Annex 1

SITE IMPROVEMENT OPTIMIZATION THROUGH AUTOMATION

Final Year Project UG 2014

By

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2014

Annex 2

This is to certify that the

Final Year Project, titled

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For the undergraduate degree

In

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ACKNOWLEDGEMENTS

"In the name of Almighty Allah, the Most Beneficent, the Most Merciful"

It is by the grace of ALLAH Almighty that we have been able to carry out this project successfully and completed it well in time. Moreover, we are thankful to our advisor Lecturer Muhammad Asim and co-advisor Dr. Syed Muhammad Jamil for their guidance and professional advices throughout in the tasks carried out. His provision of various case studies in the geotechnical field greatly helped us in achieving the accuracy in our analysis.

We would also like to mention Bilal Musani, Waqar-ul-khaf, Anas Khalid and Zaman Asif Virk of who are UG students of SEECS as they were helpful and generous enough to provide us with their time and expertise in the field of software development. They helped us steer out of the problems we faced during our project.

Lastly, we would like to thank Terzaghi, Peck, Mesri, Jie Han, Robert G. Lucas, F.C Townsend, J.B Anderson, FHWA manuals and many other researchers and engineers whose valuable literature helped us achieve this milestone.

ABSTRACT

Ground improvement is used to solve difficult geotechnical problems, especially when construction necessarily occurs in problematic soils or under difficult geotechnical conditions. Geotechnical Engineers encounter geotechnical problems, such as bearing failure, large settlements, instability, liquefaction, erosion, and water seepage. Many recent developments in equipment, materials, and design methods have made ground improvement technologies more effective, efficient, and economic. Ground improvement has become an important part of geotechnical practice.

In this project, we have developed an automated site improvement system that utilizes user inserted ground conditions, site utility and desired performance parameters to suggest the most optimum site improvement technique. This system will integrate all available ground improvement techniques into single platform. The system have user interactive interface. The system is also capable of design/analysis of the soil as per requirement.

Moreover, the system is windows form application. The development of an extensively detailed computer program is done on Microsoft Visual Studio (using C# language) using various methods and techniques for soil and site improvement.

Soil improvement is preferred on all the types of foundations:

- Shallow foundation -- All techniques
- Deep foundation -- Piles only

In the end, the user gets a detailed report about the percentage improvement of bearing capacities, settlement and cost after the improvement of soil by just putting the basic field parameter as inputs.

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CHAPTER 1

INTRODUCTION

1.1 General

Ground improvement is a tool to solve difficult geotechnical problems, especially when construction necessarily occurs in problematic soils such as soft soil, collapsible soil, organic and peaty soil, old mine pits or under difficult geotechnical conditions. Ground improvement techniques are used when behavior of the fill mass and/or the underlying soil does not meet required design criteria. Engineers have faced increased geotechnical problems and challenges, such as bearing failure, large total and differential settlements, instability, liquefaction, erosion, and water seepage. Ground improvement is carried out to improve shear strength of the fill and subsoil to ensure sufficient bearing capacity of the foundations and stability of the slopes. It is used to prevent excessive settlements of the surface of the site area when structures like buildings, roads and other foundations are loaded on it. The options to deal with problematic geomaterial and geotechnical conditions include: avoiding the site, designing superstructures accordingly, removing and replacing problematic with better and non-problematic geo-materials and improving geo-material properties and geotechnical conditions. It becomes increasingly necessary to improve geo-materials and geotechnical conditions for many projects. Ground improvement has become an important part of geotechnical practices.

As a result, it is necessary to conduct extensive soil investigations in order to obtain accurate geotechnical properties. These values facilitate in determining the most appropriate site improvement technique applicable to the given strata.

So, our group took this as our Undergraduate Final Year Project because we wanted to obtain a complete experience and understanding of the various engineering aspects related to ground/site improvement methods which will inevitably be extremely beneficial in our professional career.

1.2 Objectives

The basic purpose of taking this project is to study in depth the vast set of geotechnical principals and techniques and their application in the real life problems. Our team is trying to achieve our objectives via following practices:

1.2.1 Relate the theoretical knowledge with practical application

Gaining the theoretical knowledge is also as important as gaining the technical and practical knowledge. Through this project, we gained all the required knowledge, conditions, methods, requirements and techniques required for the ground improvement and then implementing those finding in real life projects.

1.2.2 Develop a simple yet extensive software

Extensive and rigorous calculations are main part of our project. We are well restricted in this project because we did take the example from book only due to limitation of usability. For this purpose, our team would make simple interface which were later converted into computer software which is user friendly and is able to handle all kinds of problems and cases with speed and ease. This software provides complete design or analysis of site based upon applicability of each technique. The development of this project is not only helpful for us to master our concepts about ground improvement but will be useful throughout professional careers and can have some industrial applications and uses in the near future.

1.2.3 Learning various software

In the entire working of our project we learned and mastered various software which will be useful in our project to successfully complete it:

- Microsoft Word
- Microsoft Excel
- Microsoft Visual Studio
- Graph Digitizer

1.3 Why Ground Improvement for Final Year Project?

As it is already clear from the above mentioned introduction and objective we choose Ground improvement as our Final Year Project because it is really important for the site before the construction of structure, if the soil has not the capacity to bear the large load of structure. Ground improvement does not belong to a single subject of civil engineering but it's a combination of Geotechnical Engineering, Material Engineering, Surveying and Hydraulics Engineering etc. These subjects have played a major part on our civil engineering degree so our team felt that this would be the ideal topic to sum it all up and achieve an end product of all our learnings.

1.4 Academic Project Outcomes

Other than the main objective of developing our geotechnical knowledge and practical skills the scope of this project goes beyond that thus making it a very dynamic project. We have merged various fields of engineering in our one single project ranging from geotechnical engineering to development of software.

The following are the fundamental academic outcomes of our project which encompasses these attributes:

- Understanding
- Accuracy
- Coherence
- Ease

All these attributes play a vital role when fresh engineers step into their professional carriers. These elements will become the stepping stones in a geotechnical design of a foundation, by that concluding this thesis.

CHAPTER 2

LITERATURE REVIEW

2.1 Bearing Capacity

Bearing capacity refers to the ability of a soil to support or hold up a foundation and structure. The ultimate bearing capacity of a soil refers to the loading per unit area that will just cause shear failure in the soil. It is given the symbol q_{ult}. The allowable bearing capacity (symbol q_a) refers to the loading per unit area that the soil is able to support without unsafe movement.

Bearing capacity of the shallow and deep foundations was discussed below separately:

2.2 Shallow Foundations

2.2.1 Introduction

Shallow foundations are those which transfer load to the near surface soils. The depth of shallow foundation is less or equal to width of foundation. Depending on the load imposed there are multiple types of a shallow foundation

- Square footing
- Strip footing(Continuous footing)
- Rectangular footing
- Circular footing
- Mat foundation
- Combined footing

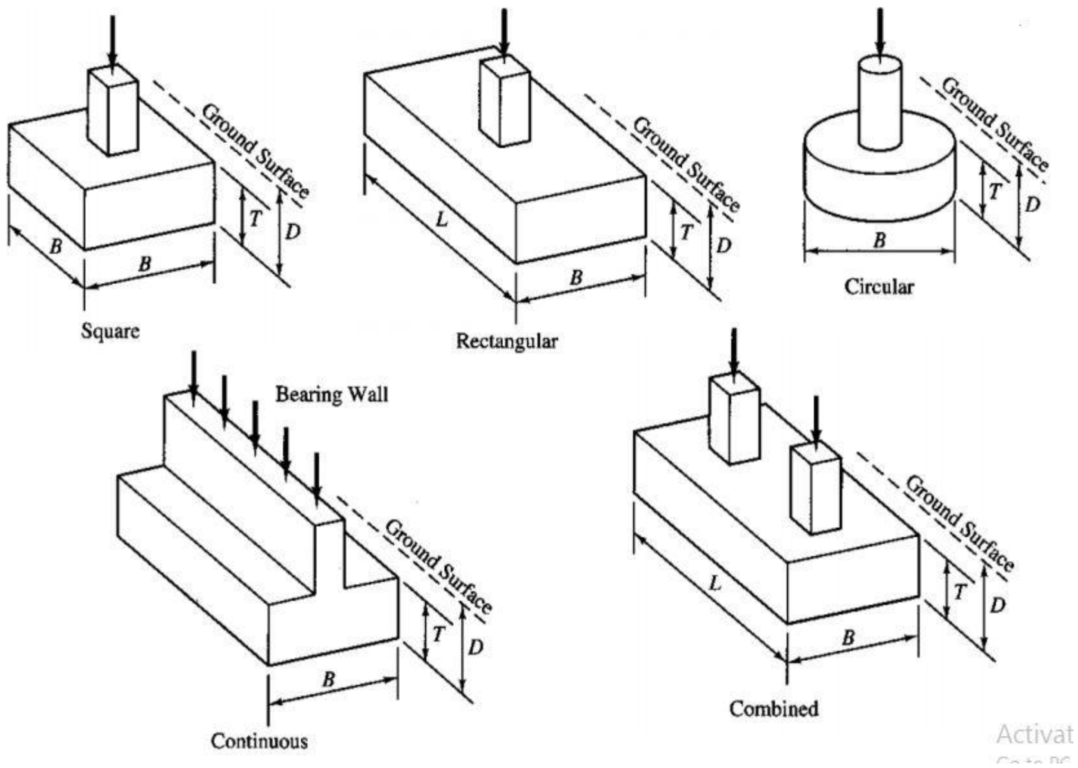


Figure 1 Types of shallow foundations

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

- The foundation must be stable against shear failure of supporting soil.
- The foundation must not settle beyond a tolerable limit to avoid damage to structure.

2.2.2 Type of Bearing Capacity Failure

A bearing capacity failure is a failure in which the shear stresses in the soil surpass the shear strength of the soil. This is further divided into three types, as follows: General shear failure, Local shear failure, Punching shear failure.

By seeing the scope of the project, we have discussed only general shear failure.

2.2.3 General Shear Failure

A general shear failure involves total rupture of the underlying soil. This failure ruptures and pushes up the soil on both sides of the footing. For actual failures in the field, the soil is often pushed up on only one side of the footing with subsequent tilting of the structure. A general shear failure occurs for soils that are in a dense or hard state.

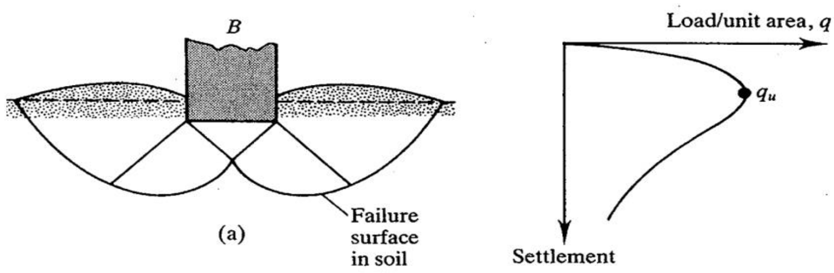


Figure 2 General shear failures

We will discuss only Terzaghi's bearing capacity theory for the general shear failure.

2.2.3.1 Terzaghi's Bearing Capacity Theory

Terzaghi (1943) first presented a comprehensive theory for the evaluation of ultimate bearing capacity of shallow foundations. In deriving the equation, he assumed the following:

- The soil is homogeneous, isotropic and holds Coulomb's law of shear strength.
- Rough base and continuous footing.
- Failure zone does not extend above the base of the foundation.
- Shear strength above the base is neglected.
- The portion from top to base of foundation is replaced by a uniform surcharge.
- Principal of superposition holds well.

Terzaghi developed bearing capacity equations for different types of footings. For general shear failure

Continuous footings (width B): $q_{ult} = c'N_c + \gamma D_f N_q + 0.5\gamma B N_\gamma$

Circular footings (radius B): $q_{ult} = 1.2c'N_c + \gamma D_f N_q + 0.3\gamma B N_\gamma$

Square footings (width B): $q_{ult} = 1.2c'N_c + \gamma D_f N_q + 0.4\gamma B N_\gamma$

Where

C' = cohesion of soil

γ = unit weight of soil

N_c , N_q , N_γ = Bearing capacity factors

The equation of all bearing capacity factors used in above equations is given as following:

$$N_q = e^{\pi tan\emptyset'} tan^2\left(45 + \frac{\emptyset'}{2}\right)$$

$$N_c = \frac{N_q - 1}{tan\emptyset'}$$

$$N_\gamma = (Nq - 1) * tan(1.4\emptyset')$$

For $\emptyset' = 0$, cohesive soil the values of these bearing capacity factors are $N_c = 5.14$, $N_q = 1.0$, $N_\gamma = 0$

For Purely cohesion less soil, c = 0 the value of $N_c = 0$.

2.2.3.2 Effect of water table on bearing capacity

Based on location of water table below ground surface there may be three different cases when water table have effect on bearing capacity.

Case-1: If depth of water is between ground surface and depth of footing, 0≤ $D_w \leq D_f$, then q in bearing capacity equation is calculated as:

$$q = effective\ surcharge = D_1\gamma + D_2(\gamma_{sat} - \gamma_w)$$

Case-2: If depth of water is such that it is below footing but should not more than width of footing,$D_f \leq D_w \leq (D_f + B)$. Then,

$$\bar{\gamma} = \frac{1}{B}\{\gamma d + \gamma'(B - d)\}$$

Case-3: If depth of water is below the footing such that $D_w \geq D_f + B$, then water will have no effect on ultimate bearing capacity.

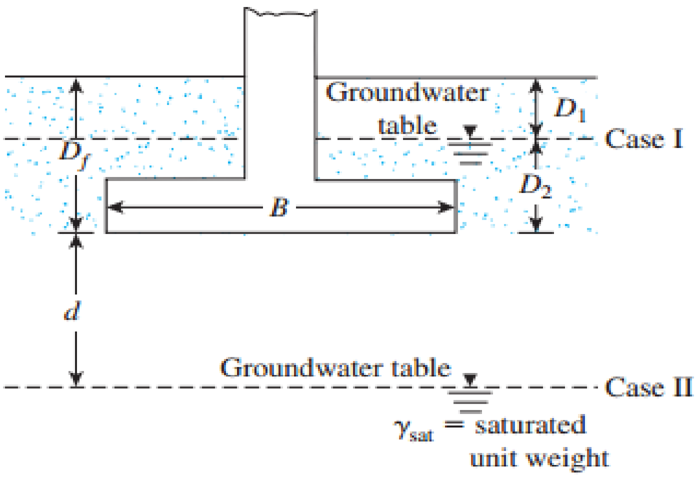


Figure 3 Effect of water table on bearing capacity

2.2.4 Bearing Capacity Theories Settlement Criteria

The bearing capacity of footing on clay is not affected by the size of footings, it remains constant. However, the settlement is increases with an increase in size of the footing. It is essential to consider both settlement criteria and shear (bearing capacity) criteria to decide safe bearing pressure.

For soft clay and weak soils settlement analysis is necessary.

2.2.4.1 Effects of settlement

A structure may settle in two different ways; it may settle uniformly or differential settlement may occur.

Generally, there are two major components of foundation settlement one is elastic settlement and other is consolidation settlement. Consolidation settlement consists of two parts one is primary consolidation settlement and other is secondary consolidation settlement. Elastic settlement is mainly for foundation on granular soils. There is different method to calculated elastic settlement for the foundation on granular soil.

2.2.4.2 Terzaghi and Peck's method

Terzaghi and Peck proposed a relation for the calculation of elastic settlement based on observed settlement in 1948. This relation is between allowable bearing capacity, standard penetration test (SPT) N value and width of footing.

$$S_e = C_w C_D \frac{3q}{N_{60}} \left(\frac{B}{B+0.3}\right)$$

Where,

C_W= ground water table correction

C_D = depth of embedment correction = $1 - (\frac{D_f}{4B})$

D_f = depth of embedment (footing)

If the depth of water table is equal to or greater than 2B below the foundation, the magnitude of C_w is equal to 1, and if the depth of water table is less than or equal to B below the foundation it is equal to 2.

2.2.4.2 Meyerhof's method

Meyerhof proposed relationships for the elastic settlement based on observed settlement in 1956, for foundations on granular soil. But later on he had applied correction for water table location and depth of footing.

For B≤1.22m,

$$S_e = C_w C_D \frac{1.25q}{N_{60}}$$

For B>1.22m,

$$S_e = C_w C_D \frac{2q}{N_{60}} \left(\frac{B}{B + 0.3}\right)^2$$

Where

C_w = water dept correction = 1.0

C_D = depth of footing correction = $1 - (\frac{D_f}{4B})$

2.3 Deep Foundations

2.3.1 Introduction

Pile foundations are used to transfer loads of the structure to underlying soil strata. They go deep into the soil unlike shallow foundations. Shallow foundation is always cheaper than deep foundations and also easier to build and take less time for construction but under some conditions where shallow foundations can't provide structural safety it is necessary to construct deep foundations. Some of the situations under which it is necessary to go for deep foundations are:

- Upper soil is weak and can't provide enough support to the loads of structure
- Presence of lateral forces.
- Presence of expansive or collapsible soils on the site.
- To resist the uplifting force.
- Soil erosion at the ground surface.
- Large values of concentrated loads.

Pile distribute load of superstructure to the ground in one of the following ways

- Skin friction.
- End bearing.
- Combination of both skin friction and end bearing.

Different material which can be used for deep foundations are

- Steel.
- Timber.
- Concrete.

We have only considered **driven piles.**

2.3.2 Pile Load Transfer

To understand that how pile transfers the load to underlying soil, consider a pile is loaded with load Q on its top. Some of this load will be taken pile surface along the length of the pile and the remaining by end resistance. As the load is increased, most of the side frictional portion along the pile length will be developed when the pile moves 5 to 10 mm, and doesn’t depend on pile size and its length. On the other hand, the maximum tip resistance will not be developed unless the pile has moved about 10 to 25% of the pile dia. It indicates that as compare to the point resistance, side friction along the pile can be developed at a much smaller pile displacement.

2.3.3 Estimation of Pile Capacity

In addition to the strength of the pile itself, pile capacity is limited by soil’s supporting strength. The load carried by a pile is transmitted to the soil surrounding the pile by friction or adhesion between the soil and the pile surface, and/or the load is transmitted directly to the soil just below pile’s tip.

To find ultimate pile capacity following equation can be used

$$Q_u = Q_{friction} + Q_{tip}$$

Where,

Q_u = ultimate (at failure) bearing capacity of a single pile

Q_{tip} = load resistance at pile point

$Q_{friction}$= skin resistance from the soil–pile interface

2.3.4 Pile Capacity in Sand

2.3.4.1 End Bearing capacity in sand

The bearing capacity of pile tip (end bearing) is given by

$$q.A_{tip} = (\sigma'_v.N_q^*).A_{tip}$$

Where,

σ_v' = effective vertical stress adjacent to pile tip

N_q^* = bearing capacity factor

N_q^* Is related with the angle of internal friction (ϕ) of sand located in general vicinity of where the pile tip will ultimately rest.

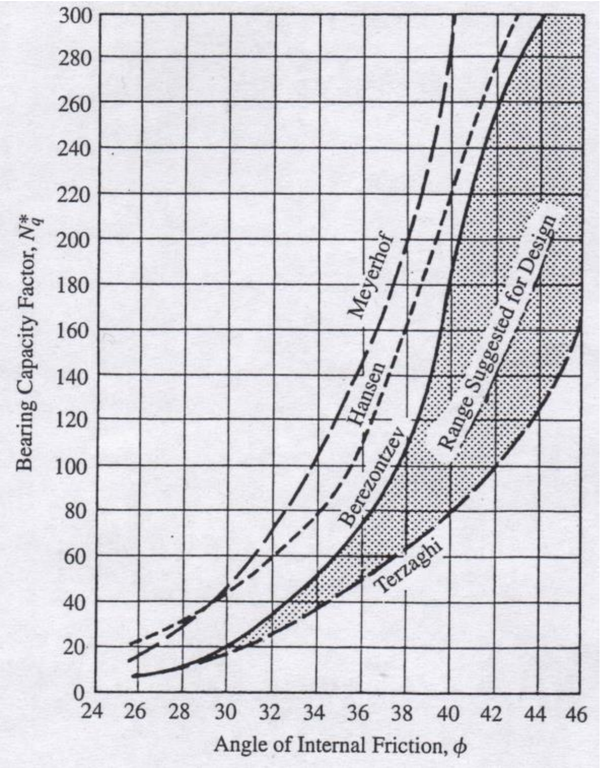


Figure 4 Angle of internal friction, ϕ

.2.3.4.2 Frictional Resistance in Sand

As described above, the frictional resistance

$$Q_{friction} = f.A_{surface}$$

$f.A_{surface}$= (Pile circumference).(Area under σ_v' diagram).(K).(tanδ)

Value of pile circumference is different for different geometry of pile i.e. circular, square and rectangular.

K = earth pressure coefficient

$\sigma_v{}'$ = effective overburden pressure

δ = friction angle of soil

The values of δ ranges from 0.5ϕ′ to 0.8ϕ′ Or value of tan δ can be taken from following table:-

Material	Tan δ
Concrete	0.45
Wood	0.4
Steel (smooth)	0.2
Steel (rough, rusted)	0.4
Steel (corrugated)	Use tan ϕ of sand

Figure 5 Table for coefficient for friction between sand and pile material

In the case of sand, one thing should be kept in mind that the unit skin friction value increases up to certain value of depth and then its value become constant. Its value is 10 pile diameter for loose soil and 20 pile diameter for dense sand.

The value of *K* changes with depth; at the top it is equal to the Rankine passive earth pressure coefficient, K_p of the pile and at a greater depth its value is equal to at-rest pressure coefficientK_0. For use following values are recommended.

Value of K is assumed to vary between 0.6 and 1.25, with lower values used for silty sands and higher values for other deposits (Bowles, 1977).

Pile Type	K
Bored or jetted	$K_o = 1 - sin\emptyset'$
Low-displacement driven	$K_o = 1 - sin\emptyset'$ to $1.4K_o = 1.4(1 - sin\emptyset')$
High-displacement driven	$K_o = 1 - sin\emptyset'$ to $1.8K_o = 1.8(1 - sin\emptyset')$

Figure 6 Average values of effective earth pressure coefficient K

2.3.5 Capacity in Clay

To find capacity in clay, we will use α-Method.

$$Q_{ultimate} = f.A_{surface} + q.A_{tip}$$

2.3.5.1 Frictional Resistance in Clay

In case of piles driven in clays, term "f" in above equation is adhesion between the soil and the sides of the pile.

$$f.A_{surface} = \alpha c.A_{surface}$$

Where,

α = Adhesion factor

The adhesion factor α can be determined from unconfined compressive strength q_u of clay. The values can be found from below graph.

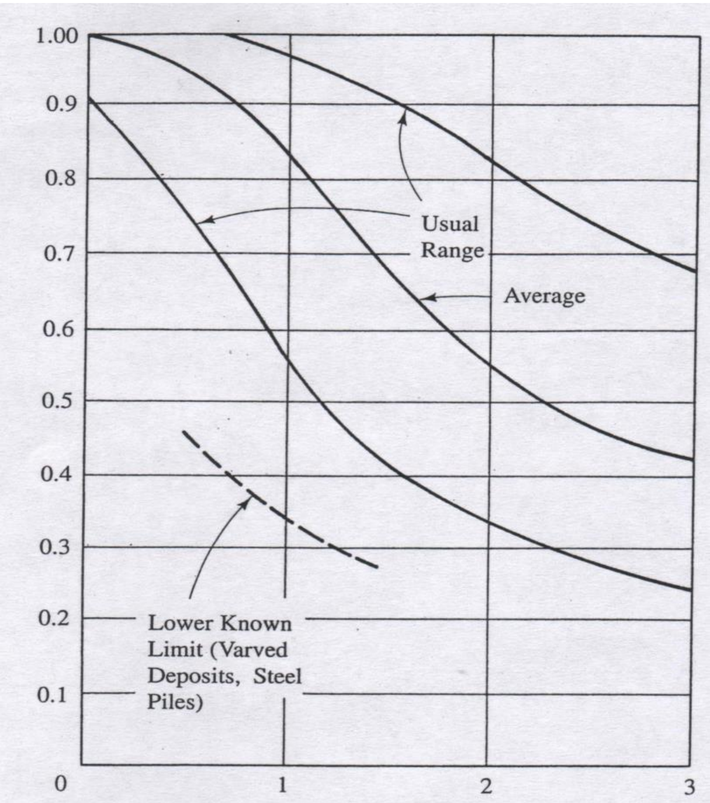


Figure 7 Adhesion factor α

2.4 Empirical Relation

We have used numerous empirical relations to find different values used in bearing capacity and settlement.

These relations are as follows:

2.4.1 Relationship of N-value with Cohesion

$$\frac{C_u}{P_a} = 0.06N_{60} \qquad \text{(Kalhawy and Mayne)}$$

Where,

C_u = Cohesion

P_a= Atmospheric pressure (1000 KPa)

N_{60} = Corrected SPT N-value

2.4.2 Relationship of N-value with Friction Angle

$$\emptyset' = tan^{-1}[\frac{N_{60}}{12.2+20.3(\frac{\sigma'}{P_a})}]^{0.34} \qquad \text{(Kalhawy and Mayne)}$$

Where,

$\emptyset$ = Friction angle

N_{60} = Corrected SPT N-value

σ' = Overburden Pressure

P_a= Atmospheric pressure (1000 KPa)

2.4.3 Relationship of N-value with Elastic Modulus of Soil

For clay,

$$E_s = 5884C_u \qquad \text{(Bowles, 1997)}$$

For sand,

$$E_s = 7355 + 785N_{60} \qquad \text{(Bowles, 1997)}$$

Where,

E_s = Elastic modulus of soil in KPa

2.4.4 Relationship of Cohesion with OCR

$$\frac{S_u}{\sigma'_{vo}} = 0.22(OCR)^{0.8} \qquad \text{(Jamiolkowski et al,1985)}$$

Where,

S_u = Cohesion

σ'_{vo} = Overburden Pressure

OCR = Over consolidation Ratio

2.4.5 Relationship of Cohesion with Pre-consolidation Pressure

$$\frac{S_u}{\sigma'_c} = 0.22 \qquad \text{(Mesri, 1975)}$$

S_u = Cohesion

σ'_c = Pre-consolidation pressure

2.4.6 Relationship of Compression Index with Liquid Limit

$$C_c = 0.009(LL - 10) \qquad \text{(Skempton, 1944)}$$

Where,

C_c = Compression Index

LL = Liquid Limit

2.4.7 Relationship of Recompression Index with Plasticity Index

$$C_r = 0.00194(PI - 4.6) \qquad \text{(Nakase et al, 1988)}$$

Where,

C_r = recompression Index

PI = plasticity Index

2.4.8 Relationship of Compression Index with Void ratio

$$C_c = 1.15(e_o - 0.35) \qquad \text{(Nishida, 1956)}$$

Where,

$C_c =$ Compression Index and e_o = void ratio

2.5 Ground Improvement Techniques

In this project we will discuss eight unique ground improvement methods. These are explained below:

2.5.1 Deep Dynamic Compaction

2.5.1.1 Introduction

Deep dynamic compaction is to repeatedly drop a weight (tamper) freely from a height onto the ground surface in a pattern to compact problematic geomaterial to a deep depth. Repeated impacts reduce voids, densify the geomaterial, and induce ground movement. A tamper typically has a weight of 5–40 tons and drops from a height of 10–40 m. Different from shallow compaction, deep dynamic compaction can compact problematic geomaterial down to a depth of 10m.2.5.1 Deep Dynamic Compaction

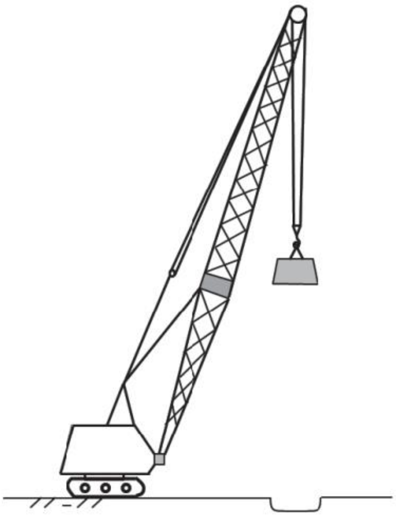


Figure 8 Dynamic compaction

2.5.1.2 Suitability

Deep dynamic compaction is suitable for the following conditions:

- Loose and partially saturated fills
- Saturated free-drained soils
- Silts with plasticity index less than 8
- Clayey soil with a low degree of saturation (moisture content lower than plastic limit)

2.5.1.3 Application

Deep dynamic compaction has been used to improve problematic geomaterials by increasing bearing capacity, reducing settlement, minimizing collapsible potential, and mitigating liquefaction for commercial and residential buildings, storage tanks, highways and railways, airports and harbors.

2.5.1.4 Design Consideration

Depth and area of improvement:

$$D_i = n_c\sqrt{W_t H_d}$$

Where D_i = depth of improvement (m)

W_t = weight of tamper (ton)

H_d = height of drop (m)

n_c = constant (take 0.35 as an average value)

Drop height and energy:

$$H_d = (W_t H_d)^{0.54}$$

Where $W_t H_d$ is energy per drop of tamper (ton-m) which is determined from above equation based on the required depth of improvement.

Pattern and Spacing of Drops:

$$S = 2 * d_t$$

Where $d_t$diameter of is tamper and S is grid spacing.

Depth of crater:

$$d_{cd} = 0.075\sqrt{W_t H_d}$$

Where, d_{cd} is depth of crater.

Number of drops and passes:

$$AE = \frac{N_d W_t H_d P}{S^2}$$

Where AE = applied energy

N_d = number of drops in one pass

P = number of passses

Ironing pass energy:

$$UAE = \frac{AE}{D_i}$$

Where UAE is Unit applied energy.

$$AE_{IP} = UAE.d_{cd}$$

Where AE_{IP} is applied energy by an ironing pass.

Total applied energy:

$$TAE = AE - AE_{IP}$$

Peak particle velocity:

$$PPV = 70(\frac{\sqrt{W_t H_d}}{x_{dp}})^{1.4}$$

Where PPV = peak particle velocity (mm/s)

x_{dp} = distance to the drop point (m)

2.5.2 Dewatering

2.5.2.1 Introduction

Dewatering is to lower an existing groundwater table by open pumping (sumps, trenches, and pumps) and a well system (well points or deep wells). The most common purpose for dewatering is for construction excavations. Dewatering for construction excavations is mostly temporary. Dewatering is to lower a ground water table from the existing level to a lower level.

2.5.2.2 Suitability

The suitability of a dewatering technique depends on:

- Location, type, size, and depth of excavation
- Thickness, stratification, and permeability of geomaterials
- Required depth of the groundwater to be lowered
- Potential damage resulting from failure of the dewatering system
- Cost of installation and operation

2.5.2.3 Application

Dewatering has been mostly used for construction excavations. It has also been used as a permanent dewatering system for permanent structures and highways. Dewatering is sometimes used for improving soil properties and resistance to liquefaction. There is a case history that uses permanent pumping wells within the building footprint to lower the groundwater table to eliminate the risk of soil liquefaction.

2.5.2.4 Advantage and Limitation

Dewatering systems are easy to install and can eliminate possible problems associated with water during excavations and permanent uses. Dewatering may induce ground subsidence and cause damage to adjacent structures. It should be used with caution when there are nearby existing structures and utilities. Dewatering requires disposal or recycling of water removed from the ground and continuous power supplies.

2.5.2.5 Design parameters

The design of dewatering require parameters such as size of excavation , soil conditions (unconfined or confined condition), depth of water to be lowered, stage of well points, number of deep wells, spacing of well points and pipe size.

Unconfined Condition

The rate of water flow can be solved in unconfined condition as:

$$Q_w = \frac{\pi k ({h_{w0}}^2 - {h_{w1}}^2)}{\ln(\frac{R_i}{r_0})}$$

Unconfined Condition

Under the confined condition, discharge can be calculated as:

$$Q_w = \frac{2\pi k h_{dr} ({h_{w0}}^2 - {h_{w1}}^2)}{\ln(\frac{R_i}{r_0})}$$

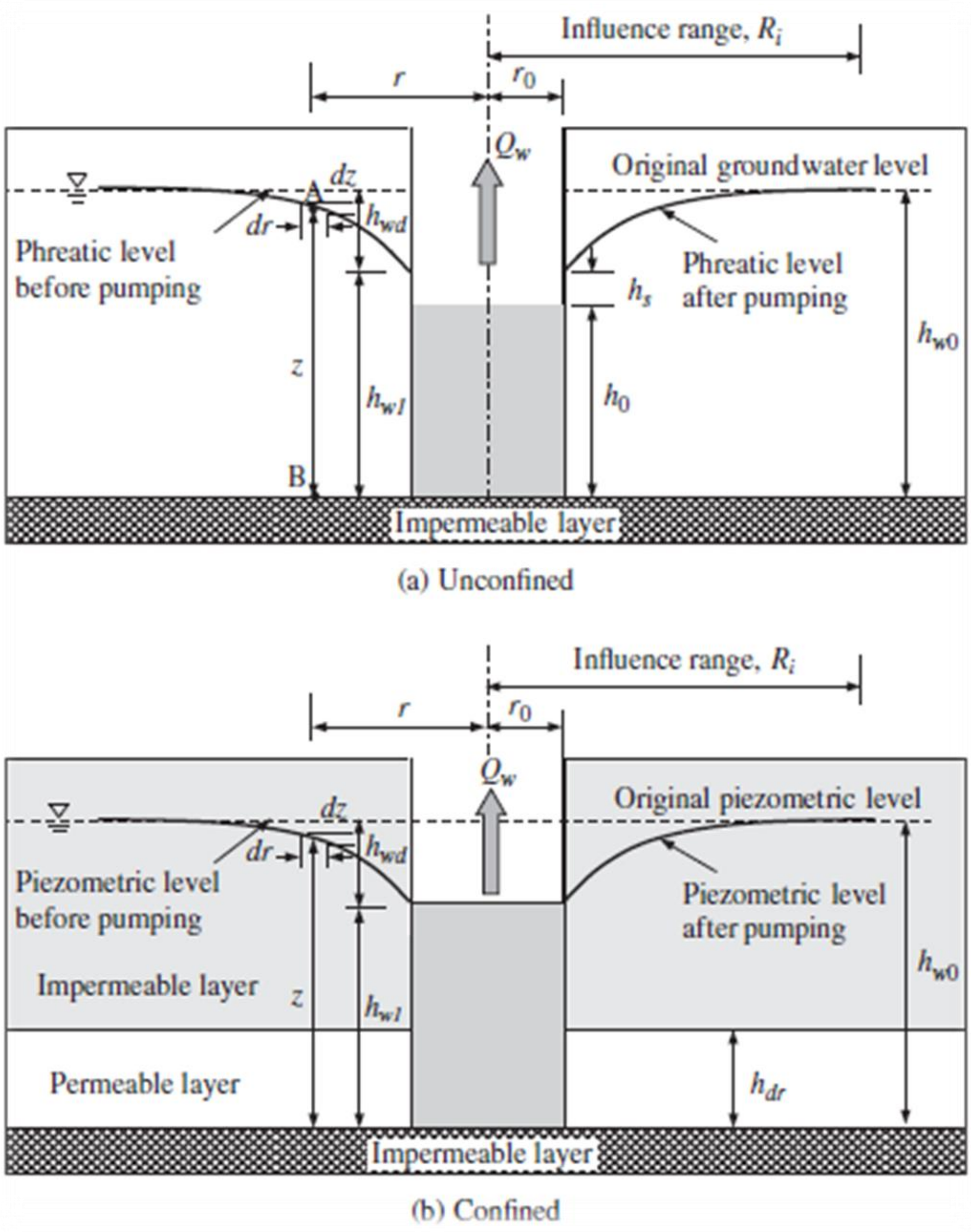


Figure 11 Flow of water into a well with an unconfined or confined permeable layer

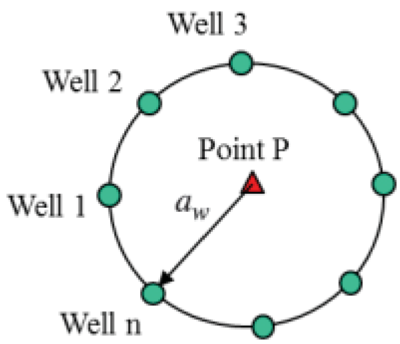


Figure 9 Multiwells in a circular arrangement

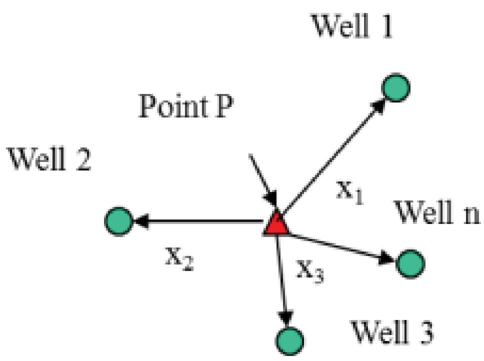


Figure 10 Multiwells in a random arrangement

Single well

$$R_i = C'(h_{w0} - h_{w1})\sqrt{k}$$

R_i = influence radius (m)

C' = 3000 for wells or 1500–2000 for single-line well points

h_{w0} = height of phreatic level from impermeable layer before pumping (m)

h_{w1} = height of the phreatic level at the edge of the well after pumping (m)

k = permeability of soil (m/s)

Multi-wells

In random arrangement,

$$Q_w = \frac{\pi k({h_{w0}}^2 - z^2)}{\ln R_i - (1/N_w)\ln(x_1 x_{2\ldots\ldots} x_{Nw})}$$

Where N_w is number of wells.

In circular arrangement,

$$Q_w = \frac{\pi k({h_{w0}}^2 - z^2)}{\ln R_i - \ln a_w}$$

Where z is depth of impervious layer.

Radius of site

$$r_0 = \sqrt{\frac{LB}{\pi}}$$

Where L and B are length and width of site.

Number of wells

$$N_w \geq \frac{Q_w}{Q_{w1}}$$

Spacing of wells

The spacing of well points can be estimated using below figure for well points in clean and uniform sand.

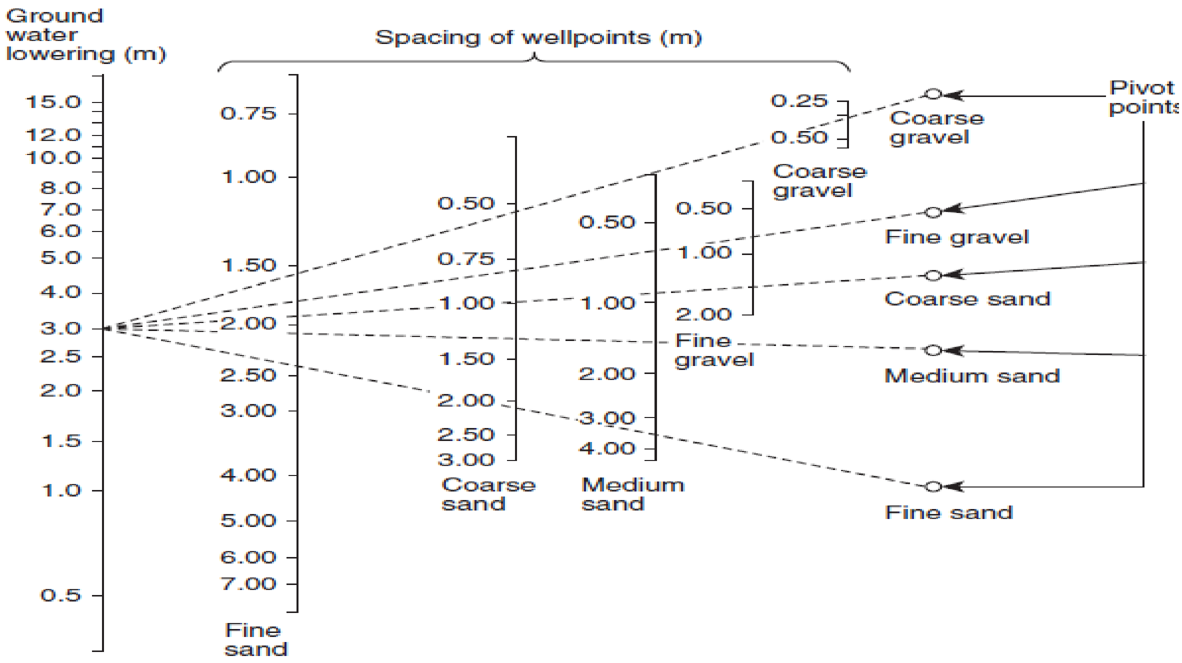


Figure 12 Design chart for well point spacing

2.5.3 Overexcavation and Replacement

2.5.3.1 Introduction

The basic concept of this method is to remove a problematic geo material and replace it with non-problematic fill. Replacing fill are rock, gravel, sand and chemically stabilized soils can be used as well. Onsite geomaterial may be excavated and the re-compacted back to the original location with and without addition of lime and cement. When the bottom of exaction is below the ground water table, dewatering is necessary

2.5.3.2 Suitability

Problematic soils are uncontrolled fill, loose sand and silt, soft soil and expansive soil, collapsible soil and liquefiable soil which may have less bearing capacity and excessive deformations. This method is often used for the following conditions:

- Limited area of excavation.
- Depth of improvement is less than 3m.

- No or limited shoring and dewatering is required.
- No existing structure is close to overexcavation area.
- Fill material is readily available and removed material is easily reused and disposed.

2.5.3.3 Application

Based upon problematic soil conditions this method is used to:

- Increase bearing capacity
- Reduce settlement
- Eliminate expansion and shrinkage of expansive soil
- Eliminate freeze-thaw effect of frozen soil

2.5.3.4 Limitations

Depending on site conditions, this method may be limited by:

- Deep excavation required
- High groundwater table
- Onsite or nearby existing structures and utility lines
- Limited truck access to the site
- Time

2.5.3.4 Principles

2.5.3.4.1 Stress Distribution

The basic principle of overexcavation and replacement is to eliminate potential problems by removing a problematic geomaterial and replacing it with non-problematic fill. The net pressure applied on the base of the footing is:

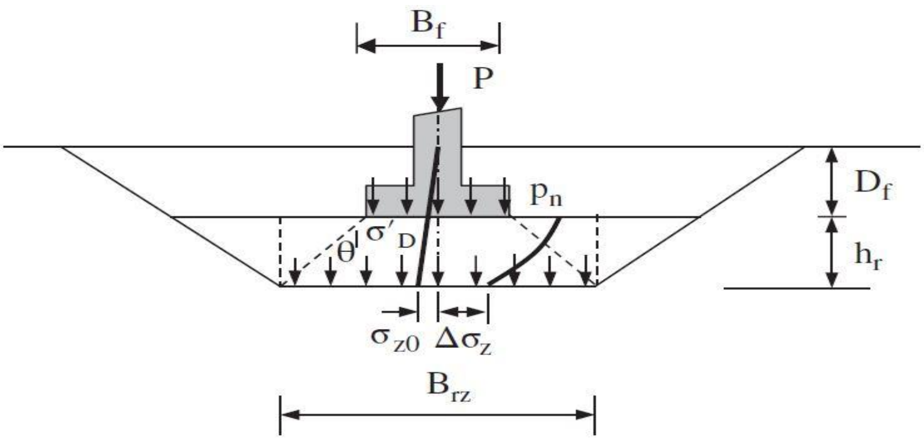


Figure 13 Stress distribution in replaced zone

$$p_n = \frac{P+W_f}{A_f} - \sigma'_D$$

Where P = column load applied on the footing

W_f = weight of the footing

A_f = cross-sectional area of the footing

σ'_D = effective overburden stress at the base of the footing

The additional vertical stresses at the center and bottom of the replaced zone induced by net pressure at the base of footing can be estimated by stress distribution as follows:

For rectangular footing,

$$\Delta\sigma_z = \frac{p_n A_f}{A_f} = \frac{p_n L_f B_f}{L_f{}' B_f{}'}$$

$$L_f{}' = L_f + 2h_r tan\theta$$

$$B_f{}' = B_f + 2h_r tan\theta$$

For circular footing,

$$\Delta\sigma_z = \frac{p_n A_f}{A_f} = \frac{p_n d'_{f^2}}{d'_{f^2}}$$

$$d_f{}' = d_f + 2h_r tan\theta$$

$$\Delta\sigma_z = \frac{p_n B_f}{B'_f}$$

Where,

A_f, B_f, L_f, d_f= area, width, length and diameter of footing respectively

$A_f{}', B_f{}', L_f{}', d_f{}'$= area, width, length and diameter of distributed foundation

h_f = thickness of replaced zone.

θ = distribution angle

2.5.3.4.2 Failure Modes

There are four possible failure modes in excavation and replacement.

- General shear failure
- Punching shear failure through replaced zone
- Failure of distributed foundation
- Punching shear failure of replaced zone

The mode of general failure within the replaced zone is likely develops under at least one of the following conditions: the fill is too weak, the area of footing is too small, the embedment depth of the footing is too shallow, and the applied load is too high.

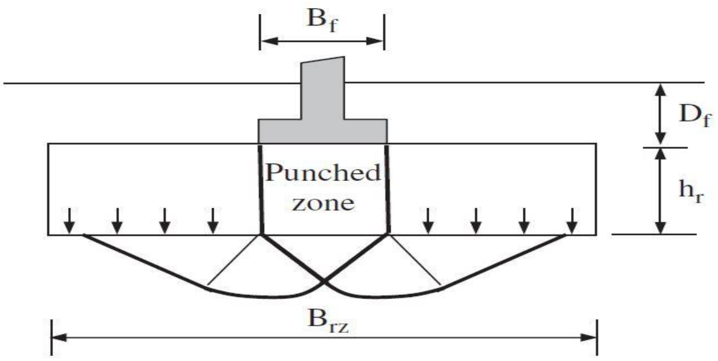


Figure 14 General failure within replaced zone

The mode of a possible punching failure through a replaced zone likely occurs when the thickness of the replaced zone is too thin and the underlying soil is too weak.

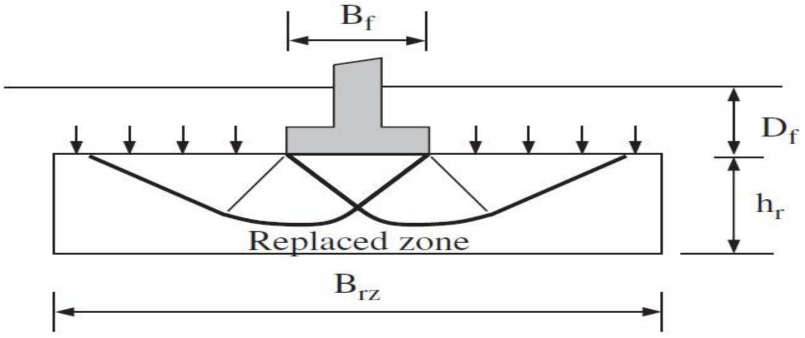


Figure 15 Punching failure through replaced zone

The distributed failure through a replaced zone is controlled by the strength of the underlying soil.

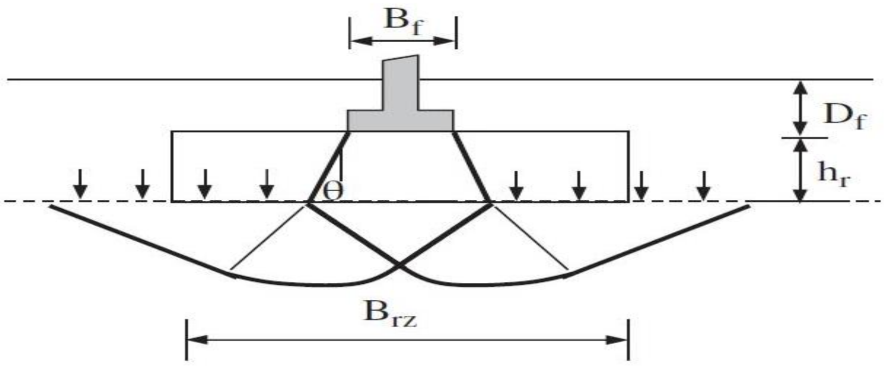


Figure 16 Distributed foundation failure

The Punching failure of the replaced zone into the underlying soil happens when the area of the replaced zone is too small and the underlying soil is too weak. This failure mode is mostly dominated by the area of the replaced zone.

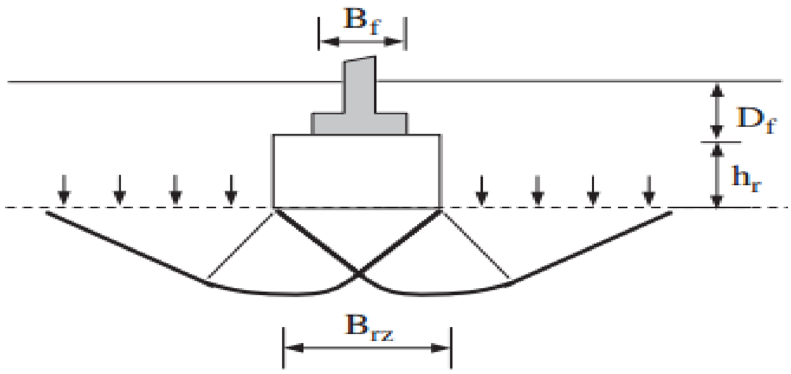


Figure 17 Punching failure of the replaced zone

2.5.3.5 Design Consideration

The following parameters should be determined during design:

- Shape and dimensions of footings
- Dimensions of replaced zone
- Applied load on footing
- In-situ geomaterial conditions like unit weight, water table and cohesion etc.
- Fill quality including strength and modulus of fill

2.5.3.5.1 General Failure within the Replaced Zone

This failure mode is used to determine the required strength of fill if footing parameters are fixed. The ultimate bearing capacity of fill can be estimated by:

$$q_{ult} = N_c d_c s_c c + 0.5\, \gamma' B_f N_\gamma s_\gamma d_\gamma + \sigma_z{}' N_q s_q d_q$$

c = cohesion of fill

γ' = effective unit weight of fill

σ_z' = effective overburden stress at the base of the footing

N_γ, N_q, N_c = bearing capacity factors

$s_c, s_{\gamma,} s_q$ = shape factors

$d_c, d_{\gamma,} d_q$ = depth factors

The shape and depth factors can be calculated by given below table:

Friction Angle	Shape Factor
$\varphi = 0°$	$s_c = 1 + 0.2\left({B_f}/{L_f}\right)$
	$s_\gamma = s_q = 1$
$\varphi = 10°$	$s_c = 1 + 0.2\, K_p({B_f}/{L_f})$
	$s_q = s_c = 1 + 0.1 K_p({B_f}/{L_f})$

Where $K_p = tan^2(45° + \frac{\varphi}{2})$

Friction Angle	Depth Factor
$\varphi = 0°$	$d_c = 1 + 0.2({D_f}/{L_f})$
	$d_\gamma = d_q = 1$
$\varphi \geq 10°$	$d_c = 1 + 0.2\sqrt{K_p}({D_f}/{L_f})$
	$\mathrm{d}_\gamma = \mathrm{d}_q = 1 + 0.2\sqrt{\mathrm{K_p}}({\mathrm{D_f}}/{\mathrm{L_f}})$

Where $K_p = tan^2(45° + \frac{\varphi}{2})$

2.5.3.5.2 Punching Failure through Replaced Zone

Meyerhof and Hanna (1978) proposed a method to calculate the ultimate bearing capacity of one strong soil layer over a weak soil layer.

$$q_{ult} = q_b + \frac{U_p P_h tan\varphi_1 + U_p h_r c_1 - W_{pz}}{A_f}$$

Where,

q_b = Ultimate bearing capacity of soil beneath replaced zone

U_p, h_r = Perimeter and length of punched zone

W_{pz} = Weight of punched zone.

σ_z' = Effective overburden stress at the base of the footing.

φ_1, c_1 = Friction angle and cohesion of replaced zone.

P_h = Lateral earth pressure thurst acting along perimeter surface

$$P_h = K_s(\gamma'_1 D_f h_r + 0.5\gamma_1' {h_r}^2)$$

Where K_s is estimated by graph in which

$$q_1 = c_1 N_{c1} + 0.5\gamma_1' N_{\gamma 1}$$

$$q_2 = c_2 N_{c2} + 0.5\gamma_2' N_{\gamma 2}$$

Where,

$N_{c1}, N_{\gamma 1}$ = Bearing capacity factors of fill

$N_{c2}, N_{\gamma 2}$ = Bearing capacity factors of underlying soil

c_1, c_2 = Cohesion of fill and underlying soil respectively

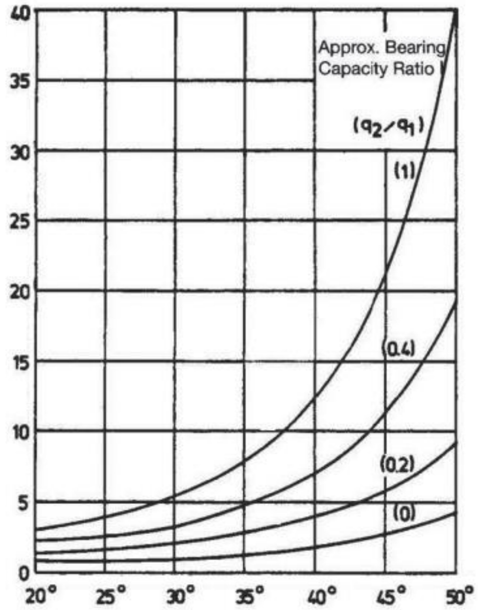


Figure 18 Coefficient of punching shear

2.5.3.5.3 Punching Failure through Replaced Zone

The ultimate bearing capacity of distributed foundation as the rigid footing on the underlying soil can be calculated as follows:

$$q_{ult} = N_{c2} d_c' s_c' c_2 + 0.5\, \gamma'_2 B_f' N_{\gamma 2} s'_\gamma d_\gamma' + \sigma_{zO}' N_{q2} s_q' d_q'$$

Where,

γ'_2 = Effective unit weight of underlying soil

σ_z' = Effective overburden stress at the bottom of replaced zone.

$N_{\gamma 2}, N_{q2}, N_{c2}$ = Bearing capacity factors

s_c', s_γ', s_q' = shape factors of distributed foundation

d'_c, d'_γ, d'_q = depth factors of distributed foundation

The formula to calculate bearing capacity at the base of foundation is

$$q_{ult} = (q_b - h_r \gamma'_1) \frac{A_f'}{A_f}$$

Where,

A_f', A_f = Area of footing and distributed foundation

γ'_1 = Effective unit weight of fill

2.5.3.5.4 Minimum Bearing Capacity and Factor of Safety

The minimum ultimate bearing capacity $q_{ult(min)}$ is the least of all the bearing capacities calculated based on the failure modes. The minimum bearing capacity and the corresponding failure mode control the design in terms of the bearing capacity.

$$FS = \frac{q_{ult(min)}}{p}$$

The factor of safety (FS) should be greater than the required factor of safety.

2.5.4 Stone columns

2.5.4.1 Introduction

Deep replacement methods improve the ground to a great depth by partially excavating or displacing problematic soils, which are replaced with better quality or densified fill. Stone column construction involves partial replacement of unsuitable subsurface soils with a compacted vertical column of stone that usually completely penetrates the weak strata. When jetting water is used the process is named wet process. When used without jetting water in partially saturated soils such as old rubble etc., this method is called dry process.

2.5.4.2 Suitability

This method is used to increase shear strength of cohesive soil less than 15kPa. Stone column technology is suitable for soft to stiff clays, loose silt and sand to dense sand, and uncontrolled fill. This method may cause difficulties when ground water table is high. The depth of improvement is approximately 10-20m.

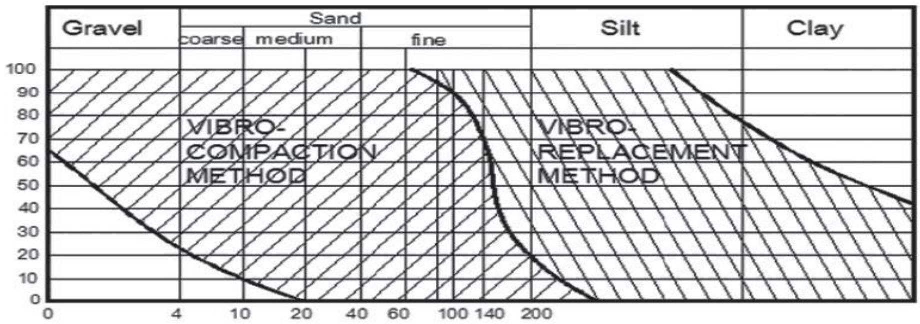


Figure 19 Soil suitable for vibro-compaction and vibro-replacement

2.5.4.3 Applications

Columns with surrounding soils to form a composite ground or foundation can increase bearing capacity, reduce increase shear strength for slope stability, and increase resistance to liquefaction. Most of the deep replacement columns can be used to support industrial, residential, retail buildings, storage tanks, embankments and walls, bridge abutments, roadway widening, wind turbines, and utilities and pipelines.

2.5.4.4 Advantages and Limitations

Stone columns have a relatively rapid installation procedure, have a high level of compaction, can pre-stress surrounding soil, and have an easy QC/QA procedure. However aggregate columns have limited improvement depth and are difficult to be installed in clean sands with a high groundwater table. Stone column provide better drainage system. This technique is not suitable when soil has shear strength less than 15kPa due to excessive bulging near the ground surface.

2.5.4.5 Failure Modes

When the applied axial load is higher than the strength of column the will crush. The shear failure may occur in stone column. The punching failure may happen to short granular or concrete columns without an end-bearing layer. Bulging failure more likely happens to granular columns in soft soils within the top portion of two to three times the diameter of the column.

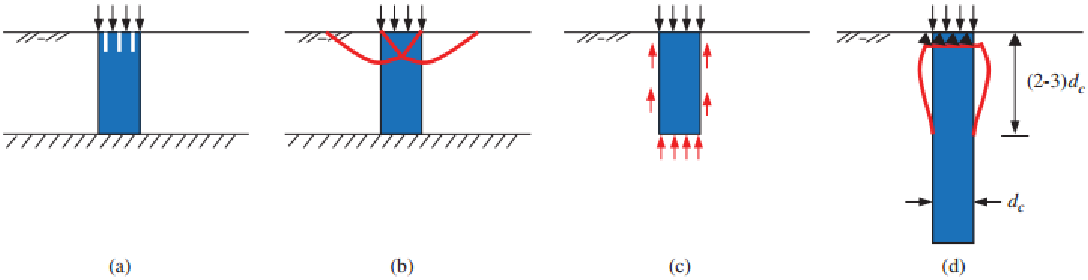


Figure 20 Possible failure modes of individual columns subjected to vertical loads: (a) crushing, (b) shear, (c) punching and (d) bulging

2.5.4.6 Design Considerations

2.5.4.6.1 Area Replacement Ratio

When columns are installed, the area replacement ratio is defined as the ratio of the cross-sectional area of a column to the tributary area of the column

$$a_s = \left(\frac{A_s}{A_c}\right) = C\left(\frac{d_c}{s}\right)^2$$

$a_s = area\ replacement\ ratio$

$A_c = cross\ sectional\ area\ of\ column$

$d_c = diameter\ of\ column$

$A_s = tributry\ \ area\ of\ column$

$s = center\ to\ center\ spacing\ between\ the\ column$

$C = constant$

2.5.4.6.2 Depth of improvement

The depth of improvement depends upon location of firm stratum, when it exists at a relatively shallow depth, the depth of improvement should reach this stratum. If it exists on higher depth, improvement depth is determined to meet the performance requirement.

2.5.4.6.3 Composite foundation

Under rigid loading, stress distribution on the columns is based on the force equilibrium. The following relationship can be established.

$$\Delta\sigma_z A = \Delta\sigma_s(A_e - A_c) + \Delta\sigma_c A_c$$

$$\Delta\sigma_z a_s = \Delta\sigma_s(1 - a_s) + \Delta\sigma_c a_s$$

$$\Delta\sigma_z = [(1 - a_s) + na_s]\Delta\sigma_s$$

$$n\Delta\sigma_s = \Delta\sigma_c \qquad , \qquad \Delta\sigma_s = \mu\Delta\sigma_z$$

$$\mu = \frac{1}{1 + (n-1)a_s}$$

$\Delta\sigma_z$ =Average vertical stress applied composite foundation

A_e = Tributry area Of one column

A_c = Cross sectional area of column

$\Delta\sigma_c$=Vertical stress on the column

$\Delta\sigma_s$ = Vertical stress on the column

$n = stress\ conentration\ factor$

2.5.4.6.4 Bearing and Load Capacity

The ultimate bearing capacities of one column and surroundings are estimated by:

$$q_{ult,c} = 20\, c_u\ , \qquad q_{ult,s} = 5\, c_u$$

$$q_{ult} = q_{ult,c} a_s + q_{ult,s}(1 - a_s)$$

$$c_{avg} = c_s(1 - a_s)$$

$$\emptyset_{avg} = \arctan[a_s tan\emptyset_c + (1 + a_s) tan\emptyset_s]$$

$c_{avg} = Average\ Cohesion$

$c_s = soil\ cohesion$

$\emptyset_s = soil\ friction\ angle$

$\emptyset_c = column\ friction\ angle$

$\emptyset_{avg} = average\ friction\ angle$

Ultimate load capacity is calculated by:

$$q_{ult} = 9c_u \left(\frac{1 + sin\emptyset_s}{1 - sin\emptyset_s}\right)$$

$$q_{ult} = N_c\ c$$

Where $N_c = 18 \quad when\ PI > 30,$

$$N_c = 22 \quad when\ PI \leq 30$$

And,

$$q_{ult} = \sigma_3 tan^2\beta + 2c_{avg} tan\beta$$

$$\sigma_3 = 9c_u$$

2.5.4.7 Design Parameters

These are the design parameters of stone columns:

- Soil type
- Depth of ground water table
- Undrained shear strength
- Required Allowable Bearing Capacity
- Allowable Settlement

2.5.5 Preloading

2.5.5.1 Introduction

The basic concept of this technique is to reduce void ratio of geomaterial through consolidation by applying loads on the ground surface for the certain time period then removing it for the construction of permanent structure. During the preloading, settlement develops with loading and time. When the fill is removed at the end of preloading, there is a rebound. Construction of the permanent structure induces new settlement due to the increase of the load. For soft clays, the consolidation may take longer time to complete due to their low permeability. If the time for preloading and construction of the structure exceeds the available time, vertical drains can be installed to shorten drainage distance thus accelerating the rate of consolidation and reducing the time for soil consolidation and settlement.

2.5.5.2 Suitability

Preloading is often cost effective to improve saturated, low strength, and highly compressible clays and silts when time is not a major concern. Vertical drains can be used to shorten the time for preloading if time is a major concern. Vertical drains are typically installed to a depth of 30m. Preloading is effective when the loading is higher than soil pre-consolidation stress. Fill preloading is more suitable if fill material is inexpensive and readily available.

2.5.5.3 Applications

This method has many applications in highways, buildings, airports, land reclamation and storage tanks.

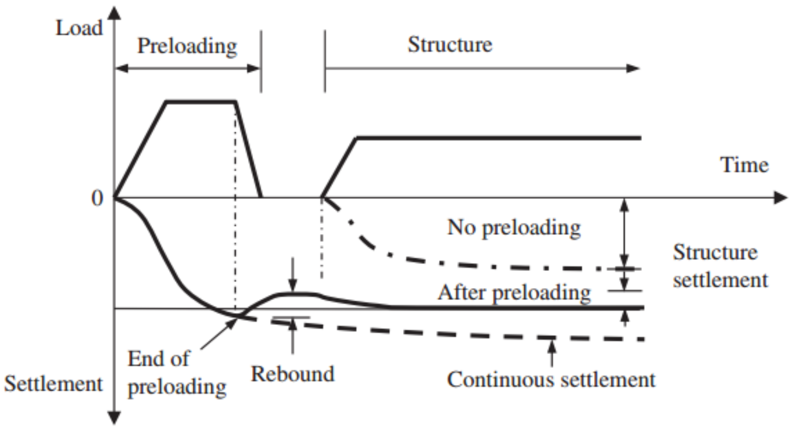


Figure 21 Fill preloading and application

2.5.5.4 Advantages and Limitations

The method is limited when the time is very short because consolidation is required longer time for low permeable soil however to accelerate rate of consolidation, vertical drains are used. Due to induced ground movement and transportation of material, this method does not work.

2.5.5.5 Principles

2.5.5.5.1 Pre-compression

The basic principle of this method is to increase shear strength and reduce future settlement by removing void ratio.

2.5.5.5.2 Stress and Ground Movement

Fill preloading induces positive excess pore water pressure and induces unequal vertical as well as horizontal stresses .Outward movement induced in the horizontal direction through fill preloading.

2.5.5.5.3 Preloading Consolidation Theory

Settlement due to applied foundation load:

$$S_F = \frac{H}{1+e_0}\log\left(\frac{P_0' + \Delta P_F}{P_0'}\right)$$

Settlement due to foundation load and surcharge load:

$$S_{F+S} = \frac{H}{1+e_0}\log\left(\frac{P_0' + [\Delta P_F + \Delta P_s]}{P_0'}\right)$$

The degree of consolidation after load application:

$$U = \frac{S_F}{S_{F+S}} = \frac{\frac{H}{1+e_0}\log\left(\frac{P_0' + \Delta P_F}{P_0'}\right)}{\frac{H}{1+e_0}\log\left(\frac{P_0' + [\Delta P_F + \Delta P_s]}{P_0'}\right)}$$

H= $Thickness\ of\ layer$

$e_0 = Initial\ void\ ratio$

$P_0' = Intial\ effective\ stress$

$\Delta P_F, \Delta P_S = load\ of\ foundation\ and\ load\ of\ surcharge\ respectively$

$S_{F+S}, S_F =$ Settlement due to $\Delta P_F\ and\ \Delta P_S$ respectively

Time rate of consolidation with vertical drainage:

$$T_v = \frac{c_v t}{H^2}$$

For $U_v \leq 52.6\%$

$$T_v = \frac{\pi}{4}(\frac{U_v}{100})^2$$

For $U_v > 52.6\%$

$$T_v = 1.781 - 0.933 \log_{10}(100 - U_v)$$

2.5.5.6 Prefabricated Vertical Drains

Prefabricated vertical drains are installed in triangular or square pattern. The equivalent diameter can be approximated by:

$$d_e = 1.13s \qquad for\ square\ pattern$$

$$d_e = 1.06s \qquad for\ triangular\ pattern$$

$d_e = equivalent\ influence\ diameter$

$s = spacing\ between\ two\ adjacent\ vertical\ drains$

2.5.5.7 Degree of Radial consolidation:

$$U_r = 1 - e^{\left(-\frac{8T_r}{m}\right)}$$

$$m = n^2 \frac{\ln(n)}{(n^2-1)} - \frac{3n^2-1}{4n^2} \text{ And } n = \frac{d_e}{2r_w}$$

Where,

$U_r = Average\ degree\ of\ consolidation\ due\ to\ radial\ flow$

$T_r = Time\ factor\ due\ to\ radial\ flow$ i.e. ($T_r = \frac{c_r t}{{d_e}^2}$)

$r_w = radius\ of\ sand\ drain$

Overall degree of consolidation can be estimated by (Carillo,1942)

$$U_{vr} = 1 - (1 - U_r)(1 - U_v)$$

U_{vr} = Overall degree of consolidation

$U_r = Degree\ of\ consolidation\ in\ radial\ consolidation$

$U_r = Degree\ of\ consolidation\ in\ vertical\ consolidation$

2.5.6 Grouting

2.5.6.1 Introduction

Grouting is the method of injecting pump able materials (lime or cement) into ground formation to change its physical characteristics. Grouting can be used in rock and soil. The main objectives of grouting are densification, prevent settlement, mitigate liquefaction, and reduction of permeability and water control. There are five main type of grouting which are given in table below.

Grouting type	Function
Permeation grouting	Voids in the soil are filled by this method
Compaction grouting	It is used to densify soils by injecting stiff and high viscosity grout which cause displacement of particles
Hydro-fracture grouting	The soil mass is fractured and the fractures are injected with stiff grout
Compensation grouting	Ground loss that happens as the result of construction works is compensated
Jet grouting	It employs grout to erode the soil at depths and then mix it with grout to form columns or walls

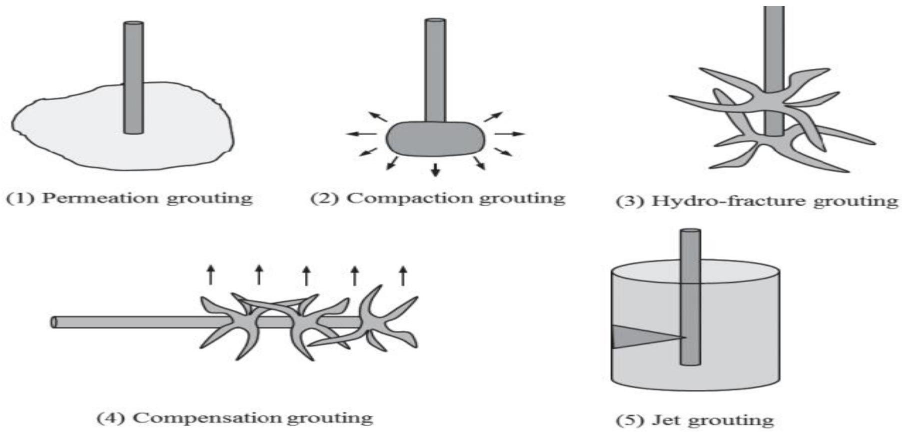


Figure 22 Type of Grouting

2.5.6.2 Suitability

The permeation grouting (cement/slurry) is suitable for cohesion less soil. The cement slurry grouting is more suitable for gravel while the chemical grouting is more suitable for sand. Compaction grouting is mostly used for sand. The hydro-fracture grouting is suitable for sand, silt, and clay. The jet grouting method is suitable for all kinds of soil types. Permeation grouting, compaction grouting, and hydro-fracture grouting have also been used for decomposed rock and fissured rock.

2.5.6.3 Application

Grouting is used to achieve the following modification in soil response and properties:

- Densification of sands
- Raising settled structures
- Controlling settlement
- Underpinning
- Support on excavation sides
- Protection of structures while tunneling
- Liquefaction mitigation
- Control on water

2.5.6.4 Advantages and Limitations

Grouting has the following advantages as compared with alternate technologies:

- No need for removal and replacement
- Effective for underpinning and protecting existing structures
- Easy to access and operate within constrained space

- Low mobilization cost

The limitations associated with grouting are:

- Quantity of grout is hard to estimate
- Effectiveness of some applications cannot be predicted
- Area of improvement is sometimes uncertain.
- Grouting may cause ground movement and distresses to existing structures.
- Certain chemical grouts may contain toxicity and have adverse impact to groundwater and underground environment
- Specialty contractors are required for the operation.

In our project, we will only discuss the design and procedure of permeation grouting.

2.5.6.5 Design Parameters

The design parameters for permeation grouting include:

- Soil type, density, and D_{15} or rock type and width of fissure
- Depth to be improved
- Groundwater table
- Type and properties of grout including grout D_{85} for soil or D_{95} for rock, grout unit weight, and grout viscosity
- Grout penetration radius and injection rate

2.5.6.5.1 Groutability

The following two parameters were proposed by Mitchell and Katti (1981) based on Terzaghi's filter criteria:

$$N_{gs} = \frac{(D_{15})_{soil}}{(D_{85})_{grout}} \quad \text{(For soil)}$$

$$N_{gr} = \frac{t_f}{(D_{85})_{grout}} \quad \text{(For rock)}$$

Where $N_{gs} =$ groutability of soil

$N_{gr} =$ groutability of rock

$(D_{15})_{soil}$ = soil particle size corresponding to 15% passing

$(D_{85})_{grout}$ = grout particle size corresponding to 85% passing

t_f = width of fissure in rock

In soil, cement grout is groutable when $N_{gs} > 11$ but consistently groutable when $N_{gs} > 24$.

In rock, cement grout is groutable when N_{gr} > 2 but consistently groutable when N_{gr}> 5

2.5.6.5.2 Grout head and Pressure

It can be estimated by the following formula (Raffle and Greenwood, 1961):

$$\Delta h_w = \frac{Q_g}{4\pi k}[\beta_g\left(\frac{1}{r_0}+\frac{1}{R}\right)+\frac{1}{R}]$$

Where k = permeability of soil

β_g = grout to water viscosity ratio

$r_0 = 0.5\sqrt{Ld}$ = radius of spherical injection source

R = radius of penetration in the ground

Q_g = rate of grout injection

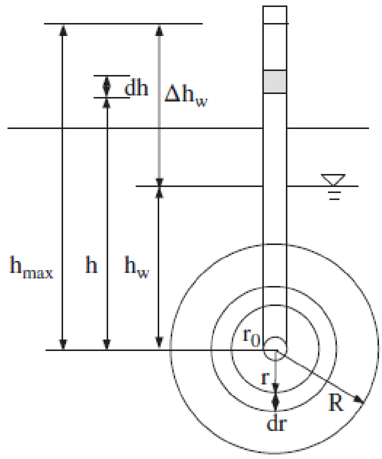


Figure 23 Model of injection

The required grout pressure can be calculated as follows:

$$p_g = \gamma_w(h_w + \Delta h_w) - \gamma_g h_{gp}$$

Where p_g = unit weight of grout

γ_w = unit weight of water

h_{gp}= height of the grout to the injection point

h_w= height of the groundwater table to the injection point

2.5.6.5.2 Allowable Grout Pressure

The allowable injection pressure can be estimated as follows (Chinese Ground Improvement Manual Committee, 1988):

$$p_{ga} = 100(\alpha_p p + C_{gs}\beta_{gm}\lambda_{sc}z)$$

p_{ga} = allowable grout pressure (kPa)

α_p = surcharge factor

p = surface surcharge (kPa)

C_{gs} = grouting sequence factor

β_{gm}= grouting method factor

z = depth of the injection point from the ground surface (m)

2.5.7 Deep Soil Mixing

2.5.7.1 Introduction

The deep soil mixing (DSM) method mixes in situ soil with a hardening agent (cement, lime, slag, or other binders) at depths by augers. Deep mixing can be accomplished by a wet or dry method. A wet method uses binder in a slurry form while a dry method uses the binder in a powder form.

2.5.7.2 Suitability

Deep mixing has been mostly used to improve soft cohesive soils, but sometimes it is used to reduce permeability and mitigate liquefaction of cohesionless soils. Deep mixing becomes difficult if the ground is very stiff, very dense, and contains boulders or other obstructions. Typically, deep mixing requires unrestricted site access and overhead clearance due to large equipment used in most projects. Deep mixing can reach a depth of up to 70 m in marine work and 30 m for land operations.

Property	Favorable Soil Chemistry
pH	Should be greater than 5
Natural water content	Should be less than 200% (dry method) and less than 60% (wet method)
Organic content	Should be less than 6% (wet method)
Loss on ignition	Should be less than 10%
Humus content[a]	Should be less than 1.0%
Electrical conductivity	Should be greater than 0.04 mS/mm

Figure 24 Table showing soil properties favorable for DSM

2.5.7.3 Applications

DSM has been used for many applications in soft soils:

- Support of superstructures, including buildings, walls, embankments etc.
- Waterfront and marine applications including quay walls, wharf structures, and breakwaters
- Stabilization of slopes
- Lateral support
- Containment of water and pollutant
- Liquefaction mitigation
- Vibration reduction

2.5.7.4 Advantages and Limitations

The deep soil mixing method has the following advantages:

- Applicable for most soil types
- Installed at great depths
- Relatively fast installation
- Low noise and vibration level
- Formation of a DM wall for earth retaining and water barrier at the same location and time
- Less spoil soil, especially for the dry method

However, the deep mixing method may have the following limitations:

- Relatively high mobilization cost
- High variability in column quality
- Lack of standardized quality control methods

2.5.7.4 Principles

2.5.7.4.1 Chemical Reaction

Lime stabilization involves hydration of binder, ion exchange reaction, and formation of pozzolanic reaction products. When quicklime (CaO) is mixed with a moist soil, it absorbs the moisture in the soil and has the following chemical reaction:

$$CaO + H_2O = Ca(OH)_2 + Heat$$

This chemical reaction generates heat and reduces the moisture in the soil. The reduction of the moisture in the soil increases the strength of the soil. This chemical process is also called the hydration of binder. The products produced from this process are called the pozzolanic reaction products. These products turn the soil into a hardened solid with high strength and stiffness.

2.5.7.4.2 Possible Failure Modes

Deep mixed columns have been installed in the field in a form of individual columns, blocks, walls, or grids. When individual columns are subjected to a vertical compressive load, they may have crushing, shear, and punching failures. Columns may crush when the applied load is higher than the capacity of the columns. This failure more likely happens to DSM columns because they are relatively brittle. The shear failure may happen to DSM columns at low binder content. The punching failure may happen to short DSM columns without an end-bearing layer.

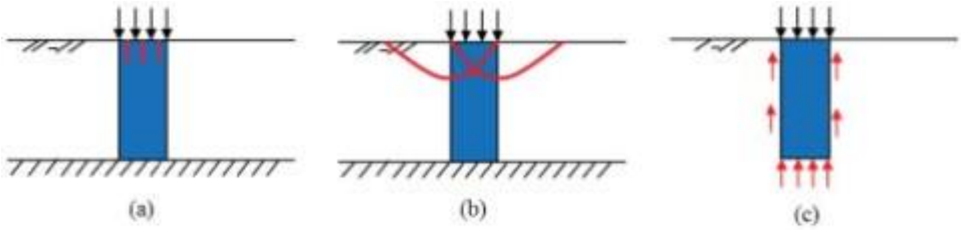


Figure 25 Failure modes of DM columns under vertical loads: (a) crushing, (b) shear and (c) punching failure

2.5.7.4.3 Stress Transfer

Due to the modulus difference between DSM columns and the surrounding soil, higher stresses develop on DM columns than those on the surrounding soil when the composite soil foundation is subjected to applied loads. This stress difference is often described by a stress concentration ratio, which is defined as the ratio of the stress on the column to that on the surrounding soil.

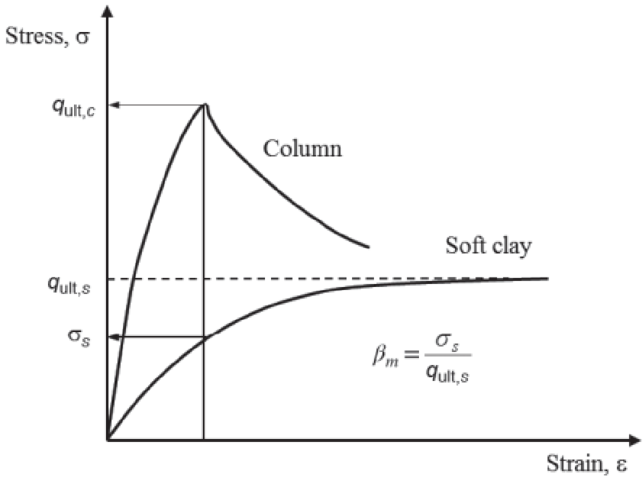


Figure 26 Stress–strain relationships of the column and the soil

The graph shows the stress–strain relationships for the column and the soil. At an equal strain, more and more stresses are transferred to the column with the increase of the strain until the column reaches the ultimate strength. At the ultimate strength of the column, the soil strength is not fully mobilized. The ratio of the mobilized stress to the ultimate bearing capacity of the soil is referred to as the mobilization factor of the bearing capacity for the soil, which is defined as follows:

$$\beta_m = \frac{\sigma_s}{q_{ult,s}}$$

This mobilization factor depends on the length of the column and the end-bearing condition.

2.5.7.5 Design Consideration

2.5.7.5.1 Design Parameter

When deep mixed columns or walls are used for foundation support, the design may include the following parameters:

- Soil type, natural moisture content, organic content, groundwater table, permeability or coefficient of consolidation and soil strength and modulus
- Depth of improvement
- Project requirements (allowable bearing capacity, tolerable settlement, factor of safety against slope failure for embankments)
- Loading condition (applied pressure)
- Type of binder (lime, cement, lime–cement, and other binder)
- Method of installation (dry or wet method)
- Binder content
- Required unconfined compressive strength of stabilized soil
- Size and pattern of columns

2.5.7.5.1 Bearing Capacity

The ultimate load capacity of an individual DSM column depends on the strength of the column, the side friction between the column and the soil, and the toe resistance of the column. The ultimate load capacity of an individual column can be estimated as the lesser of the following two capacities (Han et al, 2002):

$$Q_{ult,c} = q_{u,c} A_c$$

$$q_{u,c} = \alpha_1 \alpha_2 q_{u1}$$

$$Q_{ult,c} = f_s U_c L_c + q_{tm} A_c$$

$$q_{tm} = \lambda_E q_t$$

Where,

$q_{u,c}$ = field unconfined compressive strength of the column

q_{u1} = laboratory unconfined compressive strength of the stabilized soil sample

α_1 = laboratory to field strength correction factor

α_2 = small cored sample to full-scale column correction factor

A_c = cross-sectional area of columns

f_s = average skin friction between the column and the surrounding soil

U_c = perimeter of the column

L_c = column length

λ_E = mobilization factor of the end-bearing

q_{tm} = modified end-bearing capacity of the column toe

q_t = end bearing capacity, estimated based on pile toe bearing capacity

The Building Center of Japan (1997) provides the following guidelines for the skin friction and the end bearing capacity:

For clayey soil:

$$f_s = C_u$$

$$q_{tm} = 6C_{ut}$$

For sandy soil:

$$f_s = \frac{10N}{3} \quad \text{(kPa)}$$

$$q_{tm} = 75N_t \quad \text{(kPa)}$$

C_u = average cohesion of the soil along the column shaft (kPa)

C_{ut} = cohesion of the soil below the column toe (kPa)

N = average SPT N value of the soil along the column shaft

N_t = SPT N value below the column toe

The ultimate bearing capacity of a DM column composite foundation (q_{ult}) can be calculated as follows (Han et al, 2002):

$$q_{ult} = a_s \frac{Q_{ult,c}}{A_c} + \beta_m (1 - a_s) q_{ult,s}$$

a_s = area replacement ratio of the column to the soil

$q_{ult,s}$ = ultimate bearing capacity of the surrounding soil

β_m = mobilization factor of the bearing capacity of the soil.

A factor of safety of 2.0–3.0 may be used to calculate the allowable bearing capacity.

2.5.7.5.2 Settlement

Settlement of DSM column can be found using stress reduction method:

$$S' = \frac{1}{1 + a_s(n-1)} S$$

Where,

S = settlement of natural ground

a_s = area replacement ratio of columns

n = stress concentration ratio.

The stress concentration ratio n can be estimated using the below design chart:

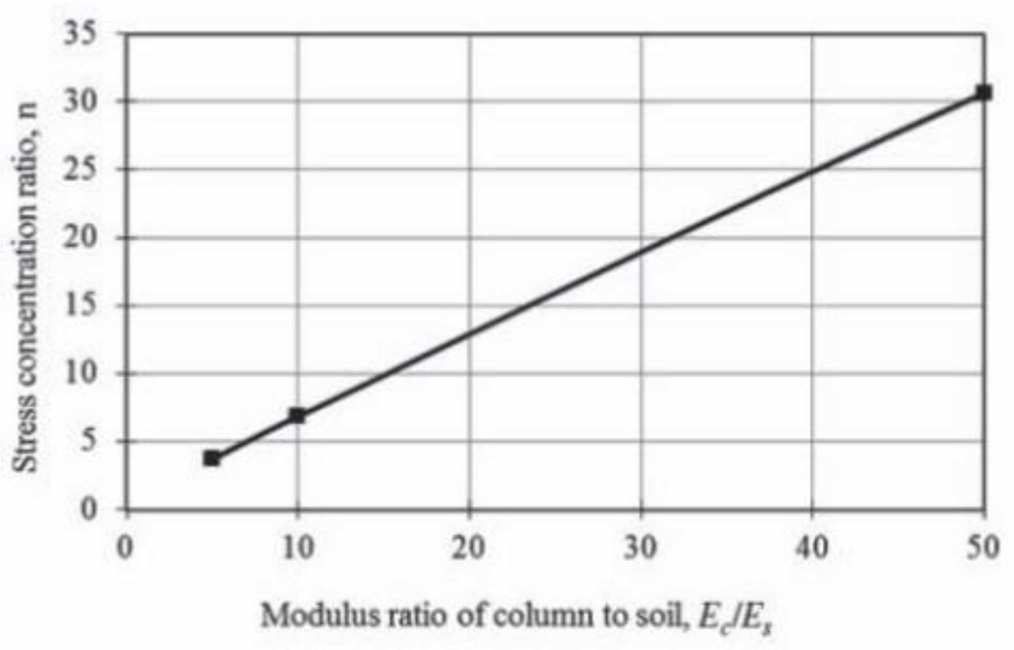


Figure 27 stress concentration ratio versus modulus ratio column to soil

CHAPTER 3

METHODOLOGY

3.1 Introduction

This chapter explains the methodology of our Final Year Project in which we adopted the following steps to achieve our project objective. Now that we have completed our literature review and established all probable theories applicable we move on to the next step. Please note that the prime objective of this project is the ground improvement and the end product being the computer software, constant references from the previous chapter are used here.

In this chapter we will explain the steps and strategies to attain our goals. After having completed the literature review we found that only few techniques are available due to their applicability and effectiveness in Pakistan. There are many methods to improve the soil which depends upon soil strata, ground water table and site constraints etc.

Our methodology can be divided into the following major parts:

1. Bearing capacity
2. Settlement
3. Development of individual algorithm of techniques
4. Development of overall algorithm of the program
5. Development of computer software
6. Troubleshooting of program code
7. Verification of the software with different examples

3.2 Bearing Capacity

3.2.1 Shallow Foundation

In first step, we have calculated bearing capacity (qu) based upon Terzaghi's Bearing Capacity approach for shallow foundation by using different parameters as listed below:

- Saturated Unit Weight
- GWT depth
- Dry unit weight

Evaluate bearing capacity factors and overburden pressure by using cohesion and friction angle. Identify the geometry of foundation such as square, rectangular, continues or circular. Estimate allowable load capacity by dividing required factor of safety.

3.2.1 Deep Foundation

We have only done analysis of the deep foundation (driven piles). If the ground conditions are suitable, we will go for the deep foundation. At first, the pile load capacity is calculated using its dimensions including length and diameter. After that the efficiency of group piles is determined. However, this is not the primary goal of our software but is included as a secondary option. This is because of the reason that mostly ground improvement is just required for the shallow depth. In case, greater depth is required to be improved, we switch pile foundation (generally speaking) because of its economic feasibility.

3.3 Settlement

We have estimated the settlement of shallow foundation by bearing capacity theories of settlement. Settlement of fine grained and coarse grained soil is calculated using Terzaghi and Peck method, and Meyerhoff's method. For this purpose, we have used the empirical relationship of N_{60} with friction angle and cohesion from the literature review in chapter 2. This N-value is taken from the geo technical investigation report.

3.3 Development of Individual Algorithm of Techniques

There are many methods to improve the soil which depends upon soil strata, ground water table and site constraints. The techniques which we have studied and make individual algorithm are listed below:

- Deep Dynamic Compaction
- Stone Column
- Fill Preloading
- Grouting
- Deep Soil mixing
- Over excavation and Replacement
- Piles Foundation

The parameters which have been used in different site improvement techniques were identified at first. We wrote these different inputs which based upon soil strata type and ground water table depth. And then the dependent and independent variables were defined which would be used for coding hereafter.

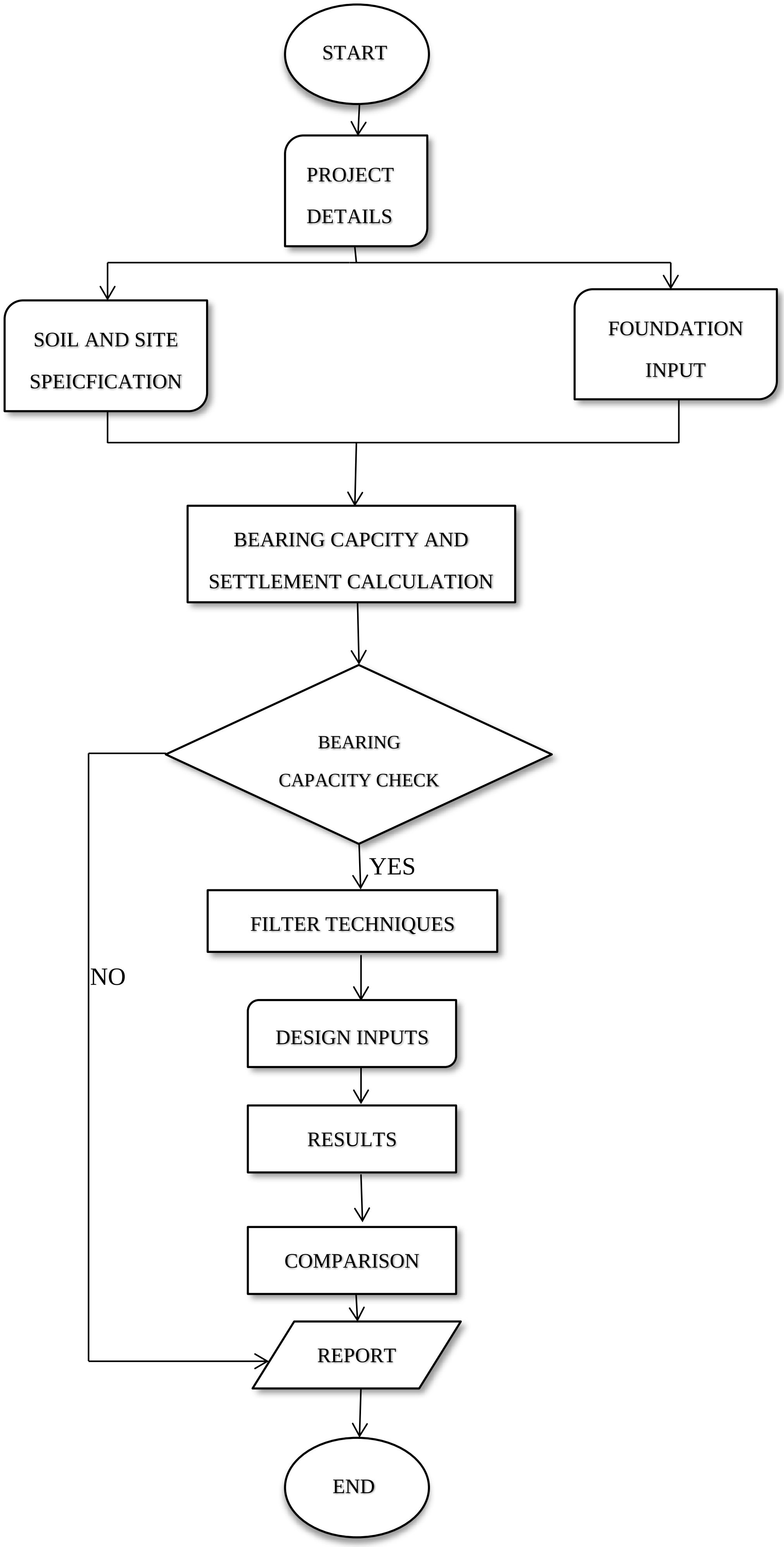
We then developed the individual algorithm by connecting them with settlement and bearing capacity factors, so that we would be able to check performance of feasible technique. The programs were established so that it take minimum input and maximize automation in order to make user friendly. We established the algorithm of each technique by relating different variables.

3.3 Development of Overall Algorithm of Program

Ground improvement methods classification has its reasoning and desirable features but also have some constraints. This situation results from the fact that several ground improvement methods can fit in one or more categories. Therefore it was required to combine them based upon soil properties.

We have combine all the individual technique with design inputs, bearing capacity, settlement and along each other to develop overall framework of the software program. After integration of all individual algorithms, we have checked for possible error and bugs in the algorithm.

The flowcharts of the combined algorithm are given below:


START
PROJECT DETAILS
SOIL AND SITE SPEICFICATION
FOUNDATION INPUT
BEARING CAPCITY AND SETTLEMENT CALCULATION
BEARING CAPACITY CHECK
YES
NO
FILTER TECHNIQUES
DESIGN INPUTS
RESULTS
COMPARISON
REPORT
END

3.4 Development of Computer Software

After the formation of all combine algorithms we started off with the programming of these into a software code. The language we decided to work on was C# as it the modern form of Visual Basic with much advance control and syntax. We used Microsoft Visual Studio as the development tool for this software. The entire algorithm formed was converted in to this program code. In the following chapter we will discuss more about the capabilities and operation of the software.

3.5 Troubleshooting of the Program Code

After developing the software, we trouble shoot the program code for the possible errors and bugs. All the error and bugs are removed and verified the code with short examples from the literature. We fixed all the problems and error of the program code, and maintain little margin of error from solved examples. Finally, we improved the program with necessary modifications and finalized it with important graphic design properties.

3.6 Verification with Examples

The last stage of this project was the verification of the software code by the help of various examples found in the literature along with some real life examples and cases we found during our study period. The answers of these examples were compared with the hand calculation and the software after which the deviation in these results was checked which came out be negligible. These examples are further discussed in the following chapters.

CHAPTER 4

SOFTWARE

4.1 Introduction

Our project mainly revolves around ground improvement using as many as possible geotechnical theories. As manual hand calculation can be too long and cumbersome, therefore, automation was the way to go. Calculations for the problems and various parameters throughout the geotechnical analysis require iterations many times; therefore a program development was the solution.

The preliminary yet extensive research and development was carried out using the literature review. Initially the algorithms were developed and utilized to achieve our objectives, but later on, developed algorithm were used to develop an application which is a stand-alone program.

4.2 Microsoft Visual Studio

The logics and algorithms were developed and imported to Microsoft Visual Studio. Visual Studio provided a platform to write the entire algorithm on C#, and develop an application based on Windows forms. This platform allowed us the manipulation and creation of the customized user interface and the division of program into multiple modules, thereby simplifying the task yet increasing the overall efficiency.

4.2.1 Technical Specifications

Language: C# (C Sharp)

GUI: Windows Forms

Modules: 8(~5000+ lines of code)

Development Tool: Microsoft Visual Studio Community Edition 2015

4.2.2 Elements of the Program

The program is the composite of all the geotechnical theories pertaining to bearing capacity, settlement and ground improvement. Like all program, this one is also divided into multiple components/modules. Each component has its own set of functions with further sub divisions. We will be detailing the functionalities and how to proceed with program subsequently.

Project Details

In the project detail screen, the inputs are the project name, organization, user name, assignment ID and date. The dropdown for soil types includes such as:

- Weak and wet fine grained soils
- Unsaturated loose granular soils
- Saturated loose granular soils
- Problematic soils i.e. expansive soils, collapsible soil
- Voids and sinkholes
- Rock fissures

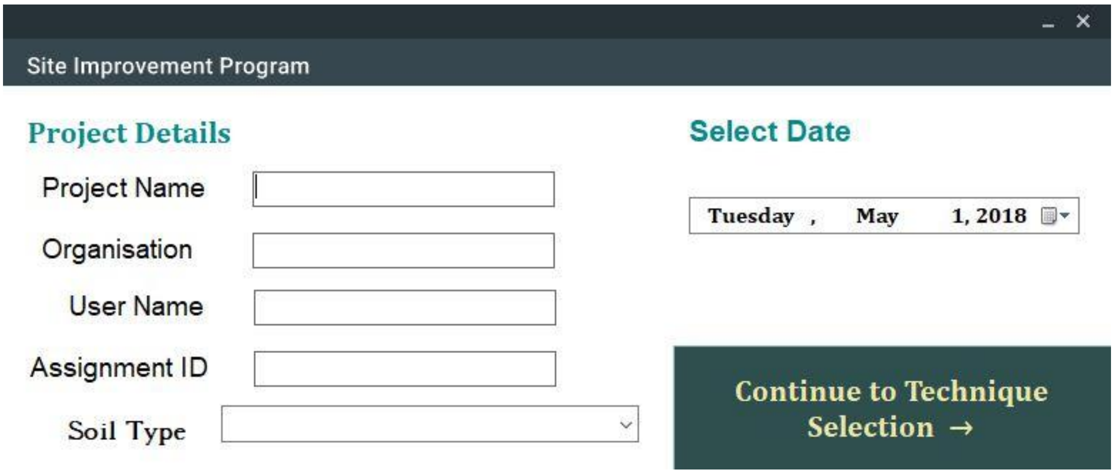


Figure 28 Project detail screen

Soil and Foundation input

The next screen after pressing the button (continue to technique selection) on project detail screen is the Soil and foundations input screen. Here you get the option to choose unitary system which is SI and FPS system.

In soil and site input bar, there are options for user to enter site area, depth of improvement, and other soil parameters. In foundation input bar, user will have to check foundation type (square, circular, continuous etc.), dimension of footings, and load of column and factor of safety. User will give N_{60} value from geotechnical investigation from site.

After having the details of the site, soil and foundation, the software will now calculate bearing capacity and settlement.

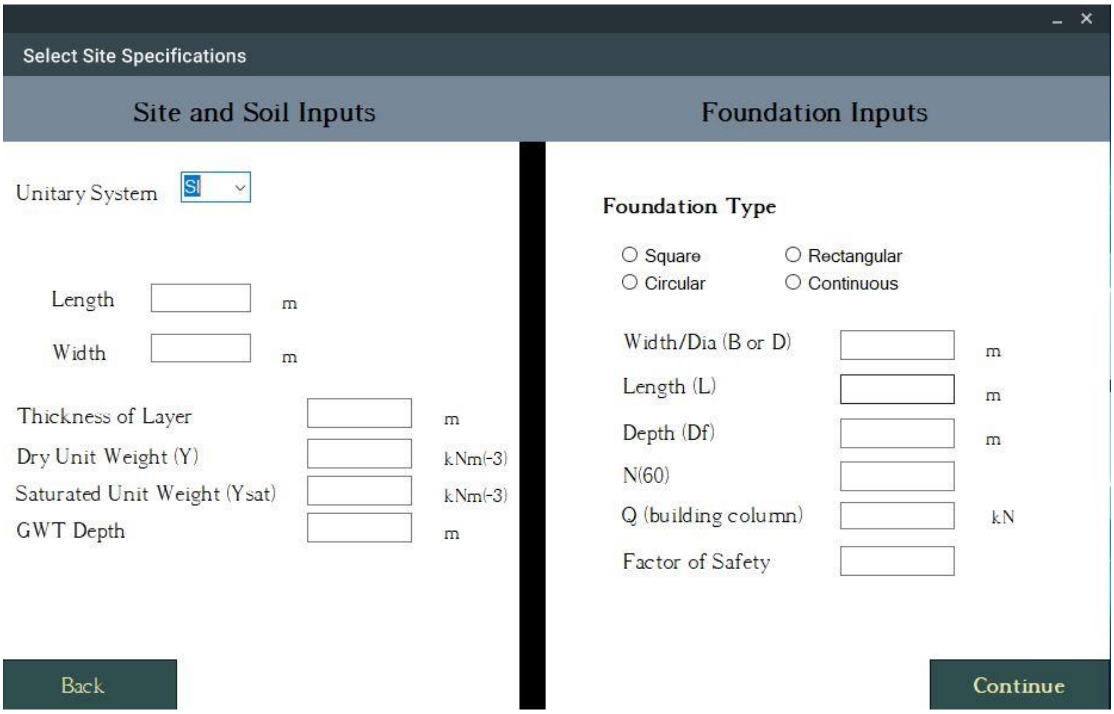


Figure 29 Soil and foundation input screen

Bearing Capacity check

If you press the continue button, the software will calculate cohesion, friction angle, bearing capacity and load, and settlement in this screen. The code will use the value of N_{60} estimate the value of friction angle and cohesion by given empirical relations. It will calculate bearing capacity and settlement and this load capacity is compared with load of building and this load of building evaluate from structural mechanics. It will compared loads and resistances if ultimate bearing capacity is less than loads of the building than the message is displayed that ground improvement is required. On the other hand, ultimate bearing capacity is greater than building than message will be shown that no ground improvement is needed.

If ground improvement is required as the result of these determinations the need for ground improvement is established as the best approach for the mitigation of the problems then two options are available to design the ground improvement method for the user:

- Filter techniques
- Continue to menu

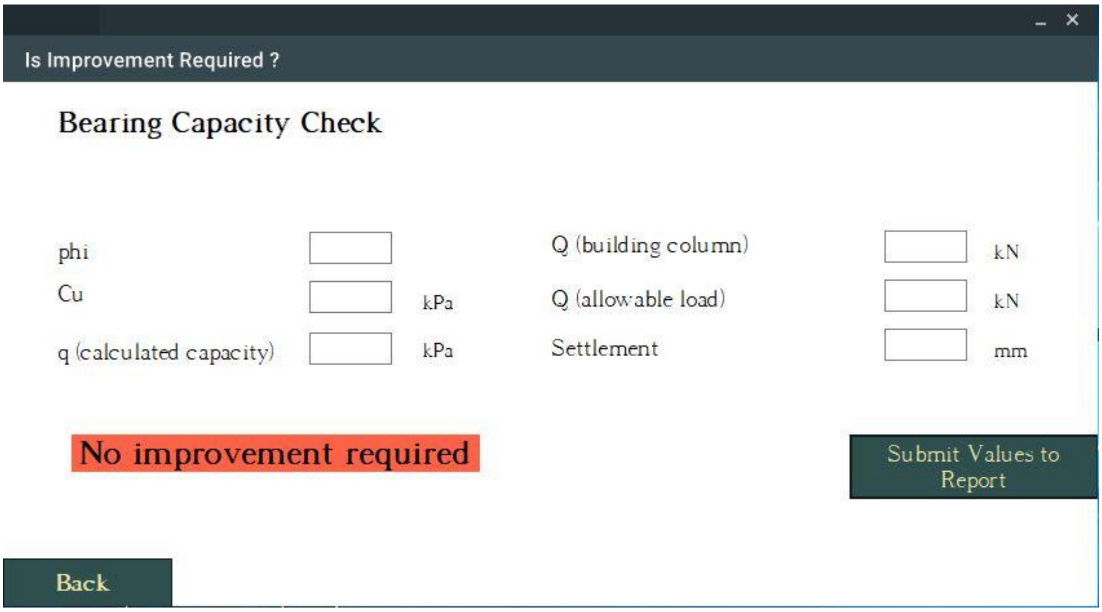


Figure 30 Bearing capacity check screen

Filtered techniques

If you press the filter button, a new form will be open which will include all the suitable techniques for improvement. After selecting this option the new form will be generated having the best suitable method based upon soil properties and limitations. There are two types of buttons which are clickable and non-clickable. Clickable buttons will enable the user to input the parameters including dependent and independent variables. Non-clickable buttons are locked which means the methods are not appropriate for the given parameters.

At this stage the software will filter techniques on the basis of these two criteria:

- Soil types
- Depth of improvement

User will select one of the filter technique to improve the site. The user will have to enter to specific parameter of that technique to calculate required or improved bearing capacity and settlement.

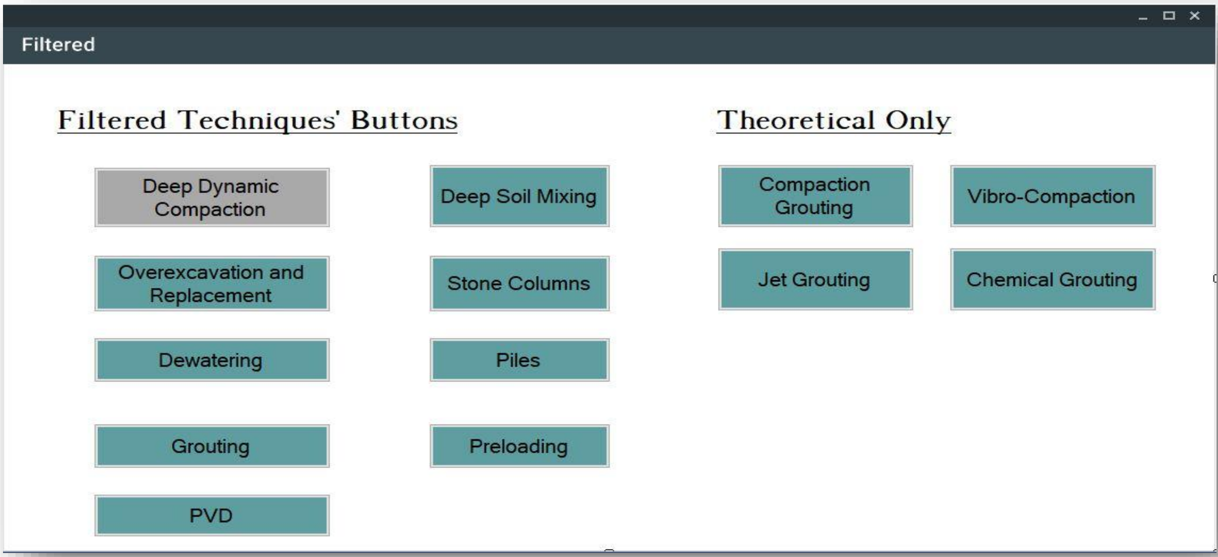


Figure 31 Filter technique screen

Menu of techniques

By clicking "Continue to menu" button on the bearing capacity check screen, the software will display the list of all individual technique in menu of technique screen. By clicking any one of technique, the user will be able to run the particular ground improvement technique.

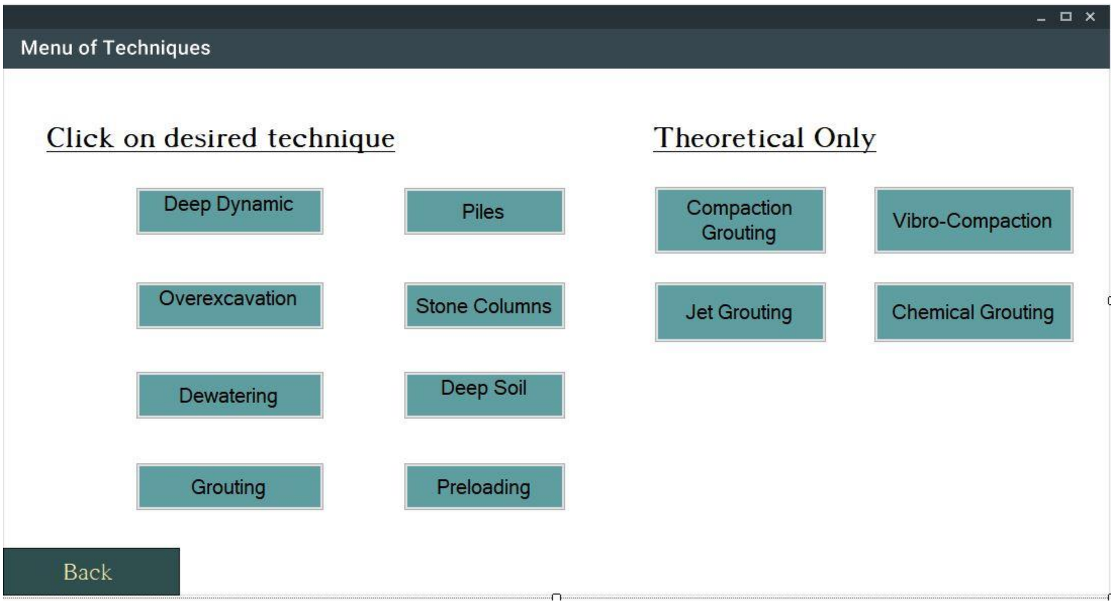


Figure 32 Menu of techniques screen

Design input and output of improvement methods

The menu of techniques screen contains all the applicable techniques on the form. By clicking on the specific method, screen for the design input of the method will be opened. In this form, user will enter all the required input for calculation. The application will calculate all the desired outputs and display in the next screen.

The given below pictures are design input and output screen of grouting.

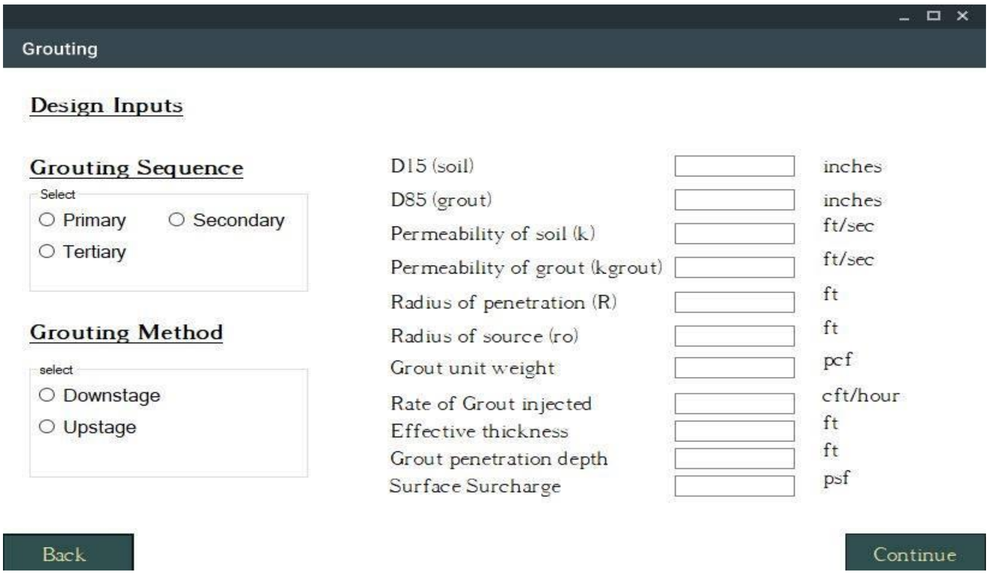


Figure 33 Design input screen for grouting

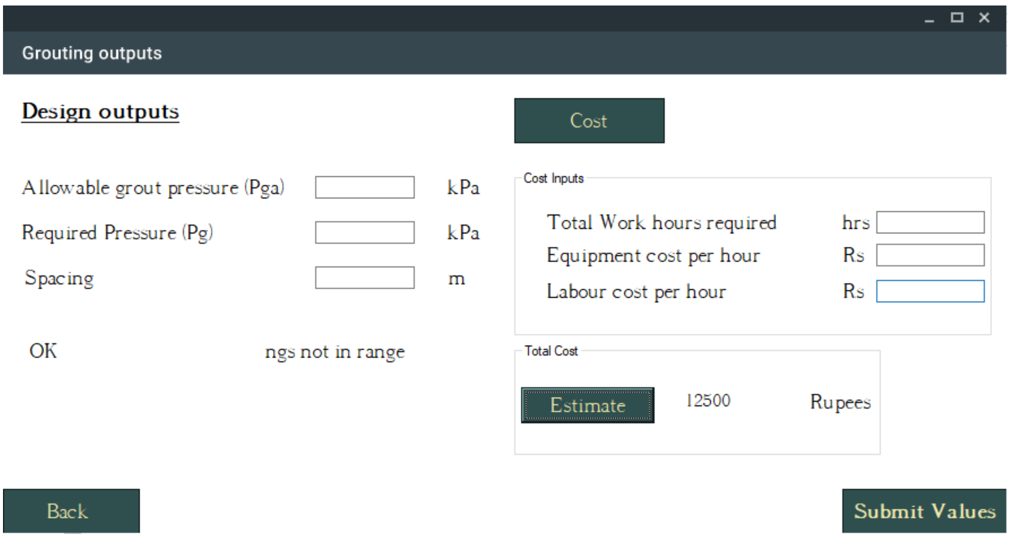
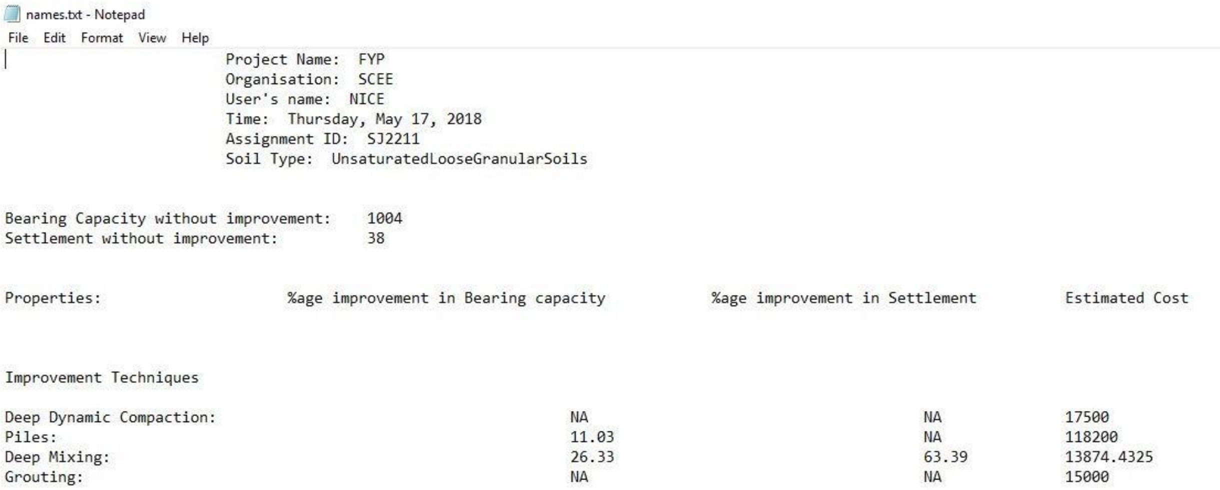


Figure 34 Design output screen for grouting

Report

At the end the program, user will click on the open report button. A report will be generated in the text file which contain project detail such as project name, date, soil type etc. It will also contain percentage improvement of bearing capacity and settlement of different method along with the total estimated cost of these methods.

The sample of report is shown below:


names.txt - Notepad
File Edit Format View Help

Project Name: FYP
Organisation: SCEE
User's name: NICE
Time: Thursday, May 17, 2018
Assignment ID: SJ2211
Soil Type: UnsaturatedLooseGranularSoils

Bearing Capacity without improvement: 1004
Settlement without improvement: 38

Properties:	%age improvement in Bearing capacity	%age improvement in Settlement	Estimated Cost
Improvement Techniques			
Deep Dynamic Compaction:	NA	NA	17500
Piles:	11.03	NA	118200
Deep Mixing:	26.33	63.39	13874.4325
Grouting:	NA	NA	15000

Figure 35 Sample of report generated from software

4.3 Cost analysis

We have provided the brief overview and illustration of methods that may be used in ground improvement applications. For confirmation point of view very little case histories are present in the literature as well as real life projects. Such as Specialty contractors’ experience, different methods in their daily life and related information especially about construction materials, procedures and costs. With respect to cost, the total cost of any specific technique depends upon many factors and it is difficult to generalize. Nonetheless, different treatment methods may cost differently. For example grouting might be more expensive than stone columns irrespective of that they are used for same purpose. Therefore we have employed rough estimate of cost.

To estimate the cost, we have taken the cost of the labor, equipment and total time of the improvement. The user will prompt to enter the total cost of specific project including treatment cost under normal conditions and equipment mobilization cost.

4.4 Limitations of Software

- The program is restricted to homogeneous soil only because of no case history has been studied in the literature.
- Only few techniques are taken into considerations due to rare applicability in our country.
- Ground reinforcement methods are not encountered such as mechanically stabilized earth walls (MSE walls), Ground nailing and ground anchors.
- Cost Estimation process is approximate analysis because there is no such manual which considered all the treatment methods and their cost under normal conditions.

4.5 Recommendations

- This program can be developed considering different layers of soil strata, by using finite element method and other mathematical relationships.
- The graphics design of this program can be made better further and run time processing can be done.
- Number of ground improvement methods can be increased.
- With the help of more practical histories this program can be improved.
- Cost can be optimized and comparable if useful data can be taken from fields.

CHAPTER 5

VERIFICATION OF SOFTWARE

5.1 Introduction

We have taken examples from various books to verify the results of the software that our group developed.

5.1.1 Example 1

A site consists of a loose medium sand (D_{15} = 0.1 mm), which is located at a depth from 4.3 to 5.3 m. The groundwater is at the depth of 2 m. The permeability of sand is 0.001 m/s and grout is 1.6×10^{-4} m/s. This loose sand has liquefaction potential; therefore, it should be improved. Permeation grouting is selected for this improvement. Design the permeation grouting. (Ground Improvement Book, Jie Han)

Results

Parameters	Grout head (m)	Grout pressure (kPa)	Allowable grout pressure (kPa)
Hand Calculation	9.4	36.024	224
Software	9.4	36.04	224

5.1.2 Example 2

A job site requires an excavation of a rectangular area (220 m × 170 m) to a depth of 15 m. The existing groundwater table is at 5 m. Below the ground surface is 30m thick gravel with a permeability of 5.0 × 10-5 m/s, which is underlain by bedrock. The groundwater table should be lowered to 1.5 m below the bottom of the excavation. Deep wells are used to dewater the site. Calculate the total required discharge. If 200-mm-diameter deep wells are used, how many deep wells are required?

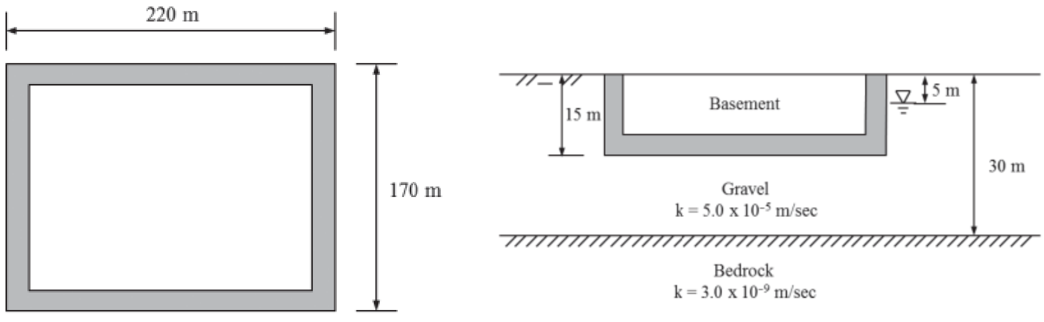


Figure 36 Plan and cross sectional view of example

Results

Parameters	Influence radius (m)	Discharge (m3/s)	Number of wells
Hand calculation	244	0.0864	10
Software	244	0.08	10

5.1.3 Example 3

A square foundation (2 m) is built on unsaturated loose granular strata of thickness 4 m. The depth of the footing is 1 m and ground water table is 5 m below surface. The dry unit weight of the soil is 18 kN/m3 and saturated unit weight is 20 kN/m3. The load of column on footing is 1200 kN. The SPT value is 5 and take factor of safety 3.

Results

Parameters	Capacity (kPa)	Allowable load (kN)	Settlement (mm)
Hand calculation	248	992	40.1
Software	251	1004	37.93

5.1.4 Example 4

A site given in example 3 needed ground improvement for construction of structure.

Deep dynamic compaction is used to improve the soil. Mass and diameter of the tamper is 15 Mg and 1.5 m. Adjacent structures from the site is 50 m away. Use values of example 3 for calculations.

Results

Parameters	Hand calculation	Software
No. of drops	6	6
Spacing (m)	2.8	3
Height of drop (m)	4.3	4.27
Energy per blow (Mj)	62.25	64
Particle velocity (mm/s)	5.29	5.38

5.1.5 Example 5

Deep soil mixing method is applied on the site given in example 3 for the purpose of ground improvement. The lime column has the length 10 m and diameter 0.5 m. The unconfined compressive strength is 1000 kPa. The elastic modulus of soil and lime column is 1000 and 6000 kPa.

Results

Parameters	Bearing capacity of single column (kN)	Ultimate bearing capacity (kPa)	Allowable BC (kPa)	Settlement (mm)
Hand calculation	140.52	480.63	315.2	15.98
Software	137.44	475.46	316.96	13.89

5.1.6 Example 6

The ground improvement method does not give satisfactory result on the site given in example 3. Therefore, engineers have decided to go for deep foundation. The circular pile has selected which has diameter 0.75 m and length 12 m. The pile should be driven in the soil. Take factor of safety of 2.

Results

Parameters	Ultimate load capacity of pile (kPa)	Allowable load capacity of pile (kPa)	Settlement (mm)
Hand calculation	2220.02	1110.01	20.78
Software	2228.63	1114.32	25.35

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