Estimation of Modulus of Elasticity from Mean Grain Size of Granular Soil and Spreadsheet of Settlement Computational Methods

FINAL YEAR PROJECT UG-2014

Submitted by

Haroon Ahmad Saeed	NUST 2014 -32260
Talha Atique	NUST2014-32647
Muhammad Ahsan	NUST2014-32670

NUST Institute of Civil Engineering

School of Civil and Environmental Engineering

National University of Sciences and Technology, Islamabad, Pakistan

2018

This is to certify that the

Final Year Project, titled

Estimation of Modulus of Elasticity from Mean Grain Size of Granular Soil and Spreadsheet of Settlement Computational Methods

Submitted by

Haroon Ahmad Saeed	**NUST 2014 -32260**
Talha Atique	**NUST2014-32647**
Muhammad Ahsan	**NUST2014-32670**

Has been accepted towards the requirements

for the undergraduate degree

in

CIVIL ENGINEERING

____

Dr. S. Muhammad Jamil

Dean SCEE

NUST Institute of Civil Engineering

School of Civil and Environmental Engineering

National University of Sciences and Technology, Islamabad, Pakistan

ACKNOWLEDGMENTS

We would first like to thank our advisor Dr. S. Muhammad Jamil, here at NUST Institute of Civil Engineering. He remained helpful throughout the project and kept us motivated. I am gratefully indebted to his valuable comments and critique on our work and directions he provided throughout this project.

I would also like to thank my co-advisor Lec. Muhammad Asim for his continued support, encouragement and innovative Ideas throughout the project. He always addressed issues we faced during project and helped us to resolve them.

Finally, I must express my very profound gratitude to my parents for providing me with unfailing support and continuous encouragement throughout my years of study. This accomplishment would not have been possible without them. Thank you.

ABSTRACT

Soil Young's modulus (E), commonly referred to as soil elastic modulus, is an elastic soil parameter and a measure of soil stiffness. It is defined as the ratio of the stress along an axis over the strain along that axis in the range of elastic soil behavior. The elastic modulus is often used for estimation of soil settlement and elastic deformation analysis.

Soil elastic modulus can be estimated from laboratory or in-situ tests or based on correlation with other soil properties. In laboratory, it can be determined from tri-axial test or indirectly from odometer test. On field, it can be estimated from Standard penetration test, Cone penetration test, pressure meter or indirectly from dilatometer test.

The main hurdle in estimating the settlement for a desired structure is to identify the value of elastic modulus (E). As a lot of work has done for the calculation of elastic modulus but a proper estimation for elastic modulus has yet to achieve.

Our basic aim is to increase our understanding of interaction between elastic modules and Soil Penetration Test value (N value), elastic modules and Cone Penetration Test value (q_c value). We aim to achieve a relationship between D_{50} and elastic modulus for general granular soils

We aim to make automated excel spread sheets for various Settlement methods proposed by different geotechnical Scientists and researchers by using elastic modulus of soil and their comparison with settle 3D.

Table of Contents

ACKNOWLEDGMENTS 3

ABSTRACT 4

LIST OF FIGURES 8

Chapter 1 10

INTRODUCTION 10

1.1 BACKGROUND 10

1.2 OBJECTIVES AND LIMITATIONS OF STUDY: 11

1.3 LEARNING OUTCOMES: 12

CHAPTER 2 13

LITERATURE REVIEW 13

2.1 GENERAL 13

2.1.1 FOUNDATION ENGINEERING 13

2.1.2 GEOTECHNICAL ENGINEERING 13

2.1.3 STRUCTURAL ENGINEERING 14

2.1.4 UNCERTAINTIES 14

2.2 SETTLEMENT 14

2.2.1 COMPONENTS OF SETTLEMENT 15

THE COMPONENTS OF SETTLEMENT OF A FOUNDATION ARE: 15

2.2.1.1 IMMEDIATE SETTLEMENT 15

2.2.1.2 CONSOLIDATION SETTLEMENT 15

2.2.1.3 SECONDARY COMPRESSION 16

2.2.1.4 SETTLEMENT LIMITS 16

2.3 PUBLICATIONS 16

2.4 METHODS REVIEWED 18

2.4.1 CANADIAN FOUNDATION ENGINEERING MANUAL (1975, 1985, 1992) (CFM)18

2.4.2 TSCHEBOTARIOFF (1953, 1971) 20

2.4.3 CARRIER AND CHRISTIAN (1973) 22

2.4.4 THOMAS (1968) 23
2.4.5 D'APPOLONIA ET AL. (1970) 27
2.4.6 PAPADOPOULOS (1992) 27
2.4.7 SCHMERTMANN ET AL. (1978) 29
2.4.7.1 RECENT MODIFICATIONS IN STRAIN-INFLUENCE FACTOR 31
DIAGRAMS 31
2.5 STANDARD PENETRATION TEST 34
2.6 CONE PENETRATION TEST 38
CHAPTER 3 44
METHODOLODY 44
3.1 DESIGN CHART METHODOLOGY 44
3.1.1 INPUT DATA 44
3.1.2 BEARING CAPACITY CALCULATIONS 45
3.1.3 SETTLEMENT CALCULATIONS 45
3.1.3.1 NON-COHESIVE SOIL 45
3.1.3.2 USING AUTOMATED WORKBOOK 45
3.1.3.3 CORRELATIONS 46
3.1.3.4 CALCULATION OF ELASTIC MODULUS 46
3.1.3.5 FRICTION ANGLES OF GENERAL GRANULAR SOILS 47
3.1.4 CORRELATION OF ELASTIC MODULUS AND MEAN GRAIN SIZE 48
3.1.5 THE TERM W 49
3.2 ADDITIONAL FEATURE 49
3.2.1 GRADATION CURVE 49
3.3 SOIL TEST DATA 49
3.4 OUTPUT OF SPREADSHEET 50
3.4.1 SETTLEMENT 50
3.4.2 ELASTIC MODULUS 50
CHAPTER 4 51

RESULTS AND DISCUSSIONS 51
4.1 DESIGN CHARTS 51
4.2 SETTLEMENT SPREAD SHEETS 52
4.2.1 HOME PAGE 52
4.2.1.1 INPUT SECTION 53
4.2.1.2 OUTPUT SECTION 55
CHAPTER 5 56
VERIFICATION OF RESULTS 56
5.1 VERIFICATION USING SETTLE 3D 56
5.1.1 EXAMPLE1: SETTLE – 3D (SETTLEMENT) 56
5.1.2 EXAMPLE 1: SPREAD SHEET (SETTLEMENT) 57
5.1.3 RESULTS OF EXAMPLE 1 (SETTLEMENT) 57
5.2 EXAMPLES: ELASTIC MODULUS AND MEAN GRAIN SIZE RELATION 57
5.2.1 VERIFICATION OF E & D_{50} RELATION 58
5.2.1.1 EXAMPLE 1 58
5.2.1.2 EXAMPLE 2 58
LIMITATIONS AND RECOMMENDATIONS 59
REFERENCES 60

LIST OF FIGURES

Figure 1: Chart for influence factor, I_c, after Kany (1959) from Canadian Foundation Manual (1985) 20

Figure 2: Compression of a Truncated Pyramid of Elastic Material 21

Figure 3: Layer Thickness Correction Factor, C, (after Tschebotarioff 1953, 1971) 22

Figure 4: Settlement Ratio Curves Presented by Carrier and Christian (1973). 23

Figure 5 Fox (1948) Embedment Correction Factor. 25

Figure 6: Thomas (1968) Elastic Modulus from CPT 26

Figure 7: Settlement Ratio as a Function of Load Level (Thomas 1968). 26

Figure 8: Papadopoulos (1992) Settlement Factor. 29

Figure 9: Theoretical and experimental distribution of vertical strain influence factor below the center of a footing 30

Figure 10: Revised strain influence factor diagram suggested by Schmertmann et al. (1978) 30

Figure 11 Strain influence diagram suggested by Terzaghi et al. (1996). 32

Figure 12 Variation of I_z'/ I_z with D_f/B (after Terzaghi et al. 1996) 33

Figure 13 Correlation between E_s and q_c for square and circularly loaded areas [adapted from Terzaghi et al. (1996)]. 34

Figure 14 Comparison of end of construct ion predicted and measured S_e of foundations on sand and gravel 34

Figure 15 estimate of Soil Modulus from SPT & CPT - comparison 36

Figure 16 Elastic Modulus for SPT Data 37

Figure 17 Elastic Modulus for SPT (continued) 38

Figure 18 Modulus from SPT and CPT (after Bowles 1988) 40

Figure 19 Soil Modulus from CPT (from Mitchell and Gardner) 41

Figure 20 Different Correlations for Calculating Modulus (various) 42

Figure 21 Different Correlations for Calculating Modulus (continued) 43

Figure 22 Correlation of N and phi ϕ 46

Figure 23 Kulhawy & Mayne (1991) Correlation 47

Figure 24 Proposed Relation between W and Elastic Modulus in tsf 51

Figure 25 General Overview of Home page of Settlement Sheet 52

Figure 26 Data Input Section - Foundation Inputs 53

Figure 27 Data Input Section - Soil Layer Inputs 53

Figure 28 Output Section - Settlement Maximum, Minimum and Comparison Box 55

Figure 29 Output Section - Modulus of Elasticity 55

Figure 30 SETTLE 3d Example Input Data & Settlement Calculated 56

Figure 31 Calculated Settlement on Spreadsheet for the example data 57

Chapter 1

INTRODUCTION

1.1 Background

Shallow foundations are among the most intensive applications of principles of basic geotechnical engineering and have served as a reliable, efficient, simple method for load transfer from the super structure to the strata below. Due to the critical nature of their design, the foundations must be rigorously evaluated not only against structural failure and cracking but also against failure due to differential settlement and bearing failure. Due to difficulty in quantizing the exact behavior of soil and the fact that whole interactions of soil particles can never be taken into account when calculating the essential soil strength parameters, the calculated strengths are often estimates based on empirical relationships and statistical models; consequentially a factor of safety of three is mostly applied to account for any over estimation of strength parameter
A controlling bearing capacity estimate (after application of factor of safety) is calculated as the minimum of the bearing capacity estimate provided by the settlement based expression and the expression taking into account the bearing of soil. It has been noted that once the foundation width exceeds 1.5 m, the settlement generally starts controlling the maximum safe bearing capacity

Settlement being the major factor in designing Foundation of a structure has to be calculated first hand. The most popular methods for settlement predictions, discussed commonly in text books, are the ones proposed by Terzaghi and Peck (1948), Schmertmann (1970), Schmertmann et al. (1978) and Burland and Burbidge (1985). Meyerhof (1956) and Peck and Bazaraa (1969) methods are similar to the one proposed by Terzaghi and Peck (1948). Two of the more recent methods are after Berardi and Lancellotta (1991) and Mayne and Poulos (1999). Sivakugan and Johnson (2004) proposed a probabilistic approach quantifying the uncertainties associated with the settlement prediction methods.

These methods, for the sake of ease of identification can be divided into methods based on the general elastic solution mechanics, the semi empirical methods incorporating factors from both statistical models as well as mechanical based models as well as completely empirical ones based on data sets obtained from test foundations

In calculating settlement the main controlling factor is to get correct value of elastic modulus of soil. Determining the value of elastic modulus is the most time taking work as its value cannot be calculated directly and various relations have to be used for estimating it. Elastic modulus could also be calculated from in-situ tests but the spread in the results raises questions about the reliability of the dataset as a whole. The correlations between elastic modulus and cone penetration test value (E) also have discrepancies such as $E=2q_c$ to $3.5q_c$ by Schmertmann (1978) and $E=11q_c$ by Lambart (1991).

Therefore elastic modulus has very vast ranges and to get an elastic modulus value which is a fair estimate of the strength of soil is as difficult as it is important.

1.2 Objectives and Limitations of study:

Basic aim of project is to provide a correlation generally for elastic modulus of granular soils in term of Mean Grain Size D_{50} which provides an acceptable value of elastic modulus. By using the values of elastic modulus we can get appropriate settlement as settlement is very critical factor in foundation design.

Since foundations provide basis for every civil engineering structure, so it is important to learn about various methods that are available to get settlement of foundations. This project then aims to study the limitations and assumptions of each method and get a practical idea of where to apply a certain method. The limitations of the project are that not all the methods for calculating settlement are catered in this project. The methods based on standard penetration test and Cone penetration tests were used to make excel sheets. Only granular soils were catered. Elastic behavior was assumed. A selective representative list of relations was used for computing modulus of elasticity

1.3 Learning Outcomes:

Developed a keen insight about

1. Variation of elastic modulus of soil and the effect on the ranges of elastic modulus with the variation of soil.
2. Learned different Settlement computational methods.
3. Developed an acceptable and comprehensive correlation between mean grain size (D_{50}) and elastic modulus of soil.
4. Learned how to use computer software for predicting and analyzing settlement using settle 3D.
5. Learned how to use advanced Microsoft excel (MS EXCEL).

CHAPTER 2

LITERATURE REVIEW

2.1 General

"A structure is no stronger than its connections" although this statement usually invokes images of connections between individual structural members it also applies to those between a structure and the terrain that supports it. These connections are known as its foundations. Even the ancient builders knew that the most carefully designed structure can fail if they are not supported on suitable foundations for example tower of Pisa in Italy is perfect example.

2.1.1 Foundation Engineering

Foundation engineer should have the knowledge of all the multiple disciplines of Civil Engineering i-e,

1. Geotechnical Engineering
2. Structural Engineering

2.1.2 Geotechnical engineering

All foundations interact with the ground, so the design must reflect the technical characteristics and behavior of the soil and adjacent rocks, so the engineer has to understand geotechnical engineering, and most civil engineers are also geotechnical engineers.

2.1.3 Structural Engineering

The factors that must be considered in the design of the foundation include; the transfer of the load to the ground and support for the structure. A foundation is a structural element that transmits the load acting on the ground. So whoever can transfer the imposed burdens must understand the principles and practices of building construction. In addition, the foundation supports a structure. We have to understand the sources and the nature of the structural burdens and the tolerance of the movements of the founding structure.

2.1.4 Uncertainties

"Despite much progress in the theory of foundation engineering, there are still gaps in understanding In general, uncertainties are the result of our limited knowledge of soil conditions." The prediction of the compaction of granular soils is also very uncertain because the location of the modulus of elasticity of the granular soil that are difficult to calculate depend on there being many relationships and correlations for modulus of elasticity and soil penetration (number of shots per foot) and test of cone penetration (q_c value).

2.2 Settlement

A soil shear failure can result in excessive building deformation and even collapse. Excessive settling can result in structural damage to a building frame nuisances such as sticking doors and windows, cracks in tile and plaster, and excessive wear or equipment failure from misalignment resulting from foundation settlements.

It is necessary to investigate both base shear resistance (ultimate bearing capacity) and settlements for any structure. In many cases settlement criteria will control the allowable bearing capacity.

Except for occasional happy coincidences, soil settlement computations are only best estimates of the deformation to expect when a load is applied.

2.2.1 Components of Settlement

The components of settlement of a foundation are:

1. Immediate settlement
2. Consolidation Settlement, and
3. Secondary compression (creep)

$$\Delta H = \Delta H_i + U\ \Delta H_c + \Delta H_s$$

ΔH = total settlement, ΔHc = consolidation settlement, ΔH = secondary compression, U = average degree of consolidation. Generally, the final settlement of a foundation is of interest and U is considered equal to 1 (i.e. 100% consolidation).

2.2.1.1 Immediate Settlement

Immediate settlement takes place as the load is applied or within a time period of about 7 days. Predominates in cohesion-less soils and unsaturated clay. Immediate settlement analysis are used for all fine-grained soils including silts and clays with a degree of saturation < 90% and for all coarse grained soils with large co-efficient of permeability (say above 10.2 m/s)

2.2.1.2 Consolidation Settlement

Consolidation settlements are time dependent and take months to years to develop. The leaning tower of Pisa in Italy has been undergoing consolidation settlement for over 700 years. The lean is caused by consolidation settlement being greater on one side. This, however, is an extreme case. The principal settlements for most projects occur in 3 to 10 years.

Dominates in saturated/nearly saturated fine grained soils where consolidation theory applies. Here we are interested to estimate both consolidation settlement and how long a time it will take or most of the settlement to occur.

2.2.1.3 Secondary Compression

Creep occurs under constant effective stress due to continuous rearrangement of clay particles into a more stable configuration.

Creep predominately occurs in highly plastic clays and organic clays.

2.2.1.4 Settlement Limits

Total settlement is the magnitude of downward movement. Differential settlement is non-uniform settlement. It is "the difference of settlement between various locations of the structure. Angular distortion between two points under a structure is equal, to the differential settlement between the points divided by the distance between them.

Theoretically, no damage will be done to a structure if it settles uniformly as a whole regardless of how large the settlement may be. The only damage would be to the connections of the underground utility lines. However, when the settlement is non-uniform (differential), as is always the case, damage may be caused to the structure.

The tolerable, settlements of different structures, vary considerably. Simple-span frames can take considerably greater distortion than rigid frames. A fixed-end arch would suffer greatly if the abutments settle or rotate. For road embankments, storage silos and tanks a settlement of 300mm - 600mm may be acceptable, but for machine foundations the settlement may be limited to 5mm 30mm. Different types of construction materials can withstand different degrees of distortion. For example, sheet metal wall panels do not show distress as readily as brick masonry. In Sands maximum total settlement = 40 mm for isolated footings = 40 to 65 mm for rafts .Maximum differential settlement between adjacent columns = 25 mm.

2.3 Publications

A representative list of literature referred to for the purpose of this project from where these methods were taken are listed below

- SETTLEMENT OF SHALLOW FOUNDATIONS ON GRANULAR SOILS Final Report Alan .lutenegger & Don j. Degroot
- ELASTIC SETTLEMENT OF SHALLOW FOUNDATION ON GRANULAR SOIL- A CRITICAL REVIEW by Braja. M. Das
- SETTLEMENT OF SHALLOW FOUNDATIONS ON GRANULAR SOILS: AN OVERVIEW by Sivkugan (2016)
- CORRELATION OF STANDARD AND CONE PENETRATION TESTS FOR SANDY AND SILTY SAND TO SANDY SILT SOIL Bashar Tarawneh Ph. D, P.E Assistant Professor Civil Engineering Department The University of Jordan Amman, Jordan 11942
- A UNIFIED CPT-SPT CORRELATION FOR NON-CRUSHABLE AND CRUSHABLE AND COHESION LESS SOILS Sayed M. Ahmad a, Sherif W. Agaibyb, Ahmed H. Abdel-Rahman

Generally these publications divide soil settlement methods into two different types; Theoretical elasticity based methods that compute layer strain and general empirical, semi empirical and/or statistical methods based on direct field measurement inputs.

While rigorous computation based methodology has been developed in later years to cater for the elastoplastic behavior of soil, the bulk of settlement computational methods treat soil under stress as an elastic medium. These methods generally employ a solution of the form s=$^{qB}/_{E}$. The major variation in different methods is the calculation of a dimensionless influence factor that dictates to what extent would each substratum layer receive the increase in stress, and to what extent that load would produce vertical strains and to what extent would those strains contribute to total settlement. Varying from simpler computations like Tschebatarioff and Canadian Foundation Manual to more rigorous strain based computations like Das, Bowles, Whals and Gupta’s and Oweiss’s methods which calculate different factors to further estimate the value of influence factor I.

The other kind of calculations incorporates empirical, semi-empirical and/or statistical approach based on the regression analysis of large sets of data from settlements on test foundations and on plate load tests on river sands. These methods incorporate field measurements such as blow counts for Soil Penetration Test and Cone tip resistance for Cone

Penetration Test along with correction factors to estimate settlement and range from simpler methods such as Terzaghi, Peck and Mesri to multi-layered methods that incorporate elements from bearing capacity design of foundations such as Stroud's and Hough's methods.

2.4 Methods Reviewed

In this project the main focus is on the immediate settlement of granular soils and the correlation of the mean grain size (D_{50}) and elastic modulus of soil. The methods which were reviewed and used during the project for calculating the immediate settlement and making automated Microsoft excel sheet are as follows

1. Canadian Foundation Engineering Manual (1975, 1985, 1992) (CFM)
2. Tschebotarioff (l953, 1971)
3. Carrier and Christian (1973)
4. Thomas (1968)
5. D'Appolonia et al. (1970)
6. Papadopoulos (1992)
7. Schmertmann et al.(1978)
8. Schmertmann (1970)
9. Terzaghi Peck & Misri (1996)
10. Lee & Salgado (2008)

2.4.1 Canadian Foundation Engineering Manual (1975, 1985, 1992) (CFM)

Canadian Foundation Manual often referred to as CFM calculates the settlement by dividing the soil into layers and using a pre-determined value of elastic modulus with a correction factor and the general elastic solution to calculate settlement such:

$E_i = q_i/E_{soil,}$

where:

q_i = stress at the ith layer

E_{soil}= Soil modulus of elasticity

The total settlement is obtained from:

s = Σ Ei*hi,

or

$s = \Sigma(q_i/E_s)h_i$

where:

s = settlement

h_i =thickness of the ith layer

CFM also presented the equation as a recommendation as:

$s= (q*B*I_c)/E_s$

where:

s = settlement

q_o = applied net footing stress

B = footing width

E_z = apparent modulus elasticity

I_c = influence factor settlement of each layer.

The I_c influence factor was reproduced from Kanye (1959) and like other similar factors incorporates changes and variations in foundation geometry and types among other factors. The factor is reproduced on the next page and includes different lines for variation in L/B and Z/B on y axis

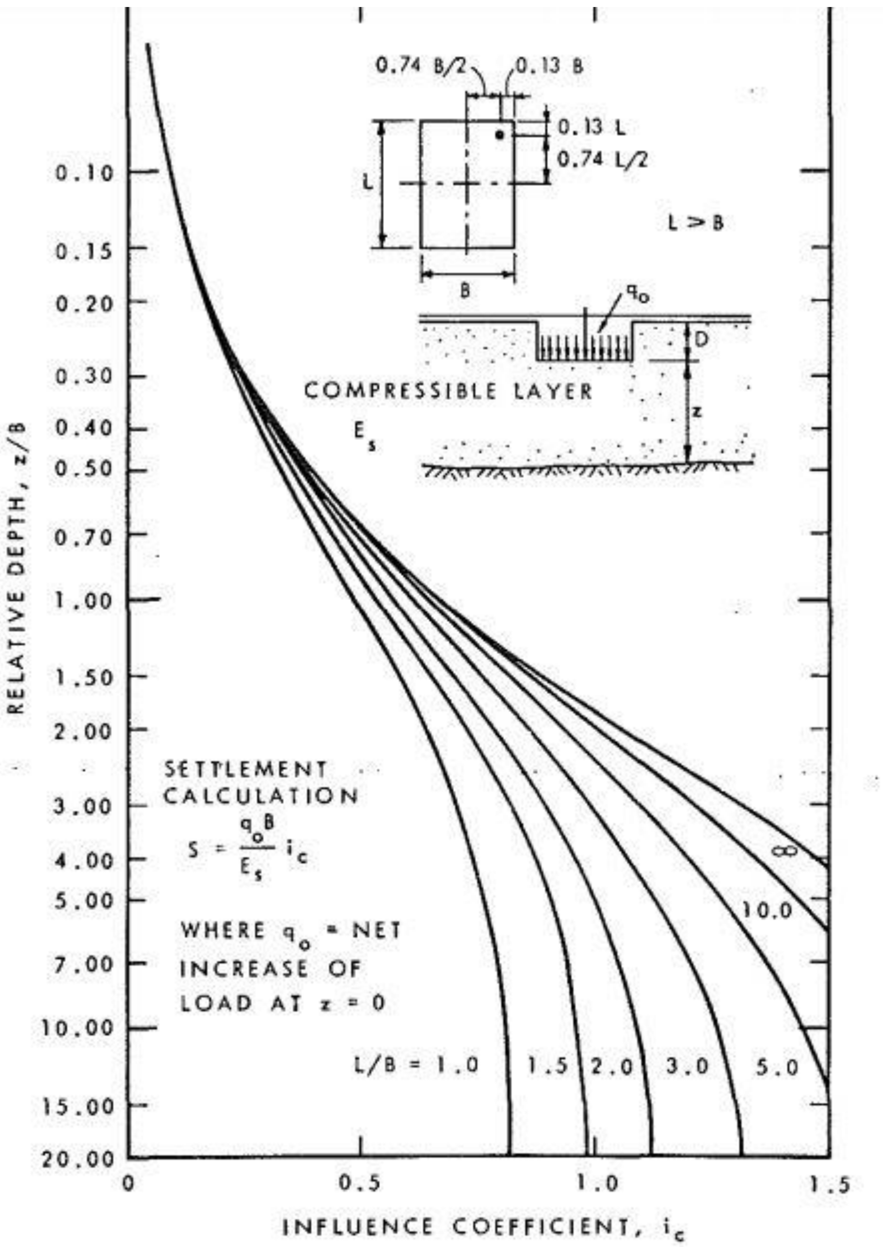

Figure 1: Chart for influence factor, I_c, after Kany (1959) from Canadian Foundation Manual (1985)

2.4.2 Tschebotarioff (1953, 1971)

Tschebotarioff (1953, 1971) suggested a simplified method which works on the assumption that the stresses applied to the soil vertically propagate in a pyramid formation and that each layer will have its own stress share of the pyramid; the individual settlement of each layer can be summed up to produce the total settlement that will occur as a result of application of loading. The method assumes that the area occupation by each layer is A= (b + 2H tanϕ)*2, where α is defined as shown in Figure 2. For an assumed value of ϕ = 30', the method provides a reduced expression for computing settlement as:

s = (0.867 q*b*Cs)/E

where:

q = applied footing stress

b = footing width

C_s = layer thickness correction factor

E = Young's Modulus

The correction factor, while catering for changes in foundation geometry and embedment, also caters for increasing values of H, as with increase in H, increases the area under the "pyramid" leading to a higher value of Predicted Q

s = [(2.0*q*b)/ E]*log [1 + (1.154*H)/b]

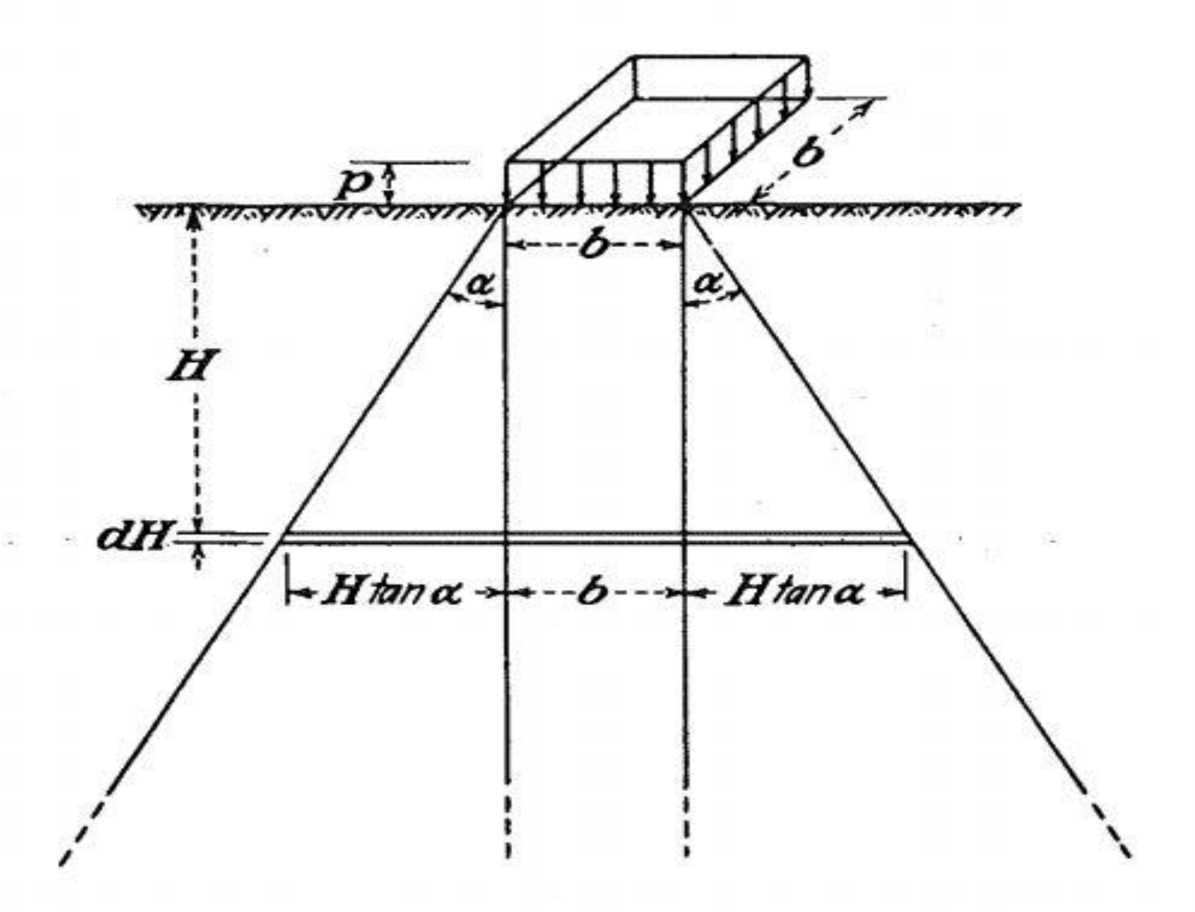

Figure 2: Compression of a Truncated Pyramid of Elastic Material

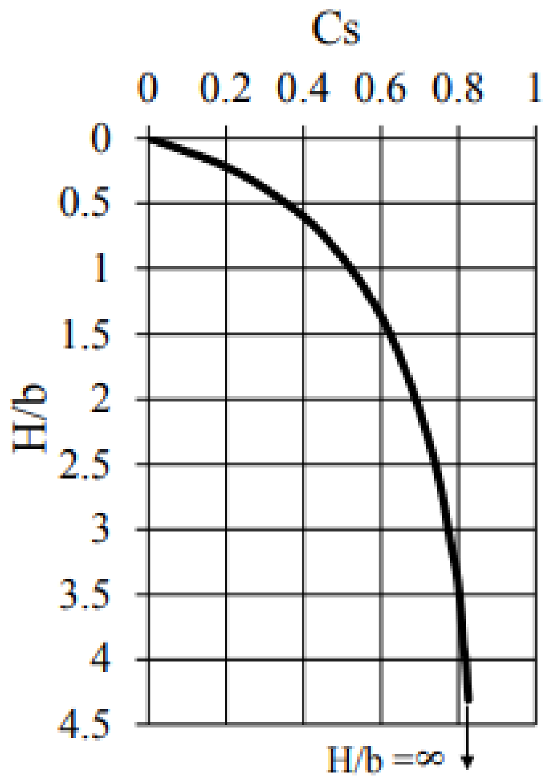

Figure 3: Layer Thickness Correction Factor, C, (after Tschebotarioff 1953, 1971)

2.4.3 Carrier and Christian (1973)

Carrier and Christian (1973) used the finite element approach to solve the dislocation and stresses induced by a rigid circular plate resting on an inhomogeneous elastic half space defined by a Young's modulus (E) of increasing depth increases with:

$E=E_o + K$

where:

E_o= Young's modulus at surface (i.e., z = 0)

K = rate of increase in E with depth

The results of comparison with methods that treat E as non-varying with depth were presented by considering the elastic settlement ratio as a function of foundation width, similar to what had previously been presented by Terzaghi and Peck (1948, 1967) and Bjerrum and Eggestad (1963). As shown in Figure 4, solutions were shown for various ratios of E/K ranging from 0 to infinity. These results show that the settlement ratio of footings on a non-homogenous half-space increases linearly with the logarithm of footing width.

An alternative approach may be to use the results of penetration tests, such as the CPT or SPT to evaluate the variation in soil modulus with depth to obtain the value of K. The value of E_o would then be obtained, as before, using a plate load test on a 0.3m (1ft.) wide plate.

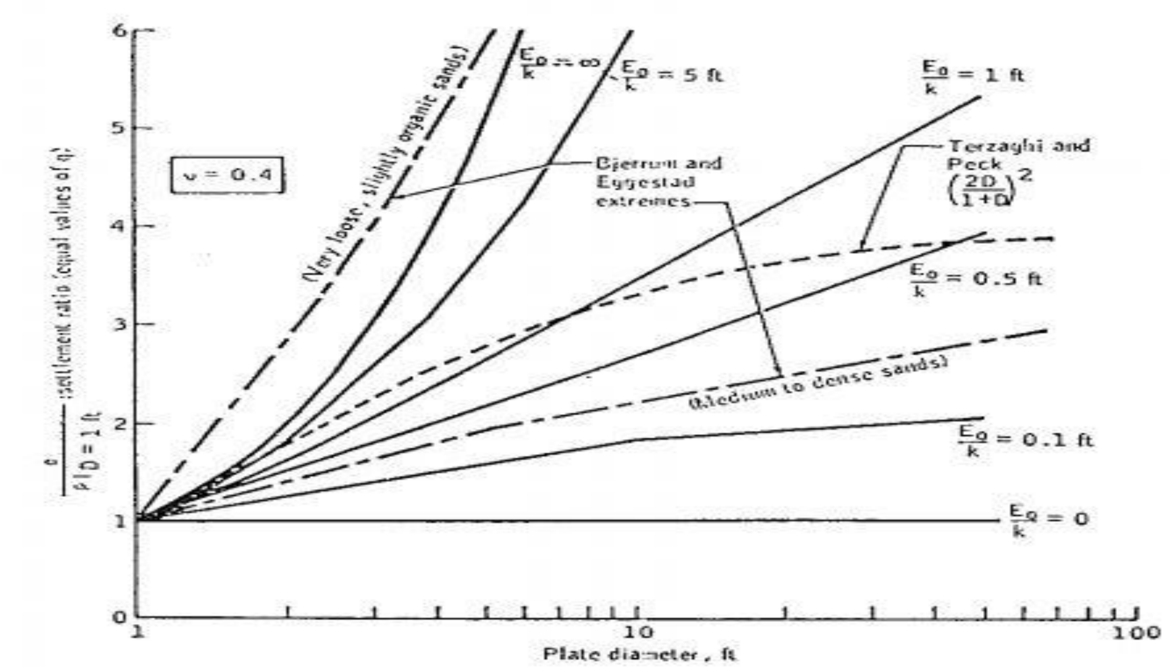

Figure 4: Settlement Ratio Curves Presented by Carrier and Christian (1973).

2.4.4 Thomas (1968)

Thomas (1968) suggested a method based essentially on the calculation of consolidation settlements using an expression identical to that of DeBeer and Martens (1957). However, whereas the previous suggestion of DeBeer and Martens (1957) had been to define the constant of compressibility C as:

$c = 1.5\ q/p'_o$

which can also be used to express the elastic modulus of the soil as:

$E=1.5*q_c$

Thomas found that for a normally consolidated sand, the elastic modulus was related to cone tip resistance as $3q < E < 12q$ with the lower coefficient corresponding to high cone tip resistance and grain crushing.

The elastic expression of Section 4 was used to calculate settlements as:

$s = I*q*B*(1 - \mu^2)/E$

where:

s = settlement (in ft.)

I= an influence factor which depends on footing geometry (LIB)

q = applied footing stress (in tsf)

B = footing width (in ft)

E = Elastic modulus (in tsf)

μ =Poisson's ratio

The influence factor, I, incorporating correction for embedment was obtained using the expressions proposed by Fox (1948; see Figure 5).

Thomas (1968) found that the ratio of estimated to observed settlement, defined as R,, was related to the level of loading, defined as a percentage of the ultimate bearing capacity as q/B (q) as shown in Figure 7. Schmertmann (1969) pointed out that this method tends to seriously underestimate settlement.

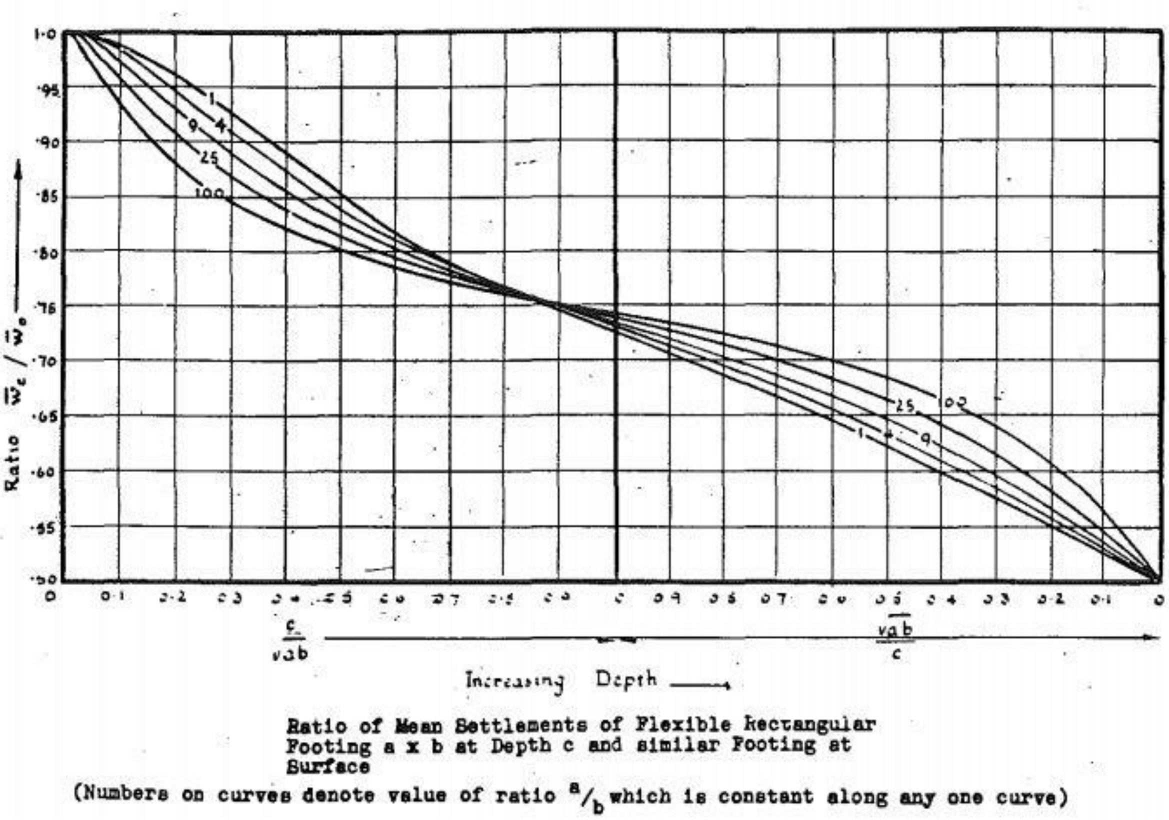

Figure 5 Fox (1948) Embedment Correction Factor.

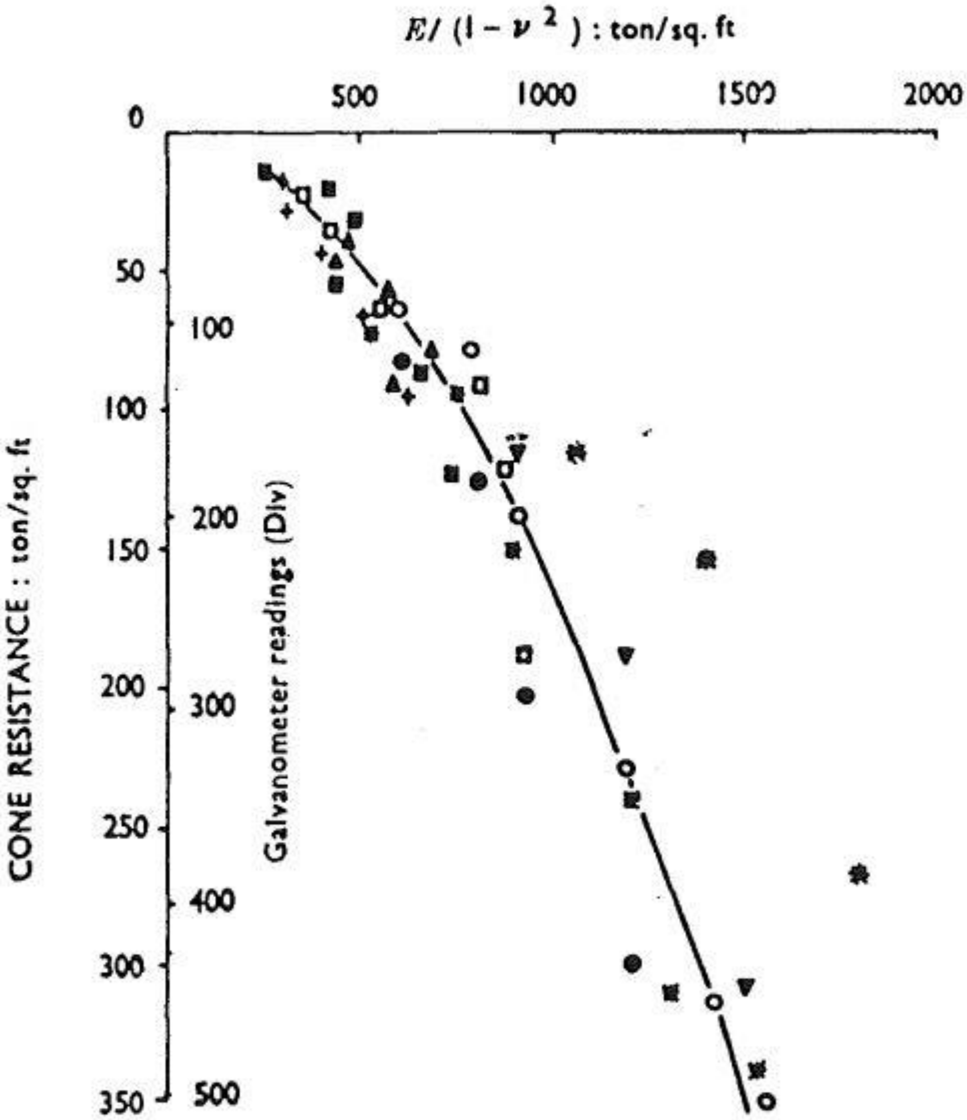

Figure 6: Thomas (1968) Elastic Modulus from CPT

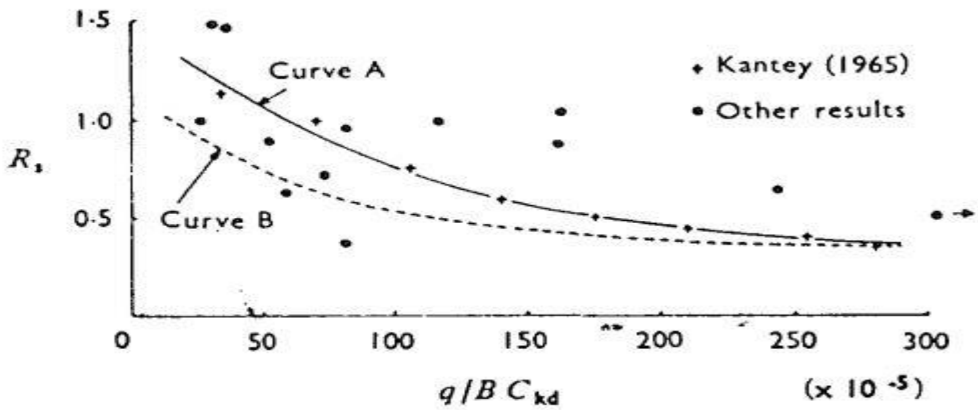

Figure 7: Settlement Ratio as a Function of Load Level (Thomas 1968).

2.4.5 D'Appolonia et al. (1970)

In an extensive study of the settlement performance of a large number of footings on sand, D'Appolonia et al. (1968) used a modification of the Terzaghi and Peck (1948, 1967) and Meyerhof (1956, 1965) methods to predict settlement.

Based on their observations, they suggested that settlement should be estimated as:

$s = [16*q/(3N_c)]*Cd$

$s = (8*q/N_c)*[B/(B+ 1)]^2*C_d$

$s = (8q/N_c)*C_d$

where:

s = settlement (in inches)

q = footing stress (in tst)

B = footing width (in ft.)

N, = corrected blowcounts

C_d = embedment correction = 1 - 0.25 (D/B)

2.4.6 Papadopoulos (1992)

Papadopoulos (1992) suggested a method of estimating the settlement of footings resting on granular soils of the elastic solution type as:

s = settlement

q = foundation stress

B = width of a rectangular foundation

E_c = constrained modulus of the soil for the appropriate stress range

f = a dimensionless factor which depends on soil stress history, geometry, loading and the relation between constrained modulus and effective stress.

According to Papadopoulos (1992) the settlement factor, f is related to the geometry of the foundation (depth and dimensions), the stress history of the soil, the foundation loading, and the relation between the constrained modulus and the effective stress, σ' as shown in Figure 4.18. The influence of history of applied stresses in soil and other miscellaneous effecting factors were statistically compiled into a factor α such as:

$\alpha = q/(y'*B)$

is indicated in Figure 8.

The constrained modulus, Es is related to the effective stress for stresses σ' = 600 kPa by a linear expression:

$Es = E_o + \lambda.\sigma'$ where:

E_0 = constrained modulus for zero effective stress

λ=the rate of E_s increase with stress.

Due to difficulty in calculating the value of gamma. The following expressions for estimating soil modulus were suggested by Papadopoulos:

$E = 2.5*q_c$ (for CPT results)

$E = 7.5 + 0.8N$ (MPa) (for SPT results)

A comparison between the settlements predicted using this method and settlement observations showed that in more than 90% of the cases the deviation of the estimated settlement from the measured settlement was ± 50%. Hence the reliability of this method remains low.

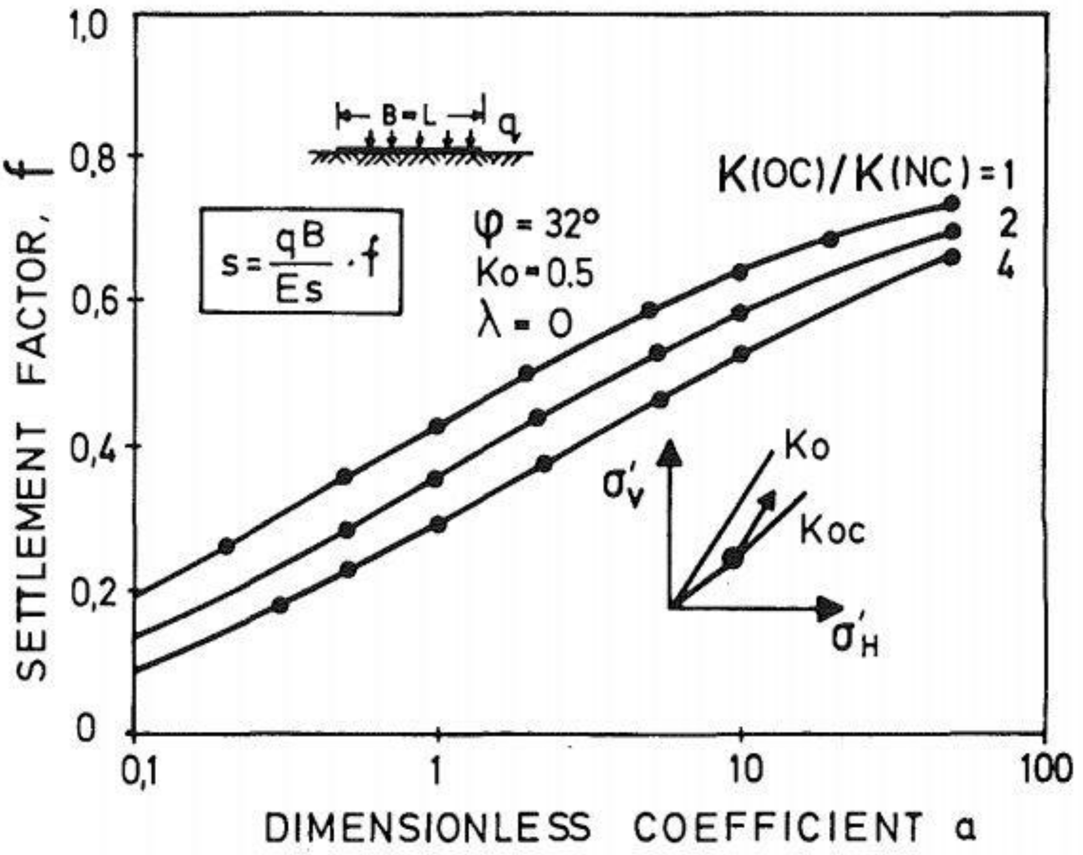

Figure 8: Papadopoulos (1992) Settlement Factor.

2.4.7 Schmertmann et al. (1978)

Schmertmann et al. (1978) modified the strain influence factor variation (2B–0.6I_z) shown in Figure 9. The revised distribution is shown in Figure 10 for use in equations

According to this, for square or circular foundation:

$I_z = 0.1$ at $z = 0$

I_z (peak) at $z = z_p = 0.5B$

$I_z = 0$ at $z = z_o = 2B$

For foundation with $L/B \geq 10$:

$I_z = 0.2$ at $z = 0$

I_z (peak) at $z = z_p = B$

$I_z = 0$ at $z = z_o = 4B$

where L = length of foundation. For L/B between 1 and 10, interpolation can be done. Also

I_z (peak) = 0.5+ 0.1*(q/σ$_o$')^0.5

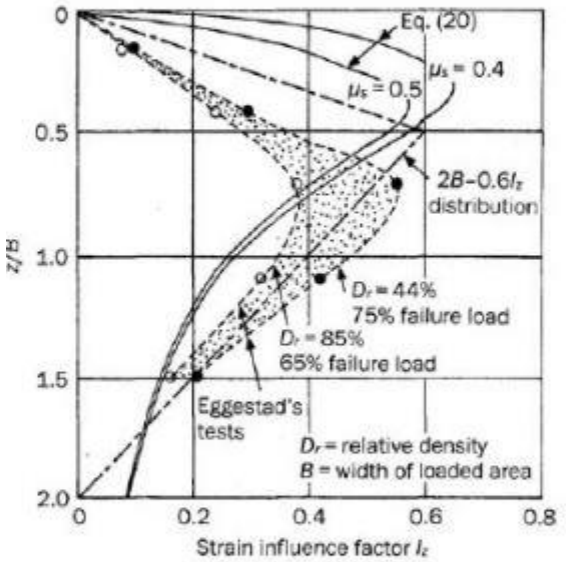

Figure 9: Theoretical and experimental distribution of vertical strain influence factor below the center of a footing

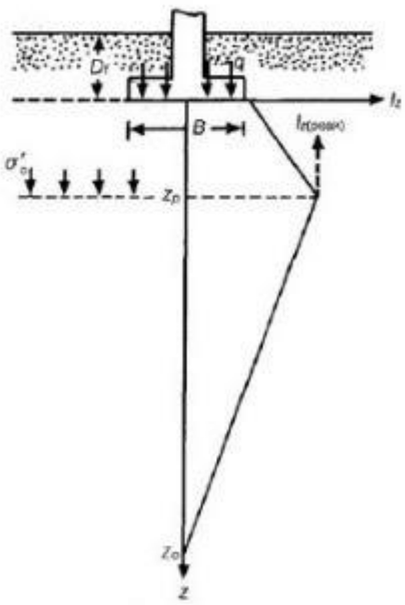

Figure 10: Revised strain influence factor diagram suggested by Schmertmann et al. (1978)

The value of σ_o' is the effective overburden pressure at a depth where I_z(peak) occurs. Salgado (2008) gave the following interpolation for Iz at z = 0, z_p, and z_o for L/B = 1 to L/B ≥ 10.

I_z(at z=0)= 0.1+ 0.0111*(L/B)^0.2

Z_p/B=0.5+ 0.0555*(L/B-1) ≤ 1

Z_o/B=2+ 0.222*(L/B-1) ≤ 4

Schmertmann et al. (1978) recommended that.

E_s=2.5*q_c (for square and circular foundations)

And

E_s = 3.5*q_c

With the modified strain-influence factor diagram,

S_e= C_1*C_2*q*∑ I_z/E_s Δz

2.4.7.1 RECENT MODIFICATIONS IN STRAIN-INFLUENCE FACTOR

DIAGRAMS

More recently some modifications have been proposed to the strain-influence factor diagram suggested by Schmertmann et al. (1978).

Modification Suggested by Terzaghi, Peck and Mesri (1996)

The modification suggested by Terzaghi et al. (1996) is shown in Figure 10.

For this case, for surface foundation condition (that is, D_f/B = 0)

I_z = 0.2 at z = 0

I_z = I_z(peak) = 0.6 at z = z_p = 0.5B

I_z = 0 at z = zo

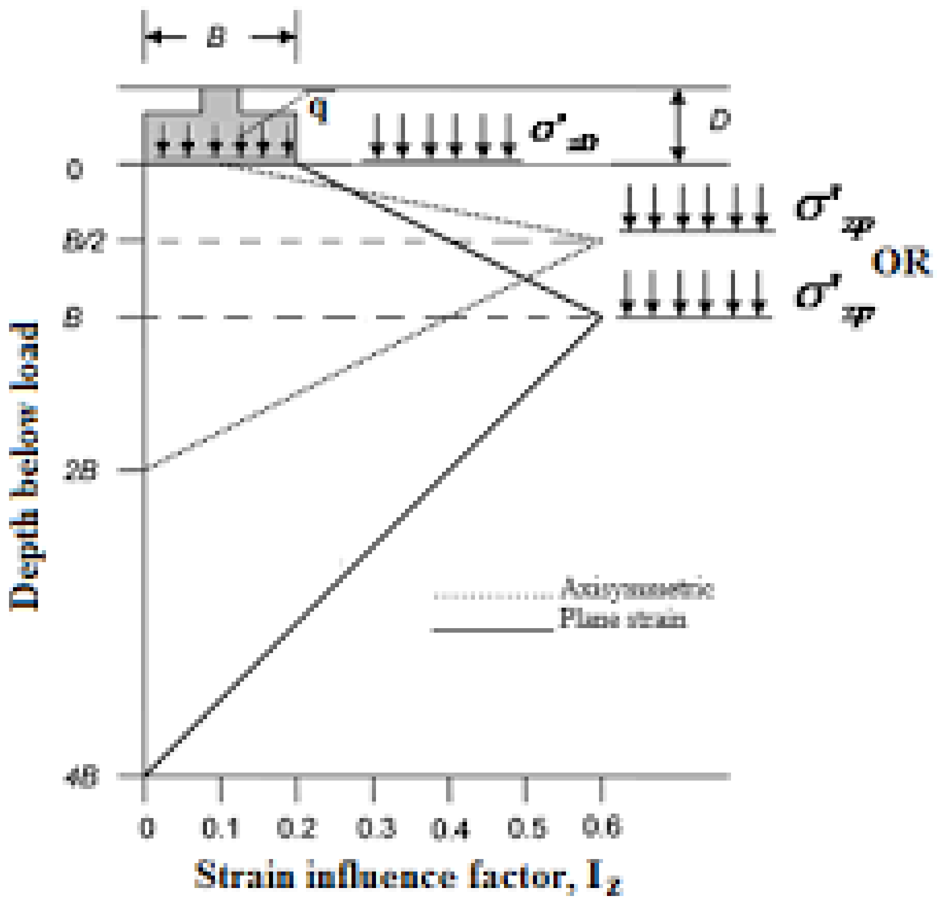

Figure 11 Strain influence diagram suggested by Terzaghi et al. (1996).

$Z_p=2[1+\log(L/B)] <= 4$

For $D_f/B > 0$, I_z should be modified to I_z'. Figure 12 shows the variation of I_z' / I_z with D_f/B.

The end of construction settlement can be estimated as

$S_e = q*\sum I_z/E_s \Delta z$

The settlement due to creep can be calculated as

$S_{(creep)} = (0.1/q_c)*Zp*\log(t(days)/1day)$

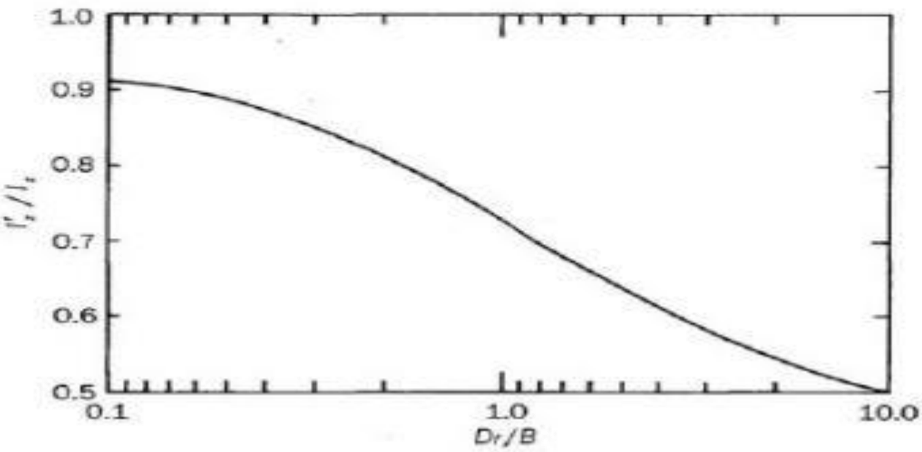

Figure 12 Variation of I_z'/ I_z with D_f/B (after Terzaghi et al. 1996)

Where

q_c = weighted mean value of measured q_c values of sub layers between z = 0 and z = z_o (MN/m 2).

It has also been suggested that

$E_{s(L/B)}/E_{s(L/B=1)} = 1 + 0.4*\log(L/B) <= 1.4$

where

$E_{s(L/ B)}$ = qc * 3.5

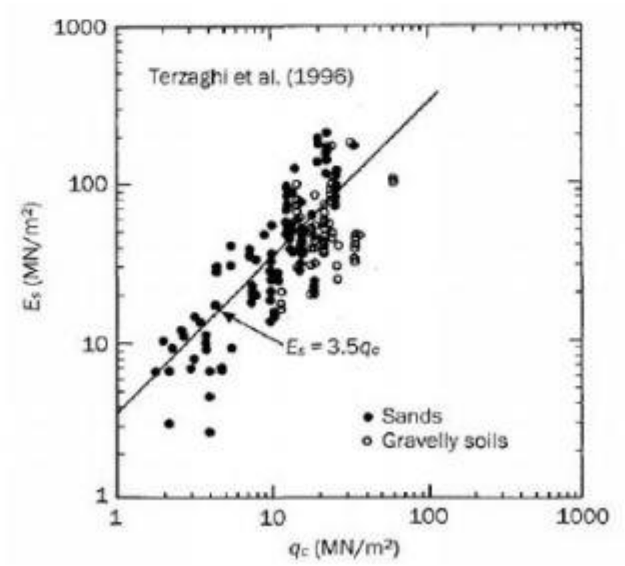

Figure 13 Correlation between E_s and q_c for square and circularly loaded areas [adapted from Terzaghi et al. (1996)].

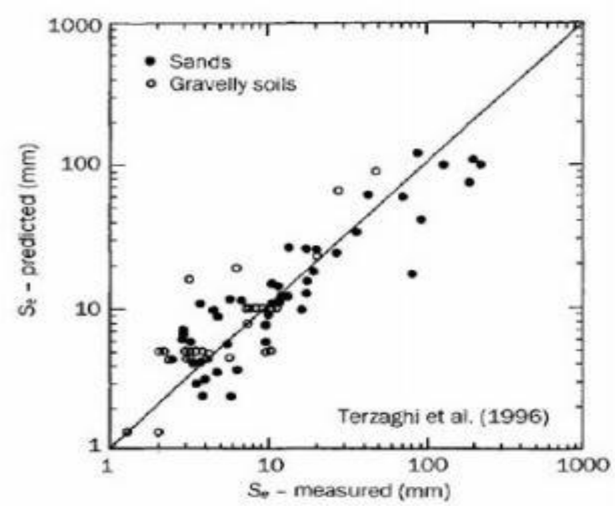

Figure 14 Comparison of end of construct ion predicted and measured S_e of foundations on sand and gravel

2.5 Standard Penetration Test

Numerous suggestions have been made to use the SPT for estimating the elastic modulus of granular soils (e.g., Schultze and Menzenbach 1961; Schultze and Melzer 1965, etc). Most of these correlations have the form of:

$E=a*(N+b)$

where:

E =soil modulus

N = blow counts

a and b = constants (empirical factors)

Alternatively, other forms have been used. In addition to the correlations presented in Tables 4.2 and 4.3, a number of other suggestions have been made. These are summarized in Table 4.4.

Other attempts have been made to correlate the results of the SPT to the constrained modulus of the soil (M) as a function of overburden stress (e.g., Schultze and Melzer 1965). D'Appolonia et al. (1970) suggested correlations between M and SPT blow count N recognizing the influence of stress history. These correlations are presented in the next section of this report and are subsequently shown in Figure 5.7.

Since the constrained modulus, M, is related to the elastic Young's modulus, E, as:

$M = [E (1-\mu)]*I*[(l+\mu) (1-2\mu)]$

an estimate of Poisson's ratio is required to estimate E from M. For most granular soils in drained loading conditions, the constrained modulus probably varies in the range of $1.2E_s$ to $1.5E_s$.

Unfortunately, the realization must be made that there is considerable scatter in suggested correlations between E or M and SPT blowcount N. This should in fact be not altogether unexpected since there is a considerable scatter in SPT results, even at a single site, because of large variations in test procedures that may occur. Additionally, since the source of correlations between modulus and N is highly variable and includes laboratory tests on reconstituted samples, results of field plate tests and results of settlement observations from full scale structures, the correlations will have implicit variability just because of differences in assumptions made. Additionally; since the modulus is strain level dependent, the correlations include comparisons at a range of strain levels.

The various relations between Standard Penetration Test blow counts and soil modulus are shown in the following tables:

Table 4.2 Estimates of Soil Modulus from SPT and CPT (after Bowles 1988).

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $E_s = (15000 \text{ to } 22000) \ln N$ $E_s^{\S} = (3500 \text{ to } 50000) \log N$	$E_s = 2 \text{ to } 4q_c$ $E_s^{\dagger} = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	
Sand (overconsolidated)	$E_s^{\ddagger} = 18000 + 750N$ $E_{s(OCR)} = E_{s(nc)}(OCR)^{1/2}$	$E_s = 6 \text{ to } 30q_c$
Gravelly sand and gravel	$E_s = 1200(N + 6)$ $E_s = 600(N + 6) \quad N \leq 15$ $E_s = 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = 3 \text{ to } 6q_c$
Silty sand	$E_s = 300(N + 6)$	$E_s = 1 \text{ to } 2q_c$
Soft clay	-------	$E_s = 3 \text{ to } 8q_c$

Notes:

E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf.

N values should be estimated as N_{55} and not N_{70}

† Vesic' (1970)

‡ Author's equation from plot of D'Appolonia et al. (1970).

§ USSR (and may not be standard blowcount N).

Figure 15 estimate of Soil Modulus from SPT & CPT - comparison

Table 4.3 Soil Modulus from Standard Penetration Resistance (modified from Mitchell and Gardner 1975)

Reference	Relationship	Soil Types	Basis	Remarks
Schultze and Melzer (1965)	$E_s = Na_{0.522}$ kg/cm^2	Dry sand	Penetration tests in field and in test shaft. Compressibility based on e, $e_{max.}$ and $e_{min.}$ (Schulzte and Moussa, 1961)	Correlation Coefficient = 0.730 for 77 tests
Webb (1969)	$E_s = 5(N+15)$ tons/ft^2 $E_s = 10/3(N+5)$ tons/ft^2	Sand Clayey Sand	Screw Plate Tests	Below water table
Farrent (1963)	$E_s=7.5(1-m^2)N$ tons/ft^2 m=Poisson's ratio	Sand	Terzaghi and Peck loading settlement curves	
Begemann (1974)	$E_s= 40 + C(N-6)$ for N>15 kg/cm^2 $E_s= C(N+6)$ for N<15 kg/cm^2 C= 3(silt with sand) to 12(gravel with sand)	Silt with sand to gravel with sand		Used in Greece
Trofimenkov (1974)	$E_s=$ (350 to 500) logN (kg/cm^2)	Sand		USSR practice

Notes:
N is penetration resistance in blows per 30 cm. (blows/ft.)
E_s = soil modulus
e = void ratio

Figure 16 Elastic Modulus for SPT Data

Table 4.4 Other Expressions for Soil Modulus from SPT

Equation	Soil Type	Reference
$E_s^{4/3} = (44N)$ (tsf)	Sand	Chaplin (1963)
$E_s = 48 + 4N$ (tsf)	Sand	Webb (1969)
$E_s = 7(N)^{0.5}$ (MPa)	Sand	Denver (1982)
$E_s = 3.5N$ to $40N$ (MPa)	Sand	Clayton et al. (1980)
$E_s = 7.5 + 0.8N$ (MPa)	Sand	Papadopoulos (1982)

Figure 17 Elastic Modulus for SPT (continued)

2.6 Cone Penetration Test

The modulus of soils has also been correlated to the results of tip resistance measurements (q_c) obtained from the CPT test. Most early correlations between q_c and E were of the general form:

$E = \alpha * q_c$

Where:

α = (empirical factor)

Since the performance of the CPT involves considerably less variation than the SPT and is prone to less errors in execution, it is suspected that the primary source of scatter indicated in Tables 4.5 and 4.6, and for that matter in Tables 4.3 and 4.4 is the soil itself and not the test method. Variations in soil mineral composition, initial void ratio, grain-size distribution, stress history, etc., as well as differences in initial effective stress level (octahedral) and change in stress during loading result in differences in the "operational" or "apparent" modulus of elasticity producing deformation. These factors, combined with the stress and strain level dependency of a "local" soil modulus for a given application all affect the reported correlations between the so-called soil modulus and in situ test results.

In recent years, the use of large calibration chamber tests on reconstituted samples of sands have helped to elucidate certain key variables that can influence correlations between modulus and CPT results. For both normally consolidated and over-consolidated sands, the ratio of constrained modulus, M, to CPT tip resistance, q" decreases with increasing relative density, all other factors being equal. Kulhawy and Mayne (1990) have summarized a number of available chamber test results which are shown in Figure 4.20.

Jamiolkowski et al. (1988) have also noted that for a given sand the ratio M/q, and E/q, is clearly related to stress history and current stress level for sands at different relative density.

Because of the wide range in correlation constants that may exist between the results of in situ penetration tests and a singular value of soil modulus it is doubtful that any method which relies on these techniques for the accuracy of settlement estimates will be of much value, other than that created by local correlations developed from full scale field observations of performance. However, there are several techniques that have recently been suggested that account for nonlinearity in modulus and show distinctly strong correlations between observed "operational" modulus and the results of SPT or CPT tests. These methods are discussed further in subsequent sections of this report.

Table 4.2 Estimates of Soil Modulus from SPT and CPT (after Bowles 1988).

Soil	SPT	CPT
Sand (normally consolidated)	$E_s = 500(N + 15)$ $E_s = (15000 \text{ to } 22000) \ln N$ $E_s^{\S} = (3500 \text{ to } 50000) \log N$	$E_s = 2 \text{ to } 4q_c$ $E_s^{\dagger} = (1 + D_r^2)q_c$
Sand (saturated)	$E_s = 250(N + 15)$	
Sand (overconsolidated)	$E_s^{\ddagger} = 18000 + 750N$ $E_{s(OCR)} = E_{s(nc)}(OCR)^{1/2}$	$E_s = 6 \text{ to } 30q_c$
Gravelly sand and gravel	$E_s = 1200(N + 6)$ $E_s = 600(N + 6) \quad N \leq 15$ $E_s = 600(N + 6) + 2000 \quad N > 15$	
Clayey sand	$E_s = 320(N + 15)$	$E_s = 3 \text{ to } 6q_c$
Silty sand	$E_s = 300(N + 6)$	$E_s = 1 \text{ to } 2q_c$
Soft clay	-------	$E_s = 3 \text{ to } 8q_c$

Notes:
E_s in kPa for SPT and units of q_c for CPT; divide kPa by 50 to obtain ksf.
N values should be estimated as N_{55} and not N_{70}
† Vesic' (1970)
‡ Author's equation from plot of D'Appolonia et al. (1970).
§ USSR (and may not be standard blowcount N).

Figure 18 Modulus from SPT and CPT (after Bowles 1988)

Table 4.5 Soil Modulus from Cone Penetration Test (modified from Mitchell and Gardner 1975)

Reference	Relationship	Soil Types	Remarks
Buisman (1940)	$E_s = 1.5q_c$	Sands	Over predicts settlements by a factor of about two
Trofimenkov (1964)	$E_s = 2.5\ q_s$ $E_s = 100 + 5\ q_s$	Sand	Lower limit Average
De Beer (1967)	$E_s = 1.5\ q_s$	Sand	Overpredicts settlements by a factor of two
Schultze and Melzer (1965)	$E_s = (1/m_v)v\sigma^{0.522}$ where $v = 301.1 \log q_s - 382.3p_o + 60.3 \pm 50.3$	Dry Sand	Based of field and lab penetration tests compressibility based on e, e_{max} and e_{min} Correlation coefficient = 0.778 for 90 tests valid for p_o = 0 to 0.8 kg/cm²
Bachelier and Parez (1965)	$E_s = \alpha q_s$		
	α = 0.8-0.9	Sand	
	α = 1.3-1.9	Silty sand	
	α = 3.8-5.7	Clayey-sand	
	α = 7.7	Soft clay	
De Beer (1967)	$A = C(A_{oed}/C_{oed})$	Over-consolidated sand	C from field tests A_{oed} and C_{oed} from lab oedometer tests $C_{oed} = 2.3(1+e)/C_C$ $A_{oed} = 2.3(1+e)/C_S$
Thomas (1968)	$E_s = \alpha q_s$ $\alpha = 3 - 12$	3 sands	Based on penetration and compression tests in large chambers Lower values of a at higher values of q_c; attributed to grain crushing

Figure 19 Soil Modulus from CPT (from Mitchell and Gardner)

Table 4.5 Cont'd

Reference	Relationship	Soil Types	Remarks
Webb (1969)	$E_s = 2.5(q_c+30)$, tsf	Sand below water table	Based on screw plate tests Correlated well with settlement of oil tanks
	$E_s = 1.67(q_s+15)$, tsf	Clayey sand below water table	
Meigh and Corbett (1969)	$E_s = 1/m_v = \alpha q_c$	Soft silty clay	
Vesic (1970)	$E_s = 2(1+D_R^2)q_c$ D_R = relative density	Sand	Based on pile load tests and assumptions concerning state of stress
Schmertmann (1970)	$E_s = 2q_s$	Sand	Based on screw plate tests $\Delta\sigma$ = 2 tsf

Table 4.5 Cont'd

Reference	Relationship	Soil Types	Remarks
Bogdanovic (1973)	$E_s = \alpha q_c$		Based on analysis of silo settlements over a period of 10 years
	$q_c > 40$ kg/cm² $\alpha = 1.5$	Sands, sandy gravels	
	$20 < q_c < 40$ $\alpha = 1.5 - 1.8$	Silty saturated sands	
	$10 < q_c < 20$ $\alpha = 1.8 - 2.5$ $5 < q_c < 10$ $\alpha = 2.5 - 3.0$	Clayey silts with silty sand and silty saturated sands with silt	
Schmertmann et al. (1978)	$E_s = 2.5\ q_c$	NC sands	L/B=1 to 2 axisymmetric
	$E_s = 3.5\ q_c$	NC sands	L/B = ≥ 10 plane strain
DeBeer (1974b)	$C > 3/2(q_c/\sigma_0)$	NC sands	Belgian practice
	$A > \varepsilon\ 3/2(q_c/\sigma_0)$	OC sands	$3 < \varepsilon < 10$, Belgian practice
	$E_s = 1.6\ q_c - 8$	Sand	Bulgarian practice
	$E_s = 1.5q_c$, $q_c > 30$ kg/cm² $E_s = 3q_c$, $q_c < 30$ kg/cm²	Sand	Greek practice
	$E_s > 3/2\ q_c$ or $E_s = 2\ q_c$	Sand	Italian practice
	$E_s = 1.9\ q_c$ $E_s = 2.5(q_c + 3200)$, kPa $E_s = 1.67(q_c + 1600)$, kPa	Sand Fine to Medium sand Clayey sands, PI < 15%	South African practice
	$E_s = \alpha q_c$, $1.5 < \alpha < 2$	Sands	U.K. practice
Trofimenkov (1974)	$E_s = 3\ q_c$	Sands	U.S.S.R. practice
	$E_s = 7\ q_c$	Clays	
Alperstein and Leifer (1975)	$E_s = (11 - 22)q_c$	Overconsolidated sand	E_s determined by lab tests on reconstituted samples of sand
Dahlberg (1974)	$E_s = \alpha q_c$ $1 < \alpha < 4$	NC and OC sand	E_s back-calculated from screw plate settlement using Buisman-DeBeer and Schmertmann methods; α increases with increasing q_c

Figure 20 Different Correlations for Calculating Modulus (various)

Table 4.6 Other Expressions for Soil Modulus from CPT

Expression	Soil	Reference
$E_s = 11q_c$	Sand	Lambrechts and Leonards (1978)
$E_s = 2.5q_c$	Sand	Roth et al. (1982)
$E_s = \alpha q_c$ $\alpha = 1.7$ to 4.4 average = 2.5	Med. sand	Das Neves (1982)
$E_s = 8(q_c)^{0.5}$, q_c in MPa	Sand	Denver (1982)
$E_s = 2.9q_c$	Sand	Garga and Quin (1974)

Figure 21 Different Correlations for Calculating Modulus (continued)

CHAPTER 3

METHODOLODY

3.1 Design Chart Methodology

3.1.1 Input Data

In input data you have to define the parameters

Non- cohesive soils:

- Shape
- Poisson's Ratio
- L/B ratio in the form of Length and Breadth
- Footing Depth "D"
- Depth of Water Table D_w
- Relative Density D_r
- Unit weight γ
- SPT N value
- CPT q_c value
- OCR/NC
- Soil type

The option to identify multiple layers has been included to accommodate changes in Soil Parameters (layer of Silty Sand over Medium Gravel for example). The depth to incompressible strata will be taken as the depth to the bottom of last layer identified in the layer Input Window. In the case of the thickness of the first layer exceeding the width of foundation, it is recommended to divide the first layer into 2 or 3 sublayers to ensure correct computations.

3.1.2 Bearing Capacity Calculations

Some data given in input were utilized to get bearing capacity for that particular soil. Two methods were used.

- Meyerhof
- Teng

Terzaghi used limit equilibrium method to compute bearing capacity. It was the first limit equilibrium method which was accepted by worldwide. Later on Meyerhof (1963), Brinch Hansen (1970) and then Vesic's formulation appeared in 1973 that is based on experimental results redefined the method.

3.1.3 Settlement Calculations

3.1.3.1 Non-Cohesive Soil

Schmertmann's method is preferred over other methods that are purely empirical, because it is based on physical model of settlement which has been calibrated using empirical data. Sivakugan and Johnson (2004) made a comparison between different methods: Schmertmann et al. give relatively more accuracy as compared to that of "Burland and Burbidge" (empirical method). With improvement in defining Influence factors, accuracy of Schmertmann's method is improved.

3.1.3.2 Using Automated Workbook

Automated Microsoft excel sheets were made for ease of use

- Input required data
- Click "Compute Settlement" to compute
- Click "compute Settlement Controlled Q" to compute back calculated Bearing capacity for 1 inch of settlement

3.1.3.3 Correlations

We have friction angles of general soils, from that we can calculate SPT N blowcount using correlation. Other correlation used was to inter-convert SPT N blowcount data and CPT (q_c) data that inputs the mean grain size of soil D_{50}.

The correlation used are:

- SPT N Value – Friction Angle of Soil by CARTER & BENTLEY 1991
- Kulhawy & Mayne Correlation for Mean Grain Size and CPT/SPT

3.1.3.4 Calculation of Elastic Modulus

As determining the soil elastic modulus is the most important factor in calculating the settlement. In the case of absence of soil test data the following procedure could be done.

First determining the friction angle ϕ of the soil, after which the SPT (N value) could be obtained by using correlation between friction angel ϕ and SPT blow count (N) shown in figure 15

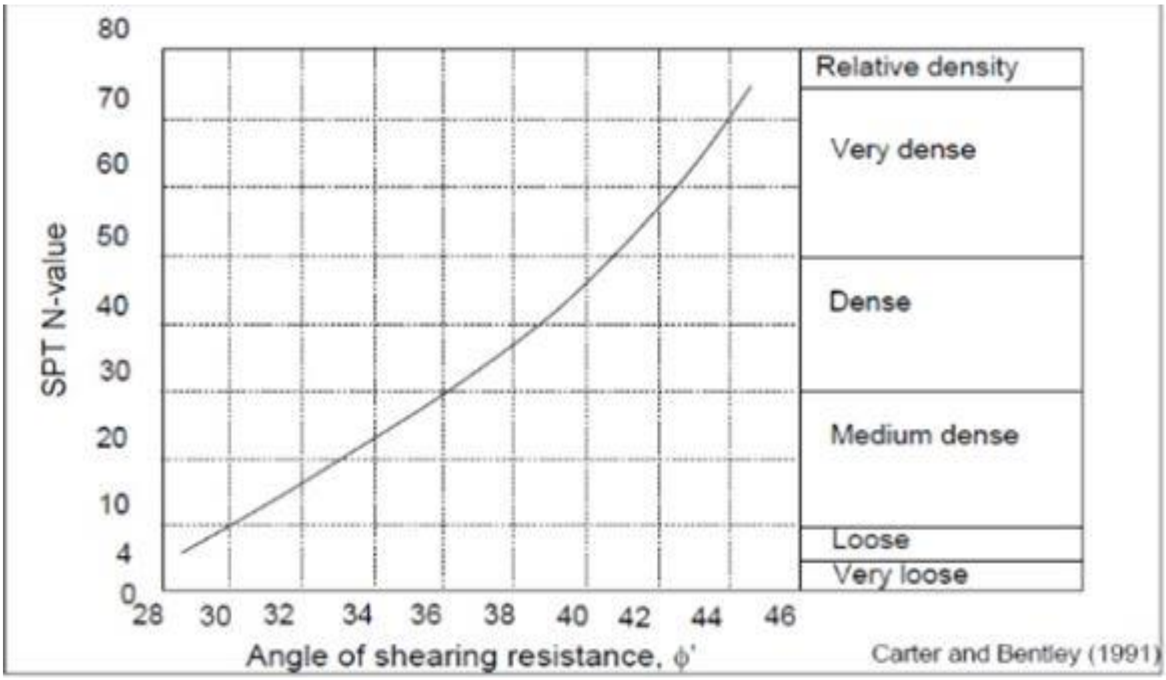

Figure 22 Correlation of N and phi ϕ

Now by using the correlation "Kulhawy & Mayne Correlation" for Mean Grain

Size and CPT, SPT Kulhawy & Mayne correlation,

$$q_c/N = \text{func}\ (D_{50})$$

$$q_c = N * \text{func}\ (D_{50})$$

by using this equation q_c is obtained in terms of D_{50} which is then can be used in the settlement sheets for obtaining E in terms of D_{50} and then E can be easily calculated.

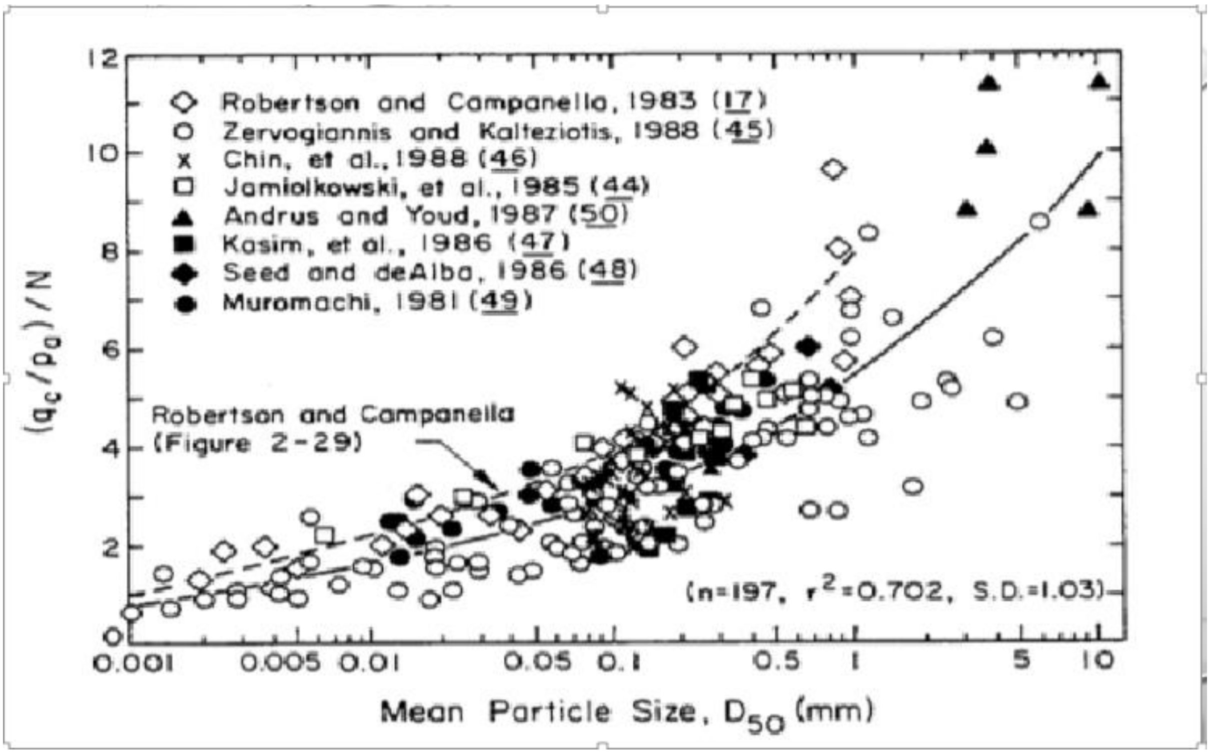

Figure 23 Kulhawy & Mayne (1991) Correlation

3.1.3.5 Friction Angles of General Granular Soils

Coarse Gravel	40
Medium Gravel	36
Fine Gravel	30
Coarse Sand	36-38

Medium Sand	32
Fine Sand	28

Swiss Standard SN 670 010b, Characteristic Coefficients of soils, Association of Swiss Road and Traffic Engineers.

3.1.4 Correlation of Elastic Modulus and Mean Grain Size

1. The first pre requisite for calculation of elastic modulus from the mean grain size was availability of ranges of mean grain sizes for different kind of granular soils.
2. The ranges of different mean grain sizes were given in Kulhawy & Mayne (1991).
3. For each soil type, the typical value of angle of friction were obtained from the data above.
4. The graph between SPT blowcount and angle of friction in Carter and Bentley (1991) was digitized and it the subsequent equation was used the angle of friction from (3) to obtain designated values of blowcounts.
5. The digitized graph of Kulhawy & Mayne yielded an equation of $\frac{q_c}{N}$=5.44*$D_{50}^{0.2778}$
6. Inputting values of SPT blowcount from (4) into (5), the values of q_c were obtained in terms of mean grain size.
7. Inputting the results from (6) into our spreadsheets correlations for computing modulus of elasticity we obtained ranges of modulus of elasticity including the maximum, the minimum and the average modulus in terms of mean grain size.
8. The values of mean grain size from (1) were input into the results from (7) to obtain values of modulus of elasticity.
9. The final step including plotting the modulus of elasticity ranges obtained in (8) on a graph with the variable W = $D_{50}^{0.2778}$ (for ease of computations) on the X axis.
10. The line of best was plotted and the equation was calculated using MS EXCEL's Line of best fit tool.

3.1.5 The Term W

For ease of usage and computations, this project identifies a new variable referred to as "W" and takes it to be equal to $D_{50}^{0.2778}$ .The reason was to create ease for calculation of a line of best fit, since the Kulhawy and Mayne's correlation provides an expression for q_c/N in terms of $D_{50}^{0.2778}$. The whole of the subsequent calculation steps involves expressions in terms of $D_{50}^{0.2778}$. Hence, identifying a new variable makes it easier for the procedure to proceed.

3.2 Additional Feature

Sometimes the soil test data in not available on such cases correlations have been developed to estimate the soil test data and to achieve the required results. The correlations used consists of two main things which have to be used for calculations

- Gradation Curve

3.2.1 Gradation Curve

Gradation curve is used when the soil tests data in not available for some reason. Gradation curve give the mean grain size from which the elastic modulus of soil could be calculated by using different relations.

3.3 Soil test data

The soil test data comprises of soil penetration test (no of blow counts) and cone penetration test (q_c value) has to be known for calculating or determining the settlement. But if the tests are not yet performed or the test are out of the budget the correlations could be used for estimation of N (no. of blow counts per foot) and cone penetration test value (q_c).

3.4 Output of Spreadsheet

The output of the project is in the following forms

- Elastic modulus
- Settlement
- Elastic Modulus of Soil from Mean Grain Size D_{50}

3.4.1 Settlement

In case of soil settlement by using different methods for calculating the outputs are

- Maximum settlement
- Minimum settlement
- Average settlement

The major output of this project is comparison of different methods in terms of maximum settlement, minimum settlement and average settlement which gives better understanding of settlement and shows which method gives more reasonable result.

3.4.2 Elastic Modulus

The elastic modulus of soil is calculated by using different correlations are as follows

- Maximum elastic modulus
- Minimum elastic modulus
- Average elastic modulus

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 Design Charts

The correlation between E and D_{50} for different kinds of soil types is given below

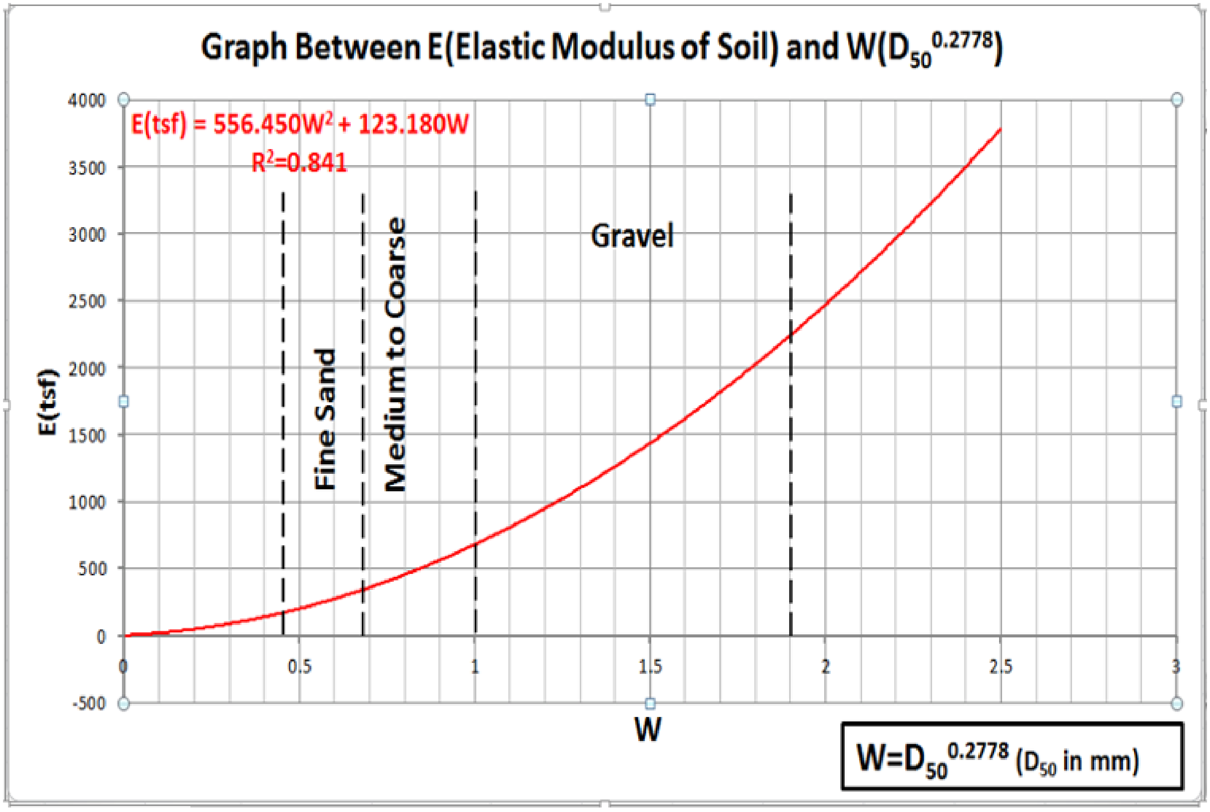

Figure 24 Proposed Relation between W and Elastic Modulus in tsf

4.2 Settlement Spread Sheets

4.2.1 Home Page

The home page of the spread sheets we make shows the

- Inputs that are given by the user
- Footing parameters
- Loading data
- Settlement calculation data
- Soil modulus data
- Layer data (field testing).

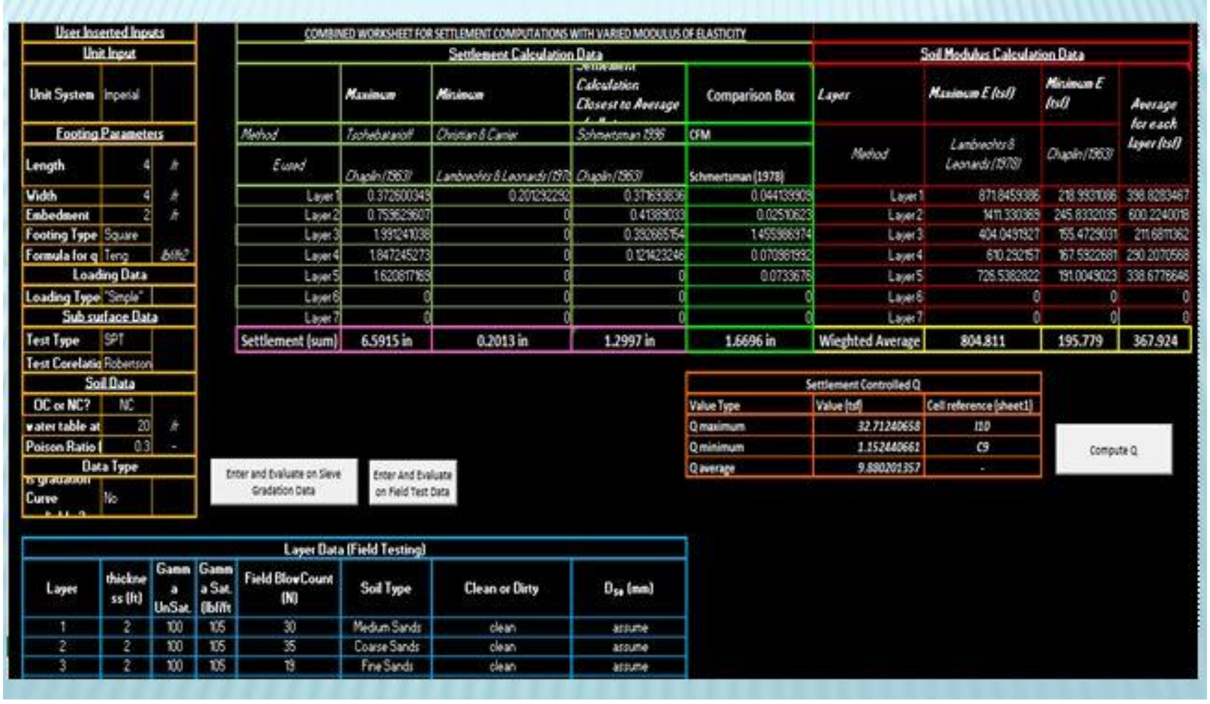

Figure 25 General Overview of Home page of Settlement Sheet

4.2.1.1 Input Section

Input section comprises of

- unit input section
- footing parameters
- loading data
- sub- surface data
- soil data, data type.

User Inserted Inputs		
Unit Input		
Unit System	Imperial	
Footing Parameters		
Length	4	ft
Width	4	ft
Embedment	2	ft
Footing Type	Square	
Formula for q	Teng	lbf/ft2
Loading Data		
Loading Type	"Simple"	
Sub surface Data		
Test Type	SPT	
Test Corelation	Robertson	
Soil Data		
OC or NC?	NC	
water table at:	20	ft
Poison Ratio (v)	0.3	-
Data Type		
Is gradation Curve avaliable ?	No	

Figure 26 Data Input Section - Foundation Inputs

Layer Data (Field Testing)							
Layer	thickness (ft)	Gamma UnSat. (lbf/ft3)	Gamma Sat. (lbf/ft3)	Field BlowCount (N)	Soil Type	Clean or Dirty	D_{50} (mm)
1	2	100	105	30	Medium Sands	clean	assume
2	2	100	105	35	Coarse Sands	clean	assume
3	2	100	105	19	Fine Sands	clean	assume
4	2	100	105	21	Medium Sands	clean	assume
5	2	100	105	25	Medium Sands	clean	assume
6							
7							

Figure 27 Data Input Section - Soil Layer Inputs

4.2.1.1.1 Unit Type

The unit types selected for the spread sheets are

- Imperial Units.
- SI Units

4.2.1.1.2 Footing Parameters

The footing parameters consist of

- Length
- Width
- Embedment
- footing type
- formula for q

4.2.1.1.3 Testing Data

The data of the soil for which the soil test data has to calculate by using correlations has to be put in this section.

4.2.1.1.4 Soil Data

The soil data comprises of the following

- OC or NC
- Water table depth
- Poisson’s ratio

4.2.1.1.5 Gradation Curve Data

The gradation curve helps us to estimate the mean grain size which further gives the friction angle of the soil.

4.2.1.2 Output Section

Output section comprises of

- Maximum settlement
- Minimum Settlement
- Average Settlement
- Comparison of Settlement
- Elastic Modulus Computations (max, min)

COMBINED WORKSHEET FOR SETTLEMENT COMPUTATIONS WITH VARIED MODULUS OF ELASTICITY

Settlement Calculation Data

	Maximum	Minimum	Settlement Calculation Closest to Average of all stages	Comparison Box
Method	Tschebatarioff	Christian & Carrier	Schmertsman 1996	CFM
E used	Chaplin (1963)	Lambrechts & Leonards (1978)	Chaplin (1963)	Schmertsman (1978)
Layer 1	0.372600349	0.201292292	0.371693836	0.044139909
Layer 2	0.759629607	0	0.41389033	0.02510623
Layer 3	1.991241038	0	0.392665154	1.455986974
Layer 4	1.847245273	0	0.121423246	0.070981992
Layer 5	1.620817169	0	0	0.0733676
Layer 6	0	0	0	0
Layer 7	0	0	0	0
Settlement (sum)	6.5915 in	0.2013 in	1.2997 in	1.6696 in

Figure 28 Output Section - Settlement Maximum, Minimum and Comparison Box

Soil Modulus Calculation Data

Layer	Maximum E (tsf)	Minimum E (tsf)	Average for each layer (tsf)
Method	Lambrechts & Leonards (1978)	Chaplin (1963)	
Layer 1	435.9229693	130.2140814	216.6898789
Layer 2	524.2084227	150.2654103	245.4791784
Layer 3	531.6436746	191.0049023	267.8544275
Layer 4	871.8459386	218.9931086	398.8283467
Layer 5	1017.153595	245.8332035	458.5991897
Layer 6	0	0	0
Layer 7	0	0	0
Wieghted Average	676.155	180.602	317.490

Figure 29 Output Section - Modulus of Elasticity

CHAPTER 5

VERIFICATION OF RESULTS

5.1 Verification using Settle 3D

Multiple Examples were solved

- Results compared with average settlement from our sheet.
- Values of Modulus calculated using our chart was compared with the values suggested in SETTLE 3d

5.1.1 Example1: SETTLE – 3D (Settlement)

We solved the following problem on settle-3D

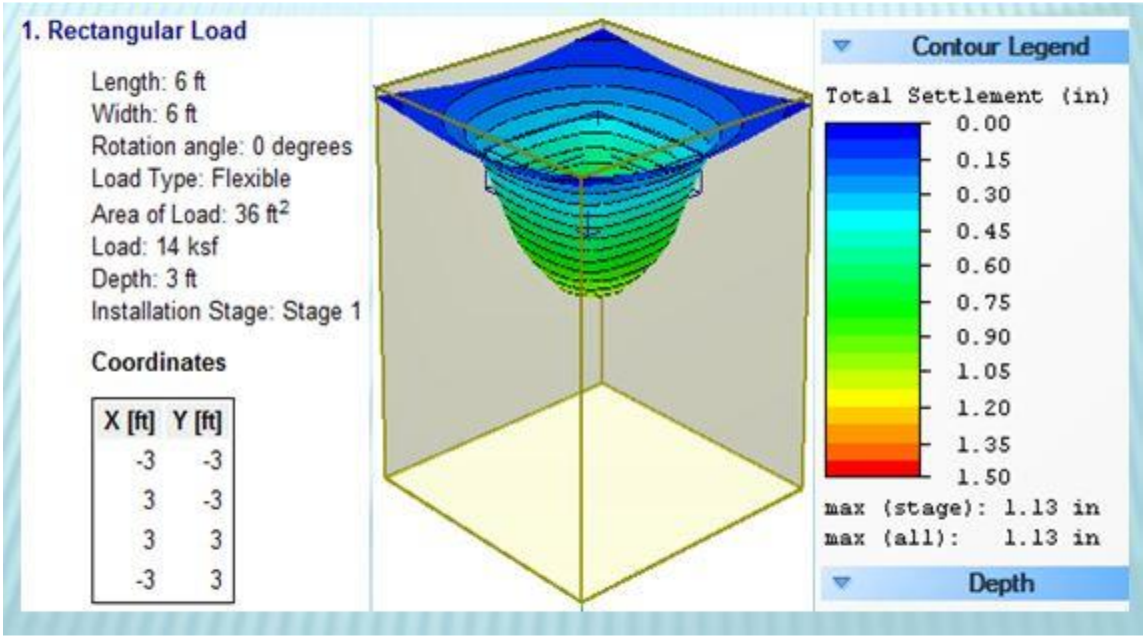

Figure 30 SETTLE 3d Example Input Data & Settlement Calculated

5.1.2 Example 1: Spread Sheet (Settlement)

The same data was input into our spreadsheet

User Inserted Inputs		
Unit Input		
Unit System	Imperial	
Footing Parameters		
Length	6	ft
Width	6	ft
Embedment	3	ft
Footing Type	Square	
Formula for q	(User Input) Q	lbf/ft2
Loading Data		
Footing pressure	7	tsf
Sub surface Data		
Test Type	SPT	
Test Corelation	Robertson	
Soil Data		
OC or NC?	NC	
water table at:	10	ft
Poison Ratio (v)	0.3	-

COMBINED WORKSHEET FOR SETTLEMENT COMPUTATIONS WITH VARIED MODULUS			
Settlement Calculation Data			
	Maximum	*Minimum*	*Settlement Calculation Closest to Average of all stages*
Method	*Tschebatarioff*	*Christian & Carrier*	*Schmertsman 1970*
E used	*Chaplin (1963)*	*Lambrechts & Leonards (1978)*	*Bogdanovi*
Layer 1	0.297939582	0.234398083	0.117055543
Layer 2	0.638694985	0	0.21693234
Layer 3	1.474600784	0	0.58935134
Layer 4	2.085862337	0	0.376249961
Layer 5	1.830185487	0	0.105349989
Layer 6	0	0	0
Layer 7	0	0	0
Settlement (sum)	**6.3273 in**	**0.2344 in**	**1.4049 in**

Figure 31 Calculated Settlement on Spreadsheet for the example data

5.1.3 Results of Example 1 (Settlement)

Settlement from Settle 3D:

- Settlement = 1.13 in

Settlement from our sheet:

- Settlement = 1.40 in

So, answers generated by our automated workbook are close to the answers yielded by Settle-3D.

5.2 Examples: Elastic modulus and mean grain size relation

Verified from:

- General ranges of Elastic modulus values known

- Calculating Settlement from our relation of E and D_{50} and comparing with average settlement on other Es (Elastic Modulus)

5.2.1 Verification of E & D_{50} relation

5.2.1.1 Example 1

Spreadsheet:

D_{50} = 0.5mm

E = 480 tsf

Settlement = 0.84 in

Settle 3D:

E = 480 tsf

Settlement = 0.86 in

5.2.1.2 Example 2

Spreadsheet:

D_{50} = 2mm

E = 970 tsf

Settlement = 0.7 in

Settle 3D:

E = 970 tsf

Settlement = 0.45 in

LIMITATIONS AND RECOMMENDATIONS

The following limitation were encountered in this project

- Elastoplastic Behavior of Soil.
- Non-Homogeneity of Soil.
- More settlement methods can be added.
- More Elastic Modulus relations can be added and checked.
- Our correlation can be tested from field tests or field data.
- Our correlation can be expanded to other soil types also if field data of other soil types is known that will need extensive experimentation.

REFERENCES

Settlement of Shallow foundations on Granular Soils by Alan J. Lutenegger et al 1995

Elastic Settlement of Shallow foundations on granular soil: A critical review, by DAS

Settlement of Shallow foundations on granular soil: An Overview, by Sivkugan 2016

Estimated and measured settlement of Shallow foundation supporting bridge substructure, by Bashar Tarawneh et al 2013

Correlation of SPT and CPT case study from Tekirdag (Turkey)

Observed and Computed Settlements of Two Shallow Foundation on Sands by Michele Maugeri et al.

Engineering Properties of Materials (Geologic and Otherwise) GEOL 615

SPT-CPT CORRELATIONS By P. K. Robertson,1 R. G. Campanella / M. ASCE, and A. Wightman3

Manual for estimating soil properties for Foundation Design by Kulhawy & Mayne 1990

Swiss Standard SN 670 010b, Characteristic Coefficients of soils, Association of Swiss Road and Traffic Engineers.