Evaluate allowable structural capacity of cased length

Material dimension and property

Use casing 150mm outside diameter and 9.17mm wall thickness

Reduce outside diameter by 1.6mm to account for corrosion i.e OD $_{casing}$=150-2*1.6=146.8mm

Pile casing inside diameter ID $_{casing}$=150-2*9.17=131.66mm

Pile casing steel area $A_{casing} = \frac{\pi}{4}(OD^2 - ID^2) = 3309.47mm^2$

Casing yield strength $F\text{-}c_{asing} = 241MPa$

Reinforcing bar- use 20mm grade 420 steel reinforcing bar.

Bar area $A_{bar} = 314mm^2$

Bar yield strength $F\text{-}_{y\ bar} = 420MPa$

Grout area $A_{grout} = 13293.47mm^2$

For strain compatibility between casing and rebar, use for steel yield stress $F\text{-}y_{steel}$=minimum of $F\text{-}y_{bar}$ or $F\text{-}c_{asing}$, $Fy_{casing} = 241MPa$

- Allowable tension load

$Pt_{all} = 0.55Fy\text{-}_{steel}(A_{bar} + A_{casing})$

$= 0.55*241(314+3309.47)$

$Pt_{all} = 480.29kN$

Allowable compression load

$Pc_{all} = 0.4fc_{grout}*A_{grout} + 0.47*Fy_{steel}(A_{bar} + A_{casing})$

$= 0.4*30*13293.47 + 0.47*241(314+3309.47)$

$Pc_{all} = 569.95kN$

2. Evaluate structural capacity of uncased length

Pile uncased length allowable load

Material and dimension property

Soil condition and the method of installations can affect the resulting diameter of the pile bond length.

Assume a drill hole diameter of 50mm greater than the casing outside diameter and to improving soil condition and pore-water pressure effect grout diameter is given by $D_g = 1.17*Db$, but D_b=50mm+cassing dia.

Therefore grout body diameter is =1.17*(50+131.66) =0.21073m

Take D_g=210mm

Bond length grout area, $A_{grout}= \frac{\pi}{4} *D_g^2-A_{bar}$ =34544.144mm^2

- Allowable tension load

$Pt_{all} = 0.55*Fy_{bar}*A_{bar}$

$= 0.55*420*314$

Pt_{all} = 72.54N

- Allowable compression load

$Pc_{all} = 0.4fc'_{grout}*A_{grout}+0.47Fy_{bar}*A_{bar}$

$= 0.4*30*131991.69+0.47*420*314$

Pc_{all} = 476.52kn

3. Evaluate allowable geotechnical capacity

An allowable geotechnical bond load PG –all≥353kN must be provided to support the structural loading.

The bond length Lb required to provide PG-all is calculated as

$$L_b = \frac{P_{Gall}*FS}{\alpha_{bond}*\pi*D_g}$$

$$L_b= \frac{353*2}{150*\pi*0.21}$$

= 1.778m, Take L_b = 2m

4. Verify allowable loads

The controlling axial design load for the pile is 353kN (compression).

The summary of allowable load is;

Structural upper cased length =569.95N

Structural lower uncased length =476.54kN

Geotechnical capacity of bond length =370.54kN

The allowable load are all greater than the design load 353KN, therefore the axial pile deign is ok and use this pile also for design load of 314kN, and 237kN.

- **Pile cap design**

Data: pile dia=200mm

Spacing =900mm

Column dim. 300*300mm

Factored load= 353kN

Factored moment Myy=-2.3189KN.m

Factored moment Mxx=0.5445kN.m

Compressive strength of Concrete C-25

Steel S-400

a. pile cap dimension

Width = breadth of pile cap =$spacing\ of\ pile + \frac{dia}{2} + 150 + \frac{dia}{2} + 150$

=900+100+150+100+150

=1400mm

b. check for pile load capacity

Assume d=0.25m

Total factored axial load compressive load= $\frac{Mu}{n} + \frac{My}{x*x} + \frac{Mx}{y*y}$

Self-weight of pile cap=1.4*1.4*0.25*25*1.5=18.375kN

Axial load=353+18.375=371.38Kn

X-coordinate of pile cap=0.45m

Y-coordinate of pile cap=0.45m

Compressive load= $\frac{371.38}{4} + \frac{0.5445*0.45}{0.45*0.45} + \frac{-2.3189*0.45}{0.45*0.45}$

P_c =94.43kN

Safe load on pile= $\frac{94.43}{1.5}$=62.95kN < 88.25kN

c. bending moment=94.43*$\frac{0.9-0.3}{2}$

M_u =28.33kN.m

d. check for effective depth

Mu=0.138f_{ck}*b*d*d

28.33=0.138*0.8*25*1400*d^2

d^2= $\frac{28.33}{3864}$

=0.00733

$d_{required}$ = 0.0856m, =85.6mm<250mm….ok

$d_{available}$ = 250-30-12-6=202mm> $d_{required}$=85.6mm

e. check for punching shear at 0.5d from face of column

Perimeter of critical section = 4 (0.5dava. + Column dim. +0.5dava)

=4(0.5*202+300+0.5*202)

=2.008m

Allowable punching shear=0.25*($25^{0.5}$) =1.25

Punching shear= $\frac{load}{perimeter*d\text{ava}}$

= $\frac{371.38}{2.008*0.202}$

=0.87< allow shear =1.25 …..ok

f. main reinforcement

$M=0.32*f_{cd}*b*d^2$

$=0.32*13.6**1*0.202^2$

=177.58kNm

$$\rho=\frac{f_{cd}}{f_{yd}}(1-\sqrt{1-\frac{2M}{f_{cd}bd^2}})$$

$$\rho=\frac{13.6}{347.83}(1-\sqrt{1-\frac{2*177.58}{13.6*1000*0.25^2}})$$

$\rho=0.00926>\rho_{min}=0.0015$

$As=\rho*b*d=0.00926*1000*202=1870.52mm^2$

Use bar dia 14mm, spacing = $\frac{\pi*14*14*1400}{4*1870.52}$

=115.mm, Use spacing=110mm

5. Estimation of micro pile settlement

i. calculate elastic settlement single micro pile.

- Casing length elastic stiffness

The upper 2m depth of soil is soft clay, so it is weak in bond and provide pile casing of 3m length.

Steel modulus of elasticity E_{steel}=200,000MPa

Grout modulus of elasticity E_{grout}=31,000MPa

Area of grout $A_{grout}=13293.47mm^2$

Area of steel $A_{steel}=A_{bar}+A_{casing}=3623.47mm^2$

Pile compression stiffness value

$EA_{casinglength}=A_{grout}*E_{grout}+A_{steel}*E_{steel}$=1,136,791,394kN

- Casing length elastic stiffness

Pile uncased length or bond length, L_b= 2m

Area of steel= A_{bar}=314mm^2

Area of grout A_{grout}=34544.144mm^2

Pile compression stiffness value

$EA_{uncasedlength}$= A_{grout}*E_{grout}+A_{steel}*E_{steel} =1,133,668,452kN

Elastic displacement of cased length = $\frac{P_C * L_{casing}}{E * A_{casing}} = \frac{353*3*1000}{1136791394.33}$ =0.000931569mm

Elastic displacement of uncased length = $\frac{P_C * L_{uncasing}}{E * A_{uncasing}} = \frac{353*3*1000}{1133668452.18}$=0.000311379mm

Total elastic settlements of micro pile are;

S_e = 0.000931569 + 0.00031137= 0.00124mm

ii. Immediate settlement of sand

$$S_e = \mathrm{qo} * \mathrm{m} * \mathrm{B}\left(\frac{1-\mu s}{\mathrm{Es}}\right) * \mathrm{Is} * \mathrm{If}$$

$$I_s = F_1 + \left(\frac{1-2\mu s}{1-\mu s}\right) F_2$$

F1 and F2 are read from table by combining of m' and n', B=1.1, L=1.1 μ=0.3 where m'= $\frac{L}{B}$ = $\frac{1.1}{1.1}$ =1, n'= $\frac{H}{B}$ ', B'= $\frac{B}{2} = \frac{1.1}{2}$ =1 and n'= $\frac{H}{B'} = \frac{1.1}{0.55}$=2 then F_1=0.142, F_2=0.064

$I_s = 0.125 + \left(\frac{1-2*0.3}{1-0.3}\right) 0.109 = 0.187$ and Es=300(N_{55}+6) =300(16+6)=6600

$\frac{Df}{B}$ =1 =1.125, <=1 take $\frac{Df}{B}$ =1 and $\frac{B}{L}$ =1, μs=0.3, then If=0.65, table 5.10 (24)

qo= $\frac{Q}{B*B} = \frac{353}{1.1*1.1}$ =160.4kN/m^2

$$S_e = 160.45 * 4 * 0.55 \left(\frac{1-0.3}{6600}\right) * 0.1486 * 0.65 = 0.00362\mathrm{m} = 3.62\mathrm{mm}$$

se_{rigid} = 0.93*3.62=3.36mm

ii. Primary settlement

σ_o = ϒ1h1+ ϒ2h

$=2*16+18(\frac{1.33+4.17}{2})$

σ_o =113.37kPa

$\Delta\sigma = \frac{q}{(B+Z)^2}$,

$= \frac{353}{(1.1+2.1)^2}$

$\Delta\sigma'$ =34.47kPa

$S_i = \frac{Cc*Ho}{1+e_o}\log(\frac{\sigma o+\Delta\sigma}{\sigma o})$

$=\frac{0.27*4.17}{1+1}\log(1.31)$

S_i=0.06474m=64.74mm

The total settlement is St=0.0034+3.62+64.74=68.36mm<75mm ok

Appendix D: Isolated Footing and Micro pile Foundation layout

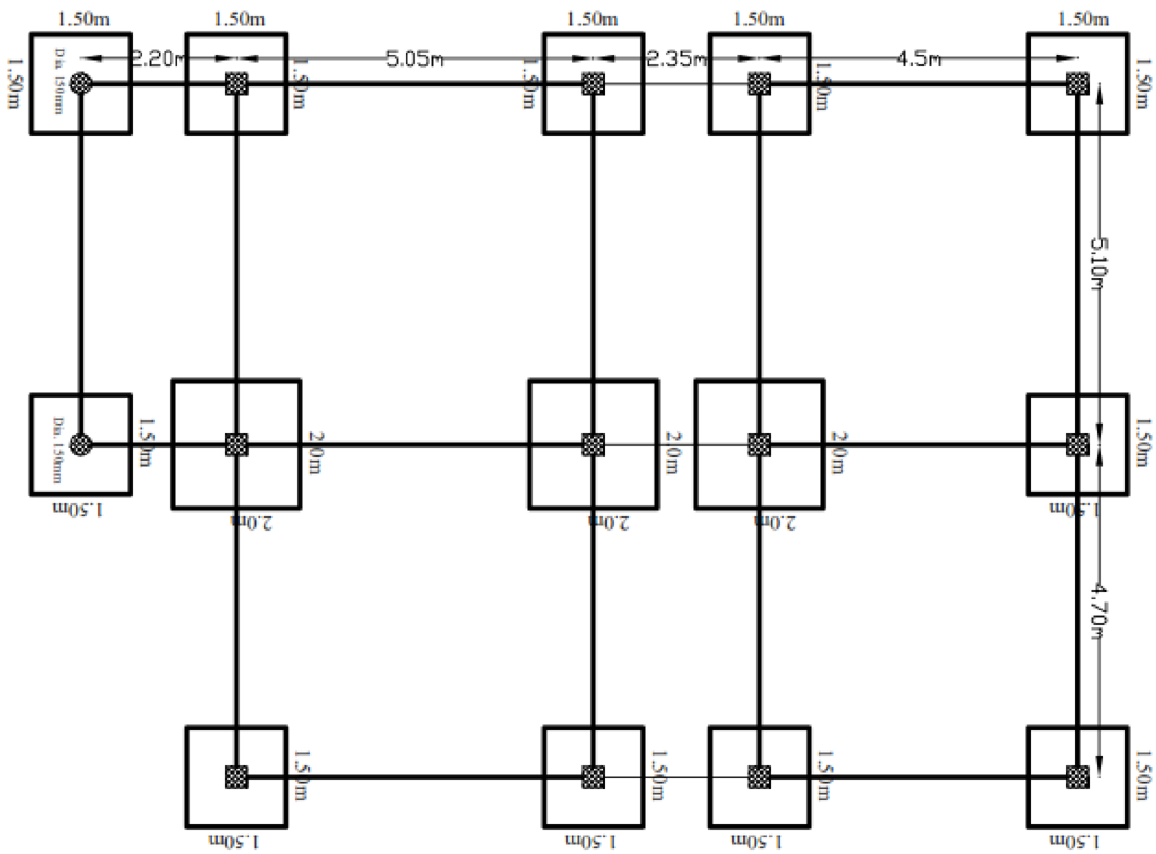


Figure D.1: Isolated footing lay out

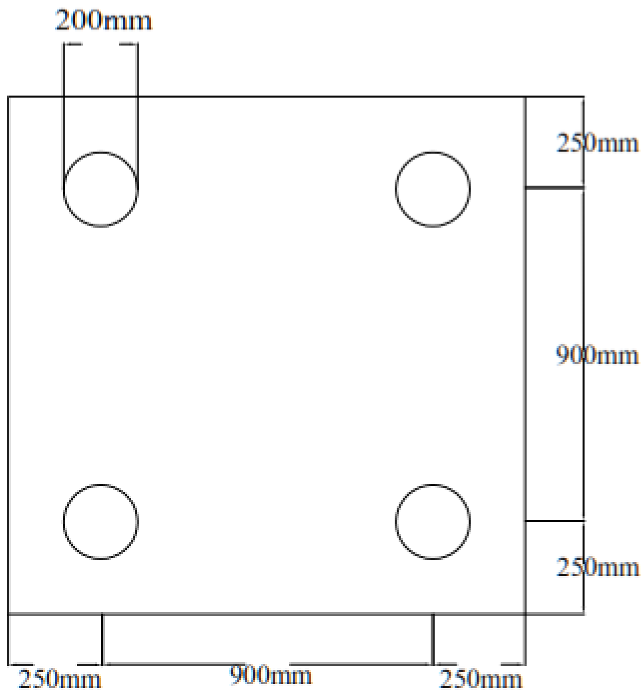


Figure D.2: Micro-Pile plan view

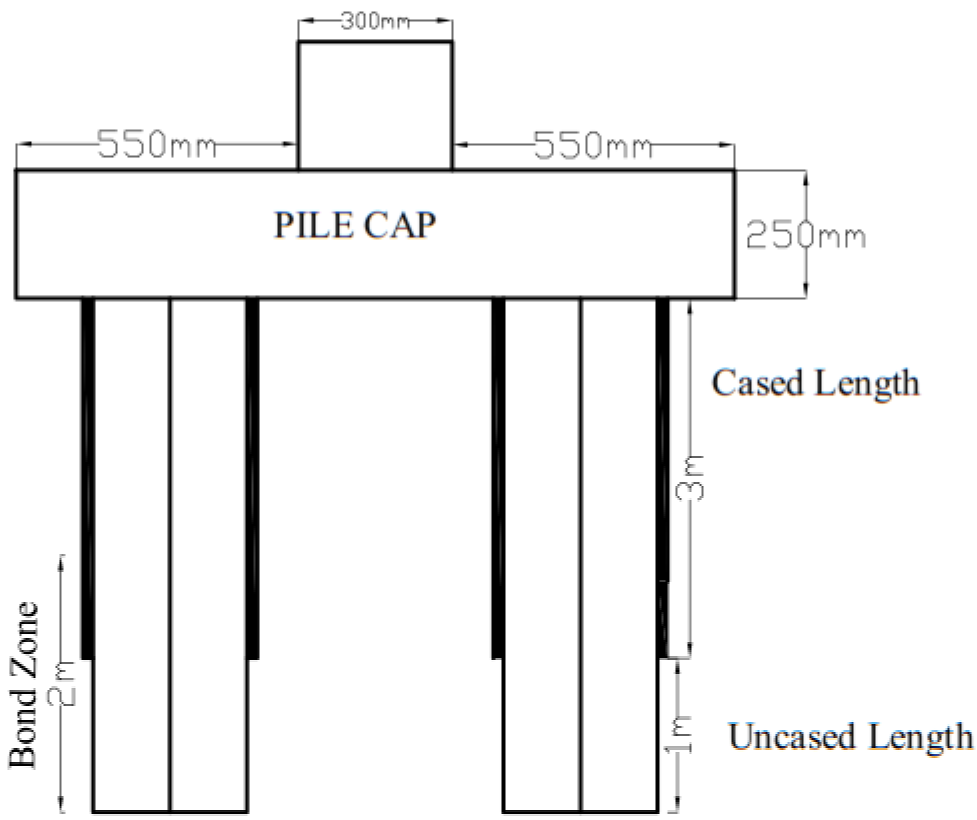


Figure D.3: Micro-Pile elevation view

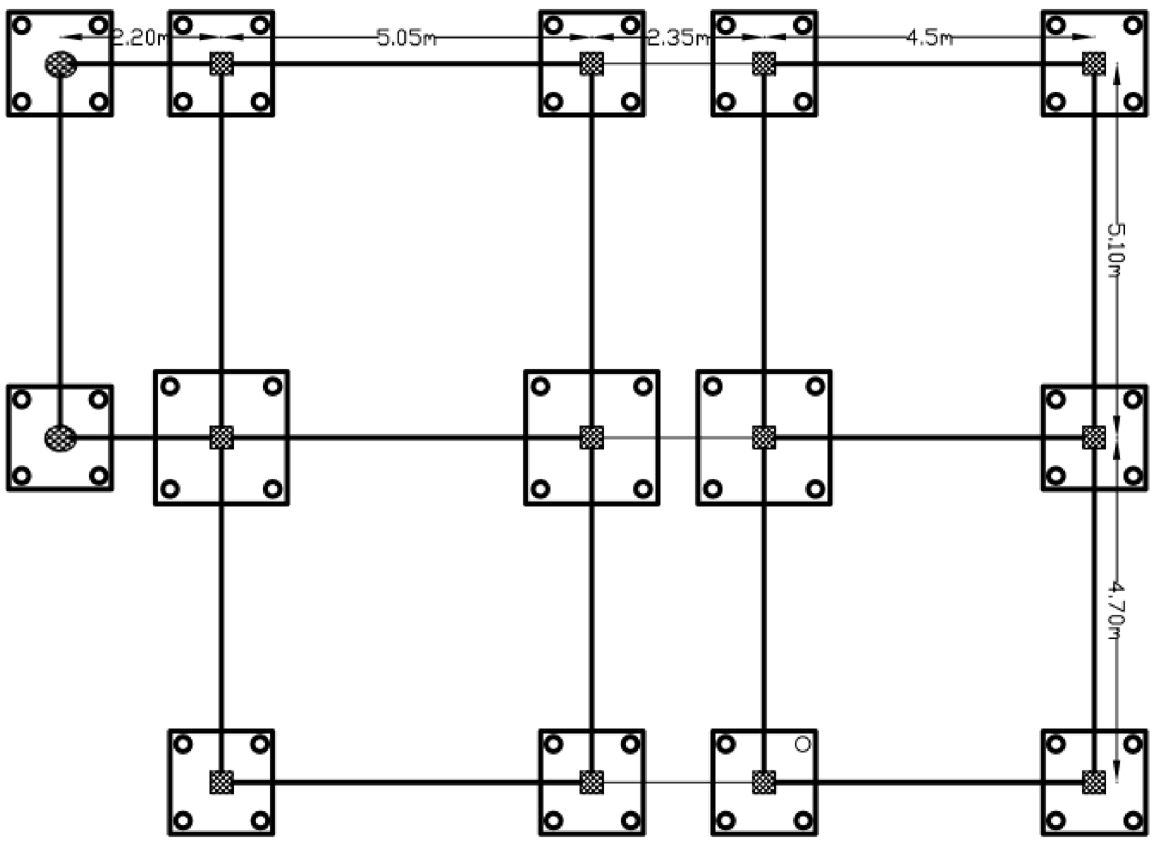


Figure D.4: Micro pile foundation lay out