NUMERICAL ANALYSIS AND PARAMETRIC STUDY OF GRANULAR ANCHOR PILE ON EXPANSIVE SOIL USING FINITE ELEMENT METHOD: CASE OF ADDIS ABABA, BOLE SUB-CITY

Ephrem Abera Shenkute

A Thesis Submitted to

The Department of Civil Engineering

School of Civil Engineering and Architecture

Presented in Partial Fulfillment of the Requirement for the Degree of Master's in Civil Engineering (Specialization in Geotechnical Engineering)

Office of Graduate Studies

Adama Science and Technology University

Adama, Ethiopia

June, 2020

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APPROVAL PAGE

We, the undersigned, members of the Board of Examiners of the final open defense by Ephrem Abera Shenkute have read and evaluate his/her thesis entitled "*Numerical Analysis and Parametric Study of Granular Anchor Pile on Expansive Soil Using Finite Element Method: Case of Addis Ababa, Bole Sub-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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LIST OF ACRONYMS AND ABBREVIATIONS

GAP – Granular Anchor Pile;

CNS – Cohesive Non-swelling;

3D – Three dimensional;

D – Diameter of the pile;

L – Length of the pile;

s – spacing between the piles;

Ip – Plastic Index;

S - Swelling potential of a soil;

A – Activity;

FVM – Finite Volume Method;

$E^{ref}{}_{50}$ – Reference triaxial loading stiffness;

$E^{ref}{}_{ur}$ – Reference unloading stiffness;

$E^{ref}{}_{oed}$ – Reference oedometer loading stiffness;

ϕ – Internal friction angle;

c- cohesion;

ν_{ur} – unloading/reloading Poisson’s ratio;

ψ – Dilatancy angle;

Es – Elastic Modulus of a soil;

Ep – Elastic Modulus of a pile;

N – Standard Penetration test blow count;

SPT – Standard Penetration Test;

R_{inter} – Strength Reduction Factor;

Su – Undrained shear strength;

K – Lateral earth pressure coefficient;

ETB – Ethiopian Birr;

INR – Indian Rupees;

ABSTRACT

A wide range of heaving problems of expansive soil in Addis Ababa become a major difficulty for the construction of low rising buildings by causing different damages in the building. Hence an attractive and economical design solution may be required for such type of problem. Since Granular Anchor Pile is one of the recent ground improvement technique which devised for resisting pull out forces. In a Granular Anchor Pile, the footing is anchored to a mild steel plate placed at the bottom of the granular pile through a reinforcing rod or a cable. Therefore, this research paper is done for the objective of numerical analysis on the performance of Granular Anchor Pile under the heave of expansive soil using PLAXIS 3D program by studying the effect of pile length, diameter and pile spacing and examining the suitability of Granular Anchor Pile as an alternative solution for heave problems of expansive soils for low rising building found in Addis Ababa city especially in Bole Sub-City. According to the result, the pile can have an uplift capacity of a minimum of 510 kN and a maximum of 4,477 kN under the analysis for a pile dimension of L=1m, d=0.4m, and s=3d and L=6m, d=0.4m and s=1.5d respectively. Also, the percentage increase in uplift capacity of the pile is 109, 58, and 39% when the pile length increased from 1 to 3m, 3 to 4m, and 4 to 6m respectively. This is due to an increase in the self-weight of the pile and friction mobilized between the pile and soil interface. Additionally, the uplift capacity of the pile reduced by 5.5, 17.1, and 32% when the diameter of the pile increased from 0.2 to 0.3m, 0.3 to 0.4m, and 0.4 to 0.6m respectively. The pile capacity also reduced by 12 and 10% when pile spacing increased from 1.5d to 2d and 2d to 3d respectively. These are due to the reduction of the number of piles in the group and soil confinement. According to the suitability analysis, the construction of the pile can spend about 2800 Ethiopian birr for simple G+1 building per footing. Generally, it is observed that Granular Anchor Pile has high performance in resisting uplift loads coming from the heaving of expansive soil and variables that considered in the analysis (pile length, diameter, and spacing) have a significant influence on the performance of the pile and the pile is feasible to apply for the actual problem of Expansive soil in a low rising building constructed in the country because of its convenience for all considerations.

Keywords: Expansive soil, Granular Anchor Pile, PLAXIS, Suitability Analysis

1. INTRODUCTION

1.1. Background of the study

Soils are aggregates of mineral particles, and together with air and/or water in the void spaces, they form three-phase systems (Das, 2008). Generally, soils are classified into two major categories which are coarse grain soil and fine grain soil. These soils are composed of a variety of minerals, most of which do not expand in the presence of moisture. However, some minerals in clay soil are expansive.

Reactive soils contain minerals such as Montmorillonite that are capable of absorbing water. When they absorb water, they increase in volume. The more water they absorb, the more their volume increases. This change in volume can exert enough force which is called uplift force. Reactive soils will also shrink when they dry out. This shrinkage can remove support from buildings or other structures. Fissures in the soil can also develop.

This movement in the soil results in structural damages such as cracking of floor slab, wall, and grade beam especially in lightweight structures such as one- or two-story buildings, warehouses, retaining walls, sidewalks, basement floors, pipelines, and foundations.

Distribution of expansive soil is generally a result of geological history, sedimentation, and local climatic conditions. Expansive soil covers an appreciable part of Ethiopia. This soil is observed in areas such as Mekelle, Bahirdar, Gambela, Arba Minch, and the most Southern, south-west, and South-eastern parts of the capital city Addis Ababa (Uge, 2011).

The losses due to extensive damage to structures founded on expansive soils are estimated to be in billions of dollars all over the world (Sisay, 2004). These soils pose the greatest hazard in regions with pronounced wet and dry seasons. This damage can occur within a few months following construction, or may develop slowly over a period of about 5 years, or may not appear for many years until some activity occurs to disturb the soil moisture.

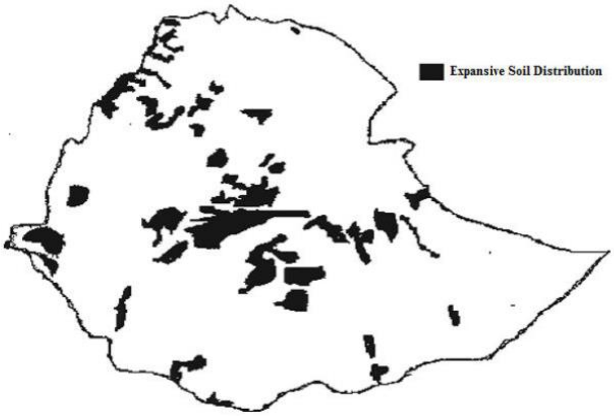


Figure 1.1. Distribution of Expansive Soils in Ethiopia (Uba. 2011).

Many researchers assess the damaging effect of expansive soil in a different part of Ethiopia. On damage assessment of buildings constructed on expansive soils of Addis Ababa on the houses in Bole, Olympia, Nifasilk, Lafto, Old Airport, Mekanisa, Gergi and Bole Bulbula localities of the city, and found that 64 % of the randomly surveyed buildings to be adversely affected by heave problem of Expansive soil (Sisay, 2004). Another research is done on the assessment of randomly selected buildings from 20 different locations in Bahir Dar town and 87 % of them are adversely affected due to this reactive soil heave (Tibebu , 2015).

The main factor for expansive soil damage is a low level of understanding of the mechanisms of heaving and the measures required to counter such problem associated with underestimating of swelling pressures by foundation designers, inadequate use of safety associated with the conservative design, and improper soil investigations and interpretation of soil parameters (Daniel, 2003). Due to this the design engineer design common usual type of foundation or isolated footing for all structure in different site condition especially for low rising building without considering whether there is an expansive soil or not in the site. Another major factor is economical problem related to using advanced techniques and different machinery, which are helpful to eliminate the heaving problem of expansive soil and lack of skilled manpower.

There are different innovative remedial measures suggested by a different researcher for remedying the swell-shrink problems posed by expansive soils corresponding to changes in moisture. They can be categorized as physical alteration, chemical alteration, and tension-resistant foundations. The sand cushion technique and the cohesive non-swelling (CNS) layer technique are examples of physical alteration. Blending expansive soils with chemicals such as lime, $CaCl_2$, fly ash, and Portland cement falls under the category of chemical alteration. Many well-known tension-resistant foundations in practice are under-reamed piles, belled piers, and drilled piers and mat foundation (Rao et al., 2007). These physical and chemical alteration techniques either replace the expansive soils or change the mineralogy of the clay particles and thus reduce the volume changes in expansive soils, and the tension-resistant foundations absorb tensile/uplift force caused on the foundation by the swelling soils.

But the above-listed remedies have some drawbacks, such as being time-consuming, uneconomical, and requiring higher-skilled or trained laborers and special equipment (Kranthikumar et al., 2016). To resolve such types of inconvenience there is a better type of foundation, which is a recent innovation, is called Granular Anchor Pile (GAP). A Granular Anchor Pile foundation, which is a modification of granular pile or stone column, is a recent foundation technique that is quite effective in reducing heave and improving the engineering behavior of expansive clay beds (Kranthikumar et al., 2016). Granular piles are anchored at the base with the help of an anchor plate and anchor rod, hence commonly termed as the Granular Anchor Pile (GAP).

So, this research paper covered the study of the behavior of GAP-Foundation System under the heave using numerical analysis and shows the suitability of GAP as an alternative solution for heave problems in expansive soils for our country condition.

1.2. Statement of the problem

Ethiopia becomes the fastest growing country in Africa as well as in the world. Following this development, urbanization becomes a serious issue in the urban part of the country. Addis Ababa is among the fastest-growing urban area in the country. According to the researches, the population in Addis Ababa has increased rapidly (Wendell, 2012). This rapid growth of population brings on the city to rebuild itself on and around its urban core. There are many new constructions of public and private condominiums and large new low rising residential buildings for residents in Addis Ababa, especially in Bole sub-city. But the wide range of heaving problem of expansive soil in the city become a major challenge for the construction sector especially in constructing low rising buildings by causing a different problem such as distortion and cracking of floor slab; cracks in grade beams and walls; jammed or misaligned Doors and Windows; failure of blocks supporting grade beams. And hence an attractive and economical design solution may be required for such type of problem. Therefore, this study investigated the performance of the Granular Anchor Pile (GAP) system to use it as an alternative solution for the heave problem of expansive soil in a low rising building in the city.

1.3. Objective

1.3.1. General objective

The general objective of this study is analysis and parametric study on the behavior of GAP-foundation systems under the heave of Expansive soil and showing the suitability of GAP as an alternative solution for heave problems in expansive soils mostly for low rising buildings constructing in Addis Ababa, Bole Sub-City.

1.3.2. Specific objectives

- Studying the effect of GAP Length on the uplift resistance capacity of the pile,
- Inspecting the effect of GAP diameter on the uplift capacity of the pile,
- Investigate the effect of Group of GAP system on the capacity of the pile,
- Examining the Group pile’s efficiency of the GAP system for different pile spacing,
- Preparing a regression analysis to quantify the observed trend in the uplift capacity of the GAP system reinforced in the Expansive soil,
- Examining the suitability of the GAP system as a dependable solution for problems that occurred in low rising buildings due to the swelling effect of expansive soil by considering different actual factors.

1.4. Research questions

1. what is the effect of different pile lengths in the performance of GAP to resist the uplift forces?
2. What is the effect of different pile diameter in the performance of GAP to resist uplift forces?

3. What is the effect of a group of piles on the resistance of GAP against uplift force?
4. What is the effect of pile spacing on the efficiency of a group of GAPs?
5. How much the pile is suitable to use as an alternative solution for low rising buildings constructed in an area where the swelling effect of soil become serious?

1.5. Significance of the study

The finding of this study helps to understand the performance of GAP system in resisting the uplift pressure of expansive soil and publicizing the advantage of Granular Anchor Pile system and showing how much it is practicable to use as an uplift resistance foundation for low rising building constructed in the city, in an area where expansive soil becomes a serious problem for the construction of a lightweight building.

1.6. Scope of the study

The scope of this study is limited to a numerical and suitability (feasibility) analysis of the group of GAP system. For numerical analysis, FEM, PLAXIS 3D software is used with a constitutive model of Hardening soil model to know the effect of different parameters with respect to upward displacement of the pile. such as Pile length (L), Pile diameter (D), Pile group, and efficiency of the pile with different spacing in reducing the heave effect of expansive soil for low rising buildings found in Addis Ababa, Bole Sub-city. Suitability analysis of the GAP system is undertaken with respect to its actual applicability for low rising buildings by considering different factors such as Economy of construction, local availability of construction material, Installation process condition, Environmental friendship, Time management, and Performance. This numerical analysis is done on the GAP system of diameter d= 0.2, 0.3, 0.4 and 0.6 m, length of L= 1, 3, 4, and 6 m and pile spacing of s=1.5d, 2d, and 3d with a fixed square footing of 2*2 m on it.

1.7. Research Methodology

The present study started by setting a clear framework of the research by reviewing secondary resources from literature, previous studies, and the internet. Next to this reliable soil data were collected for the study. Then parameters for base model were identified and arranged to use as input parameters for the selected constitutive model. Validation of the software program and sensitivity analysis was also undertaken.

After all these things parameters that have a significant effect on the performance of the GAP system were considered in the analysis. Accordingly, the effect of pile length from 1 - 6 m, pile diameter of 0.2 - 0.6 m, and pile spacing 1.5d, 2d, and 3d were considered. Finally, a suitability analysis of the GAP system was evaluated by considering different factors such as economy of construction, local availability of material, installation process condition, environmental benefit, construction time, and performance of the pile.

1.8.Organization of the Thesis

The thesis is organized into Five Chapters.

The first Chapter is discussed about the current problems associated with Expansive soil in low rising buildings and different remedial measures, Statement of the problem, objectives of the study, Research question, and scope of the thesis.

Chapter two discussed about Literature review which includes ways of identification of Expansive soil, system, and concept of GAP system, Numerical analysis, and Geotechnical problem and reviewing of the previous investigation on Granular Anchor Pile (GAP).

Chapter Three presents study area description, parameter identification, and organization for the base model, methods used for numerical analysis and suitability analysis, sensitivity analysis, and finally a description of the process of numerical modeling.

In Chapter four analysis and parametric study of the Granular Anchor Pile system are presented. The effect of different lengths of GAP, Diameter of GAP, Group of GAP, and Efficiency of Group of GAP is analyzed and discussed. Additionally, suitability analysis of the Granular Anchor Pile system based on its applicability for the low rising building is also furnished by making economical comparison of GAP with other measures and by considering additional factors like local availability of material environmental benefit, time management.

The last chapter, Chapter Five, discusses the conclusions derived from this study which is the summary idea of works undertaken in chapter 4. And recommendations for future work. Relevant figures and data generation for the PLAXIS 3D program are presented in the appendices.

2. LITERATURE REVIEW

2.1. General

The expansive soil is a type of clay soil that tends to increase in volume when water is available and decreases in volume if the water is removed. Expansive soils swell because of the hanging-up of water during the rainy season, reduce in density, and become wet, but in dry seasons it shrinks because of evaporation of water and becomes hard due to an increase in density. In summer season polygonal shrinkage breaks show up close to the surface, reaching out to a deep of around 2 m. Expansive soils are commonly known as black cotton soils, because of their color and their suitability for growing cotton (RaviTeja, M. & Murthy, N. K., 2018).

Expansive soils because of their alternate swelling and shrinkage, they result in detrimental cracking of lightly loaded civil engineering structures such as foundations, retaining walls, pavements, airports, sidewalks, canal beds, and linings In the world most of the countries are facing problems with these soils are Australia, Egypt, Europe, Canada, China, India, Israel, United States (RaviTeja, M. & Murthy, N. K., 2018).

2.2. Identification of Expansive soil

2.2.1. Based on mineralogical composition

The degree of volume changes experienced by an expansive soil depends on its mineralogical composition. The three basics groups of clay minerals are montmorillonite, illite, and kaolinite.

Kaolinite has a structure that consists of one silica sheet and one alumina sheet bonded together into a layer about 0.72 mm thick stacked repeatedly. The layers are held together by strong hydrogen bonds.

Illite consists of repeated layers of one alumina sheet sandwiched by two silicate sheets. The layers, each of thickness 0.96 mm, are held together by potassium ions.

Montmorillonite has a similar structure to illite, but layers are held together by weak van der Waals forces and exchangeable ions. Water can easily enter the bond and separate the layers resulting in swelling, while that of illite contributes a little less, kaolinite other hand is non-expansive.

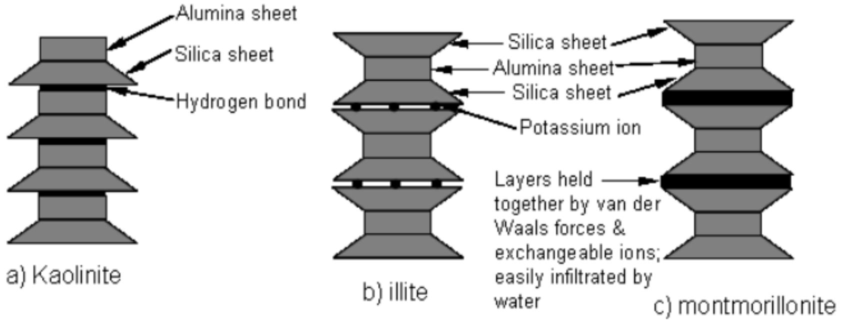


Figure 2.1. Structure of kaolinite, illite and montmorillonite.

2.2.2. Identification from Index properties

Plasticity index, Ip, and the liquid limit, w_L are useful indices for determining the swelling characteristics of most clays. Since the liquid limit and the swelling of clays both depend on the amount of water clay tries to absorb, it is natural that they are related.

The relation between the swelling potential of clays and the plasticity index has been demonstrated in the Table 2.1.

Table 2.1. Relationship between Plasticity Index and Swelling potential (Anderson, K.O. & Thompson, S. , 1969)

Plasticity Index (Ip)	Swelling potential (S)	Degree of Expansion
20	1.5	Low
20-31	1.5-4.0	Medium
31-39	4.0-6.0	High
>39	>6.0	Very High

The swell potential is supposed to be related to the opposite property of the shrinkage limit measured in a very simple test.

Table 2.2. Relationship between shrinkage limit and Degree of expansion (Holtz, 1954)

Shrinkage limit (%)	Degree of Expansion
>15	Low
10-16	Medium
7-12	High
<11	Very high

2.2.3. Identification from Activity

The activity of soil is normally calculated as the ratio of the plasticity index of the soil to its percentage clay size fraction. Expansive soil can be classified based on their activity as shown in the Table 2.3.

Table 2.3. Relation between Soil Activity and Expansive soil property (Holtz, 1954)

Activity (A)	Nature of the soil	Degree of swell potential
<0.75	Inactive	Low
0.75-1.25	Normal	Medium
>1.25	Active	High

2.2.4. Direct measurements of swelling pressure

1. **Oedometer tests**: - the most useful and reliable assessment of the swelling potential of soil could be obtained from the conventional oedometer swell tests.

According to Holtz, (1954) the criterion for the classification of degree of soil expansivity is the total volume change of soil from air dry to a saturated condition under a surcharge of 7 kPa in an oedometer. And according to Seed, H. B. & Lundgren, R., (1962), the criterion for classification of degree of soil expansivity is the percent expansion exhibited by the soil compacted at maximum dry density and OMC under a surcharge of 7 kPa. Table 2.4. shows the relation between Swelling potential with oedometer test result for both scientist's investigation.

Table 2.4. Relation between Swelling potential and Oedometer

Swelling Potential	% Expansion in Oedometer (Holtz, 1956)	% Expansion in Oedometer (Seed et.al, 1962)
Low	<10	0-1.5
Medium	10-20	1.5-5
High	20-30	5-25
Very High	>30	>25

2. **Free swell tests**: - are commonly used for identifying expansive clays and to predict the swelling potential of the soil. It is two types: -

- **Free Swell Value (FSV)**: - this test consists of pouring slowly 10 cm^3 of oven-dried soil passing 425 µm sieve into 100 cm^3 measuring jar filled with distilled water and noting the equilibrium volume of the sediment formed. The free swell value is then calculated as the increase in the volume of the soil expressed as a percentage of the initial volume.
- **Differential Free Swell Test**: - in this method, the initial dry mass of the soil to be taken for the free swell test is 10 g of oven-dried soil samples passing 425 µm sieve placed in 100 ml graduated measuring jars containing distilled water and kerosene respectively, after an equilibrium period of a minimum of 24 hr. the swell potential of the soil based on the Free swell index (FSI) is classified as per the guideline given in the Table 2.5.

Table 2.5. Relation between Swelling potential and FSI (IS: 2720 1977)

Swelling Potential	FSI (%)
Low	<50
Medium	50-100
High	100-200
Very high	>200

- **Free Swell ratio (FSR) method**: - Free swell ratio is defined as the ratio of the equilibrium sediment volume of 10 g oven-dried soil sample passing 425 µm sieve placed in 100 ml graduated measuring jar containing distilled water to that in kerosene, after an equilibration period of a minimum of 24 hr. The guidelines to classify the degree of expansivity of a soil based on its FSR are indicated in the Table 2.6.

Table 2.6. Relation between Swelling potential and Free Swell Ratio (Sridharan, A. & Prakash, K., 2000)

Free Swell Ratio	Clay type	Swelling Potential	Dominant mineral type
<1.0	Non-swelling	Negligible	Kaolinitic
1.0-1.5	Mixture of swelling and non-swelling	Low	Mixture of kaolinitic and montmorillonitic
1.5-2.0	Swelling	Moderate	Montmorillonitic
2.0-4.0	Swelling	High	Montmorillonitic
>4.0	Swelling	Very high	Montmorillonitic

2.3. Remedial measure for expansive soil

2.3.1. Mechanical alternations

This alteration method includes excavation of the reactive soil and replaces with non-expansive material, based on the shallowness of the active zone and availability of suitable replacement material. There are two types: -

- **Sand cushion method: -** In this method, the entire depth of the expansive soil stratum or a part of it may be removed and replaced with a sand cushion, compacted to the desired density and thickness (Phanikumar, B. R. & Suri, S., 2009).
- **Cohesive non-swelling (CNS) layer: -** In this method about top 1 m to 1.2 m of an expansive layer is removed and replaced by a cohesive non-swelling soil layer. Saturation of expansive soil forces is developed up to a depth of 1 to 1.2 m and counteract heave (Phanikumar, B. R. & Suri, S., 2009).

2.3.2. Chemical alternation

This involves the addition of chemicals to expansive clay to reduce the swelling nature of clay minerals. Such alteration includes the addition of lime, cement, gypsum, fly ash, Calcium Chloride ($CaCl_2$), Calcium Sulfate ($CaSo_4$), Potassium Chloride (KCl), Aluminum Chloride ($AlCl_3$), etc.

2.3.3. Adopting special foundation techniques

- **Pad foundations -** pad foundations are a series of individual footing pads placed on the upper soil and spanned by grade beams.
- **Drilled piers: -** piers of little diameter but of significant length are drilled into expansive soils up to the zone unaltered by moisture content.
- **Belled piers:** - These are the piers with enlarged base adopted to increase the carrying capacity.
- **Friction piers:** - Where the bedrock is very deep and upper layers of the soil are expansive; friction piers can be used.
- **Under reamed piles:** - Under reamed piles are bored cast-in-situ concrete piles having one or more number of bulbs formed by enlarging the pile stem.
- **Granular Anchor Pile**: - is one of the recent ground improvement techniques in devised for resisting pull out forces. It is achieved with a slight modification in the granular pile. A

base plate is provided at the bottom of the pile and it is connected with an anchor rod to resist the uplift forces coming to the foundation.

2.4. Some of the advantage of using Granular Anchor Pile (GAP)

1. Granular Anchor Pile (GAP) is one of the recent innovative foundation techniques advised for a structure subjected to tensile/uplift force.
2. Among the other foundation type Granular Anchor Pile the most successful tension-resistant foundation type.
3. Granular Anchor Pile (GAP) is a relatively new, economical, and efficient ground improvement technique.
4. Installation does not require heavy machinery or skilled labour like a pile foundation.

2.5. System for Granular Anchor Pile (GAP) installation

A first thing is a borehole preparation in the soil, and a casing pipe is inserted to support the surrounding soil to prevent the collapse of surrounding soil into the borehole due to insufficient cohesion between the soil particles if necessary. Then a Mild steel anchor plate MS which is connected to the anchor rod at one end is inserted into the borehole and placed at borehole bottom, and then the borehole is filled with granular material. Generally, a well-graded mixture of locally available crushed-stone aggregates and sand is used. The granular material is filled in layers of equal thickness. A uniform compaction procedure is adopted to fill each layer to achieve a uniform density of the material using Vibro-compaction. The other end of the MS anchor rod is connected to footing that is constructed on top of a pile (Kranthikumar et al., 2016).

This arrangement provides tensile resistance to the GAP system and enables it to offer resistance to the uplift force exerted on the foundation by the surrounding soil.

Vibro-compaction is a technique that compacts granular soils and rearranges the soil particulars into a denser state. The compaction is attained using depth vibrators, typically suspended from a crane or mounted on piling equipment. Using depth vibrators, the soil density can be increased and homogenized independently from the groundwater. The positive features of this compaction techniques include fast and economical process, noise and vibration level are low, environmentally friendly as natural and in-situ material is used, can be carried out to almost any depth and suited to soil profiles with a high-water level (Vibro-compaction, 2014).

2.6. Concept of Granular Anchor Pile (GAP)

The Granular Anchor Pile (GPA) is an innovative foundation technique, devised for mitigating the heave of expansive clay and improving their engineering behavior. It is a modification of the conventional granular pile, wherein an anchor is provided in the pile to render it tension resistant (Aljorany, 2014). Granular piles are a well-known ground improvement technique used for reducing the settlement and increasing the load-carrying capacity of soft clay beds (Hugher, J. M. O. & Withers, N. J. , 1974). Figure 2.2. shows the concept of a granular pile anchor GPA and the various forces acting on the foundation. The uplift force *Pu* acting on the base of the foundation in the vertical direction is due to the swelling pressure *Ps* of the expansive soil. This uplift force is resisted by the weight of the granular pile *Wgp* acting in the downward direction. The friction mobilized along with the pile-soil interface also resists the upward movement of the foundation. This friction is generated mainly because of the anchor in the system. The upward resistance is further augmented by the lateral swelling pressure *ksps*, which confines the granular pile anchor radially, increases the friction along with the pile-soil interface, and prevents it from being uplifted (Rao et al., 2007).

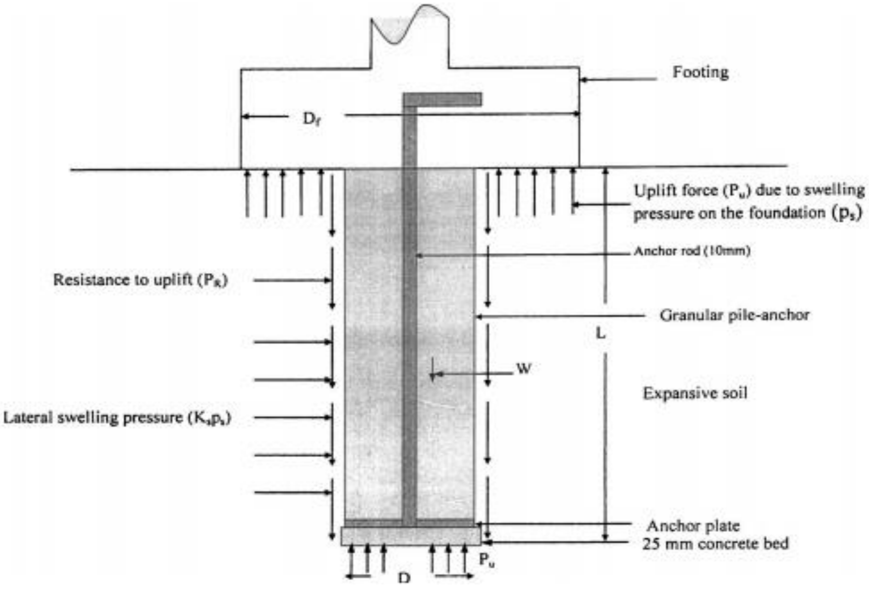


Figure 2.2. Concept of granular Anchor Pile (Rao et al., 2007).

2.7. Limit equilibrium approach for prediction of ultimate uplift capacity of GAPs

The bottom portion of GAP equal to critical height *H*c is considered to bulge due to uniform lateral stress σ_r in subsoil due to a gradual increase in uplift stress q and consequently σ_r in pile body. The cylindrical zone around the bulged pile having a radius R_u will undergo a state of plastic equilibrium. Beyond this zone of plastic equilibrium of radius R_p, the soil is considered to be in an elastic state. The ultimate uplift force applied at the pile top is considered to be resisted by the weight of GAP and force required to provide restraint against bulging of GAP. The ratio of the radius of the plastic zone and cylindrical cavity (R_p/R_u), reduced rigidity index I_{rr} and lateral limiting stress σ_{rL} are parameters controlling uplift capacity (Kumar, et.al., 2018).

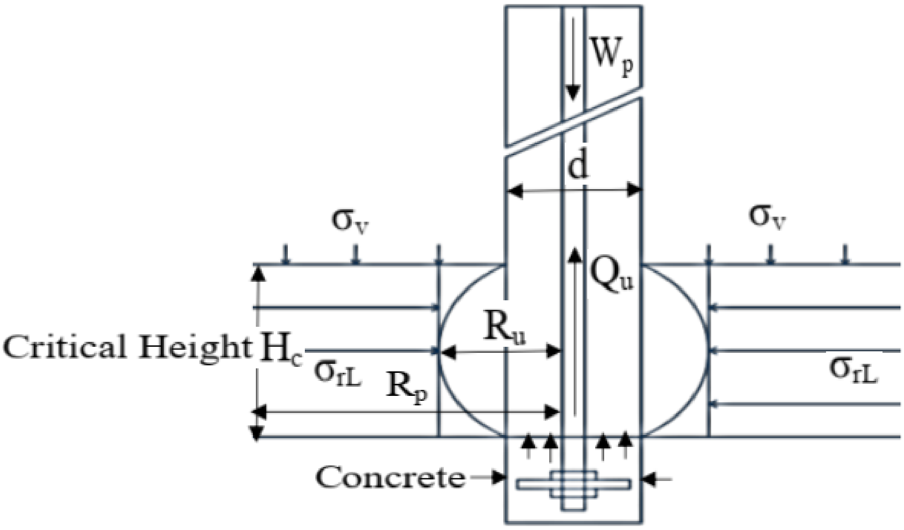


Figure 2.3. Bulging at bottom of GAP (Kumar, et.al., 2018).

- **Ultimate uplift capacity Q_u is calculated using the following steps:**

1. Effective normal stress δv at bulging

   Assume $Hc = 5 * Dpile$ (2.1)

   $\delta v = \gamma * z = \gamma * (L - 5 * D)$ (2.2)

2. Effective mean normal stress δm

   $\delta m = (\frac{1+2Ko}{3})\delta v$ where Ko = 1-sin (1.2ϕ) (2.3)

3. Elastic soil modulus is obtained from

   $Es = 2q_c(1+R_D^2)$ (2.4)

   or using other correlation

But q_c – static cone penetration resistance

R_D – relative density of soil

- Corrected Modulus Ecor. = Es∗ $(\frac{\delta v}{100})^{0.5}$ (2.5)

4. Rigidity Index Ir

$$Ir = [\frac{0.5*\frac{Ecor}{(1+\mu)}}{c+\delta m tan\phi}] \tag{2.6}$$

Therefore Irr = $\frac{Ir}{1+IrEv}$ where Ev = $\frac{\delta m}{Kbulk}$ (2.7)

Dimensionless cavity expansion factor Fq and Fc

$$Fq = (1+sin\emptyset)(Irrsec\emptyset)^{sin\emptyset/(1+sin\emptyset)} \tag{2.8}$$

$Fc = (Fq-1)cot\phi$ and (2.9)

$Fc = 1+lnIrr$ for $\phi = 0$ (2.10)

5. Lateral limiting stress δ_{rL} are

$$\delta rL = FcCu + Fq\delta m \tag{2.11}$$

6. Ultimate resistance in bulging qult

$$qult = Kp\delta rL = \frac{1+sin\emptyset p}{1-sin\emptyset p} * \delta rL \tag{2.12}$$

7. Resistance in bulging Qo

$$Qo = qult * Ap \tag{2.13}$$

8. Weight of GAPs

$$Wp = \gamma p * Ap * L \tag{2.14}$$

9. Ultimate uplift capacity Qu

$$Qu = Qo + Wp \tag{2.15}$$

2.8. Numerical analysis and Geotechnical Problems

Numerical analysis is a method which attempts to satisfy all theoretical requirements, include realistic soil constitutive models and incorporate boundary condition that realistically simulates field conditions. Because of the complexities involved and the nonlinearities in soil behavior, all methods are numerical in nature. Approaches based on Finite difference and Finite element methods are those widely used. Numerical analysis naturally finds application in all fields of engineering and the physical sciences. The overall goal of the field of numerical analysis is the design and analysis of techniques to give approximate but accurate solutions to hard problems.

Soil is a complex multiphase material its stress, strain, and strength are represented by pressure dependency with coupling between shear and volumetric behavior. With these complexities, it is not possible to think in terms of developing a completely generalized model for all soils. It is fundamental to modify the modeling of material behavior to the particular problem of interest and the required accuracy of the solution. This explains why numerical analysis has been used in many cases of geotechnical problems for several years. There are several types of popular software that are used in the numerical analysis of geotechnical problems based on different numerical analysis methods such software are PLAXIS, ABAQUS, FLAC, and so on.

2.9. Theoretical considerations for geotechnical problems

2.9.1. Requirements for a general solution

In general, a theoretical solution must satisfy Equilibrium, Compatibility, material constitutive behavior, and the Boundary condition (Force and displacement).

- **Equilibrium equation**

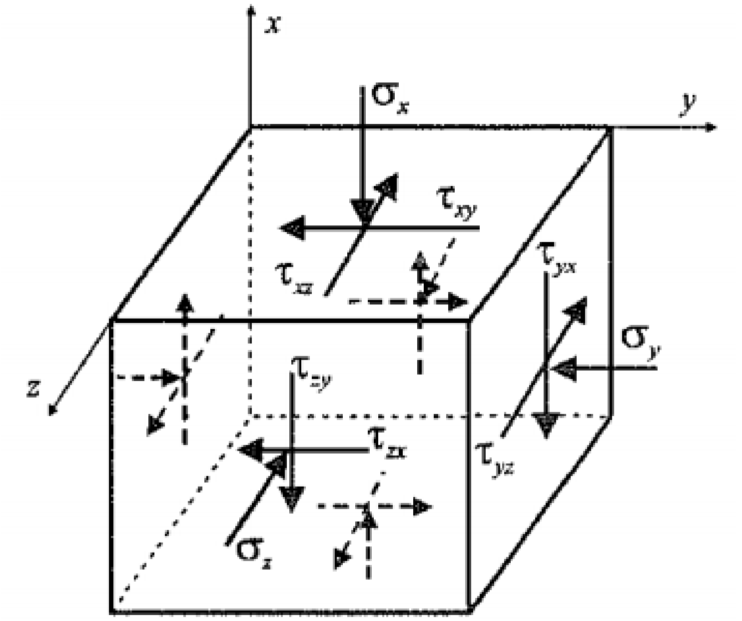


Figure 2.4. Stress on a typical element

$$\frac{\partial \sigma xx}{\partial x} + \frac{\partial \sigma xy}{\partial y} + \frac{\partial \sigma xz}{\partial z} + fx = 0 \quad (2.16)$$

$$\frac{\partial \sigma yx}{\partial x} + \frac{\partial \sigma yy}{\partial y} + \frac{\partial \sigma yz}{\partial z} + fy = 0 \quad (2.17)$$

$$\frac{\partial \sigma zx}{\partial x} + \frac{\partial \sigma zy}{\partial y} + \frac{\partial \sigma zz}{\partial z} + fz = 0 \quad (2.18)$$

Where: -

Fx, Fy, and Fz are body force (such as gravity force) per unit volume

Stress is a tensor

- **Compatibility**

Physical compatibility: - compatible deformation involves no overlapping of material and no generation of holes.

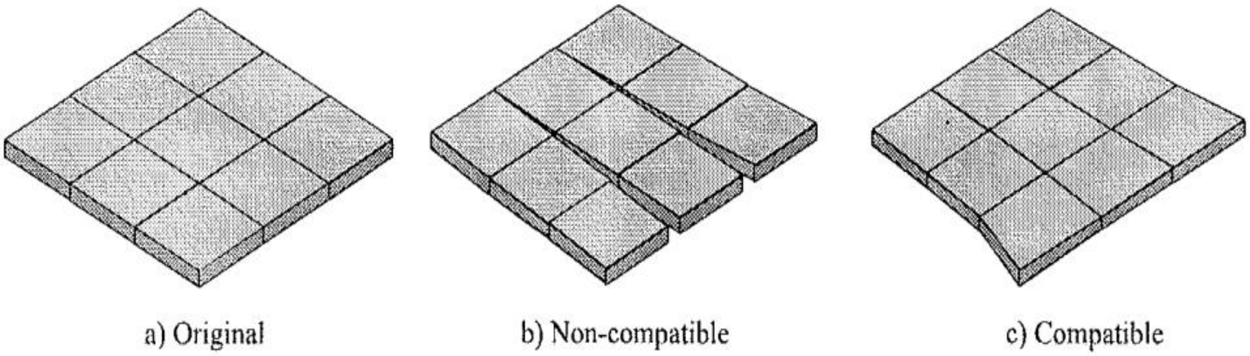


Figure 2.5. Modes of deformation

Mathematical compatibility: - the above physical interpretation of compatibility can be expressed mathematically, by considering the definition of strains.

$$\varepsilon x = \frac{\partial u}{\partial x}\ ; \varepsilon y = \frac{\partial v}{\partial y}\ ; \varepsilon z = \frac{\partial w}{\partial z} \quad (2.19)$$

$$\gamma xy = \frac{\partial v}{\partial x} + \frac{\partial u}{\partial y}\ ; \gamma yz = \frac{\partial w}{\partial y} + \frac{\partial v}{\partial z}\ ; \gamma xz = \frac{\partial w}{\partial x} + \frac{\partial u}{\partial z} \quad (2.20)$$

In matrix form

$$\begin{Bmatrix} \varepsilon x \\ \varepsilon y \\ \varepsilon z \\ \gamma xy \\ \gamma yz \\ \gamma xz \end{Bmatrix} = \begin{bmatrix} \frac{\partial}{\partial x} & 0 & 0 \\ 0 & \frac{\partial}{\partial y} & 0 \\ 0 & 0 & \frac{\partial}{\partial z} \\ \frac{\partial}{\partial y} & \frac{\partial}{\partial x} & 0 \\ 0 & \frac{\partial}{\partial z} & \frac{\partial}{\partial y} \\ \frac{\partial}{\partial z} & 0 & \frac{\partial}{\partial x} \end{bmatrix} \begin{Bmatrix} u \\ v \\ w \end{Bmatrix}$$

$$\varepsilon = Du \tag{2.21}$$

Where; - D- constitutive matrix

This mathematical equation express **Strain and displacement** relationship

- **Equilibrium and compatibility conditions**

Combining the equilibrium and compatibility, gives: -

Unknowns: - 6 stresses + 6 strains + 3 displacement = 15

Equation: - 3 equilibrium + 6 compatibility = 9

To obtain a solution, therefore, requires 6 more equations. These come from the constitutive relationship.

- Constitutive behaviour: -

Is the strain-stress behavior of the soil. It usually takes the form of a relationship between stresses and strains and therefore provides a link between equilibrium and compatibility. And expressed mathematically: -

$$\varepsilon xx \ = \frac{1}{E}\ (\sigma xx - \nu(\sigma yy + \sigma zz)) + 0\sigma xy + 0\sigma xz + 0\sigma yz \tag{2.22}$$

$$\varepsilon yy \ = \frac{1}{E}\ (\sigma yy - \nu(\sigma xx + \sigma zz)) + 0\sigma xy + 0\sigma xz + 0\sigma yz \tag{2.23}$$

$$\varepsilon zz \ = \frac{1}{E}\ (\sigma zz - \nu(\sigma xx + \sigma yy) + 0\sigma xy + 0\sigma xz + 0\sigma yz \tag{2.24}$$

$$\varepsilon xy \;=\; 0\sigma xx + 0\sigma yy + 0\sigma zz + \frac{2(1+\nu)}{E}\sigma xy + 0\sigma xz + 0\sigma yz \tag{2.25}$$

$$\varepsilon xy \;=\; 0\sigma xx + 0\sigma yy + 0\sigma zz + 0\sigma xy + \frac{2(1+\nu)}{E}\sigma xz + 0\sigma yz \tag{2.26}$$

$$\varepsilon yz \;=\; 0\sigma xx + 0\sigma yy + 0\sigma zz + 0\sigma xy + 0\sigma xz + \frac{2(1+\nu)}{E}\sigma yz \tag{2.27}$$

In matrix Form

$$\begin{Bmatrix} \varepsilon xx \\ \varepsilon yy \\ \varepsilon zz \\ \varepsilon xy \\ \varepsilon xz \\ \varepsilon yz \end{Bmatrix} = \frac{1}{E}\begin{bmatrix} 1 & -\nu & -\nu & 0 & 0 & 0 \\ -\nu & 1 & -\nu & 0 & 0 & 0 \\ -\nu & -\nu & 1 & 0 & 0 & 0 \\ 0 & 0 & 0 & 2(1+\nu) & 0 & 0 \\ 0 & 0 & 0 & 0 & 2(1+\nu) & 0 \\ 0 & 0 & 0 & 0 & 0 & 2(1+\nu) \end{bmatrix}\begin{Bmatrix} \sigma xx \\ \sigma yy \\ \sigma zz \\ \sigma xy \\ \sigma xz \\ \sigma yz \end{Bmatrix}$$

$$\{\varepsilon\} = E^{-1}\{\sigma\} \tag{2.28}$$

- $\{\sigma\}$ = E $\{\varepsilon\}$ (2.29)

Where: - [E] is a constitutive matrix

As noted above, fundamental considerations assert that for an exact theoretical solution the requirements of equilibrium, compatibility, material behavior, and boundary conditions must all be satisfied.

To apply the above concepts to a real geotechnical problem, certain assumptions and idealizations must be made. In particular, it is necessary to specify soil behavior in the form of a mathematical constitutive relationship. It may also be necessary to simplify and/or idealize the geometry and/or boundary conditions of the problem.

2.10. Mathematical Expression for GAP system

A single granular pile of length *L*, and diameter *d* with a modulus of deformation E and Poisson's ratio μ, is considered for analysis. A typical element of the granular pile is presented in Figure 2.6, along with various forces acting on it.

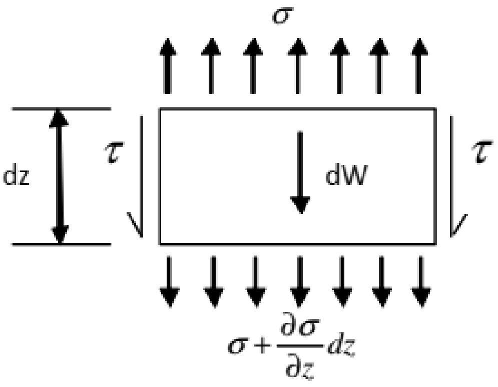


Figure 2.6. Equilibrium of a GAP Element

If γ is the unit weight of GP material, then the weight of element of length dz is given by $d\ W = \gamma Ad\ z$, where A is the cross-sectional area of the element ($\frac{\pi d^2}{4}$) and d is the diameter of the element. The axial stress σ and shear stress τ can be linked by considering the equilibrium of vertical forces on the element.

$$\sigma * A\ -\ (\sigma + \frac{\partial\sigma}{\partial z}dz) * A\ -\ \tau * (\pi d) * dz\ -\ \gamma * A * dz = 0 \qquad (2.30)$$

Then Equation 2.30 can be reduced to Equation 2.31 as shown

$$\frac{\partial\sigma}{\partial z} = -(4\frac{\tau}{d} + \gamma) \qquad (2.31)$$

The axial stress "σ" and shear stress "τ" can be related to displacement "u" with the following relation.

$$\sigma = E\varepsilon_z = E\,\frac{du}{dz} \qquad (2.32)$$

And $\tau = -ku$ (2.33)

In the above expression k is the shear interaction coefficient for the soil-GP interface. The value of which is related to shear modulus G of soil by the expression given by Scott, (1981) as

$$k = 2 * G * d * ln(50) \qquad (2.34)$$

Combining Equation 2.32 and 2.33 into Equation 2.31, the following governing differential equation can be obtained.

$$\frac{\partial^2 u}{\partial z^2} - \frac{4*k*u}{d*E} = -\frac{\gamma}{E} \qquad (2.35)$$

If the constant a is assumed as,

$$\alpha = \sqrt{\frac{4k}{dE}}. \qquad (2.36)$$

Then, the general solution of the above linear constant-coefficient nonhomogeneous differential equation (Equation 2.35) is given by the following expression:

$$u = C1e^{\alpha z} + C2e^{-\alpha z} + \frac{\gamma d}{4k} \qquad (2.37)$$

Constants C_1 and C_2, can be obtained using the boundary condition at the top and tip of the granular pile.

$\sigma = \sigma\text{o}\ at\ z = 0\ and\ \sigma = 0\ at\ z = -L$

After implementing the above two boundary conditions, the final form of the solution is given by the following expression.

$$u = \frac{\sigma o}{E\alpha(1-e^{-2\alpha L})} * e^{\alpha z}(1 + e^{-2\alpha(z+L)}) + \frac{\gamma * d}{4k} \qquad (2.38)$$

The axial stress in the granular pile can be obtained from differentiating the above equation and then multiplying by the modulus of deformation E.

$$\sigma = E\varepsilon$$

$$\sigma = E\frac{du}{dz}$$

$$\sigma = E\frac{d}{dz}(\frac{\sigma o}{E\alpha(1 - e^{-2\alpha L})} * e^{\alpha z}(1 + e^{-2\alpha(z+L)}) + \frac{\gamma * d}{4k})$$

Therefore, $\sigma = \frac{\sigma o}{1-e^{-2\alpha L}} e^{\alpha z}(1 - e^{-2\alpha(z+L)})$ (2.39)

The general solution that is determined in equation 2.38 and 2.39 can be used to determine the displacement and axial stress at any depth "z" in the pile length.

2.11. Finite Element for 3D problems

For the 3D element, it is difficult to determine the exact solution for each discretized element based on the above exact equations but the solution can be approximated using the Finite element method.

The finite element method generally involves the following steps.

1. Discretization of the Domain

This is the process of modeling the geometry of the problem under investigation by an assemblage of small regions, termed finite element. These elements have nodes defined on the element boundaries, or within the element.

2. Element equations

Use of an appropriate variational principle to derive element equations.

$$\mathbf{ku} = \mathbf{f} \tag{2.40}$$

Where: - **k** – (element) stiffness matrix, **u** – (element nodal) displacement and **f** – (element nodal) force vector.

Displacement **u** within an element is interpolated from nodal d.o.f. **d**; that is, **u=Nd**, where **N** is the shape function matrix which is used to relate displacement function with nodal displacement. If nodes have only translational (displacement) d.o.f and n is the number of nodes per element, **N** has 3n column for a 3D element. Thus, for 3D solids, **u=Nd** is

$$\begin{bmatrix} u \\ v \\ w \end{bmatrix} = \begin{bmatrix} N1 & 0 & 0 & N2 & 0 & 0 & \cdots \\ 0 & N1 & 0 & 0 & N2 & 0 & \cdots \\ 0 & 0 & N1 & 0 & 0 & N2 & \cdots \end{bmatrix} \begin{bmatrix} u1 \\ v1 \\ w1 \\ u2 \\ v2 \\ w2 \\ \vdots \end{bmatrix}$$

Formulas for k: - Substitution of **u=Nd** into the strain-displacement relation yields the strain-displacement matrix **B**. which in turn enters the integrand of the formula for stiffness matrix **k**, with n number of nodes per element, and translational d.o.f. only, these relations for 3D are as follows.

$$\underset{6\times 1}{\boldsymbol{\varepsilon}} = \underset{6\times 3n}{\mathbf{B}}\ \underset{3n\times 1}{\mathbf{d}} \tag{2.41}$$

$$\underset{3n\times 3n}{\mathbf{k}} = \int\int\int \mathbf{B}^T\ \underset{6\times 6}{\mathbf{E}}\ \mathbf{B}\ dx\ dy\ dz \tag{2.42}$$

3. Global equation

Combine element equations to form global equations

$$\mathbf{KU} = \mathbf{F} \tag{2.43}$$

Where: - **K** – global stiffness matrix, **U** – the vector of all incremental nodal displacement, **F** – the vector of all incremental nodal forces

4. Apply Boundary conditions

Formulate boundary conditions and modify global equations. Loads (e.g. line and point load, pressures and body forces) affects **F**, while the displacements affect **U**.

5. Solving the global equations and Post-processing

The global equation (2.43) is in the form of a large number of simultaneous equations. These are solved to obtain the displacements **U** at all the nodes. From these nodal displacement's secondary quantities, such as stresses and strains are evaluated using post-processing.

However, it is complex to determine the displacement at each node using hand calculation. Because of the largeness of the elemental as well as global stiffness matrix. Therefore, it needs a finite element supporting software. Since in this research the numerical analysis is done using a PLAXIS 3D program. It is an advanced computer program that performs three-dimensional finite element analyses (FEA) of deformation and stability in geotechnical engineering and rock mechanics.

2.12. Previous Investigation on Granular Anchor pile (GAP)

Rao, et.al., (2007) reported the result of a field-scale test program conducted to study the pull-out response of GAP embedded in expansive clay beds.

These pull-out load tests were conducted on GAPs by varying the length and diameter of the anchor pile. And also, one pull-out test was performed on a group of GAP system to study the group effect on the behavior of the GAP under pull-out load. Then finally the following results were found: -

- The upward load required to be applied to the GAP to cause a given upward movement increased with the increasing length of GAP. For example, for GAPs of length 500, 750, and 1000 mm, the uplift load required to cause an upward movement of 25 mm in the GAP was 9, 12, and 14 kN, respectively. When the length of the GAP was increased from 500 to 750 and 1000 mm, the percentage increase in the uplift load required for an upward movement of 25 mm was 33.3 and 55.5 %, respectively.

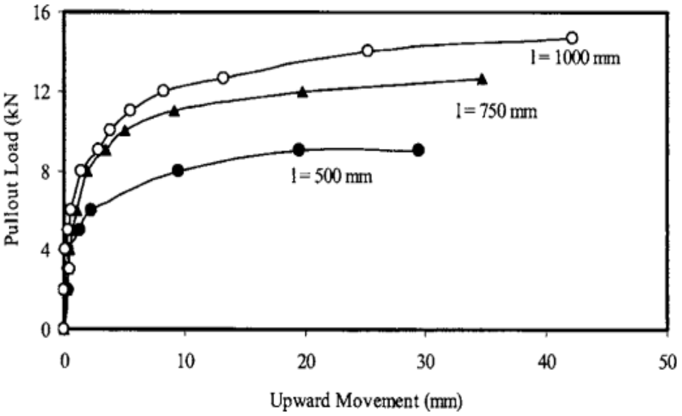


Figure 2.7. Pull-out behavior of granular pile anchors of diameter 200 mm (Rao, et.al., 2007).

- The uplift load or failure pull-out load increased with the increasing diameter of the GAP also. This is because the resistance to uplift increased with increasing surface area of the pile-soil interface consequent upon an increase in the diameter. For example, the uplift load required to be applied on the GAP of constant length L =1000 mm for an upward movement of 25 mm was respectively 11, 11.5, and 14.2 kN for GAP of diameter 100, 150 and 200 mm.

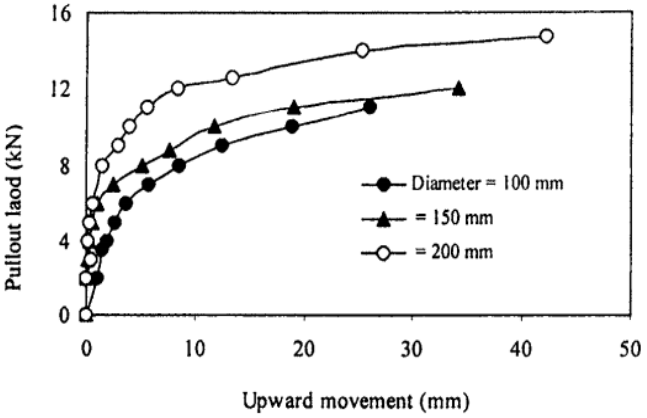


Figure 2.8. Pull-out behavior of granular pile anchors of 1000 mm length (Rao, et.al., 2007).

- The granular pile anchor under the effect of a group of five GAPs resulted in an increased uplift load for a given upward movement in comparison to that of the GAP when tested as a single pile anchor. This is because of the group action of GAPs. The failure pull-out load of the GAP when tested under group was 18 kN as against a failure pull-out load of 12 kN for the GAP when tested single, indicating a 50 % improvement in the failure pull-out load.

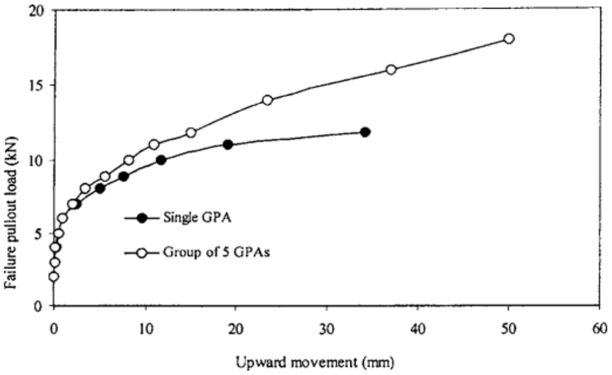


Figure 2.9. Pull-out behavior of single granular pile anchor and a group of granular pile anchors diameter150 mm and length1000 mm (Rao, et.al., 2007).

Ismail, et.al., (2011) presents results from finite element analysis undertaken on Granular Anchor Pile foundations in a reactive soil using PLAXIS 2D software. The study investigates the ability of a single pile to resist forces induced by heave and shrinkage. The analysis result confirms the potential of the GAP system and indicates that the resistance to both heave and shrinkage can improve dramatically. Figure 2.10 and 2.11 shows the performance of the GAP in a case of heave and shrinkage of the reactive soil respectively.

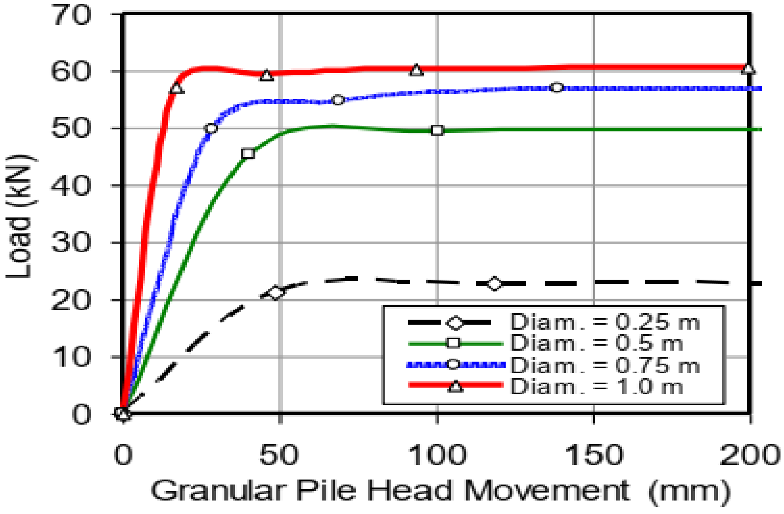


Figure 2.10. Load Displacement Curve of Pile Anchor (Ismail, et.al., 2011).

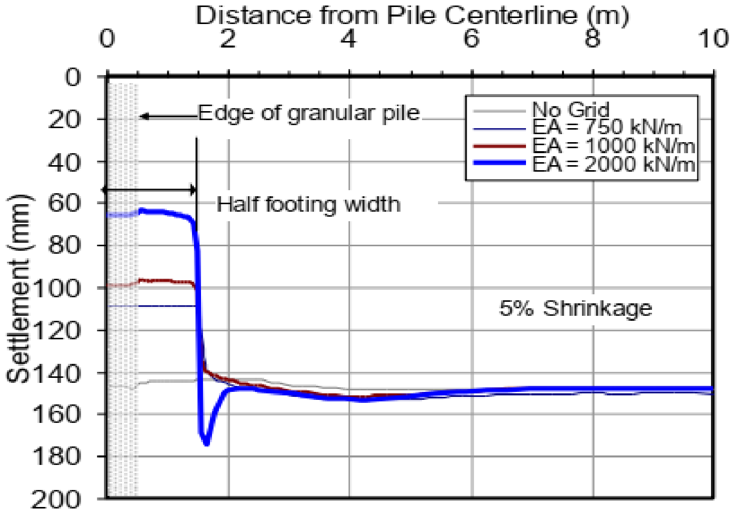


Figure 2.11. Settlement trough during a shrinkage event (Ismail, et.al., 2011).

Sivakumar, et.al., (2012) study the load-displacement response, failure mode, and ultimate pullout capacity of granular anchor pile. And importantly how this anchor can be appropriately integrated into routine Civil Engineering construction.

The experimental studies reported in this paper were performed in two parts. The focus of the first part was to compare the ultimate pull-out capacity of granular anchors in direct pullout against that of conventional concrete anchors cast in situ. These tests were performed at old filled deposits, with the experimental program considering the assessment of two variables; namely anchor lengths (*L*) of 0.5, 1.0, and 1.5 m, and anchor diameters (*D*) of 0.07 and 0.15 m.

The second part of the investigation was performed at lodgment fill deposit and examined in greater detail the performance of granular anchors having different configurations, again considering the two variables of length *L* = 0.45 – 1.62 m and diameter *D* = 0.15 or 0.20 m nominally.

The result shows that in the first part the pull-out capacities of the granular and concrete anchors of *L* × *D* = 0.5 m × 0.07 m were 5.5 and 5.2 kN respectively. Both of these anchors failed on shaft resistance mobilized along the column length. The soil surrounding the concrete anchor did not undergo any significant displacement (either heave or subsidence) until the failure state was achieved. However, the soil surrounding the granular anchor progressively heaved as the anchor was incrementally loaded to failure. Anchors of *L* × *D* = 0.5 m × 0.15 m also failed on shaft resistance experiencing ductile and sudden pull-out behavior for granular and concrete constructions, respectively, with mobilized pull-out capacities of 6.7 and 8.0 kN respectively. The granular anchor of *L* × *D* = 1.0 m × 0.07 m experienced ductile failure, undergoing localized end-bulging whereas the concrete anchor experienced sudden pull-out, failing in shaft resistance. Pull-out capacities of 16.1 and 16.3 kN were mobilized for these granular and concrete columns respectively.

The experimental results of the second part of the study Short anchors of *L* × *D* = 0.5 m × 0.196 m and 0.45 m × 0.148 m failed on shaft resistance, mobilizing a pull-out capacity of 12 kN. An increase in anchor length and (or) diameter produced an increase in pull-out capacity. Anchors having *L* = 0.96, 1.0, and 1.3 m with *D* = 0.219 m mobilized pull-out capacities of 39, 42, and 44 kN, respectively. In the case of 0.168 m diameter anchors, the pull-out capacities were 33, 40, and 42 kN for *L* = 0.8, 1.47, and 1.62 m, respectively.

Krishna, et.al., (2013) conducted laboratory and field test on Granular Anchor Pile and Conventional Concrete Pile to study the pull-out capacity of the piles and the results of the two piles were finally compared. Tests were carried out both in unsaturated and in fully wet conditions of soil mass to estimate the loss of pull-out capacity upon wetting. The tables 2.9. show the laboratory and field test results on the piles.

Table 2.9. The pull-out capacity of piles in the laboratory (Krishna, et.al., 2013) (Diameter of plie = 500 mm and Length of pile = 200 mm)

S/N	Type of Pile	Pull-out load (N)	Pull-out resistance (kN/m^2)
Unsaturated Condition			
1	Concrete Pile	1900	60.5
2	Granular Anchor Pile	6600	210.1
Saturated Condition			
1	Concrete Pile	1700	54.1
2	Granular Anchor Pile	5300	1687

Table 2.10. Pull-out capacity of piles in the field (Krishna, et.al., 2013)

S/N	Type of Pile	Condition	Length (m)	Diameter (m)	Uplift Load (N)	Uplift resistance (kN/m^2)
1	GAP	Un-Saturated	1.0	0.10	61580	196.0
2	GAP	Un-Saturated	1.5	0.10	95190	202.0
3	GAP	Un-Saturated	1.0	0.15	98140	208.3
4	GAP	Saturated	1.0	0.10	45500	144.8
5	GAP	Saturated	1.5	0.10	71160	151.0
6	GAP	Saturated	1.0	0.15	73520	156.0
7	CP	Un-Saturated	1.0	0.10	22240	70.8
8	CP	Un-Saturated	1.5	0.10	40010	84.9
9	CP	Un-Saturated	1.0	0.15	43000	91.2
10	CP	Saturated	1.0	0.10	15400	49.0
11	CP	Saturated	1.5	0.10	26400	56.0
12	CP	Saturated	1.0	0.15	28900	61.3

According to the above table result, the pull-out tests both in laboratory and field under both saturated and unsaturated conditions have revealed that the pull-out resistance of the granular anchor piles is around 3 to 4 times that of similar concrete piles. From the laboratory test results, it can be observed that on saturation, the pull-out resistance is decreasing by about 14 % that of the unsaturated condition in case of granular anchor piles, whereas it is decreasing by 26 % in case of concrete piles. And in case of field studies, the decrease is by about 25 % for granular anchor piles and by 32 % for concrete piles.

The high value of pull-out resistance for granular anchor piles could be attributed to thorough packing and lateral displacement of granular fill due to ramming. Such lateral displacement of granular fill develops close interaction and interlocking with the sides of the borehole; which is questionable in the case of concrete piles.

Aljorany, et.al., (2014) conducted a laboratory test on an experimental model and numerical modeling and analysis using PLAXIS 2D software to study the behavior of GAP foundation system in expansive soil. The effects of different parameters, such as GAP length, diameter, footing diameter, expansive clay layer thickness, and presence of non-expansive clay are studied. The result proved the efficiency of GAP in reducing the heave of expansive soil. The heave of GAP is controlled by three independent variables these are (L/D) ratio, (L/H) ratio, and (B/D) ratio. The heave can be reduced by up to (38 %) when (GAP) is embedded in expansive soil layer at (L/H = 1) and reduced by about (90 %) when (GAP) is embedded in expansive soil and extended to non-expansive clay (stable zone) at (L/H = 2) at the same diameter of (GAP) and footing.

Table 2.8. Summary of the maximum heave of unreinforced and expansive soil reinforced with (GPA) models at different lengths and diameters (Aljorany, et.al., 2014)

GAP Diameter (m)	GAP length (m)	L/D ratio	L/H ratio	B/D ratio	Maximum Heave (mm)
Footing resting on Unreinforced Expansive Soil					260.00
0.20	2.00	10.00	0.50	10.00	225.00
	4.00	20.00	1.00		189.00
	6.00	30.00	1.50		130.00
	8.00	40.00	2.00		115.00
0.40	2.00	5.00	0.50	5.00	215.00
	4.00	10.00	1.00		170.00
	6.00	15.00	1.50		100.00
	8.00	20.00	2.00		84.00
0.60	2.00	3.33	0.50	3.33	209.00

	4.00	6.67	1.00		124.00
	6.00	10.00	1.50		56.00
	8.00	13.33	2.00		40.00
0.80	2.00	2.50	0.50	2.50	204.00
	4.00	5.00	1.00		81.00
	6.00	7.50	1.50		38.00
	8.00	10.00	2.00		25.00

Phanikumar, (2016) performs a laboratory experiment on GAPs reinforced with geogrids to present the effect of the number of geogrid layers, the spacing between them, and the location of bottom geogrid from the anchor plate on heave and pull-out behavior of GAP.

To distinguish the clear effect of geogrid reinforcement the test was conducted based on three conditions such as test on unreinforced expansive clay beds, test on clay beds reinforced with plain GAP without any geogrid reinforcement, and test on clay beds reinforced with GAP and Geogrid reinforcement. The result shows that an expansive clay bed reinforced with a plain GAP resulted in a 31 % reduction in heave compared to that of the unreinforced expansive clay bed. Reinforcing the bottom portion of the GAP with geogrids of n = 2, 3, and 4 reduced the heave further by 40, 48, and 57 % respectively. Further, heave decreased with decreasing spacing between geogrids. When the spacing decreased from 30 to 20 mm, heave decreased by 18 % in clay bed reinforced with GAP having geogrid reinforcement layers of n = 3. The closer the bottom geogrid to the anchor plate of GAP, the greater was the increase in the uplift capacity. This improvement is due to Reinforcing the granular pile with horizontal geogrid layers that can minimize the bulging of granular material through interlocking. Geogrid reinforcement prevents granular material reorientation due to uplift stress concentration on the anchor plate, leading to failure.

Table 2.11. Uplift capacity (N) of granular pile-anchors reinforced with geogrids (Phanikumar, 2016)

Spacing = 20,30 mm						
	Uo = 20 mm		Uo = 30 mm		Uo = 40 mm	
	Pu	% increase	Pu	% increase	Pu	% increase

n = 2	160,156	12.3, 9.47	156, 150	9.47, 5.26	150, 147	5.26, 3.15
n =3	192,183	34.7, 28.42	187, 178	31.23, 24.49	183, 173	28.42, 21.4
n = 4	236,222	65.61, 55.78	227, 214	59.29, 50.17	222, 204	55.78, 43.15

Johnson, et.al., (2016) presented an investigation on the Granular Anchor Pile (GAP) system by studying the effect of the relative density of fill material, granular pile diameter on the pull-out capacity of the Granular Anchor Pile and the comparison of encased and the non-encased granular pile has been done using laboratory model test on black cotton soil collected from Coimbatore, Tamil Nadu at a depth of 1 m from the ground level and it was blackish. According to the research, the relative density of sand was varied as 50 %, 60 %, and 70 %. The pull-out test is conducted for a 30 mm diameter anchor pile at a water content of 40 % for surrounding clay bed. According to the result, the ultimate pull-out load increases with an increase in the relative density of the fill material. The pull-out test was also conducted to study the effect of change in the diameter of the anchor pile on the pull-out behavior by keeping constant the water content of clay bed and relative density of fill material. So, the result showed that the ultimate pull-out load increases with an increase in the diameter of the granular pile anchor. For a relative density of 50 %, there is a percentage increase of 30 % when the diameter was increased from 30 mm to 50 mm. The pull-out tests were carried out on a granular pile encased with non-woven geotextile. The tests were carried out for 30 mm granular pile. The obtained results were compared with the results of a non-encased granular pile having a diameter of 30 mm. The tests were performed at a water content of 40 % and relative density 60 %. The result showed that the ultimate pull-out load for the encased pile is greater when compared to that of the non-encased pile.

Kranthikumar, et.al., (2016) evaluated the performance of a group of GAPs under uplift loading conditions by using finite element analysis with PLAXIS 3D software. An Attempt was made to investigate the influence of factors such as the number of piles, depth of water table, soil-pile property on the uplift capacity of the GAP system, and group efficiency of the GAP system in loose sandy soils under uplift loading was also determined.

According to the study result, the uplift resistance of the GAP system in a loose sandy soil increases with increase in the length and diameter of the pile, and The efficiency of pile group decreases with increase in the number of piles for constant spacing, because if the number of piles in a group increases, the load-carrying capacity of pile group may reduce due to the overlapping of the stresses transmitting in the piles to the surrounding soil. Also, the uplift capacity decreases with increasing the depth of the water table.

Additionally, the Efficiency of group piles of the GAP system lies between 0.9 and 1.0 for a different number of piles with different L/D ratios with fixed Diameter of the pile.

Table 2.7. Uplift capacity and efficiency of a GAP system for a given diameter D = 0.5 m (Kranthikumar, et.al., 2016).

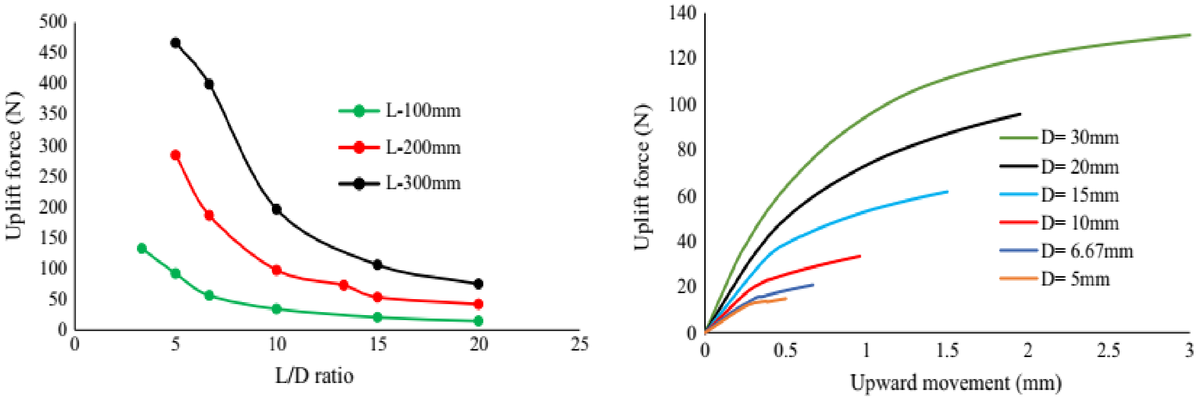
Diameter (m)	Length (m)	L/D	Uplift Capacity (kN)				Efficiency (%)		
			1 pile	2 piles	3 piles	4 piles	2 piles	3 piles	4 piles
0.5	2.5	5	62.5	119.32	174.25	230.72	95.47	92.94	92.3
0.5	3.75	7.5	92.5	181.65	265.96	350.19	98.1	95.75	94.56
0.5	5	10	112.53	221.55	326.48	431.59	98.44	96.71	95.88
0.5	6.25	12.5	128.7	254.36	375.36	497.14	98.81	97.22	96.57
0.5	7.5	15	144.1	284.68	420.59	557.8	98.79	97.3	96.79

Kranthikumar, et.al., (2016) again conducted a small-scale laboratory test and finite-element analysis using PLAXIS 3D to study the performance of the GAP system in loose sandy soil. The effects of key parameters, such as length and diameter of the pile and elastic modulus of the surrounding soil, on the uplift capacity of a GAP, were examined. The load-displacement response and ultimate pullout capacity of the GAP were also analyzed. According to the result the uplift resistance of the GAP system in a loose sandy soil increases with the embedded length, the diameter of the pile, and elasticity modulus of soil, The bulging failure occurs in the GAP system for higher modulus ratios, but the shaft failure occurs for lower values of the modulus ratio for a constant elastic modulus of a pile and Relative density of the surrounding soil has considerable impact on the ultimate capacity. An increase in the ultimate capacity is almost linear with the increase in relative density.

Bharati, et.al., (2017) conducted a laboratory model test to study the efficiency of Granular Anchor Pile in reducing heave of foundations, as natural stone is used for the installation. The laboratory test was done on GAP with fixed length L = 300 mm and varying the Diameter D = 40, 50, 60 mm to show the swelling potential of the pile with time and the result is compared with foundation rested on untreated soil.

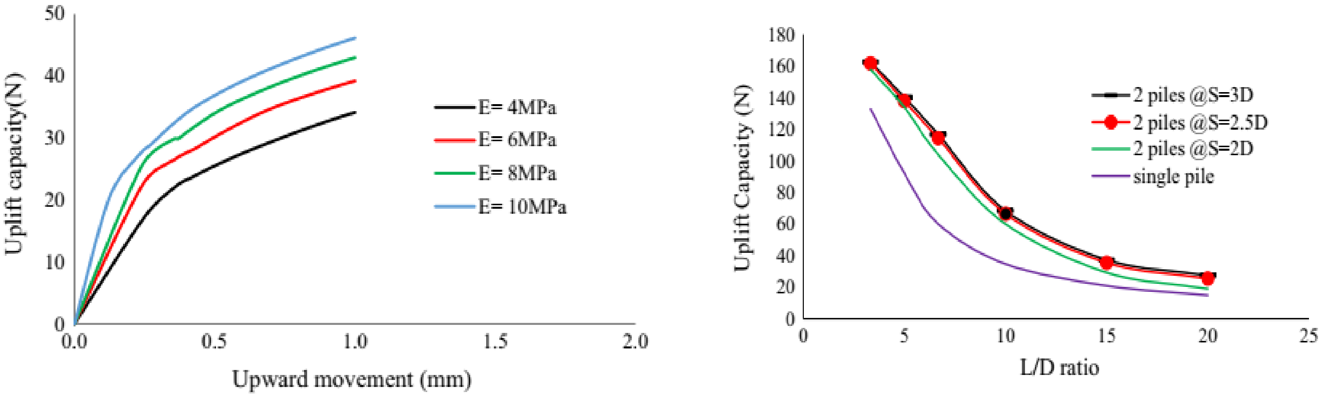
Additionally, this study covers the effect of pile spacing in the response of the pile to the uplift forces by conducting the effect of 2 piles with 2d and 2.5d spacing. Finally, the result shows that the swelling potential of an expansive soil bed decreases with an increase in the surface area of the pile. This means that the increase of the pile diameter results in a greater reduction of swell potential and the heave decreases with the decrease in spacing between the piles. When the spacing is equal to 2 times the diameter, the heave is reduced to an insignificant value.

Sharma, et.al., (2019) undertake a numerical study to predict the uplift performance of Granular Pile Anchors in expansive soil using 3D finite element PLAXIS software. The effect of length (L), diameter (D), and a number of the GAP at different spacing (s) values on uplift capacity are analyzed by applying prescribed displacement of 10 % of pile diameter at the center of granular pile anchor. Further, length to diameter (L/D) ratio of single pile and group pile was analyzed using different parameters. The resistance of granular pile anchors against uplift forces with a variation of modulus of elasticity of soil is also analyzed. Finally observed that the uplift capacity of granular pile anchor increased with an increase in length and diameter of the pile for both single pile and piles arranged in a group. Further, on the increasing value of modulus of elasticity of soil, there is an increase in pull-out resistance.



(a) (b)

Figure 2.12. a) Effect of Length, L and length to diameter ratio L/D b) Effect of diameter



(c)

(d)

Figure 2.13. c) Effect of modulus of elasticity of soil d) Effect of spacing of pile

Table 2.12. Literature Summary

S/N	Autor’s Name	Objectives	Methodology used	Description	Research output
1	Kranthikumar et.al., (2016)	To evaluate the performance of GAP under uplift loading of Group pile.	Numerical method using FEM (PLAXIS 3D)	• Loose sandy soil was investigated • Mohr-coulomb model used • Applied prescribed, displacement of 10*D on the top of the footing, • Analyze the influence factor of: - 3 different pile configuration, L/D ratio (D=0.5 m and L = 2.5, 3.75, 5, 6.25, 7.5 m) • Thickness of soil layer = 10 m	The uplift resistance of the GAP system in a loose sandy soil increases with increase in the length and diameter of the pile, and the efficiency of pile group decreases with increase in the number of piles for constant spacing.
2	Kranthikumar et.al., (2016)	To study the behavior of single GAP in loose sandy soil.	Experimental study (small-scale laboratory test)	• Loose sandy soil was investigated • Sample taken from 1-1.5 depth from the surface. • Analyze effect of pile length and diameter (L= 10, 20, 30,40 c.m and D= 1,2,3,4 c.m)	The uplift resistance of the GAP system in a loose sandy soil increases with the embedded length, diameter of the pile, and elasticity modulus of surrounding soil.
			Numerical method using FEM (PLAXIS 3D)	• Loose sandy soil was investigated • Mohr-coulomb model used • Up-ward prescribed displacement of 0.1 m was applied • Analysed the effect of L and D and elastic modulus (L = 2, 3, 4, 5, 6 m and D = 0.2, 0.3, 0.4, 0.5 m) • Thickness of soil layer = 4 m	

				• Shallow square footing of 2m width was used	
3	Rao et.al., (2007)	To study the pull-out response of single and group of GAP embedded in expansive soil.	Experimental study (Field scale test)	• Expansive clay bed was investigated • Sample taken from 1.5 m depth • Analyze the effect of single pile with (L = 0.5, 0.75, 1 m and D = 0.1, 0.15, 0.2 m) • Analyze the effect of group of 6 pile with (L = 1 m and D = 0.15 m)	The pull-out capacity of the pile increases with increasing length and diameter of the pile and have more uplift resistance when act as a group.
4	Johnson et.al., (2016)	To investigate the effect of relative density of fill material, GAP diameter and comparison of encased and non-encased GAP on single pile system	Experimental study (Laboratory model test)	• Black cotton soil was investigated • Sample taken from 1m depth • Relative density = 50, 60 and 70 % and D= 30 and 50 mm with L = 300 mm	According to the result ultimate pull-out load increases with increase in the relative density of the fill material, and increase in the diameter of the granular pile anchor. the ultimate pull-out load for the encased pile is greater when compared to that of the non-encased pile.
5	Aljorany et.al., (2014)	To study the heave behavior of GAP system based on different factor	Experimental study (Laboratory model test)	• Expansive soil was investigated • Sample taken from 1-1.5 m • Analysed the effect of L, D, Footing area and expansive soil thickness (L = 10, 20, 30, 40 c.m and D = 1, 2, 3, 4 c.m)	The capacity of GAP increases with increasing the dimension of the pile and when the pile is

		like L, D, footing diameter and expansive soil layer thickness on a single pile system	Numerical method using FEM (PLAXIS 2D)	• Expansive soil was investigated • Mohr-coulomb model used • Swelling of soil was applied by positive volumetric strain of 6.5 % • Analysed the effect of L, D, Footing area and expansive soil thickness (L = 2, 3, 4, 5, 6, 7, 8 m and D = 0.2, 0.3, 0.4, 0.5, 0.6, 0.7, 0.8 m) • Thickness of expansive soil = 4 m • Shallow circular footing of 2 m width was used	penetrated to the stable zone or underlaid soil
6	Ismail et.al., 2011	To investigate the behavior of a footing reinforced with either a single or a group of piles	Numerical method using FEM (PLAXIS 2D and 3D)	• Reactive soil was investigated • Hardening soil model was used • Swell/shrink were applied using volumetric strain • To analysed single pile L = 3 m, D = 0.25, 0.5, 0.75, 1 m and Footing diameter of 2 m were used • To analysed group pile Spacing of 2d, 3d, 4d footing dimension of 2.5*2.5 m with fixed L and D is used • Thickness of reactive soil = 3 m	The analysis result confirms the potential of GAP system and indicate that the resistance to both heave and shrinkage can improve dramatically.
7	Sharma et.al., (2019)	Study to predict the uplift performance of GAP in expansive soil	Numerical method using FEM (PLAXIS 3D)	• Expansive soil was investigated • Mohr-coulomb model was used • Applied prescribed displacement of 10*D at the centre of GAP • To study the effect of L and D (L= 100, 200, 300 mm and D = 5, 6.67, 10, 13.33,	The uplift capacity of granular pile anchor increased with increase in length and diameter of pile for both single pile and piles arranged

		for single and group pile		15, 20, 30, 45, 60 cm) used for single pile and Spacing (2d, 2.5d, 3d) used for 2 pile group. • Expansive soil thickness = 0.6 m • Circular footing is used which equal to anchor plate.	in a group. Further, on increasing value of modulus of elasticity of soil, there is increase in pull-out resistance.
8	Bharati et.al., (2017)	To study the efficiency of single and group of Granular Anchor Pile in reducing heave of foundations	Experimental study (Laboratory model test)	• Expansive clay soil was investigated • Sample taken from an average depth of 1.5 m to 2 m. • Analyse the effect of L = 300 mm and varying the Diameter D = 40, 50, 60 mm and Spacing S = 2d and 2.5d	The swelling potential of an expansive soil bed decreases with increase in the surface area of the pile. Which means that the increase of the pile diameter results in greater reduction of swell potential and the heave decreases with the decrease in spacing between the piles.
9	Krishna et.al., (2013)	To compare performance of Granular Anchor Pile and conventional concrete pile on pull-out capacity	Experimental study (Laboratory model test and Field scale test)	• For laboratory test, piles with L = 200 mm and D = 500 mm with saturated and unsaturated condition is done. • For field test, piles with L = 1 and 1.5 m and D = 0.1 and 0.15 m is used. • Expansive clay soil was investigated	The pull-out resistance of the granular anchor piles is around 3 to 4 times that of similar concrete piles
10	Phanikumar, (2016)	To present the effect of number of geogrid	Experimental study	• Number of geogrids of 2, 3 and 4 with spacing of 20 and 30 mm and with bottom	The expansive clay bed reinforced with a plain GPA resulted in a

		layers, spacing between them and the location of bottom geogrid from the anchor plate on heave and pull-out behavior of GAP	(Laboratory model test)	geogrid of 20, 30 and 40 mm were conducted. • Thickness of expansive clay = 300 mm • GAP with L = 300 mm, D = 40 mm and Dr = 0.6. • Geogrid reinforcement of d = 40 mm	reduction in heave compared to that of the unreinforced expansive clay bed. Reinforcing the bottom portion of GPA with geogrids of n = 2, 3 and 4 reduced the heave significantly. Further, heave decreased with decreasing spacing between geogrids
11	Sivakumar et.al., (2012)	To study the load-displacement response, failure mode and ultimate pull-out capacity of granular anchor pile	Experimental study (Laboratory model test)	• To compare the ultimate pull-out capacity of GAP and concrete pile based on L = 0.5, 1 and 1.5 m and Anchor diameter of 0.07 m and 0.15 m on old fill deposit. • To investigate GAP capacity in lodgement fill deposit with L = 0.45-1.62 m and diameter of 0.15 and 0.2 m	An increase in anchor length and (or) diameter produced an increase in pull-out capacity.

The research paper discussed the capacity of Granular Anchor Pile in detail by assuming different pile spacing and feasibility of the pile to apply for low rising buildings, unlike the above-mentioned literature. Also, this research is specific to expansive soil found in Ethiopia specifically in Addis Ababa. So, it helps to solve problems associated with Expansive soil in the city for low rising buildings.

3. MATERIALS AND METHODS

3.1. Description of the study area

Addis Ababa is the capital city of Ethiopia. Addis Ababa lies at an elevation of 2,355 meters (7,726 ft) and is a grassland biome, located at 9°1′48″ N 38°44′24″ E. The city lies at the foot of Mount Entoto and forms part of the watershed for the Awash. From its lowest point, around Bole International Airport, at 2,326 meters (7,631 ft) above sea level in the southern periphery, Addis Ababa rises to over 3,000 meters (9,800 ft) in the Entoto Mountains to the north. Based on the 2007 Census conducted by the Central Statistical Agency of Ethiopia (CSA), the city has a population of 2,739,551 inhabitants. The city is divided into 10 boroughs, called sub-cities. The 10 sub-cities are Addis Ketema, Akaky Kaliti, Arada, Bole, Gulele, Kirkos, Kolfe Keranio, Lideta, NIfas Silk-Lafto, and Yeka sub-city.

The geological condition of Addis Ababa city is an assortment of different soil and rock type as shown in the figure below: -

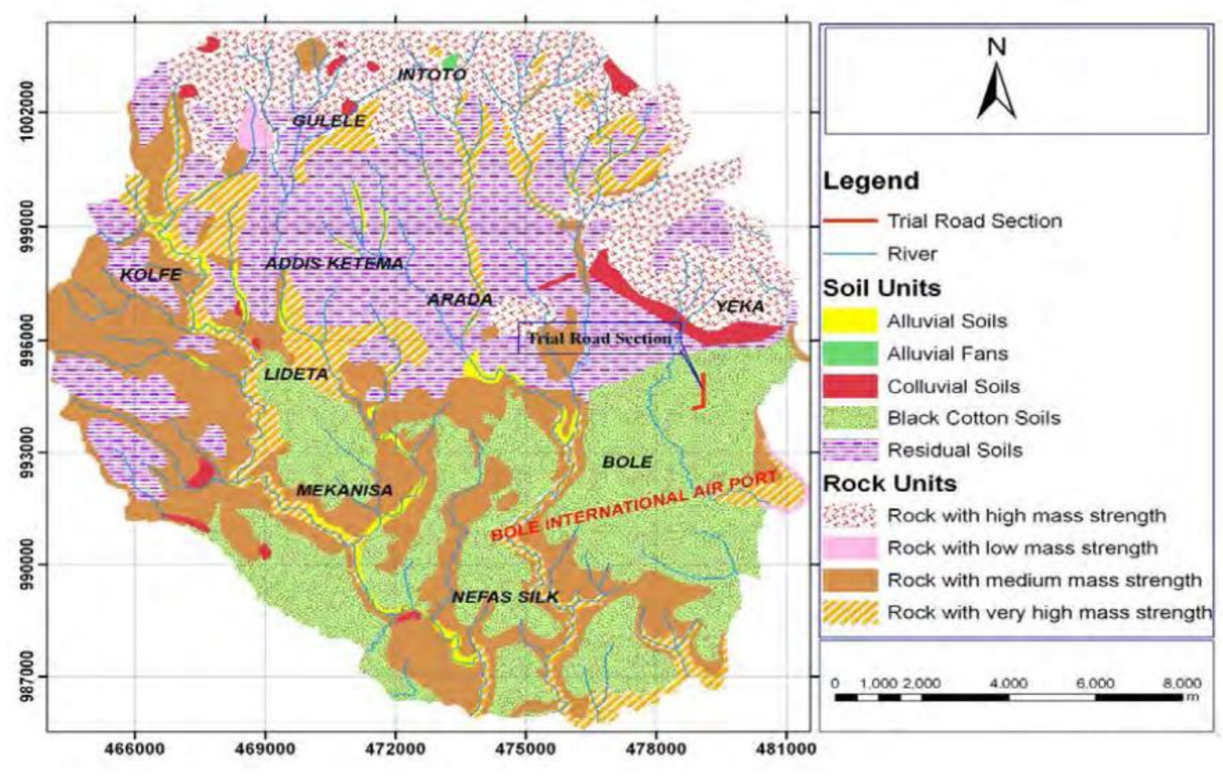


Figure 3.1. Engineering Geological Map of Addis Ababa (kebede et.al, 1990)

According to the above geological map the study area (i.e. Bole sub-city) is almost covered by Black Cotton Soil which is suspectable to high swelling and shrinkage effect.

3.1.1. Site Geology and Subsurface Condition

The selected site and its surroundings are covered with the made ground at the top underlain by silt CLAY and clayey silt/silty clay developed from complete weathering of underlying vesicular BASALT. A sub-surface investigation was done by Addis Geosystems Plc in 2017 for the construction 3B+G+19 mixed-use building in Addis Ababa Bole area has confirmed the presence of Made ground, Stiff to very stiff gray silty CLAY, Medium dense light gray clayey sandy SILT, Very stiff to hard light brown silty CLAY and Light gray to dark gray fine-grained moderately weathered medium-strong vesicular BASALT. The geological section showing the lateral and vertical extent of these layers is shown in Appendix A Figure A1.

3.1.2. Ground Water Observation

Measurement of groundwater level was made on the borehole. Drilling operation recorded that there was no groundwater within a depth of 30 m (Addis Geosystems Plc., 2017).

3.2. Methods used in the numerical analysis of the performance of the GAP system

In this paper, the performance of the GAP system can be evaluated by studying the effect of some parameters such as the diameter of the pile, length of the pile, and different pile spacing or pile group. The study is performed using FEM, PLAXIS 3D program. This program is the most efficient and fast tool for geotechnical modeling for model creation, realistic assessment of stress and displacement, and power full and versatile post-processing (Brinkgreve, et.al, 2013).

The constitutive models used in this numerical analysis is the Hardening soil model. This model is an advanced model for the simulation of soil behavior. Contrary to other models such as the Cap model or the Modified Cam Clay, the magnitude of soil deformations can be modeled more accurately by incorporating three input stiffness parameters as listed in the next paragraph (i.e. accounts for the stress-dependency of stiffness moduli) (Truty, A. & Obrzud, R. , 2011). This means that all stiffnesses increase with pressure. This model can be used to accurately predict displacement and failure for general types of soils in various geotechnical applications (Ti, et.al., 2009).

Hardening soil model requires a total of seven parameters, those parameters are stiffness parameters:- Reference secant stiffness from the drained triaxial test ($E^{ref}50$), Reference tangent stiffness for oedometer primary loading ($E^{ref}oed$), Reference unloading/reloading stiffness ($E^{ref}ur$), and strength parameters: - Internal Friction angle (ϕ), Cohesion (c) and Unloading/reloading Poisson's ratio (ν_{ur}) and Dilatancy angle (ψ).

3.3. Parameter Identification for Base Model

3.3.1. Granular fill parameters for the base model

Parameters for the base model of coarse material that is used as a fill-in Granular Anchor Pile system can be imitated from parameters that are commonly used by previous different investigations done on numerical simulation of Granular Anchor Pile. The reason is those literatures are used a coarse fill material parameter with high compression capacity. Table A1 in Appendix A shows a set of parameters for coarse fill material that are taken from different literatures and the most common one in the table is taken for the analysis.

3.3.2. Expansive soil parameters for the base model

Parameters for the base model of Expansive soil can be taken from the data gathered from Addis geosystems P.L.C. Four Sites data were selected based on the thickness of the Expansive clay layer. Such data are Atterberg Limit, Shear strength parameters, Swelling property, unit weight, and Specific Gravity. The summary of laboratory test data is given in Appendix A Table A2. From those four sites, the one which has soil with high swelling potential can be used for numerical simulation. Based on the Table A2 laboratory result, Site 2 clay soil has high free swell value when compared to the remaining 3 sites. This means that the soil shows high volume expansion when it is in contact with moisture. So, due to this Site 2 clay soil parameters are used for the numerical analysis because it is a critical soil type (Worst situation) than the others. The remaining parameters (i.e. strength parameters and stiffness Parameters) can be taken from correlation-based on SPT data of the corresponding site.

Kumar, et.al. (2016) suggested a different strong correlation between Poisson’s ratio and SPT N value for different soil types. Since the selected clay soil is stiff as shown in Figure A1 in Appendix A, so we can take the following equation to determine the Poisson’s ratio for the soil.

$$\upsilon = 0.125 + 0.0125N \ldots \text{for stiff clay (SPT N range from 6 to 30)} \qquad (3.1)$$

To determine Young's modulus for clay soil first we need to determine the undrained shear strength of the clay (S_u) from SPT N value result. Sanglerat,1972 suggested the following equation.

$Su = 12.5N$ (For Pure clay soil) (3.2)

$Su = 10N$ (For Silty clay soil) (3.3)

Since site 2 soil is more of silty clay, therefore equation 3 is a more appropriate equation to determine the undrained shear strength of the soil.

Once the undrained shear strength of the soil (S_u) is determined, Bowels 1996 suggested a correlation between S_u and Es for clay soil in general as follow: -

$Es = KSu$ (3.4)

Where: - $K = 4200 - 142.54PI + 1.73PI^2 - 0.0071PI^3$

PI – Plasticity Index (use 20% $\leq$ PI $\leq$ 100%)

Therefore, Young's modulus, poison's ratio, and undrained shear strength for silty clay soil of site 2 soil can be summarized in Table A3 in Appendix A.

3.3.3. Stiffness parameters for Hardening soil model

In hardening soil model the basic parameters for soil stiffness are E^{ref}50: Secant stiffness in the standard drained triaxial test (kN/m^2), E^{ref}oed: Tangent stiffness for primary oedometer loading (kN/m^2), E^{ref}ur : Unloading/reloading stiffness (kN/m^2), Unloading-reloading Poisson's ratio ($\nu_{ur)}$ and Dilatancy Angle (Ψ). These all stiffness parameters for Granular fill material as well as for the Expansive soil layer are determined and summarized in Appendix B.

3.3.4. Parameters for Anchor Plate, Footing plate and Anchor road

For Anchor plate, Anchor rode, and footing the mild steel material with a high flexural rigidity was used for avoiding buckling and deformations. According to the metal supermarket guideline, A36 steel is used. This steel is low carbon steel, this allows the steel to be easily machined, welded, and formed, making it extremely useful as general-purpose steel (Grade Guide: A36 steel, 2017). The mechanical properties of this steel are summarized in Appendix A Table A4.

3.4. Summary of Parameters for the Base model

The following parameters are used as input for base model generation during the whole analysis.

Table 3.1.: - Expansive soil parameters for the base model

Depth (m)	Soil Consistency	Es (MPa)	$E^{ref}50$ (MPa)	$E^{ref}ur$ (MPa)	$E^{ref}oed$ (MPa)	ν_{ur}	Su (kN/m^2)	$\gamma_{unsat.}$ (kN/m^3)	γ_{sat} (kN/m^3)
1	Stiff	42.25	38.06	152.14	38.06	0.2	100	17.68	18.73
2									
3									
4									
5	Very Stiff	54.92	49.48	197.92	49.48	0.2	130	17.68	18.73
6									
7	Very Stiff	63.37	57.09	228.36	57.09	0.2	150	17.68	18.73
8									
9	Very stiff	84.49	76.12	304.48	76.12	0.2	200	17.68	18.73
10									
11	Extremely Stiff	101.39	91.34	356.36	91.34	0.2	240	17.68	18.73
12									
12.5									

Table 3.2.: - Granular fill parameter for the base model

Granular Fill Material	Parameters									
	Es (MPa)	$E^{ref}50$ (MPa)	$E^{ref}ur$ (MPa)	$E^{ref}oed$ (MPa)	ν_{ur}	C (kN/m^2)	Φ (deg)	Ψ (deg)	$\gamma_{unsat.}$ (kN/m^3)	γ_{sat} (kN/m^3)
	240	133	399	133	0.2	0	36	6	22	24

Table 3.3.: - Anchor Plate, Footing Plate and Anchor rod parameters for the base model

Parameters	Anchor Plate	Footing	Anchor rod
E(GPa)	200	200	200
ν	0.26	0.26	0.26
Thickness (t)	0.025	0.06	-
Diameter (mm)	-	-	30
EA (kN)	-	-	$140*10^3$

3.5. Sensitivity analysis

Sensitivity analysis is used to define or fix the soil boundaries horizontally and vertically. This is can be computed by considering one point or node in the Finite element model for different dimensions of the model. When the model dimension becomes increased, after some point the stress effect on the selected node becomes constant. Therefore, the model dimension that the effect becomes constant considered as the best suit for numerical analysis. In this paper, the soil model of dimension from 4*4 m to 10*10 m for single GAP with diameter 0.3 m and length 6 m were checked. The plan dimension of the footing pad used is 2*2 m. As shown in the figure below almost 8*8 m and 10*10 m bring the same result. Therefore, the soil plan dimension of 10*10 m is used for the whole analysis. The vertical dimension (depth) of the soil is used as 10 m because the clay soil is existing up to a depth of 10 m in the selected site.

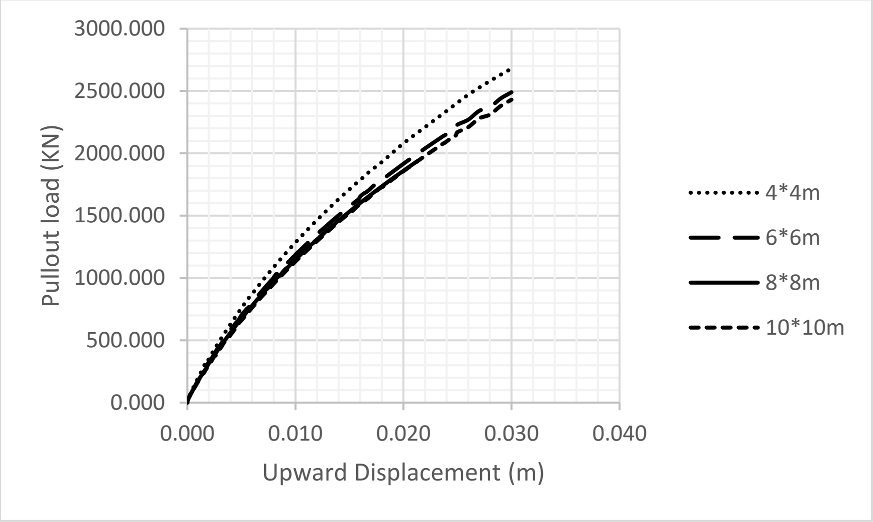
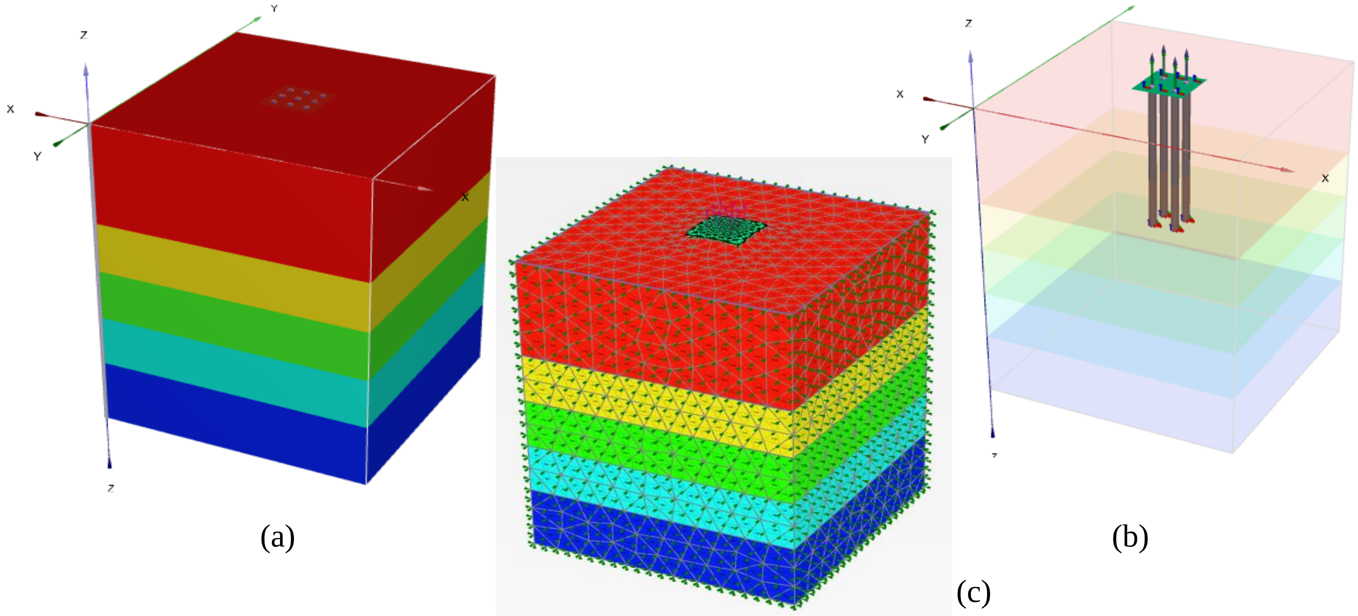


Figure 3.2. Sensitivity analysis curve

3.6. Description of Numerical Modelling

A model of Granular Anchor Pile of various length L = 1 m, 3 m, 4 m and 6 m, diameters d = 0.2 m, 0.3 m, 0.4 m, and 0.6 m and spacing s = 1.5d, 2d and 3d is modeled in PLAXIS 3D software for the analysis of the uplift capacity of the pile. The soil surrounding the GPA is considered as expansive soil with different five soil consistency which extends to a depth of 10 m underneath the ground surface. The soil layer is modeled using borehole option in PLAXIS 3D by taking suitable plan dimensions. The plan dimensions of the soil layer are taken as 10*10 m (from sensitivity analysis) and depth is kept as 10 m. The single granular pile is modeled in PLAXIS 3D software by poly-curve and extrude options. The modeling of the pile group is done by using an array option in structural mode. The modeling of the footing and anchor plate is done with the plate elements. To model the anchor rod of the GPA system and to tie the anchor plate, footing plate, and GPA together, a node-to-node element is used. For plate elements and node-to-node anchor, a linear elastic model is adopted. Hardening soil model is applied for defining the relationship between the granular pile and clay. The modeling of volume elements is done using ten-node tetrahedral elements, and the plate elements are modeled using six-nodded triangular elements. The boundary condition used in the analysis is fixed in both X and Y direction as well as the bottom of the model and prescribed at the top. To analyze the group effect of GPA, three different spacing values 1.5d, 2d, and 3d for different diameter and length are analyzed as shown in Appendix A Figure A1, A2, and A3. To analyze the uplift behavior of the GPA system, the results are plotted graphically taking upward movement (mm) on X-axis and the corresponding pullout load (kN) on Y-axis. Upward displacement of 10 % of the pile diameter of GPA is applied in each model and the corresponding uplift is calculated. The material properties of soil, GPA, and structural elements used in numerical modeling are given in Tables 3.1., 3.2. and 3.3. respectively.



(a)

(b)

(c)

Figure 3.3. (a) Soil model, (b) GAP, plate and Point displacement model, (c) Meshing and fixity of the model

3.7. Interface modelling

The numerical modeling of the Granular Anchor Pile system requires the use of interface boundaries between the dissimilar materials to simulate the discontinuity and transfer of normal and shear stresses from the soil to the facing component. The interface is defined as a layer connected with soil and structure elements as shown in Figure 3.4. The interfaces in PLAXIS are generally modeled using zero-thickness elements. The strength and stiffness of the interface are defined as part of the strength and stiffness of soil adjusted to interface strength. These interfaces have properties of friction angle (ϕ_i), cohesion (c_i), dilatancy angle (ψ_i), shear modulus (G_i), Poisson's ratio (with a fixed value of v_i = 0.45), and oedometer modulus ($E_{oed,i}$). The values of interface properties in PLAXIS can be set directly by using a strength/stiffness reduction factor ($Ri \leq 1.0$). The default value is R_i = 1.0 represents a fully bonded interface. This factor is applied to the properties of the adjacent soil as follows:

$$c_i = R_i * c_{soil} \tag{3.5}$$

$$\phi_i = \tan^{-1}(R_i * \tan\phi_{soil}) \tag{3.6}$$

$$\psi_i = \begin{cases} 0 & R_i < 1 \\ \psi_{soil} & R_i = 1 \end{cases} \tag{3.7}$$

$$G_i = R_i^2 * G_{soil} \tag{3.8}$$

$$E_{oed,i} = 2Gi * \frac{1-\nu i}{1-2\nu i} \tag{3.9}$$

Where: - c_{soil} is the soil cohesion, ϕ_{soil} is the soil friction angle, ψ_{soil} is the soil dilatancy angle, G_{soil} is the soil shear modulus.

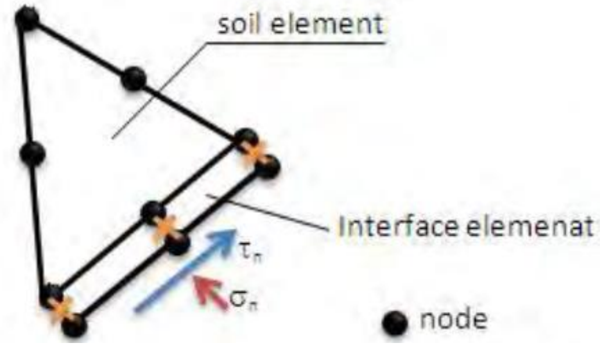


Figure 3.4. The geometry of interface element

In this study, PLAXIS 3D modeling contains two interface zones. These are between the GAP shaft and the surrounding soil and isolated footing bottom with the underlying soil. Therefore, Strength reduction factor for GAP shaft and Soil and Footing pad and underlying soil were taken as 0.9 and 0.7 respectively.

3.8. Methods used to check the suitability of the GAP system as a dependable solution for problems in expansive soil for low rising building

In Ethiopia here to fore many geotechnical researches are performed. Most of them investigate actual geotechnical problems that are an impediment to the construction industry in the country and many of them put a direction for solutions to the problems. But those researches cannot be clearly clarifying how much the solutions are best suitable for the problem in terms of its applicability for the mention problems based on the country's capacity like technological advancement, economy, number of skilled manpower and so on. However, knowing these things really help to decide how much the researches are realistic and practical for actual problems.

In this study, once the performance of the pile is identified then Suitability analysis of the GAP system was checked as much as possible based on different factors that are stated below to know how much the pile is practical for the intended purpose.

- **The economy of construction**: - the economy has a crucial role in the applicability of the GAP system for all types of low rising building types such as a three-story, two-story, or one-story building which are constructed in Expansive soil bed. Because the pile system must be economically better and safe than the other remedial measures previously adopted in the country and must be affordable to use for the different economic levels of societies. So, at this point, a cost comparison is done between a single GAP system and Simple shallow isolated footing which are design for the G+1 building constructed on Expansive soil. This can be achieved by quietly estimating the cost of construction material, machinery cost, and labour cost based on the current available price. Finally, the cost and performance of both the foundation system are compared to each other and the corresponding advantage of the GAP system also suggested.
- **Local availability of construction material**: - Since Granular Anchor Pile (GAP) uses Granular fill material (Coarse aggregate) and Mild steel anchor plate and anchor rod as a construction material, therefore, in this point this research investigates the local availability of those materials in an efficient amount especially for the studied area by checking the

number of factories or production units that produces and supplies the above-mentioned construction materials in the vicinity and their production and supply capacities.

- **Installation process condition**: - At this point, this research assesses the difficulty and easiness of installation of the Granular Anchor pile for the proposed constructed building such installation procedure is digging of ground for installation, installation of Anchor plate and Anchor rod and filling of granular material and compaction method.
- **Environmental benefit**: - now a day construction sector adversely affects the environment by making air pollution, climatic change, drinking water pollution, and landfill waste. In this section, the research assessed how much Granular Anchor Pile (GAP) is environmentally friendly which means that checking the environmental benefits of GAP in terms of environmental protection from pollution.
- **Construction time**: - time management is the major issue for the construction sector to finish the construction at the given schedule. Proper use of time management techniques can result in the completion of the project on time and create a positive testimonial for the contractor. Therefore, at this point, it is assessed that what is the contribution of the Granular Anchor Pile (GAP) system in minimizing the time of construction.
- **Performance of the pile**: - since the pile has high performance, therefore it requires less material and energy consumption to resist or withstand the relatively high swelling potential of a soil. So, in this section, it is assessed that how much is the performance of GAP relative to the extent of the problem.

4. RESULT AND DISCUSSION

4.1. Introduction

In this chapter analysis and results are reported to draw the inferences of the study. The analyses of data are presented in tabular form and figures or pictorial presentation. In the discussion part, the results are interpreted in detail. In this study, two major points are investigated which are the parametric study of different parameters of the Granular Anchor Pile (GAP) system and the Suitability analysis of GAP for the actual problem of the country based on different factors. So, in this section different results from both analyses are stated and discussed briefly. In addition to this validation of the Finite element program is also done.

4.2. Validation of Finite Element program (PLAXIS 3D)

Laboratory testing on the performance of the GAP system can give a better outcome than numerical study but some difficulties incapacitate not to do laboratory modeling of GAP. Those difficulties are: -

- The test is difficult to conduct for different parameters such as length, diameter, spacing, and so on for heaps of trials. Because it is hard to set up the equipment repeatedly for a different trial.
- To simulate the heave, loading hydraulic jack with specifically designed and fabricated attachment is required,
- The uplift movement is recorded through dial gauges fixed at the top of the pile. So, for this purpose at least a calibrated dial gauges are necessary.
- To record the data, it needs continuous follow up for a longer period of time at least 40hrs for a single test. this makes too difficult to record exhaustive result.

Because of this reason doing laboratory tests are an onerous thing for this study. Therefore, in this study the finite element analysis results cannot be validated with laboratory measurements, instead, the validation is undertaken on PLAXIS 3D program whether it represents the actual condition by simulating other researcher model data which investigate both on laboratory and numerical analysis using PLAXIS 3D and confirm whether the present PLAXIS 3D output is equivalent with researcher’s output.

Kranthikumar, et.al (2016) present the behavior of a GAP system in loose sandy soil. The small-scale laboratory tests were conducted on loose dry sand to investigate the effect of the embedded length of the pile on the uplift capacity of the GAP system. The results were compared with the numerical analysis using three-dimensional finite-element analysis software. The size of the soil testbed used in the analysis was 1.25*1.25*1.25 m. The dry unit weights and saturated unit weight of the pile and soil were 21 and 24 and 17 and 19 kN/m^3, respectively. The diameter of the anchor plate was the same as the diameter of the pile, and the thickness was 0.0254 m. The length of the anchor rode was the same as the length of the pile with a diameter of 0.0125 m. For both the anchor rod and anchor plate, the MS material with a high flexural rigidity was used to avoid buckling and deformations (E = 200 GPa and ν = 0.15). Soil modulus was computed as 2 MPa and the pile modulus as 11 MPa with a poison's ratio of 0.3 for both materials. Then finally the uplift forces obtained from the laboratory and the numerical analyses for a given upward movement of 0.025 m were 1.48 and 1.6 kN for a 0.75-m-length pile, and 2.6 and 2.5 kN for a 1.0-m-length a pile for upward movement of 0.032 m as shown in Figure 4.1. The result informed that a close agreement is observed between the experimental and numerical results. Therefore, the PLAXIS 3D program is validated using the above model parameter to check whether the program gives a result near to the researcher's result. The result is plotted in Figure 4.2 for 1m and 0.75 m of pile length. According to the analysis, the results are more likely the same as the result of Kranthikumar, et.al. So, PLAXIS 3D has infallibilities for analyzing the parametric study of this research and the result are the best representative of the actual condition.

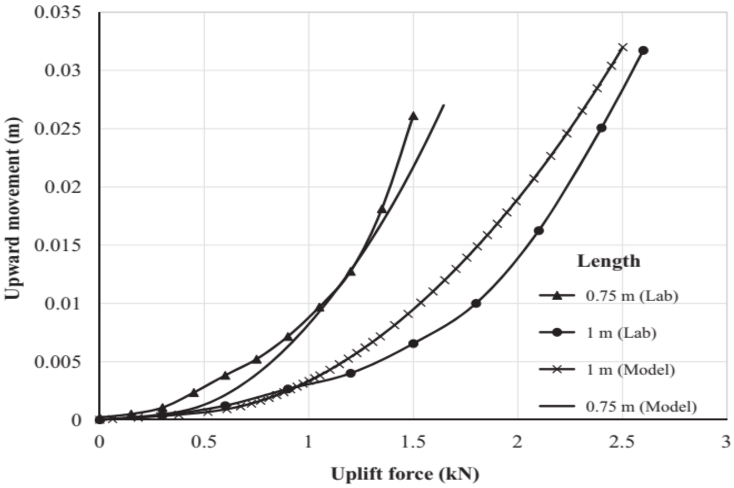


Figure 4.1. Comparison of uplift force between laboratory investigation and numerical analysis (Kranthikumar, et.al., 2016)

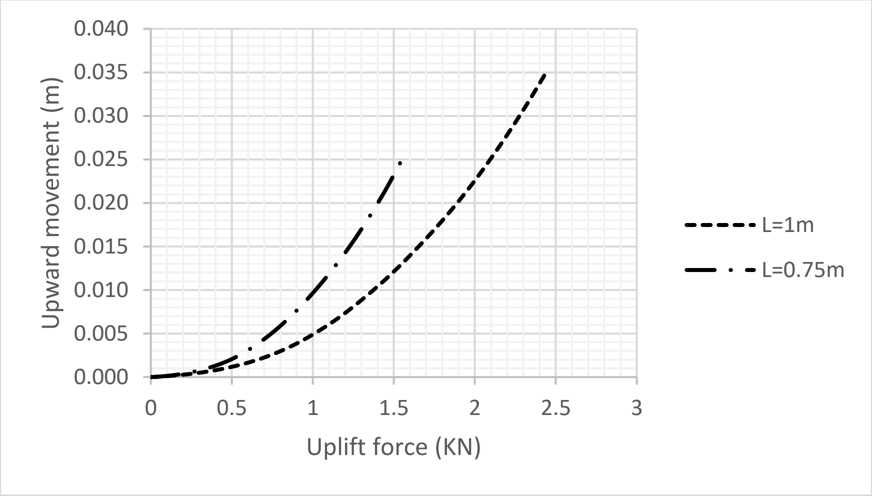


Figure 4.2. The current result of PLAXIS 3D

4.3. Analysis and Parametric Study

There are several parameters to be considered if one is interested in conducting a comprehensive parametric study. However, some of the variables have little effect in the performance of Granular Anchor Pile (GAP), and running of several parameters takes longer calculation time and needs a good computer processer when using advanced running programs like PLAXIS 3D. In this paper, variables that have practical importance and reported to have a significant influence in the performance of the GAP system were discussed. In this section, 48 runs are analyzed using PLAXIS 3D software and a representative parametric study is presented. The parametric study conducted included a number of alternative arrangements of variables under consideration. This includes the effect of GAP length, GAP Diameter, and Different Pile Spacing, Efficiency of pile group. The results from all parametric studies are shown in Appendix C. When one of the above parameters is varied, the rest are kept constant. Accordingly, the effect of varying a parameter for Expansive soil of dimension 10*10*10 m with their construction phases is manifested by obtaining the variation in the quantities of upward displacement with corresponding pullout load.

4.3.1. Effect of Pile Diameter on the uplift capacity of the GAP system

The diameter of the granular pile has an important bearing on the performance of the GAP system to resist uplift forces coming from the soil because it is related to the load-transfer mechanism of the GAP system. A parametric study was carried out to study the effectiveness of the diameter of the pile on the uplift capacity of the GAP system. GAPs of different diameters (0.2, 0.3, 0.4, and

0.6 m) for a constant GAP length and pile spacing were modeled to determine the behavior of the GAP system. Generally, three different cases are inspected under this condition.

Case 1: For a GAP system, the uplift capacity for a 0.02 m upward displacement is observed as 706 kN for 0.2 m diameter GPA, 690 kN for 0.3 m diameter GPA, 633 kN for 0.4 m and 460 kN for 0.6 m diameter GPA, for pile length of 1 m and Pile Spacing s=1.5d as shown in Table 4.1.

Table 4.1. The uplift capacity of a pile for different pile diameter

Pile spacing and pile length	Uplift capacity (kN)			
	d=0.2 m	d=0.3 m	d=0.4 m	d=0.6 m
s=1.5d & L= 1 m	706	690	633	460
s=1.5d & L= 3 m	1442	1377	1126	818
s=1.5d & L= 4 m	2351	2243	2211	1412
s=1.5d & L= 6 m	3328	3269	2783	1933

The percentage decrease in uplift capacity of GPA for length 1m and s=1.5d is approximately 2.32 % when the diameter of GPA is increased from 0.2m to 0.3m, 9 % for a diameter of GPA increased from 0.3 m to 0.4 m and 37.6 % for a diameter of GPA increased from 0.4 m to 0.6 m. A similar trend is observed for a pile length of 3, 4, and 6 m with spacing s=1.5d and different diameter values as shown in Figures. 4.3., 4.4., 4.5. and 4.6. as well as mentioned in Table 4. 1.

Hence, the result shows that, with the increasing diameter of a pile, the uplift capacity of the GAP system decrease. This happens when the diameter of the pile increase, the spacing between the pile group will be increased resulting in decreasing in pile number in the group. Therefore, the total self-weight of the piles and total surface area of piles in the group which serves a frictional resistance to the pile will also decrease, this causes declination in pile capacity to resist the upcoming pullout load.

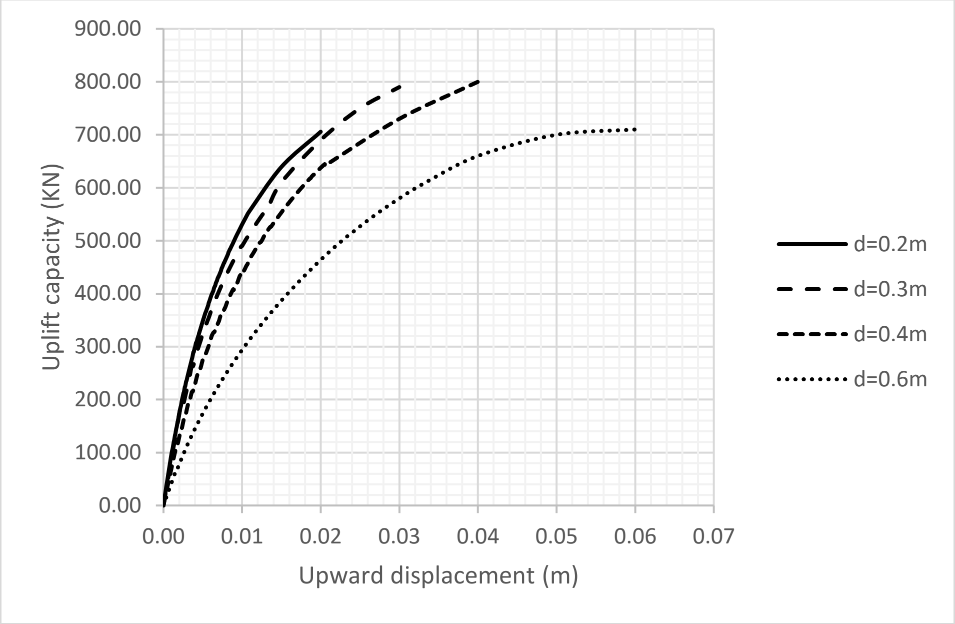


Figure 4.3. Upward movement versus uplift force of GAP for L = 1 m and s=1.5d

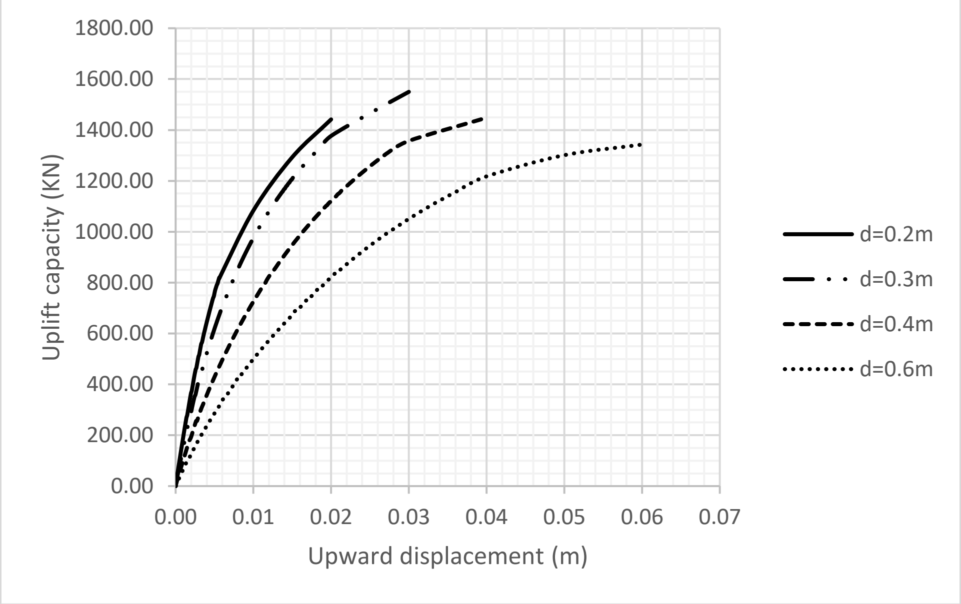


Figure 4.4. Upward movement versus uplift force of GAP for L = 3 m and s=1.5d

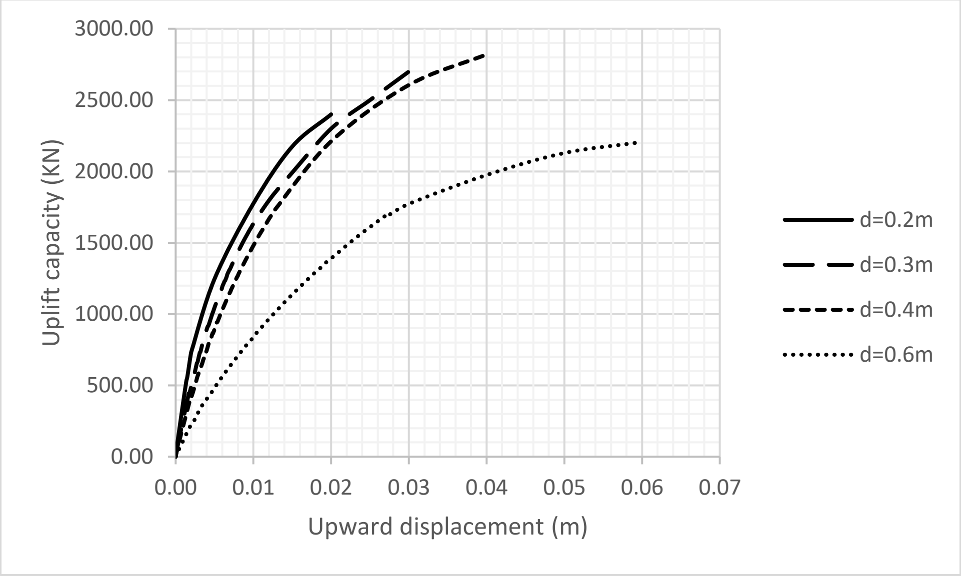


Figure 4.5. Upward movement versus uplift force of GAP for L = 4 m and s=1.5d

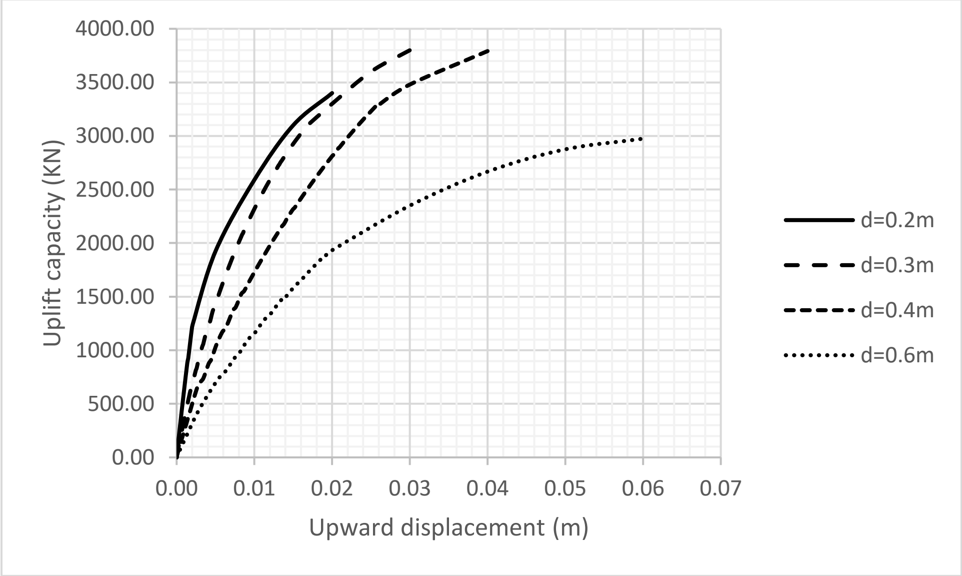


Figure 4.6. Upward movement versus uplift force of GAP for L = 6 m and s=1.5d

Case 2: For spacing s=2d and pile length of L= 1, 3, 4, and 6 m the uplift capacity of a pile for d=0.3 m is slightly lesser than the capacity of d=0.4 m as shown in Figure 4.7. and 4.8. The reason is that the number of pile in the group for both diameters (i.e. d=0.3 m and 0.4 m) is equal (or four) as shown in Appendix A Figure A2 but the total self-weight and surface area of group pile are greater in the case of d=0.4 m than d=0.3 m due to increment in pile diameter. This makes the uplift capacity of the pile greater in the case of d=0.4 m than d=0.3 m.

Case 3: In the case of s=3d and L= 1, 3, 4, and 6 m the uplift capacity of a pile for d=0.4 m is somewhat lesser than piles of d=0.6 m as shown in Figure 4.9. and 4.10. The reason is that the number of piles in a group in both cases (i.e. d=0.4 m and 0.6 m) is the same or single pile as shown in Appendix A Figure A3 but total self-weight and surface area of the pile is greater in case of d=0.6 m than d=0.4 m because of increment of pile diameter. Therefore, the uplift capacity of a pile with d=0.6 m is greater than that of d=0.4 m. The rest figures are put in Appendix C Figure C1, C2, C3, and C4.

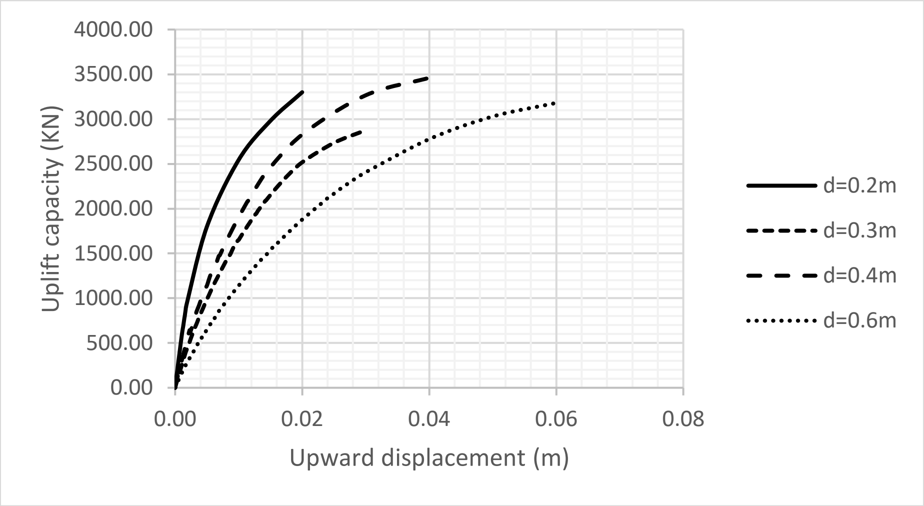


Figure 4.7. Upward movement versus uplift force of GAP for L = 6 m and s=2d

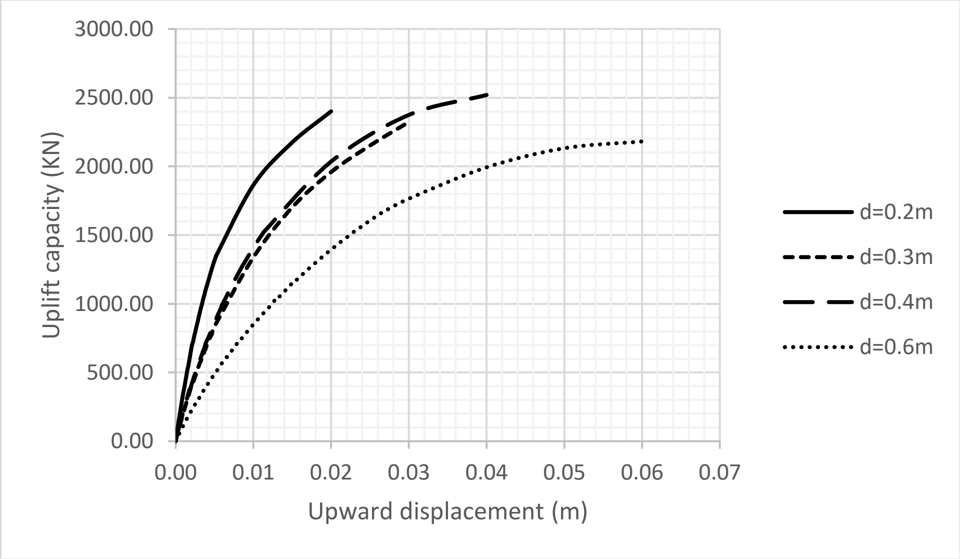


Figure 4.8. Upward movement versus uplift force of GAP for L = 4 m and s=2d

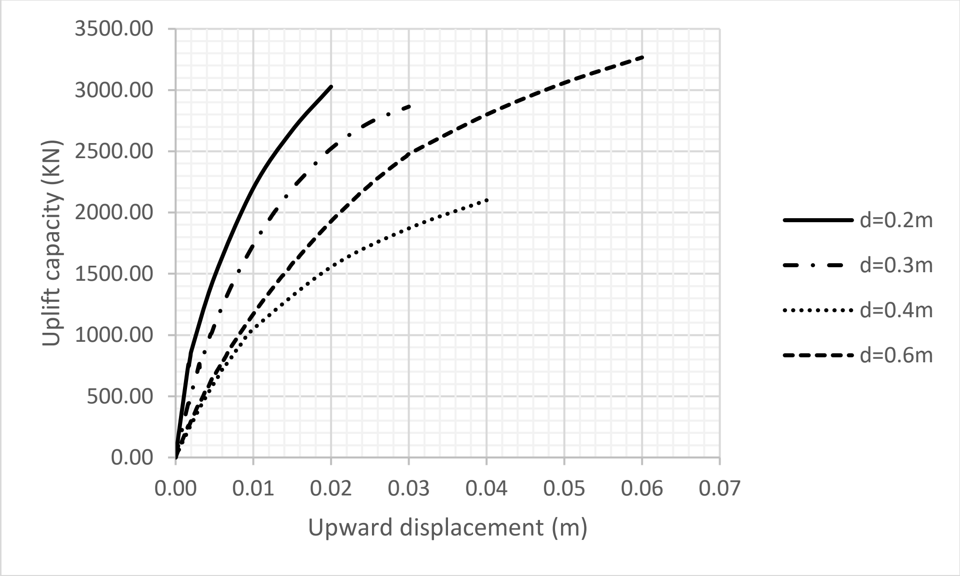


Figure 4.9. Upward movement versus uplift force of GAP for L = 6 m and s=3d

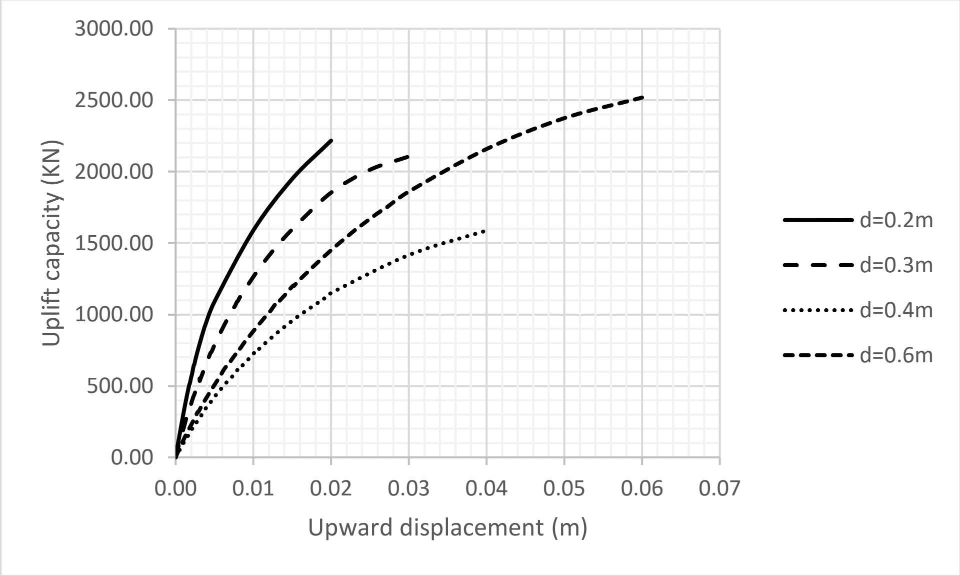


Figure 4.10. Upward movement versus uplift force of GAP for L = 4 m and s=3d

4.3.2. Effect of Pile Length on the uplift capacity of the GAP system

A parametric study was carried out to examine the uplift behavior of the GAP system for different pile lengths with constant pile spacing and diameter as shown in the sample Figure 4.11., 4.12. and 4.13. Accordingly, the figures show that with an increase in the length of a pile, the uplift capacity of a pile also increases for given spacing and diameter. The reason is if the length the pile increase, there were an increase in friction mobilized between the pile-soil interface and also increment of self-weight of the GAP system due to the lengthening of the piles in the group. This condition makes the pile's uplift capacity to be increased. The remaining figures are attached in Appendix C Figure C5 – C13.

Ultimate uplift force corresponding to the prescribed upward displacement of 10 % of pile diameter (d) for different lengths, spacing, and diameters of a GAP system is summarized in Table 4.2. The uplift force of the GAP system was increased with an increase in the length of the pile for constant diameter and spacing as mentioned in the above paragraph. But also, Table 4.2. shows that the percentage increase in the uplift capacity of the GAP system. As an example, the percentage increase in the uplift capacity is approximately 89, 64, and 35 % for an increase in length from 1 to 3 m, 3 to 4 m, and 4 to 6 m respectively, for d=0.6 m and s=3d. Also 95, 55, and 48 % for an increase in length from 1 to 3 m, 3 to 4 m, and 4 to 6 m respectively, for d=0.3 m and s=3d. This shows that the uplift capacity of a pile improves significantly with an increase in the length of a pile up to a certain length, after which the increase in the length does not contribute much toward the load sharing. Since the GAP system resists the uplift force by the weight of the granular pile acting in the downward direction and the friction mobilized along with the pile-soil interface. In clay soil, the skin friction is developed due to adhesion between the pile material and clay. But generally accepted that adhesion does not increase with depth (Rajapakse, 2016). Therefore, there is a somewhat reduction in uplift capacity due to the reduction of skin resistance in the pile surface.

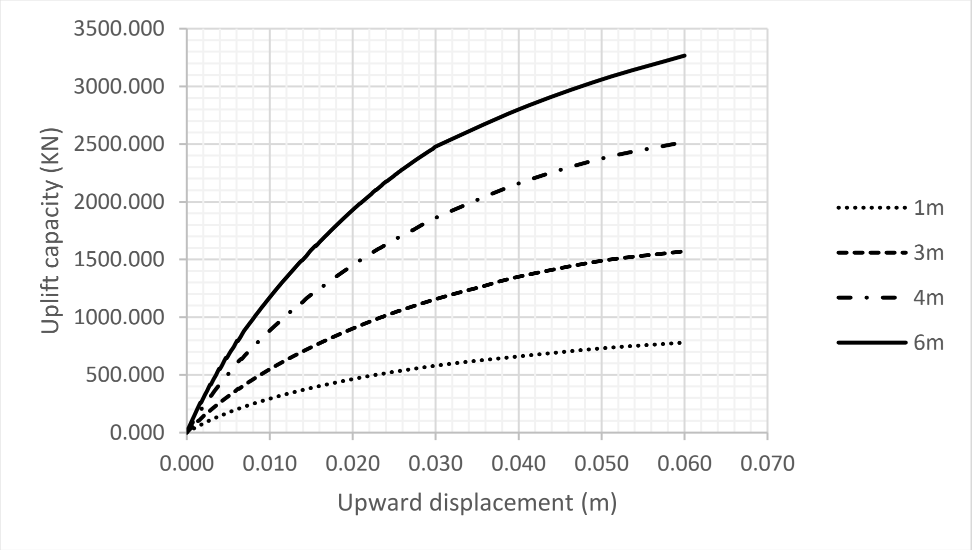


Figure 4.11. Upward movement versus uplift force of GAP for s=3d, d=0.6 m

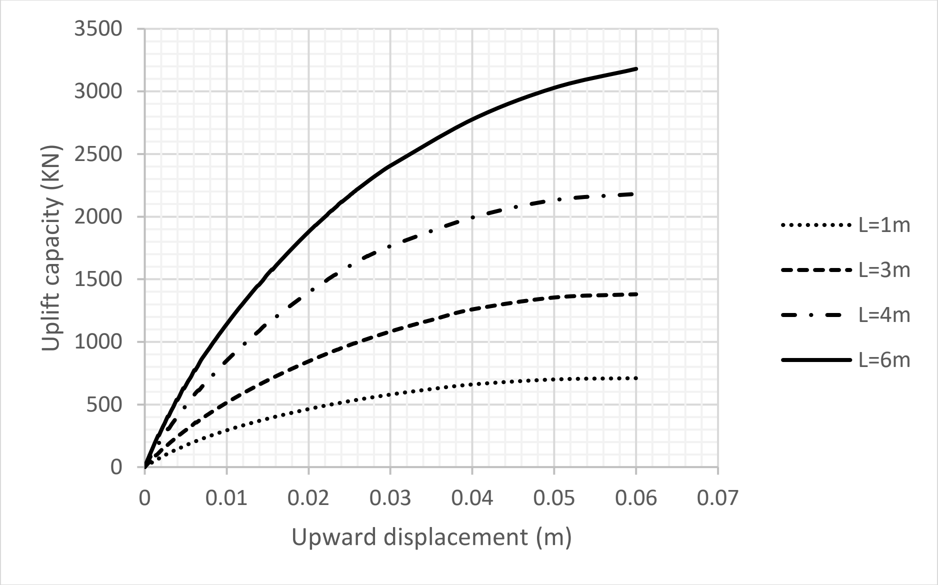


Figure 4.12. Upward movement versus uplift force of GAP for s=2d, d=0.6 m

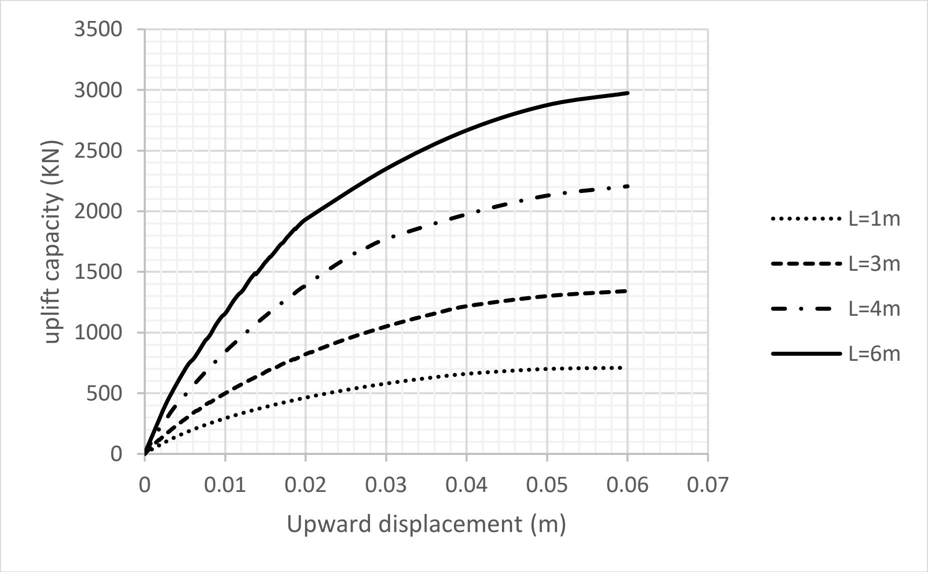


Figure 4.13. Upward movement versus uplift force of GAP for s=1.5d, d=0.6 m

Table 4.2. Uplift capacity of GAP for different pile length

Diameter (m)	Length (m)	L/D	Ultimate uplift capacity (kN)					
			S=1.5d	%age increase	S=2d	%age increase	S=3d	%age increase
0.6	1	1.67	710		710		780	
	3	5	1343	89	1380	94	1571	101
	4	6.67	2206	64	2181	58	2518	60
	6	10	2974	35	3180	46	3266	30
0.4	1	2.5	800		700		510	
	3	7.5	1883	135	1732	147	979	92
	4	10	2820	50	2519	45	1588	62
	6	15	4200	49	3461	37	2100	32
0.3	1	3.33	790		700		600	
	3	10	1540	95	1505	115	1356	126
	4	13.33	2382	55	2323	54	2106	55
	6	20	3536	48	3137	35	2865	36
0.2	1	5	705		775		620	
	3	15	1441	104	1475	90	1320	113
	4	20	2351	63	2400	63	2217	68
	6	30	3212	37	3300	37	3028	37

4.3.3. Effect of Group of Granular Anchor Piles

A finite element analysis is carried out on a group of GAP's with constant pile length and diameter varying spacing as 1.5D, 2D, and 3D, and corresponding uplift capacities are predicted. Generally, 4 different cases are examined for the effect of a group of GAP system in resisting uplift forces.

Case 1: According to the analysis result it is observed that when the spacing between the pile increases from 1.5d to 2d and 2d to 3d, the uplift capacity of group pile was decrease for diameter d=0.3 m and length L=1, 3, 4 and 6 m as shown in the sample Figure 4.14. and 4.15. The reason is that when the pile spacing between the pile increases the soil confinement between two pile will decrease result in the reduction of friction mobilized between soil and pile. In addition to this when the pile spacing increased the number of piles in the group was decreased which result in a reduction in the capacity of the pile group in general. Because of these two reasons, the uplift capacity of the group pile will be decreased when increasing the pile spacing. The remaining figures are attached in Appendix C Figures C16 and C17.

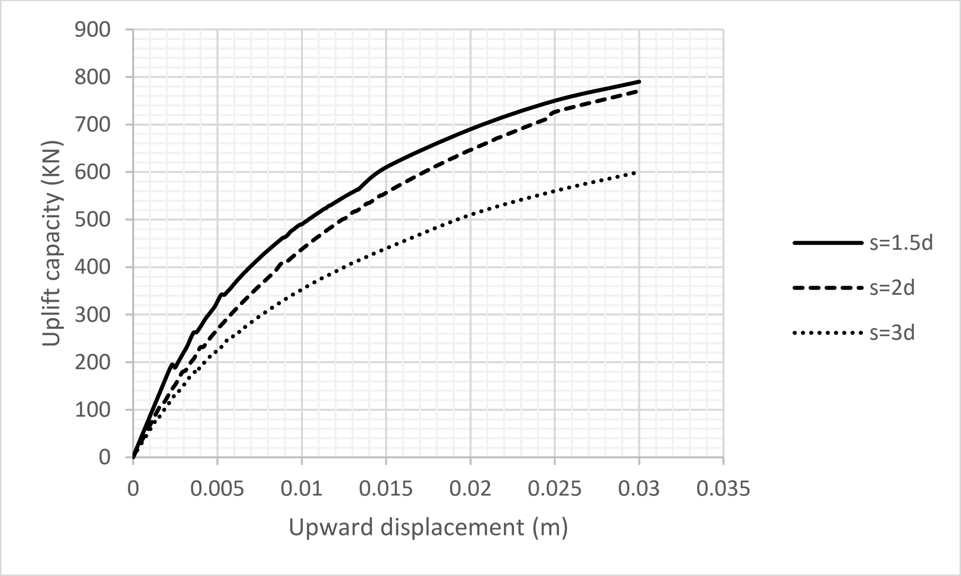


Figure 4.14. Upward movement versus uplift force of GAP for d=0.3 m, L=1 m

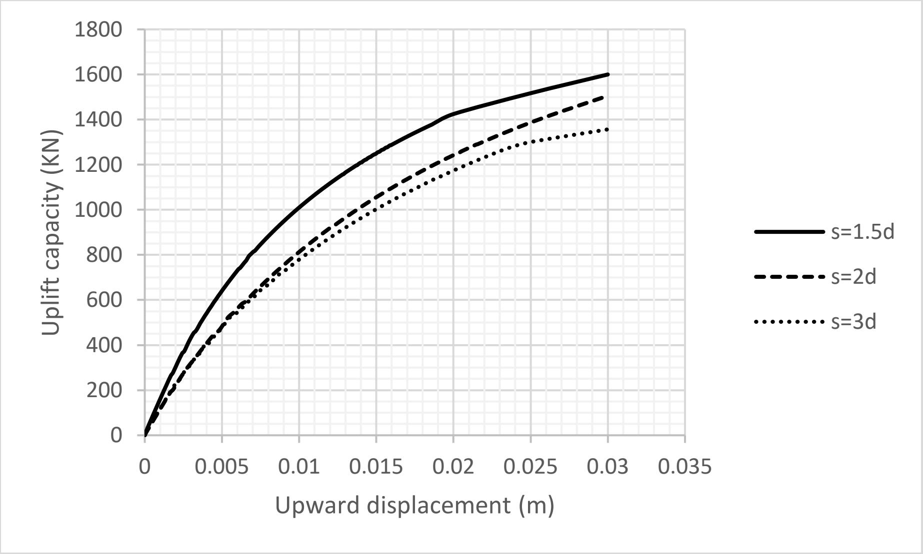


Figure 4.15. Upward movement versus uplift force of GAP for d=0.3 m, L=3 m

Case 2: For the case of diameter d=0.2 m and length L=1, 3, 4, and 6 m the uplift capacity of the pile group for s=2d is greater than that of s=1.5d for a constant number of the pile in the group (i.e. 16 piles) as shown in the Figure 4.16. and 4.17. This is happened due to the soil stresses caused by the piles tend to overlap between piles in the case of s=1.5d is higher than s=2d because piles are too close to each other. This condition causes the soil is highly stressed in the zones of overlapping of pressures which tends the soil to fail which results in a reduction of the uplift capacity of the pile in the group due to decrement of frictional resistance. The remaining Figures are attached in Appendix C, Figure C18 and C19

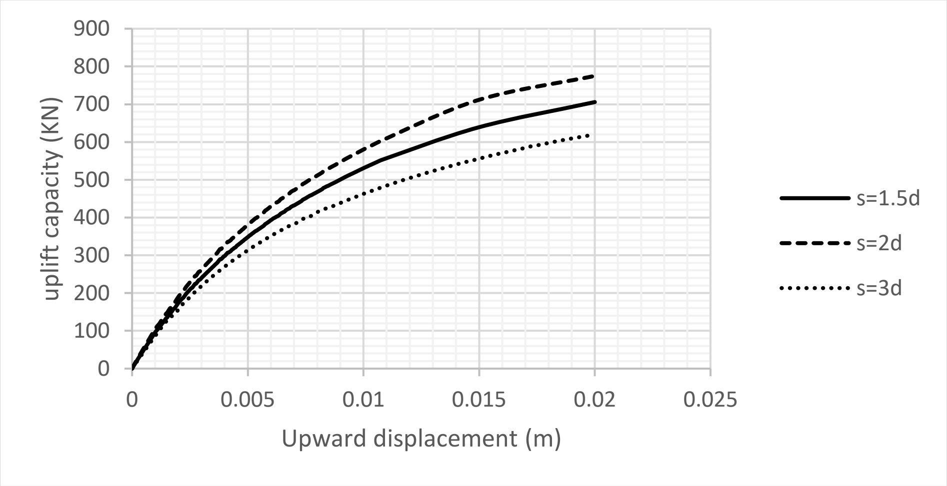


Figure 4.16. Upward movement versus uplift force of GAP for d=0.2 m, L=1 m

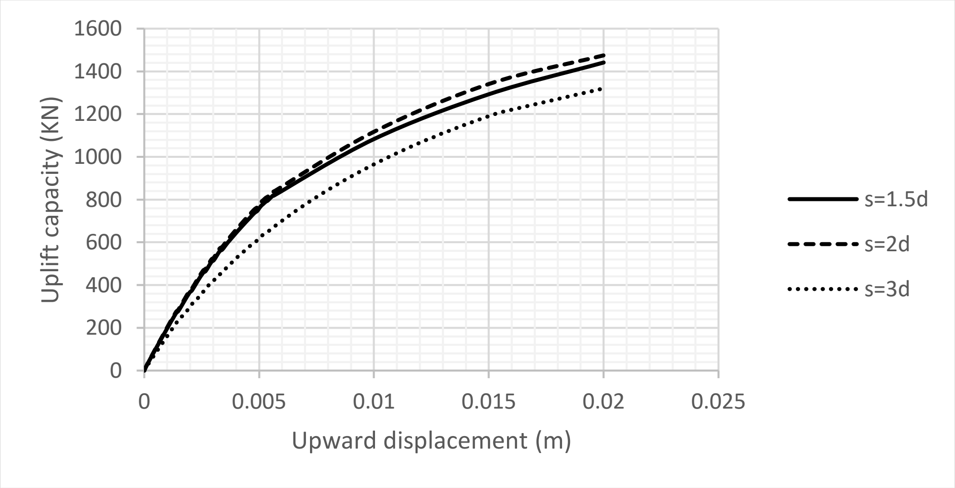


Figure 4.17. Upward movement versus uplift force of GAP for d=0.2 m, L=3 m

Case 3: The percentage increase in the uplift capacity is 37.25 and 11.43 % for the decrease in spacing from 3d to 2d and 2d to 1.5d respectively for d=0.4 m and L=1 m, the percentage increase in the uplift capacity is 76.9 and 8.66 % for the decrease in spacing from 3d to 2d and 2d to 1.5d respectively for d=0.4 m and L=3 m, the percentage increase in the uplift capacity is 58.63 and 11.95 % for the decrease in spacing from 3d to 2d and 2d to 1.5d respectively for d=0.4 m and L=4 m and percentage increase in the uplift capacity is 64.76 and 27.2 % for the decrease in spacing from 3d to 2d and 2d to 1.5d respectively for d=0.4 m and L=6 m as shown in the Figures 4.18., 4.19., 4.20. and 4.21. The result shows that the percentage increase in pile uplift capacity from 3d to 2d is much greater than from 2d to 1.5d the reason is that the number of piles in the group for the case of s=1.5d and 2d and diameter of d=0.4 m is four but for the case of s=3d and d=0.4 m the number of piles is one or single pile. Thus, the difference in the number of piles causes the capacity gap between the pile group.

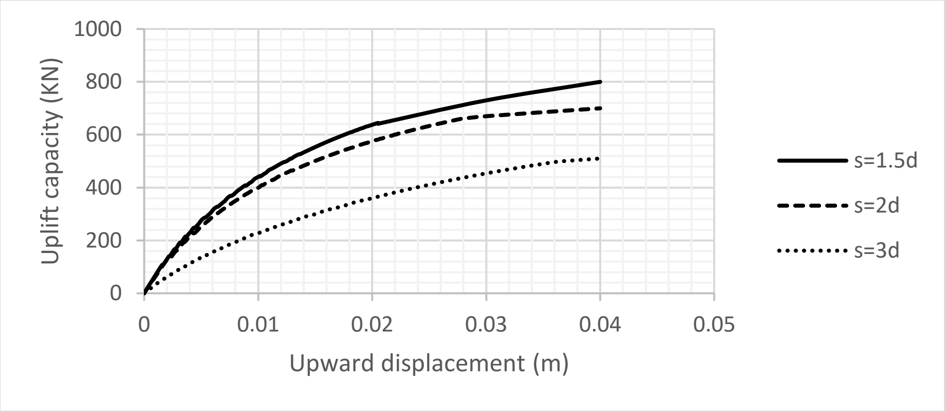


Figure 4.18. Upward movement versus uplift force of GAP for d=0.4 m, L=1 m

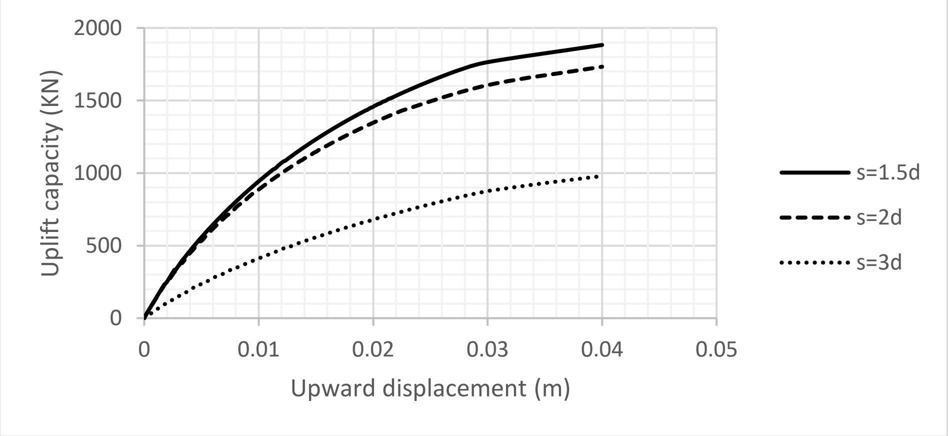


Figure 4.19. Upward movement versus uplift force of GAP for d=0.4 m, L=3 m

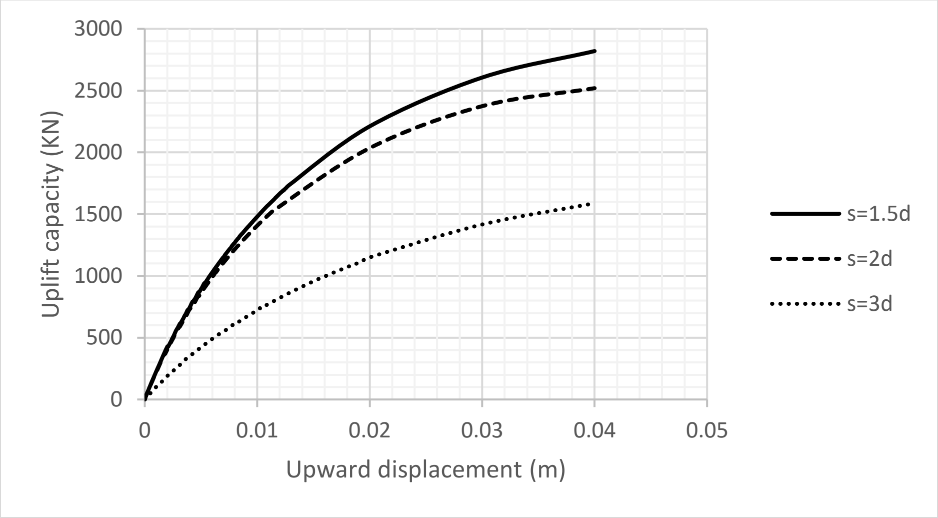


Figure 4.20. Upward movement versus uplift force of GAP for d=0.4 m, L=4 m

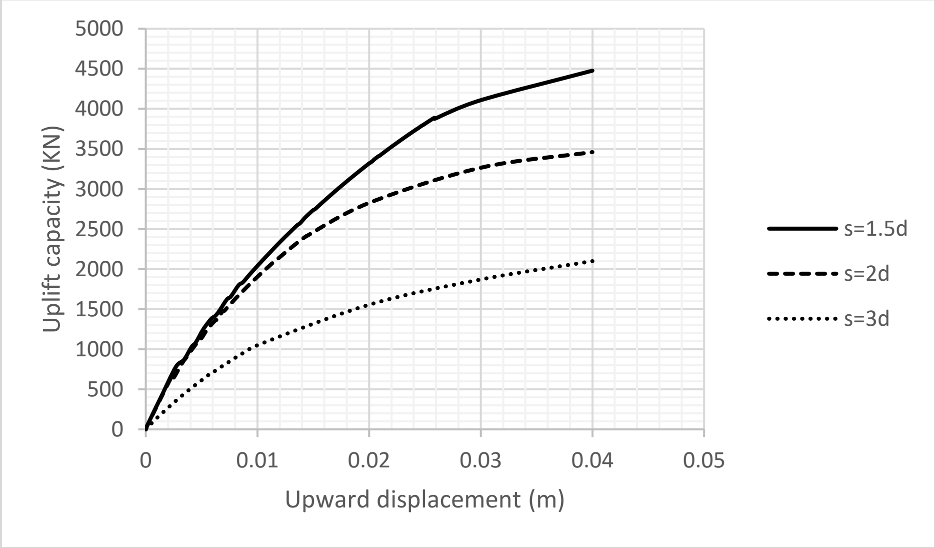


Figure 4.21. Upward movement versus uplift force of GAP for d=0.4 m, L=6 m

Case 4: For spacing s=1.5d, 2d and 3d with d=0.6 m and length L=1, 3, 4, and 6 m the uplift capacity is almost equal for the three spacing as shown in the sample Figures 4.22. and 4.23. the rest is attached in Appendix C Figures C14 and 15. The reason is that for all three cases (i.e. s=1.5d, 2d, and 3d) the number of piles in the group is one or single pile. Therefore, the uplift capacity for all three-case becomes equal.

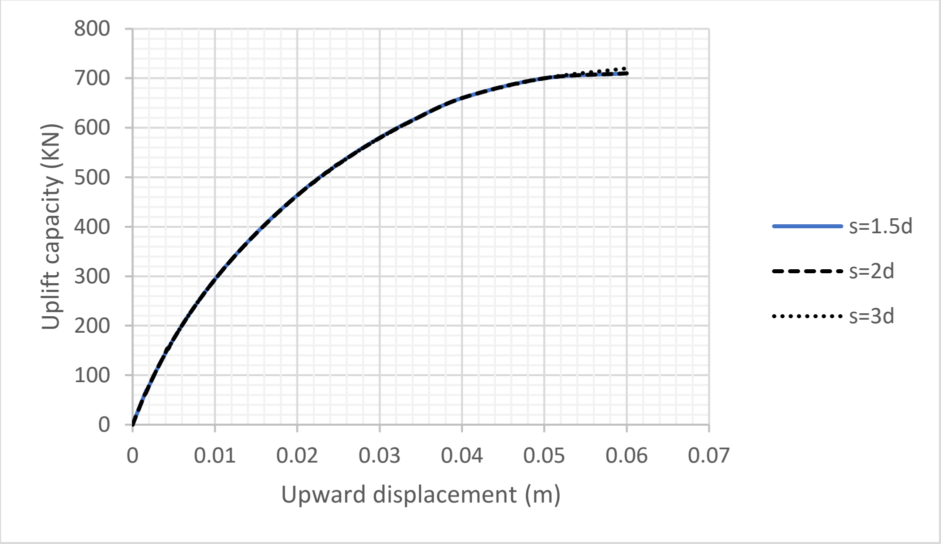


Figure 4.22. Upward movement versus uplift force of GAP for d=0.6 m, L=1 m

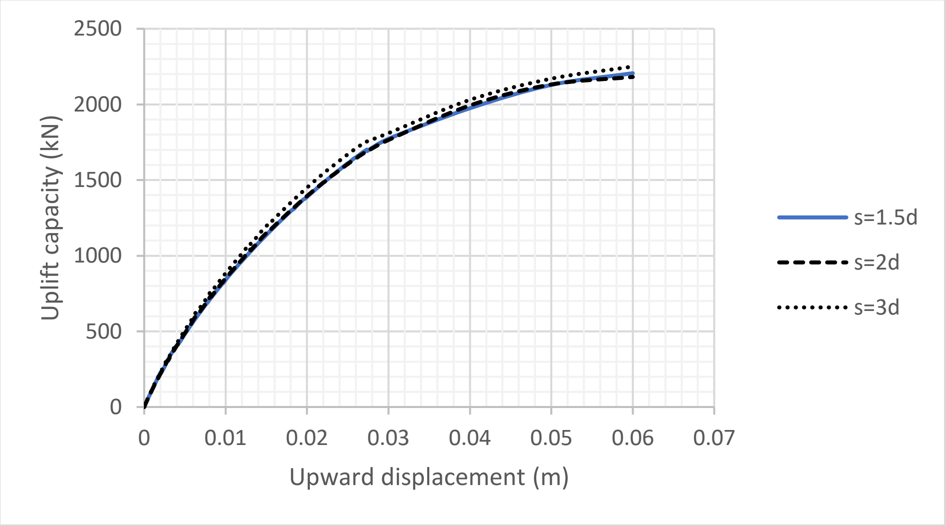


Figure 4.23. Upward movement versus uplift force of GAP for d=0.6 m, L=4 m

4.3.4. Efficiency of a group of Granular Anchor Pile system

The capacity of the pile group is the sum of the individual capacities of piles, but it is influenced by the spacing between the piles. If piles are spaced sufficient distance apart, then the capacity of the pile group is the sum of the individual capacities of piles. However, if the spacing between piles is too close, the zone of stress around the pile will overlap and the ultimate load of the group is less than the sum of the individual pile capacity, where the efficiency of the pile group is much less.

The efficiency of the pile group mainly depends on the Spacing of piles, a total number of piles in a group, and Characteristics of the pile (i.e. diameter and length). Hence, the efficiency η_g of the pile group can be expressed by using the following formula: -

$$\eta_g = \frac{Qg(u)}{NQu}*100 \tag{4.1}$$

where: - η_g – efficiency of pile group, Qg(u) – ultimate uplift capacity of a group of the pile, N – number of piles in the group, and Qu – ultimate uplift capacity of a single pile.

Thus, the pile group efficiency is equal to the ratio of the load capacity of a pile group to the total load capacity of the piles acting as individual piles. Table 4.3. shows the efficiency of a pile group of different spacing.

Table 4.3. Uplift capacity of GAP for different pile spacing and corresponding efficiency

Length (m)	Diameter (m)	L/D	No of pile			Pile capacity (kN)			Efficiency (%)		
			1.5d	**2d**	**3d**	**1.5d**	**2d**	**3d**	**1.5d**	**2d**	**3d**
1	0.2	5.00	16	16	9	705	775	620	59	65	92
	0.3	3.33	9	4	4	790	700	600	56	92	79
	0.4	2.50	4	4	1	800	700	510	80	70	100
	0.6	1.67	1	1	1	710	710	780	100	100	100
3	0.2	15.00	16	16	9	1441	1475	1320	58	59	95
	0.3	10.00	9	4	4	1540	1505	1356	54	96	87
	0.4	7.50	4	4	1	1883	1732	979	98	90	100
	0.6	5.00	1	1	1	1343	1380	1571	100	100	100
4	0.2	20.00	16	16	9	2351	2400	2217	58	60	98
	0.3	13.33	9	4	4	2382	2323	2106	54	97	88
	0.4	10.00	4	4	1	2820	2519	1588	91	81	100
	0.6	6.67	1	1	1	2206	2181	2518	100	100	100
6	0.2	30.00	16	16	9	3212	3300	3028	58	60	97
	0.3	20.00	9	4	4	3536	3137	2865	59	98	90
	0.4	15.00	4	4	1	4477	3461	2100	97	75	100
	0.6	10.00	1	1	1	2974	3180	3266	100	100	100

The Efficiency of the pile group is decreased in the case of d=0.2 m when the spacing between the piles decreased from 3d to 2d and 2d to 1.5d. The reason is that when the spacing between piles is too close or decreased the zone of stress around the pile will overlap causing a reduction in friction mobilized between the soil-pile interface due to soil failures. This condition reduces the capacity of a pile group in general.

In the case of d=0.3m the efficiency of group of pile for s=2d > s=3d > s=1.5d. The reason is that the first thing the number of piles for the case of s=2d and 3d is equal but the uplift capacity is greater in the case of 2d than 3d. Therefore, the efficiency is greater in the case of 2d than s=3d. while for s=1.5d, the number of piles in the group is nine and piles are closest to each other. Therefore, it is suspectable to stress overlap between the piles and soil failure occurrence, due to this the group capacity is much lower than the capacity of the sum of the individual pile that is why the efficiency of the piles is the lowest among the three.

For the case of d=0.4 m the efficiency of group pile for s=3d > s=1.5d > s=2d. The reason is for the case s=3d there is only one pile in the group that acts as a single pile. Therefore, the efficiency is 100 %. In the case of 1.5d and 2d, the number of piles in the group is equal but the uplift capacity is greater in the case of 1.5d than 2d. This shows there is no problem with Stress overlap in the group. So, efficiency is greater in 1.5d than 2d.

In the case of d=0.6 m, the number of piles in the group for all three cases (i.e. s=1.5d, 2d, and 3d) is equal to one or a single pile. Therefore, it considers a single pile or contains one pile with d=0.6 m. So, efficiency is 100 % for all cases.

4.4. Regression Analysis of Uplift Capacity of GAP System

An attempt was made to quantify the observed trend in the uplift capacity of the GAP reinforced in the Expansive soils. For this purpose, the observed values of uplift capacity (Pu) were normalized with axial rigidity (EA) of anchor rod as Pu'=Pu/EA. Similarly, the length of the pile was normalized in terms of L/D ratio L'=L/D. Variation in normalized uplift capacity (Pu') for different L/D ratios L' considered for different spacing (i.e. 1.5d, 2d, and 3d). To develop the mathematical relationship, a regression analysis was performed on the results obtained from the numerical analysis using the MS Excel spreadsheet as shown in Figure 4.24., 4.25. and 4.26.

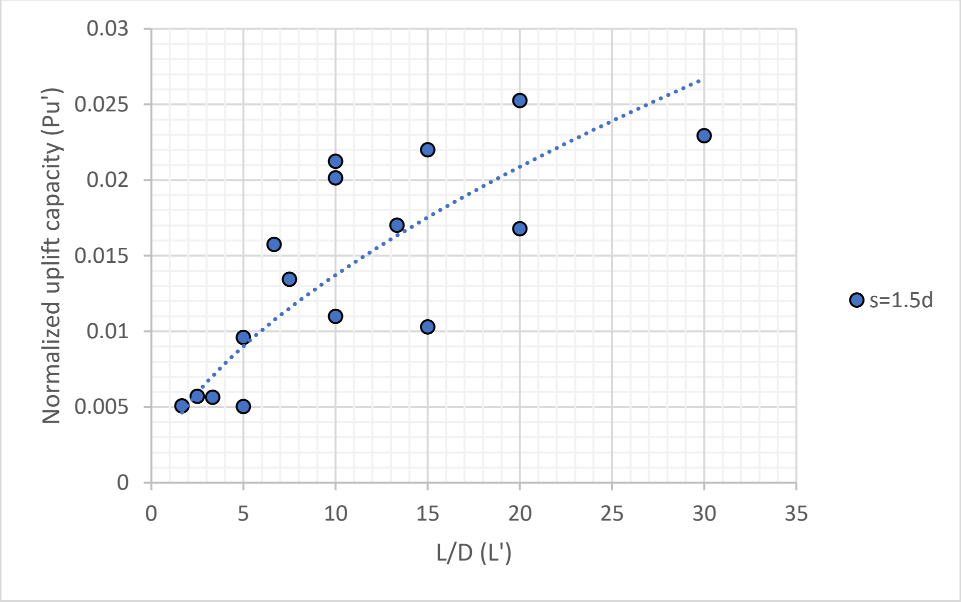


Figure 4.24. Relation between L/D ratio and normalized uplift capacity of the GAP system for s=1.5d

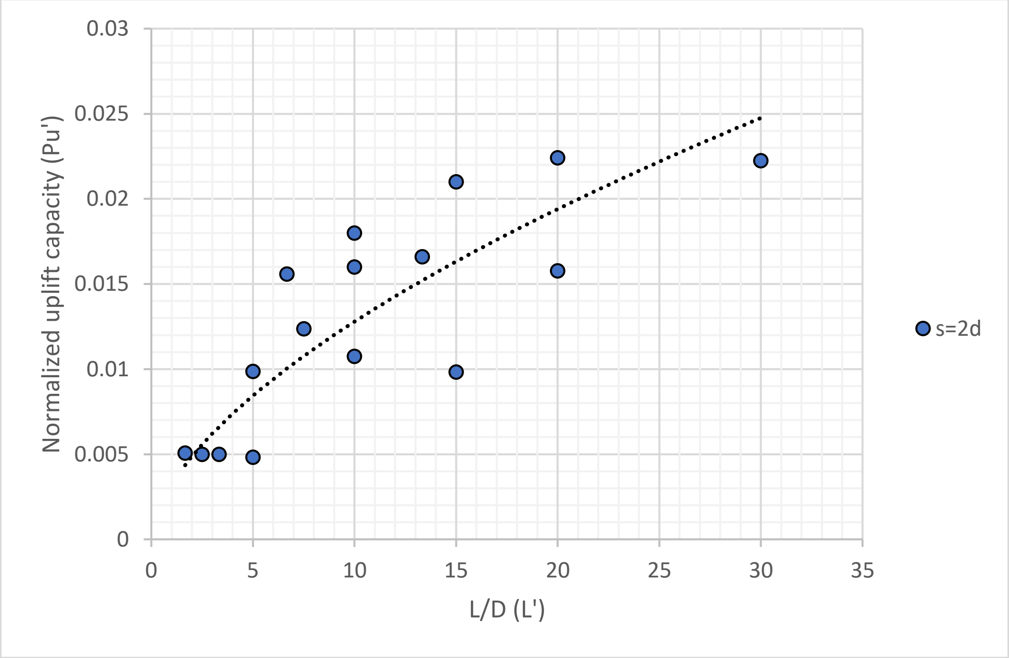


Figure 4.25. Relation between L/D ratio and normalized uplift capacity of the GAP system for s=2d

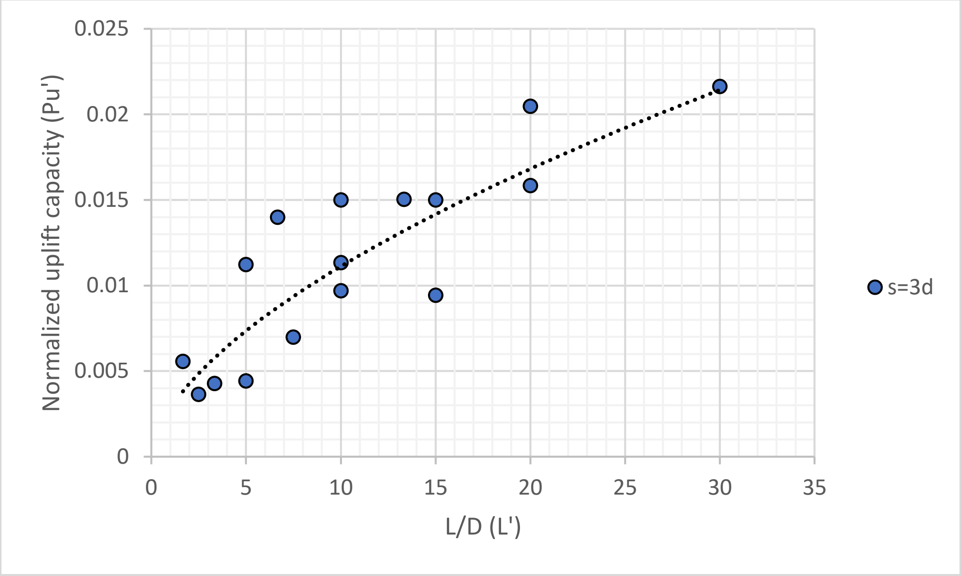


Figure 4.26. Relation between L/D ratio and normalized uplift capacity of the GAP system for s=3d

A simple mathematical expression was developed to relates the dependent variable normalized uplift capacity (Pu’) to the L/D ratio L’ as an independent variable as shown in the equation 4.2, 4.3. and 4.4.

For s=1.5d

$$Pu' = 0.0034 * (L')^{0.6055}, \text{R} = 0.85 \quad (4.2)$$

For s=2d

$$Pu' = 0.0032 * (L')^{0.6022}, \text{R} = 0.86 \quad (4.3)$$

For s=3d

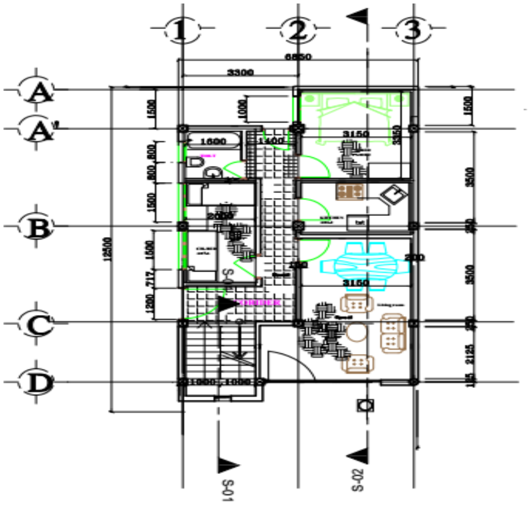
$$Pu' = 0.0028 * (L')^{0.5968}, \text{R} = 0.843 \quad (4.4)$$

The correlation coefficient (R) is a measure of the strength of the straight line or linear relationship between two variables. Generally, a value of R greater than 0.7 is considered a strong correlation. Anything between 0.5 and 0.7 is a moderate correlation, and anything less than 0.4 is considered a week or no correlation (John, 2018). Therefore, the above three equations can predict the L/D ratio of GAP in a better way for given uplift capacity. But the application of the aforementioned expression is limited to the parameters Es = 42.25 – 54.92 MPa and Ep = 240 MPa.

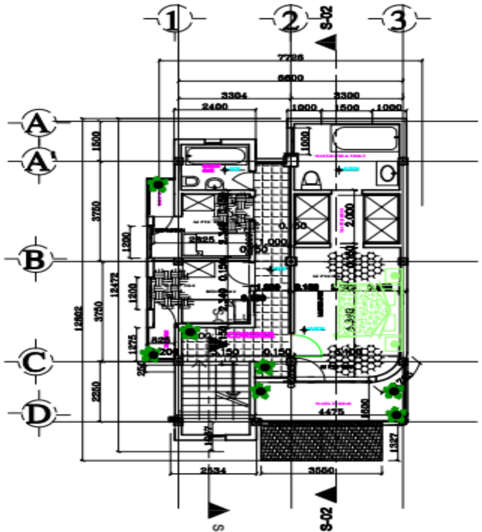
4.5. Suitability analysis

4.5.1. Economy of construction

In Addis Ababa, most of the low rising buildings are constructed using simple shallow isolated footing without considering or undervaluing the existence of reactive soil in the area. It is a common trend in the city because it considered the cheapest footing type. In the designing process of the shallow foundation, the designing engineer considers the bearing capacity of the underlined soil to define the dimension of the isolated footing that only withstands the upcoming superstructure load by underestimating soil's swelling potential. But this footing design method may not be safe against the heave effect for lightweight structure constructed in the area where potential expansive soil widely exists. Because if the soil has high swelling potential possibly the footing can not withstand the swelling effect of the soil which leads to instability in the foundation. This condition can cause serious damage to the foundation as well as in the super-structure part. Therefore, to solve such types of problems the proposed footing must have a capacity to counterbalance the upcoming uplift pressure by increasing the self-weight of the foundation or using another economical alternative method to resist the swelling potential. In this part cost of shallow isolated footing which is practiced currently in the city for lightweight building in expansive soil, and shallow isolated footing attached with a single GAP system was economically compared for the G+1 residential building constructed in Addis Ababa, Bole area around HORRA TRADING building. Figure 4.27. shows the plan area and Front elevation of the building. All necessary soil data are gathered from Addis Geo-System plc for the targeted area as shown in Appendix A, Table A2.



GROUND FLOOR PLAN



FIRST FLOOR PLAN

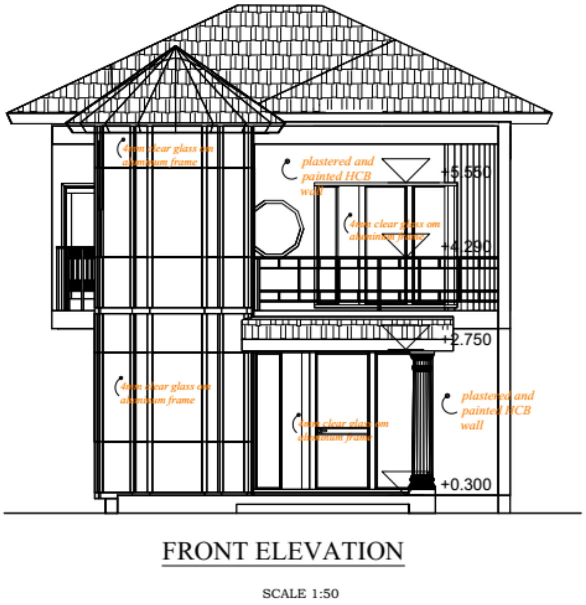


Figure 4.27. Ground Floor and First Floor plan of G+1 Residential Building (Sutphin, 2006)

Before designing any foundation, first, it needs to determine the final column loads which are acted on the foundation. To analyze these loads ETABS 9.7 is used and only the dead load of the building is taken for the analysis by neglecting the live load which acts on the building. This is because it is the worst condition for the building to be failed by the heaving effect of the soil. The modeling of the G+1 building structure and foundation plan is shown in Figure 4.28. and 4.29. respectively. Finally, the column load and moment in X and Y direction of the 12 columns of the building are determined and tabulated in Table 4.4.

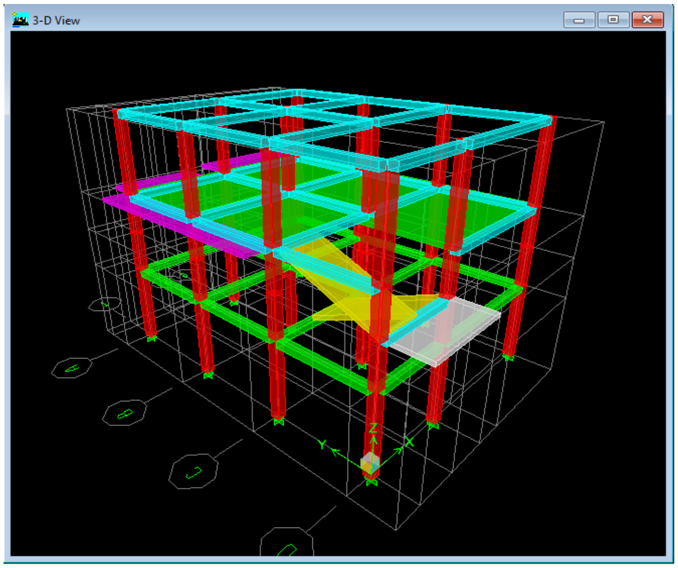


Figure 4.28. Model of G+1 building (Habibullah, 1995)

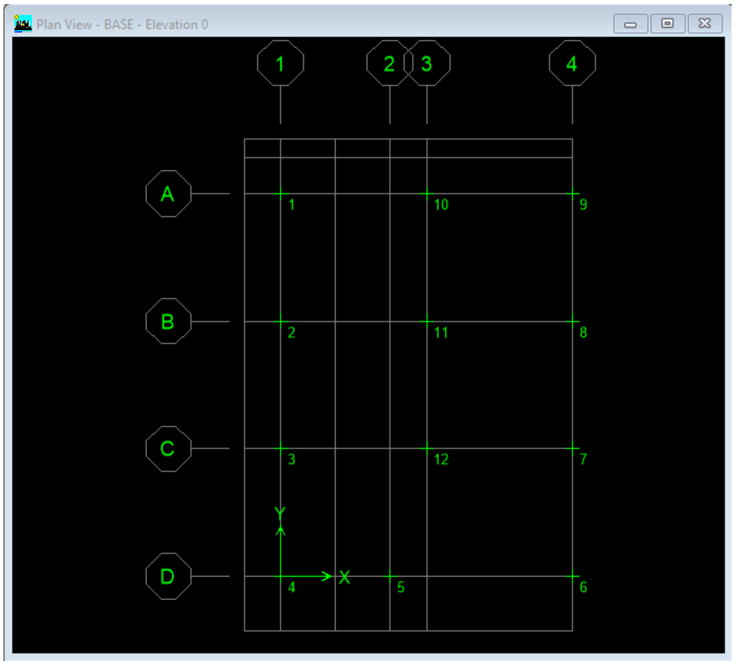


Figure 4.29. Foundation plan of the G+1 Building (Habibullah, 1995)

Table 4.4. Column loads and Moment coming from superstructure load of G+1 Building

Story	Column Point	FZ (kN)_	MX (kNm)	MY (kNm)
BASE	A1	156	-1	0
BASE	B1	212	-1	-1
BASE	C1	205	-1	2
BASE	D1	172	-4	0
BASE	D2	242	6	6
BASE	D4	155	-2	-1
BASE	C4	173	-1	-1
BASE	B4	162	-1	0
BASE	A4	159	2	3
BASE	A3	257	2	-2
BASE	B3	277	-1	-1
BASE	C3	258	-1	-8

From the above columns loads, for this analysis, a column with the minimum and maximum load was considered. Because foundation with small column load it is highly suspectable to effect of heaving effect due to its lightness in load (Worst scenario) and foundation with maximum load expected to have wide footing surface area which reduces the contact pressure of the footing this leads to swelling pressure to be higher than the applied or contact pressure. Therefore, column D4 and B3 is selected with static column load of 155 kN and Mx = -2 kNm and My = -1 kNm and 277 kN and Mx = -1 kNm and My = -1 kNm and column dimension of 0.3*0.3 m.

The uplift pressure of an Expansive soil for the area where the building is constructed was calculated using the following correlation equation which is derived for Addis Ababa Expansive soil by Daniel, 2003.

$$LogPs \ = \ -5 - 0.0002064LL + 0.003477PI + 0.005827\gamma d \tag{4.5}$$

Where: - LL – liquid limit (%), PI - plastic limit (%), and γ_d – dry unit weight (kg/m^3) therefore, according to the data in Appendix A, Table A2 for site 2 the swelling pressure is calculated. The other sites (sites 1, 3, and 4) have less swelling pressure than site 2. Therefore, in this analysis, the critical value was considered.

Hence, P_s = 312.78 kpa

It must be noted that the swelling pressure obtained from a laboratory result can be 1.5 times higher than the actual swelling pressure in the field (Dafalla et al., 2010). This is due to the size and surface friction of the sample ring used in the test and size and shape of the sample, level of disturbance, and error in measurement.

Therefore, Ps(field) = Ps*2/3 = 312.78*2/3 = 208.52 kpa, this swelling pressure value is used for the analysis.

Conventional isolated footing design

The common trend used in the designing of shallow isolated footing is determining the dimension of the footing pad which safe for shear failure and settlement. Nonetheless, it may be safe for the effect of swelling if the contact pressure is greater than the swelling pressure of the soil. Otherwise, it will be suspectable to the damaging effect of the soil. Therefore, it is a must to determine the weight of the footing to check whether it resists or counterbalance the uplift pressure of the soil or not. But it is not practical in our country's condition. If the applied stress due to column load and weight of foundation is less than the uplift pressure of the soil, it needs an additional supporting mechanism to resist the residual swelling pressure. This concept is related to designing of shallow isolated footing by considering the effect of heave of a soil. The safe width of an isolated footing for shear failure can be determined using the Limit state design method that the maximum contact pressure must be less than or equal to the allowable bearing capacity of the soil.

i.e. $\sigma_{max} \leq q_{all}$ (4.6)

where: - $\sigma_{max} = \frac{P}{A} * [1 - \frac{6eb}{b} - \frac{6el}{l}]$ - for a negative moment of M_x and M_y

$q_{all} = qu/FS$

Accordingly, for preliminary estimation of the ultimate bearing capacity, the following equation for square footing is used to determine the bearing capacity of the soil as follows:

$qu = 1.3cNc + \gamma DNq + 0.4B\gamma N\gamma$ (4.7)

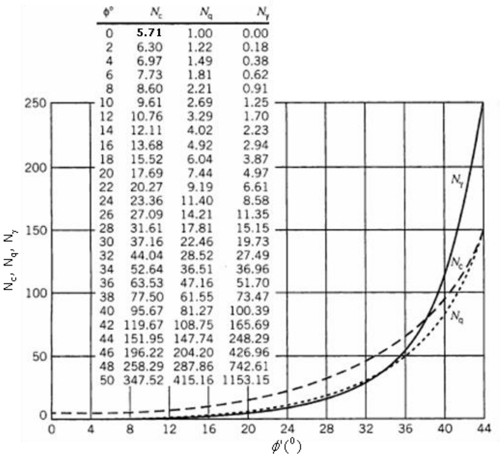
where: - Su- cohesion = 164 kN/m^2

Nc, Nq, Nγ – bearing capacity factors based on the value of $\phi = 0^0$ (undrained condition)

γ - The bulk unit weight of the soil = 17.68 kN/m^3

Depth of foundation (Df) = 2 m… (D/B $\leq$1 for shallow footing)

The value of bearing capacity factors (Nc, Nq, Nγ) can be easily determined from Figure 4.30.



ϕ°	N_c	N_q	N_γ
0	**5.71**	1.00	0.00
2	6.30	1.22	0.18
4	6.97	1.49	0.38
6	7.73	1.81	0.62
8	8.60	2.21	0.91
10	9.61	2.69	1.25
12	10.76	3.29	1.70
14	12.11	4.02	2.23
16	13.68	4.92	2.94
18	15.52	6.04	3.87
20	17.69	7.44	4.97
22	20.27	9.19	6.61
24	23.36	11.40	8.58
26	27.09	14.21	11.35
28	31.61	17.81	15.15
30	37.16	22.46	19.73
32	44.04	28.52	27.49
34	52.64	36.51	36.96
36	63.53	47.16	51.70
38	77.50	61.55	73.47
40	95.67	81.27	100.39
42	119.67	108.75	165.69
44	151.95	147.74	248.29
46	196.22	204.20	426.96
48	258.29	287.86	742.61
50	347.52	415.16	1153.15

Figure 4.30. Relation between bearing capacity factor and ϕ using Terzaghi’s theory (Krizek, 1965)

Accordingly, the value of Nc, Nq, and Nγ are 5.71, 1.0, and 0 respectively.

Therefore, based on the inequality of equation 4.6. the dimension of the footing safe against bearing capacity failure or shear failure is approximated to 1*1 m and 1.2*1.2 m for minimum and maximum load respectively as shown in Appendix D1.

The next step is checking the footing area proportioned above for settlement. To check this, it needs to determine the total settlement which is the sum of immediate and consolidation settlement.

$$\mathrm{St} = S_i + S_c = \left[\frac{qn*B}{E} * (1-\nu^2) * I\right] + [\frac{H}{1+eo}\ Cc \log\left(\frac{po+\Delta p}{po}\right)] \qquad (4.8)$$

Where: -

q_n – contact pressure = (155 kN/1 m^2) + (0.25 m*25 kN/m^3) = 161.25 kN/m^2 for (p=155 kN),

q_n – contact pressure = (277 kN/1.44 m^2) + (0.35 m*25 kN/m^3) = 201.11 kN/m^2 for (p=277 kN),

B = 1 m, E = 69,289 kPa, ν = 0.332, I – influence factor = 0.82, H (thickness of clay layer) = 10 m, Cc = 0.3115 and e_o(initial void ratio) = $\frac{Gs\gamma w - \gamma d}{\gamma d}$ = 1.16.

Since the thickness of the clay layer is 10m, therefore, the settlement calculation can be done on each 1m depth. Therefore, the overburden pressure and pressure increment can also calculate in each layer mid-depth. Accordingly, the total settlement is the summation of consolidation settlement of each layer plus immediate settlement and it becomes 75.223 mm and 90.1 mm for minimum and maximum load respectively as shown in Appendix D2. Since the total allowable settlement is limited to 75 mm for clayey soil (EBCS). Therefore, the foundation pad is not safe for total settlement for both conditions. So, it needs to increase the dimension of the foundation pad. Hence, when the footing pad is increased to 1.1*1.1 m and 1.4*1.4 m for minimum and maximum load the settlement reduced to 65.1 mm and 71.7 mm respectively as shown in Appendix D3 which is within the permissible limit. The thickness of foundation pads which is safe for wide-beam shear and punching shear is equal to 0.25 m and 0.35 m as per the design by assuming concrete cover 50 mm and bar diameter of 12 mm.

Generally, the safest dimension of the isolated footing to carry the static load of 155 kN and Mx = -2 kNm and My = -1 kNm is determined as F6 =1.1*1.1*0.25 m with a reinforcement bar of ϕ_{14} c/c 450 mm in both directions. And for the static load of 277 kN and M_x=-1 kNm and M_y=-1 kNm is determined as F11 = 1.4*1.4*0.35 m with a reinforcement bar of ϕ_{14} c/c 300 mm in both directions.

Accordingly, the allowable bearing capacity is calculated as

σ_{all}(p=155 kN) = (1.3cNc + γDNq + 0.4BγNγ)/3 = 353.3 kN/m^2

σ_{all}(p=277 kN) = (1.3cNc + γDNq + 0.4BγNγ)/3 = 354.7 kN/m^2

Therefore, the contact pressure of the footing also reduced to: -

$q_{n(P=155KN)}$ = (Column load/Pad area) + (self-weight of the pad)

= (155KN/1.21m^2) + (0.25*25) KN/m^2

= **134.35 kN/m^2**

$q_{n(P=277KN)}$ = (Column load/Pad area) + (self-weight of the pad)

= (277KN/1.96m^2) + (0.35*25) KN/m^2

= **150.07 kN/m^2**

As stated above the uplift pressure of soil in the field is assumed to be 208.52 kN/m^2 and the total pressure exerted by the footing is calculated as 134.35 kN/m^2 and 150.07 kN/m^2 for minimum and maximum load respectively. Therefore, the swelling pressure can exceed the applied stress with 74.17 kN/m^2 and 58.45 kN/m^2 for both cases. Swelling pressure of greater than about 20KN/m^2 may be regarded as of much consequence (Rajan, 2007). Accordingly, the residual uplift pressure (i.e. 74.17 kN/m^2 and 58.45 kN/m^2) is greater than the recommended value (i.e. 20 kN/m^2). Therefore, it is expectable that the soil can cause considerable damage to the structure at a certain time when it gains much of water or become more saturated. According to the result, designing of shallow isolated footing without considering the expansiveness of the soil cannot fully keep away the effect of heave in an area where the swelling effect is high like the selected site. But to counterweight this uplift pressure and making the structure safe it needs to consider the effect of heave of soil in the design of the foundation.

Designing of Granular Anchor Pile

The next step is determining the dimension of Granular Anchor Pile which resists the residual upcoming uplift pressure of 74.17 kN/m^2 and 58.45 kN/m^2 with minimum upward displacement. The rest are already resisted by the footing.

To estimate the probable dimension of the GAP system to resist the uplift pressure an empirical Equation stated in Equation 2.1. and 2.15. are used. Additionally, it is possible to reduce the pad dimension until the swelling pressure cannot cause any significant effect or below 20 kN/m^2. This makes the design economical by reducing the footing dimension and GAP dimension. Hence, 1*1m and 1.3*1.3 m footing dimension are used under the GAP, below this value cannot need any remedial measure because it counterbalances the swelling effect fully by its weight. Accordingly, the design of GAPs can be analyzed for the designed footing dimension and reduced pad dimension as shown in Appendix D4 with corresponding swelling pressure for both maximum and minimum column load and tabulated in Table 4.5.

Table 4.5. Single Granular Anchor Pile system with minimum upward displacement

Load (kN)	Contact pressure (kN/m$^{2)}$	Residual pressure (kN/m$^{2)}$	Pad dimension	Vertical uplift load (kN)	GAP dimension
155	134.35	74.17	1.1*1.1m	84.33	L=2m & d=0.3m
	161.25	47.27	1*1m	44.06	L=1.5 m & d=0.2 m
277	150.07	58.45	1.4*1.4m	110.46	L=2.5 m & d=0.3 m
	17265	35.86	1.3*1.3m	56.1	L=1.6 m & d=0.3 m

According to the result, it can be used the optimum GAP dimension for the cost comparison purpose to be economical.

After investigating all these things, finally, a cost analysis between shallow isolated footing and GAP system rested on the reduced dimension of isolated footing is done and tabulated in Table 4.6. and 4.7. The cost analysis includes Material cost, Equipment and Machinery rental cost, and Labor cost. The information is gathered from material production companies, machinery rental companies, senior persons in the construction industry, and from a well-known construction website in Ethiopia called Constructionproxy.com and ethiopianconstruction.com Therefore, the cost analysis is calculated in the next page.

Table 4.6. Cost analysis of shallow isolated footings

Item	Unit	Unit rate (ETB)	Qty* (1.1*1.1*0.25m)	Qty** (1.4*1.4*0.35m)	Amount* (ETB)	Amount** (ETB)
Material Cost						
Cement	Bag	187.5	3	6	561.6	1,125
Aggregate	m^3	510	0.324	0.648	165.24	330.48
Sand	m^3	700	0.21	0.42	147	294
Reinforcement bar	kg	38	12.56	26.14	477.28	993.32
Form-work (Zigba-form work) 2.5 cm local wood	m^2	300	1.125	1.995	337.5	598.5
Equipment and Machinery Cost						
Concrete Mixer	Hr.	100	1	2	100	200
Labour Cost						
Carpenter	m^2	70	1.125	1.995	78.75	139.65
Barbender	kg	6	12.56	26.14	75.36	156.84
Total Cost					**1,942.73**	**3,837.79**

Table 4.7. Cost analysis of GAP system rested on shallow isolated footing

Item	Unit	Unit rate (ETB)	Qty* GAP of L=1.5 m & d=0.2 m with footing (1*1*0.25m)	Qty** GAP of L=1.6 m & d=0.3 m with footing (1.3*1.3*0.35m)	Amount* (ETB)	Amount** (ETB)
Material Cost						
Cement	Bag	187.5	2	5	375	937.5
Aggregate	m^3	510	0.216	0.54	110.16	275.4
Sand	m^3	700	0.14	0.35	98	245
Aggregate for GAP	m^3	510	0.0283	0.0678	14.43	34.578
Sand for GAP	m^3	700	0.0188	0.0452	13.16	31.64
Q345 A36 25 mm thick mild steel plate	Ton	19,500	0.0069	0.015	134.55	262.5
A36 30 mm carbon MS round rod bar	Ton	20,329	0.0091	0.0097	194.99	197.191
Reinforcement bar	kg	38	8.712	20.57	331.056	781.66
Form work (Zigba form work 2.5cm local wood	m^2	400	1.025	1.855	410	742
Equipment and Machinery Cost						
Concrete Mixer	Hr.	100	0.66	1.6	66	160
Drilling Machine	m	1000	1.5	1.6	1,500	1,600

Welding Machine	Hr.	83	1	1	83	83
Vibro-compactor	Hr.	600	2	2	1,200	1,200
Labour Cost						
Carpenter	m^2	70	1.025	1.855	71.75	129.85
Barbender	kg	6	8.712	20.57	52.272	123.42
Total Cost					**4,654.37**	**6,803.74**

According to the result, there is an additional cost of 2,711.64 ETB and 2,965.95 ETB to provide a GAP system per shallow isolated footing for minimum and maximum column load respectively. This shows that the cost of constructing isolated footing alone is cheaper than installing the GAP system. But the designed isolated footing cannot fully counterbalance the upcoming uplift pressure. Different structural and non-structural damage is occurred on the building depending upon the amount of moisture in the soil. such as jammed or misaligned Doors and Windows, distortion and cracking of floor slab, cracks in grade beams and walls, failure of blocks supporting grade beams. These failures can need routine maintenance, replacement of some non-structural parts and if it is serious structural stability weakened, and maybe the building is out of service. These all things are shown using simply isolated footing alone in highly Expansive soil bed can cost high maintenance cost even loss of the building.

In contrast providing of GAP system under the shallow footing has a higher initial cost when compared to shallow isolated footing but it can sustain an uplift pressure of soil efficiently or there is an extremely small movement of soil. This makes the building considerably safe and there is no maintenance cost associated with the effect of swelling pressure of soil. This makes the foundation affordable or sensible in cost.

Generally, using of GAP system in resisting uplift pressure of an expansive soil for a low rising building is affordable and more preferable in reducing the damaging effect of expansive soil in a greater amount and minimizing the maintenance cost of the structure.

Kumar, et.al. (2018) compare the material cost of GAP with a concrete pile for Expansive soil as shown in Figure 4.8. According to the result, the material cost of GAP is nearly half of the concrete pile of the same dimension.

Table 4.8. Comparison of the material cost of GAP and Concrete pile (Kumar, et.al. 2018)

No	L/d	Material cost of GAP (INR)	Material cost of concrete pile (INR)	% difference
1	6.66	836.8	1653.25	97.57
2	10	1096.64	2248.58	105.04
3	13.3	1356.48	2843.91	109.65
4	20	1876.16	4034.57	115.04

4.5.2. Local availability of construction material

When someone thinks about sustainable products, does transportation distance, material origin, or manufacturing site comes to mind. But locally sourced products can boost its sustainability credibility.

Locally available construction materials are the resources that can be found readily in efficient quantity in the vicinity or within a specified radius (distance) from the construction site. Using locally available resources can give several advantages such as products are delivered quickly, getting the materials at a fair price, reduce transportation cost, it makes dealing with problems simpler and encourage local producers.

Since Granular Anchor Pile (GAP) uses stone aggregate, sand, Mile steel anchor plate, anchor rod as a construction material. Therefore, it is necessary to find local producers or providers and the amount of production of the material in the locality of Addis Ababa.

- **Aggregate and Sand Producers and Suppliers**

GAP system can use about 60 % aggregate and 40 % sand mix proportion for its granular fill around the anchor rod to get a maximum density and this granular fill covers the majority part of the pile. Therefore, it is necessary to know there is an efficient material resource in the vicinity of the construction site. In Addis Ababa, there are a few aggregate crusher plants. But these few companies can produce and supply a large number of aggregate materials to the construction sector of Addis Ababa and the outskirts areas. The most well-known aggregate crushing plants in this area are called Tikur Abay stone crusher plant and ASER crusher plant with a capacity of producing an average of 150 Tons/hr. Additionally, several companies work as a construction material supplier in the city. Some of them are Addisu Getenet construction supplier, KSY mining and manufacturing plc, Degfe Construction Material Supplier, and others. These local companies can primarily provide selected material, aggregate, sand, cursed sub-base, and base course for local users in Addis Ababa as per the customer's amount of needs. Therefore, it is apparent that there is a sustainable and enough supply of aggregate and sand in Addis Ababa for the construction of GAP.

- **Steel plate and steel rod manufacturer and supplier**

Steel materials (Anchor plate and Anchor rod) are the second essential part of the GAP system next to granular fill. Therefore, it is necessary to identify the types and number of suppliers or producers of these steel materials in the locality. Accordingly, there are several steel factories and suppliers established in Ethiopia especially in Addis Ababa aiming to provide sustainable production of steel products for all steel users. Some of them are Akaki steel factory, Ethiopian steel PLC, Alem steel PLC, Ocfa metal manufacturing, BMK Tila steel factory, East steel PLC, Steely RMI PLC, and there are more than 25 steel and iron suppliers in Addis Ababa according to the information of ethiopiamarket.com. Since the GAP system uses A36 carbon mild steel grade for plate and anchor rod. Therefore, some of the above mention companies can produce steel plates and steel rod with different form and quality (grade) and special orders as per customer's design ("producers of steel products," 2017). Some of the remaining are imported high-quality steel products from a different foreign country with a fair price.

4.5.3. Installation process condition

After all necessary investigations are carried out on the soil and bulk excavation at the desired depth is excavated, a borehole of required dimension is drilled in the ground using simple ground hole drilling machine or Crawler mounted rotary drilling machine. Then, lean concrete is poured at its bottom through a pipe. After that, a pre-fabricated anchor plate and anchor rod are welded together using a welding machine (Arc welding) and then lowered and positioned at the bottom of the borehole. Then Granular Anchor Pile is installed with a mixture of stone aggregate and sand in predetermined layers. Each layer was given a uniform amount of compaction energy using Vibro-compaction. Finally, the isolated footing with proper dimension is constructed on it which is in contact with the GAP system using one end of the anchor rod. Therefore, according to the installation processes indicated, the process does not require any advanced technology which does not exist in the country and special follow-up and care. Thus, the GAP system can be easily installed even in a small scale.

4.5.4. Environmental Benefit of Granular Anchor Pile

It's no secret that the construction industry has a significant impact on the environment. Construction is responsible for up to 50 % of climate change, 40 % of energy usage globally, and 50 % of landfill waste and also pollution of air, water and noise, and destruction of natural habitats ("3 ways the construction," 2019). As the effects of climate change and other environmental issues become more pronounced, it is more important than ever to find ways to protect the environment. Granular Anchor Pile is a type of pile that used to resist uplift forces acting on any civil engineering structure founded on it. But this pile to be secured and sustainable solution for such a problem it must be environmentally friendly. Unlike the other piles, Granular Anchor Pile has some environmental benefit: -

- It can be installed in any weather condition. Which can preserve the project from unnecessary delays, economic loss, and absence of quality work.
- It can be principally used locally available materials that do not adversely impact the environment.
- Has less waste material. The environmental benefits of reducing waste include less waste going to landfill, less use of natural resources, less carbon dioxide (CO_2) emission from producing, transporting, using of materials and disposing of waste materials, and lower risk of pollution incidents.
- It creates less noise and vibration during installation and compaction to the surrounding environment, due to this it reduces the disturbance of foundation soil and structural damage of surrounding structure which occurred due to vibration, and eliminate problems because of excessive noise like a general annoyance, interference with speech communication, sleep disturbance and even hearing impairment.

4.5.5. Construction time

Time management can play a crucial role in the construction sector. It helps to organize and implement a strategy related to the time required for work activities on a project. Effective time management is essential to successfully and efficiently meet budget and program targets, as well as achieving profitability. Granular Anchor pile system can contribute its own sketch in relation to minimizing construction time of work.

- Granular Anchor pile can bear weight immediately after its installation, no need of time for attaining full strength or there is no curing time.
- Since the GAP system can bring high performance in resisting the upcoming uplift force with a smaller dimension of the pile as shown in the numerical analysis part. Therefore, the installation process of the Granular Anchor Pile system can take a shorter time because of its smallness in dimension and easiness of erection or installation.

4.5.6. Performance of the pile

Expansive soil can bring on considerable damage to structures, especially for lightly loaded structures. This damage can make the structure out of service and unusable. Such type of situation creates high economic devastation. However, there are several remedial measures recommended to minimize the potential of expansive soil and change the property of the soil in general. Such measures are Sand cushion method, Cohesive non-swelling (CNS) layer method, Chemical alternation, and Adopting special foundation techniques.

The efficiency of these measures is depending on the performance they show to counterbalance the heave effect of the soil. Using high-performance techniques can preserve from excessive utilization of resources. This means when using a technique having high performance, with minimum consumption of resources it brings high efficiency or can give a prospective result.

Accordingly, at this point, it is an attempt to check the performance of the GAP system based on the analysis result. Table 4.9. shows the ultimate uplift capacity of GAP with corresponding dimension and pile spacing.

Table 4.9. Ultimate uplift capacity of GAP

Diameter (m)	Length (m)	Ultimate uplift capacity (kN)			Ultimate uplift pressure (kN/m^2)			No of pile		
		S=1.5d	S=2d	S=3d	S=1.5d	S=2d	S=3d	s=1.5d	s=2d	s=3d
0.6	1	710	710	780	177.5	177.5	195	1	1	1
	3	1343	1380	1571	335.75	345	392.75			
	4	2206	2181	2518	551.5	545.25	629.5			
	6	2974	3180	3266	743.5	795	816.5			
0.4	1	800	700	510	200	175	127.5	4	4	1
	3	1883	1732	979	470.75	433	244.75			
	4	2820	2519	1588	705	629.75	397			
	6	4200	3461	2100	1050	865.25	525			
0.3	1	790	700	600	197.5	175	150	9	4	4
	3	1540	1505	1356	385	376.25	339			
	4	2382	2323	2106	595.5	580.75	526.5			
	6	3536	3137	2865	884	784.25	716.25			
0.2	1	705	675	620	176.25	168.75	155	16	16	9
	3	1441	1375	1320	360.25	343.75	330			
	4	2351	2209	2217	587.75	552.25	554.25			
	6	3212	3113	3028	803	778.25	757			

According to the above table result, the GAP system can exert high performance as shown in terms of ultimate uplift pressure with minimum pile dimension. These results proper utilization of resources which contribute a lot of benefits in preventing the environment, time consumption, and saving of money.

5. CONCLUSION AND RECOMMENDATION

5.1. Conclusion

The effect of key parameters such as the length and diameter of the pile, group of the pile, and efficiency of the pile on the performance of the GAP system were examined. Therefore, based on the present study the following conclusion can be made:

- The uplift capacity of the pile improves significantly with an increasing length of the pile up to a certain length. The percentage increase in uplift capacity of the pile is 109, 58, and 39% in average when the pile length increased from 1 to 3m, 3 to 4m, and 4 to 6m respectively. But after which the increase in the length does not show a significant effect on the load sharing of the pile. This is due to the reduction of the adhesion between the pile material and clay with depth.
- The uplift capacity of GAP decreases with increasing the diameter of the piles in the group for constant spacing. The capacity was reduced by 5.5, 17.1, and 32% in average when the diameter of the pile increased from 0.2 to 0.3m, 0.3 to 0.4m, and 0.4 to 0.6m respectively. This is due to the reduction of the number of piles in the group because of the increment of spacing between the pile. Since spacing is expressed in terms of the diameter of the pile.
- The uplift capacity of GAP decreases when the spacing between the pile in the group increase for constant diameter of the pile in the group. The capacity was reduced by 12 and 10% when pile spacing increased from 1.5d to 2d and 2d to 3d respectively. This is because of a reduction in soil confinement between the piles due to increment of spacing between the piles and reduction of the number of piles in the group.
- The analyses also show that using a group of piles instead of a single pile under a footing can reduce the efficiency of the GAP system.
- According to the suitability analysis result, it is confirmed that the GAP system is the best alternative solution for the problem of Expansive soil on a low rising building and it is beneficial for the owners, contractors, and countries in general by reducing maintenance cost associated with heave effect.

5.2. Recommendation

- It is recommended to investigate the performance of the GAP system for long structure, (i.e. transmission towers, high raised buildings) rested on loose soil like loose sandy soil subjected to a high tensile force.
- It is better to do further investigation on the performance of the GAP system by considering additional parameters like the effect of the number of anchor plates in a single pile, Geomembrane covering, and different footing dimensions to know their contribution on the performance improvement of the GAP.
- It is better to investigate the capacity of the GAP system in increasing the bearing capacity and reducing the settlement of weak soil or compressible soil as a stone column.

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Adama, Ethiopia

REFERENCES

1. Aljorany, A. N.-A. (2014). Heave Behavior of Granular Pile AnchorFoundation System. *Journal of Engineering, 20(4)*, 1-22.
2. Anderson, K. T. (1969). Modification of expansive soils of western Canada with. *Texas conference on expansive soils*.
3. Anderson, K.O., & Thompson, S. . (1969). Modification of expansive soils of western Canada with. *Texas conference on expansive soils*.
4. Bharati, K. S. (2017). Experimental Study of Granular Pile. *International Journal for Technological Research, 6(2)*.
5. Bharati, K. S., Ramakrishna, M., & Gopalalerishpa P. . (2017). Experimental Study of Granular Pile Anchor Foundation. *International Journal for Technological Research, 6(2)*.
6. Bolton, M. (1986). The strength and dilatancy of sands. *G´eotechnique, 36(1):65{78. 86}*.
7. Bowles, L. E. (1996). *Foundation analysis and design.* McGraw-hill.
8. Brinkgreve, R. B. J., Engin, E., & Swolfs, W. M. . (2013). *PLAXIS 3D 2013 user manual.* Plaxis bv, Delft.
9. Budhu, M. (2000). *Soil Mechanics and Foundations.* . John Wley& Sons. Inc. New York.
10. Dafalla, M. A., Shamrani, M. A., Puppala, A. J., & Ali, H. E. (2010). Use of rigid foundation system on Expansive soils. *In GeoFlorida 2010: Advances in Analysis, Modeling & Design* , 1680-1689.
11. Daniel, T. (2003). Examining the swelling pressure of Addis Ababa Expansive soil. *Addis Ababa University* .
12. *Grade Guide: A36 steel*. (2017, feb. 14). Retrieved from http://www.metalsupermarket.com/grade-guide-a36-steel/.
13. Habibullah, A. (1995). ETABS Three Dimensional Analysis of Building Systems. *Computers and Structures*.
14. Holtz, W. (1954). Engineering properties of expansive clays. *Transactions of the American Society of Civil Engineers, 121,* 641-677.
15. Hugher, J. M. O., & Withers, N. J. . (1974). Reinforcing of soft cohesive soils with stone columns. *Ground engineering, 7(3)*.
16. Ismail, M., & Shahin, M. (2011). Finite element analyses of granular pile anchors as a foundation option for reactive soils. *In Proceedings of the International Conference on*

Advances in Geotechnical Engineering (pp. 1047-1052). Australian Geomechanics Society.

17. John, C. (2018). *Pearson correlation coefficient*. Retrieved from http:// www.magoosh.com.
18. Johnson, N., & Sandeep, M. N. . (2016). Ground Improvement Using Granular Pile Anchor Foundation. *Procedia Technology, 24* , 263-270.
19. Kebede T. , & Tadesse H. (1990). *Engineering Geological Map of Addis Ababa.* Addis Ababa, Ethiopia: Ethiopian Institute of Geological survey.
20. Kezdi, A. (1974). *Handbook of Soil Mechanics.* Elsevier, Amsterdam. 71, 108.
21. Kranthikumar, A., Sawant, V. A., & Shukla, S. K. . (2016). Numerical modeling of granular anchor pile system in loose sandy soil subjected to uplift loading. *International Journal of Geosynthetics and Ground Engineering, 2(2),* 15.
22. Kranthikumar, A., Sawant, V. A., Kumar, P., & Shukla, S. K. (2016). Numerical and experimental investigations of granular anchor piles in loose sandy soil subjected to uplift loading. *International Journal of Geomechanics, 17(2), 04016059.*
23. Krishna, P. H., & Murty, V. R. (2013). Pull-out capacity of granular anchor piles in expansive soils. *IOSR Journal of Mechanical and Civil Engineering, 5(1).*
24. Kumar, P., Kumar, M., Chandaluri, V. K., & Sawant, V. A. (2018). Uplift Capacity of Single and Group of Granular Anchor Pile System. *Journal of Civil Engineering, Science and Technology, 9(1),* 34-40.
25. Kumar, R., Bhargava, K., & Choudhury, D. (2016). Estimation of Engineering Properties of Soils from Field SPT Using Random Number Generation. *INAE Letters, 1(3-4),* 77-84.
26. Mowafy, M. Y., & Bauer, G. E. . (1985). Prediction of swelling pressure and factors affecting the swelling behaviour of an expansive soils. *Transportation research record, 1032*, 23-28.
27. *Numerical analysis*. (2020, Feb. 18). Retrieved from https://en.wikipedia.org/wiki/Numerical_analysis.
28. Phanikumar, B. R. (2016). Influence of geogrid reinforcement on pullout response of granular pile-anchors (GPAs) in expansive soils. *Indian Geotechnical Journal, 46(4),* 437-444.
29. Phanikumar, B. R., & Suri, S. (2009). Expansive soils—problems and remedies. *In Indian geotechnical conference,* (pp. 907-913). Guntur, India .

30. Plc, A. G. (2017). *Geotechnical Investigation Report for 2B+SB+G+18 Mixed Use.* Addis Ababa, Bole area.

31. Plc, A. G. (2017). *Geotechnical Investigation Report for 2B+SB+G+18 Mixed Use.* Addis Ababa, Bole Area.

32. Prat, M., Bisch, E., Millard, A., Mestat, P., & Cabot, G. (1995). La modelisation des ouvrages. *Hermes, Paris. 71,* 108.

33. *Producers of steel products in Ethiopia*. (2017). Retrieved from https://www.2merkato.com.

34. Rajan, G., & Rao, A. S. R. (2007). *Basic and applied soil mechanics.* New Age International.

35. Rajapakse, R. A. (2016). *Pile design and construction rules of thumb.* Butterworth-Heinemann.

36. Rao, A. S., Phanikumar, B. R., Babu, R. D., & Suresh, K. (2007). Pullout behavior of granular pile-anchors in expansive clay beds in situ. *Journal of Geotechnical and Geoenvironmental Engineering, 133(5),* 531-538.

37. RaviTeja, M., & Murthy, N. K. (2018). Review on foundation practices for expansive soils, special emphasis on granular anchor pile foundation. *International Journal of Engineering & Technology, 7(2.1),* 84-86.

38. Sanglerat, G. (1972). Interpretation of penetration diagrams-theory and practice. *Developments in geotechnical engineering.* Amsterdam, The Netherlands: Elsevier publishing company.

39. Schanz, T. , & Vermeer, P. (1998). On the stiffness of sands. . *In Jardine, R., Davies, M., Hight, D., Smith, A., and Stallebras, S., editors, Pre-failure deformation behaviour of geomaterials,* 383.

40. Seed, H. B., & Lundgren, R. (1962). Prediction of swelling potential for compacted clays. *Journal of the soil mechanics and foundations division, 88(3),* 53-87.

41. Sharma, R. K. (2019). A numerical study of granular pile anchors subjected to uplift forces in expansive soils using PLAXIS 3D. *Indian Geotechnical Journal, 49(3),* 304-313.

42. Sisay, A. (2004). *Assessment of Damage of Buildings Constructed in Expansive Soil Areas of Addis Ababa.* Ethiopia: Addis Ababa University.

43. Sivakumar, V., O'Kelly, B. C., Madhav, M. R., Moorhead, C., & Rankin, B. (2012). Granular anchors under vertical loading–axial pull. *Canadian Geotechnical Journal, 50(2),* 123-132.

44. Sridharan, A., & Prakash, K. (2000). Classification procedure for expansive soil. *Proceeding of the Institution of Civil Engineers-Geotechnical Engineering, 143(4)*, 235.

45. Sutphin, J. (2006). AutoCAD 2006 VBA: a Programmer's reference . *Apress*.

46. Ti, K. S., Huat, B. B., Noorzaei, J., , Jaafar, M. S., & Sew, G. S. (2009). A review of basic soil constitutive models for geotechnical application. *Electronic Journal of Geotechnical Engineering, 14*, 1-18.

47. Tibebu , S. (2015). *Assessment of Damages Caused by Expansive Soil on Buildings Constructed in Bahir Dar*. Ethiopia: Addis Ababa University.

48. Truty, A., & Obrzud, R. . (2011). *The Hardening soil model-a practical guidebook.* Zace Services Ltd, Software engineering. Lausanne.

49. Uge, B. U. (2011). Expansive soils in Ethioipa: a Review.

50. *Vibro-compaction*. (2014, Feb. 29). Retrieved from https://www.franki.co.za.

51. Wendell, C. (2012). *The Evolving Urban for: Addis Ababa*. Retrieved from https://www.newgeography.com/content/003203-the-evolving-urban-form-addis.

APPENDICES

Appendix A

Table A1: - Summary for granular fill parameters

SN	Autor	γ (kN/m^3)	γ_{sat} (kN/m^3)	C (kN/m^3)	Φ(deg.)	E(MPa)	υ	ψ
1	Kranthikumar et.al., 2016	22	24	0	36	240	0.3	-
2	Sharma, 2019	22	24	0	36	240	0.3	-
3	Kranthikumar et.al., 2016	22	24	0	36	240	0.3	-
4	Aljorany et.al., 2014	17	20	0.1	40	270	0.3	0

Table A2: - Summary of Laboratory test for the selected four sites

Site NO	Clay Layer depth (m)	Soil Description	Natural Moisture Content (%)	Bulk Unit Weight (kN/m^3)	Dry unit Weight (kN/m^3)	Swelling pressure (kN/m^2)	Free Swell (%)	Atterberg Limit		
								LL (%)	PL (%)	PI (%)
1	9	Silty Clay	24.03	17.56	12.27	117.6	160	82.46	49.66	32.8
2	11.5	Silty Clay	25.05	17.68	12.26	208.5	205	109	54.6	54.4
3	9.5	Silty Clay	33.37	17.07	11.09	26.97	125	99.84	47.6	52.2
4	8.5	Silty Clay	37.91	18.44	11.59	54.8	160	98.7	46.7	52.3

Table A3: - Summary of Stiffness Parameters for Site 2 soil

Depth (m)	SPT N value	Su (kPa)	Es (kPa)	υ
1.00	10.00	100.00	42249.36	0.25
2.00		100.00		
3.00		100.00		
4.00		100.00		
5.00	13.00	130.00	54924.17	0.29
6.00		130.00		
7.00	15.00	150.00	63374.04	0.31
8.00		150.00		
9.00	20.00	200.00	84498.72	0.38
10.00		200.00		
11.00	24.00	240.00	101398.46	0.43
12.00		240.00		
12.50		240.00		

Table A4: - Summary of parameters for steel material

Parameters	Anchor Plate	Footing	Anchor rod
E(GPa)	200	200	200
N	0.26	0.26	0.26
Thickness (m)	0.025	0.06	-
Diameter (mm)	-	-	30
EA (MPa)	-	-	140

Table A5: - Summary of laboratory test data for Expansive soil found in the Bole area, Addis Ababa around African Union Commission Deputy Chairperson’s Residence.

No	Soil description	% pass sieve No-200	Specific Gravity (Gs)	Swelling Pressure (kPa)	Free swell (%)	Atterberg limit			Shear strength	
						LL	PL	PI	C (kN/m^2)	Φ (Deg.)
1	Silty Clay	98	2.48	95	145	82.46	49.66	32.8	42	13

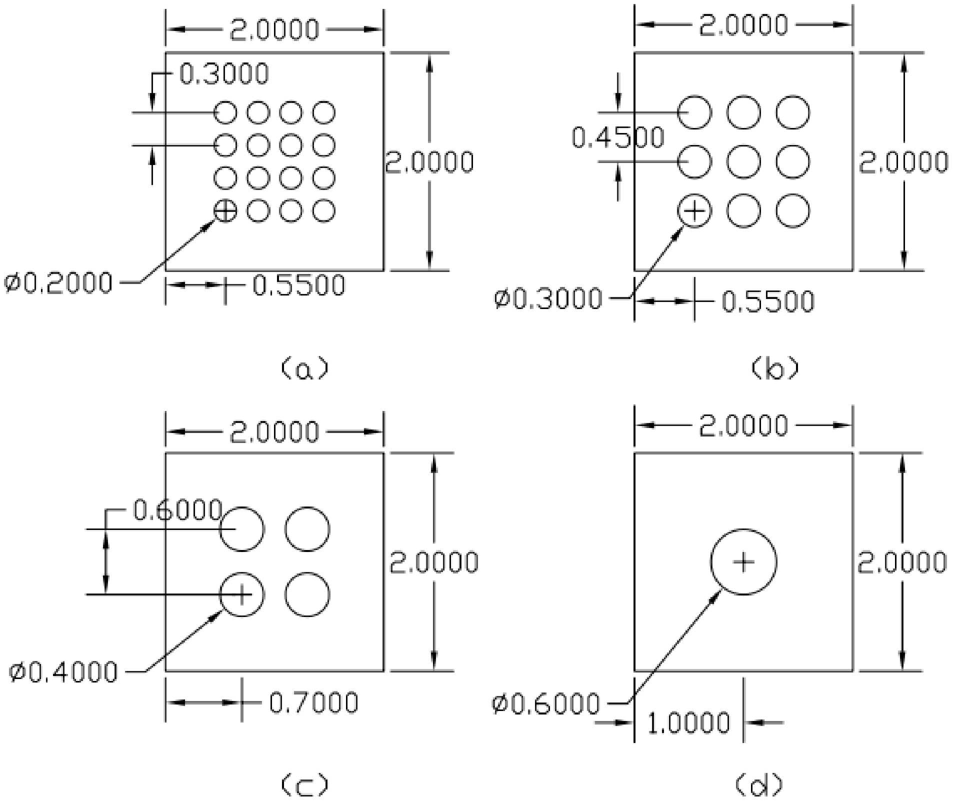


Figure A1: - S=1.5d and (a) d=0.2 m, (b) d=0.3 m, (c) d=0.4 m, (d) d=0.6 m

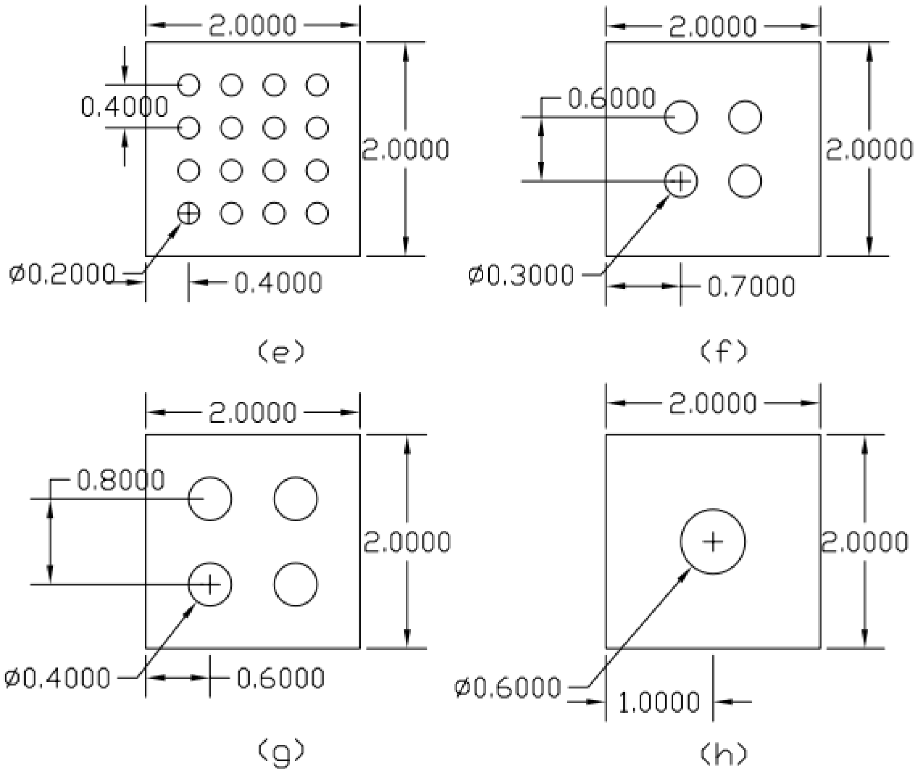


Figure A2: - S=2d and (e) d=0.2 m, (f) d=0.3 m, (g) d=0.4 m, (h) d=0.6 m

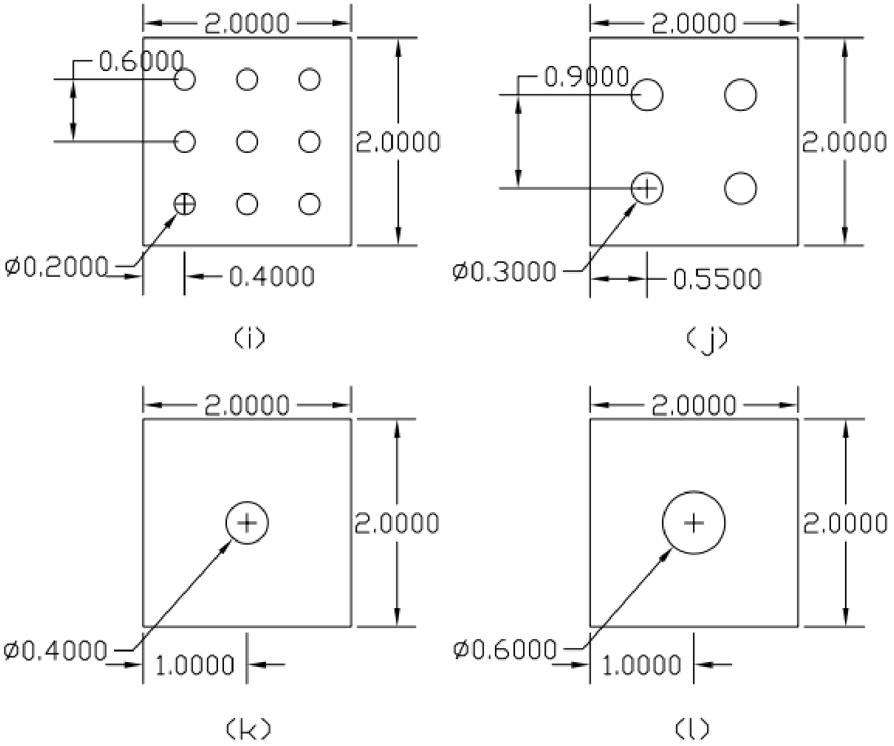
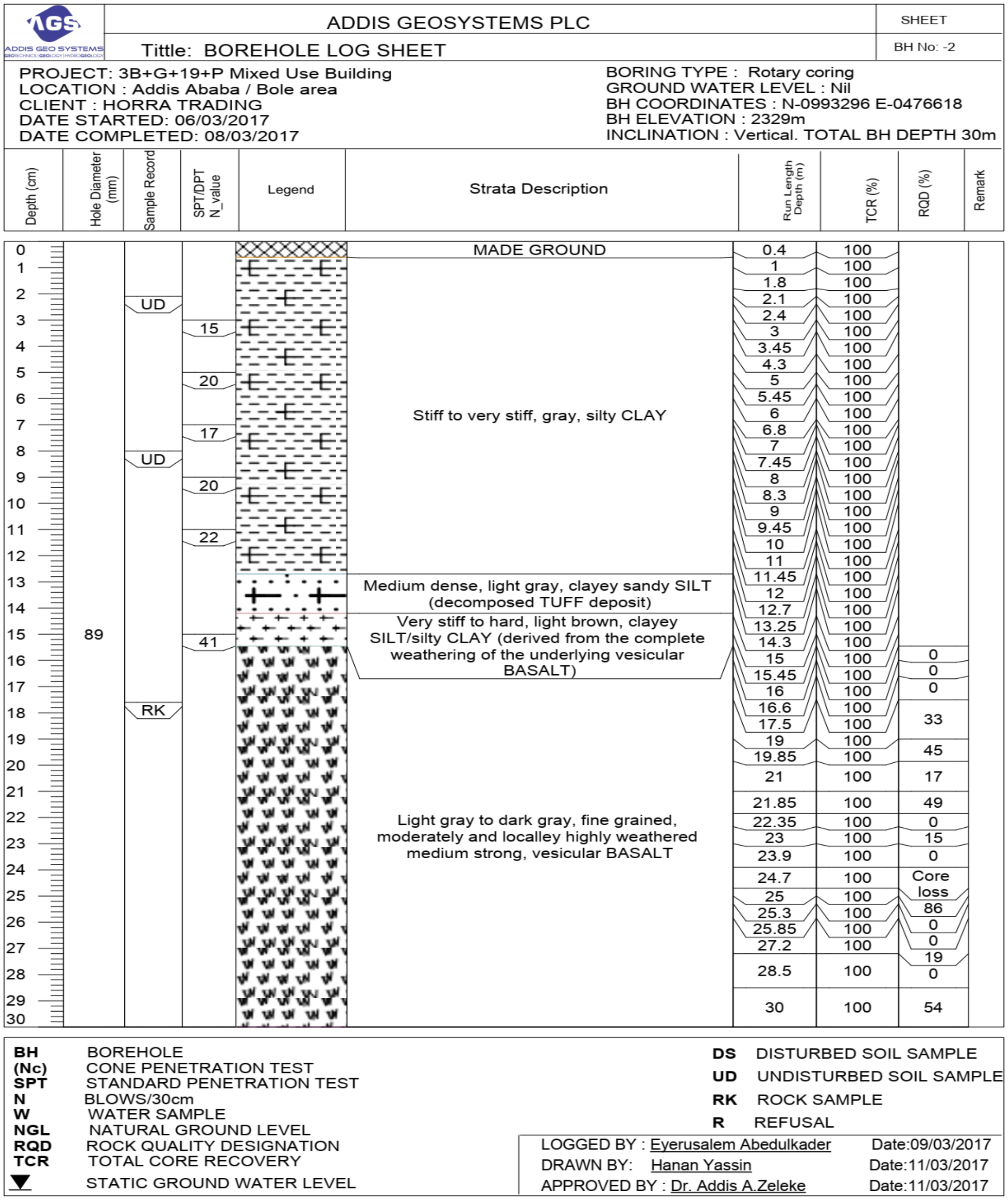


Figure A3: - S=3d and (i) d=0.2 m, (j) d=0.3 m, (k) d=0.4 m, (l) d=0.6 m



ADDIS GEOSYSTEMS PLC

SHEET

Tittle: BOREHOLE LOG SHEET

BH No: -2

PROJECT: 3B+G+19+P Mixed Use Building
LOCATION : Addis Ababa / Bole area
CLIENT : HORRA TRADING
DATE STARTED: 06/03/2017
DATE COMPLETED: 08/03/2017

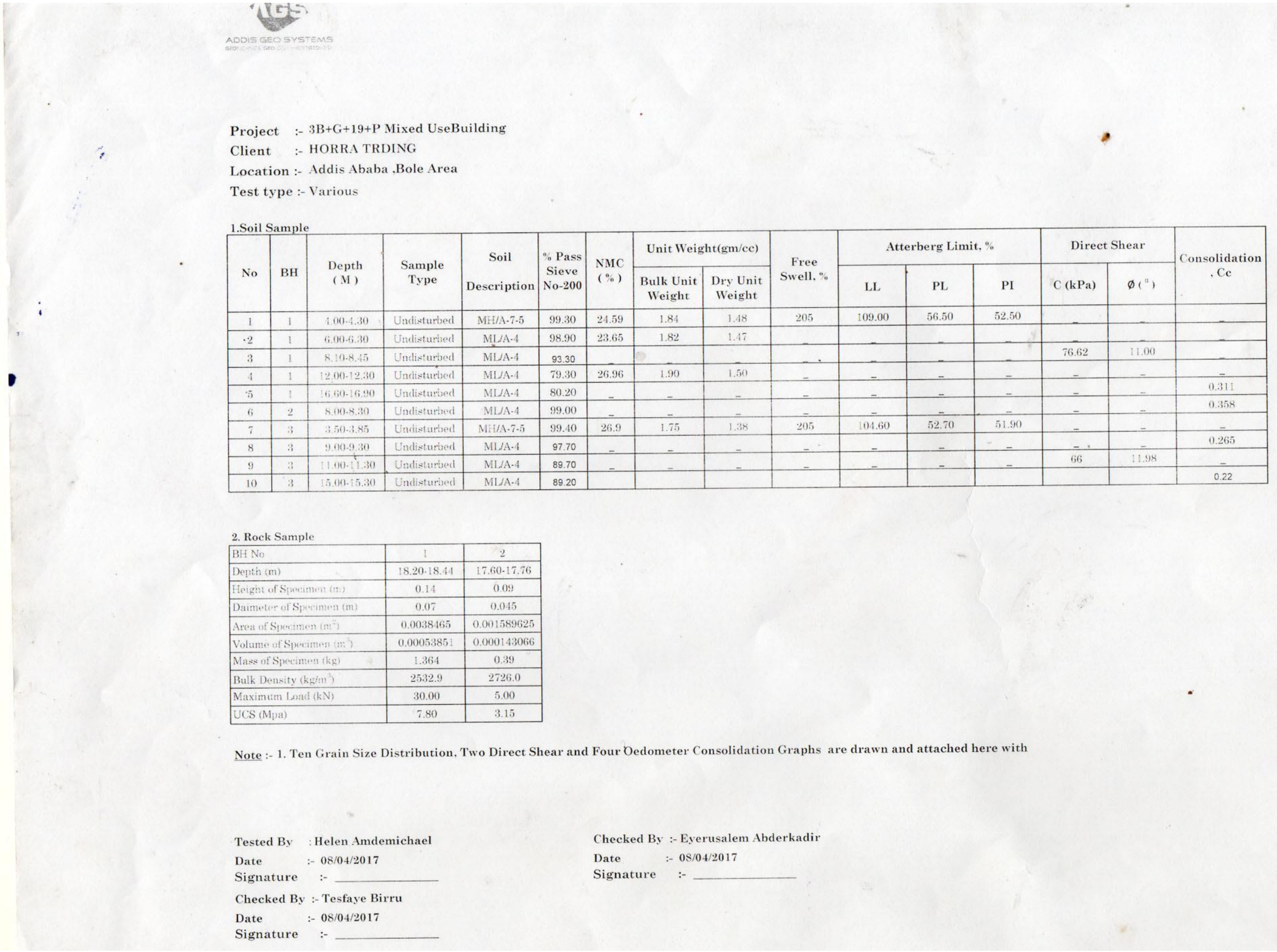
BORING TYPE : Rotary coring
GROUND WATER LEVEL : Nil
BH COORDINATES : N-0993296 E-0476618
BH ELEVATION : 2329m
INCLINATION : Vertical. TOTAL BH DEPTH 30m

Depth (cm)	Hole Diameter (mm)	Sample Record	SPT/DPT N_value	Legend	Strata Description
0					MADE GROUND
1					Stiff to very stiff, gray, silty CLAY
2		UD			
3			15		
4					
5			20		
6					
7			17		
8		UD			
9			20		
10					
11			22		
12					
13					Medium dense, light gray, clayey sandy SILT (decomposed TUFF deposit)
14					Very stiff to hard, light brown, clayey SILT/silty CLAY (derived from the complete weathering of the underlying vesicular BASALT)
15	89		41		
16					
17					Light gray to dark gray, fine grained, moderately and localley highly weathered medium strong, vesicular BASALT
18		RK			
19					
20					
21					
22					
23					
24					
25					
26					
27					
28					
29					
30					

Run Length Depth (m)	TCR (%)	RQD (%)	Remark
0.4	100		
1	100		
1.8	100		
2.1	100		
2.4	100		
3	100		
3.45	100		
4.3	100		
5	100		
5.45	100		
6	100		
6.8	100		
7	100		
7.45	100		
8	100		
8.3	100		
9	100		
9.45	100		
10	100		
11	100		
11.45	100		
12	100		
12.7	100		
13.25	100		
14.3	100		
15	100	0	
15.45	100	0	
16	100	0	
16.6	100	33	
17.5	100		
19	100	45	
19.85	100		
21	100	17	
21.85	100	49	
22.35	100	0	
23	100	15	
23.9	100	0	
24.7	100	Core loss	
25	100		
25.3	100	86	
25.85	100	0	
27.2	100	0	
28.5	100	19 0	
30	100	54	

BH	BOREHOLE
(Nc)	CONE PENETRATION TEST
SPT	STANDARD PENETRATION TEST
N	BLOWS/30cm
W	WATER SAMPLE
NGL	NATURAL GROUND LEVEL
RQD	ROCK QUALITY DESIGNATION
TCR	TOTAL CORE RECOVERY
▼	STATIC GROUND WATER LEVEL
DS	DISTURBED SOIL SAMPLE
UD	UNDISTURBED SOIL SAMPLE
RK	ROCK SAMPLE
R	REFUSAL

LOGGED BY : Eyerusalem Abedulkader Date:09/03/2017
DRAWN BY: Hanan Yassin Date:11/03/2017
APPROVED BY : Dr. Addis A.Zeleke Date:11/03/2017



ADDIS GEO SYSTEMS

Project :- 3B+G+19+P Mixed UseBuilding
Client :- HORRA TRDING
Location :- Addis Ababa ,Bole Area
Test type :- Various

1.Soil Sample

No	BH	Depth (M)	Sample Type	Soil Description	% Pass Sieve No-200	NMC (%)	Unit Weight(gm/cc)		Free Swell, %	Atterberg Limit, %			Direct Shear		Consolidation , Cc
							Bulk Unit Weight	Dry Unit Weight		LL	PL	PI	C (kPa)	Ø (°)	
1	1	4.00-4.30	Undisturbed	MH/A-7-5	99.30	24.59	1.84	1.48	205	109.00	56.50	52.50	_	_	_
2	1	6.00-6.30	Undisturbed	ML/A-4	98.90	23.65	1.82	1.47	_	_	_	_	_	_	_
3	1	8.10-8.45	Undisturbed	ML/A-4	93.30	_	_	_	_	_	_	_	76.62	11.00	_
4	1	12.00-12.30	Undisturbed	ML/A-4	79.30	26.96	1.90	1.50	_	_	_	_	_	_	_
5	1	16.60-16.90	Undisturbed	ML/A-4	80.20	_	_	_	_	_	_	_	_	_	0.311
6	2	8.00-8.30	Undisturbed	ML/A-4	99.00	_	_	_	_	_	_	_	_	_	0.358
7	3	3.50-3.85	Undisturbed	MH/A-7-5	99.40	26.9	1.75	1.38	205	104.60	52.70	51.90	_	_	_
8	3	9.00-9.30	Undisturbed	ML/A-4	97.70	_	_	_	_	_	_	_	_	_	0.265
9	3	11.00-11.30	Undisturbed	ML/A-4	89.70	_	_	_	_	_	_	_	66	11.98	_
10	3	15.00-15.30	Undisturbed	ML/A-4	89.20										0.22

2. Rock Sample

BH No	1	2
Depth (m)	18.20-18.44	17.60-17.76
Height of Specimen (m)	0.14	0.09
Daimeter of Specimen (m)	0.07	0.045
Area of Specimen (m²)	0.0038465	0.001589625
Volume of Specimen (m³)	0.00053851	0.000143066
Mass of Specimen (kg)	1.364	0.39
Bulk Density (kg/m³)	2532.9	2726.0
Maximum Load (kN)	30.00	5.00
UCS (Mpa)	7.80	3.15

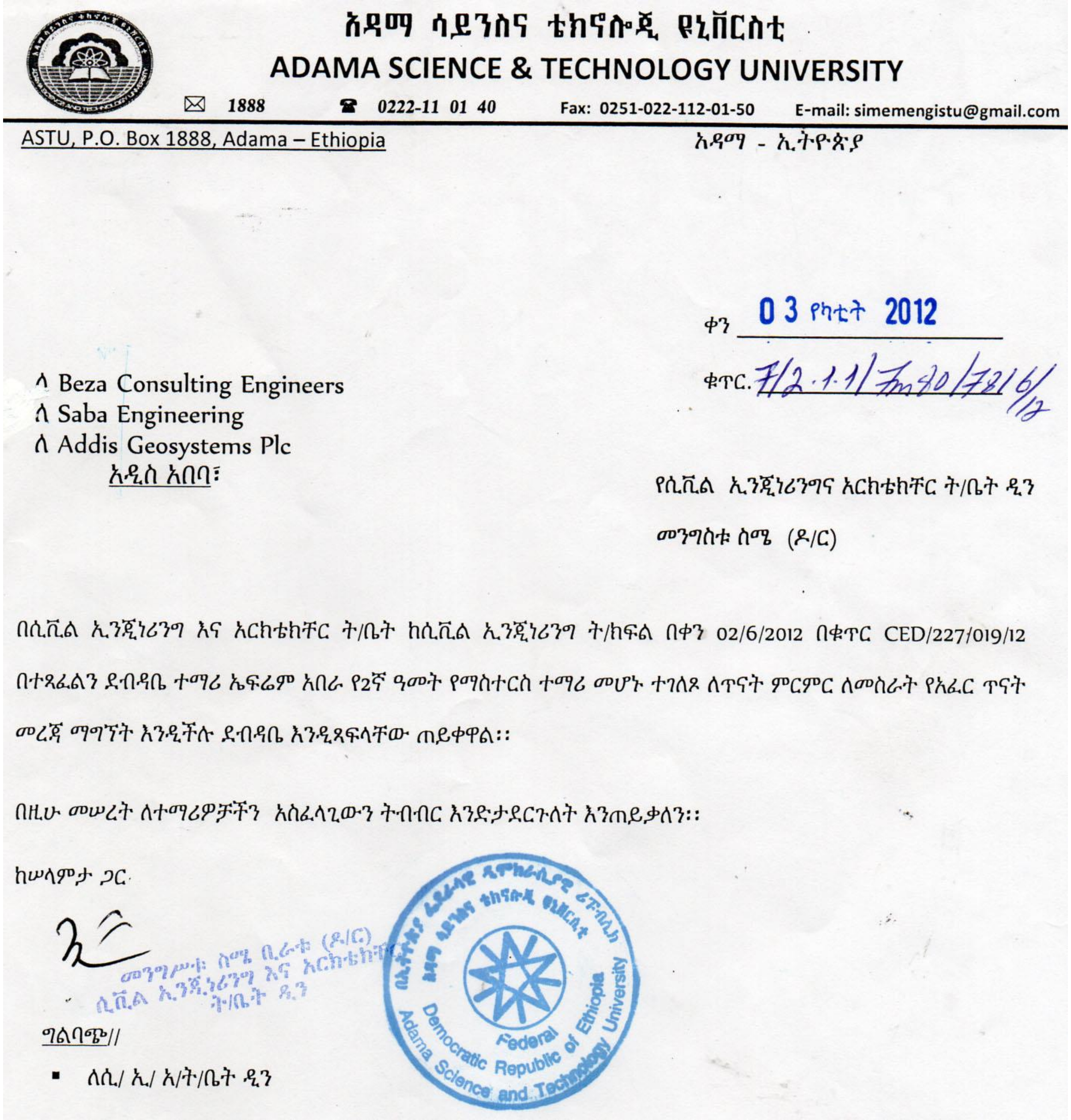
Note :- 1. Ten Grain Size Distribution, Two Direct Shear and Four Oedometer Consolidation Graphs are drawn and attached here with

Tested By : Helen Amdemichael
Date :- 08/04/2017
Signature :- _______________

Checked By :- Tesfaye Birru
Date :- 08/04/2017
Signature :- _______________

Checked By :- Eyerusalem Abderkadir
Date :- 08/04/2017
Signature :- _______________

Figure A4: - Representative borehole log sheet for site 2 and corresponding Soil Property



አዳማ ሳይንስና ቴክኖሎጂ ዩኒቨርስቲ
ADAMA SCIENCE & TECHNOLOGY UNIVERSITY
✉ 1888 ☎ 0222-11 01 40 Fax: 0251-022-112-01-50 E-mail: simemengistu@gmail.com
ASTU, P.O. Box 1888, Adama – Ethiopia
አዳማ - ኢትዮጵያ

ቀን 03 የካቲት 2012
ቁጥር. ቻ/2.1.1/ከn80/781/6/12

ለ Beza Consulting Engineers
ለ Saba Engineering
ለ Addis Geosystems Plc
አዲስ አበባ፣

የሲቪል ኢንጂነሪንግና አርክቴክቸር ት/ቤት ዲን
መንግስቱ ስሜ (ዶ/ር)

በሲቪል ኢንጂነሪንግ እና አርክቴክቸር ት/ቤት ከሲቪል ኢንጂነሪንግ ት/ክፍል በቀን 02/6/2012 በቁጥር CED/227/019/12 በተጻፈልን ደብዳቤ ተማሪ ኤፍሬም አበራ የ2ኛ ዓመት የማስተርስ ተማሪ መሆኑ ተገለጾ ለጥናት ምርምር ለመስራት የአፈር ጥናት መረጃ ማግኘት እንዲችሉ ደብዳቤ እንዲጻፍላቸው ጠይቀዋል፡፡

በዚሁ መሠረት ለተማሪዎቻችን አስፈላጊውን ትብብር እንድታደርጉለት እንጠይቃለን፡፡

ከሠላምታ ጋር

መንግሥቱ ስሜ ቢራቱ (ዶ/ር)
ሲቪል ኢንጂነሪንግ እና አርክቴክቸር
ት/ቤት ዲን

<u>ግልባጭ</u>//
- ለሲ/ ኢ/ አ/ት/ቤት ዲን

Figure A5: - Supporting letter from Adama Science and Technology University

Appendix B

Hardening model Stiffness Parameters of Granular fill for the base model

- **Secant and unloading-reloading moduli**

In general, the relation between the stiffness moduli is as follows:

$$E_{50} < Es < E_{ur} \tag{B1}$$

where Es denotes "static" modulus or secant modulus is taken from the initial part of the "ε_1-q the experimental curve at "ε_1 = 0.1% or can be represented with:

$$Es = \frac{q}{0.001} = \frac{1}{\frac{1}{2E50} + \frac{0.001*Rf}{qf(\phi,c1)}} \tag{B2}$$

In the case when E_{50} or Eur cannot be directly determined from experimental curves, it may be relevant for many practical cases to set:

$$\frac{E^{ref}ur}{E^{ref}50} = 2 \text{ to } 6 \text{ (an average can be assumed equal to 3)} \tag{B3}$$

However, note that the following condition should be satisfied:

$$\frac{E^{ref}ur}{E^{ref}50} > 2 \tag{B4}$$

Equation B2 can be represented graphically in Figure B1 and can be used to estimate E_{50} from the known value of *Es*.

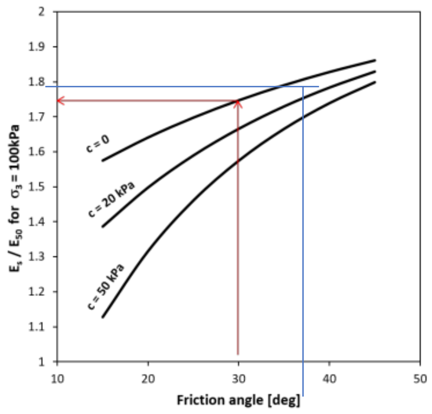


Figure B1: - Estimating the ratio between the static modulus Es and secant modulus E_{50}

From the above figure for Friction angle of ϕ = 36° and Cohesion c = 0 the ratio of $Es/E_{50} \approx 1.81$

Since we have Es = 240 MPa = $240*10^3$ kPa from table A1, Therefore $E_{50} = \frac{Es}{1.81}$,

$E_{50} = \frac{240*10^3}{1.81}$ = $133*10^3$ kPa

But $E^{ref}50 = E_{50}\ (\frac{ccos\phi + \sigma3sin\phi}{ccos\phi + prefsin\phi})^{-m}$

Where: - c = 0, ϕ = 36^0, P^{ref}: reference confining pressure (PLAXIS default 100 kPa) σ3 = 100 kPa (from above figure) and stiffness exponent (m) = 0.5 for gravel and sand.

Therefore: - $E^{ref}50 \approx E_{50} = 133*10^3$ kPa

Once we know $E^{ref}50$, therefore $E^{ref}ur = 3*E^{ref}50 = 3*133*10^3 = 399*10^3$ kPa

Oedometric modulus

In case of lack of oedometric test data for granular material the oedometric modulus can approximately be taken as:

$E^{ref}oed \cong E^{ref}50$ (B5)

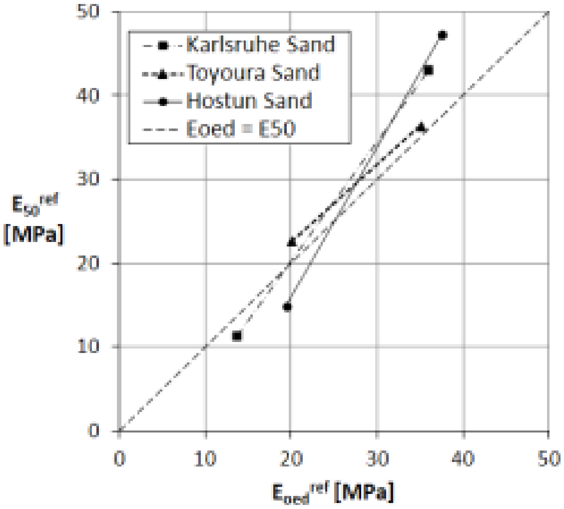


Figure 2.3. Comparison of reference stiffness moduli for sands from oedometer and triaxial tests (Schanz and Vermeer, 1998)

Therefore $E^{ref}oed \cong E^{ref}50 = 133*10^3$ kPa

Hardening model Stiffness Parameters of Expansive soil for the base model

- **Secant and unloading-reloading moduli**

In the case when one of the stiffness moduli cannot be directly determined, it may be relevant for many practical cases to set:

$\frac{E^{ref}ur}{E^{ref}50}$ = 3 to 6 (an average can be assumed equal to 4) (B6)

However, note that the following condition should be satisfied:

$\frac{E^{ref}ur}{E^{ref}50} > 2$ (B7)

The secant modulus E_{50} can be approximated based on the known value of the "static" modulus *Es* and according to the approach described in Figure below.

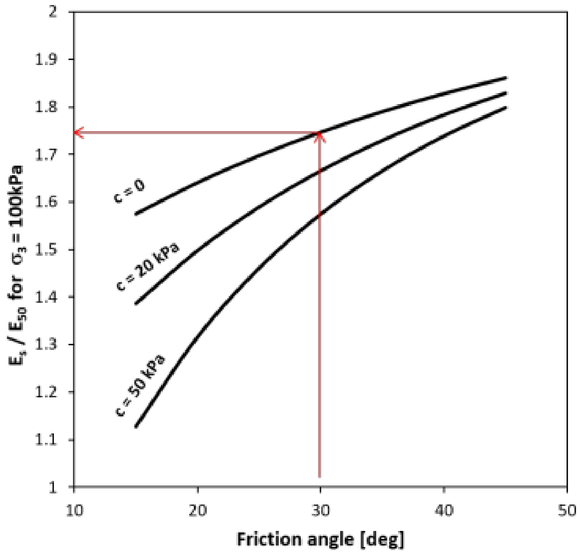


Figure B2: - Estimating the ratio between the static modulus Es and secant modulus E_{50}

From the above figure for Friction angle of $\phi = 11^o$ and Cohesion c = 76.62 kN/m^2 the minimum ratio is $Es/E_{50} \approx 1.11$

But $E^{ref}50 = E_{50} \left(\frac{c\cos\phi + \sigma 3 \sin\phi}{c\cos\phi + pref\sin\phi}\right)^{-m}$

Where: - c = 76.62, $\phi = 11^0$, P^{ref}: reference confining pressure (PLAXIS default 100 kPa) $\sigma3 = 100$ kPa (from above figure) and stiffness exponent (m) = 1. (high plastic soil)

Therefore: - $E^{ref}_{50} \approx E_{50}$

Once we know $E^{ref}50$, therefore $E^{ref}ur = 4*E^{ref}50$

Oedometric modulus

In case of lack of oedometric test data for granular material the oedometric modulus can approximately be taken as:

$E^{ref}oed \cong E^{ref}50$ (B8)

Since there is five young modulus value for clay soil through the depth, therefore the result is summarized in the table below as shown.

Table B1: - Summary of hardening model Stiffness parameters

Depth (m)	Es (MPa)	$E^{ref}50$ (MPa)	$E^{ref}ur$ (MPa)	$E^{ref}oed$ (MPa)
1 – 4	42.25	38.06	152.24	38.06
4 – 6	54.93	49.48	197.92	49.48
6 – 8	63.37	57.09	228.36	57.09
8 – 10	84.49	76.12	304.48	76.12
10 – 12.5	101.39	91.34	365.36	91.34

Unloading-reloading Poisson's ratio

Experimental measurements from local strain gauges show that the initial values of Poisson's ratio in terms of small mobilized stress levels (q/qmax) vary between 0.1 and 0.2 for clays, sands, and rocks. Therefore, the characteristic value for the elastic unloading/reloading Poisson's ratio of $\nu_{ur} = 0.2$ can be adopted for most soils.

Dilatancy Angle

Bolton, 1986 suggested reasonable approximation of ψ for well-compacted granular soil using the following equation: -

$\Psi = \phi - 30^0$ for the value of friction angle larger than 30^0

Therefore, for the granular fill, the dilatancy angle $\psi = 36 – 30 = 6^0$

An estimation of for clays relies on geotechnical experience. For instance, it can be observed in laboratory tests that normally consolidated clays may exhibit no dilatancy at all. Therefore, it is proposed that for cohesive soils, it can be assumed that dilatancy depends on the pre-consolidation state and can be approximately taken as (Truty, et.al, 2011)

- ➢ $\psi = 0^0$ for normally- and lightly-overconsolidated soil
- ➢ $\psi = \phi/6$ for overconsolidated soil
- ➢ $\psi = \phi/3$ for heavily overconsolidated soil

And Bowels, 1996 suggested the following tables for the determination of the over-consolidation ratio of soil.

Table B2: - Relation between Different water content and Degree of consolidation (Bowels, 1996)

If ω_n is close to ω_L	Soil is Normally Consolidated
If ω_n is between ωp and ω_L	Soil is lightly Consolidated
If ω_n is close to ω_p	Soil is lightly to heavily Consolidated
If ω_n is larger than ω_L	Soil is on verge of being a viscous liquid

$\psi = \phi/6 = 11/6 = 1.8^0$

Saturated unit weight of clay soil

$\gamma_{sat} = (\frac{Gs+e}{1+e})\gamma_w = (\frac{2.7+0.87}{1+0.87})*9.81 = 18.73\ kN/m^3$

Appendix C

Effect of diameter in the uplift capacity of GAP system

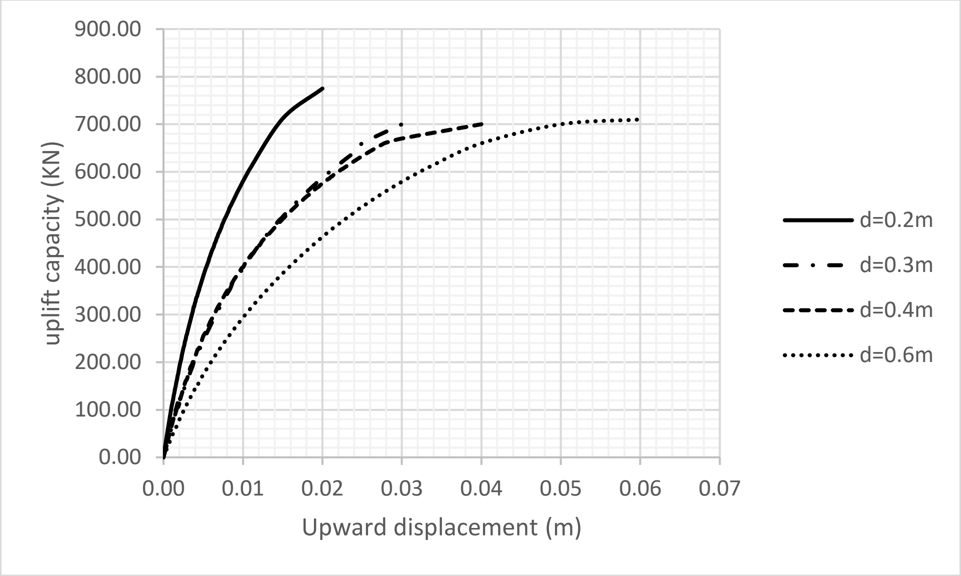


Figure C1: - Upward movement versus uplift force of GAP for L = 1 m and s=2d

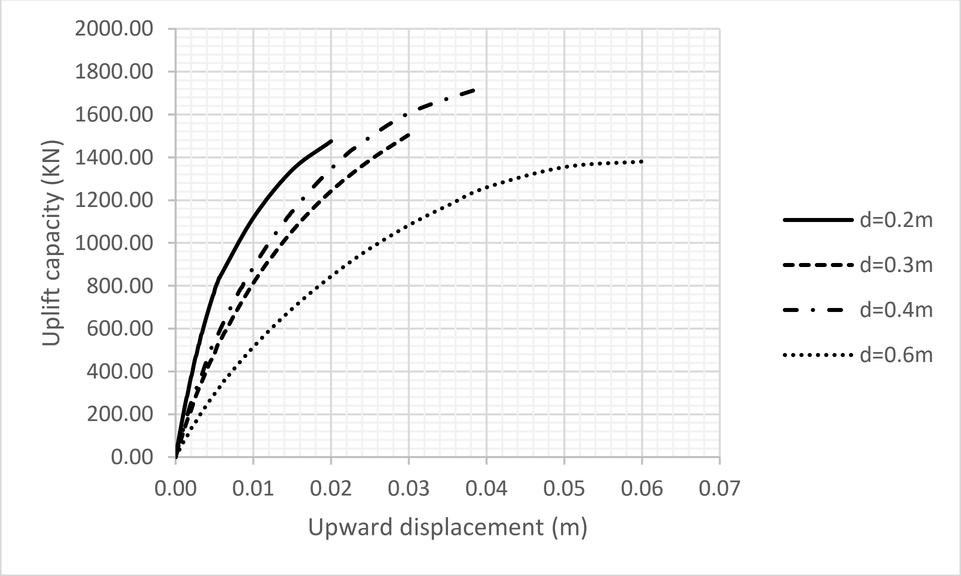


Figure C2: - Upward movement versus uplift force of GAP for L = 3 m and s=2d

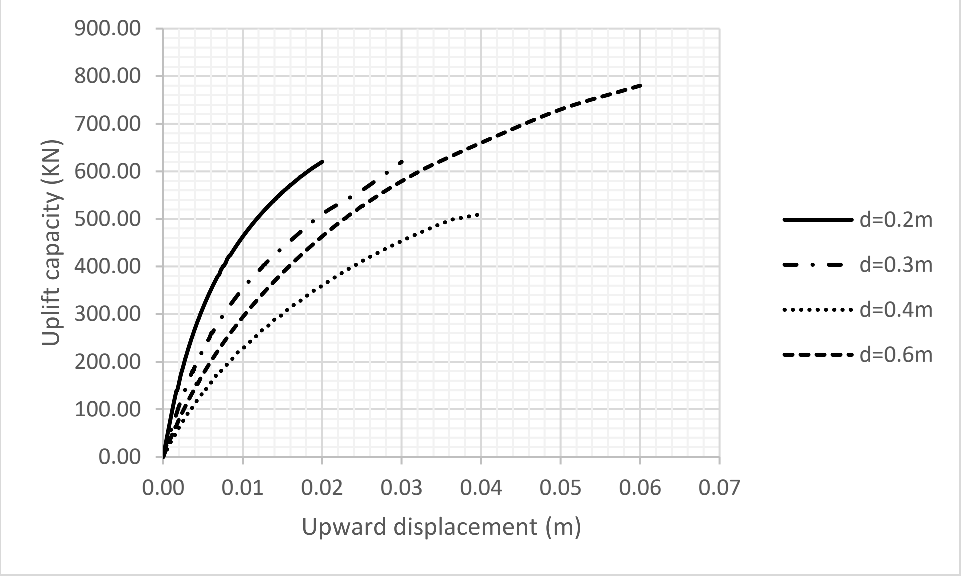


Figure C3: - Upward movement versus uplift force of GAP for L = 1 m and s=3d

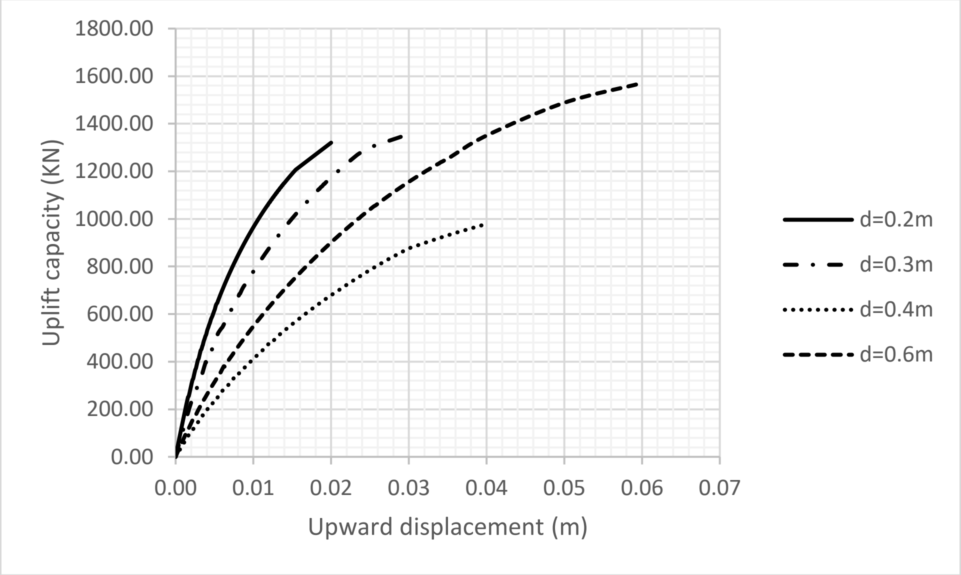


Figure C4: - Upward movement versus uplift force of GAP for L = 3 m and s=3d

Table C1: - Plan area (A), circumference (c), center to center spacing (c/c) and number of piles for different spacing and diameter of group of pile

Spacing (s) and diameter (d)	A (m^2)	C (m)	c/c (m)	No of pile
s=1.5d, d=0.2 m	0.502	10.05	0.3	16
s=1.5d, d=0.3 m	0.64	8.48	0.45	9
s=1.5d, d=0.4 m	0.502	5.028	0.6	4
s=1.5d, d=0.6 m	0.283	1.884	-	1
s=2d, d=0.2 m	0.502	10.013	0.4	16
s=2d, d=0.3 m	0.283	3.76	0.6	4
s=2d, d=0.4 m	0.502	5.026	0.8	4
s=2d, d=0.6 m	0.283	1.884	-	1
s=3d, d=0.2 m	0.283	5.65	0.6	9
s=3d, d=0.3 m	0.283	3.769	0.9	4
s=3d, d=0.4 m	0.125	1.256	-	1
s=3d, d=0.6 m	0.283	1.884	-	1

Effect of length in the uplift capacity of GAP system

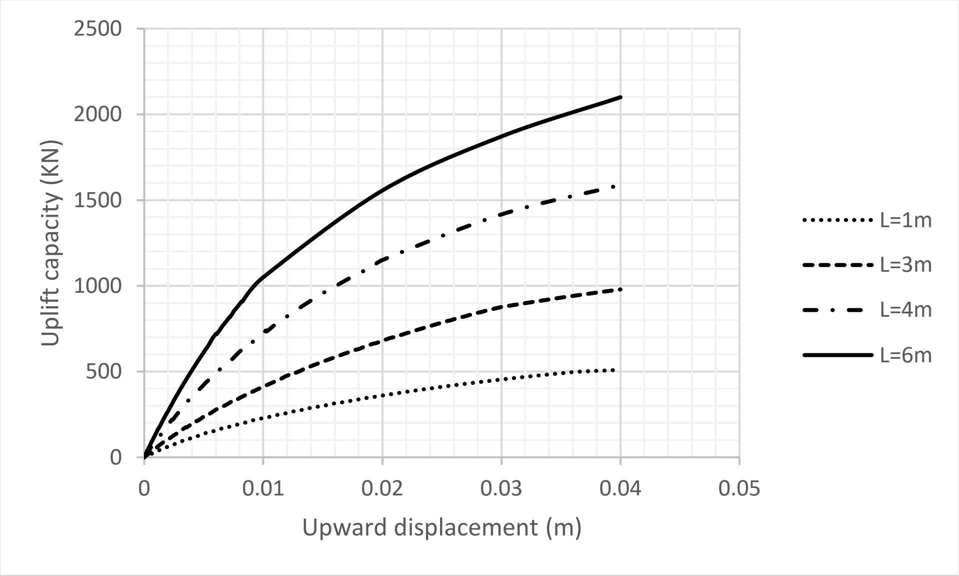


Figure C5: - Upward movement versus uplift force of GAP for S=3d, d=0.4 m

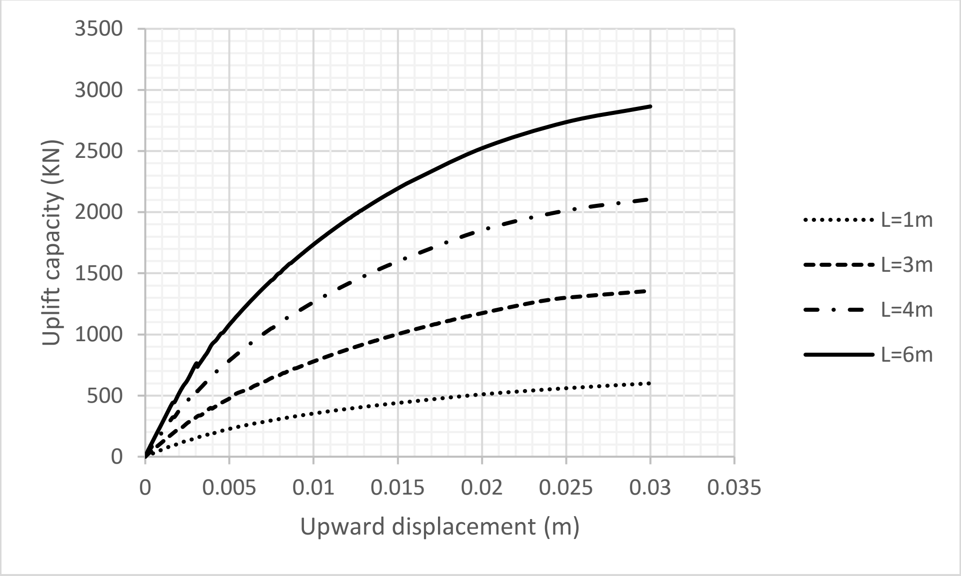


Figure C6: - Upward movement versus uplift force of GAP for S=3d, d=0.3 m

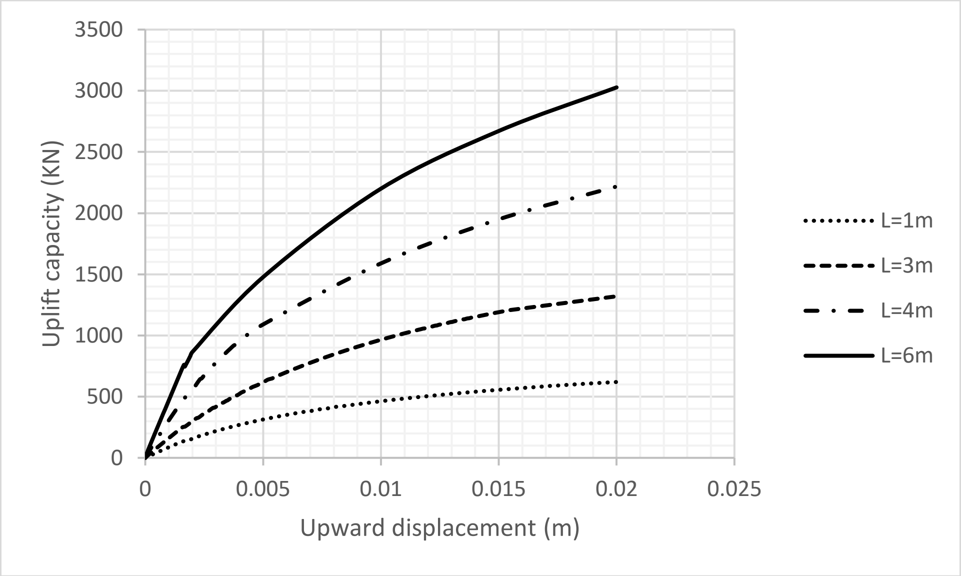


Figure C7: - Upward movement versus uplift force of GAP for S=3d, d=0.2 m

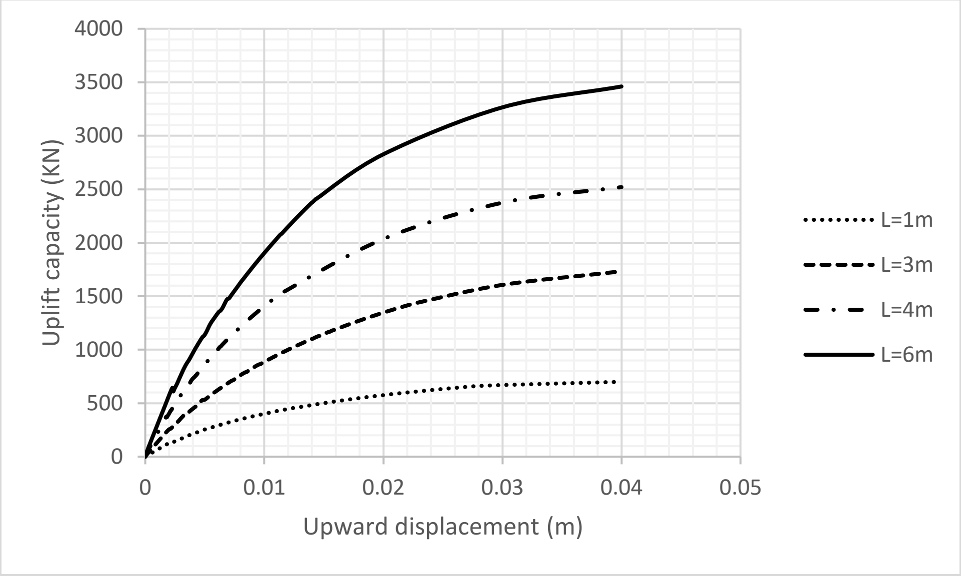


Figure C8: - Upward movement versus uplift force of GAP for S=2d, d=0.4 m

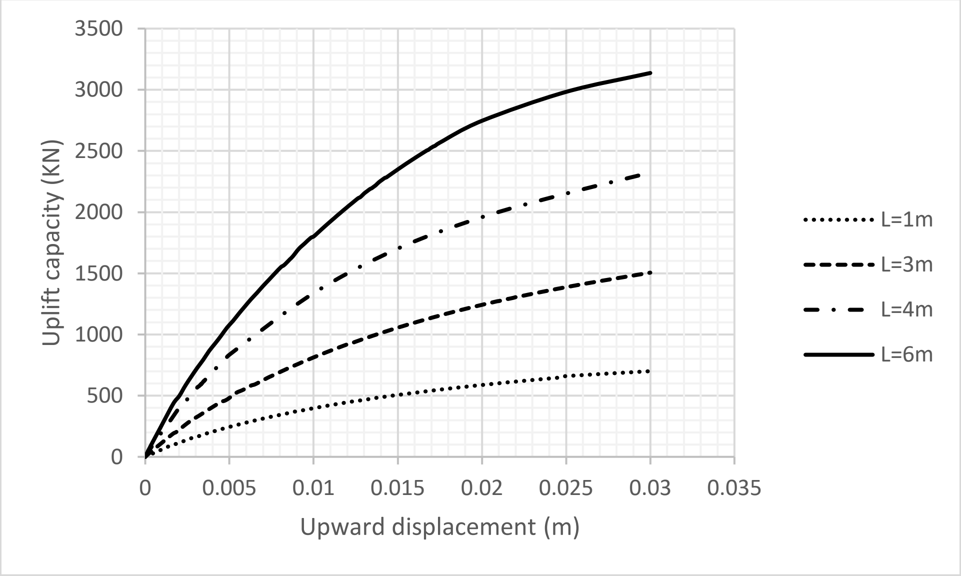


Figure C9: - Upward movement versus uplift force of GAP for S=2d, d=0.3 m

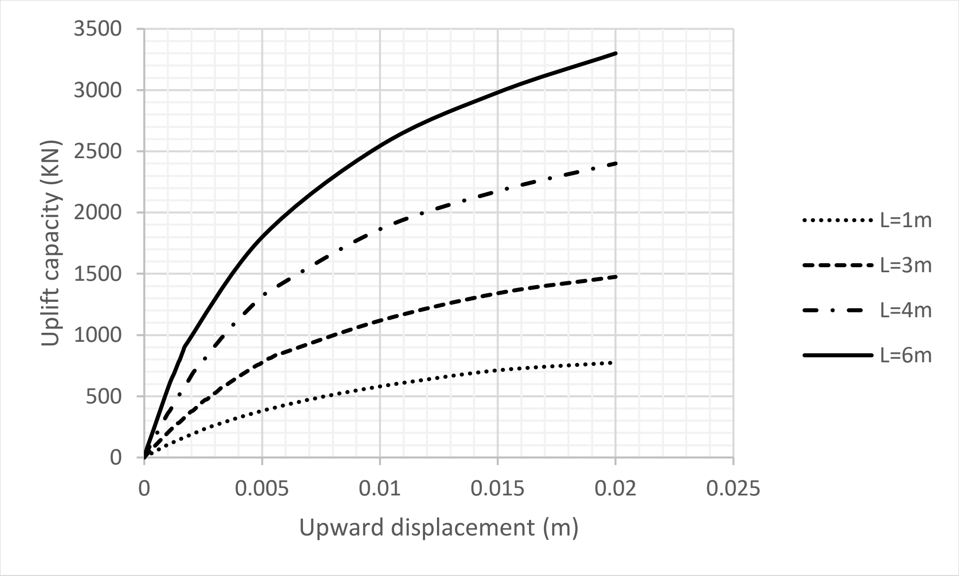


Figure C10: - Upward movement versus uplift force of GAP for S=2d, d=0.2 m

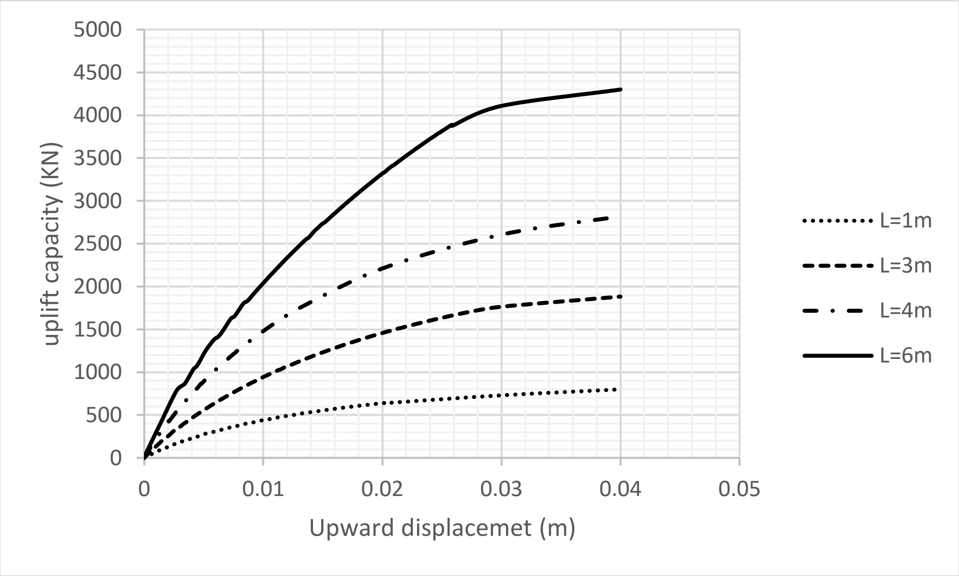


Figure C11: - Upward movement versus uplift force of GAP for S=1.5d, d=0.4 m

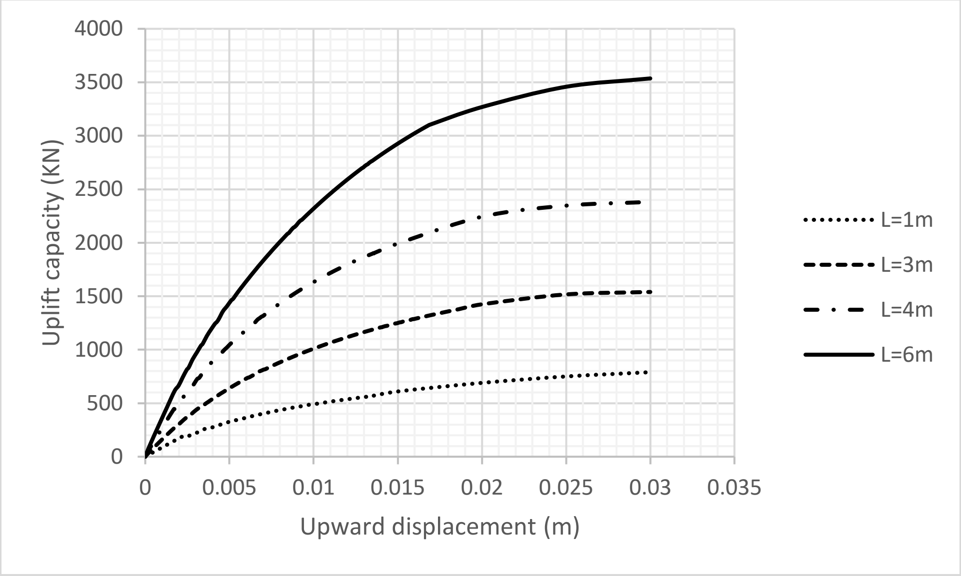


Figure C12: - Upward movement versus uplift force of GAP for S=1.5d, d=0.3 m

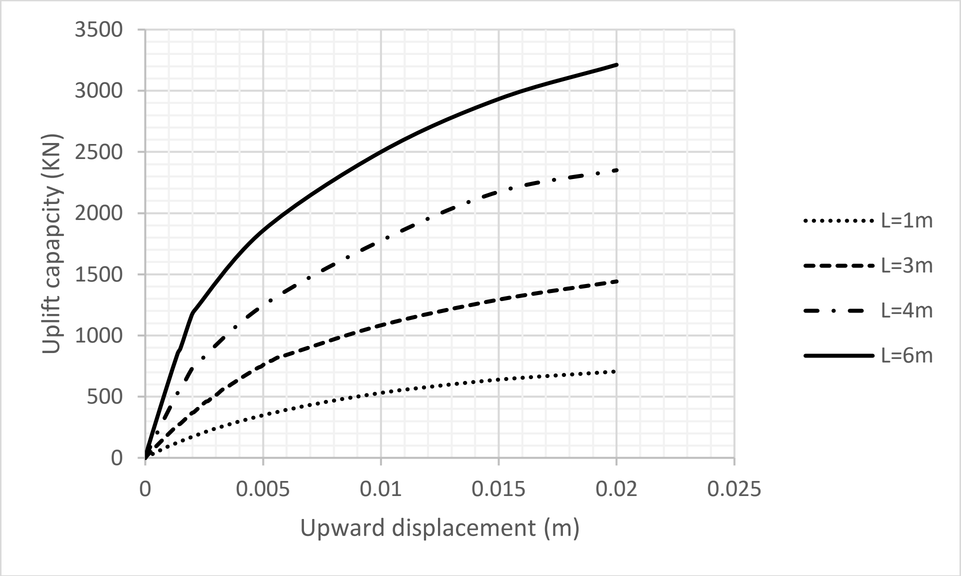


Figure C13: - Upward movement versus uplift force of GAP for S=1.5d, d=0.2 m

Effect of pile spacing in the uplift capacity of GAP system

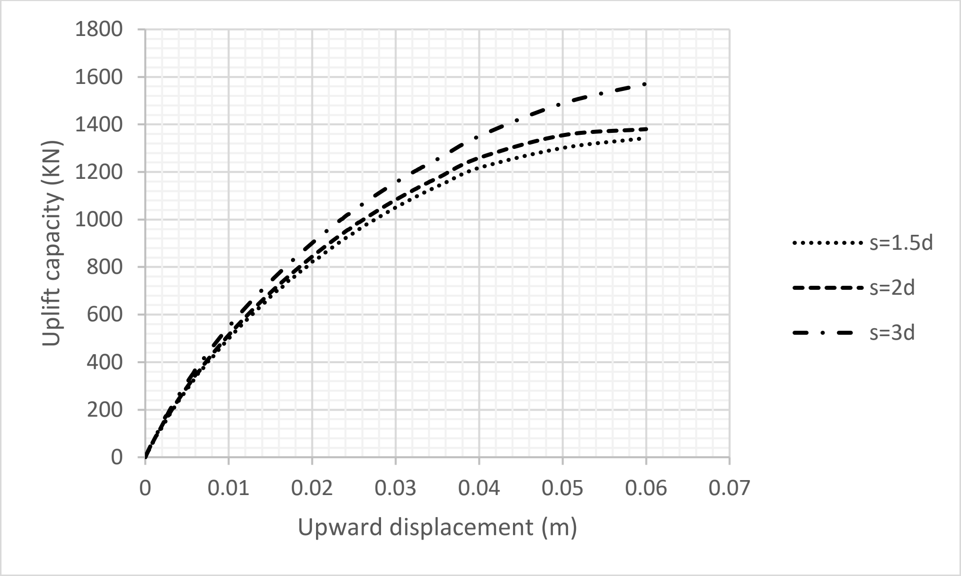


Figure C14: - Upward movement versus uplift force of GAP for d=0.6 m, L= 3 m

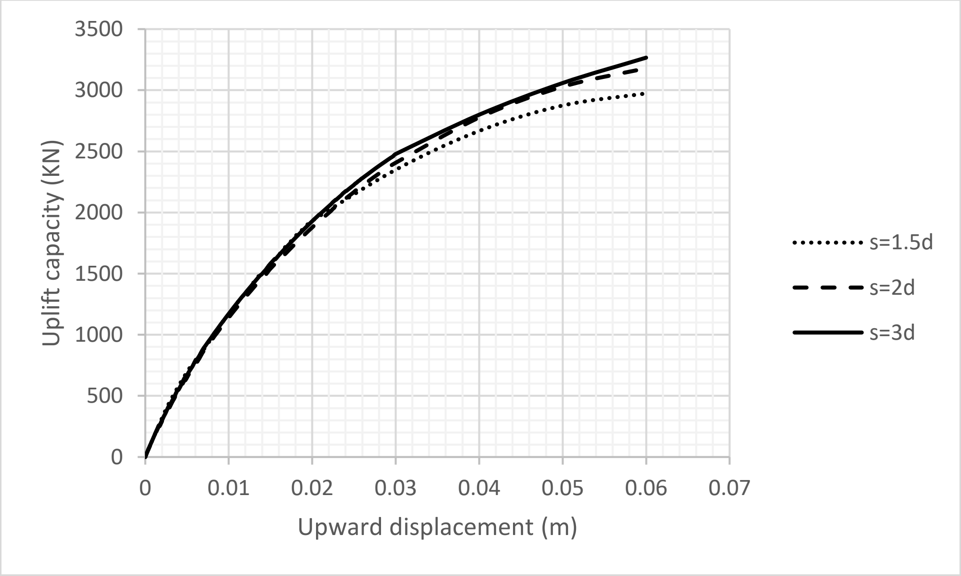


Figure C15: - Upward movement versus uplift force of GAP for d=0.6 m, L=6 m

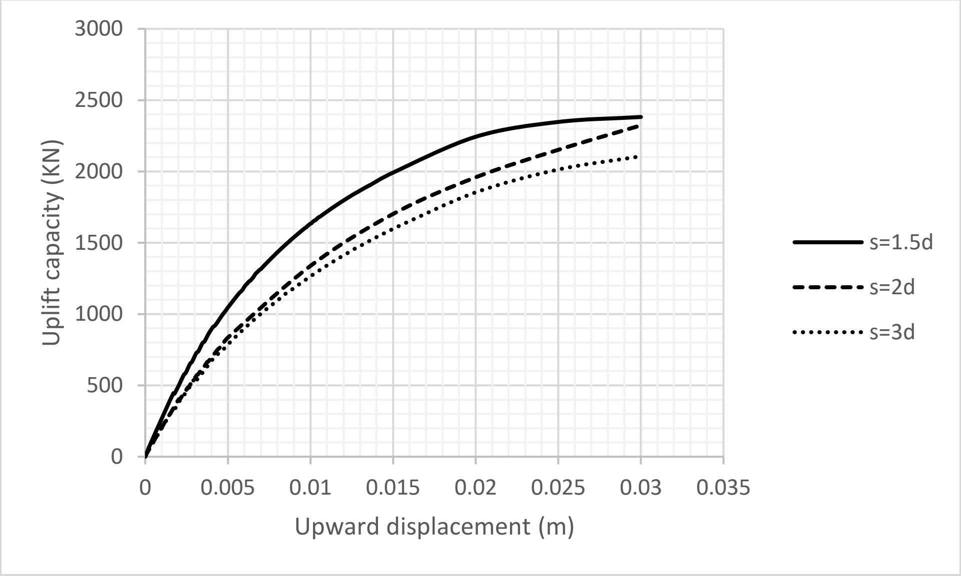


Figure C16: - Upward movement versus uplift force of GAP for d=0.3 m, L=4 m

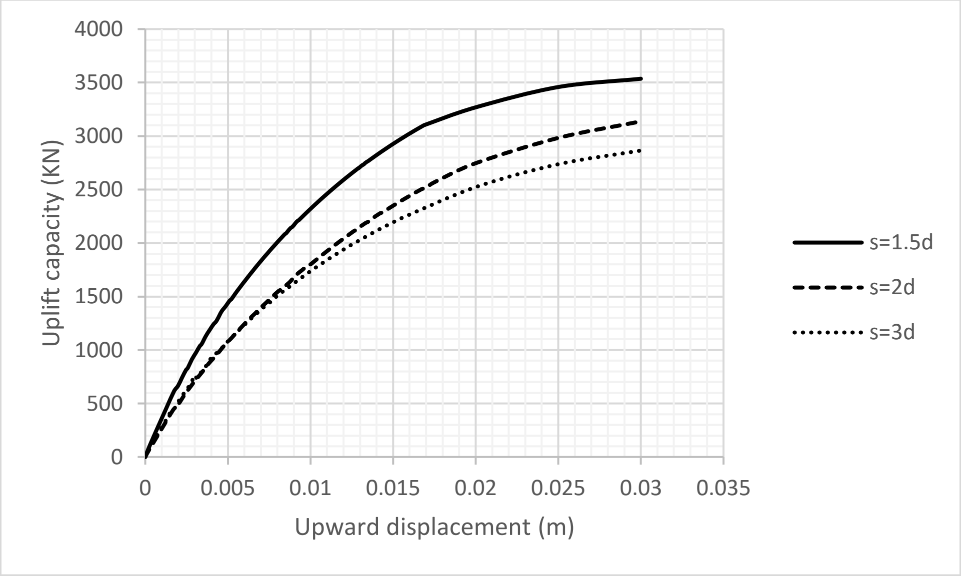


Figure C17: - Upward movement versus uplift force of GAP for d=0.3 m, L=6 m

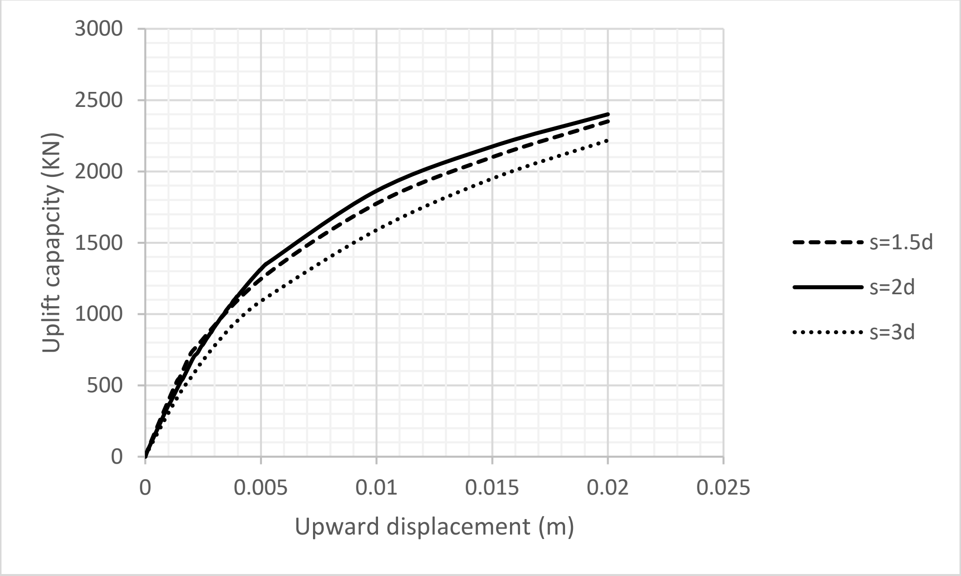


Figure C18: - Upward movement versus uplift force of GAP for d=0.2 m, L=4 m

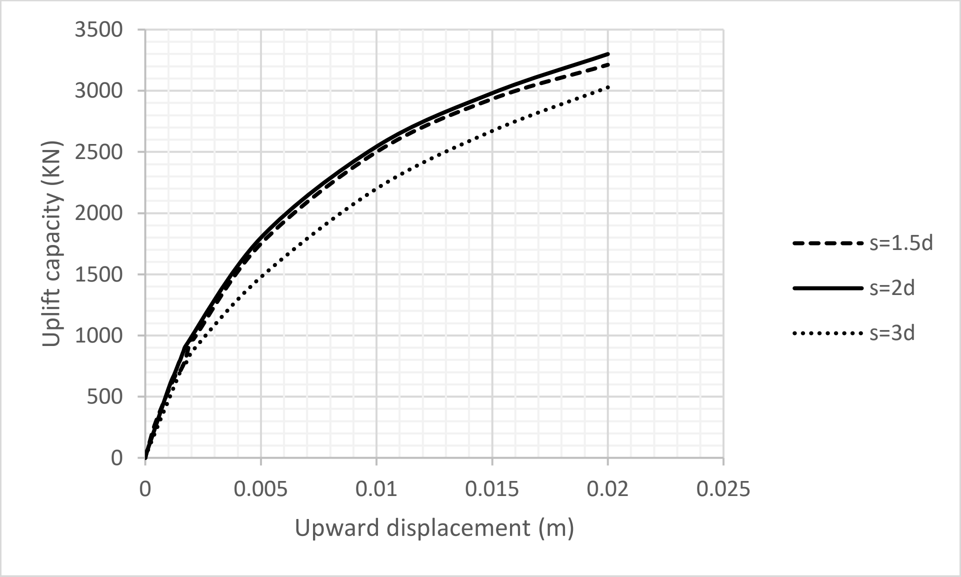


Figure C19: - Upward movement versus uplift force of GAP for d=0.2 m, L=6 m

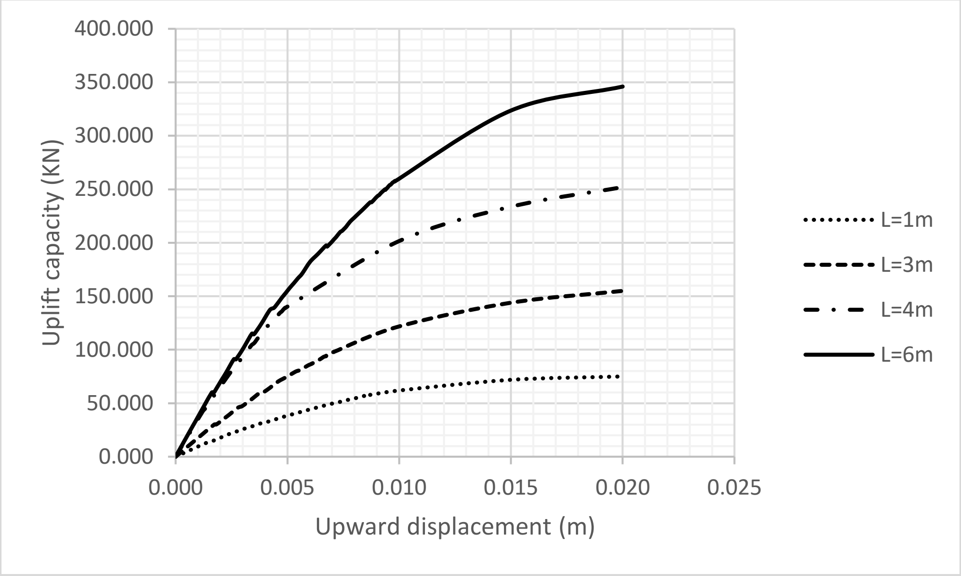


Figure C13: - Upward movement versus uplift force of single GAP for d=0.2 m

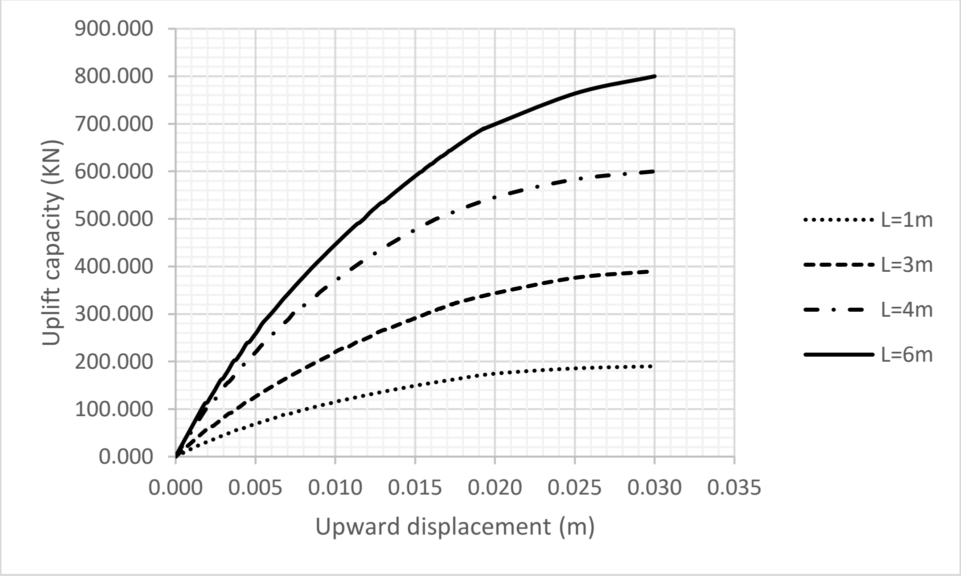


Figure C13: - Upward movement versus uplift force of single GAP for d=0.3 m

Appendix D

Appendix D1: - **Calculation of Shallow Isolated footing Design**

σmax < qall

For Minimum load (p =155 kN and Mx = -2 kNm and My = -1 kNm)

$$\frac{155}{B^2} * [1 + \frac{6*(0.013)}{B} + \frac{6*(0.0064)}{B}] = \frac{(1.3*164*5.71)+(17.68*2*1)}{3}$$

$$\frac{155}{B^2} + \frac{12.09}{B^3} + \frac{5.952}{B^3} = 405.7 + 11.78$$

$$\frac{155B+12.09+5.952}{B^3} = 417.48$$

$155B+18.042 = 417.48B^3$

$417.4B^3-155B-18.042 = 0$

B=0.661 m ≈ **1 m**

For maximum load (p = 277 kN and Mx = -1 kNm and My = -1 kNm)

$$\frac{277}{B^2} * [1 + \frac{6*(0.0036)}{B} + \frac{6*(0.0036)}{B}] = \frac{(1.3*164*5.71)+(17.68*2*1)}{3}$$

$$\frac{277}{B^2} + \frac{5.98}{B^3} + \frac{5.98}{B^3} = 405.7 + 11.78$$

$$\frac{277B+11.96}{B^3} = 417.48$$

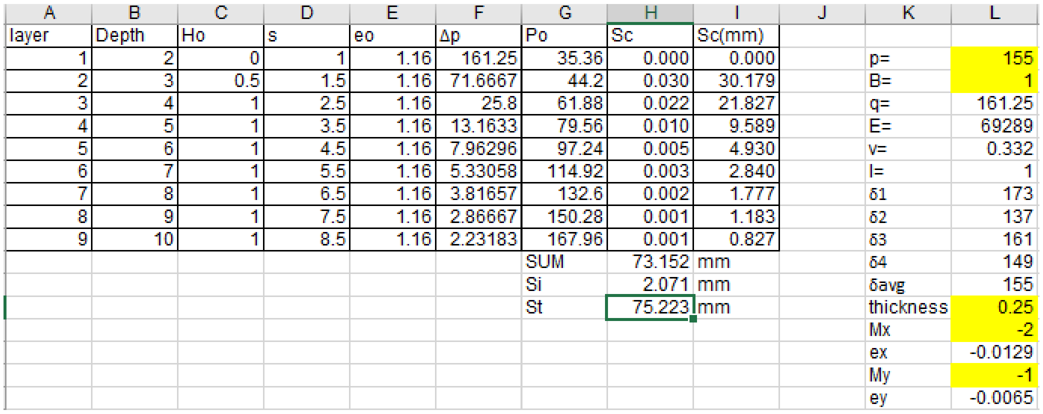
$277B+11.96 = 417.48B^3$

$417.48B^3-277B-11.96 = 0$

B=1.12 m ≈ **1.2 m**

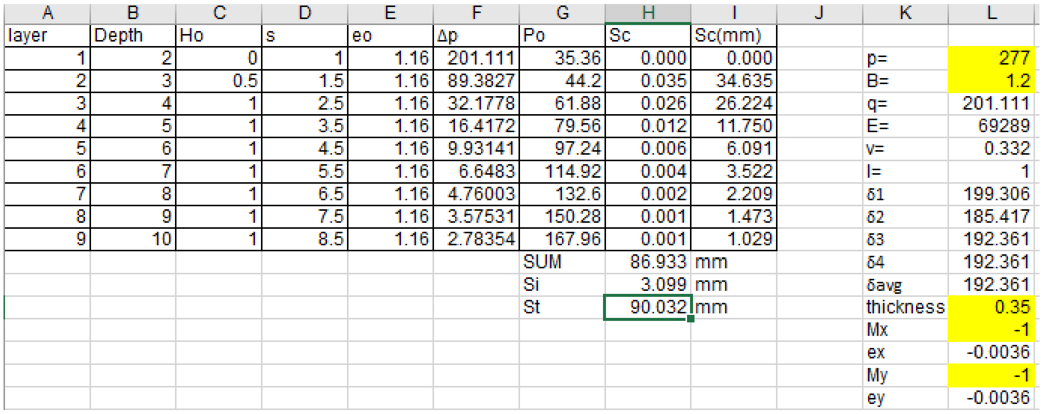
Appendix D2: - **Settlement calculation using Excel templet**

- For p = 155 kN and Footing Dimension of 1*1 m



A	B	C	D	E	F	G	H	I	J	K	L
layer	Depth	Ho	s	eo	Δp	Po	Sc	Sc(mm)			
1	2	0	1	1.16	161.25	35.36	0.000	0.000		p=	155
2	3	0.5	1.5	1.16	71.6667	44.2	0.030	30.179		B=	1
3	4	1	2.5	1.16	25.8	61.88	0.022	21.827		q=	161.25
4	5	1	3.5	1.16	13.1633	79.56	0.010	9.589		E=	69289
5	6	1	4.5	1.16	7.96296	97.24	0.005	4.930		v=	0.332
6	7	1	5.5	1.16	5.33058	114.92	0.003	2.840		I=	1
7	8	1	6.5	1.16	3.81657	132.6	0.002	1.777		δ1	173
8	9	1	7.5	1.16	2.86667	150.28	0.001	1.183		δ2	137
9	10	1	8.5	1.16	2.23183	167.96	0.001	0.827		δ3	161
						SUM	73.152	mm		δ4	149
						Si	2.071	mm		δavg	155
						St	75.223	mm		thickness	0.25
										Mx	-2
										ex	-0.0129
										My	-1
										ey	-0.0065

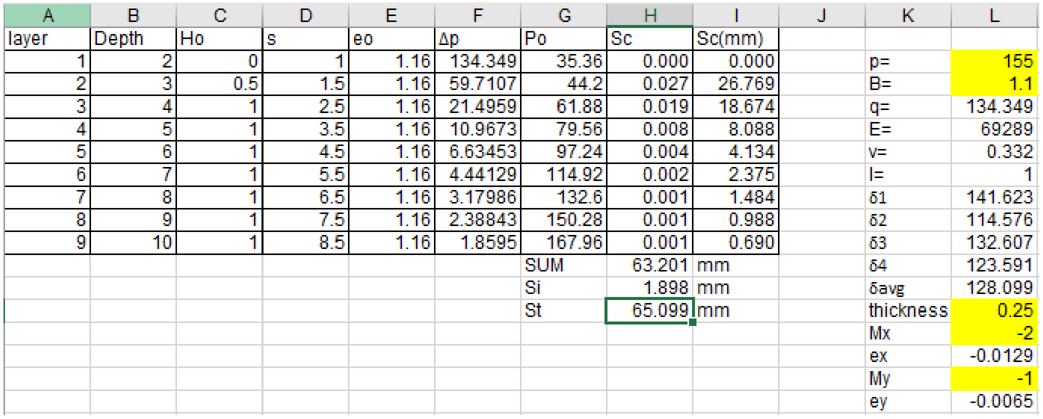
- For p = 277 kN and Footing Dimension of 1.2*1.2 m



A	B	C	D	E	F	G	H	I	J	K	L
layer	Depth	Ho	s	eo	Δp	Po	Sc	Sc(mm)			
1	2	0	1	1.16	201.111	35.36	0.000	0.000		p=	277
2	3	0.5	1.5	1.16	89.3827	44.2	0.035	34.635		B=	1.2
3	4	1	2.5	1.16	32.1778	61.88	0.026	26.224		q=	201.111
4	5	1	3.5	1.16	16.4172	79.56	0.012	11.750		E=	69289
5	6	1	4.5	1.16	9.93141	97.24	0.006	6.091		v=	0.332
6	7	1	5.5	1.16	6.6483	114.92	0.004	3.522		I=	1
7	8	1	6.5	1.16	4.76003	132.6	0.002	2.209		δ1	199.306
8	9	1	7.5	1.16	3.57531	150.28	0.001	1.473		δ2	185.417
9	10	1	8.5	1.16	2.78354	167.96	0.001	1.029		δ3	192.361
						SUM	86.933	mm		δ4	192.361
						Si	3.099	mm		δavg	192.361
						St	90.032	mm		thickness	0.35
										Mx	-1
										ex	-0.0036
										My	-1
										ey	-0.0036

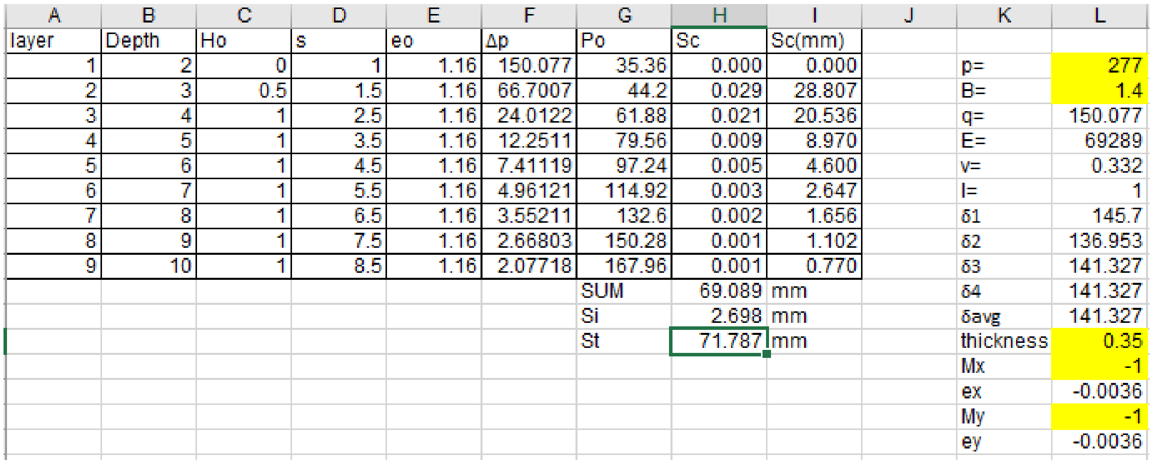
Appendix D3: - Settlement calculation for increased footing dimension

- For p = 155 kN and Footing Dimension of 1.1*1.1 m



A	B	C	D	E	F	G	H	I	J	K	L
layer	Depth	Ho	s	eo	Δp	Po	Sc	Sc(mm)			
1	2	0	1	1.16	134.349	35.36	0.000	0.000		p=	155
2	3	0.5	1.5	1.16	59.7107	44.2	0.027	26.769		B=	1.1
3	4	1	2.5	1.16	21.4959	61.88	0.019	18.674		q=	134.349
4	5	1	3.5	1.16	10.9673	79.56	0.008	8.088		E=	69289
5	6	1	4.5	1.16	6.63453	97.24	0.004	4.134		v=	0.332
6	7	1	5.5	1.16	4.44129	114.92	0.002	2.375		I=	1
7	8	1	6.5	1.16	3.17986	132.6	0.001	1.484		δ1	141.623
8	9	1	7.5	1.16	2.38843	150.28	0.001	0.988		δ2	114.576
9	10	1	8.5	1.16	1.8595	167.96	0.001	0.690		δ3	132.607
						SUM	63.201	mm		δ4	123.591
						Si	1.898	mm		δavg	128.099
						St	65.099	mm		thickness	0.25
										Mx	-2
										ex	-0.0129
										My	-1
										ey	-0.0065

- For p = 277 kN and Footing Dimension of 1.4*1.4 m



A	B	C	D	E	F	G	H	I	J	K	L
layer	Depth	Ho	s	eo	Δp	Po	Sc	Sc(mm)			
1	2	0	1	1.16	150.077	35.36	0.000	0.000		p=	277
2	3	0.5	1.5	1.16	66.7007	44.2	0.029	28.807		B=	1.4
3	4	1	2.5	1.16	24.0122	61.88	0.021	20.536		q=	150.077
4	5	1	3.5	1.16	12.2511	79.56	0.009	8.970		E=	69289
5	6	1	4.5	1.16	7.41119	97.24	0.005	4.600		v=	0.332
6	7	1	5.5	1.16	4.96121	114.92	0.003	2.647		I=	1
7	8	1	6.5	1.16	3.55211	132.6	0.002	1.656		δ1	145.7
8	9	1	7.5	1.16	2.66803	150.28	0.001	1.102		δ2	136.953
9	10	1	8.5	1.16	2.07718	167.96	0.001	0.770		δ3	141.327
						SUM	69.089	mm		δ4	141.327
						Si	2.698	mm		δavg	141.327
						St	71.787	mm		thickness	0.35
										Mx	-1
										ex	-0.0036
										My	-1
										ey	-0.0036

Appendix D4: - Designing of GAP system for original and reduced footing dimension

- **For minimum load (p = 155 kN) and Footing Dimension of 1.1*1.1 m**

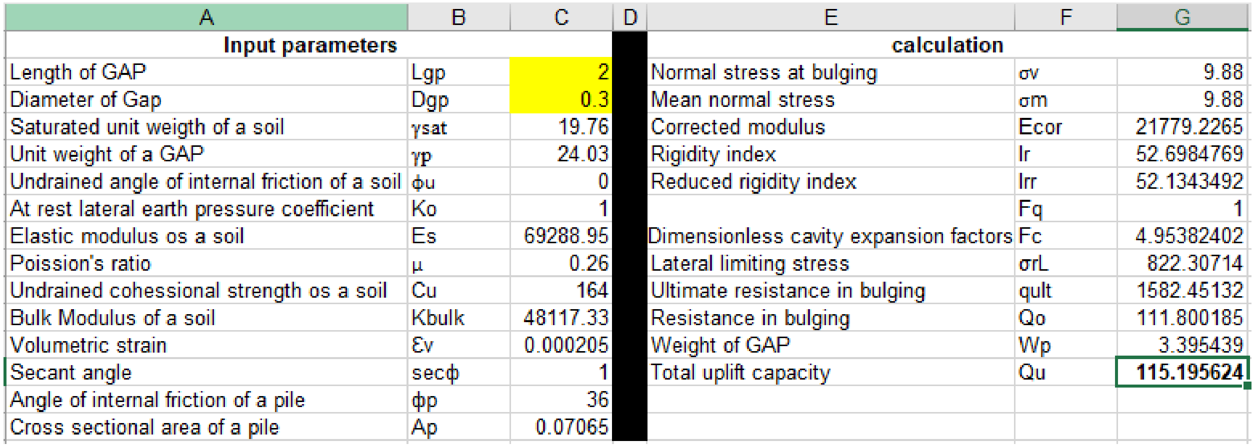
Residual swelling pressure (Ps) = 74.17 kN/m^2

Since the vertical uplift force is Pu = Ps*A$_{footing}$

Pu = 74.17*1.1*1.1 = **89.74 kN**

Assume Dgp = 0.3 m and Lgp = 2 m

Therefore, the uplift capacity of the GAP (Qu) can be calculated according to Equation 2.1 - 2.15. The calculation is summarized in table below.



A	B	C	D	E	F	G
Input parameters				**calculation**		
Length of GAP	Lgp	2		Normal stress at bulging	σv	9.88
Diameter of Gap	Dgp	0.3		Mean normal stress	σm	9.88
Saturated unit weigth of a soil	γsat	19.76		Corrected modulus	Ecor	21779.2265
Unit weight of a GAP	γp	24.03		Rigidity index	Ir	52.6984769
Undrained angle of internal friction of a soil	ϕu	0		Reduced rigidity index	Irr	52.1343492
At rest lateral earth pressure coefficient	Ko	1			Fq	1
Elastic modulus os a soil	Es	69288.95		Dimensionless cavity expansion factors	Fc	4.95382402
Poission's ratio	μ	0.26		Lateral limiting stress	σrL	822.30714
Undrained cohessional strength os a soil	Cu	164		Ultimate resistance in bulging	qult	1582.45132
Bulk Modulus of a soil	Kbulk	48117.33		Resistance in bulging	Qo	111.800185
Volumetric strain	Ɛv	0.000205		Weight of GAP	Wp	3.395439
Secant angle	secϕ	1		Total uplift capacity	Qu	**115.195624**
Angle of internal friction of a pile	ϕp	36				
Cross sectional area of a pile	Ap	0.07065				

Since Qu > Pu ……ok!

Therefore, take Dgp = 0.3 m and Lgp = 2 m for footing Dimension of 1.1*1.1 m

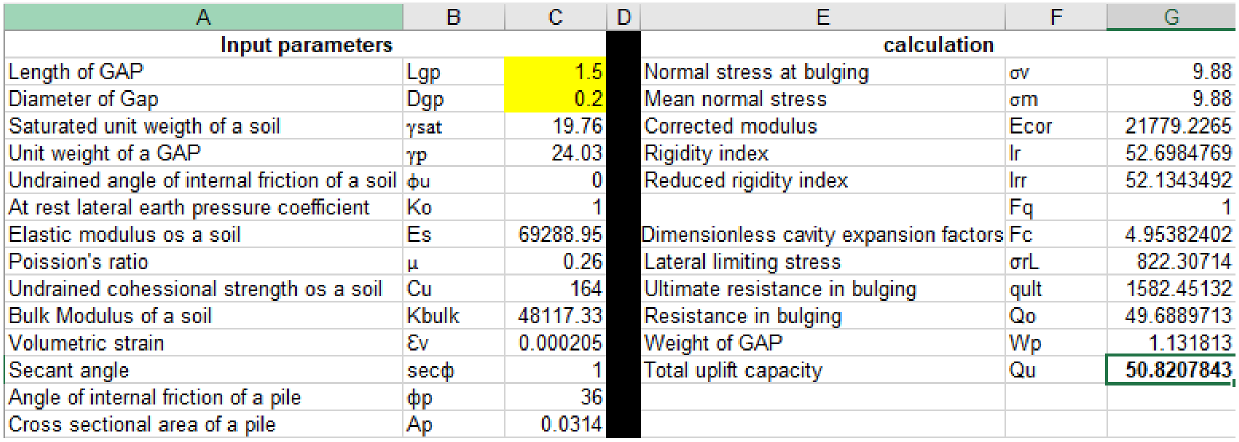
If the footing dimension decreased to 1*1 m qn = (155/1 m^2) + 0.25*25 = 161.25 kPa

Ps = 208.52 – 161.25 = 47.27 kPa > 20 kPa…… can cause damage!

Pu = 47.27*(1.0*1.0) = **47.27 kN**

Assume Dgp = 0.2 m and Lgp = 1.5 m

Therefore, Qu



A	B	C	D	E	F	G
Input parameters				**calculation**		
Length of GAP	Lgp	1.5		Normal stress at bulging	σv	9.88
Diameter of Gap	Dgp	0.2		Mean normal stress	σm	9.88
Saturated unit weigth of a soil	γsat	19.76		Corrected modulus	Ecor	21779.2265
Unit weight of a GAP	γp	24.03		Rigidity index	Ir	52.6984769
Undrained angle of internal friction of a soil	ϕu	0		Reduced rigidity index	Irr	52.1343492
At rest lateral earth pressure coefficient	Ko	1			Fq	1
Elastic modulus os a soil	Es	69288.95		Dimensionless cavity expansion factors	Fc	4.95382402
Poission's ratio	µ	0.26		Lateral limiting stress	σrL	822.30714
Undrained cohessional strength os a soil	Cu	164		Ultimate resistance in bulging	qult	1582.45132
Bulk Modulus of a soil	Kbulk	48117.33		Resistance in bulging	Qo	49.6889713
Volumetric strain	Ɛv	0.000205		Weight of GAP	Wp	1.131813
Secant angle	secϕ	1		Total uplift capacity	Qu	**50.8207843**
Angle of internal friction of a pile	ϕp	36				
Cross sectional area of a pile	Ap	0.0314				

Since Qu > Pu ……ok!

Therefore, it is safe to use 1*1 m footing dimension on the GAP with Lgp = 1.5 m and Dgp = 0.2 m

- **For maximum load (p = 277 kN) and Footing Dimension of 1.4*1.4 m**

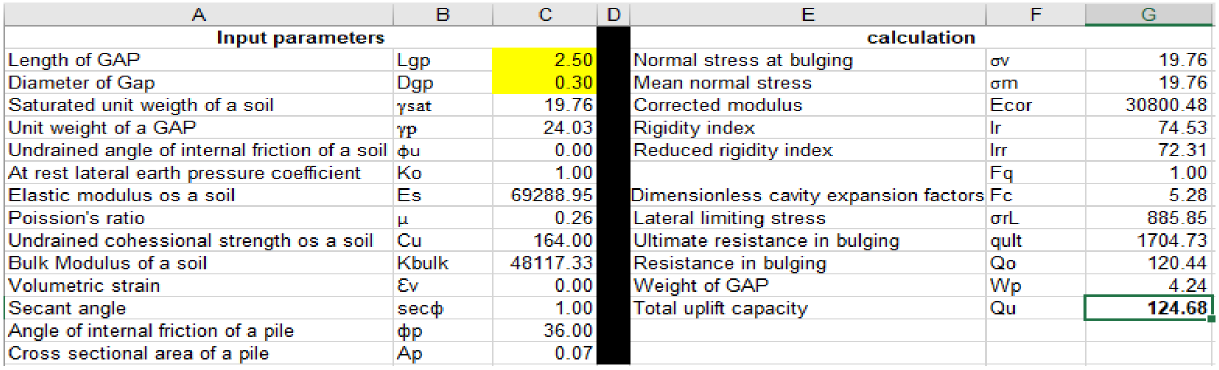
Residual swelling pressure (Ps) = 58.45 kN/m^2

Since the vertical uplift force is Pu = Ps *$A_{footing}$

Pu = 58.45*(1.4*1.4) = **114.56 kN**

Assume Dgp = 0.3 m and Lgp = 2.5 m

Therefore, the uplift capacity (Qu) become



A	B	C	D	E	F	G
Input parameters				**calculation**		
Length of GAP	Lgp	2.50		Normal stress at bulging	σv	19.76
Diameter of Gap	Dgp	0.30		Mean normal stress	σm	19.76
Saturated unit weigth of a soil	γsat	19.76		Corrected modulus	Ecor	30800.48
Unit weight of a GAP	γp	24.03		Rigidity index	Ir	74.53
Undrained angle of internal friction of a soil	ϕu	0.00		Reduced rigidity index	Irr	72.31
At rest lateral earth pressure coefficient	Ko	1.00			Fq	1.00
Elastic modulus os a soil	Es	69288.95		Dimensionless cavity expansion factors	Fc	5.28
Poission's ratio	µ	0.26		Lateral limiting stress	σrL	885.85
Undrained cohessional strength os a soil	Cu	164.00		Ultimate resistance in bulging	qult	1704.73
Bulk Modulus of a soil	Kbulk	48117.33		Resistance in bulging	Qo	120.44
Volumetric strain	Ɛv	0.00		Weight of GAP	Wp	4.24
Secant angle	secϕ	1.00		Total uplift capacity	Qu	**124.68**
Angle of internal friction of a pile	ϕp	36.00				
Cross sectional area of a pile	Ap	0.07				

Since Qu > Pu ……ok!

If the footing dimension reduced to 1.3*1.3 m

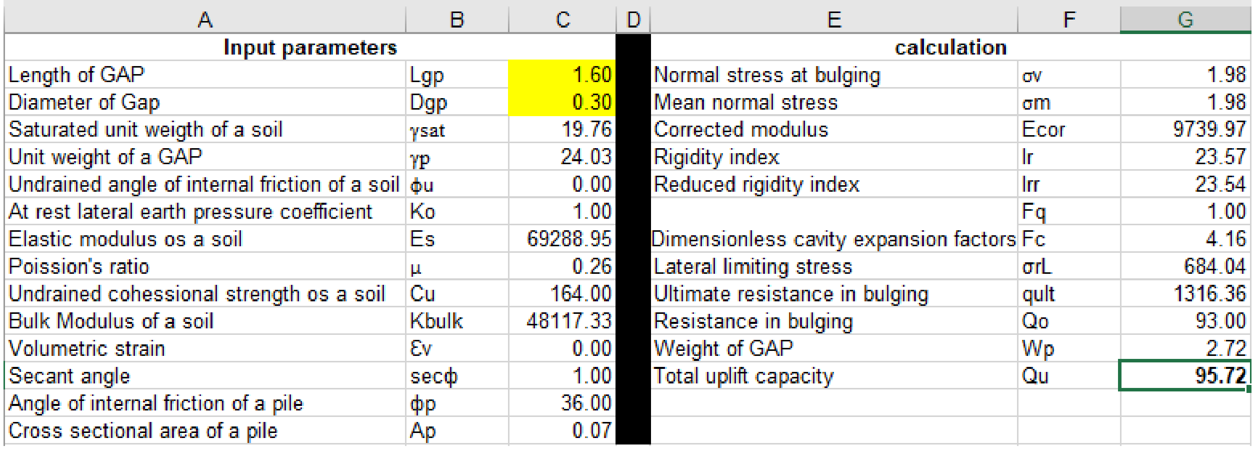
qn = (277/1.3*1.3 m^2) + 0.35*25 = 172.65 kPa

Ps = 208.52 – 172.65 = 35.86 kPa > 20 kPa…… can cause damage!

Pu = 35.86*(1.3*1.3) = **60.6 kN**

Assume Dgp = 0.3 m and L = 1.6 m

Therefore, Qu become



A	B	C	D	E	F	G
Input parameters				**calculation**		
Length of GAP	Lgp	1.60		Normal stress at bulging	σv	1.98
Diameter of Gap	Dgp	0.30		Mean normal stress	σm	1.98
Saturated unit weigth of a soil	γsat	19.76		Corrected modulus	Ecor	9739.97
Unit weight of a GAP	γp	24.03		Rigidity index	Ir	23.57
Undrained angle of internal friction of a soil	ϕu	0.00		Reduced rigidity index	Irr	23.54
At rest lateral earth pressure coefficient	Ko	1.00			Fq	1.00
Elastic modulus os a soil	Es	69288.95		Dimensionless cavity expansion factors	Fc	4.16
Poission's ratio	μ	0.26		Lateral limiting stress	σrL	684.04
Undrained cohessional strength os a soil	Cu	164.00		Ultimate resistance in bulging	qult	1316.36
Bulk Modulus of a soil	Kbulk	48117.33		Resistance in bulging	Qo	93.00
Volumetric strain	Ɛv	0.00		Weight of GAP	Wp	2.72
Secant angle	secϕ	1.00		Total uplift capacity	Qu	**95.72**
Angle of internal friction of a pile	ϕp	36.00				
Cross sectional area of a pile	Ap	0.07				

Since Qu > Pu ……ok!

Therefore, it is safe to use 1.3*1.3 m footing dimension on the GAP