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ISTANBUL GEDİK UNIVERSITY
INSTITUTE OF GRADUATE STUDIES

EFFECT OF UNCERTAINTIES OF SOIL STRENGTH PARAMETERS ON THE RELIABILITY OF FOUNDATION BEHAVIOR

MASTER THESIS

Omar Muttlaq Sulaiman SULAIMAN

Engineering Management Department

Engineering Management Master in English Program

JANUARY 2024
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Bu çalışma 26.01.2024 tarihinde aşağıdaki jüri tarafından Mühendislik Yönetimi Dalı, Mühendislik Yönetimi (Tezli Yüksek Lisans) Programı Yüksek Lisans Tezi olarak kabul edilmiştir.

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DECLARATION

I Omar Muttlaq Sulaiman Sulaiman as a result of this declare that this thesis titled "Effect of Uncertainties of Soil Strength Parameters on the Reliability of Foundation Behavior" is original work I did for the award of the master's degree in the faculty of Engineering Management. I also declare that this thesis or any part of it has not been submitted and presented for any other degree or research paper in any other university or institution. (26/01/2024)

Omar Muttlaq Sulaiman SULAIMAN

To my spouse and children,

PREACE

In the name of Allah I start my foreword, I want to direct my thanks and gratefulness to my spouse who supported with me all the way, my noble professor doctor Mohammed Yousif Fattah, who supported me all the time during studying in university of Technology in Baghdad before graduation. And keep on supporting me with my thesis and to my esteemed professor doctor Gözde Ulutağay my thesis advisor for her humanity and understanding.

I dedicated this achievement to the spirits of my parents and pray for them to have mercy.

January 2024 Omar Muttlaq Sulaiman SULAIMAN

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ABBREVIATIONS

C.B.R : California bearing test
Cu : Undrained Shear Strength
Eg : Efficiency of Stone Column Group
Et al : And Others
FLAC : Finite-Difference Code
Ic : Consistency Index
Kpa : Kilo Pascal
Ks : The Coefficient of Punching Stress
L/D : Length of Stone Column to its Diameter
n : Stress Concentration
SPSS : Statistical analysis program

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EFFECT OF UNCERTAINTIES OF SOIL STRENGTH PARAMETERS ON THE RELIABILITY OF FOUNDATION BEHAVIOR

ABSTRACT

Since the soil characteristics may vary within a specified zone therefore for a geotechnical engineering projects the uncertainty is something unavoidable. For this reason the factor of safety is to be used in deterministic approach that determined the uncertainty related with soil of such characteristics. The amount and sources of uncertainty is not to be considered in the deterministic approach. Through the use of deterministic approach the limit case design is non-predictable. Therefore it is more logical to study the probability of failure.

During this topic the procedures of the uncertainty analysis of the bearing capacity of foundation located on soil supported by stone columns or by the soil replacement ways using granular material is to be explained. The analysis requires the definition of the standard deviation of undrained shear strength and the internal angle of friction of the stone and the soil surrounding the stone. The point estimation method that in it the amount of capacity and standard deviation to be calculated. The current topic procedure is an extension of the point estimation method. Tests implemented on 21 models as a two groups on models of soft clay of variable values of undrained shear strength (Cu) ranging from 8 to 18 kpa. The first group of eight models of soil replacement located at variable depth and width, the second group of twelve models of stone columns with single column, two column group, three column group, four column group, five column group and six column group. The last model is of un enhanced soil (untreated soil).

A portable vane shear device used for calculating the undrained shear strength before and after the occurring of the failure of the model. It appeared that the undrained shear strength of the stone column increased from (5.6-20) % because of the use and the construction of the stone columns. For the models of footing resting on replaced soil partially (using granular materials) the undrained shear strength increased from (5.5-15) % because of soil replacement.

Through the implementing of two non-liner models by the use of a (statistical analysis program SPSS) an analysis performed for the experimental data and for the ex-studies in order to form a bearing capacity equation for the floating and end bearing stone columns group.

The conclusion is that for a single column or for column within group at a given factor of safety the failure probability still the same, by the increasing the safety factor the reliability index increases for both ways of enhancement (stone column and soil replacement) and by the increasing of safety factor the failure probability decreases.

Key Words: *Soil strength, Soil properties, Shear strength parameters, Soil survey*

TOPRAK MUKAVEMETİ PARAMETRELERİN BELİRSİZLİKLERİNİN TEMEL DAVRANIŞININ GÜVENİLİRLİĞİ ÜZERİNDEKİ ETKİSİ

ÖZET

Zemin özellikleri belirli bir bölge içinde değişebileceğinden, bu nedenle bır jeoteknik mühendisliği projesi için belirsizlik kaçınılmaz bir şeydir. Bu nedenle, bu özelliklere sahip zeminle ilgili belirsizliği belirleyen deterministik yaklaşımda güvenlik faktörü kullanılmalıdır. Deterministik yaklaşımda belirsizliğin miktari ve kaynakları dikkate alınmamalıdır. Deterministik yaklaşımın kullanılmasıyla, limit durumu tasarımı tahmin edilemez. Bu nedenle, başarısızlık olasılığnı incelemek daha mantıklıdır.

Bu konuda, taş kolonlarla desteklenmiş zemin üzerinde yer alan temelin taşıma kapasitesinin belirsizlik analizi veya granül malzeme kullanılarak zemin değiştirme yollar ile ilgili prosedürler anlatılacaktır. Analiz,drenajsız kesme mukavemetinin standart sapmasının ve taşın ve taşı çevreleyen toprağın iç sürtünme açısının tanımlanmasını gerektirir. Noktasal tahmin yöntemi, içinde bulunan kapasite miktarı ve standart sapma hesaplanacaktır. Mevcut konu prosedürü, nokta tahmin yönteminin bir uzantısıdır. Testler , 8 ila 18 kpa arasında değişen drenajsız kayma mukavemeti (Cu) değişken değerlerine sahip yumuşak kil modelleri üzerinde iki grup olarak 21 model üzerinde uygulanmıştır. Değişken derinlik ve genişlikte yer alan sekiz toprak değiştirme modelinden oluşan birinci grup, tek sütunlu, iki sütunlu gruplu,üç sütünlü gruplu, dört sütünlü gruplu, beş sütünlü gruplu ve altı sütünlü on iki taş sütun modelinden oluşan ikinci grup. Son model,geliştirilmemiş topraktır (işlenmemiş toprak). Modelin arızalanmasının meydana gelmesinden önce ve sonra drenajsız kesme mukavemetini hesaplamak için kullanılan portatif kanatlı kesme cihazı. Taş kolonların kullanımı ve inşası nedeniyle taş kolonun drenajsız kesme dayanımının % (5.6-2O) den arttığı görülmüştür. Kısmen değiştirilen toprağa (tanecikli malzemeler kullanılarak) dayanan temel modeller için, toprak değişimi nedeniyle drenajsız kesme mukavemeti % (5.5-15) den artmıştır.

(SPSS istatistiksel analiz programı) kullanılarak astar olmayan iki modelin uygulanmasıyla,yüzer ve uç taşıyıcı taş kolon grubu için bir taşıma kapasitesi denklemi oluşturmak amacıyla deneysel veriler ve eski çalışmalar için bir analiz gerçekleştirilmiştir.

Sonuç, belirli bir güvenlik faktöründe tek bir sütun veya grup sütun için arıza olasılığının hala aynı olduğu, güvenlik faktörünün artmasıyla güvenilirlik indeksinin her iki iyileştirme yolu için de arttığı (taş kolon ve toprak değişimi) ve güvenlik faktörünün artmasıyla arıza olasılığının azaldığıdır.

Anahtar Kelimeler: *Toprak mukavemeti, Zemin özellikleri, Kesme dayanımı parametreleri, Zemin etüdü*

1. INTRODUCTION

1.1 Premium

It is clear that the characteristic the shear strength parameters are the basic factor that the geotechnical design depends on. These parameters can be defined as cautioned estimations that affect the limit state of soil. There is always a certain degree of subjectivity in the evaluations of soil featured parameters. Because of the originated lack of the data in the geotechnical investigation, this shows how the geotechnical engineers explain the cautioned estimations. A group of shear parameters used by 90 geotechnical engineers to design a spread foundation, the engineerical dimensions resulted were clearly different from each other, the conclusion was that the geotechnical calculations are estimates not an accurate predictions.

For the complicated geotechnical projects, an accurate analysis (sensitivity) should be performed to gain knowledge about the critical scenarios. More than that, the continuous evaluation of the of the soil characteristics is so important during the construction processes. The ordinary ways such as dealing with the bearing capacity of the shallow foundations and the calculations of excavation depth, the limit equilibrium and limit analysis, all of them does not satisfy the theoretical requirements of the stability problems which leads to different solutions depending on the way used. The geological materials show variation and heterogeneity due to their inherited variation.

Although a lot of many of interesting projects were implemented in the past time such as tunnels, dams, and other geotechnical structures and important projects before reaching a formal geotechnical engineering system, but the engineers were still unable to deal with the geotechnical difficulties and it was not able for them to come up with analytical methods to describe the attitude of rocks and soils until the middle of the twentieth century. Because of bearing capacity failure and settlement failure. Furthermore the foundation should satisfy other requirements these requirements may have adverse effects on the foundation.

The foundation should be placed in proper manner to avoid adverse effects of some effects like frost influence, high volume change adjacent structure, property line. Considered. Certainty may be a description for most of engineering problems. Uncertainty problems cannot be avoided and the sources causing them may be due to:

- Insufficient knowledge of the state of soil layers.
- Inherited variance in soil parameters.
- Insufficient control for environmental changes after constructions.
- Decreased accuracy in calculating the bearing capacity by the empirical methods.
- Considering the applied loads as live loads, dead loads, frost loads and other.

1.2 The Management of Geotechnical Risks

Due to the difficulty of determination of the geotechnical circumstances, the infrastructures projects called the unknown environments. The inadequate understanding of the site geotechnical conditions leads to incapability of determining the effective geotechnical parameters. The geotechnical researchers by themselves investigate in internal erosion, characteristics of rocks and soils, investigations of the risky geotechnical situations, the analysis of the soil and the probable problems, and the polarity of the calculations of failure. (Shipton and coop, 2012), (Hong, 2010).

The analyzing of soil with focusing on the geotechnical frightens, more than that the geotechnical design symbols for the whole structure is enough to carrying loads and stability. Also the constructions procedures and the organization and the financial arrangement all of them can be classified as resources of risks. (Barr et al., 2019).

The controlling of the geotechnical risks requires deep knowledge of all the details of the project. A lot of geotechnical engineering projects have a lot of difficulties including the long term schedules of the project and the huge doubts and a huge group of different requirements and the big organizations, a high level of technical expertise, focus on the policy and the peoples and the environment. The construction operation involving a group of different players most of them have a different goals and a lack of experience in dealing with each other. The foundations works and excavations works and tunnels all of them can be called (the series system) in the field of geotechnical engineering.

(Tawalare, 2019) states that the work processes controlling and affect the sequence by which the future works are implemented. The ISO 31000 pays attention to the (risk source and consequence). Both of them put the risk source in a view that the risk source appears to be a one factor or group of factors has the ability to cause risks. The (event) is the present or modifying specific group of conditions while the result of event affecting the goals called (consequence).

1.3 The Risk – The Analysis and Evaluation

The analysis of risk is the procedure through which a detailed test of probable risks is implemented. The evaluation of risks that should be minimized and to determine the most effective ways to do that will need the cooperation.

Including all the causes and the effects of the probable threats. While evaluation the risks and analyzing them, the common practices are to consider the results and the probabilities of the events. The risks evaluations can be performed on a qualitative or quantitative methodology or both of them according to the need. (Szymanski, 2017) summarized the most used techniques of risks evaluation in geotechnical engineering as follows:

- Consequence or cause consequence.
- Fault tree analysis (FTA).
- Event tree analysis (ETA).

According to (Zhu et al., 2020) the techniques used for analyzing the complicated safety and risks and the critical controls can be expressed as shown below:

- Bayesian networks.
- Multi risk analysis.
- Decision analysis
- Probabilistic risk analysis (PRA).
- Analytical hierarchy process (AHP).

1.4 The Statement of Problem

Because of the complication of the geotechnical characteristics, and incapability of predicting them, the designers facing in the infrastructures projects unknown complicated geotechnical conditions. Often the natural soil deposits have unorganized layers resulted from a variable materials of effective characteristics that affects the behavior of the material. (Muhaimeed et al., 2014). Also in the rocky material unorganized material can be found that might effect on the rock behavior. And it is difficult to choose suitable parameters for soil to be used in design 4 circumstances and modelling geotechnical behavior is difficult. As a result of the high sensitivity of changes in implementation of the infrastructures projects resulted from delays in work activities, in most times the geotechnical risks has a huge influence on the costs of the project and on the work time schedule and in turn this leads to periodical delays and higher costs.

The events with negative influence can be avoided especially the ones that affects the company reputation or its income. The adequate safety margin must be used to insure the safety. It is necessary to focus on the geotechnical frightens for constructing the infrastructure project (Pocock et al., 2016)

1.5 Purpose of Thesis

The purpose of this research is to investigate the geotechnical risks in the calculations of bearing capacity of foundations and to enhance the geotechnical risks management by the use of independent modern methods to enhance the quality of the geotechnical works and reduce the cost and the periodical delays.

The other target of the research is to clear that the reliability and the factor of safety both of them are useable to measure an accepted design depending on the followings:

1. Soil unit weight(ɣ), angle of internal friction (φ), Cohesion (Cu).
2. The stone column dimensions and the width of footing are deterministic variables.

1.6 Stone Column and Soil Replacement

The problems related to the evaluation of the bearing capacity and choosing the suitable foundation figures more in so weak soils. At least a moderate increasing in bearing capacity for weak soils is desired to obtain constructions with satisfied properties and suitable foundations. One of the solutions for enhancing the bearing capacity of the weak soil is by using granular piles or stone columns these piles or column is to be installed on soil. A granular trench is said to be a plane strain version of a granular pile. A satisfied results can be obtained from a well-controlled and carefully applicated (Kameswara, 1993). Many of grand enhancement techniques are used to enhance the attitude of strength and compressibility of these deposits. The most important ones between them is stone columns, lime columns, and granular trenches.

For choosing a definite technique that is depending on soil profile in site, the quality of the used material in technique, and the pore fluid nature and the way of implementing the technique. More than that the quality assurance checks implemented for enhanced attitude of the ground and also performance studies which are separated during a known period of time through the maintenance of settlement records all are basic factors on them the successful ground enhancing techniques application depends. Although a lot of work was consumed during searching in these techniques, the confidence in any field engineer will be gained and developed through the successful and proper using of these applications (Somayazulu, 1993). The treatment of the ground using the stone column techniques succeeded in improving:

1. The improvement of slope stability for embankment and natural slopes.
2. The increasing of the bearing capacity.
3. The reduction of differential settlements.
4. The reduction of liquefaction of sands and the increasing of the settlement time rate.

The supporting of structures covering both very soft to firm cohesive soils and also loose

Silty sands having greater than 15 percent fines, (Barksdale and Bachus, 1983).

The high potential for useful use of stone column is mainly as a ground improvement technique for strengthening weak and soft soil. This involves the area of high way, rail way and Airport applications have also prompted a thorough investigation to determine how and why the system works well, and to establish appropriate guidelines for design and construction. This has led to the deployment of several experimental design concepts for the purpose of stone column design.

1.7 Hypothesis

Requiring the same factor of safety for all the bearing capacity applications in a (one size fits all) approach that is certain to result in inappropriate factor of safety in some cases.

A more logical approach would consider: A procedure followed in this thesis is to investigate the reliability of bearing capacity equation of foundation on soft clay reinforced by a stone column or a trench of replaced soil based on reliability index rather than conventional factor of safety.

Figure 1.1: Triaxial shear stress Apparatus

Source: (internet)

In this research, a method was described to perform a reliability analysis of Foundation

Tolerance on soils improved by stone columns or partially replaced by granular materials. The method requires the definition of the standard deviation of the value of the undrained shear stress, the angle of internal friction, the specific weight of the surrounding soil and the crushed stone. Context is an extension of the dot-point speculative method from which the expected values of the standard deviation of both the demand and amplitude functions can be calculated. The probability of failure, reliability, central safety coefficient and reliability index were calculated in an appropriate way. The total number of inspection models is 21 models divided into two different groups of clay soils with undrained shear stress ranging from 8 to 18 kPa. The first group consists of eight soil replacement models of different dimensions and depths, while the second group consists of twelve models of stone columns, one stone column, two columns, a third column, four columns, five columns and six columns. In addition to one model for untreated soils. The strength of the undrained shear was measured using a laboratory shear inspection device (Trixial Shear Test Apparatus).

Before and after the examination for each model, it was found that the stress of the undrained shear increases by (5.6-20) % for improved models with stone columns and increases by (15-5.5) % for improved models by partial replacement of soil with crushed stone.

Nonlinear Mathematical models were created using SPSS software. Laboratory work data and previous studies were analyzed to construct an equation to estimate a tribe bearing foundations built on soil improved with floating stone columns and stone columns with end-bearing bearing. It was found that for a certain safety coefficient, the probability of failure is close whether for one stone column or for several columns, and that the reliability index increases with the increase in the safety coefficient of soil improved by partial replacement or improved by stone columns. And that the probability of failure decreases as the safety coefficient increases.

1.8 Thesis layout

The thesis representing five chapters as expressed below:

- Chapter one: representing the risks in geotechnical projects and the problems and aims. Discussing the background of this field of knowledge and the main concept of the geotechnical risks in infrastructure projects.
- Chapter two: gives a wide view about the soil improvement techniques, the risks threaten the geotechnical projects and previous researches from other studies.
- Chapter three: this chapter involved the studied parameters, methodology and elements affecting the testing and the analytical calculations. (The analysis in the excel file within the appendices).
- Questionnaire: to be after chapter three (the analysis in the excel file within the appendices.
- Chapter four: laboratory works (experimental works) in summary.
- Chapter five: the last chapter consists of the: conclusions, recommendations and references, resume.
- The appendices are four separated files; two of them are excel files and consisting of the analysis of the questionnaire and study analytic solutions and results. The third one is document (word file) of Statistical Analysis of Designed Experiments - 2009 - Tamhane - Appendix C Statistical Tables. The forth file is article.

2. REVIEW of LITERATURE

2.1 Introduction

Locations of soft soil are found especially in coastal areas where these soils have decreased compressibility and low strength. Two problems are recorded while implementing constructions upon these soils, the first problem is the increased settlement the second one is the low shear strength, (Barksdale and Bachus, 1983; Bergado, 1996). They are named clays according to (Terzaghi, 1936) and they with normal or light over consolidation. And they are of liquidity index more than 0.5, and less than 10kPa is the undrained shear strength Cu of these clays. (Terzaghi andPeck, 1967) , defined clays as very soft clays when their unconfined compressive strength is less than 25 kPa and to be soft when their unconfined compressive strength is between 12 and 50 kPa. The clayey soil classified as soft clay when their undrained shear strength is less than 40 kPa. (Brand and Brenner, 1981), the soil classified as soft when it’s undrained shear strength is between 20 to 40 kPa. According to British standard (B.S: C.P 8004: 1986), soil with Cu undrained shear strength less than 20 kPa. İs classified as very soft clay. The young balanced due to its weight soft soil and not passed under lately or second consolidation from the time of its forming are classified as clay or silty clay (Kemphert and Gebreselassie 2006). İn order to classify soil to be a soft soil to be used in constructional projects, a group of characteristics should be satisfied according to German geotechnical council (Kemphert and Gebreselassie, 2006) these characteristics are:

1. Consistency index Ic < 0.75 for soft and very soft soil.
2. Saturated or about to be saturated
3. The undrained shear strength Cu less than or equal to 40 kPa. And the soil seems to flow.
4. The plasticity of soil is light or medium.
5. High sensitivity to vibrating and changeable flow.

2.2 Improvement of Soft Soil

To improve a soft soil a lot of methods may be used, one of them is the using of the stone columns. For the using of this method there are also many ways for installation of the stone columns. But the most used way is the bottom feed vibrio-displacement method. In this method the compressed air used for flushing a vibrating probe direct into ground. The probe has a tube through it the sand or gravel to be filled, the sand still at the soil while the probe to be moved later. The probe is closed from the upper side to prevent the internal air pressure and the probe has is opened from its bottom to allow the sand or gravel to flow to soil and remain. The gravel or sand column to be compacted by the vibration of probe. (Keller, 2002). To be shown in Figure (2.1).

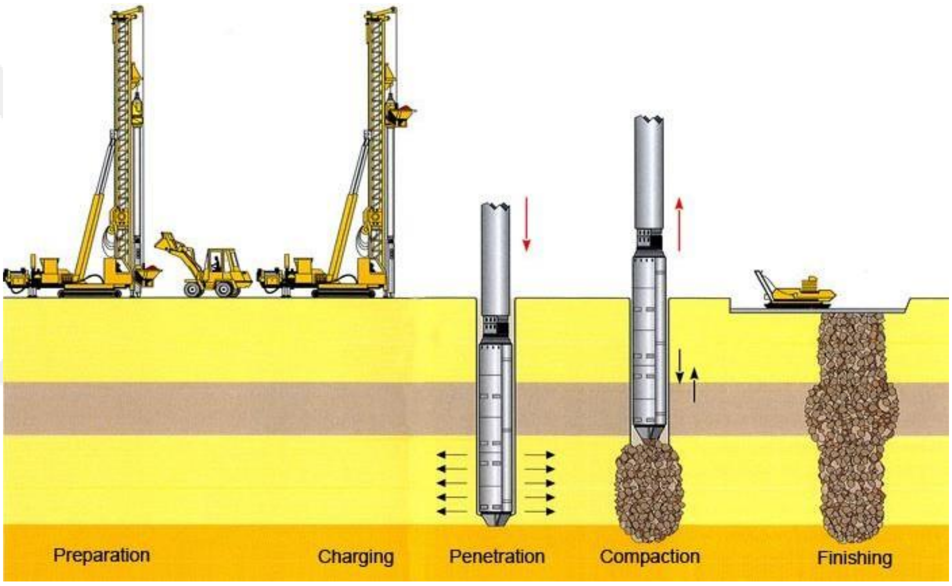


Figure 2.1: Installation of Columns

Source: (Keller, 2002)

2.3 Stone columns techniques

As a ground improvement method the granular piles are used stone columns are included for more than thirty years. These compressed particles located inside the ground, although these particles cannot stand against tensile stress, but they give high compressive stress and stiffness. And they bear the highest portion of the applied loads with less deformation as compared with the soft clay in the site. Strengthen the functions of reinforcement and the drainage, in addition to the mentioned these vertical columns also improve the bearing capacity and making settlement to be reduced. Because of the continuous installation process noticed that the lateral stresses in the native ground around the improvement particles is (the lateral stresses) higher than the other values. As a result (Bergado et al, 1991) made a report explaining the case of threat about the improvement ways for the ground. In the soft clay the using of column stones especially in the embankment is an economical way since it increases the value of the shear stress and reducing settlement and gives accelerated consolidation to resist the vertical stresses, so they may remain constant. Because of the negative earth pressure which occurred from the effect of bulging of the stone column and from the increasing of the resistance against the lateral deformation because of the applied composite load.

2.3 Basic Design Equations

2.3.1 The ratio of area replacement

According to (Bergado, et.al. 1996) the ratio of area replacement is the ratio of the cross sectional area of stone column to the total unit cell area.

$$\boldsymbol{a_s} = \frac{A_s}{A_s + A_C} \tag{2.1}$$

where:

$\mathbf{A_s}$: the cross sectional area of stone column.

$\boldsymbol{A_C}$ **:** The area of clay around the stone column.

2.3.2 The concentration ratio of stress

Since the stone column has more stiffness than the native clay therefore the concentrated stress took place in the stone column.

$$\text{n} = \frac{\sigma_s}{\sigma_c} \tag{2.2}$$

Where:

n: the ratio of concentrated stress.

$\boldsymbol{\sigma_s}$: The column stress.

$\boldsymbol{\sigma_c}$: The surrounding soil stress.

The applied stress ($\boldsymbol{\sigma}$)on the unit cell is:

$$\sigma = a_s * \sigma_s + (1-A_s) * \sigma_c \qquad (2.3)$$

$$\sigma_c = \frac{\sigma}{1+(n-1)*A_s} = \mu_c * \sigma \qquad (2.4a)$$

By solving equation (2.3) in equation (2.2) we obtain:

Stress concentration factor ($\boldsymbol{\sigma_s}$) = $\frac{\boldsymbol{n*\sigma}}{\boldsymbol{1+(n-1)*A_s}}$ (2.4b)

Where:

μ_c And μ_s are the ratio of stress in clay and stone column to the loading intensity as shown in figure (2.2).

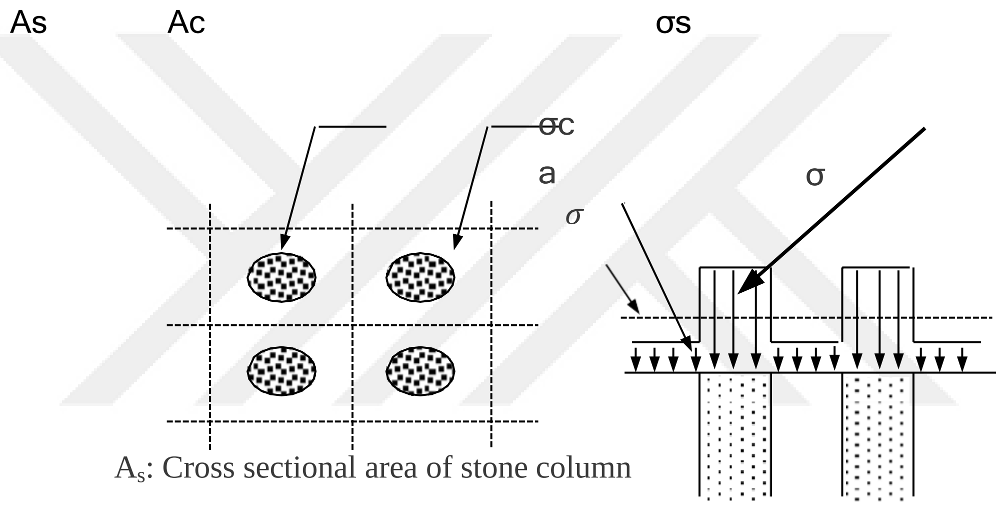


Figure 2.2: 1Stress Concentration

Source: (Aboshi et al., 1979)

Where:

As: Cross sectional area of stone column.

Ac: The area of clay for each stone column.

σ: The average loading of intencity.

2.3.3 Mechanism of failure

Three sorts of possible failure may the stone column pass through, these three sorts of possible failure are:

2.3.3.1 Shear failure

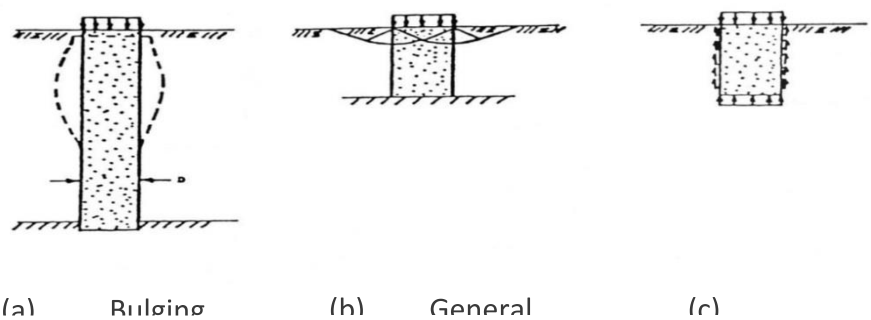
When the deformational characteristics of the soft clay and for the stone column material are of the same order an appropriate shear failure mechanism occurs. And when the column is absent or the width of the column is three times greater than the loaded area also an appropriate shear failure may take place, as explained in figure. İt is possible for failure to happen in a very short column loaded with a firm support is explained in figure (2.3b).

2.3.3.2 Bulging failure

This pattern of failure takes place when in an end-bearing stone column on a solid layer under loose soil, and a tall floating stone column more than 4-6 diameters long, as explained in figure (2.3a).

2.3.3.3 Punching failure

This type of failure is similar to a rigid pile attitude when a lateral displacement takes place in floating column with length less than 2-3 times its own diameter. This type of failure occurs before the occurring of the bulging failure. İn this type of failure a higher loading capacity is expected to be carried. But the lack of rigidity of stone column as compare with steel column or concrete piles makes this type of failure not recommended. As explained in figure (2.3c).



a. Bulging failure b. Shear failure c. Punching failure

Figure 2.3: Single Stone Column, Mechanism of Failure

Source: (Barksdale and Bachus, 1983)

(Wood et al., 2000) suggested four types of failure for the stone column this suggestion is may be applicable for the stone column. As explained in figure (2.4). The four suggested failures are as following:

1. Failure takes place in a stone column due to bulging if the stone column is not to be prevented by the adjacent columns from radially expansion. As explained in figure (2.4a).
2. When a thin stone column is laterally loaded, failure may happen. As explained in figure (2.4d).
3. Due to the lack of the column lateral resistance a diagonal shear plane may take place in the column due to a high load. As explained in figure (2.4b).
4. The column may fail if a soft soil layer under the column is to be penetrated by the column itself, when the column is stiffer than the underlying soft soil layer. As explained in figure (2.4c).

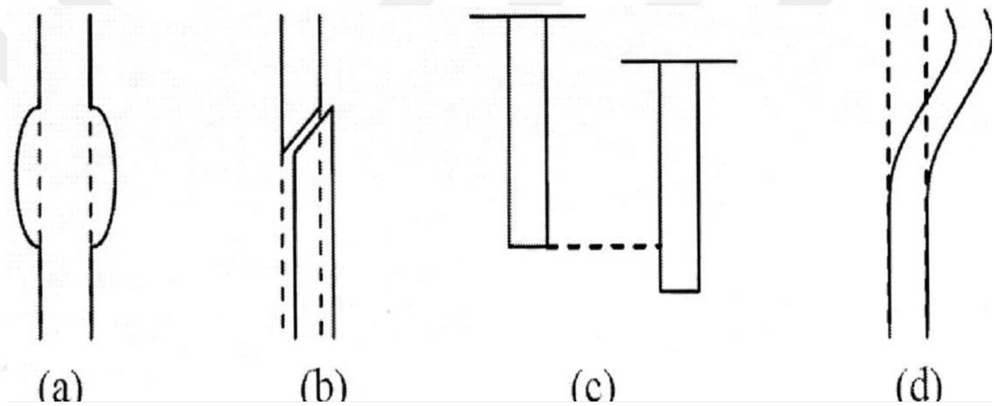


Figure 2.4: The Four Types of Failure of Stone Column

Source: (wood et al., 2000)

2.4 The bearing capacity of the stone columns

The approximated equation for calculating the allowable bearing capacity of the stone columns was given by Bowels in 1996. This equation stated:

$$q_{a}=\frac{K_P}{F_S}\left(4c+\ \sigma_r{}'\right) \tag{2.5}$$

Where:

Kp: Is the passive earth pressure coefficient.

$$Kp= tan^2(45^{\circ}+\frac{\emptyset'}{2}) \tag{2.6}$$

C: Either drained cohesion (for the large areas) or undrained shear stress $c_{u.}$

$\emptyset'$: Is the drained angle of friction of stone.

$\sigma_r{}'$: Is the effective radial stress measured by a pressure meter.

($\sigma_r{}'$) can be used as (2c) when the pressure meter data are not available and when the factor of safety (Fs) is between 1.5 to 2.

The overall allowed load on a stone column of average cross-sectional area (Ac) is equal to:

$$Q_a = q_a A_c \quad (2.7)$$

$$q_{ult} = 4c + 2a\,[c\,(K_P - 2) + c_s\sqrt{k_p}\,] \quad (2.8a)$$

Where:

c_s : Granular pile material cohesion.

he above equation (2.8b) was developed in 1995 by (Boussida et al.) it is used for group of stone columns. The equation states that the ratio between the total cross-sectional area of reinforcement and the footing area A_f is:

$$a_r = N_S\ \pi\ a2/A_f \quad (2.8b)$$

Where:

N_S : The number of stone columns.

2.5 Ex Studies Regarding Stone Columns, Researchers Conclusions

(Falah Rahil, 1992) according to an analyzing for 45 model tests implemented on both untreated soil and treated soil with the use of a layer of ballast with the exist or without the exist of stone columns under repeated loads. A group of notes were written.

1. When the undrained shear stress increased, the bearing ratio decreased.
2. When the undrained shear stress increased, the bearing improvement ratio decreased.
3. The difference in the models attitude due to the graduated change in failure. When models to be tested at Cu=9kPa AND 16Kpa, the ratio of bearing improvement will reach peak values at S/B=1% and to be followed by rapid decreasing.

Where:

S: Settlement.

B: Is the footing width.

As compared with models tested at Cu= 25kPa at S/B=1%, the ratio of bearing improvement increased and remain stable to the end of the test.

4. When Cu=9kPa the bearing ratio increased, while the settlement reduction ratio decreased and the ratio of optimum reduction is increased.

(AL-Qayssi, 2001) implemented typical tests to enhance the stone column attitude through using different types of reinforcement, using two and three discs connected to a central shaft. He studied the stone columns spacing, the footing shape effect, the ratio of area replacement and it is effect, and the stone columns numbers. He discovered that the circular footings can stand against the highest bearing ratio during the failure occurring. The second one is the square shape of footings; next to it the rectangular shape of footings. By the increasing of spacing between columns from center to center (c/c) from 2D, 2.5D and 3D, for all the three shapes of footings it causes increasing in the bearing ratio. He noticed that a simple influence happens on the efficiency of a single stone column due to the area replacement ratio.

(Maki Al-Waily, 2001) the target of the work is to establish a reference by experiment to obtain the value of the ratio of the concentrated stress through producing a single stone column model comes with rigid plates for loading supplied with measuring devices for measuring the total loads on the footing and the total loads on the stone column to be measured separately. The conclusion is:

1. In the treated soils due to the increasing of the shear strength the amount of the bearing improvement ratio decreasing.
2. In the treated soils the increasing of the shear strength leads to decrease the efficiency of stone column group (eg).
3. By increasing the shear stress in the treated soils the stress concentration ratio increased.
4. Obtaining increased stone column group efficiency by using crushed stone with (l/d=8), but by using (l/d=6) the efficiency is decreased.

(Zina Waleed, 2002) her conclusion consists of using different enhancement techniques, stone columns, geogrid reinforcement and covered stone columns(encased stone columns) and other enhancement methods of about twelve methods, these models exposed to monotonic loading, among these twelve models two of them were tested by using repeated loading. The target is achieved by using a small number of model tests. The target is to have knowledge about the stability of the ballast of a track coating a soft saturated soil exposed to both repeated and monotonic loading. Zina came up with a conclusion based on the using of model tests exposed to two undrained shear strengths of Cu=9kPa and Cu=25kPa. She figured that:

1. By the increasing of the bearing ratio (q/Cu) the ratio of settlement reduction decreases and the lowest value of the settlement reduction to be noticed at Cu=9 kPa.
2. The soil surrounding the stone columns when its shear strength increases, the bearing ratio (q/Cu) decreases. This happened when the shear strength increased (Cu) from 9kPa to 25kPa.
3. When the undrained shear strength (Cu) = 9kPa, the ratio of bearing improvement is=1.44, and when the undrained shear strength (Cu) = 25kPa, the ratio of bearing improvement is=1.17, concludes that the ratio of bearing improvement deceases by the increasing of the undrained shear strength.

(Mahdi, 2002) from the laboratory model tests, the most affecting factors or parameters are the length to diameter ratio of the stone column (Ls/Ds) and the ratio of area replacement(ar). The conclusion is the behavior of the soil will be enhanced by the using of the ordinary stone columns and the enhancement depends on the ratio of the length to the diameter (Ls/Ds) of the stone columns and the ratio of area replacement (ar).

(Malarvizhi and Ilamparuti, 2004) through applying load tests to a soft clay enhanced with a single stone column or reinforced stone column (covered with geogrid) the stone column comes with different ratio of slenderness and different types of covering material. The slenderness ratio of a reinforced stone column is the ratio between the length of the column and the column lateral dimensions. The slenderness ratio helps to resist buckling pressure and it can be calculated by dividing length of stone column by the diameter of the stone column (L/D). When the cross-sectional dimensions are smaller than its length is said to a slenderical column. When the slenderness ratio is high the stone column will fail under compression loads. The high length of the stone column to the cross-sectional dimensions will cause increased slenderness ratio. Area of stone column to the area of surrounded soil is said to be area ratio. The slenderness ratio to the area ratio figured to be 0.174 for both floating stone columns and end-bearing stone columns. Using the geogrid covered stone columns increasing the carrying loading capacity.

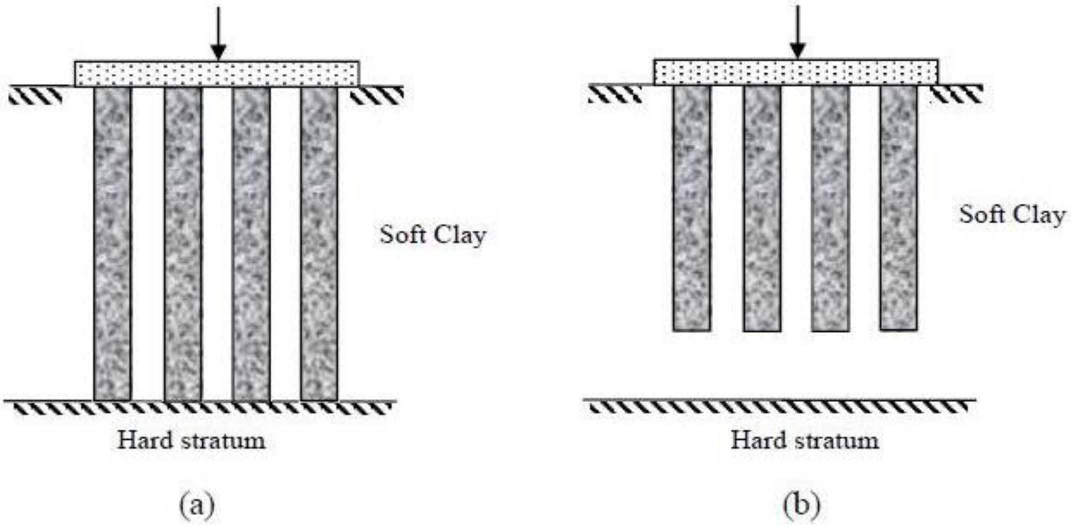


Figure 2.5: (a) End-bearing stone column. (b) Floating stone column.

Source: internet

(Rahil, 2007) performed a model tests with or without a ballast coat, over the soft soil and the influence of the undrained shear strength to be studied. The target is to investigate the behavior of soft clay strengthened with a stone columns, the soft clay is located under a rail way model. The followings were figured:

1. Through increasing the tests number of cycles and by increasing the applied stress the strength of fatigue (F.S) is decreased and a significant value of increasing can be noticed because of the using of the combination influence of stone columns and ballast layer.
2. By increasing the ratio of ballast a noticed decreasing in settlement ratio took place, and the reduction reached it is perfect values when using stone columns with the ballast layer at undrained shear strength(Cu) =9kpa.

3. By the presence of the combination of the stone columns and the ballast layer, the influence of the combination made the bearing enhancement ratio(improvement ratio) to reach it is maximum amount when the undrained shear strength=9kpa and the ballast ratio(H/B) is equal to 0.125.
4. The bearing enhancement ratio in the monotonic tests the maximum value of bearing enhancement ratio for the soft soil which is treated with ballast coat is still constant by neglecting the influence of the undrained shear strength. On the other hand in the repeated tests the number of the cycles of stress is increased while when using ballast coat or layer side by side with stone columns the result were reduction in the stress cycles number. Meaning that the combination of ballast coat and stone columns reducing the number of cycles of stress.

(Malarvizhi and Ilamparuthi, 2007) results came from laboratory studies stated that the bearing capacity of footing placed on a coated stone column (encased column) is about 1.5 to 2 more than the bearing capacity of the un coated stone column when the length to diameter ratio of the coated stone column is equal to 9 and the area ratio equal to 17%.

The studies also explained that the load carrying capacity for end loading column is twice than the load carrying capacity of a floating column. And the concentrated stress is more in the coated column (encased column) than the concentrated stress for ordinary column.

(Murugesan and Raja Gopal, 2010) studying the behavior of coated stone columns (geosynthetic) stone columns. Through several experimental tests on a single and group of encased stone columns (geosynthetic) with or without coating (encasing), the results were the geosynthetic coating increases the stiffness of the stone columns. The stone column pressure inside it increases by the increase of the geosynthetic coating modulus.

(Fattah et al., 2011) the stress concentration ratio (n) can be defined as the vertical stress effecting on the stone column to the stress effecting on the soil surrounding. Several experimental tests were implemented to gain the value of the stress concentration, a test implemented by using tow proving rings to calculate the applied load on the soil-stone column combination and also calculating the load applied on the stone column only. The dimension of the foundation which is a steel plates having 5 millimeters in thickness and 220 millimeters diameter. There are four holes in theses plates, (the distance between the holes = (2*D)), D is the diameter of the stone column. The stone column is positioned inside very soft clays with undrained shear strength between 6kPa and 12kPa. The stone column is made from crushed stone, the (L/D) the length of the stone column to the stone column diameter ratio were ranging between (L/D=6) and (L/D=8). The program procedures containing thirty tests these tests implementing on a single, two, three and four columns in order to estimate the concentration ratio of the stress and the bearing enhancement ratio(improvement ratio) (q treated/q untreated) of stone columns. The laboratory test explained that when using stone columns with (L/D=8) the ratio of stress concentration (n) =1.4, 2.4, 2.7and3.1 when the shear strength of soil treated with single, two, three and four columns (Cu=6kPa). And by using (L/D=6) the n=1.2, 2.2and 2.8, noticing that the increasing of the ratio of concentration stress (n) depends on the increasing of the shear strength of the treated soil. The increasing happened when the shear strength (Cu) between 9 and 12kPa.

(Fattah, 2012) at the time when the load applied, the pile made from granules is to be deformed by the effect of the load causing bulging in the pile the bulging effect transferred to the subsoil instead of transferring to the soil end layer (strata) and then the load to be distributed to the upper layers of soil profile making the soil supporting the load. The ultimate bearing capacity and the strength of soil can be increased and reducing the compressibility. Most of the predicting theories about the ultimate bearing capacity for isolated and single pile made from granules, the theories depending on the confined lateral stress supporting the pile is said to be the final negative resistance that the surrounding soil can gain while the granular pile bulging to outside. The economic and effective solutions for implementing embankments are the use of (piled stone columns) on soft soils. This using can save the construction time and effort and reducing settlement and reducing cost, it can be consider as an enhancement method for soft soil under the light structures such as a rail or roads embankments.

(Fattah et al., 2014) for the embankments models laying over soft soil strengthen with stone columns with different ratio of (L/D) stone columns length to the stone column diameter (2*L/D) and different distances between the stone columns.

Various heights of embankments used and 21 models implemented on a soil having (Cu) undrained shear strength almost equals to Cu=10kPa. The models having embankment of stone columns with different ratio of spacing to diameter ranging from 2.5, 3, and 4. Choosing three different heights for embankment 200 millimeters, 250 millimeters and 300 millimeters and using pressure cells for calculating effective the vertical pressure directly on the stone column positioned in the middle downward the central line of the embankment and for the stone column fixed at the edge for the entire models while the third pressure cell positioned at the bottom of the embankment between two columns for measuring the effective vertical stress applied on the strengthened soft soil, the models of the built embankments on a treated soft soil with normal stone columns located at spacing of 2.5 millimeters explained that the maximum bearing enhancement ratio for height of 200 millimeters, 250 millimeters and 300 millimeters equals to (1.21 ,1.44 and 1.7).

Showing also that the maximum ratio of settlement enhancement for heights ranging from 200 millimeters, 250 millimeters and 300 millimeters equals to (0.78, 0.67 and 0.56). Tow behaviors of the (column-wall) were compared depending on the column geometry and the column characteristics this comparison implemented by (Zhang et al., 2014). A numerical analysis (3-D) performed for investigating the stability of the embankment supported by stone column over a soft soil and also (2-D) numerical models by using the way of (column-wall) putting in mind the stress concentration factors and the ratio of area replacement and the soil states within long and short periods conditions. With short periods of time term, the numerical analysis explained that the method of the (equivalent area) made a slip surface that is critical and continuous for stone column supporting embankment laying on a soft soil. Although the (column-wall) method is not producing this slip continuous surface but it is not using any more, the (equivalent area) method(model) gives safety factor higher than that producing by (column-wall) method(model) but with long period time term the difference between the calculated safety factor to be reduced.

2.6 The Ultimate Bearing Capacity of A Footing Laying On Layered Soil (Stratified Deposit Soil)

For designing a foundation the foundation must achieve tow important requirements the strength and to be serviceable. And the soil beneath it should be capable of supporting the structural loads laying on it without shear failure and without sequenced settlement. The failure in soil surfaces took place under a specified stress and conditions. (Das, 1997) defined the bearing capacity as the soil and foundation capacity to support the applied loads due to the structures upon them without the occurring of shear failure or sequenced settlement. The naturally or artificially deposits that made inside the soil profile through each layer, in that case the soil is said to be homogenous. It is important to calculate the soil profile and the bearing capacity through them the shear strength factors of the layers will be determined, the shear strength factors such shape and size of footing, thickness of the soil upper layer, the ratio of the thickness of the upper layer to the width of footing.

Theoretical studies supposed that the subsoil is homogenous to a deep depth. But in nature the soil is a composition of clay, silt, and sand with various proportions that can be determined through the classification and analyzing of a specific profile of soil, each layer has it is composition and characteristics of strength. An average profile readings can be taken in analyzing procedures with a condition that the failure surface of soil should not pass the other layers of the soil, if that happened then the soil is said to be non-homogeneous. The current analyzing is depending on a system based on two layers of soil. The basement of the upper layer is existed on a (D) depth under the ground level; the ultimate failure surfaces may be located at the upper layer or it may cross the two layers. Sometimes the upper layer might be strong or weak and the same think might be for the lower layer. In both cases the analyzing must implement through determining the (c-φ) to classify the composition of the soil layers if they belong to sand groups or clay groups according to soil classification. The first analysis was done by (Button, 1953) who supposed ($\varphi = 0$) only for saturated clay. After the analyzing of (Button, 1953), (Brown and Meyerhof, 1969) they explained that the (Button) analyze gives inexact results. Later (Vesic) supposed the two types of soil is existed in each layer, clay and (c-φ) types of soil but there was no endorsement or agreement on this analysis. In 1974 (Meyerhof) assumed the exist of a system of two layers one of them containing a sand of a known dense lay on soft clay, while the other layer containing loosely sand lay on clay with known stiffness. (Meyerhof) stand with his theory and made a model tests for supporting his theory. Later in 1978 (Meyerhof and Hanna) supported the (Meyerhof, 1974) analysis by involving soil of (c-φ) supporting that with model tests. The (Meyerhof) analysis in 1974 and (Meyerhof and Hanna) in 1978 analysis is the reference of the current section.

2.6.1 The soil of layers (layered soil) the bearing capacity estimation (Theoretically)

State 1: weak deposit beneath strong layer

By looking at the figure (2.6) (a):

B: is the width of a strip footing.

D_f: is the depth of footing from the ground level, within the strong layer. (layer1).

H: is the depth starting from the base of footing to the weak layer boundary. (layer2).

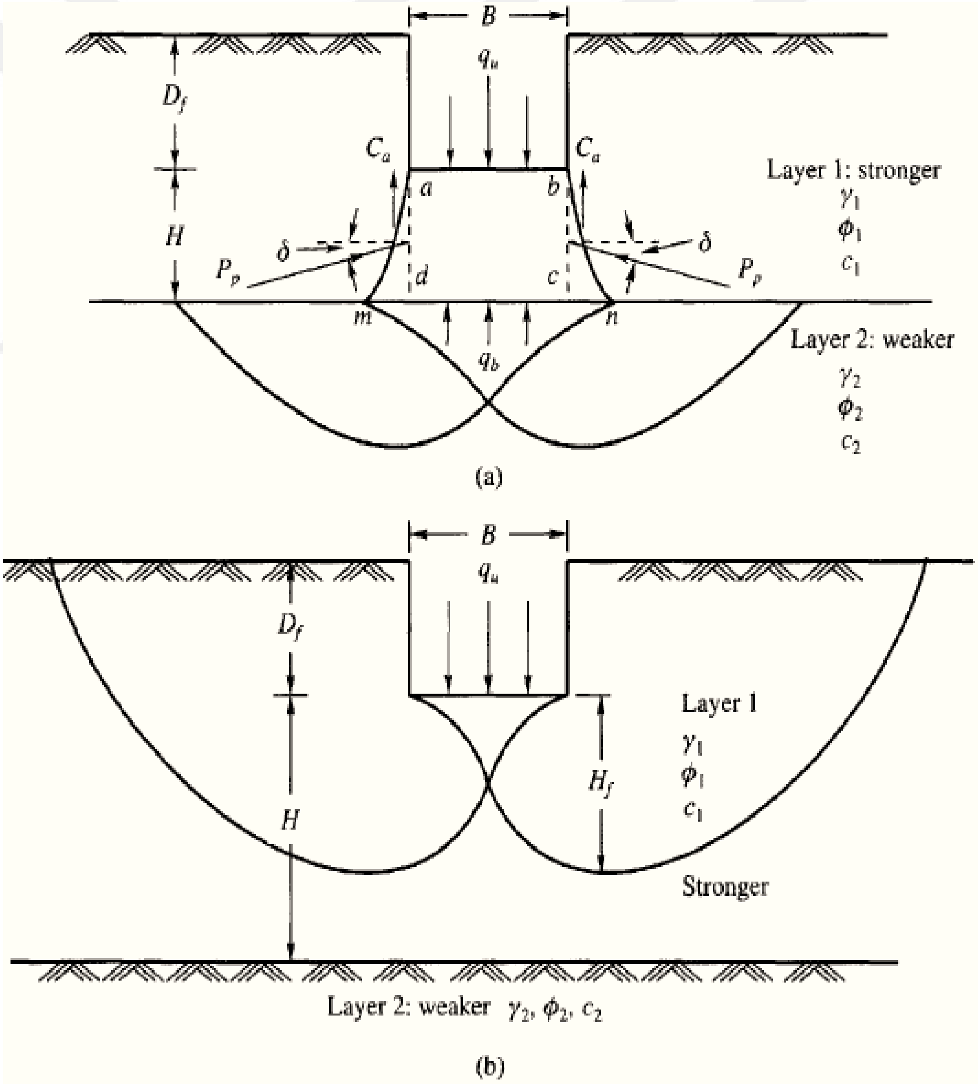


Figure 2.6: Failure Of Soil Beneath Strip Footing Undergo Vertical Load On Strong Layer Resting On Weak Layer

Source: Meyerhof and Hanna, 1978

When the (H) the depth calculated from the footing base to the boundary of the weaker layer is not adequate for forming a full plastically failure surface within the first layer the stronger layer, the load will transit to the boundary of the second layer the weaker ones causing failure in the weaker layer. When the (H) is kind of being larger, then the failure will occurring in the stronger layer the first one as explained in figure (2.6) (b). For surfaces of thick and homogenous coats the ultimate bearing capacity can be explained as follows for the layers 1 and 2:

For layer 1:

$$q_1 = C_1 N_{C1} + 0.5\gamma_1 BN_{y1} \tag{2.9a}$$

For layer 2:

$$q_2 = C_2 N_{C2} + 0.5\gamma_2 BN_{y2} \tag{2.9b}$$

Where:

N_{C1} , N_{y1} : load bearing capacity factors of layer 1 with shear resistance angle $\emptyset_1$

N_{C2} , N_{y2} : load bearing capacity factors of layer 2 with shear resistance angle $\emptyset_2$

The ultimate bearing capacity of the layer1 can be explained as:

$$q_u = q_1 = C_1 N_{C1} + q'_{\circ} 0.5\gamma_1 BN_{y1} \tag{2.10}$$

$C_a = c_a H$ Where: C_a is the adhesıve force.

$$F_f = P_P \text{ SIN } \delta \quad \text{Where: } F_f \text{ is the frictional force.} \tag{2.11}$$

P_P : is the earth passive pressure / unit length of footing.

δ : is the inclination of the earth paasive pressure P_P.

or the system of two layers, the ultimate bearing capacity equation can be written as:

$$q_u = q_b + \frac{2(C_a + P_P \text{ SIN } \delta)}{B} - y_1 H \tag{2.12}$$

Where: q_b is the ultimate bearing capacity of layer 2.

So the P_P equation can be written as:

$$P_P = \frac{y_1\ H^2}{2\cos\delta}\left(1+\frac{2\,D_f}{H}\right) k_p \tag{2.13}$$

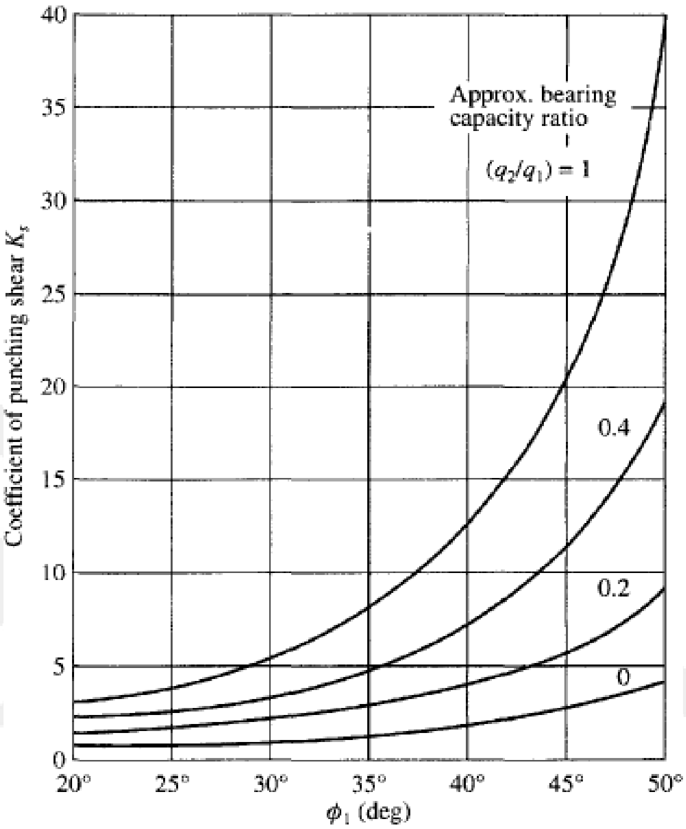


Figure 2.7: Coefficient of Punching Shear Resistance Under Vertical Loading

Source: Meyerhof and Hanna, 1978

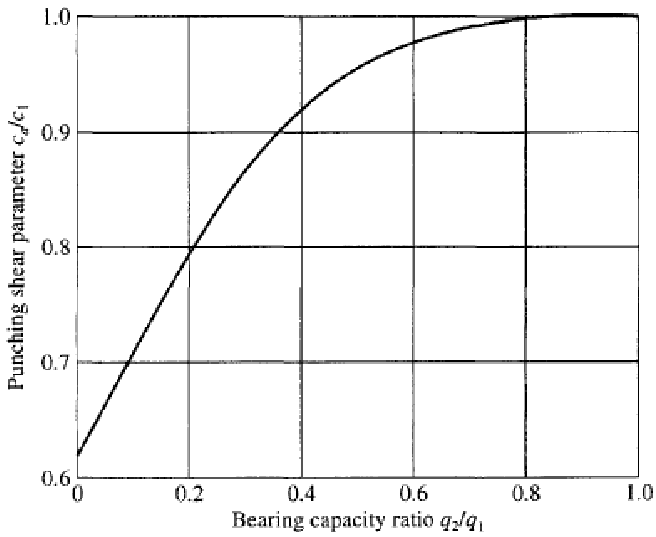


Figure 2.8: Plot $Ca/_{C1}$ versus $^{q_2}/_{q_1}$

Source: (Meyerhof and Hanna, 1978)

The ultimate bearing capacity equation for a footing of a rectangular shape can be written as:

$$q_u = q_b + \frac{2c_a H}{B} + 1 + \frac{B}{L} + \frac{\gamma 1 H^2}{B} + 1 + \frac{2Df}{H} + 1 + \frac{B}{L} k_s \tan\varphi 1 - \gamma H \le q_t \quad (2.14)$$

State 2: Saturated soft clay beneath dense sand (*φ* =0):

The value of the bearing capacity can be explained as:

$$q_b = C_2 N_{C2} S_{S2} + \gamma_1 (Df + H) \quad (2.15)$$

When:

$\varphi = 0$ *FROM TABLE* (2.1)*AND FROM TABLE* (2.2) (*Meyerhof*, 1963).

$N_C = 5.14$ And $S_{c2} = 1 + 0.2\ (\frac{B}{L})$

Table 2.1: The Bearing Capacity Factors

φ	N_C	N_q	N_y *Hansen*	N_y Meyerhof	N_y Vesic
0	5.14	1.0	0.0	0.0	0.0
5	6.49	1.6	0.1	0.1	0.4
10	8.34	2.5	0.4	0.4	1.2
15	10.97	3.9	1.2	1.1	2.6
20	14.83	6.4	2.9	2.9	5.4
25	20.71	10.7	6.8	6.8	10.9
26	22.25	11.8	7.9	8.0	12.5
28	25.79	14.7	10.9	11.2	16.7
30	30.13	18.4	15.1	15.7	22.4
32	35.47	23.2	20.8	22.0	30.2
34	42.14	29.4	28.7	31.1	41.0
36	50.55	37.7	40.0	44.4	56.2
38	61.31	48.9	56.1	64.0	77.9
40	72.25	64.1	79.4	93.6	109.4
45	133.73	134.7	200.5	262.3	271.3
50	266.50	318.50	567.4	871.7	762.84
Note: N_C and N_q *are the same for all the methods.*					

Source: (Meyerhof, 1963)

Bearıng capacıty factors of Meyerhof:

N_c , N_q , AND N_r are the bearing capacity factors of Meyerhof depending on soil friction angle φ.

S_C , S_q and S_y are the shape factors.

$d_{\ c}, d_q\ and\ d_y$ Are the depth factors.

i_c , i_q and , i_γ are the incline load factors.

Table 2.2: Meyerhof Bearing Capacity Factors

Friction angle	Shape factors	Depth factors	İncline load factors
At any φ	Sc= 1+0.2kp(B/L)	dc=1+0.2$\sqrt{kp}(\frac{D}{B})$	ic=iq=$(1-\theta/90°)^2$
$\varphi = 0$	Sq=Sγ=1	dq=dγ $= 1$	iγ=1
$\varphi\ \geq 10°$	Sq=Sγ=1+0.1kp(B/L)	dq=dγ $=$ 1+0.1$\sqrt{kp(D/B)}$	$i\gamma = \varphi$

Source: (Meyerhof, 1963)

θ: Is the angle of load in degrees.

$\theta = \tan^{-1}(\frac{Qh}{Qv})$

γ : is the unit weight of soil.

B: The width of footing.

L: is the length of footing.

D: is the depth of footing.

C: the cohesion of soil.

Kp: the coefficient of earth passive pressure, Kp= $\tan^2(45+ \varphi /2)$

Passive earth pressure: is the pressure that earth compressing in the horizontal direction. (Lateral pressure).

Again from Meyerhof tables (2.1) and (2.2):

$$q_b = 1 + 0.2\ \frac{B}{L} 5.14\ c_2 + \gamma_1\ (\ Df + H) \tag{2.16}$$

For c_1=0 then the q_1 will be explained as:

$$q_{1=} \gamma_1 Df\ N_{q1}\, S_{q1}\ + 0.5\ \gamma_1 \mathrm{B} N\gamma_1 \mathrm{S}\gamma_1 \tag{2.17}$$

Now we can write down the equation for the ultimate bearing capacity as following:

$$q_u\text{= 1+0.2}\frac{B}{L}\, 5.14C_2\ + \frac{\gamma_1 H^2}{B}\ \ 1\text{+}\frac{2Df}{H}\ 1\text{+}\frac{B}{L} k_s tan\,\varphi_{1+}\ \ \gamma_1\, Df \leq\ \gamma_1 Df$$

$$N_{q1}\, S_{q1}\ + 0.5\gamma_{1\mathrm{B}N\gamma_1 \mathrm{S}\gamma_1} \tag{2.18}$$

The $\frac{q_2}{q_1}$ ratio can be written as following:

$$\frac{q_2}{q_1} = \frac{(C_2\ NC_2)}{0.5\,\gamma_1\ BN\gamma_1} \quad = \frac{5.14C_2}{0.5\gamma_1\ BN\gamma_1} \tag{2.19}$$

State 3: loose sand beneath dense sand:

The ex-equation for ultimate bearing capacity to be used in this state:

$$q_u = \gamma_1(Df + H)N\,q_2 + 0.5\,\gamma_2\,BN\gamma_2\,S\gamma_2 + \frac{\gamma_1 H^2}{B}\quad 1 + \frac{B}{L}\quad + 1 + \frac{2Df}{H}\,K_S\,\tan\varphi 1 - \gamma_1 H \le q_t \tag{2.20}$$

Where:

$$q_t = \gamma_1\ Df\ \ \mathrm{N}q_1\,\mathrm{S}c_1\ + 0.5\ \gamma_1\ BN\gamma_1\,\mathrm{S}\,\gamma_1 \tag{2.21}$$

$$\frac{q_2}{q_1} = \frac{\gamma_2 N\gamma_2}{\gamma_1 N\gamma_1} \tag{2.22}$$

State 4: saturated soft clay ($\boldsymbol{\varphi_{2=0}}$) beneath stiff saturated clay ($\boldsymbol{\varphi_{1=0}}$):

The ultimate bearing capacity of this state can be expressed as:

$$q_u = 1 + 0.2\ \frac{B}{L}\ \ 5.14C_2\ + 1 + \frac{B}{L}\ \ \frac{2\mathrm{C}_{a}H}{B} + \gamma_1\,Df\ \le\ q_t \tag{2.23}$$

$$q_t = 1 + 0.2\ \frac{B}{L}\ \ 5.14C_1\ + \gamma_1\,Df \tag{2.24}$$

$$\frac{q_2}{q_1} = \frac{5.14C_2}{5.14C_1} = \frac{C_2}{C_1} \tag{2.25}$$

2.6.2 The ex-studies and researches regarding the techniques of soil replacement

İn a weak clays at least the soil should satisfy a moderate increasing in bearing capacity on of the enhancement ways is the use of the granular piles and stone columns. The developed plane of strain is a granular trench. The granular trench is a two- dimensional plane of strain that is various according to conditions. Formed by using a geosynthetic around it. This method is using in enhancing the characteristics of soil by replacing the native soil with a soil with better characteristics. For the current topic the a kind of a failure is supposed to take place in the granular trench, and some statistical analysis methods to be used in order to predict the ultimate bearing capacity of footing resting on an enhanced or stabilized soil. İt is very clear that the granular trench supported and strengthens the weak soil. Studying the influence of the factors of the bearing capacity upon variable groups of the affected parameters. (Madhav and Vitkar, 1978).

(Abdulbaki et al., 1993) investigated the influence of using a one reinforced stiff layer located inside the granular soil on the footing bearing capacity exposed to loads acting upon the centroid of the footing(concentric loads) and also exposed to loads acting away from the axis(the centroid of the footing) called (eccentric loads) and inclined loads. Through the experimental works the investigation was done and proved the supporting of using the reinforced stiff layer has gave a significant influence on the footing bearing capacity.

This influence is seemed to be three times more than without the using the reinforced layer. Within the investigation the length of the reinforcement was put in mind and it appeared that by increasing the length the bearing capacity is not affected when the length to be more than 1.25 of the footing width. Also the continuation of the reinforcement beneath the footing is investigated. Appeared that when the reinforcement gap is increased the bearing capacity reduces and equal to zero when the width of the gap is equal to the footing width.

(Radolsaw et al., 1995) in order to find a way that can be applied to calculate the bearing capacity of a footing laying on two layers soil, and to applied for the groups of parameters of the two layered soil, He used a kinematic method of limit analysis through which the average limit pressure can be calculated leading to find the value of the bearing capacity. The concept supposed giving the maximum limit load (the yield load) of the actual limit loads. The concept produces the collapse load (failure load) that is very close to the real loads of collapse for the elastic and having perfect plasticity characteristics materials, The concept of the maximum or upper load sees that the average of losing the energy is more than or equal to the average effort that the external forces make at any acceptable kinetic mechanism and when the material characteristics are knew and the mechanism of collapse are also knew, so the true upper limit load can be found through the equality between the average of the lost internal energy and the effort of the external forces that causing the loss in energy. The interruption in speed at the footing edges to be bent on at the layers interface. At the bending action an angle took place, the formed angle is equal to difference of internal friction angle between the two layers. This is how the mechanism of failure (collapse) to be constructed. And through the concept it was found that the depth of the collapse was depending on the clay layer strength. It was also found that the results were expressed as limit pressures and it’s not like it was a bearing capacity factors. And the limit pressures are totally depends on the angle of internal friction of the sand and depends on the thickness of the sand layer, and the surcharge pressure and the clay cohesion. Non-dimensional charts were used for representing the results for the difference values of the angles of internal friction.

(Burd and Frydman, 1996) they submitted a respectable concept dealing with the bearing capacity of the layered soil. The purpose of the concept was of two aims the first aim was to compare between the independent numerical analysis results of the concept owners and the design charts of the authors. The concept paper to be used as a reference for the analysis basement that assumed by (Haana and Meyerhof, 1980). For solving such problems. Results were obtained through the use of a numerical study of thin and layers resting on deposits of soft clay. The first group of results was gained through the use of (finite-element approach), while the other group of results was gained through the use of the (finite-difference code FLAC). Through the comparison of the results obtained from the two approaches the authors came up with a result saying that the kinematic approach for an in mind parameters can express the bearing capacity with a large scale. They also dealing with the analytical approach especially the approach that suggested by both (Hanna and Meyerhof, 1980) that was basing on the equilibrium analysis of the granular soil beneath the footing. In order to calculate the bearing capacity of the system by using this approach. It is so important to calculate the amount of shear force which effecting on the sides of this mass of soil. Through entering the value of the coefficient of punching shear (Ks) which is connected to a group of parameters, unit weight and the depth of the layer of sand. At the design charts of (Meyerhof and Hanna, 1980) these to parameters were not mentioned the unit weights (Y) and the depth of the sand layer (t), although the approach of (Meyerhof and Hanna, 1980) seems to be acceptable, the (limit-equilibrium) approach, but only for specified values of unit weight(Y) and depth of the sand layer (t) that mentioned in their design charts and the approach was basing on them. Therefore it seems to be not workable for a wider scale of values of these two parameters.

(Burd and Frydman, 1997) they suggested exist of a soil of a uniform layer of sand beneath it a homogenous and thick coat of clay. Laying over this soil a rigid plane-strain footing. Regarding the bearing capacity it is very limited method when the cases including a comparison of the thickness of the sand layer to the footing width. Considering that the in all the cases the ground surface is horizontal and the interface between the two layers are horizontal too. The concept is to consider the sand layer is a drained layer while the clay layer is undrained layer.

(Kenny and Andrawes, 1997) for a sand layer resting on clay deposits. They suggested a theoretical model for the footing placed on the two layers. At the laboratory they implemented tests for the model to estimate the relation of stress-settlement only for sand. And for the subgrade soil (the clay) only. And for the both layers (sand resting on clay). The results were represented in charts after comparing the results with the data of experiments of other researchers. The stress- settlement relationship was submitted in a non-dimensional form for all the tests. For the condition of the subgrade layer the clay layer, this layer shows local failure properties. According to (Terzaghi and Vesic) by reducing the factors that making lack in the clay shear strength this will give more predictable reliability of the bearing capacity of the sand laying on clay. Often the local ways of spreading seen to be shy of giving the real prediction of bearing capacity. But for this concept by using spreading angle for giving accurate estimations otherwise this concept will not give accurate estimation without using spreading angle of course by put in mind the effect of the local failure of clay.

(Merifieild and Sloan, 1999) they estimated and predicted the ultimate bearing capacity of a surface strip footing laying upon clay profile of horizontal layers. A lot of experimental and semi-experimental equations may be used. Through using them a close solution of the problems may be achieved. Lately (Forkiewics, 1989) he assumed a maximum (upper) limit way by assuming the exist of acceptable kinetical mechanism for failure, although this way or method useful but a limited results had been produced. The method of the upper limit largely used for estimating the layered clay bearing capacity. But the method leads to obtaining a safety factor value for design less than the actual safety factor. The better likelihood is to use the lower limit instead of the upper limit this way gives a more safety design. When the upper limit solution and the lower limit solution to be used together they lead to estimate the actual failure (collapse) load form bellow and above. The purpose that pushed (Merifield and Sloan) to make use of the advantages of the limit theory to put the actual collapse load in brackets by studying two ways of solutions for estimating bearing capacity of footing clay profile of two layered.

By using the numerical techniques that are developed by (Sloan, 1988 and Sloan and Kleeman, 1995) that were basing on the limit theories of finite elements and classical plasticity. For a profile of two layered consist of upper strong layer over soft clay layer a lot of failure mechanism were found due to the exist of the strong upper layer and its thickness when compared with the lower weak layer. Lead to the upper limit theory (upper bound theory) solutions and the solutions of semi-experimental ways incapable of putting the failure system in a model. The semi-experimental and the upper limit theory and also the experimental ways may differ from the bound solutions with a percentage up to +/- %20 and the risk may be greater when the upper layer is so strong as compared with the lower layer or the depth of the upper layer is more than the half of the width of the footing. Many ways were used in past and still using nowadays for the enhancement of the bearing capacity of soil depending on the type of the structure and the equipment's availability and the soils characteristics, the use of the granular trench the (case of the two-dimensional plane strain of the stone column) one of the ways used for enhancement. This topic introduces the finite elements approach for soft cohesive soil modeling, reinforced material and granular trench through the use of a computer application (program) named **(sigma / W),** (Fatah et al., 2010) had simulated both the cohesive and the granular soil behavior by using non-liner elastic soil model, at the other hand the liner elastic soil model used for simulating the reinforcement material. The variation took place angle of friction of trench soil, also in depth, width and the shape of the granular trench, also the variation happened in the reinforcement material in its modulus of elasticity, the locations and the layers number of reinforcement also had varied. Un- analyzing process implemented for the (sloped) granular trench into two states:

1. Lined conditions.
2. Unlined conditions.

The uses of the granular trenches increase the bearing capacity and reduce the settlement when used under the footings this fact had showed by the results. More than that the results showed that the using of polymers as a reinforcing material will have a considerable influence on both the settlement and the bearing capacity.

Polymers: material of a molecular structure to be built up mainly or completely by using numbers of units that are similar by gathering the units together for example the synthetic organic materials used as resins and plastic.

For the granular trenches that are reinforced or the non-reinforced granular trenches the ratio of depth has an effective influence on the ratio of settlement, the ratio of settlement decreases as the depth ratio increases and it found that the appropriate depth ratio is to be equal to (2). For making a granular trench with depth equal to (X) the footing width is equal to (B) the perfect influence that reduces the settlement is to be (X / B= 0.75).

(Unnikrishnan, et al., 2010): he explained the response of loose sand deposits that resting on granular trenches non-liner finite elements simulation process implemented to predict the attitude of the load of settlement during the exist of granular trench for a strip footing. A load test implemented on a strip footing model without using and by using a granular trench to satisfy the requirements of the finite element approach. A well graded sand from river used for soil replacement as a replaced soil in the granular trench and a see sand used as bulk material with low relative density. The soils characteristics were determined through the laboratory tests. Implementing an investigation for some parameters of the trench to estimate their effect such as the size and the shape of the trench by simulation process of non-liner finite element approach. The simulation results approved that the using of granular trenches in supporting the strip footing will enhance the response.

Loose sand deposits: is a type of sediment that is composed of loose sand grains that had been deposited by water, ice, and water they can be found in riverbeds, deserts and beaches. To be used in structures and in civil engineering due to their well drainage and stability by considering their characteristics.

3. RELIABILITY IN GEOTECHNICAL ENGINEERING

3.1 Introduction

In geotechnical engineering for the uncertainty there is no way out. The geotechnical engineering deals with natural raw materials and natural materials that are with variable characteristics (spatial variability). The characteristics of the material can be obtained from the field or by experimental tests. These characteristics dependable it depends on the location in which the materials located and to be taken (borehole). The characteristics depend also on borehole methods and number of sampling, method of sampling (distributed or undistributed). Procedures of laboratory tests and the ways used. The explanation of the statistical results from testing data and the method of analyzing for a certain problem like the methods implemented by Meyerhof, Terzahgi and Vesic for bearing capacity, the errors of the use of instruments and the error of the human should be in mind and also the uncertainty came up with the system performance. In some cases although the failure possibility seem to be increased but the system shows a high value of safety factor in deterministic analysis. The choose of the safety factor in deterministic analysis depends on the ex-experiments and the results of failure and the variability of the natural soil, errors of instruments (measurements), statistical approximations, the transformation of the model and the analytical limitations for models. Neglecting the value and sources of the uncertainty came up with the system. For such reasons, a value of factor of safety from 2.5-3 to be used in order to avoid the problems of the bearing capacity of geotechnical projects. And in the deterministic approach it is difficult to predict the structure serviceability as showed by (Fendon and Griffiths, 2008).

3.2 Reliability of the System

The relation between the loads the structure (the system) must carry and the real capability of the structure to carry. The reliability expressed as reliability index (β).

The system failure (P_f) is very connected with the reliability index. Each of them completes the other. The risk is unsatisfactory work or the possibility of failure while the reliability is a successful work or satisfactory work.

The reliability benefits or advantages regarding the traditional (conventional) design are:

1. The full identification of the all risks inside the project.
2. The failure possibility through the reliability can be identified for each design method.
3. Serviceability of the structure for which the design placed.
4. Help to make analyzing for the risk-cost relationship, helping with making the decisions.
5. Reliability considers any sources of risk inside the project.

In many times, the deterministic ways to be used in analyzing and in the design of the structures. These ways are dealing with the minimum limits of the safety factors and deals with the minimum limit of the required materials characteristics. The deterministic ways does not accurately explain the uncertainties in the engineering analysis and design. In order to treat the uncertainty the probability's theory were widely accepted and used in engineering design through the using of the statistical knowledge for the random variables through the use of their the values of mean and standard deviation in the applications. Probability methods and among them the reliability analysis as showed by (Row and Fraser, 1994) are to be used in a continuously in structural and geotechnical engineering. The reliability calculations give a way for evaluation the multi-faces of uncertainty and a way to recognize between conditions in there the faces of uncertainty are increased or decreased; also the reliability analysis provides capability for determining the suitable safety factors (Christian and Baecher, 2001). And the other targets of the design and leads to better evaluation for the relative importance for uncertainty face regarding the different parameters.

The complicated interaction between the theory and the application (practice). For predicting the real behavior a real assumptions must be put for the material characteristics. According to their geological resources and their behavior in the laboratory and field. And fact assumptions should be made for the limits conditions. Frequently the theory must be tested regularly to know if the theory will satisfy the requirements or not. (Terzaghi et al., 1996) and to examine each of them against the other. (The theory against the practice), and to connect each to another, thus the theory becomes more real and more cohesive.

The realistic analysis the basic value of them is to understand the problems since these problems became understood then it can be solved. The understanding of the problems gives a view through it the faces of uncertainty cannot be neglected or cannot be avoided. But it can be determined and managed, even capability for making accurate and deterministically theoretical predictions cannot be achieved even by the use of the most advanced ways. There are many fields in them the analysis may help in understanding a problem but the problem would not be solved by using a predication method. And the direct use of the field experience is so important and suitable and safe if the mechanisms are understood since these mechanisms governing the behavior. The theoretical view provides the language through it the problems can be understood and learned and to in public use. In order to achieve success the reality must be merged. This type of methodology is complicated and brings unnecessary mistakes, these mistakes or errors can be solved by the use of quality assurance, but quality assurance is applied on non-realistic calculations leads to by accident true results. The alternative solution is to use the practical guidelines carefully connected with the geology and the structure type with alerts about where the uncertainty faces located. This will provide a better and more effective methodology or approach for solving the daily problems. (Vaughan, 1994) stated that the shape of geotechnical engineering expertise that should be given to the non-expertise engineers is governing to what degree the expertise geological engineers are using their experience.

3.3 The Selection of the Coefficient of Reliability

Standard deviations of the undrained shear strength, the angle of internal friction for the stone, and the unit weight of soil must be determined as part of the technique. The coefficient of variation is the ratio of the standard deviation to the mean value.

Table 3.1: Coefficient of Variation of Geotechnical Parameters

Property	Coefficient of variation	Source
Unit weight (γ)	3-7 %	Harr (1984), Kulhawy (1992)
Effective stress friction angle (φ')	2-13 %	Harr (1984), Kulhawy (1992)
Undrained shear strength (Cu)	13-40 %	Harr (1984), Kulhawy (1992)

Source: Harr (1984) and Kulhawy (1992)

3.3.1 Moments

$$M = \sum_{i=1}^{N} P(i)X(i) \qquad (3.2)$$

For a system of disconnected (discrete) parallel vertical forces acting on a rigid beam at spacing respectively:

The forces can be written as: P (1), P (2), and P (N)

Spacing can be written as: X (1), X (2), and X (N)

M is the magnitude of the equilibrant as shown in figure (3.1a).

The application point is:

$$\bar{X} = \frac{\sum_{i=1}^{N} P(i)X(i)}{\sum_{i=1}^{N} P(i)} \qquad (3.3)$$

If we suppose that the disconnected (discrete) forces P (i) are the frequencies of the happening results (the outcomes) (N), X (1), X (2)… X (N). Then the magnitude of the equilibrant = 1. As explained in figure (3.1a). AS a result the equation (3.3) can be written:

$$E[X] = \bar{X} = \sum_{i=1}^{N} P(i)X(i) \qquad (3.4)$$

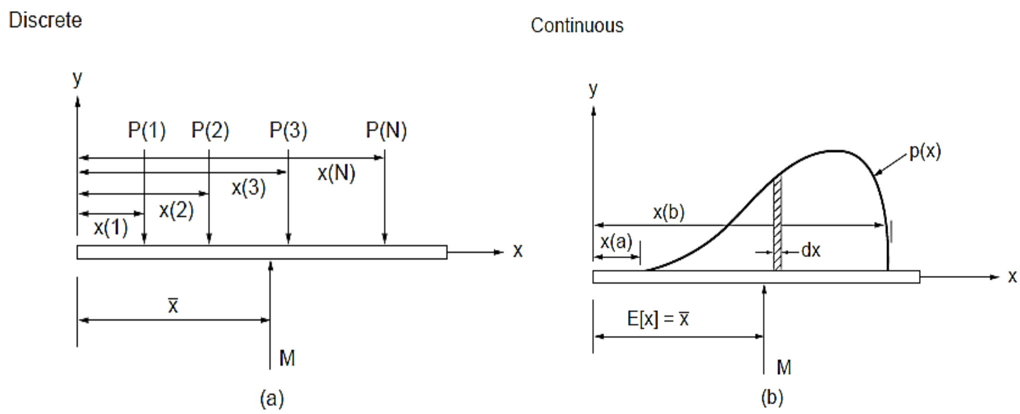


Figure 3.1: The Equilibrant of Disconnecting (Discrete) And Continuous Distribution

Source: (Harr, 2002)

The expected value (mean) gives the position of the central direction for distributing random variable in return to statics, providing the dispersion distribution of forces around the centroid axis at X= E$[X]$ as shown in figure (3.1b) is given by the moment of inertia:

$$I (y) = \int_{X(a)}^{x(b)} (x - \bar{X})^2 \ P(x)\ dx \tag{3.5}$$

The distribution of a random variable is measured by an equivalent measurement of distribution called the variable variance which expressed as: $\boldsymbol{v}[x]$

Disconnected (discrete) $\boldsymbol{v}[x] = \sum all\ x(i)\ [x(i) - \ \bar{X}\ ]^2$. P (i) (3.6)

Continuous $\boldsymbol{v}[x] = \ \int_{x(a)}^{x(b)} (x - \bar{X})^2$. P (x) dx (3.7)

These equations can be written as:

$$\sqrt{v[x]} = E\ [(x - \bar{X})^2] \tag{3.8}$$

For more acceptable equation after expansion:

$$\boldsymbol{v}[x] = E\ [x^2] - (\ E\ [x]\ )^2 \tag{3.9}$$

The equivalent equation of the parallel-axis theorem for the second moment of area (moment of inertia).

(Harr, 2002) stated that the variance is the square root of the random variable's values. The best way to measure the dispersion of a random variable (x) is to take its positive square root. (Compare with the standard deviation, the radius of mechanics' gyration).

$$\sigma[x] = \sqrt{\text{v}[x]} \tag{3.10}$$

The coefficient of variation is a relative measure of the scatter of the random variable (x):

$$\text{CoV} = \frac{\sigma\ [X]}{E\ [X]} * 100\% \tag{3.11}$$

The exponential distribution's standard deviation $\sigma[x] = 1/a$

The straight line can be considered suitable and this can be expressed as the correlation coefficient P:

$$\text{P}= \frac{CoV\ [X,Y]}{\sigma[X].\sigma[Y]} \tag{3.12}$$

According to Harr (2002), covariance from analog to statics correlates to the inertia product.

$$P= \frac{CoV\ [X,Y]}{\sigma[X].\sigma[Y]} \tag{3.12}$$

Where: $\sigma[X]\ AND\ \sigma[Y]$ are the standard deviation respectivily and the covariance is the measure of despersion of the data and expressed as:

$$CoV\ [X,Y] = \frac{1}{N}\ \sum_{i=1}^{N}[xi - \bar{y}]\ [\text{yi} - \bar{y}] \tag{3.13}$$

3.3.2 Point estimating approach for several random variables

The methodology was developed generally by Rosenblueth, (1975) for any number of connected (correlated) variables, such as the function of three random variables:

y = y $[x\ (1), x(2), x(3)]$ and P (i, j) is the correlation factor between x (i) and x (j).

$$E\ [y^N] = P\ (+ + +)\ y^N\ (+ + +) + P\ (+ + -)\ y^N\ (+ + -) + \ldots + P\ (- - -)\ y^N\ (- - -) \tag{3.14}$$

$\sigma[xi]$ Is the standard deviation for xi, the sign of P (i, j) can be obtained by the multiplication method of I and j. if the sign of i (-) and the sign of j (+) then:

(i) (j) (-)(+) = (-).

As follows:

$$y\ (\pm \pm \pm) = y\ [(\bar{x}\ 1)] \pm\ \sigma[x1], (\bar{x}\ 2) \pm \sigma[x2], (\bar{x}\ 3) \pm\ \sigma[x3]] \tag{3.15}$$

$$P\ (+ + +) = P\ (- - -) = \frac{1}{2^3}\,[1 + P(1,2) + P(2,3) + P\ (3,1)] \tag{3.16}$$

$$P\ (+ + -) = P\ (- + +) = \frac{1}{2^3}\,[1 + P(1,2) - P(2,3) - P\ (3,1)] \tag{3.17}$$

$$P\ (+ - +) = P\ (- + -) = \frac{1}{2^3}\,[1 - P(1,2) - P(2,3) + P\ (3,1)] \tag{3.18}$$

$$P\ (+ - -) = P\ (- - -) = \frac{1}{2^3}\,[1 - P(1,2) + P(2,3) - P\ (3,1)] \tag{3.19}$$

The equation (3.15) has eight terms. (2^3) . For the ($2^{\ M}$) the correlation coefficient is M (M-1) /2. (Harr, 2002).

3.4 Probabilistic Basics

(Harr, 2002) stated that the probability of success of a structure is called reliability (R).

While the probability of failure of a structure is called failure P (f).

When a set of data get distracted from its mean this is called standard deviation.

$$R + P(f) = 1 \tag{3.1}$$

3.5 Reliability Analysis

3.5.1 Capacity – demand

In the geotechnical engineering generally the adequately of the design through the comparison between the estimated resistance of the structure and the supposed loads.

The capacity (c) represents the resistance (the strength) and the (d) the demand represents the supposed loads to be applied on the structure.

Traditionally the designer form the factor of safety as (nominal values of a single – valued) of the demand (d) and the capacity (c) as expressed in figure (3.2a). Stated by (Ellingwood et al., 1980).

Through the application of footing analysis tools, Harr, (1980) demonstrated—and later developed by (Fattah, 2010)—that piles may be included in the capacity-demand concept. Known examples are the size of culverts and the amount of water they can hold, the structural capacity and the projected traffic flow on highways, the carrying capacity of the soil and column loads, and the capacity of traffic and predicted traffic flow.

$$Fs = \frac{C}{D} \quad \text{and} \quad \overline{D} = \frac{\bar{E}}{FS} \tag{3.20}$$

Generally the function of the demand results from a lot of uncertain compositions effect the system such as the loads of winds, loads of vehicles, the location of the raw water and the temperatures.

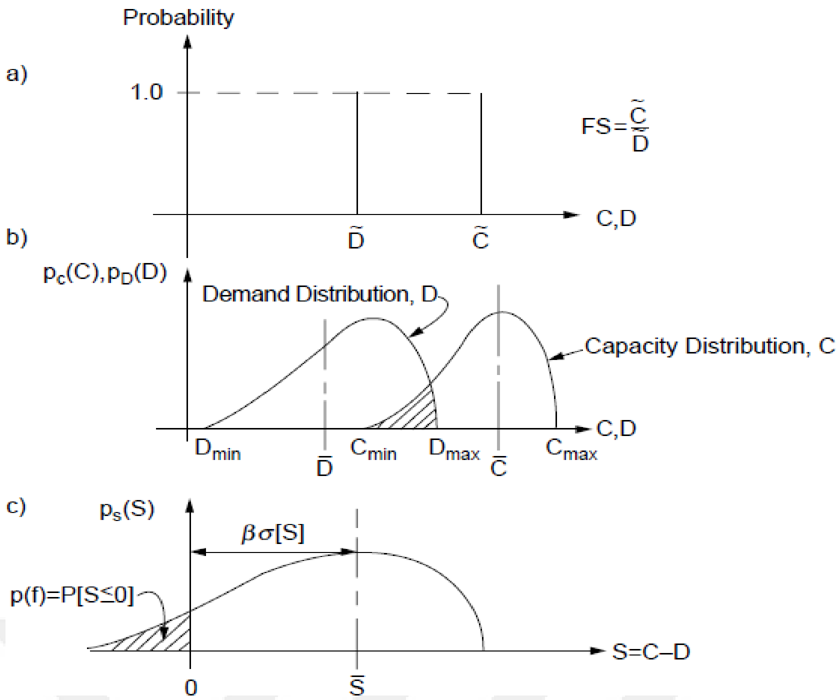


Figure 3.2: (a).Traditional factor of safety. (b).Capacity-Demand model. (c) Safety margin

Source: (Harr, 2002)

If we have allowable loads of 400 tan for each square meter, and the calculated loads is 250 tan for each square meter then the safety factor is 1.6 will be acceptable when the safety factor is larger than the minimum value gained from ex-expertise in such designs but when the safety factor is less than the minimum value then the system will be redesigned in order to minimize the induced loads.

Materials parameters variation, the errors of testing, the supervision of inspections and constructions procedures the function of capacity to be depended.

It is evident that the margin of safety is a random variable in and of itself. Failure is associated with the negative portion of the probability distribution, as figure (3.2) (c) illustrates.the area that is darkened.

This gray area represents the failure probability P (f).

The shaded area S = C - D $\leq$ 0

$$P(f) = P[(C - D) \leq 0] = P[S \leq 0] \tag{3.21}$$

Appears in the figure (3.2) (b) the functions of the capacity and demand as probability distributions. According to (Harr, 2002) if the minimum capacity is to be exceeded by the maximum demand then the probability of distribution will be interloped and the probability of failure will never equal to zero.

$$S = C - D \tag{3.22}$$

Where: (S) Is the margin of safety.

3.5.2 Reliability index

The inverted proportion of the margin of safety's coefficient of variation is the dependability index. Additionally, it is the number of standard deviations that the safety margin's mean exceeds.

$$\beta = \frac{\bar{S}}{\sigma\,[S]} \qquad \text{Type equation here.} \tag{3.23}$$

$$\beta = \frac{1}{CoV} \tag{3.24}$$

(Ditlevesen, 1981) from the definitions to be obtained:

$$E[a + bx + cy] = a + bE\,[x] + cE\,[y] \tag{3.25}$$

a, b, c are constants.

$$V\left[a + bx + cx = b^2 v\,[x] + c^2\,v\,[y] + 2b.c.CoV\,[X\,,Y]\right] \tag{3.26}$$

$$CoV\,[X, Y] \leq \sigma[x].\,\sigma[y] \tag{3.27}$$

$$V\,[a + bx + cx = b^2 v\,[x] + c^2\,v\,[y] + 2b.c.\sigma[x].\sigma[y].P\ ] \tag{3.28}$$

$$E\,[S] = E[C] - E\,[D] \text{ FROM EQUATION (3.27)} \tag{3.29}$$

$$\sigma^2[S] = \sigma^2[C] + \sigma^2[D] - 2P\ [C]\,.\,\sigma[D] \tag{3.30}$$

Then:

$$\beta = (\bar{C} - \bar{D}) / \sqrt{\sigma^2[C] + \sigma^2[D] - 2P\,\sigma\,[C]\,.\sigma[D]} \tag{3.31}$$

According to Haugen, (1968), ideal positive correlation has a maximum limit, while ideal negative correlation has a minimum limit. The total of the differences between two normal variables yields a normal variation.

Assuming that the demand and capacity functions are regular variations, we can then:

$$P\,(f) = 0.5 - \psi\,[\beta] \tag{3.32}$$

$\psi\,[\beta]$ **:** Is the normal probability distribution as mentioned in the tables of standard normal probability.

3.6 The Piles Based Design (Reliability Procedures)

(Harr, 2002) The design based on reliability should satisfy the following required requirements:

1. To be perfect the mathematical calculations to be reduced.
2. It must provide results (outputs) connected to the expected performance under the monitoring through the design life of the system.
3. It must not neglect the indices connected with the system such as the factor of safety or the reliability index and must collect knowledge about these indices and reduces the uncertainty.
4. It has to put in mind the importance of the capacity and the demand and their parameters and their contents and their influence.

3.6.1 The axially loaded piles and the application of reliability

The estimation of bearing capacity of the axially loaded piles depending on reliability calculations as recommended by (Fattah, 2010).

As (Tomlinson, 1993) stated the ultimate bearing capacity (Q) of a pile in dry sand when (Cu=0) can be expressed by the following equations:

Q = $Q_b + Q_S$ (3.33)

Where: $Q_S = K_S \,.\, \tan(\delta)\,.\, A_S\ \sigma_v$ (3.34)

$Q_b = A_b .\, \sigma_{vb}\ .\, N_q$ (3.35)

$\varphi = 0\ then\ Nq = 1\ N\gamma = 0$ (3.36)

Where: $Nq\ and\ N\gamma$ are the bearing capacity factors.

$Q_b\ = base\ resistance\ and\ \ Q_S =\ shaft\ resistance.$

Where: K_S = a factor depends on the relative density of soil and the pile type.

δ = coefficient of friction between the soil and the pile. (In concrete piles

Equal to ¾ of φ).

σ_{vb} = at the pile base is the effective overburden pressure.
σ_v = the overburden pressure along the pile.
A_b= area of the base of the pile.

$A_{S} =$ The surface area of the pile.

3.7 Reliability Analysis of a Footing on a Trench of Granular Soil

The footings supporting the experiments were partially replaced with a granular dirt trench and rested on soft clay. The impact of uncertainties in the soil parameters on the bearing capacity values will be examined using procedures of reliability analysis.

Case study (1):

In the practical practicing there are many techniques for improvement the load bearing capacity and the settlement of foundations resting on weak compressible layers of soil. One of these techniques is the use of the granular trench beneath the foundation. As expressed in figure (3.1) (Madhav and Vitkar, 1978).

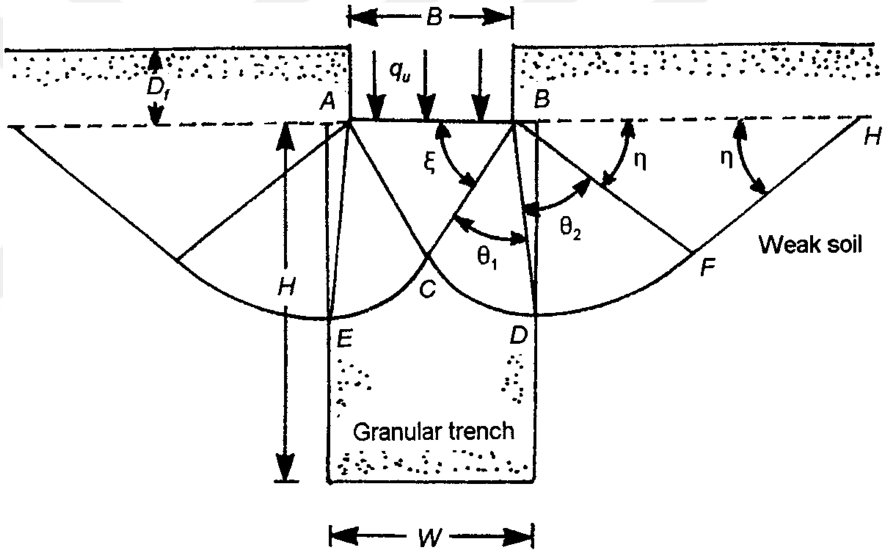


Figure 3.3: Foundation Resting On Granular Trench On Weak Soil

Source:(Madhav and Vitkar, 1978)

B: Is the width of the foundation.

W: Is the width of the trench.

H: Is the depth of the trench.

W can be smaller or larger than B. the ultimate bearing capacity of the foundation can be expressed by the following equation:

$$q_u = C_2 N_{C(T)} + D_f \gamma_2 N_{q(T)} + \left(\frac{\gamma_2 B}{2}\right) N_{\gamma(T)} \quad (3.37)$$

Where: C_2 is the undrained shear strength of the soft soil.

$N_{C(T)}, N_{q(T)}$ *and* $N_{\gamma(T)}$: are the bearing capacity factors in the presence of the trench.

The bearing capacity factors can be obtained from the figures (3.4), (3.5) and (3.6)

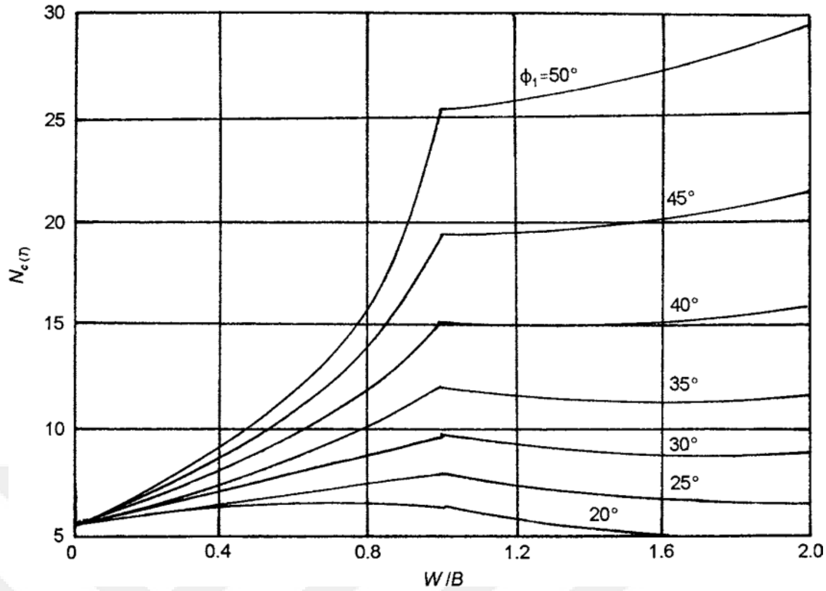


Figure 3.4: The Bearing Capacity Factors NC(T))

Source: Madhav and Vitkar, (1978).

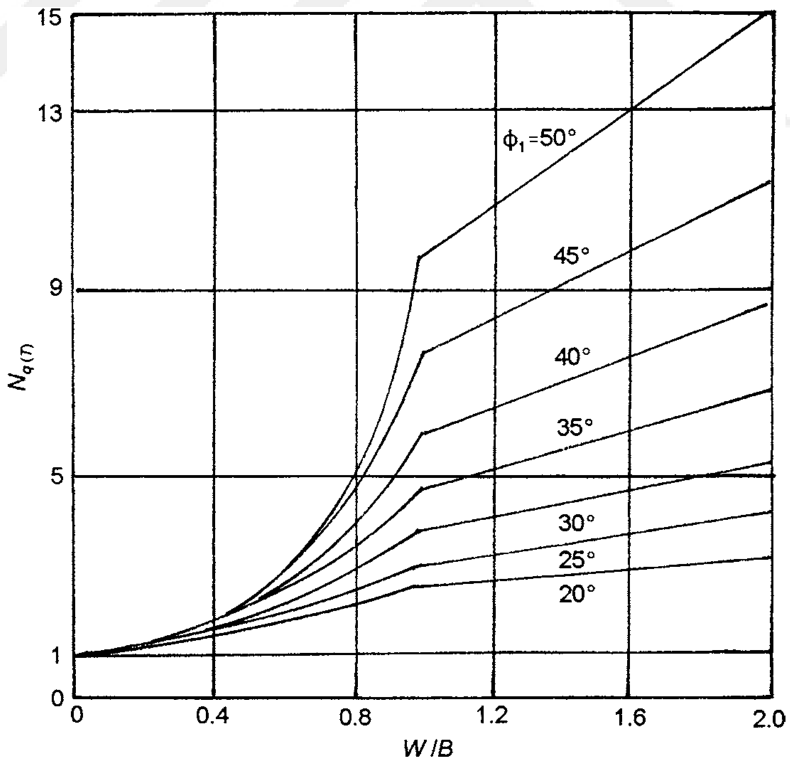


Figure 3.5: The Bearing Capacity Factors $N_{q(T)}$

Source: Madhav and Vitkar, (1978).

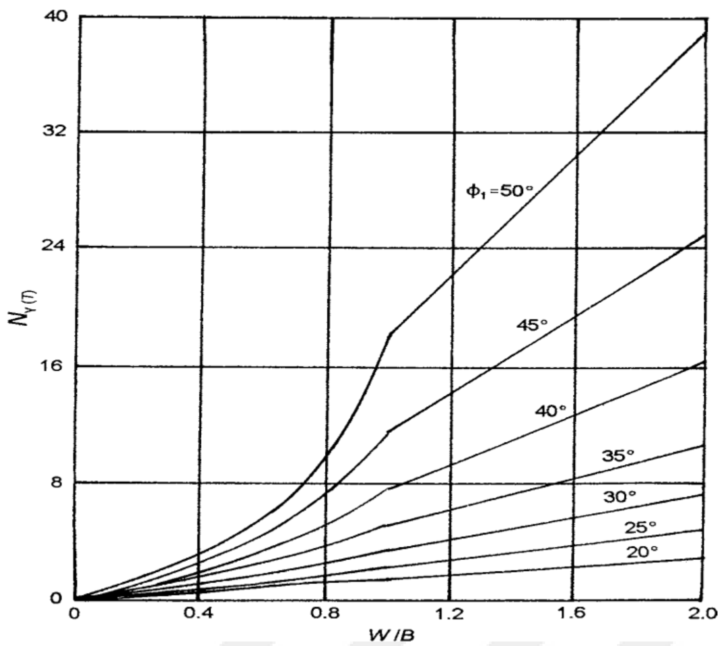


Figure 3.6: The Bearing Capacity Factors$N_{\gamma(T)}$

Source: Madhav and Vitkar, (1978).

Table 3.2: The Parameters of the Trench and Bed of Soil Used in Reliability Estimation

Parameter	Values from work	Standard Deviation	X(+)	X(-)
Unit weight, γ (kN/m^3)	15.2	5	20.2	10.2
Angle offriction of crushed stone φ(^{o})	45	5	50	40
Cohesion of the soil bed, c_u(kN/m^2)	17	5.78	22.78	11.22

Source: laboratory works

W = B = 2 m

Nq (T) = 8

Nc (T) = 19

Nγ (T) = 12

Df = 1 m

Q (φ, γ, C) (kN/m^2)	**Q^2 (φ, γ, C)**
Q(+ + +) = 523.91	274481.6881
Q(- - -)=143.34	20546.3556
Q(+ - -)=265.69	70591.1761
Q(- + -)=282.06	79557.8436
Q(- - +)=145.14	21065.6196
Q(+ + -)=520.01	270410.4001
Q(- + +)=283.86	80576.4996
Q(+ - +)=269.59	72678.7681

The bearing capacity is a function of three independent variables therefore the bearing capacity will be calculated as 2^3 = eight times.

According to (harr, 2002) the correlation coefficient ρ (φ,γ, C)= **- 0.5**

By the using of the point estimation method to find the weights ρ (i , j , k):

P (+ + +) = P (- - -) = $\frac{1}{2^3}$ $[1 + P(1,2) + P(2,3) + P\ (3,1)] = 0.3125$

P (+ + -) = P (- - +) = $\frac{1}{2^3}$ $[1 + P(1,2) - P(2,3) - P\ (3,1)]$ = 0.0625

P (+ + -) = P (- - +) = $\frac{1}{2^3}$ $[1 + P(1,2) - P(2,3) - P\ (3,1)] = 0.0625$

P (+ - -) = P (- + +) = $\frac{1}{2^3}$ $[1 - \ P(1,2) + P(2,3) - P\ (3,1)]$ = 0.0625

Equation (3.37):

$q_u = C_2 N_{C(T)} + D_f \gamma_2 N_{q(T)} + \left(\frac{\gamma_2\, B}{2}\right) N_{\gamma(T)}$ =318.91 **kN /** $\boldsymbol{m^2}$

While in the experimental work and for the same case the bearing capacity = 510 kN/m^2.

$E\ [Q]^2$ = $\bar{Q} \sum Q^2$(i, j) P (i, j) = 129376.28

Equation (3.9): THE VARIANCE IS

$\boldsymbol{v}\ \ [Q]$ = $E\ [Q]^2 - (\ E\ [Q]\ \ )^2$ = 2767.100

Equation (3.10): The Standard Deviation Is

$\sigma\ [Q]$ = $\sqrt{v\ \ [Q]}$ = 166.346

Equation (3.11): The Coefficient Of Variation Is

CoV $[Q]$ = $\frac{\sigma\ [Q]}{E[Q]}$ *100% = $\frac{451.84}{1345.42}$ * 100 = 52.160 %

- For Factor Of Safety= 4 And From Equation (3.20) The Demand Is :

$\bar{D}$ = $\frac{\bar{E}}{FS}\ =\ 79.728$

The Standard Deviation Of The Demand:

$\sigma\ [D]$ = E $[D]$ * CoV $[D]$ = 79.728 * 0.5216 = 41.586

The Safety Margin Is:

S= $\bar{C}$ - $\bar{D}$ = 318.91 – 79.728 = 239.184 kN / m^2

According to (Harr, (2002)) assumptions the correlation coefficient $\rho\,(Q, D) = +0.75$

Equation (3.31):

$$\beta = (\bar{C} - \bar{D}) / \sqrt{\sigma^2[C] + \sigma^2[D] - 2\text{P}\,\sigma\,[C]\,.\sigma[D]}$$

$$\beta = \frac{239.184}{\sqrt{(166.346)^2 + (41.586)^2 - 2\,(0.75)\,(166.346)(41.586)}} = 1.734196$$

From probability tables: $\psi\,[\beta]$ **:** Is the normal probability distribution as mentioned in the tables of standard normal probability.

$\psi\,[\beta]$ = 0.4585

Equation (3.32):

P (f) = 0.5 - $\psi\,[\beta]$ = 0.5 – 0.4585 = 0.04144

Then The Probability Of Failure = 4.14 %.

Case study (2):

By the application of the equation (2.18) on the same trench for the calculating of bearing capacity (considered two layer soil systems).

$$q_u = 1 + 0.2\frac{B}{L}\,5.14C_2 + \frac{\gamma_1 H^2}{B}\;1 + \frac{2Df}{H}\;1 + \frac{B}{L}\,k_s tan\,\varphi_{1+}\;\gamma_1\,Df \le \gamma_1 Df$$

$$N_{q1}\,S_{q1} + 0.5\gamma_{1\text{B}N\gamma_1 \text{S}\gamma_1} \tag{2.18}$$

By the using of table (3.2):

Because the bearing capacity depends on five independent variables, it may be found using the formula 2^(5) = 32 times.

The ratio of $q_2\,/\,q_1$ by the use of equation (2.19):

$$\frac{q_2}{q_1} = \frac{(C_2\,NC_2)}{0.5\,\gamma_1\;BN\gamma_1} = \frac{5.14C_2}{0.5\gamma_1\;BN\gamma_1} \tag{2.19}$$

The value of ks can be obtained from figure (2.7):

S_{q1} = 1+ B/L tan $\varphi_{\,1}$ = 1+ tan 40 = 1.84

$S_{\gamma 1}$ = 1 – 0.4 B/L = 0.6

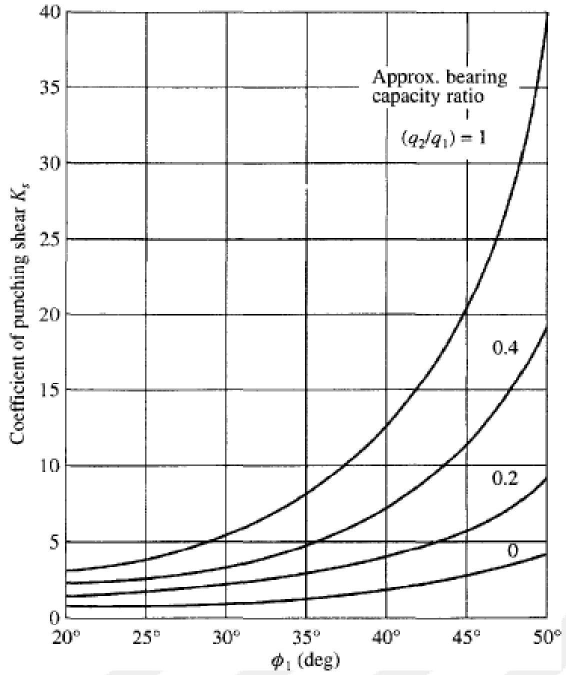


Figure 3.7: Coefficient of Punching Shear Resistance Under Vertical Loading

Source: Meyerhof and Hanna, 1978

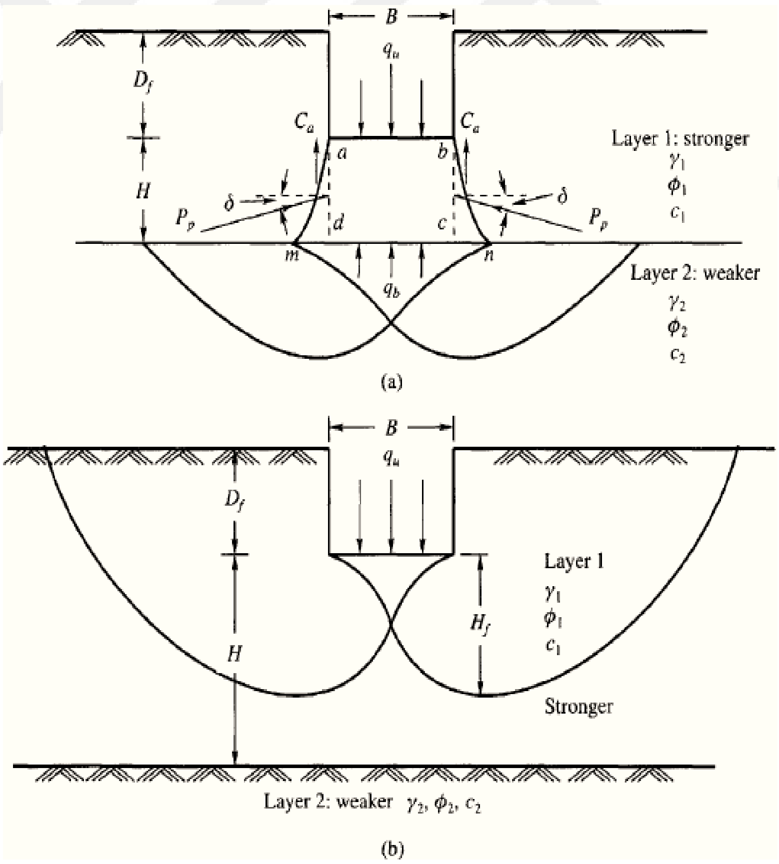


Figure 3.8: Failure of Soil Beneath Strip Footing Undergo Vertical Load On Strong Layer Resting On Weak Layer

Source: Meyerhof and Hanna, 1978

3.8 Monte Carlo Simulation Methodology

The simulation of Monte Carlo is a statistical technique to be used for taking random samples and statistical modeling and to estimate the mathematical functions and simulate the complicated systems. The technique includes the generation of random samples for the variable soil properties such as cohesion and the unit weight of gravel. According to their probability distributions. Then these samples were used for calculating the bearing capacity for each scenario. This method is effective especially for understanding how the uncertainties in input parameters affect the output of model. Leading to make it a powerful technique in the assessment of risks and take decisions.

3.9 The Analysis of the Safety Factor

The analysis of the safety factor in geotechnical engineering is the application of safety factor upon the calculated parameters of soil strength or upon the calculated bearing capacity itself to ensure safety margin in design. The safety factor expresses the faces of uncertainty in soil properties. And express the inaccuracy in model design. And the probably deviations in the construction ways. This methodology modifies the design parameters to support safety and reliability, this will reflect a conservative methodology for potential and inherited uncertainties and risks in geotechnical design. The design bearing capacity is calculated from the following equation:

$$q_{design} = \frac{q_u}{N} \tag{3.38}$$

Where: N is the safety factor and q is the bearing capacity.

3.10 Cumulative Distribution Function Analysis (CDF)

The cumulative distribution function analysis involves using the (CDF) for a probability distribution especially for the normal standard distribution. For calculating the probability of the occurring of a certain event. In this case it used to determine the probability of the decreasing of the bearing capacity under a certain threshold. The study derived the probability of failure through standardizing the input parameters and the evaluating of the standard normal (CDF).

The probability of failure can be calculated through the following equations:

$$Z = q_{design} - mean_{qu} / SD_{qu} \tag{3.39}$$

$$P\ (\text{Failure}) = \varphi\ (q_{design} - mean_{qu} / SD_{qu}) \tag{3.40}$$

Where: Z: is the standardize value

SD: is the standard deviation

Mean: is the arithmetic mean of the estimation

At last, the failure to be calculated according to all the mentioned methodologies above. In all these methodologies or ways, the probability of failure is a crucial measurement determining the potential risks in the design. Considering the variation and uncertainty in soil properties. The engineers and the decision makers help in understanding the probability of lack in satisfying of the design to the required performance.

Case study (1):

1. **Defining the variant input parameters.**

Table 3.3: The Parameters of the Trench and Bed of Soil Used in Reliability Estimation

Parameter	Values from work	Standard Deviation	X(+)	X(-)
Unit weight, γ (kN/m^3)	15.2	5	20.2	10.2
Angle offriction of crushed stone $\varphi(^{o})$	45	5	50	40
Cohesion of the soil bed, c_u(kN/m^2)	17	5.78	22.78	11.22

Source: laboratory works.

2. **Defining constant input parameters**

Defining the mean values of the soil properties and the constants to be used in the equation of bearing capacity:

B = W =2 m

Nq (T) = 8

Nc (T) = 8

$N\gamma\ (T) = 8$

D_f = 1m

3. The calculation of ultimate bearing capacity

From equation (3.1):

The ultimate bearing capacity:

$$q_u = C_2 N_{C(T)} + D_f \gamma_2 N_{q(T)} + \left(\frac{\gamma_2 B}{2}\right) N_{\gamma(T)} = 627.00 \quad kN/m^2 \tag{3.37}$$

4. The applying of the safety factor

In order to calculate the design bearing capacity, the safety factor to be added to the ultimate bearing capacity, the addition of the safety factor depends on the standards of industry by the assumption of 3. By the use of equation (3.38):

$$q_{design} = \frac{q_u}{N} = \frac{627.00}{3} = 209.00 \; kN/m^2 \tag{3.38}$$

5. Generating of the random samples

For each soil property in this very case (the undrained shear strength and the unit weight of gravel) generating a large number of random samples. These samples represent the probable values that the soil properties may have, considering the taking of the mean and the standard deviation for these samples, 10000 random samples were generated in this study for $C_2 \, AND \, \gamma_2$. Assuming the normal distribution the samples were generated.

$$C_{2,Samples} = Normal\,(C_{2,Mean}, C_{2,SD})$$

$$\gamma_{2_{Samples}} = Normal\,(\gamma_{2_{Mean}}, \gamma_{2_{SD}})$$

Then calculating the ultimate bearing capacity for each of the samples of 10,000 samples, the figure (3.3) express the histograms for the both features, while the figure (3.4) express the ultimate bearing capacity corresponding with the estimations.

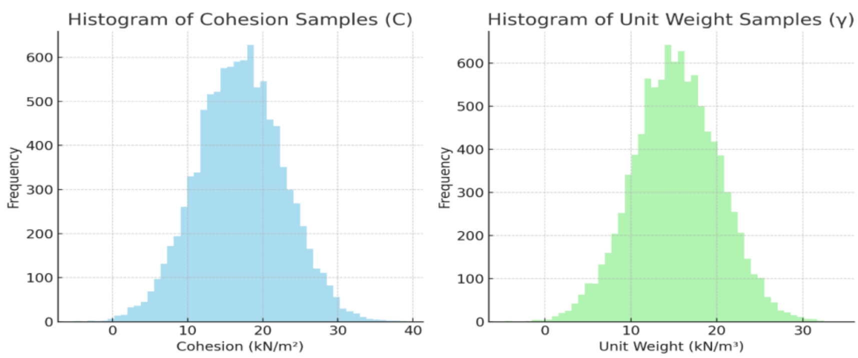


Figure 3.9: Histogram Sampling of the Estimation for Cohesion and Unit Weight

Source: Analysis sheets

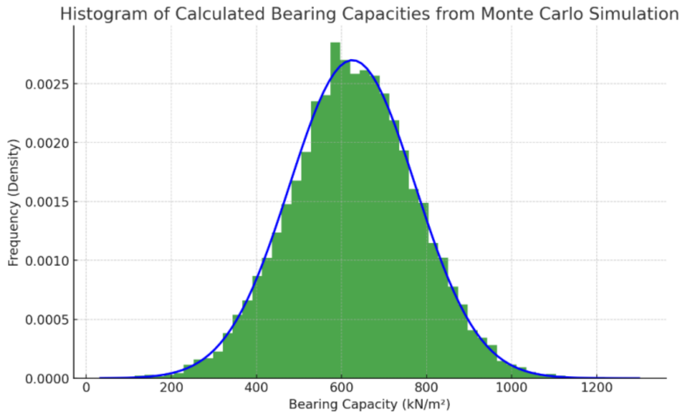


Figure 3.10: Histogram Sampling of the Estimation for Ultimate Bearing Capacity

Source: Analysis sheets

Using the mentioned values and figures of Monte Carlo criteria the following can be concluded:

Mean bearing capacity ($Mean_{\ qu}$) = 625.60 kN/m^2

SD of bearing capacity (SD_{qu}) = 147.74 kN/m^2

1. Estimating the probability of failure

The probability of failure is the probability of the bearing capacity to be less than the design bearing capacity. To be calculated by the use of the equations:

$$Z = q_{\ design} - \ \ mean_{qu} \ / \ \ SD_{qu} = \frac{209.00 - 625.60}{147.74} = -2.82 \tag{3.39}$$

From figure (3.9) and by using the (CDF) method to calculate the probability of failure:

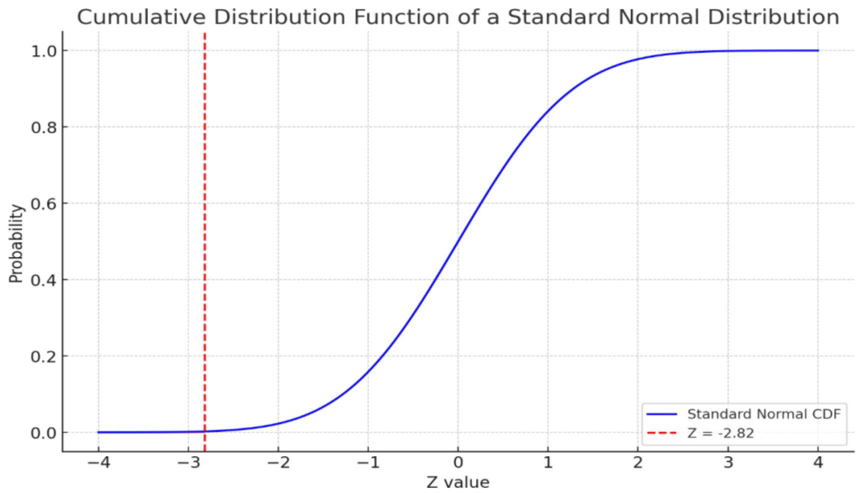


Figure 3.11: (CDF) of a Standard Normal Distribution for Probability Calculation

Source: Analysis sheets

The probability of failure calculated to be 1.24 % and it is in the acceptable limits. These steps determine the procedure of calculating the probability of failure of foundation considering the faces of uncertainties in soil properties and the application of the safety factor. This method guaranties the use of a conservative and realistic for foundation design, and achieve the equilibrium between safety and practical practice.

Case study (2):

1. Defining variant input parameters
2. Defining constant input parameters

We define the mean values for the soil properties and the constants used in the bearing capacity equation.

B = L = 2 m

Nq (T) = 8

Nc (T) = 8

$N\gamma\ (T) = 8$

D_f = 1 m

$H = 1\ m$

Sq1 = 1+ B / L

tan φ1 = 1 + tan 40 = 1.84

$S_{\gamma 1}$ = 1 – 0.4 B / L = 0.6

FROM EQUATION (2.19):

$$\frac{q_2}{q_1} = \frac{(C_2\ NC_2)}{0.5\ \gamma_1\ \ BN\gamma_1} = \frac{5.14C_2}{0.5\gamma_1\ \ BN\gamma_1} = 70.47 \quad (3.40)$$

The value of Ks can be taken from figure (2.7):

Ks = 9.888

1. **The calculation of ultimate bearing capacity**

$$q_u = 1+0.2\frac{B}{L}\ 5.14C_2\ + \frac{\gamma_1 H^2}{B}\ \ 1+\frac{2Df}{H}\ 1+\frac{B}{L} k_s tan\, \varphi_{1+}\ \ \gamma_1\ Df \leq\ \gamma_1 Df$$

$$N_{q1}\, S_{q1}\ + 0.5\gamma_{1BN\gamma_1 S\gamma_1} = 506.17\ \text{kN} / m^2 \quad \leq \quad 302.56 \quad \text{kN} / m^2 \quad (3.41)$$

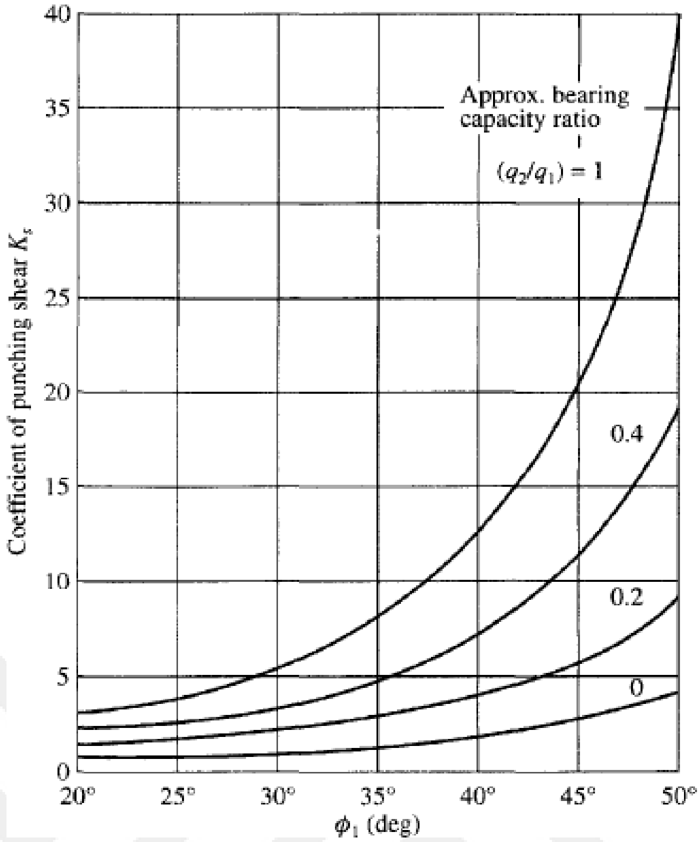


Figure 3.12: Coefficient Of Punching Shear Resistance Under Vertical Loading

Source: Meyerhof and Hanna, (1978)

2. The applying of the safety factor

The choose of the safety factor depends on industry standards to be 3. The safety factor to be applied to the ultimate bearing capacity to calculate the design bearing capacity.

From equation (3.38):

$$q_{design} = \frac{q_u}{N} = \frac{302.56}{3} = 100.854 \ kN/m^2 \tag{3.42}$$

3. Generating of the random samples

In this case for each soil properties (undrained soil strength and the unit weight of gravel) the generation of a huge number of random samples implemented. These samples represent the probable values that these properties may have in realistic. Considering obtaining the mean and standard deviation for the probable values. $(\varphi 1, C2 \ and \ \gamma 2)$ In this study 10,000 random samples generated considering the normal distribution.

$$C_{2,Samples} = Normal\ (C_{2,Mean}, C_{2,SD})$$

$$\gamma_{1_{Samples}} = Normal\ (\gamma_{1_{Mean}}, \gamma_{1_{SD}})$$

$$\varphi_{1_{Samples}} = Normal\ (\varphi_{1_{Mean}}, \varphi_{1_{SD}})$$

Then the value of the ultimate bearing capacity to be calculated for each sample of the 10,000 samples. As expressed in the figures below:

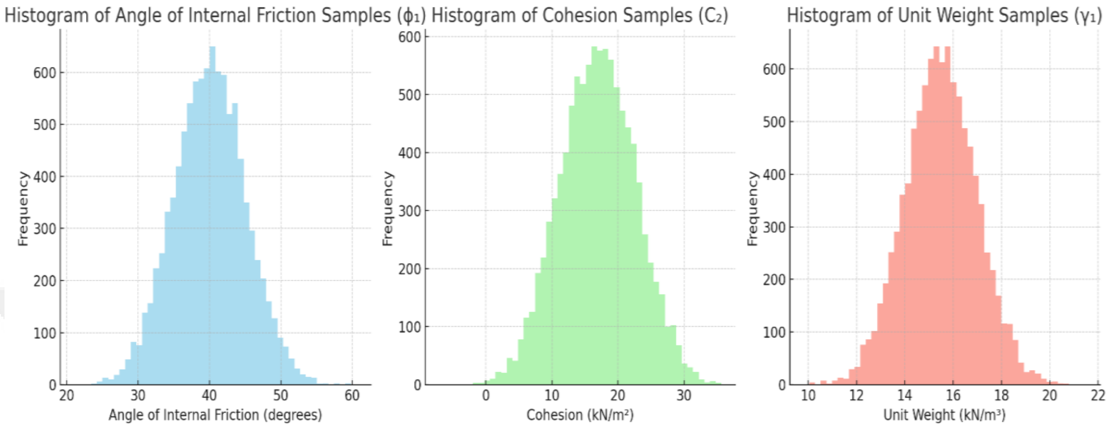


Figure 3.13: Histogram Sampling Frequency of the Estimations for Angle of Internal Friction, Cohesion and Unit Weight

Source: analysis sheets

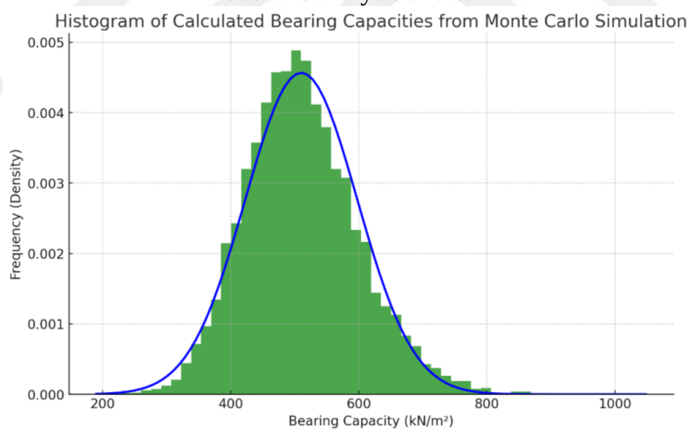


Figure 3.14: Histogram Sampling Frequency Of The Estimations For Ultimate Bearing Capacity

Source: Analysis sheets

By the use of the mentioned values and figures of Mont Carlo criteria, the following can be concluded:

Mean bearing capacity ($Mean_{qu}$) = 510.22 $\frac{kN}{m^2}$

SD of bearing capacity (SD_{qu}) = 87.83 $\frac{kN}{m^2}$

4. **Estimating the probability of failure**

The probability of failure is the probability that the bearing capacity is less than the design bearing capacity.

$$Z = q_{\,design} - \ mean_{qu} / \ SD_{qu} = \frac{100.854-510.22}{87.83} = -4.661 \qquad (3.43)$$

From figure (3.15) by the use of (CDF) calculating the probability of failure:

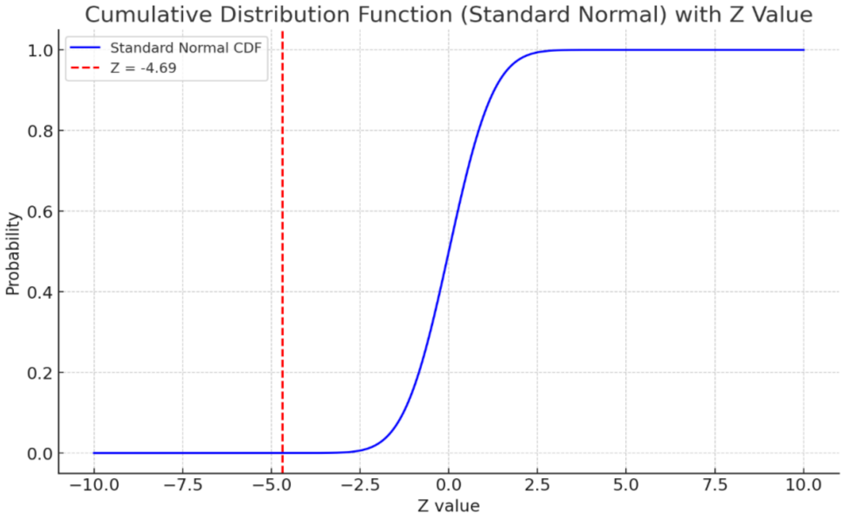


Figure 3.15: (CDF) Of A Standard Normal Distribution For Probability Calculation

Source: Analysis sheets

The probability of failure calculated equal approximately 0.00014 % which is within the acceptable limits.

3.11 The Geotechnical Problems Of Reliability Analysis - The Ex-Researches

The development for reliability analysis occurred during several years in the geotechnical engineering beginning from the ways of probabilistic in here some of these studies.

(Kulhawy, 1996) the (R.B.D) the reliability based- design. (Kulhawy, 1996) submitted a historical view for the traditional factor of safety. Focused on the basically importance for the limit state design to the reliability based-design. At last the view of reliability had done.

He also discussed the prober application for this new design approach and provided examples for the ultimate limit state design for drilled shafts, exposed to undrained uplift loadings.

The suspicious traditional approaches through the application of the reliability based-design (R.B.D) figured to be correct.

(Phoon and Kulhawy, 2004) made a public view about the development of structural and geotechnical design for years and he deal with how to deal with the faces of uncertainty. This will provide a valuable historical view for the current situation and the important unsolved cases and to highlight on the availability of the statistics to develop the simple equations of the (R.B.D) reliability based-design to solve unsolved issues through supporting experimental ways. The design based on reliability if it is simple or not supplies more organized approach to treat the uncertainty cases. Any wat it is not ideal solution, still the engineering governing a solution that cannot be neglected in many fields of the geotechnical engineering. The reliability analysis just removes the need for prediction about the influence of uncertainty on the performance of a system and it can be compared with the theory of the elastic-plastic for removing the estimation or prediction about how loads caused in pressures and deformations.

(Dasaka, et al., 2005) investigations made in probabilistic analysis for the bearing capacity of strip footing resting on cohesive less soil**.** The safety factors calculated opposite to the aim reliability index are 3, 7.3, and 5.5 respectively for the advanced and simple analysis. The safety factors are higher than those to be taken in the routine footings designs. The higher values of safety factors that connected with allowable loads gained from the reliability approach clearly express the importance of uncertainty studies

in geotechnical engineering and very require the need for involving the reliability view in the design of geotechnical engineering.

Using reliability calculations, **Fattah, (2010)** attempted to estimate the bearing capacity for axially-loaded piles. The chance of failure, reliability, central factor of safety and the reliability index. The process is an expansion of the point estimate approach, which computes the predicted standard deviation values for the capacity and demand functions. The process will be applied in two stages: in the first, a pile will be located and driven in sand, and in the second, a pile will be driven in clay. The proposal was straightforward and should be expanded to cover additional geotechnical engineering applications.

(Honjo et al., 2011) proposed a reliability-based design framework for geotechnical engineering practice. The geotechnical engineers can use this system to distinguish between the geotechnical design portion and the uncertain analysis portion of the geotechnical reliability based design. The geotechnical engineers would be more relaxed in dealing with the reliability based-design otherwise if they use the traditional reliability based-design. According to the results some discussions implemented for facing major cases that the geotechnical reliability based-design (R.B.D) is suffering from. The conclusion was that the variability in soil characteristics the (spatial variety) is one of the uncertainty sources. In many of the design problems the errors of design calculations model, the errors of transformation correlated with estimation of the soil parameters (friction angle), from the measured quantities and the errors of the statistical estimation, it is important to gain knowledge about these sources of uncertainty to develop the reliability based-design (R.B.D) and to shift the reliability based-design to a higher levels of knowledge.

Questionnaire about the: Effect of Uncertainties of Soil Strength Parameters on the Reliability of Foundation Behavior.

Studying parameters that affecting the foundation behavior in the existence of uncertainties. The safety factor can be calculated precisely by the use of the analytic methods.

Questions of the Questionnaire:

1. What is the soil property that has the greatest influence in determining the bearing capacity of the foundations?

a. Angle of friction. b. Cohesion. c. Density.

2. The choice of the factor of safety depends largely on:

a. Previous experience. b. The nature of failure. c. The applied load.

3. Reliability index is related to the probability of failure of the foundation:

a. Largely. b. Moderately. c. The relation between them is weak.

4. One of the benefits of the reliability is that the structure can be designed according to serviceability conditions:

a. Agree well. b. Agree c. Disagree.

5. In reliability analysis, the overall risk involved in the project is clearly identified.

a. Agree well. b. Agree c. Disagree.

6. Reliability analysis provides a framework for establishing appropriate factors.

a. Agree well. b. Agree c. Disagree.

7. The reliability index is seen to be the reciprocal of the coefficient of variation of the safety margin. Is this definition?

a. Appropriate b. Beneficial. c. Has no importance.

8. Which do you prefer in foundation designs?

a. Traditional factors of safety. b. Reliability coefficients. c. Both of them.

9. In the context of decision making in geotechnical engineering practice, the scenario includes being situated at a specific site. Hence, do you think that site investigation is necessary?

a. Agree well. b. Agree c. Disagree.

10. Although numerical models are highly sophisticated, the input ground profile is typically simple and deterministic. Do you think that reliable analysis is required?

a. Agree well. b. Agree c. Disagree.

11. Reliability-based design (RBD) is a rational methodology that allows practitioners to design geotechnical elements to achieve a target probability of failure.

a. Agree well. b. Agree c. Disagree.

12. The characterization of geologic uncertainties is more challenging than the aforementioned geotechnical uncertainties,

a. Agree well. b. Agree c. Disagree.

13. Can the geotechnical engineers make use of probability analysis concepts in judgments?

a. agree b. do not agree c. agree well

14. Structural failure consists of shear failure and moment failure, soil failure consists of bearing failure and settlement failure. In addition to above the foundation should satisfy other requirements.

a. agree b. do not agree c. agree well

15. The foundation should be probably located regarding frost action and volume change adjacent structure.

a. agree b. do not agree c. agree well

16. Certainty can be a description for most engineering problems.

a. agree b. do not agree c. agree well d. other

17. The sources of uncertainty are unavoidable.

a. agree b. do not agree c. agree well d. other

18. Does the accuracy of theoretical and empirical methods for calculating bearing capacity of soil considered as one of uncertainty sources.

a. agree b. do not agree c. agree well d. other

19. Variability and randomness cause a difficulty in selecting the suitable design parameter.

a. agree b. do not agree c. agree well d. other

20. The stone column technique of ground treatment has proven successful in increasing the time rate of settlement.

a. agree b. do not agree c. agree well d. other

21. Cohesion, angle of internal friction and soil unit weight are independent and uncorrelated variables.

a. agree b. do not agree c. agree well d. other

1. **Timestamps:**

- The earliest response was recorded on January 18, 2023 at 19:37:53.
- The latest response was recorded on March, 14, 2023 at 01:07:11.

2. **Academic Achievement:**

Each entry in the dataset represents a unique respondent, with each name occurring exactly once, indicating that there are 70 unique respondents.

- Bachelor`s Degree: 31 respondents
- Master`s Degree: 20 respondents
- Doctoral Degree: 11 respondents
- Higher Diploma: 4 respondents
- Technical Diploma: 3 respondents
- Student: 1 respondents

3. **Specialization:**

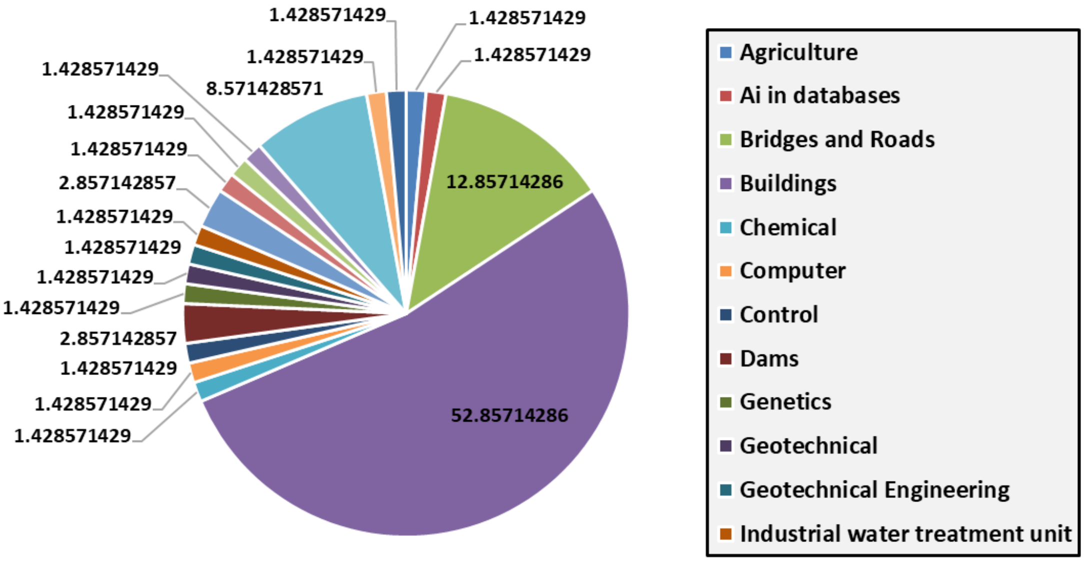
- Civil Engineering: 46 respondents
- Computer Engineering: 8 respondents
- Mechanic Engineering: 4 respondents
- Electronic Engineering: 3 respondents
- Electrical Engineering: 2 respondents
- Chemical Engineering: 2 respondents
- Other specializations (Management, physics, Agriculture, Translation, Biomedical Engineering): 1 respondent each.

4. **Years of Experience**

- More than 20 years: 22 respondents
- 1 to 5 years: 17 respondents
- 10 to 20 years: 16 respondents
- 5 to 10 years: 15 respondents

The analysis provides insights into the demographics and backgrounds of the survey participants, including when they responded, their education levels, fields of specialization, and their professional experience.

In regards to engineering projects which the participants have previously been included with, figure below shows the frequency distribution. It can be clearly concluded that mostly are involved with buildings.



1. Mode Analysis:

The figure presents a comprehensive visualization of the mode responses for each question titled "Effect of Uncertainties of Soil strength Parameters." Displayed as a series of horizontal bars on an A4 – sized layout, each bar corresponds to a specific survey question and is labeled with the most frequently chosen response (the mode) by the survey participants. The bars are color-coded for clarity and arranged in order of the questionnaire.

This graphical representation effectively conveys the prevailing opinions or choices among the respondents for each question, providing an immediate visual summary of the dominant trends and preferences in the data. The figure serves as valuable tool for quickly grasping the key findings of the survey, especially in understanding the collective viewpoints of professionals in the field regarding uncertainties in soil strength parameters and their impact on engineering practices.

2. Conclusion of the Questionaaire:

The most respondents are Civil Engineers (46 respondents of 70 respondents) , the percentage of them of about (72.857143 %) **(** 52.85714286 buildings Engineers + 12.85714286 Bridges and roads Engineers + 1.428571429 Bridges and roads Engineers + 2.857142857 Dams. + 1.428571429 + 1.428571429 Geotechnical Engineering.857142857 Dams

3. **The frequency percentage % of respondents answers versus the actual answers regarding the questions of the Questionaaire:**

<u>The questions:</u>

1. The actual answer is cohesion / respondents answer is density.
2. The actual answer is the previous-experince / respondents answer is the applied load.
3. The actual answer is largely / respondents answer is largely.
4. The actual answer is agree well / respondents answer is agree.
5. The actual answer is agree well / respondents answer is agree.
6. The actual answer is agree well / respondents answer is agree.
7. The actual answer is beneficial / respondents answer is beneficial.
8. The actual answer is both of them / respondents answer is both of them.
9. The actual answer is agree well / respondents answer is agree well.
10. The actual answer is agree well / respondents answer is agree.
11. The actual answer is agree well / respondents answer is agree.
12. The actual answer is agree well / respondents answer is agree.
13. The actual answer is agree well / respondents answer is agree.
14. The actual answer is agree well / respondents answer is agree.
15. The actual answer is agree well / respondents answer is agree.
16. The actual answer is agree well / respondents answer is agree.
17. The actual answer is agree well / respondents answer is agree.
18. The actual answer is agree well / respondents answer is agree.
19. The actual answer is agree well / respondents answer is agree.
20. The actual answer is agree well / respondents answer is agree.
21. The actual answer is agree well / respondents answer is disagree.

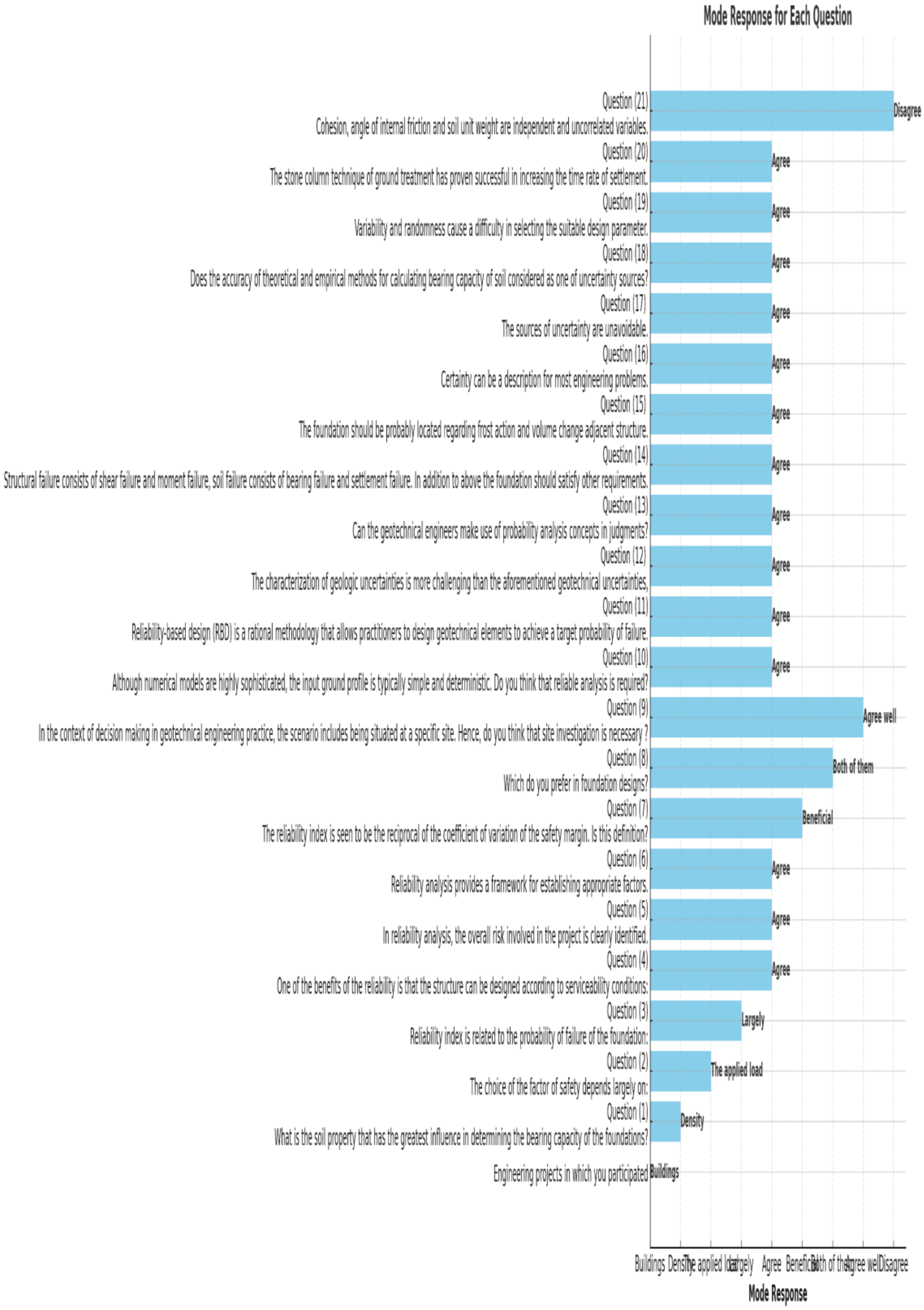


Figure 3.16: Answers To Each Question In The Model

1. **Frequency Distribution:**

The Excel file in the appendices shows the Figures of Frequency Distribution for each question.

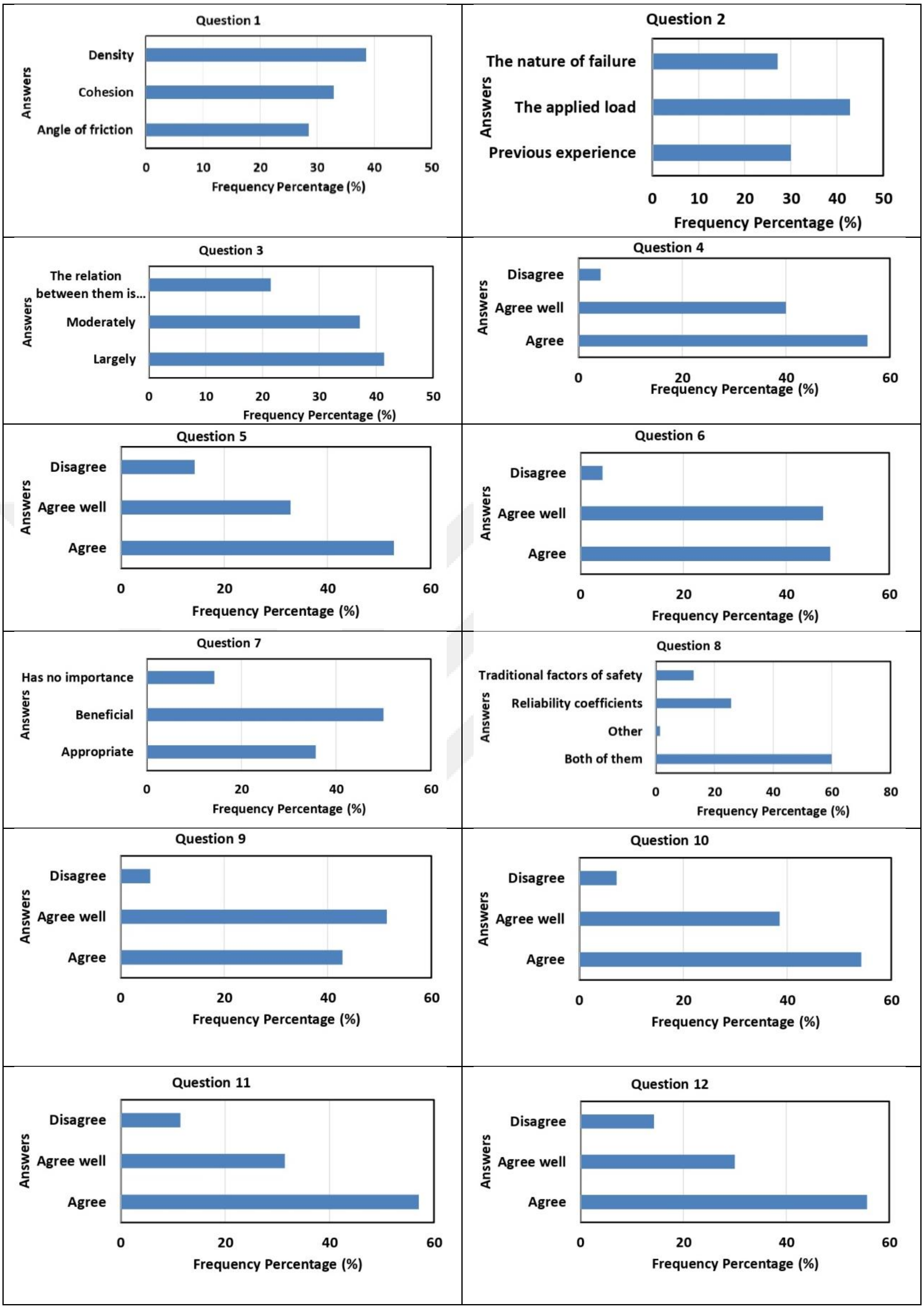


Figure 3.17: Graphical Representation Of The Answers To Each Question

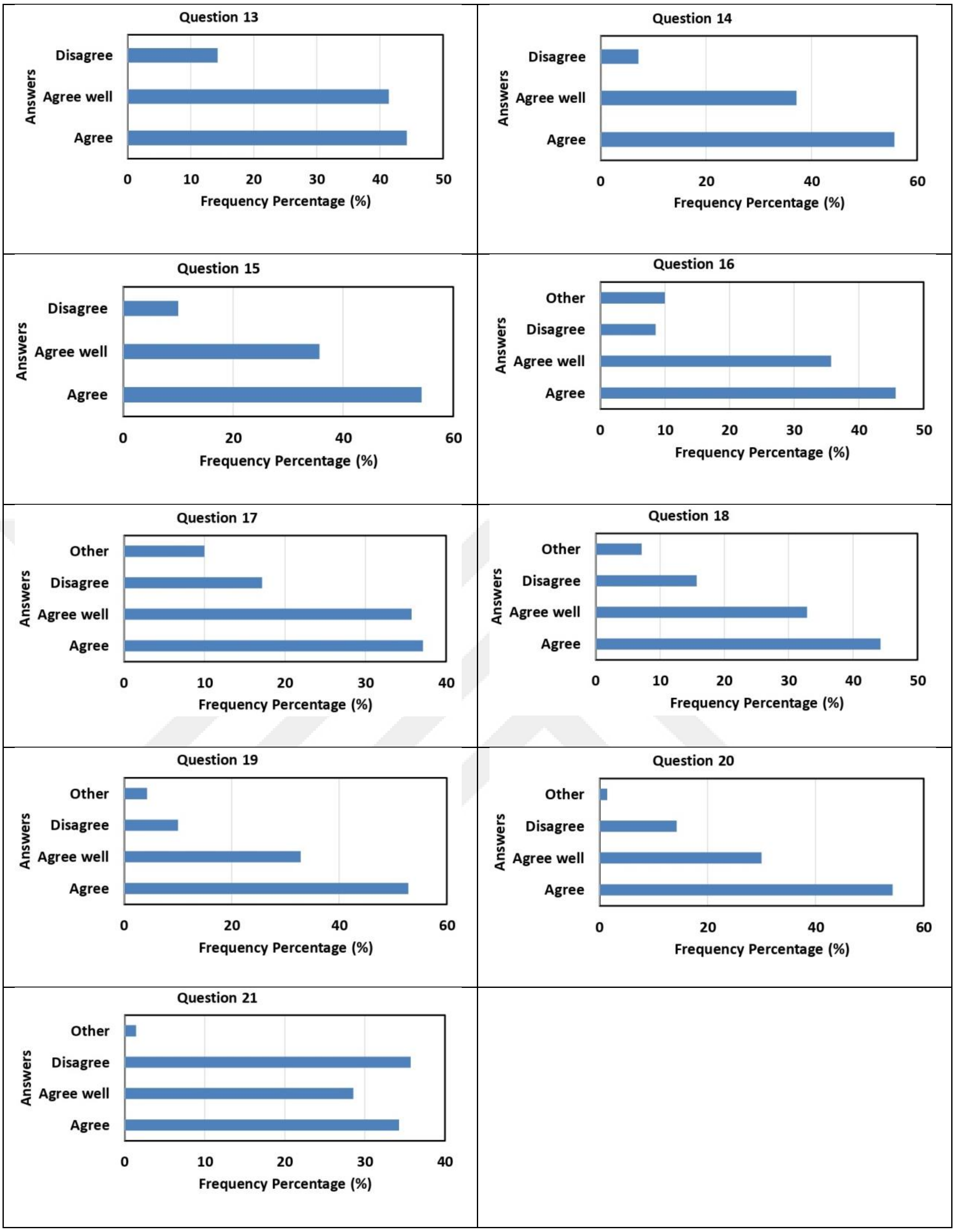


Figure 3.17: (Cont.) Graphical Representation of the Answers to Each Question

4. THE LABORATORY WORKS (EXPERIMENTAL WORKS IN SUMMARY)

4.1 Introduction

Expressing the full details of the experimental works for model tests to investigate the reliability of the bearing capacity for number of cases including one stone column, group of stone columns and two models of layered soil. Twenty one model tests implemented were divided into two series, models of soft clay with different values of undrained shear strength (Cu) ranging from 8-18 kPa. The first series consist of eight models of soil replacement to variable depths and widths.

The second series consists of twelve models of stone columns, single stone column, group of two stone columns, group of three stone columns, group of four stone columns, group of six stone columns. Tests were implemented upon these groups in two cases: end-bearing stone column and floating stone column. In addition to one model of untreated soil.

4.2 The used materials

4.2.1 The soft soil

The soil that was being used came from a city east of Baghdad, which is the official capital of Iraq.. According to the (USCS) the unified soil classification system; the soil seemed to be classified as (cl) consists of 48% clay, 35% silt and 17% sand. a sample of reformed clay that was extracted from the soil's bottom and subjected to a typical consolidation test.

4.2.2 The crushed stone

The material of the crushed stone used in the stone columns had been taken from the factory of crushing stone. Producing by the crushing of the massive stones with angular shape, The shape of the particles is uniform and the particles are come with a poor gradient, the direct shear test implemented on the samples, the samples with relative density of 73% according to the standards (ASTM-3080-2003).

4.3 Model Test Preparations

4.3.1 Soil bed preparations

Prior to the soil bed being prepared, control experiments were conducted to ascertain how the undrained shear strength varied over time at various water contents and liquidity limitations., where the undrained shear stress was measured using the vane shear stress device. In order to accomplish this task, three separate samples were prepared and put inside five coats or layers in the California Bearing Test Device (CBR test device). Each coat or layer was then gently tapped with a hammer to release any trapped air. After that, the samples were wrapped in polyethylene sheets and left for a week. The water content to be measured and the undrained shear stress.

First, the dirt was dried, and then it was crushed into little pieces using a crushing machine to further break up the material. To achieve the necessary consistency, a small amount of water was combined with the native dirt. A large mixture, holding 120 liters, was employed, with each 25 kg of dry dirt being mixed separately and combined to make the entire amount of soil. To ensure a consistent wet content, the well mixed wet soil was sealed within polyethylene bags for a full day. The soil to be utilized in the model testing is layered, with each layer having a thickness of between 50 and 75 mm, inside a steel-made container. Using a wooden tamper, each layer was leveled. Next, each layer was gently tapped with a metal hammer, which weighs 9.87 kg. Additionally, the metal hammer's size (150 x 150 mm) are necessary to release the trapped air. Each layer is added to the soil in the steel container until the thickness of the soil for the floating stone columns, end-bearing stone columns, and soil replacement models is 500 mm, 400 mm, and 350 mm, respectively. Once the last layer is completed, it must be leveled and scraped to create a flat surface. To prevent moisture loss, polyethylene sheets must then be placed on top of the final layer. A wooden pallet measuring 600 by 600 mm, the same size as the soil bed. the soil bed exposed to a sitting pressure of 5 kPa with the wooden pallet placed on top of it. for a whole day in order to regain some of its strength. The dirt bed was covered and was to be left for a duration of five days prior to the scheduled test time. Two steel containers were utilized to carry out the tests. The first container's internal measurements were (600*600*500) mm, and it was used to replace dirt. The second one measures (600*750*750) mm inside. The two of them were made from 4 mm thick steel sheets.

4.3.2 The steel foundations

The thickness of the steel foundations 10 mm. and the dimensions of the foundations are different according to the purpose of the use:

- The footing's diameter for the stone column is 64.6 mm.
- The rectangular foundation for the two stone columns measures (125*250) mm.
- The square footing for the three and four stone columns measures (250*250) mm.
- The square footer for each of the five stone columns measures (375*375) mm.
- The rectangular footing for the six stone columns measures (250*375) mm.
- For all the cases of the soil replacement the square footing is (100*100) mm.

It is noticeable that when the spacing between the columns increased the bearing pressure on the foundation decreased, (Bora and Dash, 2010) stated that the suitable spacing between the columns is to be equal to (2.5d), when the spacing reduced from (3.5d to 2.5d) the bearing pressure is high after this increasing any extra bearing pressure or excessive bearing pressure is seemed to be can neglected.

4.3.3 The assembly of loading

In the loading assembly two frames were used to apply the vertical statics loads on the model footings:

4.3.3.1 The loading assembly of soil replacement

Bearing frame designed and manufactured for applying the vertical static load on the model footing in the soil replacement models.

4.3.3.2 The loading assembly of the stone columns

The bearing steel frame manufactured by for supporting the hydraulic jack. The arrangements of the square sheets for horizontally moving for the piston along the beam. On the sides of the column there are holes helping in controlling vertically the space between the jack and the container surface.

The hydraulic jack's catalogue states that the maximum bearing load that can be used is approximately 10 tans. To regulate the hydraulic intensity, a manual regulating mechanism will be installed. A pressure gauge for measuring axial pressure is located in the upper portion of the jack. The plastic tube is utilized to transfer hydraulic fluid from the manual system to the piston. The air in the hydraulic system is removed using an Abreaction system.

The cell of load used and the weighting digital measurement device also used for measuring the net axial loads on footing.

4.4 The Stone Column Construction

After completing the soil bed preparations, the processes for the building of the stone columns were followed, and the ratio of (L/D) was selected to be equal to 6 for the floating stone column and 8 for the end-bearing stone column in accordance with the recommendations of (AL-Waily, 2008). The area replacement ratio (as) can be defined based on equation (2.1).

$$\boldsymbol{a_s = \frac{A_s}{A_s + A_c}} \tag{4.1}$$

4.5 The Stone Column – Testing Procedures

Following the curing time, the following protocols were put into place:

1. Placing the steel footing above the columns made of stone.
2. Positioning the bearing frame so that, the piston's center lines up with the footing's or the single stone column's center.
3. The loads were applied using a loading jack, which permitted an increase in weights that was maintained until the dial gauge stopped reading or a

penetration of 0.25 to 1.25 mm/min. Happened. As to the ASTM D-1143, 2000 standard.

4. The dial gauge's readings were taken prior to the subsequent increase in load.
5. The weights increased steadily until the collapse happened.
6. Using a vane shear portable instrument, the undrained shear strength was measured for the soil bed between the stone columns at a depth of 150 mm after the load testing was completed.

4.6 Soil Replacement Models – The Implementation

Beyond the preparation of the soil bed, the following procedures were implemented, the table (4.5) expressing the details of the cases of the soil replacement.

The following techniques have been put into practice:

1. The center of footing and the borders of the replaced area were present on the soil bed's surface prior to soil bed preparation.
2. Using a portable vane shear equipment, the soil bed undrained shear strength was measured in the center of the replacement area at a depth of 100 or 150 mm.
3. A hand excavator was used to carry out the excavation work until the necessary depth and width were reached.
4. Using a plastic cone to place the crushed stone in two layers at a depth of 100 mm and three layers at a depth of 150 mm in the excavated area.

Each layer has a thickness of 60 mm, and the layers are compressed using a tiny hammer to achieve the necessary dry unit weight of roughly 15.1 kN/m^3.Following the completion of the work, the dirt bed was covered and allowed to cure for a full day.

4.7 The Soil Replacement – The Testing Procedures

1. The bearing frame is positioned so that the footing center and the center of the replenished soil are in synchrony when the curing period is over.

2. The loads applied using loading discs in a manner that permits the load to be increased, with the growing continued until the dial gauge stopped or, in accordance with the standard (ASTM D-1143,2000), reached a penetration of 0.25 mm to 1.25 mm.
3. The dial gauge's recorded readings prior to the subsequent increase in load.
4. The load continued to increase until the failure happened.
5. Beyond the load tests, the measuring of the undrained shear strength implemented near and beneath the replaced soil by the use of the portable vane shear device.

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusions

The conclusion can be noticed from the results of reliability analysis of a footing on a trench of granular soil:

The bearing capacity is a function depends on three variable:

Case one: the foundation resting on a granular trench (a one layer system of soft clay).

2^M : M is the number of the variables (in this case the bearing capacity depends on:

(φ *the effective angle of friction*, γ *the unit weight of soil*, C_u *the undrained shear strength*).

2^3 = 8 so there are 8 probabilities to consider,

From the value of the ultimate bearing capacity obtained by the using of the following equation:

$$q_u = C_2 N_{C(T)} + D_f \gamma_2 N_{q(T)} + \left(\frac{\gamma_2 B}{2}\right) N_{\gamma(T)} = 318.91 \text{ kN} / m^2$$

While in the experimental work and for the same case the bearing capacity = 510 kN/m^2.

This means that the reliability analysis reduce the time, the effort and the cost.

then the probability of failure = 4.14 %. To be compared with the results of the use of:

(Monte Carlo Simulation Methodology, The Analysis of the Safety Factor and the cumulative distribution function analysis (CDF)).

For case two: for a two layers system (strong layer over weak layer)

$$q_u = 1+0.2\frac{B}{L}\,5.14C_2 + \frac{\gamma_1 H^2}{B}\;1+\frac{2Df}{H}\;1+\frac{B}{L}\,k_s \tan\varphi_{1+}\;\gamma_1\,Df \le \gamma_1 Df$$

$$N_{q1}\,S_{q1} + 0.5\gamma_{1\mathrm{B}N\gamma_1 \mathrm{S}\gamma_1}$$

The bearing capacity is a function depends on five variables:

2^5 = 32 so the probabilities are 32.

- **The analysis after the application of ((Monte Carlo Simulation Methodology, The Analysis of the Safety Factor and the cumulative distribution function analysis (CDF)).**

Case one:

$q_u = C_2 N_{C(T)} + D_f \gamma_2 N_{q(T)} + \left(\frac{\gamma_2 B}{2}\right) N_{\gamma(T)}$ = 627.00 kN / m^2

q_{design} = $\frac{q_u}{N}$ = $\frac{627.00}{3}$ = 209.00 kN / m^2

The probability of failure is the probability of the bearing capacity to be less than the design bearing capacity.

$q_u > q_{design}$

The safety factor to be taken = 3 based on industry standards.

The probability of failure calculated to be 1.24 % and it is in the acceptable limits.

1.24 % $<$ 4.14 % meaning the probability of failure occurring is decreased.

Case two:

q_u= 1+0.2$\frac{B}{L}$ 5.14C_2 $+ \frac{\gamma_1 H^2}{B}$ 1+$\frac{2Df}{H}$ 1+$\frac{B}{L} k_s tan\, \varphi_{1+}$ $\gamma_1\, Df \le \gamma_1 Df$

$N_{q1}\, S_{q1}$ $+ 0.5\gamma_{1BN\gamma_1 S\gamma_1}$= 506.17 kN / m^2 $\le$ 302.56 kN / m^2

q_{design} = $\frac{q_u}{N}$ = $\frac{302.56}{3}$ = 100.854 kN / m^2

$q_u > q_{design}$

The probability of failure calculated equal approximately 0.00014 % which is within the acceptable limits.

5.2 Recommendations

For the geotechnical engineers: an accurate works must be done regarding:

- The sources of the used material, the locations of the raw materials and the mistakes obtained from field work or from instruments or devices, all of them may be a source of uncertainties. For geotechnical projects it is necessary to use the methods of analysis to obtain the accurate foundation bearing capacity, since the results and conclusion showed the huge difference in calculations of the bearing capacity and the probability of failure. Even for the standard deviation, the results showed the number of variables to be excluded.

- The use of the histograms shows the accuracy of the results obtained regarding the bearing capacity, cohesion, unit weight and the angle of internal friction

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APPENDICES

Appendix A: The contents of the appendices are two files

- The first file is: The Statistical Analysis of Designed Experiments - 2009 - Tamhane -Appendix C Statistical Tables. (Separated file).
- The second file is: EXCEL sheets consisting of the analysis of the questionnaire (Separated file).
- Article: Based on the subject of the research.

Question	Response	Frequency (%)	Mode	
Engineering projects in which you participated	Agriculture	1,428571429	Buildings	
Engineering projects in which you participated	Ai in databases	1,428571429	Buildings	
Engineering projects in which you participated	Bridges and Roads	12,85714286	Buildings	
Engineering projects in which you participated	Buildings	52,85714286	Buildings	
Engineering projects in which you participated	Chemical	1,428571429	Buildings	
Engineering projects in which you participated	Computer	1,428571429	Buildings	
Engineering projects in which you participated	Control	1,428571429	Buildings	
Engineering projects in which you participated	Dams	2,857142857	Buildings	
Engineering projects in which you participated	Genetics	1,428571429	Buildings	
Engineering projects in which you participated	Geotechnical	1,428571429	Buildings	
Engineering projects in which you participated	Geotechnical Engineering	1,428571429	Buildings	
Engineering projects in which you participated	Industrial water treatment unit	1,428571429	Buildings	
Engineering projects in which you participated	Networks	2,857142857	Buildings	
Engineering projects in which you participated	Networks and design	1,428571429	Buildings	
Engineering projects in which you participated	Power	1,428571429	Buildings	
Engineering projects in which you participated	Project management	1,428571429	Buildings	
Engineering projects in which you participated	Water and Sanitation projects	8,571428571	Buildings	
Engineering projects in which you	design	1,428571429	Buildings	

participated				
Engineering projects in which you participated	physics	1,428571429	Buildings	
Question (1) What is the soil property that has the greatest influence in determining the bearing capacity of the foundations?	Angle of friction	28,57142857	Density	
Question (1) What is the soil property that has the greatest influence in determining the bearing capacity of the foundations?	Cohesion	32,85714286	Density	
Question (1) What is the soil property that has the greatest influence in determining the bearing capacity of the foundations?	Density	38,57142857	Density	
Question (2) The choice of the factor of safety depends largely on:	Previous experience	30	The applied load	
Question (2) The choice of the factor of safety depends largely on:	The applied load	42,85714286	The applied load	
Question (2) The choice of the factor of safety depends largely on:	The nature of failure	27,14285714	The applied load	
Question (3) Reliability index is related to the probability of failure of the foundation:	Largely	41,42857143	Largely	
Question (3) Reliability index is related to the probability of failure of the foundation:	Moderately	37,14285714	Largely	
Question (3) Reliability index is related to the probability of failure of the foundation:	The relation between them is weak	21,42857143	Largely	
Question (4) One of the benefits of the reliability is that the structure can be designed according to serviceability conditions:	Agree	55,71428571	Agree	
Question (4) One of the benefits of the reliability is that the structure can be designed according to serviceability conditions:	Agree well	40	Agree	
Question (4) One of the benefits of the reliability is that the structure can be designed according to serviceability conditions:	Disagree	4,285714286	Agree	
Question (5) In reliability analysis, the overall risk involved in the project is clearly identified.	Agree	52,85714286	Agree	
Question (5) In reliability analysis, the overall risk involved in the project is clearly identified.	Agree well	32,85714286	Agree	
Question (5) In reliability analysis, the overall risk involved in the project is clearly identified.	Disagree	14,28571429	Agree	
Question (6) Reliability analysis provides a	Agree	48,57142857	Agree	

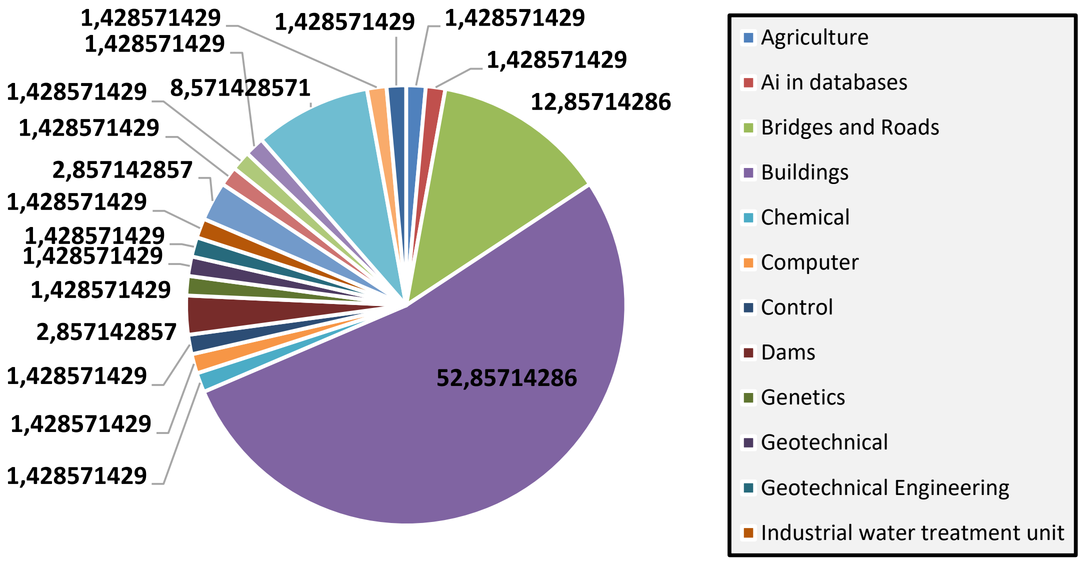
framework for establishing appropriate factors.				
Question (6) Reliability analysis provides a framework for establishing appropriate factors.	Agree well	47,14285714	Agree	
Question (6) Reliability analysis provides a framework for establishing appropriate factors.	Disagree	4,285714286	Agree	
Question (7) The reliability index is seen to be the reciprocal of the coefficient of variation of the safety margin. Is this definition?	Appropriate	35,71428571	Beneficial	
Question (7) The reliability index is seen to be the reciprocal of the coefficient of variation of the safety margin. Is this definition?	Beneficial	50	Beneficial	
Question (7) The reliability index is seen to be the reciprocal of the coefficient of variation of the safety margin. Is this definition?	Has no importance	14,28571429	Beneficial	
Question (8) Which do you prefer in foundation designs?	Both of them	60	Both of them	
Question (8) Which do you prefer in foundation designs?	Other	1,428571429	Both of them	
Question (8) Which do you prefer in foundation designs?	Reliability coefficients	25,71428571	Both of them	
Question (8) Which do you prefer in foundation designs?	Traditional factors of safety	12,85714286	Both of them	
Question (9) In the context of decision making in geotechnical engineering practice, the scenario includes being situated at a specific site. Hence, do you think that site investigation is necessary ?	Agree	42,85714286	Agree well	
Question (9) In the context of decision making in geotechnical engineering practice, the scenario includes being situated at a specific site. Hence, do you think that site investigation is necessary ?	Agree well	51,42857143	Agree well	
Question (9) In the context of decision making in geotechnical engineering practice, the scenario includes being situated at a specific site. Hence, do you think that site investigation is necessary ?	Disagree	5,714285714	Agree well	
Question (10) Although numerical models are highly sophisticated, the input ground profile is typically simple and deterministic. Do you think that reliable analysis is	Agree	54,28571429	Agree	

required?				
Question (10) Although numerical models are highly sophisticated, the input ground profile is typically simple and deterministic. Do you think that reliable analysis is required?	Agree well	38,57142857	Agree	
Question (10) Although numerical models are highly sophisticated, the input ground profile is typically simple and deterministic. Do you think that reliable analysis is required?	Disagree	7,142857143	Agree	
Question (11) Reliability-based design (RBD) is a rational methodology that allows practitioners to design geotechnical elements to achieve a target probability of failure.	Agree	57,14285714	Agree	
Question (11) Reliability-based design (RBD) is a rational methodology that allows practitioners to design geotechnical elements to achieve a target probability of failure.	Agree well	31,42857143	Agree	
Question (11) Reliability-based design (RBD) is a rational methodology that allows practitioners to design geotechnical elements to achieve a target probability of failure.	Disagree	11,42857143	Agree	
Question (12) The characterization of geologic uncertainties is more challenging than the aforementioned geotechnical uncertainties,	Agree	55,71428571	Agree	
Question (12) The characterization of geologic uncertainties is more challenging than the aforementioned geotechnical uncertainties,	Agree well	30	Agree	
Question (12) The characterization of geologic uncertainties is more challenging than the aforementioned geotechnical uncertainties,	Disagree	14,28571429	Agree	
Question (13) Can the geotechnical engineers make use of probability analysis concepts in judgments?	Agree	44,28571429	Agree	
Question (13) Can the geotechnical engineers make use of probability analysis concepts in judgments?	Agree well	41,42857143	Agree	
Question (13) Can the geotechnical engineers make use of probability analysis concepts in judgments?	Disagree	14,28571429	Agree	
Question (14)	Agree	55,71428571	Agree	

Structural failure consists of shear failure and moment failure, soil failure consists of bearing failure and settlement failure. In addition to above the foundation should satisfy other requirements.				
Question (14) Structural failure consists of shear failure and moment failure, soil failure consists of bearing failure and settlement failure. In addition to above the foundation should satisfy other requirements.	Agree well	37,14285714	Agree	
Question (14) Structural failure consists of shear failure and moment failure, soil failure consists of bearing failure and settlement failure. In addition to above the foundation should satisfy other requirements.	Disagree	7,142857143	Agree	
Question (15) The foundation should be probably located regarding frost action and volume change adjacent structure.	Agree	54,28571429	Agree	
Question (15) The foundation should be probably located regarding frost action and volume change adjacent structure.	Agree well	35,71428571	Agree	
Question (15) The foundation should be probably located regarding frost action and volume change adjacent structure.	Disagree	10	Agree	
Question (16) Certainty can be a description for most engineering problems.	Agree	45,71428571	Agree	
Question (16) Certainty can be a description for most engineering problems.	Agree well	35,71428571	Agree	
Question (16) Certainty can be a description for most engineering problems.	Disagree	8,571428571	Agree	
Question (16) Certainty can be a description for most engineering problems.	Other	10	Agree	
Question (17) The sources of uncertainty are unavoidable.	Agree	37,14285714	Agree	
Question (17) The sources of uncertainty are unavoidable.	Agree well	35,71428571	Agree	
Question (17) The sources of uncertainty are unavoidable.	Disagree	17,14285714	Agree	
Question (17) The sources of uncertainty are unavoidable.	Other	10	Agree	
Question (18) Does the accuracy of theoretical and empirical methods for calculating bearing capacity of soil considered as	Agree	44,28571429	Agree	

one of uncertainty sources?				
Question (18) Does the accuracy of theoretical and empirical methods for calculating bearing capacity of soil considered as one of uncertainty sources?	Agree well	32,85714286	Agree	
Question (18) Does the accuracy of theoretical and empirical methods for calculating bearing capacity of soil considered as one of uncertainty sources?	Disagree	15,71428571	Agree	
Question (18) Does the accuracy of theoretical and empirical methods for calculating bearing capacity of soil considered as one of uncertainty sources?	Other	7,142857143	Agree	
Question (19) Variability and randomness cause a difficulty in selecting the suitable design parameter.	Agree	52,85714286	Agree	
Question (19) Variability and randomness cause a difficulty in selecting the suitable design parameter.	Agree well	32,85714286	Agree	
Question (19) Variability and randomness cause a difficulty in selecting the suitable design parameter.	Disagree	10	Agree	
Question (19) Variability and randomness cause a difficulty in selecting the suitable design parameter.	Other	4,285714286	Agree	
Question (20) The stone column technique of ground treatment has proven successful in increasing the time rate of settlement.	Agree	54,28571429	Agree	
Question (20) The stone column technique of ground treatment has proven successful in increasing the time rate of settlement.	Agree well	30	Agree	
Question (20) The stone column technique of ground treatment has proven successful in increasing the time rate of settlement.	Disagree	14,28571429	Agree	
Question (20) The stone column technique of ground treatment has proven successful in increasing the time rate of settlement.	Other	1,428571429	Agree	
Question (21) Cohesion, angle of internal friction and soil unit weight are independent and uncorrelated variables.	Agree	34,28571429	Disagree	
Question (21) Cohesion, angle of internal friction and soil unit weight are independent and uncorrelated variables.	Agree well	28,57142857	Disagree	
Question (21) Cohesion, angle of internal friction and soil unit weight are independent and	Disagree	35,71428571	Disagree	

uncorrelated variables.				
Question (21) Cohesion, angle of internal friction and soil unit weight are independent and uncorrelated variables.	Other	1,428571429	Disagree	



RESUME

EDUCATION:

- BSc. In Civil Engineering, University of Technology, Baghdad – Iraq.
- M.Sc. in Engineering Menegement Istanbul Gedik University, Istanbul

OBLECTIVE:

I seek challenging opportunities where I can fully use my skills for the success of the organization.

SKILLS:

Civil engineer. Microsoft word.

Marketing. And Exhibitions Manager. Of course i speak Arabic. fluent in English. And about 70% Turkish. Having an adequate level in dealing with Electronics.

Since 2007. -

Contracts Engineer. Sight Engineer.

Exhibition Manager.

I worked at many sights in South and North of Iraq. Governorates of (Misan; Nasiriya; Basra; Baghdad and Erbil.

I worked in Istanbul also.