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NUMERICAL MODELLING OF SHALLOW FOUNDATIONS IN GRANULAR SOILS

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Abstract

The design of a shallow foundation in a granular soil is usually governed by the settlement of the footing rather than the bearing capacity of the soil. However, research has indicated that there are currently more than forty settlement prediction methods and theories and even the best of these methods over predict settlements and underestimate maximum allowable bearing capacity. The primary objective of this project is to develop a numerical model of the load-deformation behaviour of a shallow foundation in granular soil and to validate the model against full-scale load test data which is available in literature. FLAC is a powerful geotechnical modelling software that is used worldwide and was used to develop the numerical model. Bearing capacity theories and settlement methods were critically reviewed and studied using the numerical model and full-scale data. The model predictions were compared with predictions and results from the Settlement '94 ASCE Conference Prediction Symposium which was held at Texas A&M University, where five spread footings were load tested and participants were asked to predict the load-displacement behaviour for each footing. On comparison with the measured results and other participants in the conference, it was found that the model gave conservative predictions for loads which would result in 25 mm and 150 mm settlement, however better predictions for loads resulting 150 mm settlement were achieved using bearing capacity theories.

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List of Symbols

Symbol	Definition
B	Width of shallow foundation
c	Soil cohesion
C	Depth correction factor applied to Terzaghi and Peck's theory
C_1	Depth correction factor applied to Schmertmann's theory
C_2	Time correction factor applied to Schmertmann's theory
C_N	Depth correction factor applied to SPT blow count
D'	Constrained modulus
D_f	Depth of foundation embedment below ground level
E	Modulus of elasticity (Young's modulus)
G	Shear modulus
I_z	Displacement influence factor
K	Bulk modulus
k_n	Normal stiffness of soil-concrete interface
k_s	Shear stiffness of soil-concrete interface
L	Length of shallow foundation
N	Number of blows required for 300mm penetration in SPT
N'	SPT blow count correction for fine silt sands below the water table
N_1	SPT blow count with depth correction applied
N_{60}	SPT blow count corrected for energy dissipation
$(N_1)_{60}$	SPT blow count corrected for energy dissipation and depth

N_c, N_q, N_γ	Bearing capacity factors
q	Applied pressure
Q	Applied load
Q_{150}	Load applied to produce 150 mm foundation settlement
Q_{25}	Load applied to produce 25 mm foundation settlement
t	Time since loading
z	Depth below foundation
Δz	Change in depth
γ	Unit weight of soil
δ	Settlement of foundation
ε_z	Vertical strain
ϕ	Friction angle
η	SPT hammer efficiency
v	Poisson's ratio
ρ	Soil density
σ	Normal stress
σ'_{vo}	Effective initial vertical stress
σ_1	Axial stress
σ_3	Confining stress
τ	Shear stress

Chapter 1: Introduction

1.1 General

It is important that the settlement of shallow foundations on granular soils can be predicted with good accuracy and reliability to prevent catastrophic failure of supported structures. However, it has been well established in literature that the current methods which are being used to predict the settlements of shallow foundations in granular soils are quite poor. This is concerning, as it is often the settlement of shallow foundations in granular soils rather than the bearing capacity of the underlying soil governs the design of a structural footing. Most of the existing settlement prediction methods over predict settlements and underestimate the maximum allowable bearing capacity of a granular soil. The error associated with settlement predictions may also be compounded by the fact that the engineer must assume that the soil data acquired has been tested to the relevant standards and is representative of the soil area of interest. It is often not feasible to perform additional testing on the soil; however with new technologies constantly emerging, new models and theories may be developed to predict the settlement of shallow foundations in granular soils.

A settlement prediction symposium was held in 1994 at Texas A&M University, USA, in order to evaluate current practices used in the design of shallow foundations. In total, thirty one international 'experts' submitted predictions to be used in the study. An extensive range of insitu and laboratory soil tests were performed at the site. The conference showed that the majority of predictions were inaccurate and unreliable, and that settlement prediction still remains a problem from a practical and theoretical point of view.

The following document contains a review of existing bearing capacity theories and settlement prediction methods in addition to a review of FLAC and the numerical methods used by the software to solve the problem. More specifically, the explicit finite difference programs FLAC and FLAC^{3D} are used to develop and validate numerical models which can be used to predict settlements of shallow foundations in granular soils. Other computation tools used for numerical modelling in geotechnical engineering are also discussed.

The constructed numerical model can then be applied to the foundations constructed at Texas A&M University as though the author was a participant of the Settlement '94 conference. The results of the conference will be made available for comparisons and evaluation after initial predictions are 'submitted'.

1.2 Project Objectives

The intended objectives of this project are to:

1. Develop and validate a FLAC model for strip and circular footings;

2. Develop and validate a FLAC^{3D} model for square footings;

3. Investigate the influence of various soil properties on settlement curves;

4. Apply the model to a specific problem and compare results generated by each model to the measured results;

5. Investigate the influence of interface elements;

6. Determine if results obtained using FLAC for a circular footing can be accurately used to represent results obtained using FLAC^{3D} for square footings; and to

7. Perform a detailed comparison of the model with the other author's predictions and commonly used design methods.

The criteria which will determine the success of the project include understanding of the problems associated with settlement predictions and measuring the performance of the model in terms of accuracy and reliability when compared to field data. The model was applied to a specific problem, allowing it to be compared against predictions made using other methods. The model was developed so that modifications could easily be made, making further investigations possible, while allowing the model to be applied to other scenarios involving settlements of shallow foundations in granular soils.

The main constraint for this project is time. Resourcing requirements are minimal for the project. The model was initially developed using the student version of FLAC which can be run on any PC and be used to solve meshes with up to 500 elements. The work was then extended to the full version, where larger meshes were used. James Cook University currently has three FLAC 5.0 keys and two FLAC^{3D} keys, which are required to run the software.

1.3 Statement of the Problem

The problem proposed for the development of this thesis can be stated as follows:

The load-deformation behaviour of shallow foundations in granular soil is to be modelled and validated against full scale data available in literature using FLAC and FLAC^{3D}; two software packages which are commonly used worldwide in industry and by academics.

This thesis aims to establish if a numerical model can adequately represent the load-deformation behaviour of a shallow foundation in granular soil.

Chapter 2: Literature Review

2.1 Introduction

Foundations are structural elements that transfer loads safely to the underlying soil. Foundations can be classified as either shallow foundations or deep foundations.

Deep foundations are used to support large loads in weaker soils. The most popular types of deep foundations are piles and piers. Deep foundations are not within the scope of this thesis and will not be considered in further detail.

A shallow foundation can be defined as a footing with a width B that is equal to or greater than the depth of foundation below the ground level, D_f. Figure 2.1 shows how loads are transferred from a structure to the underlying soil. Shallow foundations are used when the structure may be supported within a stratum of a soil located at a relatively shallow depth. Skin friction is neglected for shallow foundations in granular soils; therefore it is assumed that the entire bearing pressure is distributed over the area of the base.

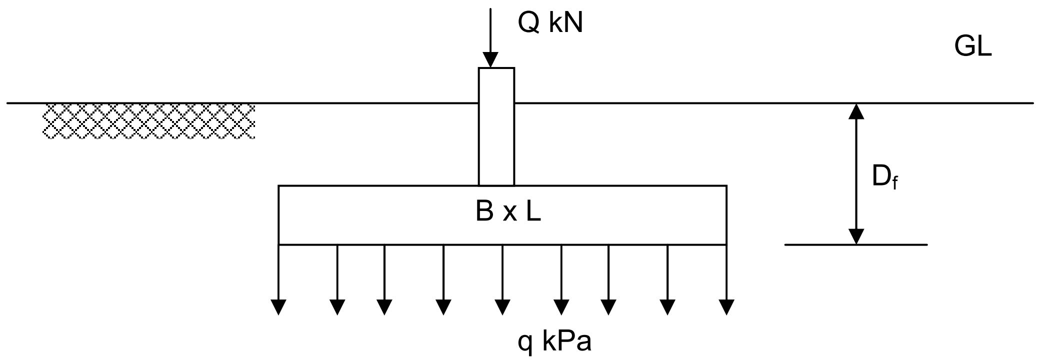


Figure 2.1: Section through a shallow footing

2.1.1. Classification of Granular Soils

Granular soils are cohesionless soils and may be defined by the size of the soil particles within a sample. Classification of soils provides a systematic and consistent system of group soils of similar behaviour and grain size. The distribution of grain sizes can be determined using a sieve analysis. There are a number of soil classification systems in use worldwide. Soil classification in Australia is performed to Australian Standard AS1726-1993, however the Unified Soil Classification System (USCS) is recognised internationally in geotechnical engineering projects. A comparison of these may be seen in Figure 2.2. In addition to USCS, most countries adopt their own classification systems.

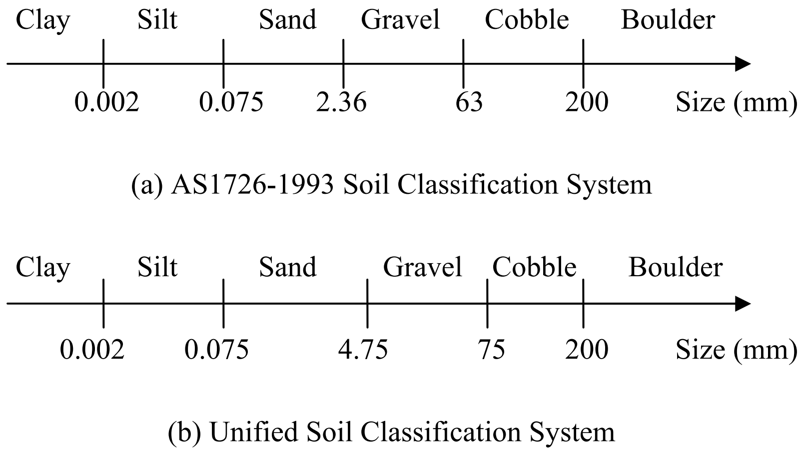


Figure 2.2: Comparisons between AS1726-1993 and USCS classification systems

It can be seen in the above figure that the limits on particle sizes for gravels differ for the two classification systems. For this thesis, the AS1726-1993 classification system will be used to define sand. However in both the AS1726-1993 and the USCS classification systems, granular soils can be defined as soils with a particle size of more than 0.075 mm diameter.

2.1.2. Types of Shallow Foundations

The most commonly used shallow foundations in structural applications are pad footings, strip footings and raft or mat footings. Pad footings are used to support singular column loads, and are usually square, circular or rectangular in shape. Circular footings impose axisymmetric loads on the underlying soil directly below the column.

Strip footings (Figure 2.3(b)) are often used to carry walls or live loads. Strictly speaking, a strip or continuous footing has a width to length ratio of zero. However, footings with a high length to width ratio may also be treated as strip footings. Plane strain loading occurs underneath strip footings and loads are more evenly distributed to the underlying soil. The depth of influence for stresses underneath surface loads for strip footings is approximately twice that of a square footing of equivalent width.

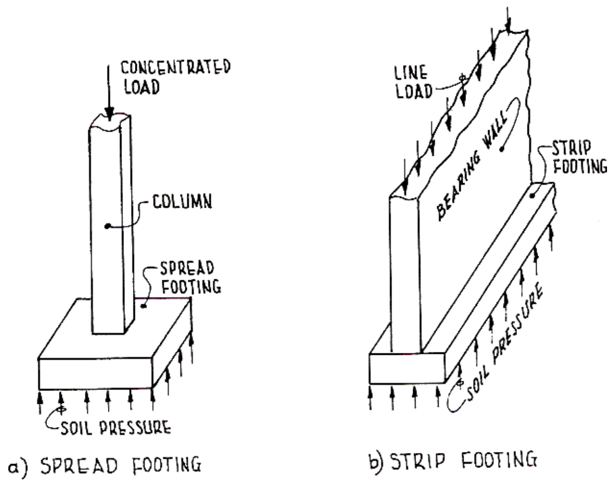


Figure 2.3: Pad and strip footings (French 1999)

Raft or mat footings consist of a single slab which covers the supporting soil beneath the entire area of the superstructure. Essentially a raft footing is a large pad which carries several column or wall loads and distributes these loads in two directions. Raft footings can be used to limit differential settlements of structural components, however the differential settlements of the supporting soil can induce bending stresses within the footing itself, leading to cracking or failure. Therefore only pad and strip footings will be considered in this thesis.

2.2 Design of Shallow Foundations in Granular Soils

There are three considerations which must be followed when designing shallow foundations in granular soils.

1. Allowable Bearing Capacity. The underlying soil must not fail.

2. Allowable Settlement. There must not be excessive settlements of the foundation. Typically a maximum of 25 mm is allowed in granular soils, slightly more in clays; and

3. Economic factors. Economic factors of foundation design are not within the scope of this thesis and will not be discussed in further detail.

It is well documented in literature that selection of allowable bearing pressures for a shallow footing on sand is most often governed by the allowable settlements (Berardi and Lancellotta 1994; Jeyapalan and Boehm 1986; Tan and Duncan 1991).

Prediction of the bearing capacity and settlements requires proper site investigation. It is almost impossible to obtain undisturbed samples of granular soils for laboratory testing;

therefore insitu testing methods are frequently used for granular soils. The most popular test methods are based on the Standard Penetration Test (SPT) and Cone Penetration Test (CPT). The results of these tests can then be correlated to the strength and stiffness of the soil through the use of empirical relationships. Granular soils can display high variability of soil properties at locations of close proximity; therefore it is important to note that these values are representative of the soil.

2.2.1. Allowable Bearing Capacity

Granular soils are considered to have good bearing capacity. Failure of soils carrying foundation loads occurs when soil particles slide over each other and takes place much sooner than crushing of soil particles. The point at which this failure occurs is the ultimate bearing capacity. Typically, shear failure occurs along a plane as shown in the figure below.

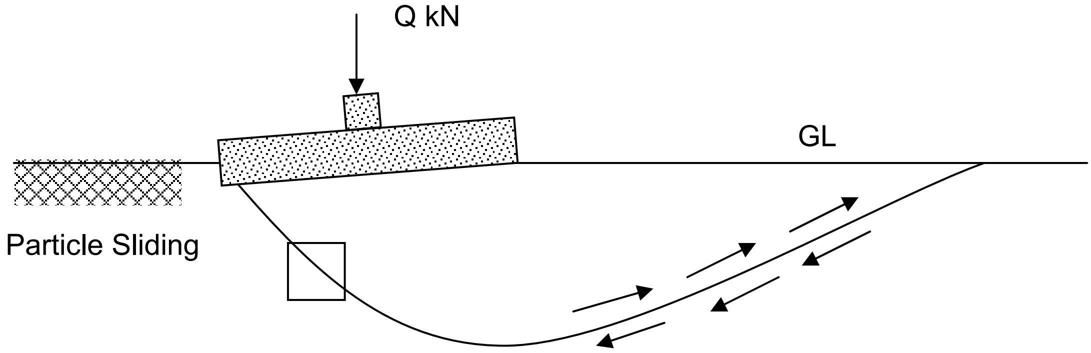


Figure 2.4: Shear failure in a soil

Mohr's circles can be used to represent the state of stress for a soil element. Each point on the Mohr circle represents the unique values of normal and shear stresses along a specific plane on the element. The Mohr circle has to be contained within an envelope for an element to be stable. When they touch, the element fails. The normal stresses at failure produce the circle that is tangential to the failure envelope. Because the soil is granular, there is no cohesion between particles. These concepts are shown in Figure 2.5, where the failure envelope is given by the Coulomb equation:

$$\tau_f = \sigma \tan\phi + c \qquad (2.1)$$

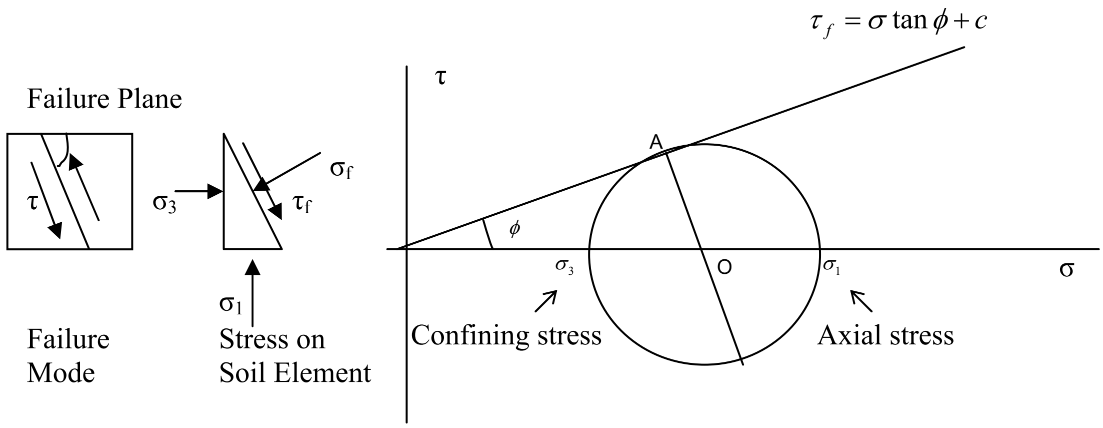


Figure 2.5: Mohr circle and state of stress at failure

Terzaghi (1943) developed a theoretical relationship on the basis of plastic theory, which can be written as:

$$q = \frac{Q}{BL} = cN_c + \gamma DN_q + \tfrac{1}{2}.\gamma BN_\gamma \tag{2.2}$$

where N_q, N_c and N_γ are bearing capacity factors which are dependent on the soil characteristics (vide Meyerhof 1963). These factors may be obtained from equations and charts. Several equations exist for the calculation of N_γ. Some of these methods provide more conservative predictions of bearing capacities than others and may produce a large variation in the ultimate bearing capacity. Meyerhof (1963) also believed that the ultimate bearing capacity of shallow foundations on granular soils could be estimated with sufficient accuracy on the basis of plastic theory and extended upon Terzaghi's basic theory by researching the effects of shape, foundation depth, eccentricity and inclination of the load. Meyerhof accounted for these effects by developing equations for factors which could be applied to Terzaghi's original theory.

The ultimate bearing capacity is reduced by a safety factor to give the allowable bearing capacity. A factor of 3 is used for shallow foundations to account for unfactored loads applied to the structure, oversimplified theory and uncertainty in design parameters.

2.2.2. Allowable Settlements

The prediction of settlements of shallow foundations is important to prevent structural damage. Accurate prediction of the settlement of a shallow foundation in granular soil is much for difficult than predicting the bearing capacity of a soil due to variability of granular soil compressibility and soil density at different locations. This variability can occur within close proximity, even in sand deposits that appear to be homogenous.

In most cases, this structural damage is caused by differential settlements of footings within the structure. Differential settlements are caused by footings within the same structure settling by differing amounts. This creates bending stresses within the structure and may lead to cracking of structural members or catastrophic failure. Limiting the amount of settlement of the footing will also limit the differential settlements.

The settlement of a shallow foundation in sand is dependent on the following parameters (Jeyapalan and Boehm 1986):

- Footing dimensions, width length and depth;
- Contact pressure;
- Soil profile;
- Depth of water table;
- Sand type; and
- Compressibility

There are more than forty settlement prediction methods currently in use (Douglas 1986). The most widely used of these methods include Schmertmann et al. (1978) and Burland and Burbidge (1985). The accuracy and reliability of these methods will be discussed in Chapter 2.6 of this thesis.

2.3 Existing Methods of Settlement Prediction

Many methods have been published for the prediction of settlements of foundations on sands and gravels. Most of these methods treat the underlying soil as a uniform layer by using an average of the soil test data within the layer. The influence depth of this assumed layer is dependent on the method being used. Two popular methods are investigated in this section. It is intended to compare the predictions of the FLAC models to these commonly used settlement prediction methods in Chapter 6.

2.3.1. Terzaghi and Peck's (1948) Method

In 1948, Karl Terzaghi and Ralph Peck developed a simple and quick method to estimate the settlement of a shallow foundation in granular soil. This method is based on laboratory tests on model footings and was validated with some field tests. Terzaghi and Peck conducted plate load tests on three different sands with SPT values of N=10, N=30 and N=50 using a

0.3 m square plate. Their experimentation involved a sand chamber and a 300 mm square loading plate. Load was incrementally applied using a loading jack and measured using a load cell. Pressure-settlement curves were developed for use in an empirical relationship between the settlement of the plate and the settlement of the loaded footing, which forms part of a design method.

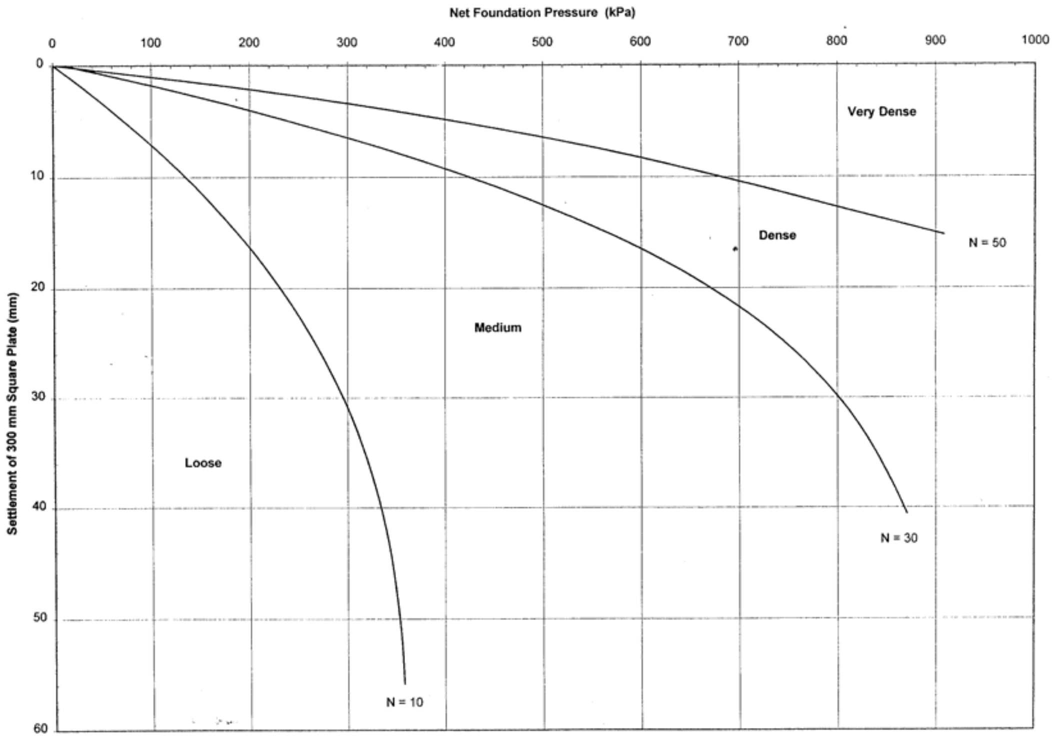


Figure 2.6: Pressure-settlement curves for 300 mm square plate (after Terzaghi and Peck 1948)

The settlement of the plate can be read from the above chart and substituted into the following equation to predict settlements for different values of B under the same pressures.

$$\delta_{footing} = \delta_{plate}\left(\frac{2B}{B+0.3}\right)^2 \tag{2.3}$$

A depth correction factor was later introduced into the method to allow for reduction of the stresses within the soil after the embedment of the foundation. Terzaghi and Peck (1948) suggested that this should be equal to the expression shown below.

$$C = \left(1 - \frac{1}{4}\frac{D_f}{B}\right) \tag{2.4}$$

Leonards (1986) suggested that this was too conservative, and suggested that the expression should be $\left(1-\frac{1}{3}\frac{D_f}{B}\right)$. It must be noted that Terzaghi and Peck's method was originally developed for square pads, but can be used with caution for rectangular and strip footings.

2.3.2. Schmertmann's Method

Schmertmann's (1970) Method uses a static cone to compute static settlement over sand. The method is based on elastic half-space theory and Finite Element Analysis. The method involves determining a simplified distribution of vertical strain under a foundation in the form of a strain influence factor, shown below.

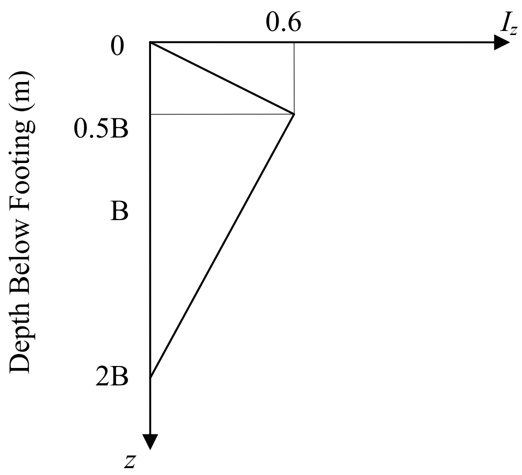


Figure 2.7: Schmertmann's (1970) 2B-0.6 influence diagram

Schmertmann proposed that the strain below a loaded foundation could be found by the following equation.

$$\varepsilon_z = \frac{q_n}{E} I_z \tag{2.5}$$

The total settlement of the foundation may be found by integrating the vertical strain over the depth of influence, which was considered as twice the width of the foundation.

$$\delta = \int_0^\infty \varepsilon_z$$

$$\delta = q \int_{z=0}^{z=2B} \frac{I_z dz}{E_z} \tag{2.6}$$

Corrections for the depth of the foundation and the amount of creep are applied to the above expression in the form of factors C_1 and C_2. However, there is some debate on the applicability of the correction factor for creep. Leonards (1988) and Holtz (1991) believe that settlements of shallow foundations in granular soils is not time dependent and therefore, C_2 is always equal to 1.0.

$$C_1 = 1 - 0.5\frac{\sigma'_{vo}}{q} \tag{2.7}$$

$$C_2 = 1 + 0.2\log\left(\frac{t}{0.1}\right) \tag{2.8}$$

Therefore, the equation for total settlement becomes,

$$\delta = C_1.C_2.q\sum_{0}^{2B}\frac{I_z}{E_z}\Delta z \tag{2.9}$$

Schmertmann also carried out research to determine the correlation between static cone bearing capacity (q_c) and the Young's modulus values used in settlement computations. His experiments involved screw-plate tests and he found that Young's modulus was equal to twice the CPT cone tip resistance (E=2q_c).

Schmertmann (1978) later expanded on the original theory by considering the axisymmetric and plane strain modes of deformation. Improved strain influence factor diagrams were developed using two strain factor distributions, one for square footings (axisymmetric) and one for strip footings (plane strain). For rectangular footings, the settlements may be calculated using both models and then linearly interpolated.

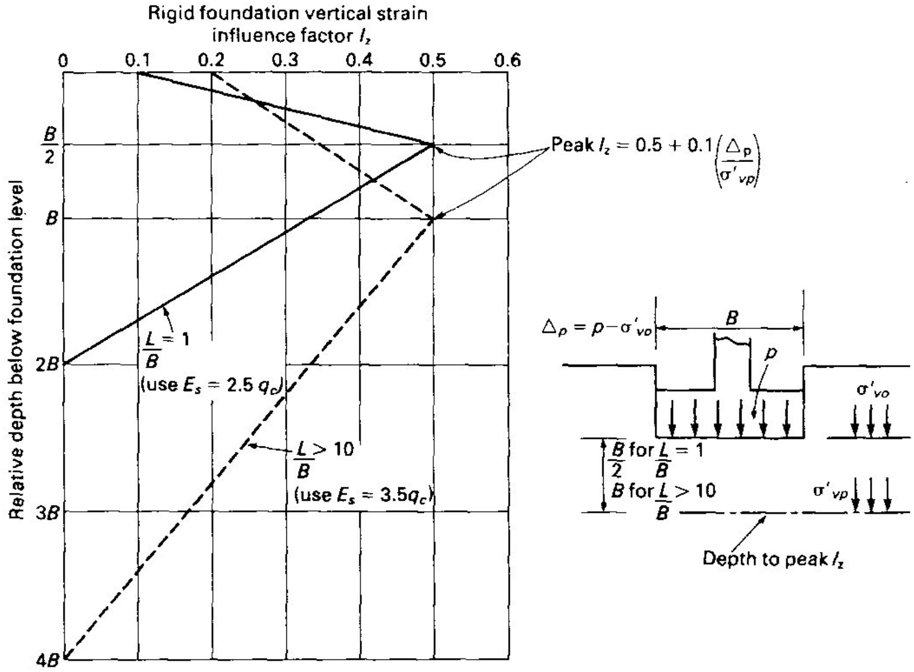


Figure 2.8: Schmertmann's (1978) influence diagrams (Tomlinson 1995)

It can be seen in Figure 2.8 that a strip footing has twice the influence depth of a square footing. These new distributions required modifications for the Young's modulus correlations with the CPT cone tip resistance. Instead of using $E = 2q_c$ for the footings, it was proposed that for square footings $E_{axisym} = 2.5q_c$ and $E_{plane\ strain} = 3.5q_c$. Terzaghi, Peck and Mesri (1996) simplified the method by suggesting that at a depth of z=0 beneath the footing, I_z should equal 0.2 and that $I_{z,peak} = 0.6$. The influence depths remained the same as for Schmertmann's (1978) method.

2.4 Computational Tools Used in Geotechnical Engineering

The use of computer-based methods in foundation engineering is not as developed or extensive as in other areas of civil and structural engineering (Tomlinson 1995). However, the development of the personal computer has seen advances in modelling software packages. There are many different finite element and finite difference software programs available for applications in geotechnical engineering. Some widely used numerical modelling software programs include FLAC, PLAXIS, ANSYS, ABAQUS and CRISP. These packages may contain similar features and soil constitutive models. The key attributes that make a software program popular include the ease of use of the software and the adaptability of the software to specific applications.

2.4.1. FLAC and Other Itasca Programs

FLAC (Fast Langrangian Analysis of Continua) is a general purpose geotechnical software which is used worldwide and can operate on personal computers to quickly solve complex problems without the requirement of a mainframe computer. FLAC was developed by Itasca Consulting Group in Minneapolis, USA. Itasca is an engineering firm and software developer that designs modelling software specifically for geomechanics. Since FLAC was introduced in 1986 there are more than 2,500 users located in 60 countries (Itasca 2005) FLAC^{3D} is a version of FLAC which has been extended into three-dimensions. Other programs developed by Itasca include UDEC (Universal Distinct Element Code; for discrete blocks such as jointed rocks) and 3DEC, its three-dimensional counterpart (Coulthard 1994). Assemblage of circular or spherical particles will require Itasca's Particle Flow Code (PFC^{2D} and PFC^{3D}). A more detailed review of FLAC and FLAC^{3D} will be given in Chapter 2.5.

2.4.2. PLAXIS

PLAXIS is a geotechnical modelling software, which shares some similarities to FLAC. PLAXIS operates with a Graphical User Interface (GUI) which many engineers consider user friendly. This creates some limitations though, as the commands are not customisable for every situation. Ease of use enables engineers to use the software without requiring an extensive knowledge of soil mechanics. However, solving complex geotechnical problems without this knowledge and some understanding of the problem can also lead to critical design errors. In some ways, this creates a "black box" effect, and unless one can calculate and estimate answer to the problem, there may be no way of checking the validity of the solution. Nevertheless, it has been used world-wide for a number of years and no serious problems have been reported to date. Like FLAC, it operates on personal computers, eliminating the requirement for mainframe computers making it a viable option for small design offices.

2.4.3. ABAQUS and ANSYS

ABAQUS is a finite element program developed for linear and nonlinear simulations of engineering problems. Although not specifically designed for geotechnical engineering, it is a useful tool which is relevant to geotechnical studies. ABAQUS can be used to find solutions to nonlinear problems, seepage and pore pressure analysis, stage construction sequences, concrete liner modelling, joint and foundation interaction and earthquake analysis (Currie 1994). Learning to use ABAQUS is regarded more difficult than learning FLAC or PLAXIS and takes some time to be able to use the features comfortably.

ANSYS is another general purpose finite element program for analysis of engineering programs. Modules can be added to ANSYS which enables specific types of problems to be modelled. The CIVIL FEM module customises ANSYS for structural and geotechnical modelling.

Both ABAQUS and ANSYS operate on mainframe computers, making them more suited to academic use or large scale research and design projects.

2.4.4. CRISP

CRISP (CRItical State Program) is a two-dimensional finite element program developed in the UK. CRISP operates with a GUI (menu-driven mode) by default, but also has an option to be command driven, and like FLAC can be operated on a personal computer. CRISP has capabilities to model three-dimensional models when run in command mode. Because CRISP shares many similarities to FLAC and PLAXIS, it will not be reviewed further.

2.5 Numerical Modelling in FLAC and FLAC^{3D}

Software modelling packages can be chosen based on cost, availability, solution method, constitutive models and their application in industry. FLAC and FLAC^{3D} were chosen over other modelling packages because of their suitability to the problem and the factors mentioned above.

FLAC is a two-dimensional explicit finite difference program which was originally developed for geotechnical and mining applications, however is also capable of the analysis and design in other applications of civil and mechanical engineering. Soils are represented as a continuum in FLAC.

The software was developed for the high speed computation of model grids containing numerous elements, and offers the ability to solve complex problems in soil mechanics. FLAC^{3D} is available for the analysis of problems that require three-dimensional modelling. Both software packages are explicit finite difference programs. The commands used in FLAC and FLAC^{3D} are very similar. This means that a two-dimensional model can be extended into three-dimensions with relative ease. A brief introduction to FLAC^{3D} has been provided in Appendix B.

2.5.1. Features of FLAC and FLAC^{3D}

FLAC can be operated in two modes of operation. The first mode is command-driven. Commands are written using recognisable words and parameters. A logical sequence of commands often correlates with the physical sequence of the actual system and can be used to indirectly keep a documentation of the analysis procedures. There are two ways for command input; “interactive” mode is when the commands are entered directly into the command prompt and executed line by line, or “batch” mode where scripts are created and modified with a text editor (Microsoft Notepad or similar). The script then is called by FLAC and executed. There are approximately fifty main commands and nearly 400 keywords which may be applied to the main commands to modify them. A simple analysis may be performed with only a few commands. Although learning this new “language” may seem daunting at first, most professionals who have been using FLAC for years operate in the command-driven mode because of its versatility of application.

In version 4.0 of FLAC, a menu-driven mode, known as GIIC (Graphical Interface for Itasca Codes) was released. Although the menu-driven mode seems easier to operate and learn at first, it lacks the versatility of the command-driven mode. It may be suitable for occasional users, but there is an inherent danger that the user will design a structure without checking the validity of the design. FLAC is not a “black box” which gives “the solution”; the behaviour of the numerical system must be interpreted (Coetzee et al. 1998). Users of FLAC should have some general understanding of soil mechanics and numerical analysis.

Special features of FLAC include a library of constitutive models which represent geologic materials, the use of interface elements, various geometry modes, structural element models, ground water and consolidation calculation, dynamic analysis and thermal modelling capability and a large strain mode.

It is not always possible to express the user’s needs using the FLAC commands alone. FLAC has an inbuilt programming language called FISH (short for FLACish) which enables the user to define new variables and functions to customise analyses to fit a specific problem. FISH is a compiler and can be used to program new variables, additional material constitutive models, apply servo controls to a numerical model and generally enhance the features available in FLAC. FISH is similar to other programming languages such as FORTRAN, or BASIC and enables conditional if statements and loops to be included within a FLAC program. FISH code is written and embedded within FLAC command scripts.

2.5.2. Comparison of Modelling Methods

Most software programs use one of two methods for solving problems; the Finite Difference Method (FDM) and the Finite Element Method (FEM). Many numerical modelling packages favour finite element methods; however, the efficiency of each method is dependent on the application and type of problem.

Finite element methods discretise a model into a system of individual elements. These elements are treated by an extension of concepts used in structural analysis. Within each element, it is assumed that selected material properties are constant and particular field variables vary in a prescribed fashion. Matrix solution schemes are common in finite element programs where a set of differential equations may be transformed in to matrix equations for each element. An element stiffness matrix relates nodal forces to nodal displacements for elastic materials. These element stiffness matrices can then be combined into a large global stiffness matrix for solving the problem.

In the finite difference method, Ordinary Differential Equations (ODEs) can be solved by transforming every derivative in the set of governing equations to algebraic expressions written in terms of field quantities such as stress or displacement at discrete points in space. These variables are undefined everywhere else. Once the continuous domain is discretised, the exact derivatives in the ODE can be approximated by Finite Difference Approximations. These Finite Difference Approximations are substituted into the ODE to obtain an algebraic Finite Difference Equations.

FLAC uses an explicit solution scheme when solving equations (Coetzee et al. 1998). This is achieved by time-marching or time-stepping, which is an iterative process that adjusts the values of each node in the mesh through a series of cycles or steps. After each cycle, the values of each node are adjusted in the mesh. The central concept of the explicit method is that the “calculation wave speed” always keeps ahead of the physical wave speed. This enables the equations to always operate on known values that are fixed for the duration of the calculation.

Implicit schemes are more common in finite elements and involve the use of matrix methods to obtain a solution (Coetzee et al. 1998). In an implicit method, all elements influence each other during one solution step. Many iterations may be required before compatibility and equilibrium conditions are reached.

The calculation cycle used in FLAC starts by using the equations of motion to derive velocities from stresses and forces. Strain rates are derived from the velocities, and new stresses are calculated from the strain rates. One timestep is equal to one cycle around this loop.

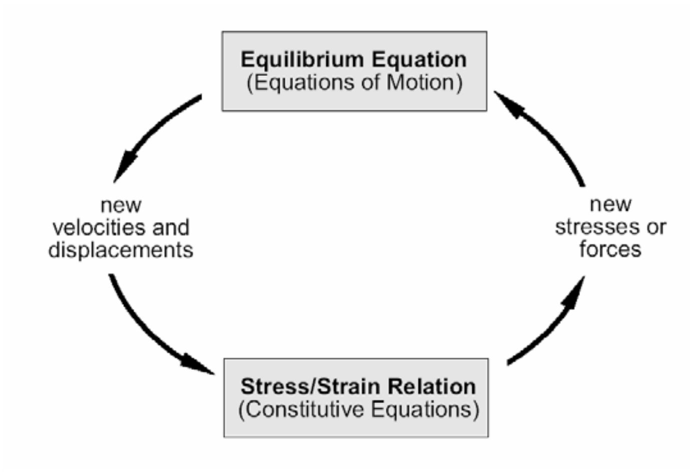


Figure 2.9: Explicit calculation cycle (Coetzee et al. 1998)

All of the grid variables are updated from known values for each of the operations shown in the boxes in Figure 2.9. The timesteps are so small that the elements cannot affect each other during the calculation cycle (Coetzee et al. 1998). If stresses are calculated from velocities, the velocity is considered to be fixed for the calculation. The velocity is considered to be fixed while control is within the operation. Propagation still occurs across elements after a number of cycles of the loop reflecting physical conditions.

Both methods produce identical algebraic equations and therefore identical solutions. It is therefore the application and type of problem that is often the deciding factor in which method to use to obtain a solution. Differences between the methods are most easily observed by considering the advantages and disadvantages that finite difference methods have over finite element methods. FLAC utilises full dynamic equations of motion to model systems that are essentially static. An explicit solution scheme is more efficient than implicit methods for non-linear, large strain or problems which are physically unstable. Matrices are rarely formed in finite difference modelling because it is reasonably efficient to develop equations at the end of each step. Therefore large two-dimensional problems can be solved using less computing resources than finite element modelling by regenerating the finite difference equations at each step.

Disadvantages of programs that utilise finite difference methods are that computation times are slower than implicit (finite element) methods for linear, small-strain problems and that they may be inefficient for models where there are large differences in elastic modulus or poorly defined grids.

2.5.3. Implementation of Solution

The finite difference grid (or mesh) consists of elements and nodes across the problem zone. The grid also defines storage locations of all state variables within the model. Vector quantities and saturation and temperature are stored at nodes. Scalar quantities and fluid flow rates are stored within element centroids. Pore pressure is stored in both nodes and element centroids.

The mesh is defined by an *i-j* plane, therefore it follows that elements and nodes are identified by *i* and *j* coordinates, which are integers. The mesh is applied in *x-y* space where the *x* and *y* coordinates are the real coordinates and can be customised to suit model geometries.

Initial and boundary conditions must be defined within the model. The state of all variables in the model prior to any changes caused by loading or disturbance is known as the initial condition. A constraint or controlled condition along a model boundary is known as the boundary condition. Boundaries may be real or artificial; real boundaries are the physical limits of the model, whereas artificial boundaries can be lines of symmetry and lines of truncation used to simplify problems.

Once the modelling space and conditions have been established, the compiled code may be executed. A solution to the problem will be found, and the information can be conveyed by FLAC in tabular form, or visually in the form of plots. There are many options for plotting data, and various plots can be overlaid if required.

2.6 Current State-Of-The-Art for Prediction Methods

Geotechnical engineering is a relatively young discipline of engineering. There have been many developments of the theories and methods used to calculate bearing capacities and settlement of shallow foundations in granular soils in the past fifty or so years. Most of the popular theories and methods seem to have been developed so that they achieve a high level of accuracy. However, only a few of the methods appear to be truly reliable. This poor state-of-the-art is well documented in literature. The origins and shortcomings of the conventional design methods are investigated in the sections below.

Many studies have been carried out to determine the accuracy and reliability of settlement prediction methods for shallow foundations in granular soils. These include Berardi and Lancellota (1994), Jeyapalan and Boehm (1986), Papadopoulos (1992), Sivakugan et al. (1998) and Tan and Duncan (1991). Probability studies have also been carried out by

Sivakugan and Johnson (2004) to quantify the uncertainty associated with settlement predictions concerning shallow foundation design.

Accuracy can be defined as the average value of calculated settlement divided by measured settlement (Tan and Duncan 1991). It is a measure of how the calculated settlements compare to the actual measured settlements. The more accurate a prediction of the settlement of a footing is, the more efficient the design will be.

Reliability can be defined as the percentage of the cases in which the calculated settlement is equal or exceeds the actual measured settlement. Under prediction of the settlement could result in catastrophic failure of the structure. The findings of these studies indicate that methods developed with reliability in mind often result in poor accuracy. In contrast, accurate methods often have poor reliability. The model developed in this thesis can be developed by considering the features of the various methods investigated below.

2.6.1. Bearing Capacity

Current theories and methods used to calculate bearing capacities of shallow foundations in granular soils are considered to provide estimates with good accuracy. Terzaghi (1943) pioneered research which led to a better understanding of the fundamentals of the behaviour of foundations. This theory was based on plastic theory and even today, the theory proposed by Terzaghi forms the basis of most bearing capacity predictions. Modifications by Meherhof (1963) accounted for shape, depth, inclination of the applied load, eccentricity and plane strain through the use of correction factors.

In the past, empirical approaches were used for the design of shallow foundations and allowable bearing pressures for different soil types rather than settlements would have governed the design (Meyerhof 1965). However it is now well established that the settlement of the foundation and the stability of the structure are the critical design factors.

2.6.2. Settlements

As documented previously in this thesis, more than forty prediction methods exist for shallow foundations in granular soils (Douglas 1986). Many of these methods (Meyerhof 1965; Papadopoulos 1992) have been based on drastically simplified hypotheses. These methods only require a small number of input parameters; however it is important that this data is accurate in order to obtain an accurate settlement prediction.

Schmertmann (1970), Schmertmann et al. (1978), Burland and Burbidge (1985), and Berardi and Lancellotta (1994) proposed the use of empirical methods to predict settlements. Some methods lack explicit soil mechanics principles and for certain cases should be applied with caution.

Berardi and Lancellota (1994) proposed that soil stiffness should be treated as a parameter that is dependent on the current state of stress, specific volume and strain rather than a material property. The process is more accurate, but becomes complex due to its iterative process.

Prediction methods based on elastic theory are commonly applied to problems involving settlements due to their simplicity. At the other extreme, non-linear constitutive laws combined with finite element methods allows for correct modelling of foundation behaviour, however these methods are often too expensive for most practical problems (Nova and Montrasio 1991). Sivakugan et al. (1998) have recently investigated Artificial Neural Networks (ANN) as a means of predicting the settlements of shallow foundations in granular soils. Once trained by data from literature, the method provided better predictions than the traditional methods.

Tan and Duncan (1991) also investigated the ease of use of various methods by performing manual calculations and measuring the time and effort required to estimate the settlements. Their findings indicated that for a singe example, a solution with reasonable accuracy may be found with relative ease.

2.6.3. Settlement '94 Conference

To examine various design methods for spread footings on granular soils, a full scale load testing exercise which entailed participants to predict the behaviour of five spread footings on sand was carried out. Predictions were presented by thirty one international experts including academics and consultants during the Settlement '94 ASCE Conference Prediction Symposium held at Texas A&M University. Five spread footings were load tested to 150 mm of displacement in sandy soil, ranging in size from 1 x 1 m to 3 x 3 m. Participants were asked to predict the loads necessary to create 25 mm and 150 mm of settlement after 30 minutes of load application. They were also asked to predict the creep settlement over 30 minutes and the settlement by the year 2014 for the 25 mm load.

On comparison with measured data, the results of these tests indicated that nobody had a complete set of answers which was consistently within ± 20% of the measured values. Two participants had predictions which fell within this margin of error. The load predictions for a given settlement value of 25 mm were on the safe side 80% of the time and underestimated by 27%, however the predictions were only underestimated by 6% and on the safe side 63% of the time for the load creating 150 mm of settlement (Briaud and Gibbens 1994).

Most of the participants used a combination of methods. Because of this, it was not possible to identify which method was most accurate, although Schmertmann's (1978) method appeared to be the most popular, making the CPT data the most used. Overall, the calculated average safety factor was very conservative and the average settlements were under predicted. This study and many others of a similar nature indicate that the design of shallow foundations in granular soils has potential to be more efficient.

2.7 Research Significance and Conclusions

It has been established in the literature review that there are shortcomings in current settlement prediction methods. Settlement mechanisms of shallow foundations in granular soils are not fully understood. This lack of understanding creates inaccuracy in predictions and these methods can often be unreliable. The problem with most of the design methods is that they over predict the settlement of the footing and that they underestimate the maximum allowable bearing pressure. This creates the need for additional modelling of settlements of shallow foundations in granular soils. However, this requirement has also resulted in many more (and some questionable) methods and theories for designers to choose from.

A numerical model produced using FLAC can be used for verifying and improving upon previous models and theories. Once the model is created, it should be able to be used to easily predict settlements.

A perfect prediction method would give 100% reliability and perfect accuracy while being easily applied to a problem. However, it must be understood that all settlement calculations in soils can only be considered as estimates because of the variability of soil properties. It is unreasonable and uneconomical to perform soil tests within a concentrated area to account for this and to use this data in models. Therefore there will always be some degree of error associated with the predictions. Despite this, there is a need for developing a more accurate and reliable model to predict settlements of shallow foundations in granular soils as an alternative to current methods and theories.

Prediction of settlements still remains a problem from a practical and theoretical point of view (Papadopoulos 1992). Nevertheless, better accuracy in prediction methods means that design of foundations will not be as unnecessarily conservative, and lead to increased confidence levels when designing footings. This in turn has the potential to produce economic advantages in construction and material costs. A numerical model created using

FLAC may be more suited to customisation of variables and modes, enabling parametric and sensitivity analyses to be conducted at a later stage.

Chapter 3: Calibration of the Numerical Model

3.1 Overview

Numerical modelling of foundations and other aspects of geotechnical engineering requires different design philosophy than other areas of engineering where the engineer has access to fabricated materials that are not site dependent. Most soils are heterogeneous and display complex stratigraphy, having stiffness properties which are non-linear and inelastic. The presence of ground water can have a major effect on the strength of a soil. Often the analysis and design of structures and excavations in or on soils must be completed with only a limited amount of field data. Performing extensive insitu or laboratory tests to determine initial stresses and material properties over the area of interest is unfeasible and impractical.

Numerical models can be used as prediction tools, or as numerical simulation tools to understand material behaviour. The models developed as part of this thesis were used in both cases. Some well founded assumptions and correlations were employed within the model itself. These will be discussed later in the chapter.

The structure of the model attempts to reflect the physical sequence of actual settlement mechanisms. Settlement of the foundation was monitored through the centreline of the footing. This feature may also be used to increase levels of understanding the mechanism.

Bugs in the compilation of any programming script are often inevitable. These can be caused by inputting or syntax errors, or from the lack of understanding of the commands within the programming language. Pseudo code is a list of commands written in simple English that outlines what it is that the user wants the program to achieve. Some pseudo code can be seen written next to the FLAC and FISH code in Appendix C.

To help with debugging, the model was split into smaller sections and analysed in parts. The results were checked at the end of each section to ensure the model behaved as expected before compiling the complete model from the sections.

3.2 Development of the Model

Five steps were performed during the model generation process. These incorporated:

1. Developing simple, idealised models
2. Validating the models
3. Refining the models

4. Running the validated models for various applications

5. Presenting the results for interpretation

3.2.1. Idealised Model

A conceptual picture of the problem can be used to increase understanding of the theory behind the model. Initially, concepts were investigated during a planning stage, which led to the development of a crude model. This crude model allowed the settlement mechanism to occur and contained all of the basic requirements to predict settlements. The problem was simplified by assuming the footing was constructed on a single homogenous soil layer with typical soil properties. A representation of this simplification is shown in Figure 3.1.

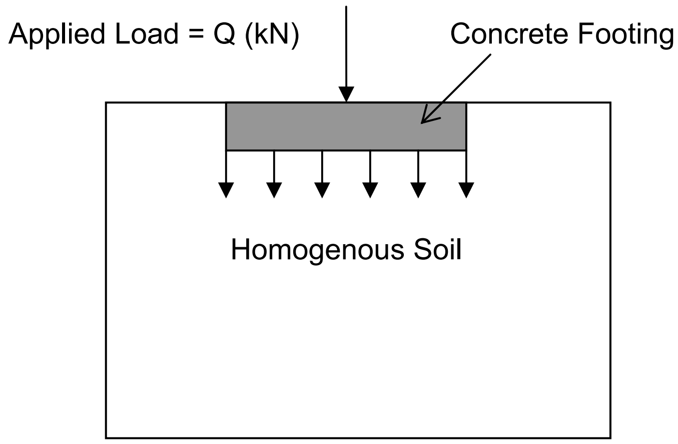


Figure 3.1: Idealised shallow foundation on granular soil

Initially, a strip footing was modelled in FLAC to obtain knowledge of the program under these simple conditions and assumptions. A plane strain analysis (Figure 3.2) was performed for this problem. An axisymmetric analysis was carried out to determine if square footings can be accurately modelled as circular footings (as shown in Figure 3.3).

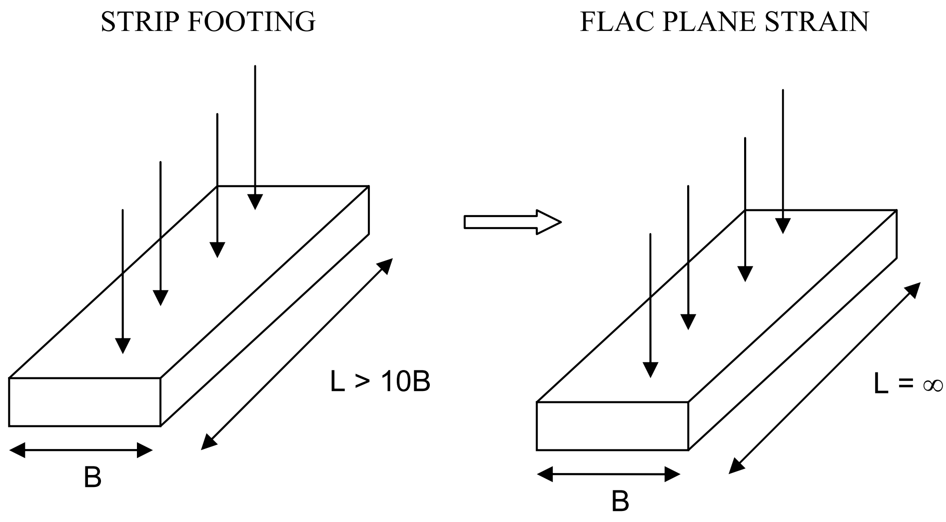


Figure 3.2: Idealised strip footing

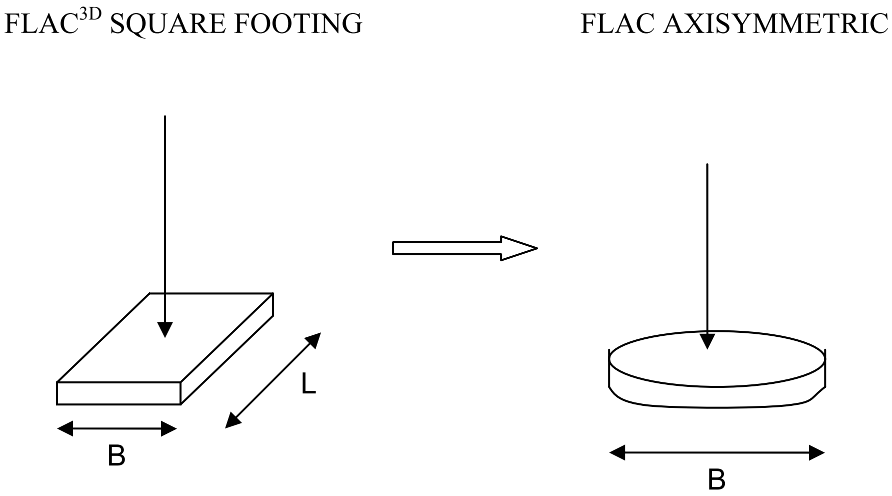


Figure 3.3: Modelling a square footing as a circular footing

3.2.2. Grid Generation

The first step in any of the finite calculation methods is that the domain must be discretised such that an accurate solution can be obtained in the region of interest. The grid is defined by i and j coordinates positioned in a real x, y space. The grid was designed so that the size of each element reflected the significance of the variations in stresses and strains within the region. Essentially, finer elements were used at regions of greater interest.

A trial problem was developed to determine the number of elements which would be used for all future investigations. Because of the substantial simplification at this early stage of model development, experimental data was unavailable for comparison. The uncertainty associated with other prediction methods also meant that a result obtained from these methods would also be unsuitable for comparison as a convergence value. Therefore, convergence was determined when the variation in the pressure-settlement curve was minimal for variation in mesh density.

Four grid densities were trialled in FLAC using typical dimensions and soil properties. The results and computation times are summarised in the figure and table below.

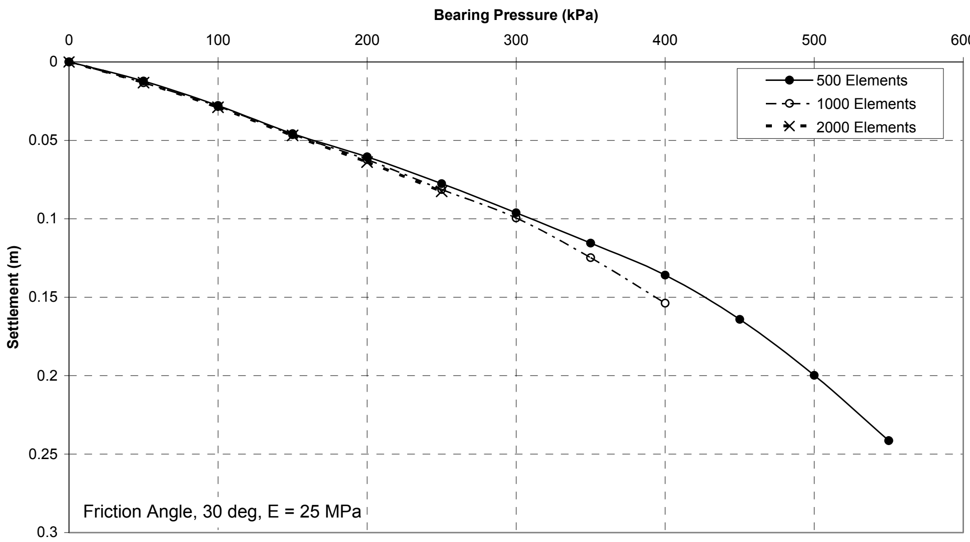


Figure 3.4: Effect of varying grid densities on FLAC plane strain model

Table 3.1: Computation times for varied grid density in FLAC

Grid Density	*i* x *j*	Number of elements	Approximate Computation Time*
Coarse	20 x 25	500	6 mins
Medium	28 x 35	980	25 mins
Fine	40 x 50	2000	1 hr 28 mins
Very Fine	80 x 100	8000	More than 3 hours

* Time taken for model to pause due to bad input geometry on a 3 m wide strip footing

There is marginal difference in the settlements over a typical bearing pressure range between each of the models. Interestingly, an increase in the number of elements produced a truncation of the pressure-settlement curve due to ‘bad geometry’ errors where the displacements became large in comparison with the smaller elements, causing excessive deformation of elements when running in large strain mode.

A grid density of 500 elements (shown in Figure 3.5) was considered to have the best balance between accuracy and computation time and was chosen for all further plane strain and axisymmetric modelling in FLAC.

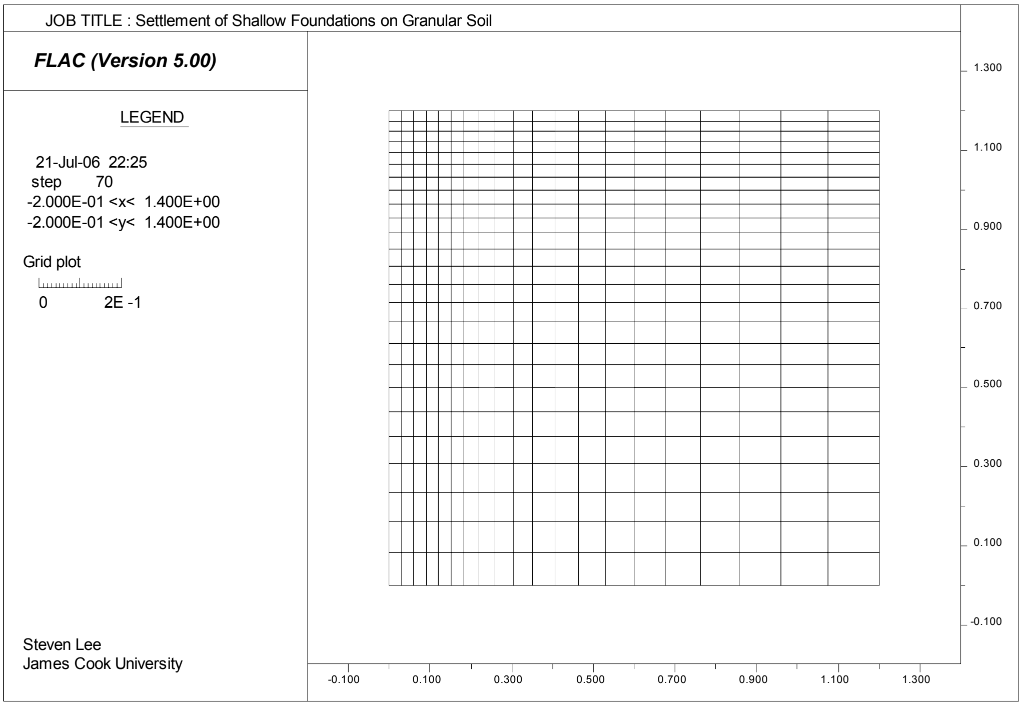


Figure 3.5: Mesh used in FLAC model

A similar analysis was carried out on a square footing using FLAC^{3D}. Brick elements were chosen for this model because this shape best suited the model geometry and allowed for ease of manipulation. These results are summarised below in Figure 3.6, Figure 3.7 and Table 3.2.

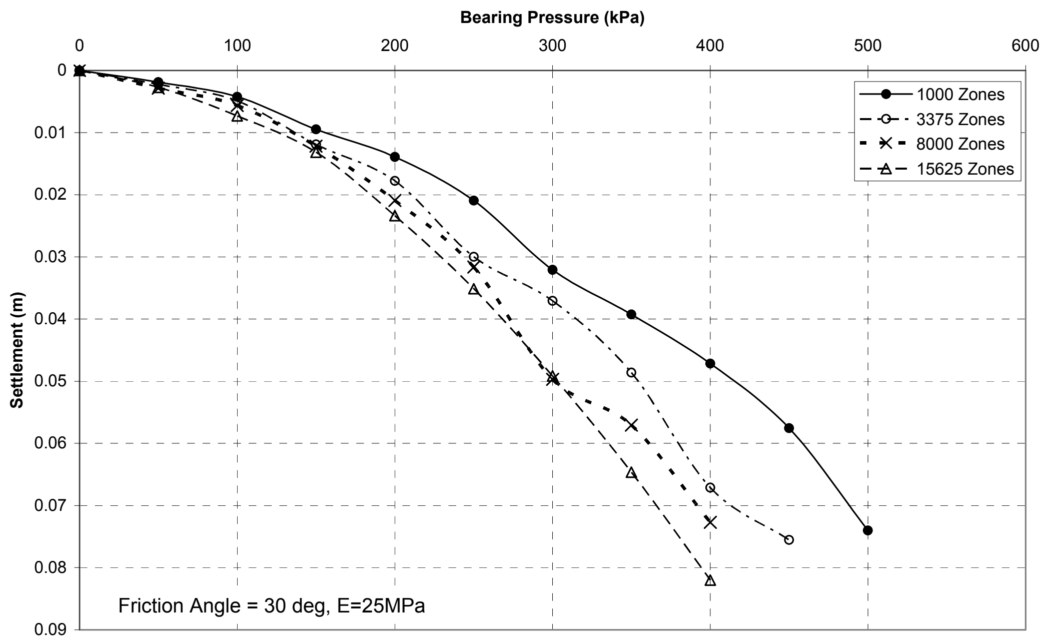


Figure 3.6: Effect of variations in grid density for $FLAC^{3D}$ square footing model

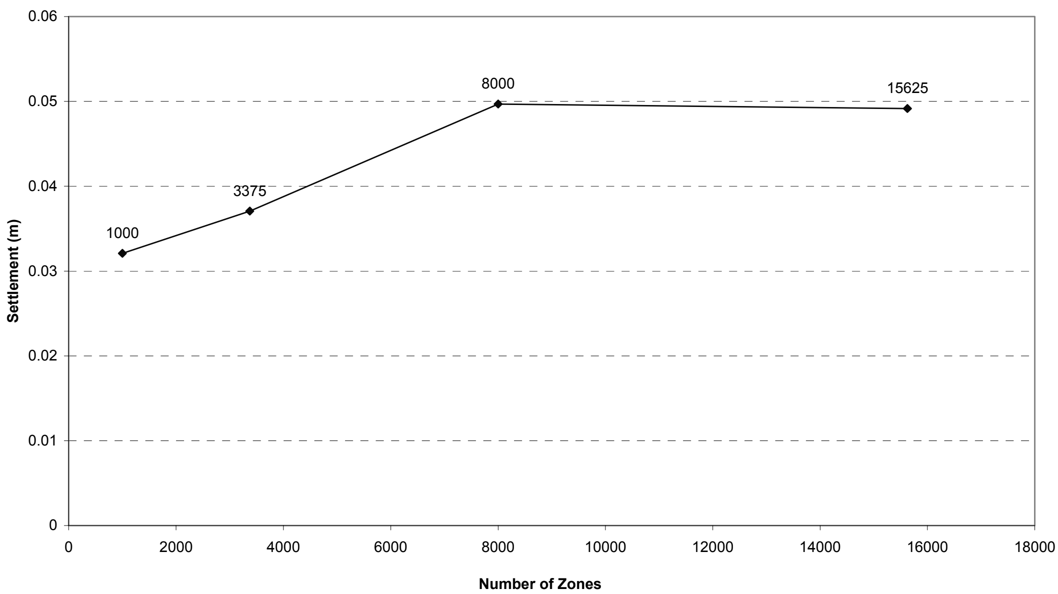


Figure 3.7: Sensitivity of mesh density for 300 kPa applied pressure in $FLAC^{3D}$ model

Table 3.2: Computation times for varied grid density in FLAC^{3D}

Grid Density	*i* x *j* x *k*	Number of zones	Approximate Computation Time
Very Coarse	10 x 10 x 10	1000	9 mins
Coarse	15 x 15 x 15	3375	35 mins
Medium	20 x 20 x 20	8000	1 hr 20 mins
Fine	25 x 25 x 25	15625	3 hrs

Figure 3.6 shows the variations in settlement predictions as the number of zones was varied when modelling in three dimensions. The coarse meshes also produced anomalies in the pressure-settlement curves, and convergence was considered to occur for 15625 zones. Computation times for a mesh of this size are reasonable. Therefore, for all predictions using FLAC^{3D}, 15625 zones were adopted, as shown in the figure below.

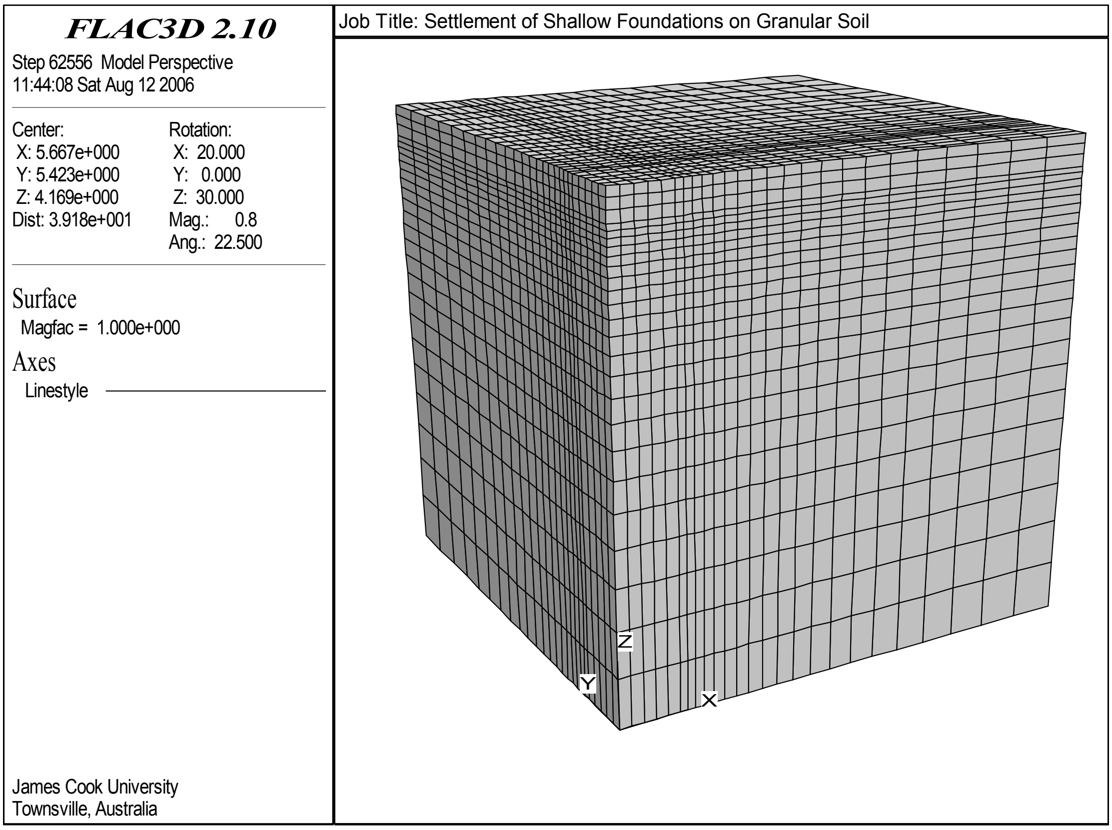


Figure 3.8: Mesh used in FLAC^{3D} model

3.2.3. Implementation of Initial and Boundary Conditions

Most numerical modelling techniques require boundary conditions to be applied to the model. The location and restraints at the boundaries will have some influence on the model results. Boundaries must be placed far enough away from the area of interest so that they do not have significant influence on the system being modelled.

Schmertmann's (1978) strain influence diagram (shown below) was used to determine a basis for the model geometry, assuming that the soil was homogenous and cohesionless. The influence depths and widths were tested for a strip footing. In accordance with the strain influence diagram, four times the footing width (4B) was chosen as the reference dimension.

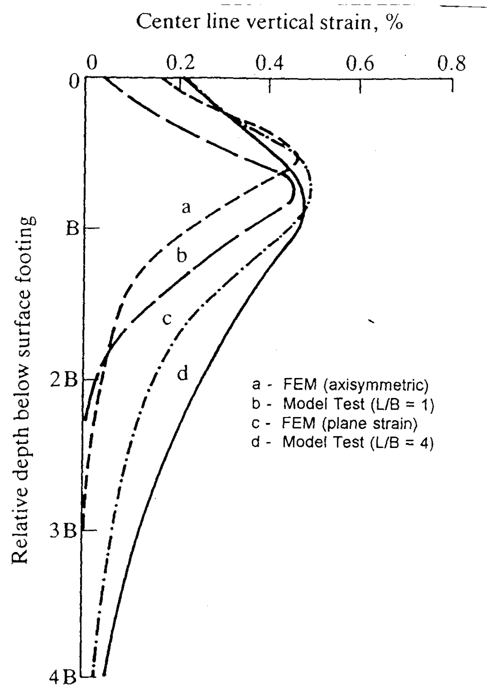


Figure 3.9: Vertical strain distributions and rigid model tests (After Schmertmann et al. 1978)

The effect of varying the influence depth was not significant. Depths of 2B, 4B and 8B were trialled, and these results can be seen in Figure 3.10. Because of the elongation of elements caused by the larger influence depth, it was considered that an influence depth of 4B was sufficient for strip footings. This influence depth was also used for analysis of the square and axisymmetric models, to ensure that the boundary extended beyond a depth of twice the footing width as suggested by Schmertmann's influence diagram.

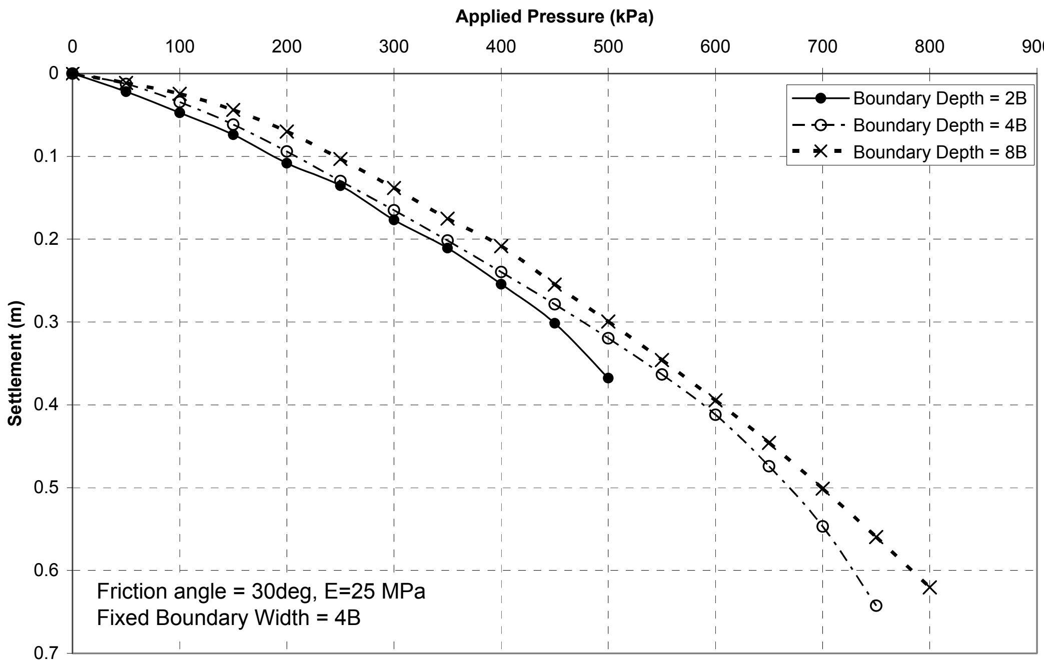


Figure 3.10: Influence of boundary depth on FLAC plane strain model

Interestingly, Figure 3.11 shows there was more of an effect when the boundary width was varied. This was somewhat unexpected, as most of the literature only mentions limits on boundary depth.

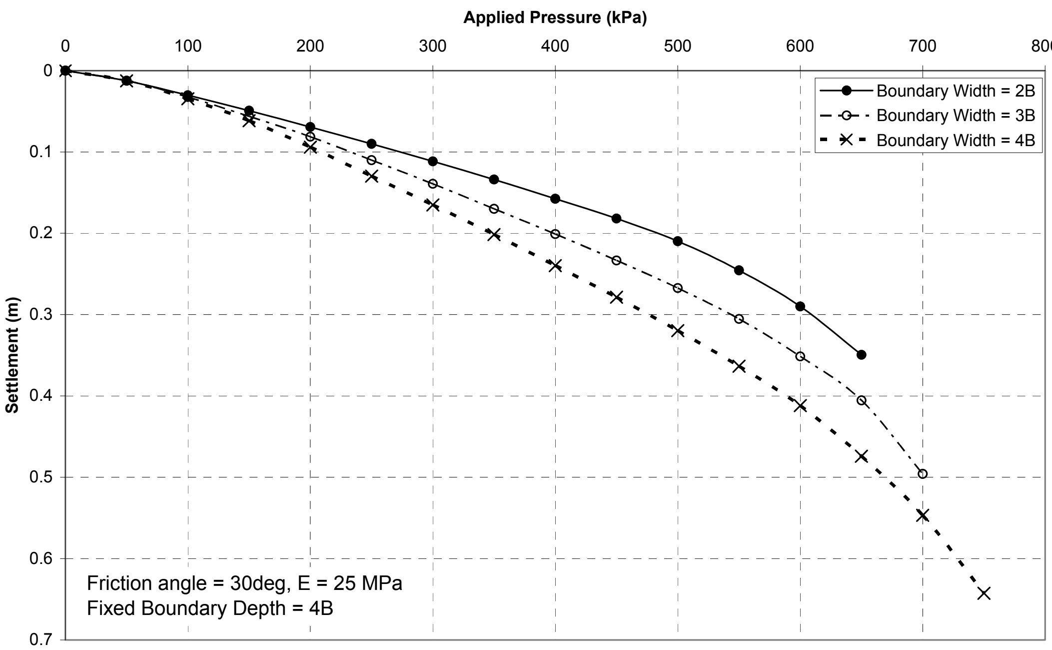


Figure 3.11: Influence of boundary width on FLAC plane strain model

These effects may have been produced by excessive elongation of grid elements (ideally elements should have an aspect ratio of no more than 5:1 so that computations can be carried out), or the effects may have been produced by restrictive lateral movements of the soil. It was decided that an influence width of 4B be used to be somewhat conservative. Because of the location of the boundaries, it was assumed that there would be very little difference between roller and pinned supports on the external boundaries.

The boundary conditions which were initially applied to the models can be seen in the following figures. The FLAC two-dimensional models have been developed so that they take advantage of geometric symmetry by only modelling one half of the system. The boundary conditions on the axisymmetric model can be seen in Figure 3.12. It is important to note that a similar restraint arrangement exists for the plane strain model.

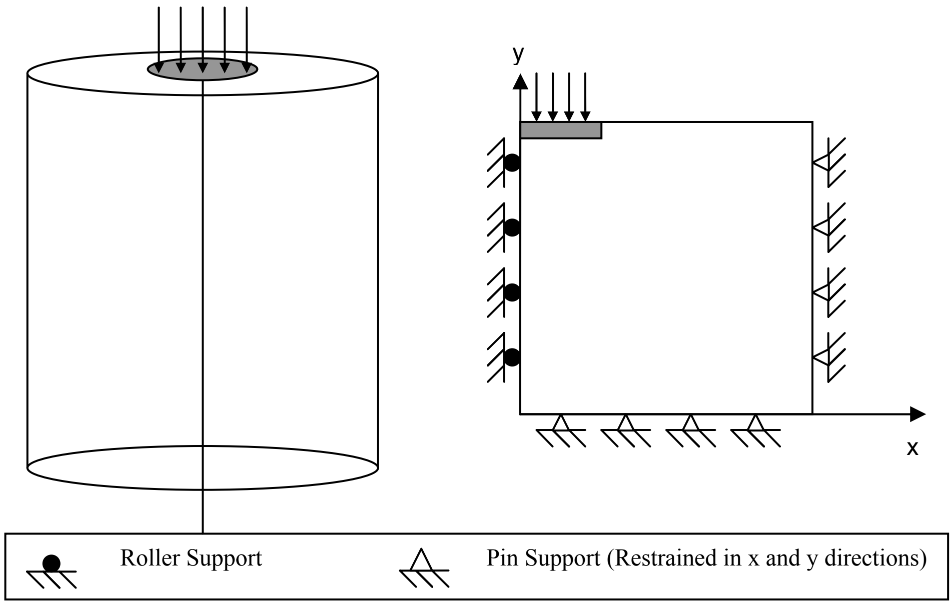


Figure 3.12: Boundary conditions applied to the two-dimensional models

Two planes of symmetry exist for square footing when modelled in three dimensions. This enables a quarter of the footing and surrounding soil to be modelled, decreasing overall computation time. A representation of this can be seen in the figure below.

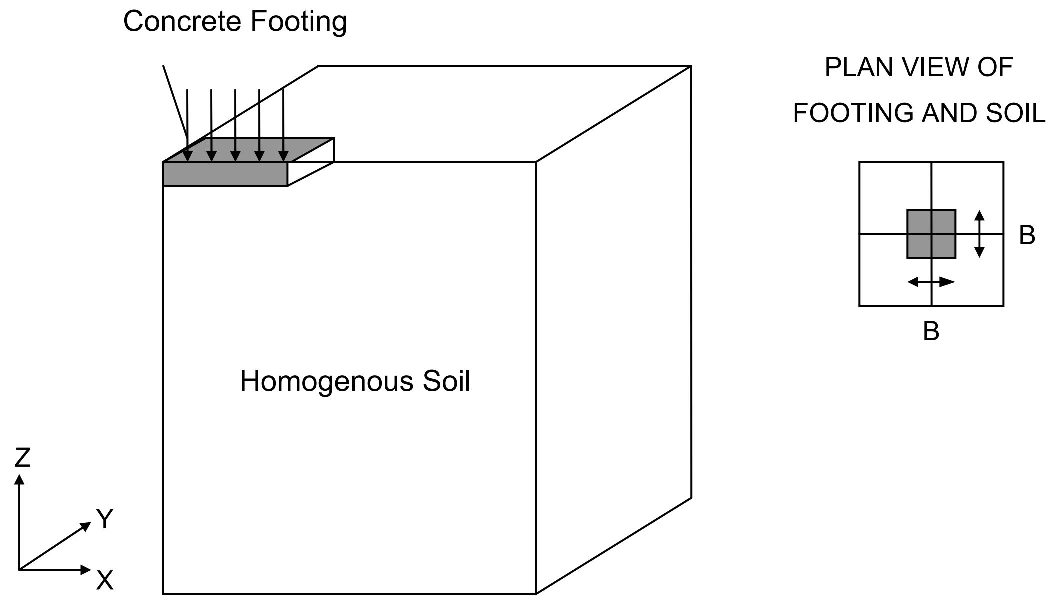


Figure 3.13: Modelling the square footing using quarter symmetry

The restraint arrangement for the three-dimensional model is similar to the fixings required for the two-dimensional models. Two planes are shown in Figure 3.14. Roller supports have been used along the planes of symmetry, and pin supports have been used on the external boundaries.

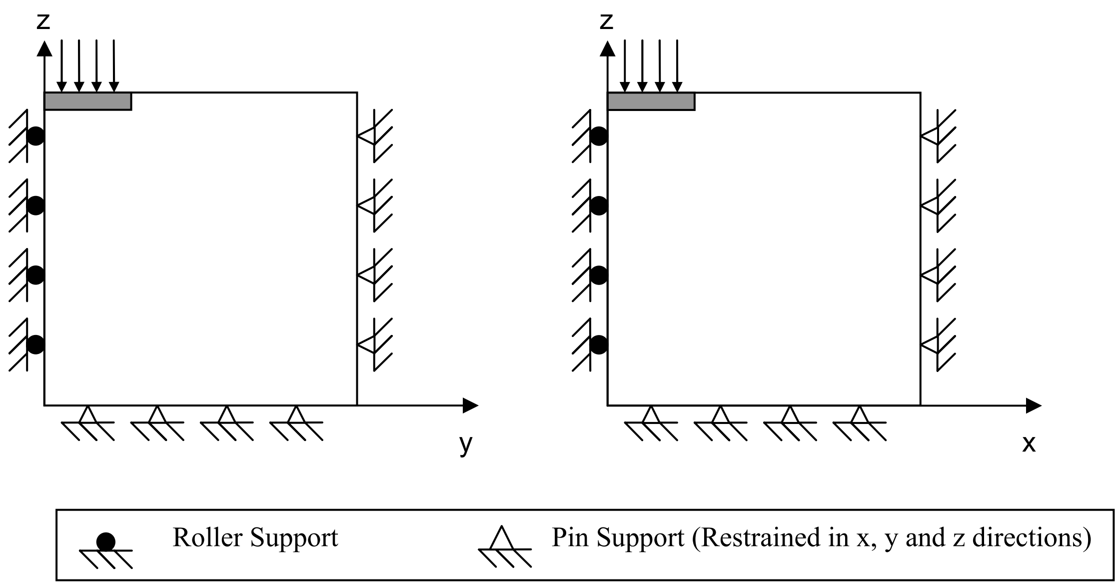


Figure 3.14: Boundary conditions applied to the FLAC^{3D} model

Initial conditions need to be established for the numerical model. Gravity was the only initial condition applied to all three of the models. By applying gravity and solving the model for equilibrium, initial stresses within the system were established prior to further loading of the underlying soil. In most geotechnical applications, vertical stress increases linearly with depth and could be confirmed by plotting the distribution of vertical stress over the grid.

The model was constructed so that the mechanical processes within the model corresponded with the physical processes associated with the construction of shallow foundations. Once the stresses within the soil were initialised, modelling of the excavation of the soil and construction of the foundation could commence. At this stage, the system was solved for equilibrium. The footing was then loaded incrementally by applying pressure and solving for system equilibrium. This continued until 1000 kPa loading or model failure, whichever occured first. Output data was then retrieved and interpreted.

An alternative approach to modelling the loading process is to apply a downward velocity to the area representing the foundation. This velocity induces the displacement within the soil. This method was not favoured because it does not offer the same level of control over the applied load as the previously mentioned method.

3.2.4. Material Property Definition

FLAC has many soil constitutive models incorporated within the program, including elastic models, or plastic models such as Mohr Coulomb, Drucker-Prager, strain-hardening or Hoek-Brown. Modelling the sand as a continuum using the Mohr-Coulomb constitutive model was considered to be the best choice for this application. The Mohr-Coulomb failure criteria is widely acknowledged as the failure mode for most soils. The concrete footing was modelled as an elastic medium, effectively ensuring that failure within the model occurred in the soil region and not within the concrete footing. Typical values of the model input parameters are shown in Table 3.3.

Table 3.3: Typical material properties

Property	**Concrete**	**Sand**
Density	2380 kg m^{-3}	1600 kg m^{-3}
Poisson's Ratio	0.15	0.2
Young's Modulus	29 GPa	25 MPa
Friction Angle	-	30°
Cohesion	-	0 kPa

FLAC requires that the soil parameters be expressed as bulk modulus (K) and shear modulus (G). The bulk modulus and shear modulus can be related to Young's modulus and Poisson's ratio by the following equations.

$$G = \frac{E}{2(1+\nu)} \tag{3.1}$$

$$K = \frac{E}{3(1-2\nu)} \tag{3.2}$$

In order for the model to be able to provide a solution to the problem, problem specific data needs to be assembled from full-scale load test data. This data includes the soil type, the test method used, results of the test methods, dimensions of the footing and the thickness of the soil stratum. From this data, model input parameters such as density, bulk modulus, shear modulus, friction angle and tensile strength can be found. Because of the uncertainties associated with the insitu state of stress, deformability and strength properties, it was assumed that the test results used in the model are representative of the sample area.

3.3 Model Refinement

Most of the settlement records available in the literature used SPT as the test method over CPT and other methods. Therefore, correlations relating the N-value to Young's modulus and friction angle have been incorporated within the model.

Control over the depth of footing embedment was introduced by separating the mesh into four sub-grids, consisting of the footing, the soil adjacent to the footing, and the soil below and surrounding the footing.

This FLAC model was then used as a basis for creating the FLAC^{3D} model. Because of the similarities in the programming language, the translation process was relatively simple. A similar process was used for the model development and refinement.

The boundary conditions were slightly modified to minimise the arching effect at the extreme boundaries of the model. The pin restraints above were changed to roller supports so that the distribution of stresses in the z-direction was linear.

3.3.1. SPT Correction Factors

Measured N-value or 'blow count' is the number of blows required for 300 mm penetration. Depth correction was applied to the N-value to produce N_1 by applying an overburden correction factor.

$$N_1 = C_N . N \tag{3.3}$$

$$C_N = 9.78\sqrt{\frac{1}{\sigma'_{vo}\,(kPa)}} \quad \text{… Liao and Whitman (1986)} \tag{3.4}$$

Energy correction was applied as N_{60}, where correction was made for the efficiency of the rig. Hammer efficiency (η) was assumed to be 60%.

$$N_{60} = N \times \frac{\eta}{60} \tag{3.5}$$

If both energy and depth corrections are applied, both factors can be applied to the field blow count to convert it to the corrected blow count, $(N_1)_{60}$.

3.3.2. Young's Modulus and Friction Angle

Young's modulus can be easily related to commonly used site investigation results such as Standard Penetration Test (SPT) and Cone Penetration Test (CPT) by using empirical relationships. Hundreds of correlations exist for soil test – soil property relationships, however not all of these are reliable. This suggests that if there are many correlations, there are many interpretations of the field test data.

To calculate the elastic modulus of the soil, Leonards (1986) suggests that $E = 0.8(N_{60})$. The relationship between SPT N-values and the angle of shearing resistance established by Peck *et al.* (1967) (vide Tomlinson 1995) is used for ordinary foundation design and can be seen in Figure 3.15. These SPT-E-φ correlations were chosen in preference to all of the other available correlations because of their common application to problems involving shallow foundations in granular soils.

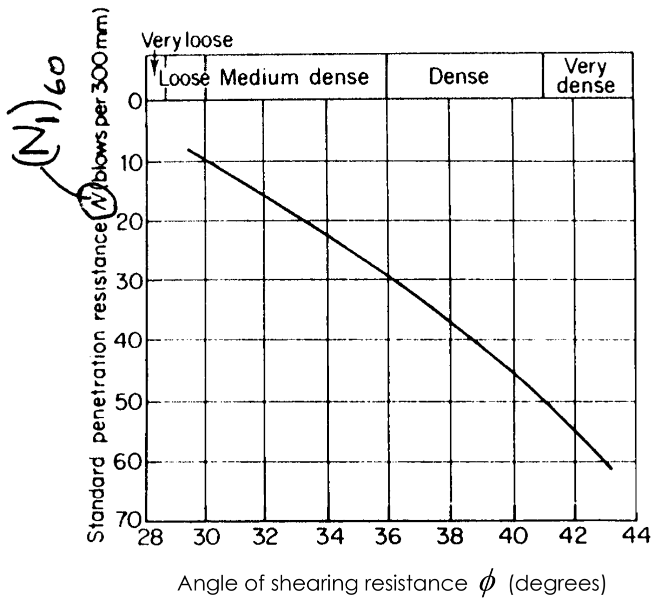


Figure 3.15: Relationship between SPT N-value and angle of shearing resistance (after Peck *et al.* 1967)

3.4 Model Validation

Model tests could be completed in the laboratory to validate the numerical model. However, this is considered to be a reasonably massive task and is unnecessary because of the abundance of full scale data available in literature. Settlement records for 79 foundations are available using case studies by Jayapalan and Boehm (1986) and Papadopoulos (1992). For this thesis, pressure-settlement curves produced by Terzaghi and Peck (1948) were used to validate the FLAC and FLAC^{3D} models for square (and circular) footings by comparing the results produced by the numerical model when using the soil and geometric properties which were used in the experiments. Simulation of the loading of the foundation is to be based on these experiments.

Some assumptions about the experimental process carried out by Terzaghi and Peck were required due to lack of information available. These include assuming a typical density of 1600 kg m^{-3} for the sand and assuming that the plate had a thickness of 25 mm. It was also assumed that the SPT N-values were uncorrected and taken at a depth B below the footing. Therefore depth correction was applied according to this assumption for these trials. The results of this investigation showed good correlation between experimental data and the modelled data for both the axisymmetric models and the FLAC^{3D} model for a loose soil (see Figure 3.16).

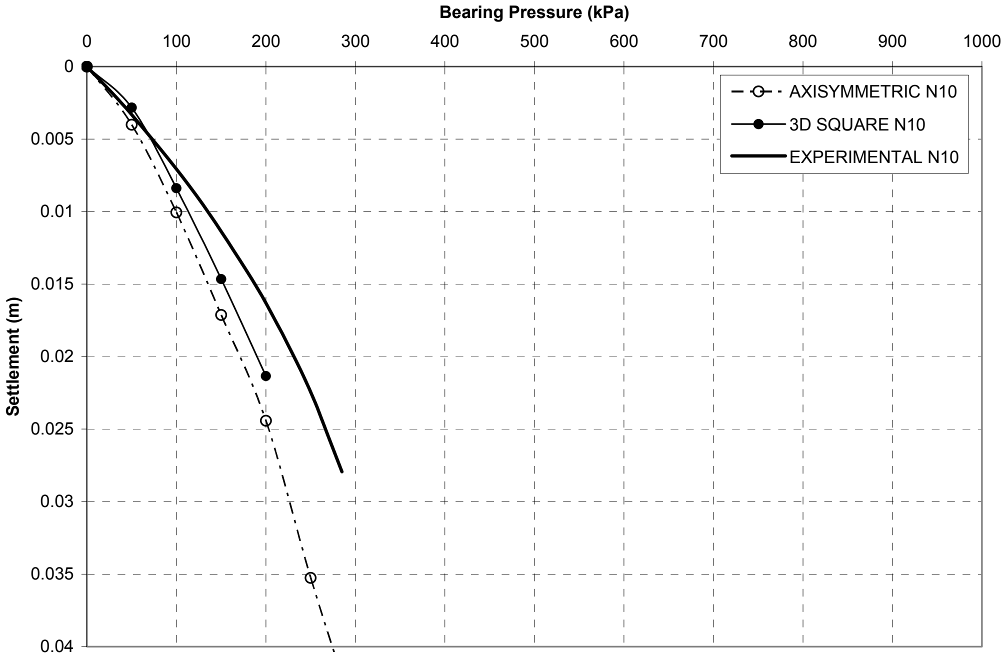


Figure 3.16: Comparison of axisymmetric and FLAC^{3D} models against Terzaghi and Peck experimental data for N=10

For stiff soils (corresponding to N=30 and N=50), the deviation from the experimental results increased as shown in Figure 3.17 and Figure 3.18. This may have been due to the assumptions made for corrections of the SPT N-values and the choices of SPT-E-φ empirical correlations used within the model. Although these are well founded, they are usually applied to full scale footings.

The graphs were prepared so that the FLAC and FLAC^{3D} results are shown on the same chart. This is to assist in the investigation to determine if FLAC can be used to accurately model settlements of square pad footings as circular footings. This may eliminate the need to purchase the more expensive FLAC^{3D} which has much longer computation times. These results showed good correlation and will be further investigated in the next chapter.

For this problem, the results and correlation to measured data were considered to be satisfactory. It is known that the problem which the model is to be applied to has a loose upper stratum of soil. Therefore, no modifications will be made to the empirical relationships or soil model, and it can be said that the models are validated.

It was decided to abandon further investigation of the plane strain model at this stage of the project, as there was an absence of full load-settlement curves available for comparison, and only a few documented settlement records for strip footings.

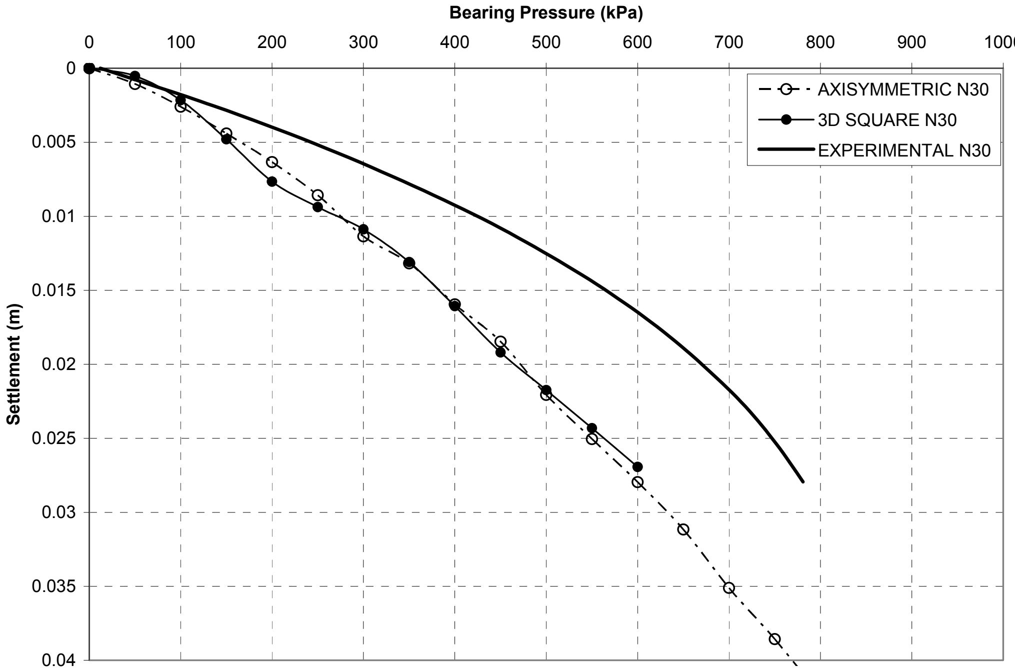


Figure 3.17: Comparison of axisymmetric and FLAC^{3D} Models against Terzaghi and Peck experimental data for N=30

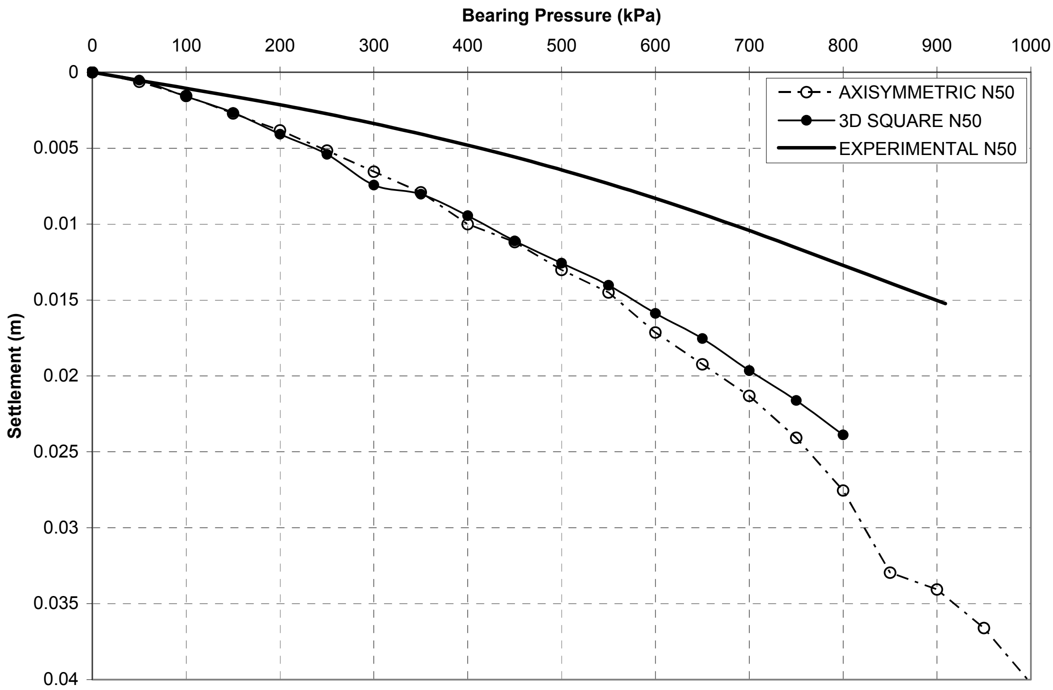


Figure 3.18: Comparison of axisymmetric and FLAC^{3D} models against Terzaghi and Peck experimental data for N=50

3.5 Sensitivity Analysis of Material Properties

An advantage of numerical modelling over other modelling techniques is that specific parameters can be controlled. This was used to perform a sensitivity analysis of the effects of material properties on the settlement of a shallow foundation on granular soil. The trials were based on the same scale model which was used for model validation. Four material properties were chosen and varied for observation. These are summarised in the table below.

Table 3.4: Varied material properties

Parameter	Density (kg/m^3)	Poisson's ratio	Friction Angle (degrees)	Young's Modulus (Pa)
Standard	**1600**	**0.2**	**30**	**25 x10^6**
Density	*1400*	0.2	30	25 x10^6
	1800	0.2	30	25 x10^6
	2000	0.2	30	25 x10^6
Poisson's Ratio	1600	*0.1*	30	25 x10^6
	1600	*0.3*	30	25 x10^6
Friction Angle	1600	0.2	*28 (v. loose)*	25 x10^6
	1600	0.2	*36 (dense)*	25 x10^6
	1600	0.2	*42 (v.dense)*	25 x10^6
Young's Modulus	1600	0.2	30	*8 x 10^5 (N10)*
	1600	0.2	30	*40 x 10^6 (N50)*

By understanding the effects of each of the properties acting alone, the SPT-E-φ correlations may be modified to produce a more accurate result. The effect of the variations on the load-settlement plots were observed in the axisymmetric, plane strain and FLAC^{3D} models. It is important to note that in reality, some of the combinations of properties would not exist, and results are intended to indicate <u>trends</u> only.

Varied Density

A range of soil densities were trialled ranging from 1400 kg m^{-3} for loose soils to 2000 kg m^{-3} for dense soils. Densities were increased in 200 kg m^{-3} increments. In all of the models, the point of yield in the soil can be recognised by the sudden change in gradient of the curve where the soil stops behaving elastically and undergoes plastic deformation. Interestingly for all of the models, the variation in density appears to have no effect on the settlement of the foundation while the soil is behaving elastically. However, the bearing capacity of the soil increases as the density increases as was expected. The two-dimensional models appear to respond to the variations in soil density more than the FLAC^{3D} model.

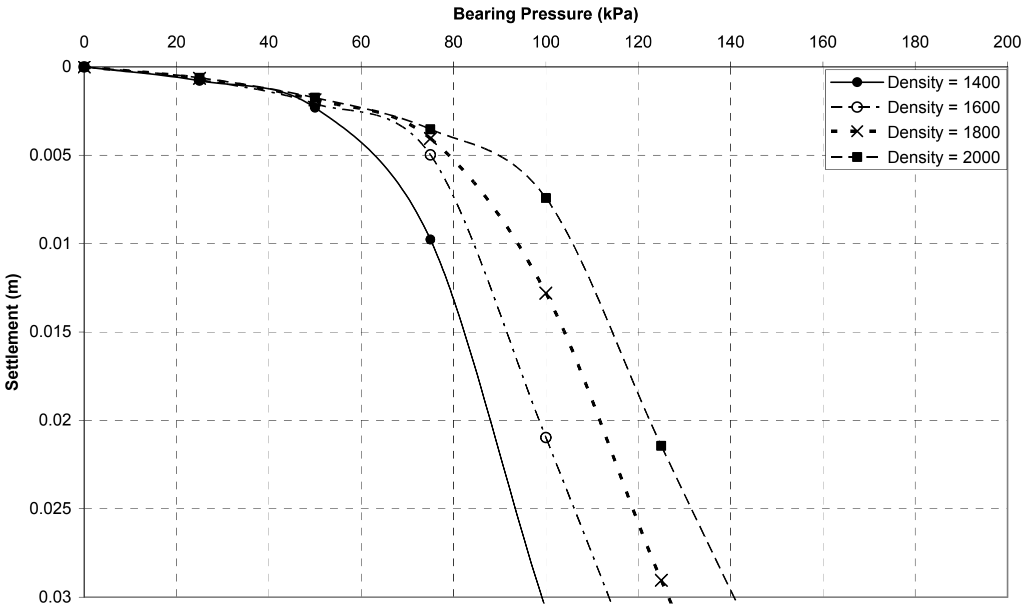


Figure 3.19: Effect of varied density on FLAC axisymmetric model

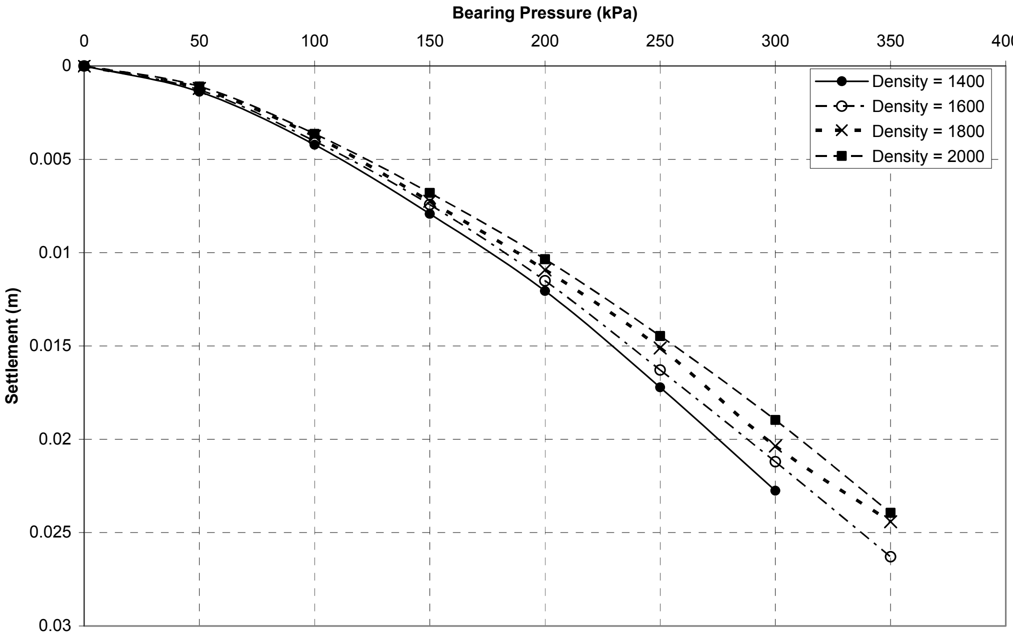


Figure 3.20: Effect of varied density on FLAC^{3D} model

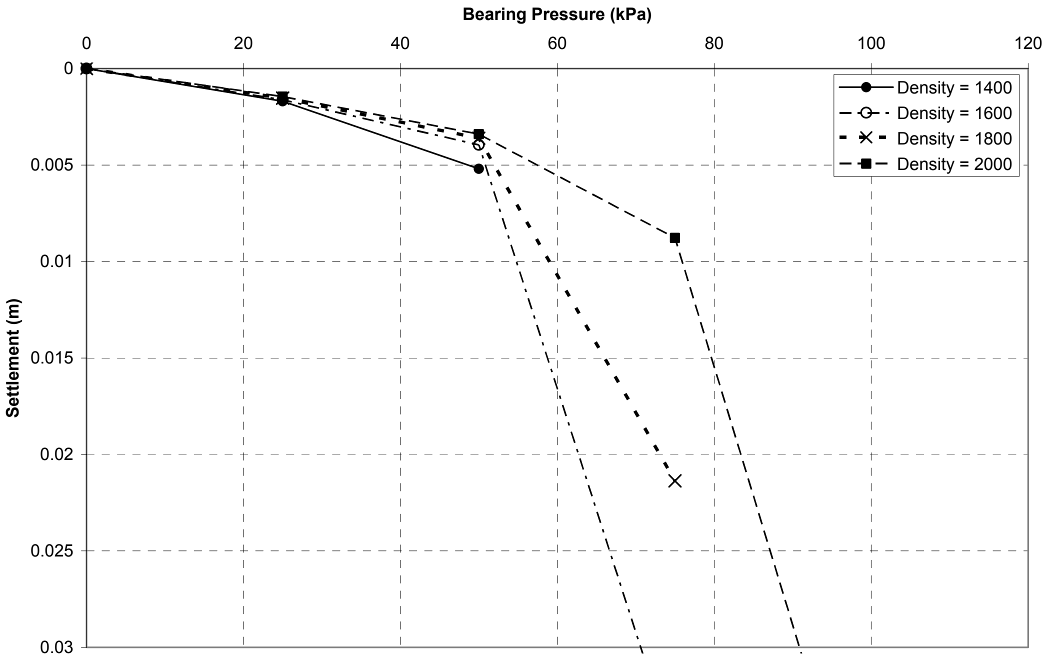


Figure 3.21: Effect of varied density on FLAC plane strain model

Varied Poisson's Ratio

Poisson's ratio was chosen as a property to vary because the bulk and shear moduli inputs required by FLAC are functions of both the elastic modulus and Poisson's ratio. It is therefore expected that a variation in Poisson's ratio will produce some variation in the pressure-settlement behaviour of the foundation. Three trials were performed for Poisson's ratios of 0.1, 0.2 and 0.3; which may be considered to be within the typical range of values for sands. There appeared to be little difference between the pressure-settlement curves for the axisymmetric (Figure 3.22) and plane strain models (Figure 3.24). However, the response for the FLAC^{3D} model (shown in Figure 3.23) was greatly influenced by these changes. The difference in the calculated shear moduli produced by varying Poisson's ratio from 0.1 to 0.3 was small (2 MPa) compared to the differences in the calculated bulk moduli (10 MPa).

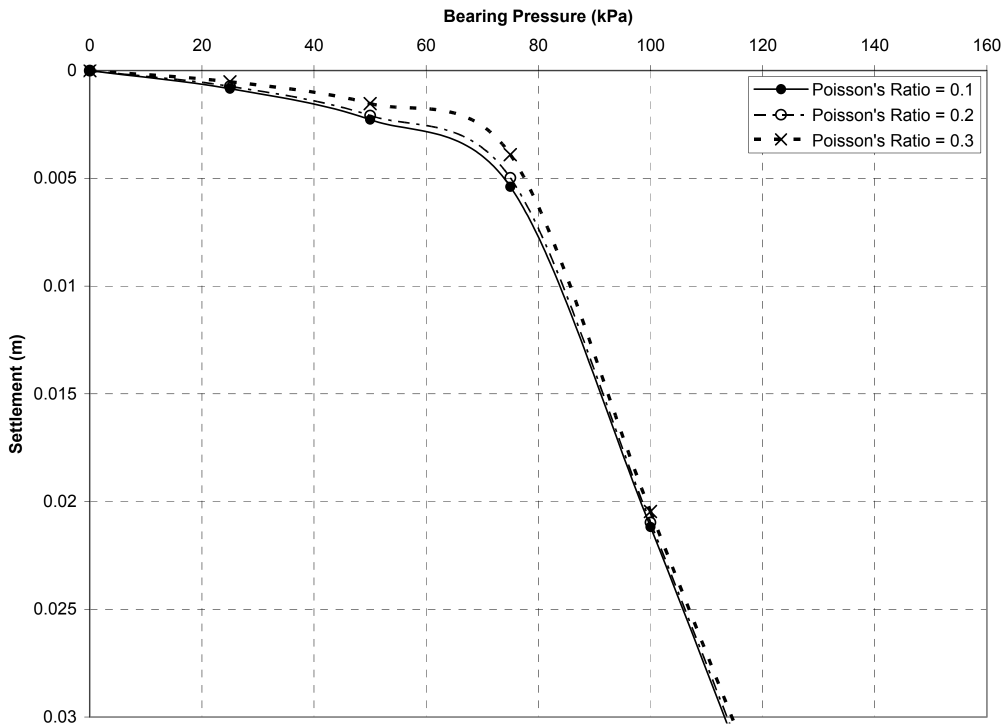


Figure 3.22: Effect of varied Poisson's ratio on FLAC axisymmetric model

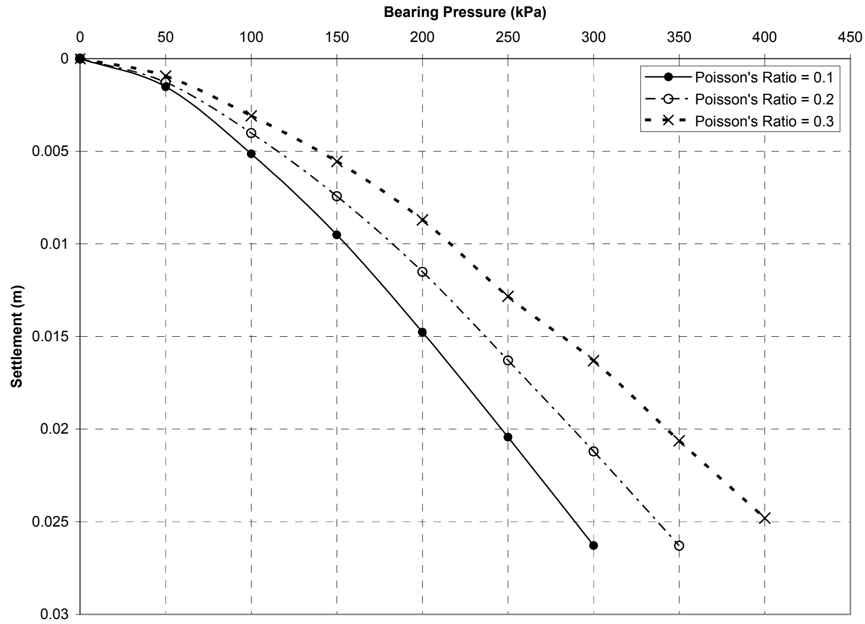


Figure 3.23: Effect of varied Poisson's ratio on FLAC^{3D} model

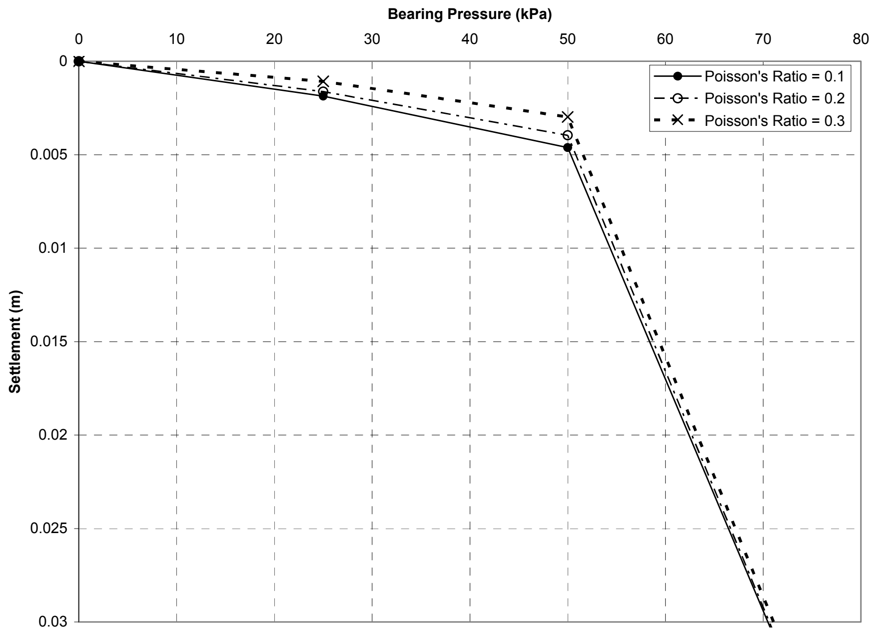


Figure 3.24: Effect of varied Poisson's ratio on FLAC plane strain model

Varied Friction Angle

The friction angle of a soil can often be loosely related to the relative density of a soil. Four different friction angles were trialled. 28˚ corresponded to very loose soil, 30˚ for loose/medium dense soil, 36˚ for dense soil and 42˚ for very dense soil. The behaviour of the system displayed outcomes similar to the variation of the SPT N-values of a soil. This was expected due to the strong correlation between SPT N-values and the friction angle of a soil. Soils with higher friction angles had a higher bearing capacity for a given settlement than a soil with a lower friction angle. This behaviour was consistent for all models and can be seen in Figure 3.24 to Figure 3.26.

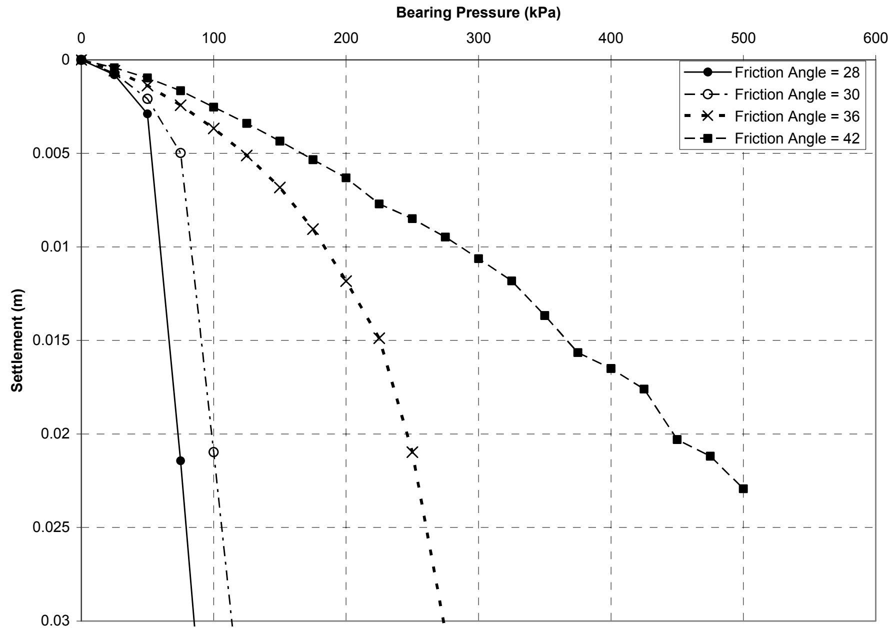


Figure 3.25: Effect of varied friction angle on FLAC axisymmetric model

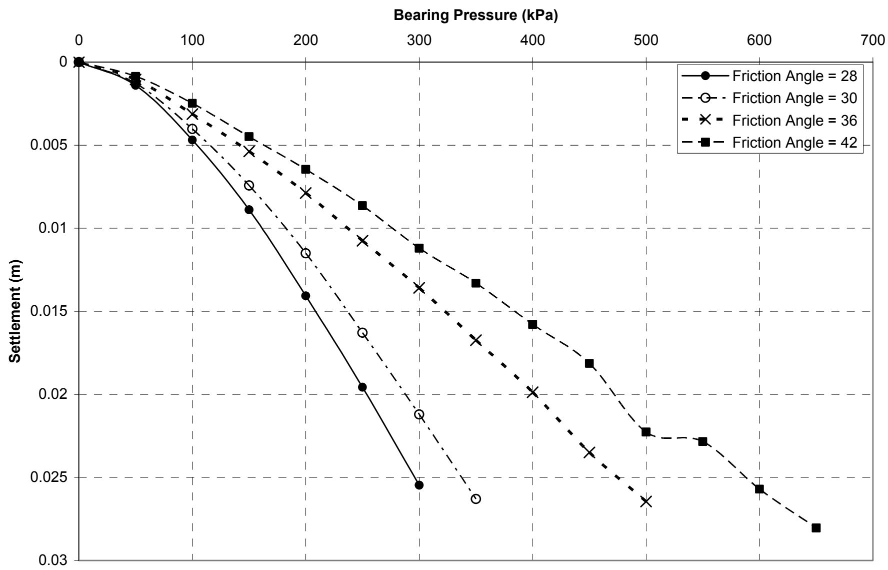


Figure 3.26: Effect of varied friction angle on FLAC^{3D} model

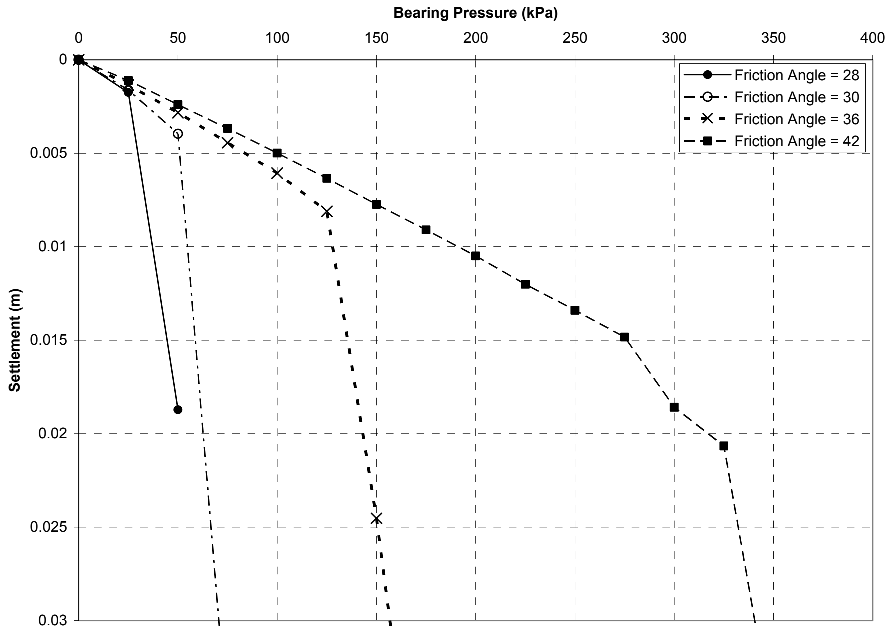


Figure 3.27: Effect of varied friction angle on FLAC plane strain model

Varied Young's Modulus

The Young's Modulus of a soil can also be related to the relative density of a soil. Based on the correlation between Young's Modulus and the SPT N-value of a soil, the variation of Young's modulus corresponds to N=10, N=30 and N=50. The results showed that soils with a higher elastic modulus had a higher bearing capacity for a given settlement. The resulting behaviour was similar to the behaviour produced by varying the friction angle, due to the SPT-φ-E relationships previously mentioned.

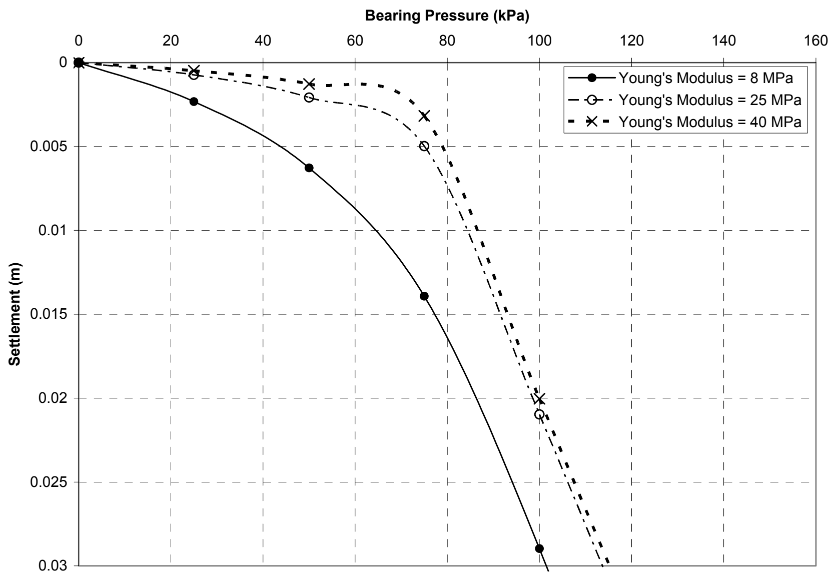


Figure 3.28: Effect of varied Young's modulus on FLAC axisymmetric model

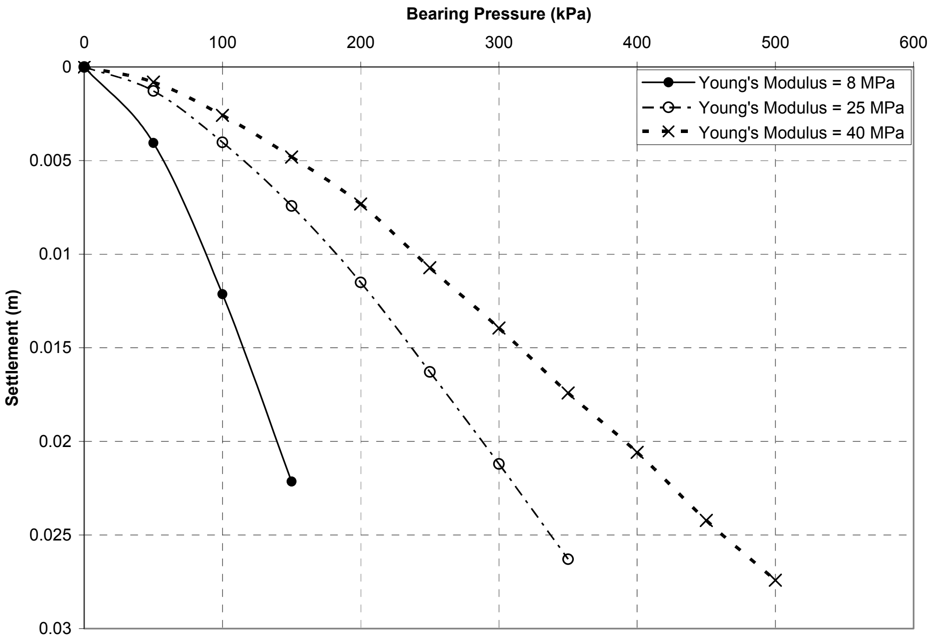


Figure 3.29: Effect of varied Young's modulus on FLAC^{3D} model

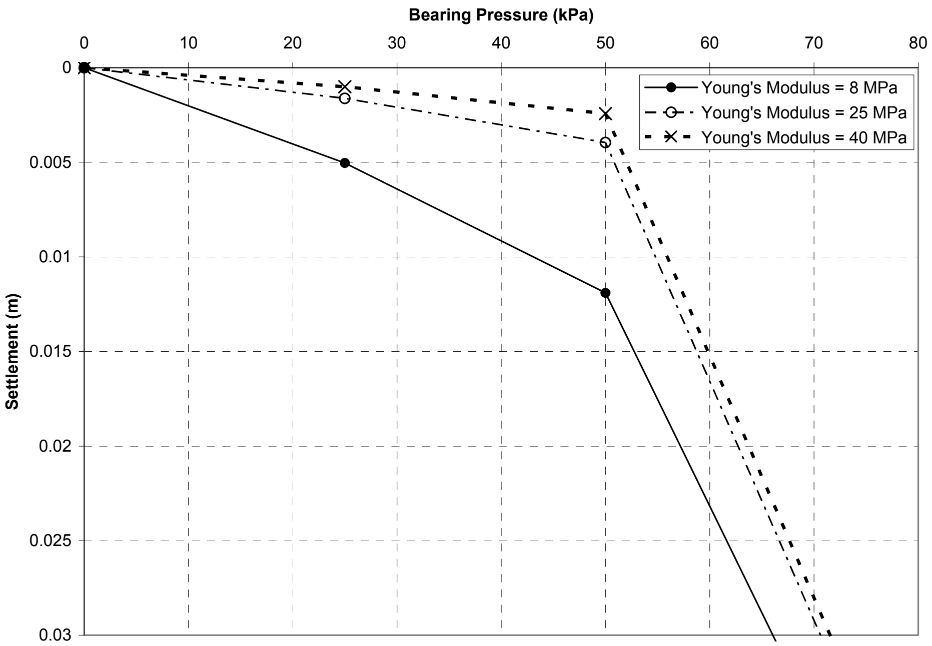


Figure 3.30: Effect of varied Young's modulus on FLAC plane strain model

3.6 Summary

Within this thesis, a numerical model was proposed and developed using FLAC and FLAC^{3D}. The development process was performed in accordance with some steps which were considered to be appropriate for geotechnical engineering applications. The approach recognises that field data will never be completely known, and incorporates empirical relationships which were chosen on the basis of their use in practice. The structure of the model was designed such that the physical sequence of events associated with the construction of a shallow foundation could be simulated. This was implemented by creating a simple model.

For correct interpretation of the obtained results, indicators were incorporated within the model to ensure convergence with respect to time. This included the investigation of optimum mesh size and boundary conditions to ensure accurate solutions could be obtained.

Validation of the model for circular and square footings was performed by modelling Terzaghi and Peck's (1948) pressure-settlement experiments and comparing the results. The model performed well when compared to the measured data for loose soils, while less accurate results were obtained for stiffer soils. Because of the lack of validation data for strip footings, the study was concentrated on the modelling of square foundations.

The model was then used as a 'numerical laboratory' by varying specific soil property parameters and investigating the trends which were produced in a sensitivity analysis. This aided in understanding the effects that could be produced by modifying or choosing different empirical relationships or assumptions made about the material properties of the sand. Using this study as a basis, it was decided that no modifications were required for the SPT-E-φ correlations employed within the model, as the specific problems being modelled within this thesis are known to have a top soil stratum that is relatively soft.

Chapter 4: Application of the Numerical Model

4.1 Introduction

In this chapter, the model's performance will be examined by predicting the behaviour of the five spread footings analysed at the Settlement '94 conference held at Texas A&M University. Predictions will be made for loads that produce 25 mm and 150 mm settlement. The performance of the model will be assessed and ranked amongst the other participants.

4.2 Foundation Dimensions and Soil Data

Five footings were constructed for the investigation: two 3 x 3 m footings, one 2 x 2 m footing, one 1.5 x 1.5 m footing and one 1 x 1 m footing. The footings will be identified as shown in the figure below. All footings are founded at a depth of 0.76 m in the sand and are 1.2 m thick.

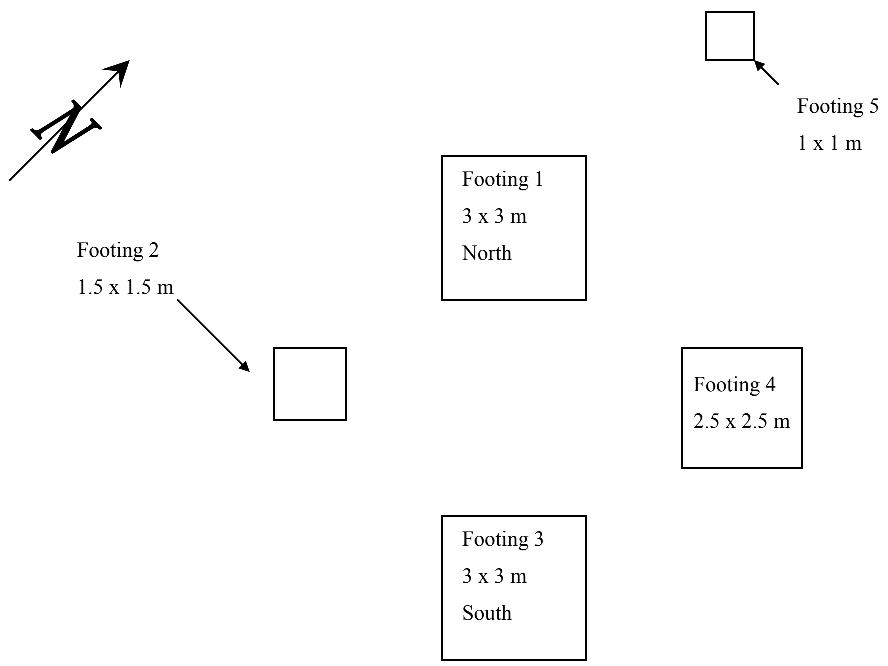


Figure 4.1: Footing layout

The insitu soil tests carried out at the test site were: CPT, PMT, SPT with Energy, DMT with axial force, Step Blade, Borehole Shear and Cross-Hole Wave tests. The laboratory soil tests which were conducted include Index Property Tests, Triaxial Tests and Resonant Column Tests. In this model, the SPT data was predominantly used as the basis for determining the required input parameters. This was because standard penetration tests are more commonly performed than in practice than CPT and other tests because of their lower cost. Locations of the test sites can be seen in Figure 4.2.

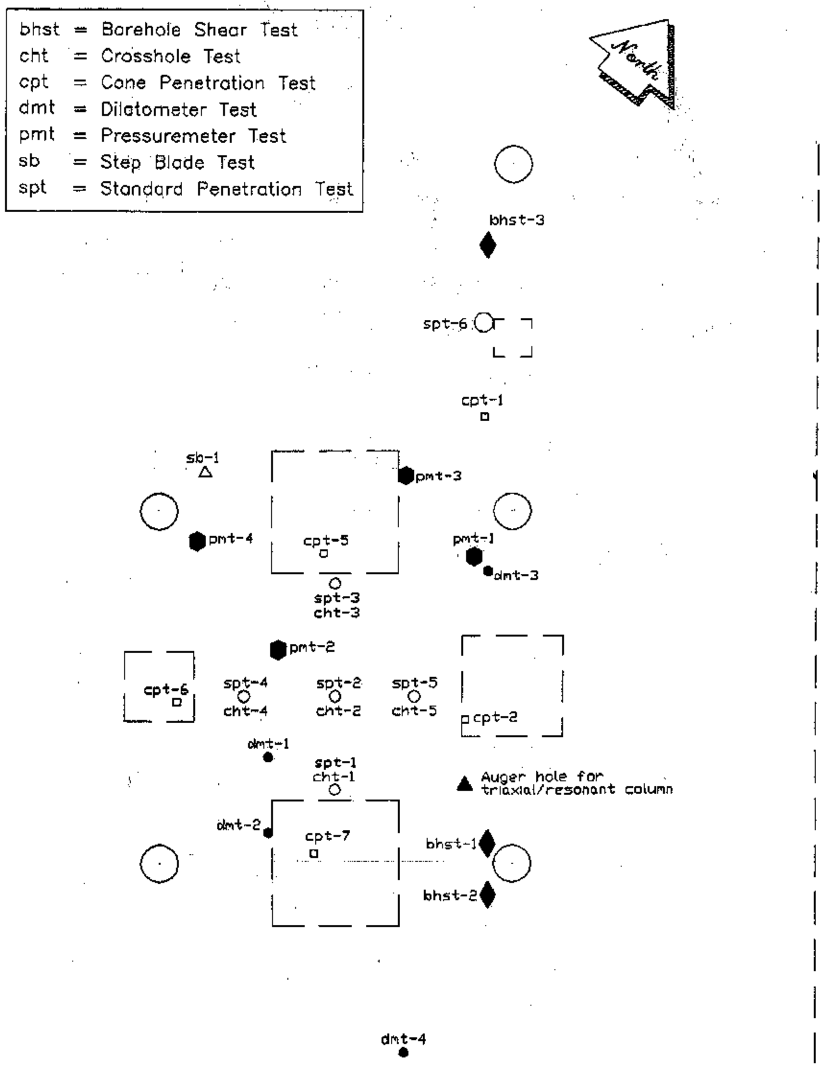


Figure 4.2: Field testing layout (Briaud and Gibbens 1994)

The soil profile was determined from the boring logs provided in the prediction package sent to participants. This was used in conjunction with the SPT results which may be found in Appendix A. Distinct layers were identified and an average N-value was determined for each apparent layer. Overburden correction was made at the middle of each defined layer rather than at a depth B below the footing due to the stratification of the soil. For all of the footings, the water table was considered to be 4.9 m below the ground surface. It was assumed that the measured N-values were influenced by the presence of the water table, therefore no further corrections were applied.

The soil profiles for each footing are shown in Table 4.1. SPT test location numbers are also shown as a reference to Figure 4.2.

Table 4.1: Soil profiles

<u>Footing 1</u> 3 m x 3 m North SPT-3	GL 5 m N=20 (sand) 4 m N=18 (sand) 3 m N=50 (clay) LIMIT OF MODEL
<u>Footing 2</u> 1.5 m x 1.5 m SPT-4	GL 5 m N=15 (sand) 1 m N=17 (sand) LIMIT OF MODEL
<u>Footing 3</u> 3 m x 3 m South SPT-1	GL 5 m N=25 (sand) 4 m N=20 (sand) 3 m N=50 (clay) LIMIT OF MODEL
<u>Footing 4</u> 2.5 m x 2.5 m SPT-5	GL 5 m N=20 (sand) 4 m N=22 (sand) 1 m N=45 (clay) LIMIT OF MODEL
<u>Footing 5</u> 1 m x 1 m SPT-6	GL 4 m N=20 (sand) LIMIT OF MODEL

4.3 Predicting the Settlement

Predictions were made for the five footings with the FLAC and $FLAC^{3D}$ models using the geometric and soil properties established in Chapter 4.2 combined with the empirical relationships established in Chapter 3. For each model, pressure was applied to the footing in 50 kPa increments and the settlement which occurred directly beneath the footing was recorded. Predictions for loads which would result in 25 mm (Q_{25}) were made using the square and circular footing models. An unknown error in the $FLAC^{3D}$ model caused premature termination of the load-settlement plots for the five footings after a certain loading or settlement value. Therefore, only the axisymmetric FLAC model was able to produce a load settlement curve which extended to 150 mm of settlement (Q_{150}).

For 150 mm of deformation, it was assumed that the soil would display plastic deformation. Therefore it may be reasonable to assume that the load which would result in 150 mm of settlement would be approximately equal to the ultimate bearing capacity of the soil. This was used as a basis to check if the FLAC model was accurately predicting Q_{150}. The results showed that there was significant difference between the bearing capacity and Q_{150}. It was decided to use the ultimate bearing capacity as a representation of Q_{150} for comparison to the measured results and other predictions.

Contours of the displacement taken from the $FLAC^{3D}$ model for Footing 4 can be seen in Figure 4.3. The results show that displacement varies within the soil in a 'bulb' like fashion, as expected. The results also show that there are no displacements within the proximity of the boundaries. The red regions show zones of high displacements, concentrated within and beneath the footing. The plot shows the final data point captured for the footing, which can be considered to be close to the yield point of the load-settlement for the footing.

To confirm that the pressure being applied to the footing was being transferred to the soil, a contour plot of stress in the z-direction was produced (Figure 4.4). The strains of a material can be related to the stresses of a material, therefore the stresses directly beneath the footing should behave according to Schmertmann's (1970) distributions of vertical strain below the centre of a loaded area. The general shape can be confirmed from the contour plot along the centreline of the foundation. Stresses appear to increase up to a depth approximately *B* beneath the footing before decreasing in magnitude to the surrounding initial vertical stress.

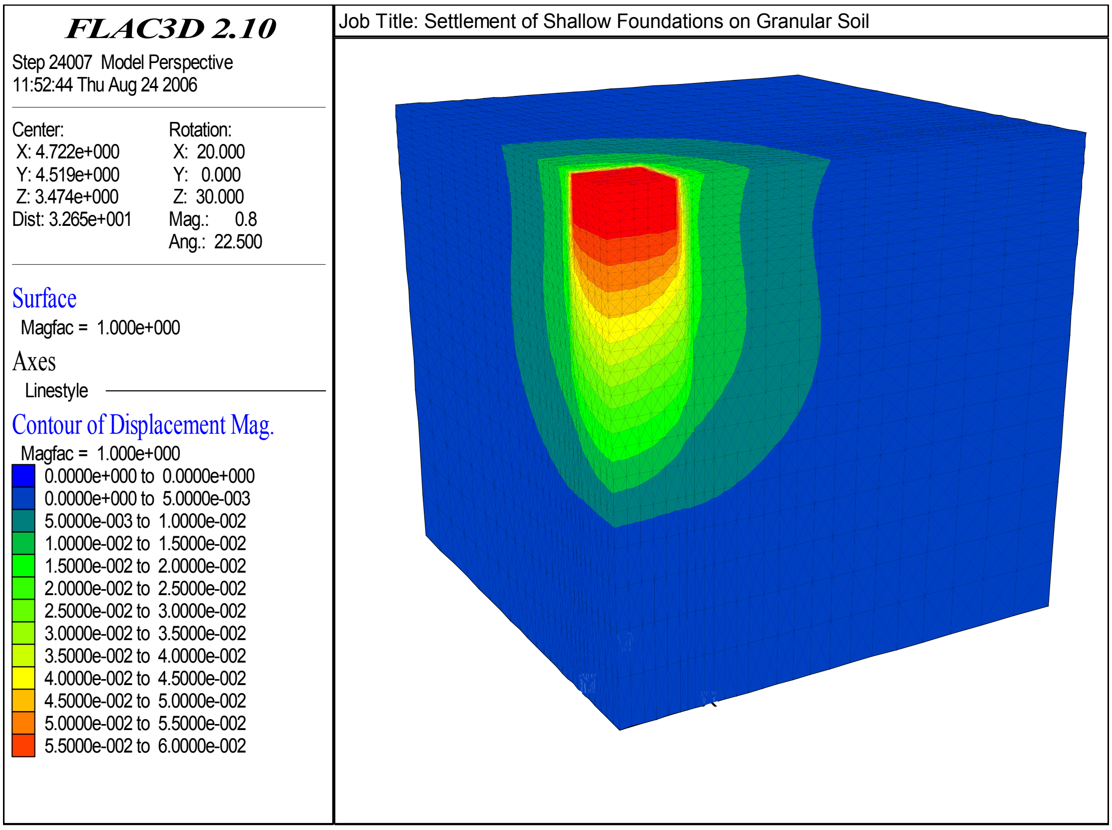


Figure 4.3: FLAC^{3D} contour of displacement for Footing 4

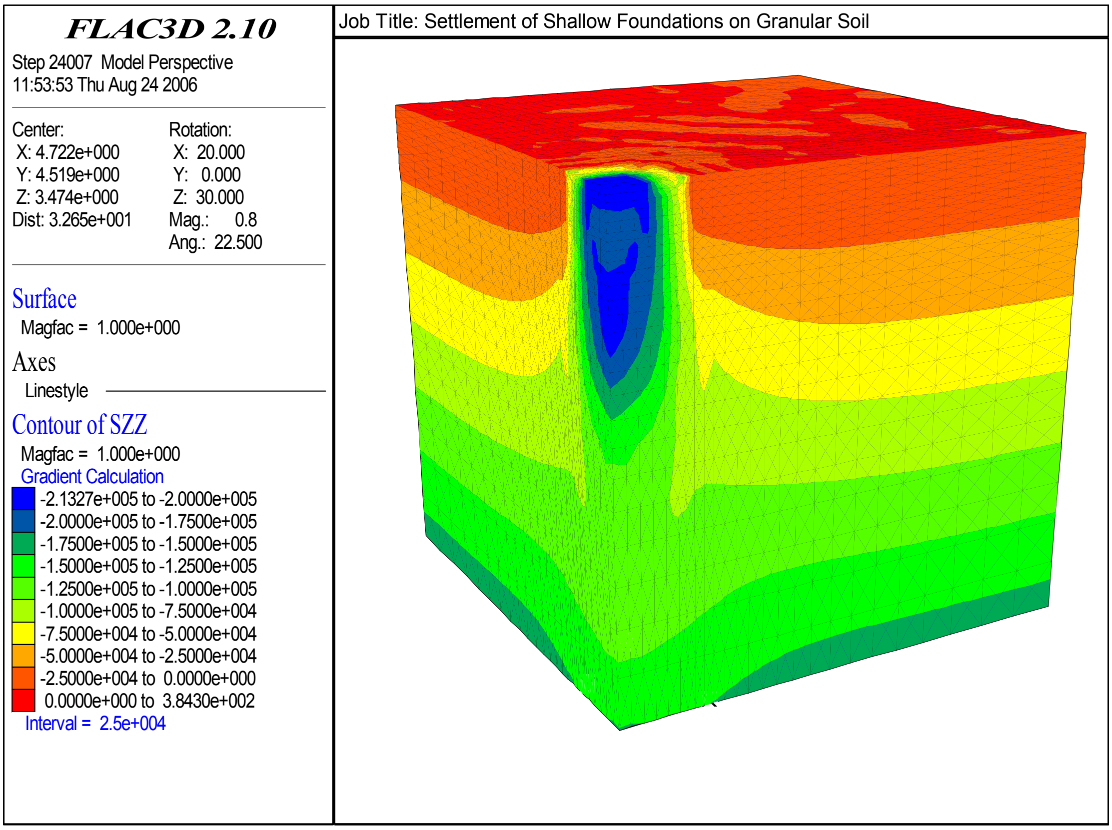


Figure 4.4: FLAC^{3D} contour plot of stresses in the z-direction for Footing 4

The predictions for Q_{25} and Q_{150} are summarised below for each of the footings.

Table 4.2: Prediction summary for FLAC axisymmetric model

	Footing 1 3 m x 3 m	Footing 2 1.5 m x 1.5 m	Footing 3 3 m x 3 m	Footing 4 2.5 m x 2.5 m	Footing 5 1 m x 1 m
Load for 25mm of settlement Q_{25} on the 30 minute load settlement curve (kN)	760 kN	250 kN	950 kN	625 kN	200 kN
Load for 150mm of settlement Q_{150} on the 30 minute load settlement curve (kN)	3350 kN	910 kN	4000 kN	2650 kN	800 kN

Table 4.3: Prediction summary for $FLAC^{3D}$ model

	Footing 1 3 m x 3 m	Footing 2 1.5 m x 1.5 m	Footing 3 3 m x 3 m	Footing 4 2.5 m x 2.5 m	Footing 5 1 m x 1 m
Load for 25mm of settlement Q_{25} on the 30 minute load settlement curve (kN)	800 kN	250 kN	950 kN	625 kN	150 kN
Load for 150mm of settlement Q_{150} on the 30 minute load settlement curve (kN)	-	-	-	-	-

The final predictions given in the table below were obtained by examining the range of results from the two models in conjunction with the results obtained from the ultimate bearing capacity calculations and determining a value which best represents the engineering judgement of the author. These predictions are summarised below.

Table 4.4: Summary of the authors' predictions

	Footing 1 3 m x 3 m	Footing 2 1.5 m x 1.5 m	Footing 3 3 m x 3 m	Footing 4 2.5 m x 2.5 m	Footing 5 1 m x 1 m
Load for 25mm of settlement Q_{25} on the 30 minute load settlement curve (kN)	800 kN	250 kN	950 kN	625 kN	200 kN
Load for 150mm of settlement Q_{150} on the 30 minute load settlement curve (kN)	9850 kN	1450 kN	13500 kN	6100 kN	800 kN

4.4 Load Test Results

Load settlement curves for the five footings were produced by recording the loads and amount of settlements at given intervals. The results shown in the table were published and made available to all participants of the conference after all predictions were submitted. These values were read from the curves produced. Figure 4.5 to Figure 4.9 shows the measured results and the predicted results using the FLAC and FLAC^{3D} models.

Table 4.5: Measured results

	Footing 1 3 m x 3 m	Footing 2 1.5 m x 1.5 m	Footing 3 3 m x 3 m	Footing 4 2.5 m x 2.5 m	Footing 5 1 m x 1 m
Load for 25mm of settlement Q_{25} on the 30 minute load settlement curve (kN)	5200 kN	1500 kN	4500 kN	3600 kN	850 kN
Load for 150mm of settlement Q_{150} on the 30 minute load settlement curve (kN)	10250 kN	3400 kN	9000 kN	7100 kN	1740 kN

4.5 Conclusions

The prediction model was prepared so that it could be applied to small and large foundation engineering projects, while utilising commonly available and affordable insitu testing techniques. Even though there was a broad range of geotechnical test data obtained for the test site, SPT data was used with empirical relationships to obtain the soil properties which were used within the model.

Various assumptions about the test site and about the application of some of the correction factors were made in the design procedure. These may have had some influence on the predicted data.

Three sets of prediction data were obtained by using a FLAC axisymmetric model, a FLAC^{3D} model and using the bearing capacity equations to determine a prediction for Q_{150}. Engineering judgement was applied to obtain the final predictions listed. These results will be examined against the measured data. It is unreasonable and unexpected that any of the predictions will yield perfect results because of the lack of experience and case studies performed at the site in the past and the uncertainty associated with the data.

The predictions made for Q_{25} using the FLAC model were very conservative, however the predictions made for Q_{150} using bearing capacity theory produced reasonably accurate results. Comparisons will be made between the measured and predicted results in Chapter 6.

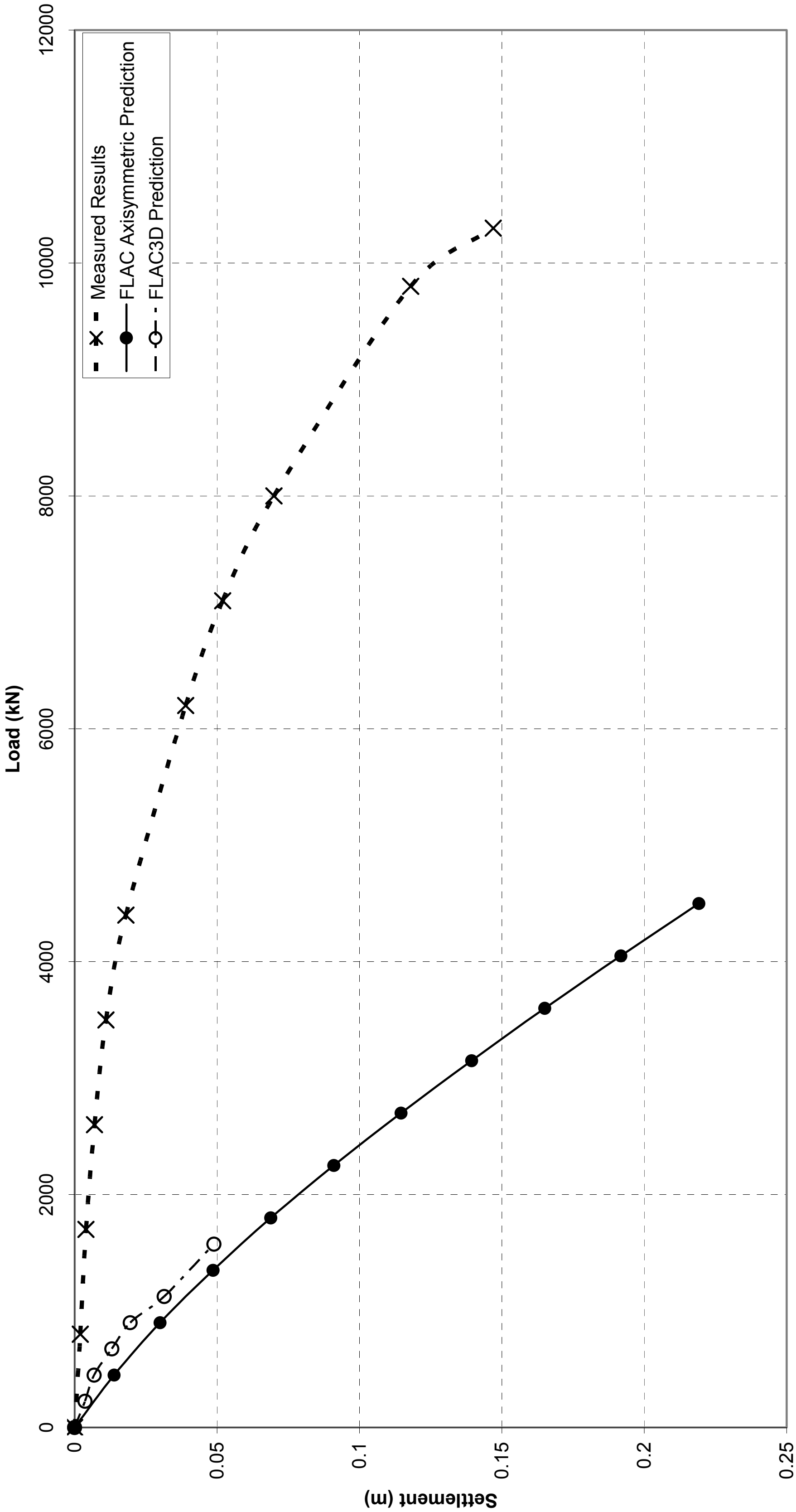


Figure 4.5: Prediction curve and measured load settlement curve for Footing 1

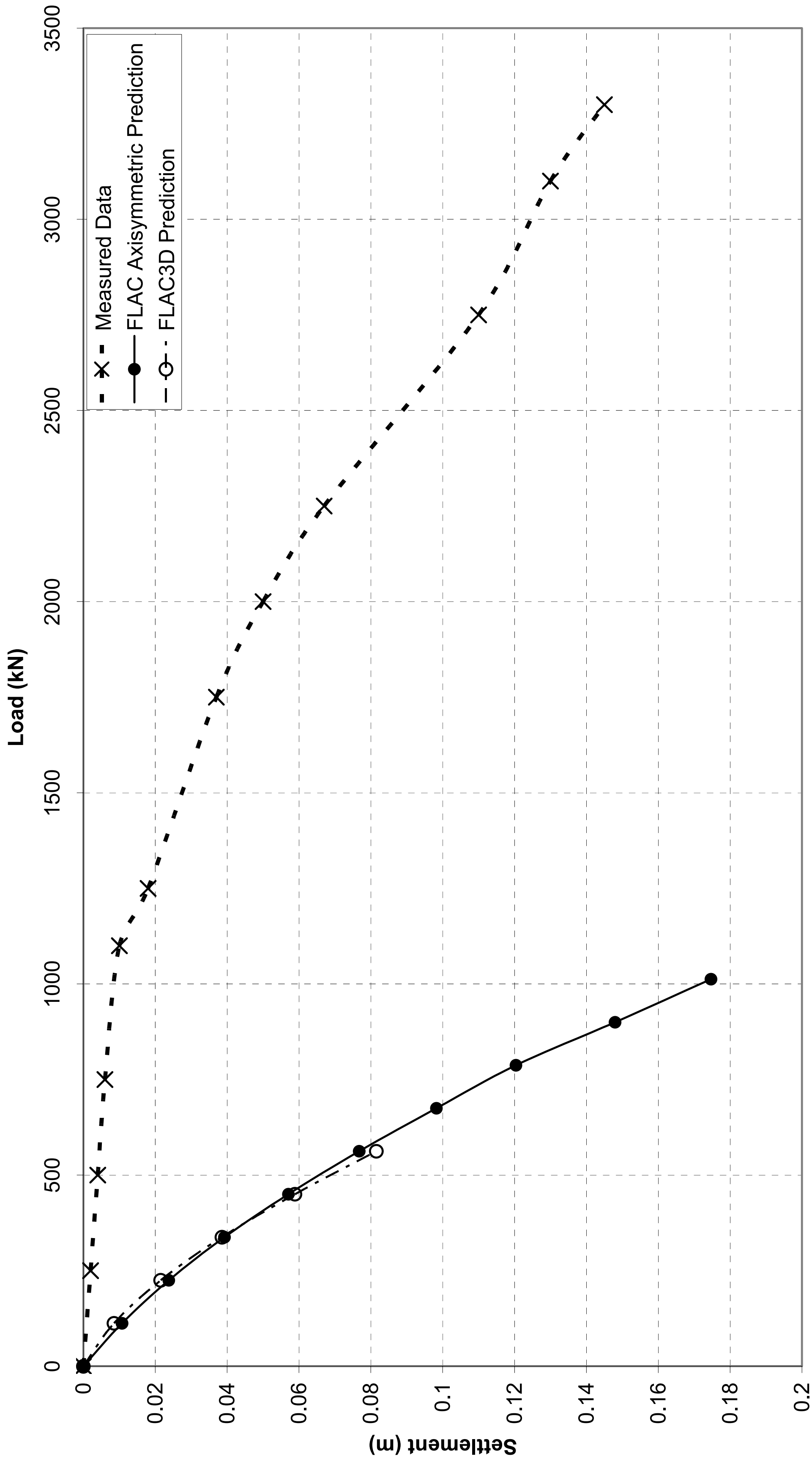


Figure 4.6: Prediction curve and measured load settlement curve for Footing 2

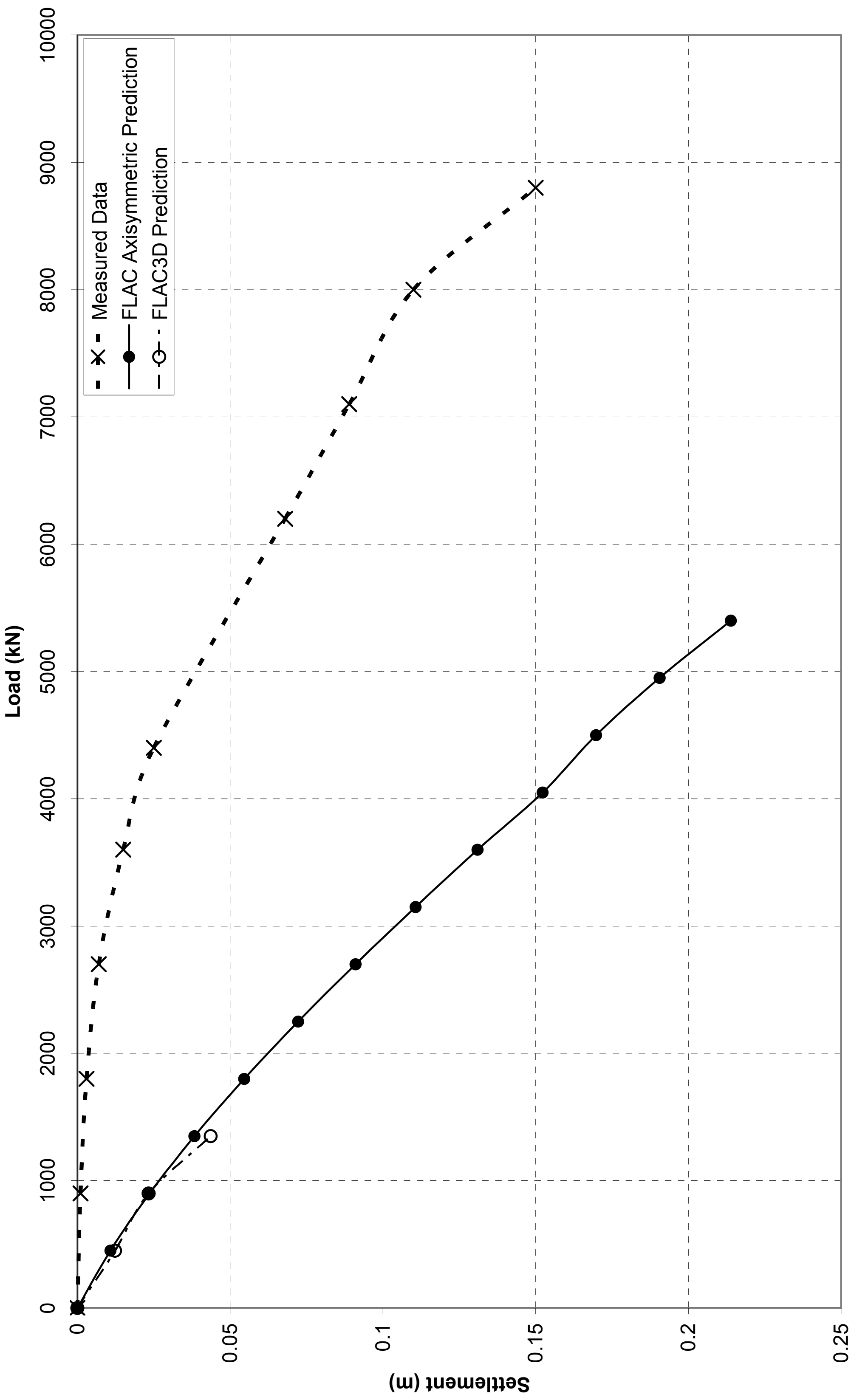


Figure 4.7: Prediction curve and measured load settlement curve for Footing 3

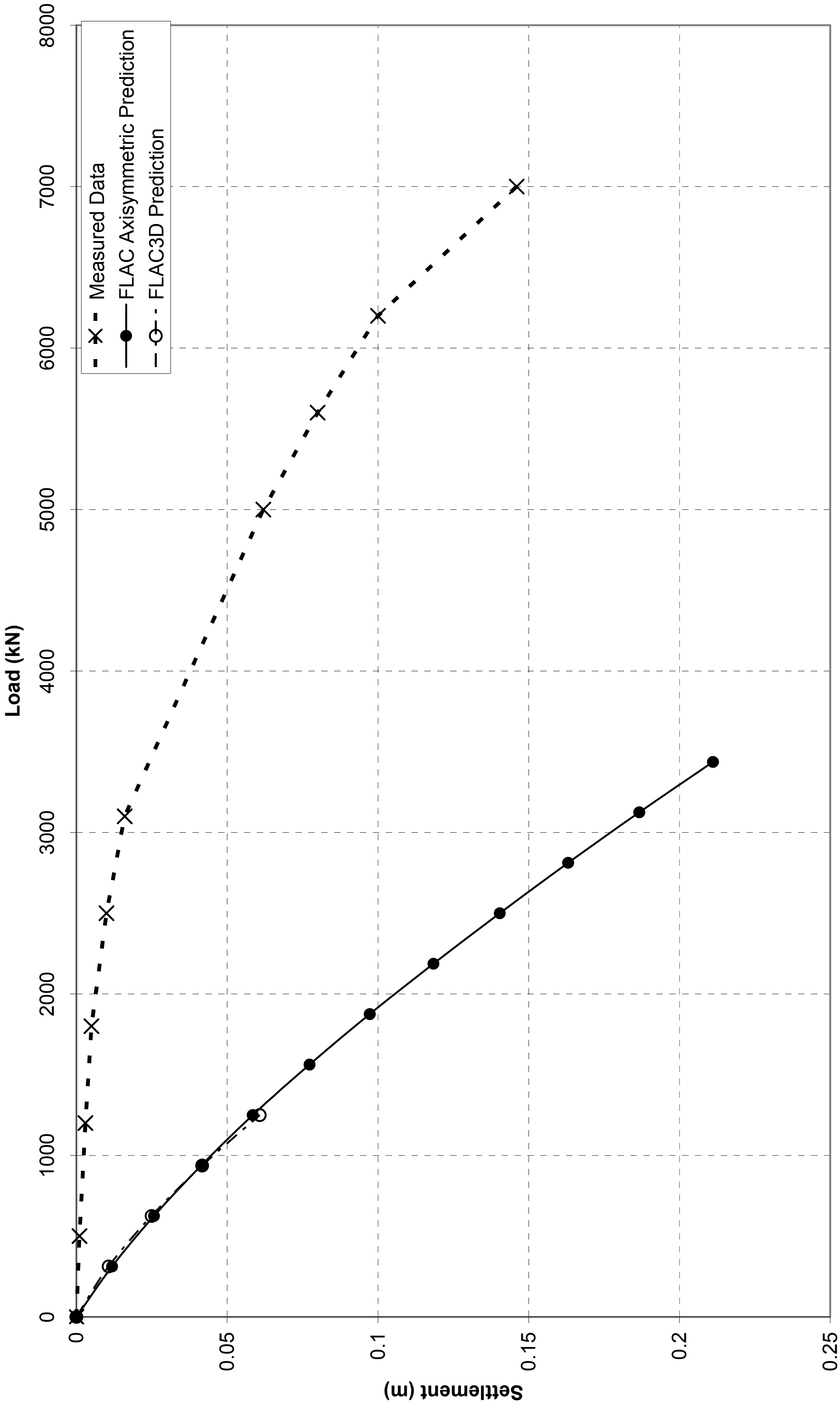


Figure 4.8: Prediction curve and measured load settlement curve for Footing 4

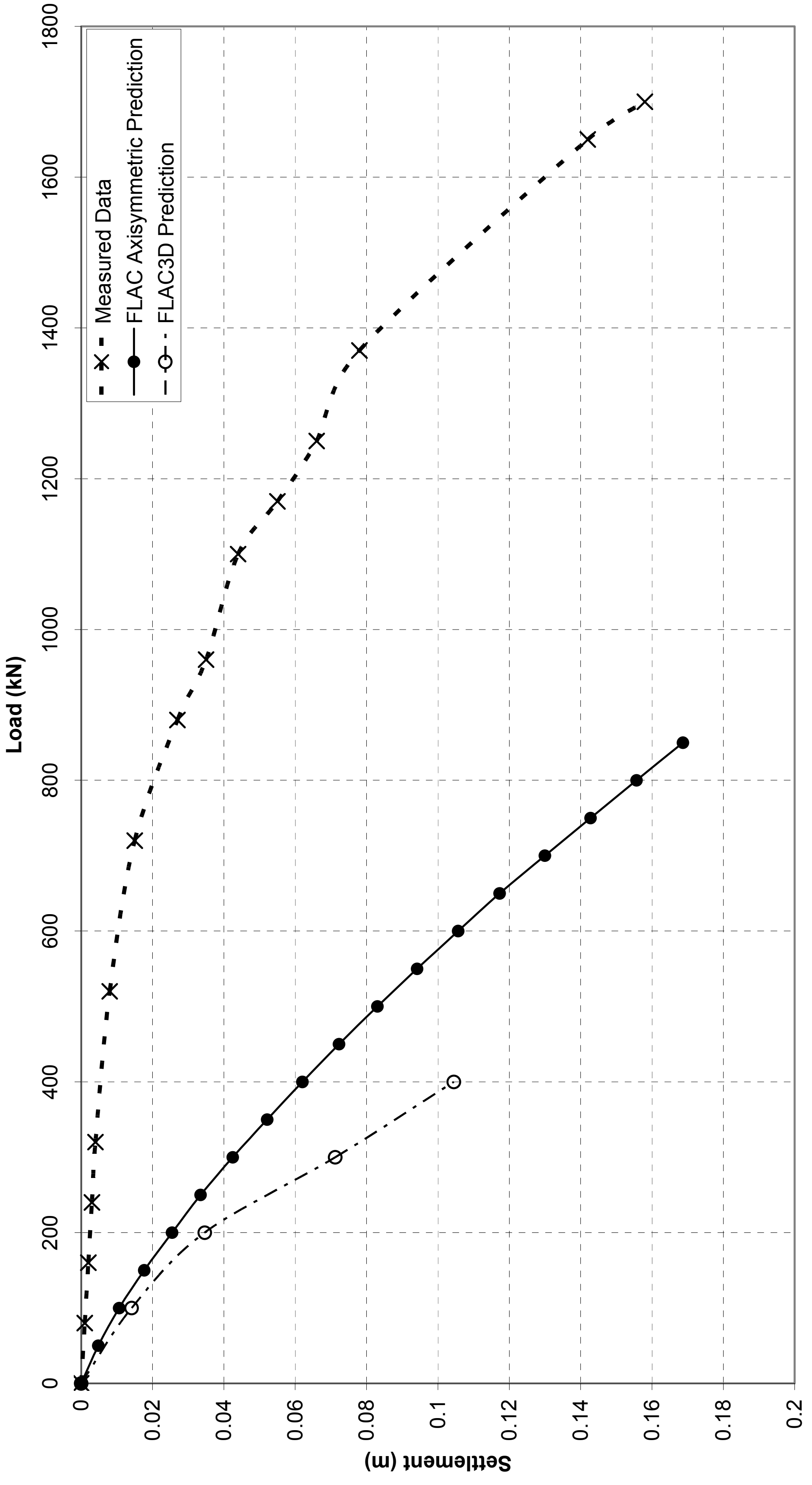


Figure 4.9: Prediction curve and measured load settlement curve for Footing 5

Chapter 5: Interface Elements

5.1 Introduction

In order to replicate the soil-foundation interaction upon loading of a concrete footing constructed within a granular soil layer, interface elements were used between the footing and the soil. Slippage was assumed to occur within the soil and interface elements were applied to the FLAC axisymmetric model only.

For this model, the interface can be characterised by Coulomb sliding. In FLAC, Coulomb interfaces are represented by the normal and shear stiffness between two contact planes and the friction angle of the interface. Cohesion was assumed to be zero due to the sandy soil. Direct shear tests are commonly used to obtain the properties required for the interface, however for this model, commonly used correlations shown below were employed. The friction angle of the interface may be given by:

$$\delta = \frac{2}{3}\phi \tag{5.1}$$

Using a rule of thumb provide in the FLAC manual, the following relationship was obtained to find the normal and shear stiffness of the interface.

$$k_n = k_s = 10 \times \max\left[\frac{\left(K + \frac{4}{3}G\right)}{\Delta z_{\min}}\right] \tag{5.2}$$

where $\Delta z_{\min}$ is the smallest width of an adjoining zone in the normal direction.

The above equation should be applied to the side of the interface where the sand is present. This is because the concrete is much stiffer than the sand and the deformability of the entire system will be governed by softer material. Figure 5.1 shows the slip occurring within the model.

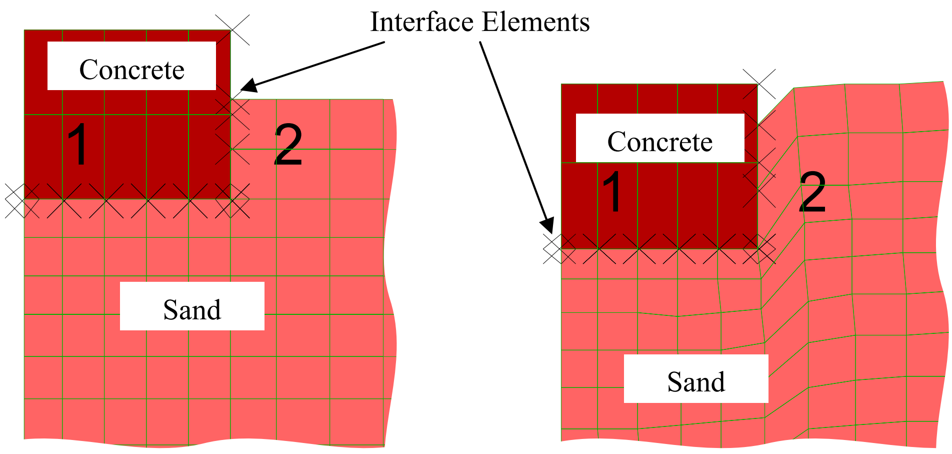


Figure 5.1: Settlement of the foundation using interface elements

5.2 *Influence of Interface Elements on the Model*

It was anticipated that the interface elements would improve the accuracy of the predictions made in Chapter 4. Therefore a comparison of the results obtained with and without the use of interface elements was performed. These results can be seen in Figure 5.2 and Figure 5.3.

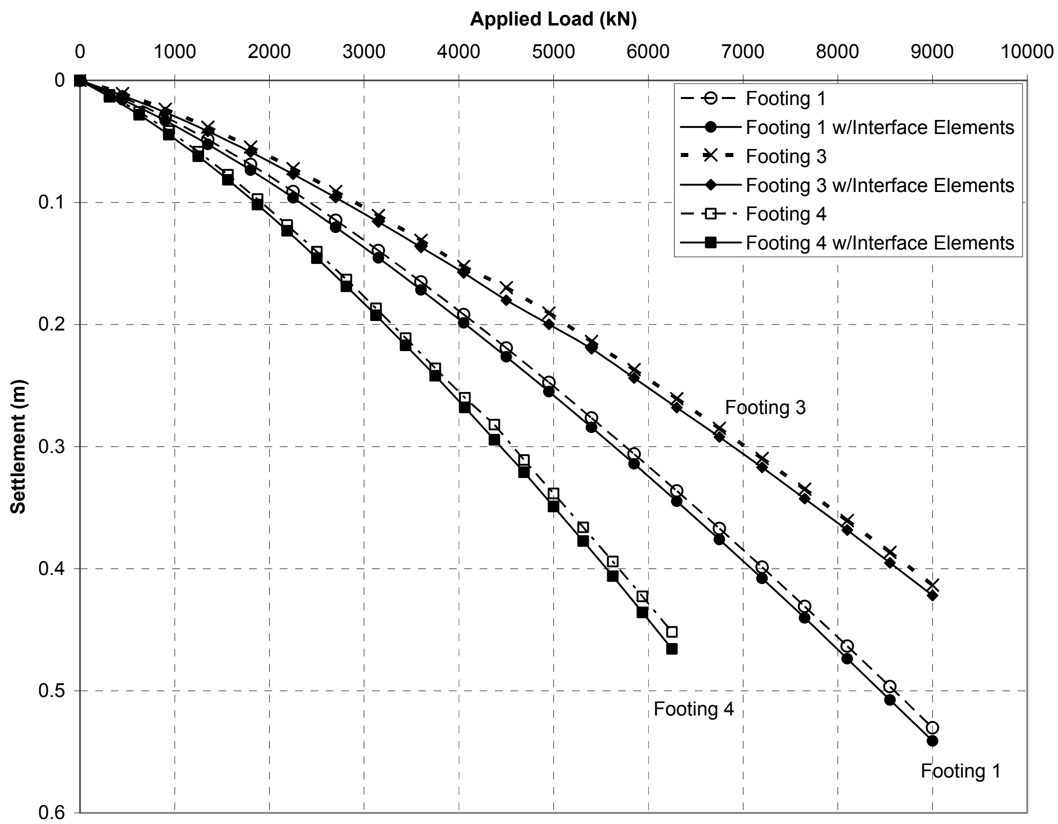


Figure 5.2: Load-settlement predictions with and without interface elements for Footings 1, 3 and 4

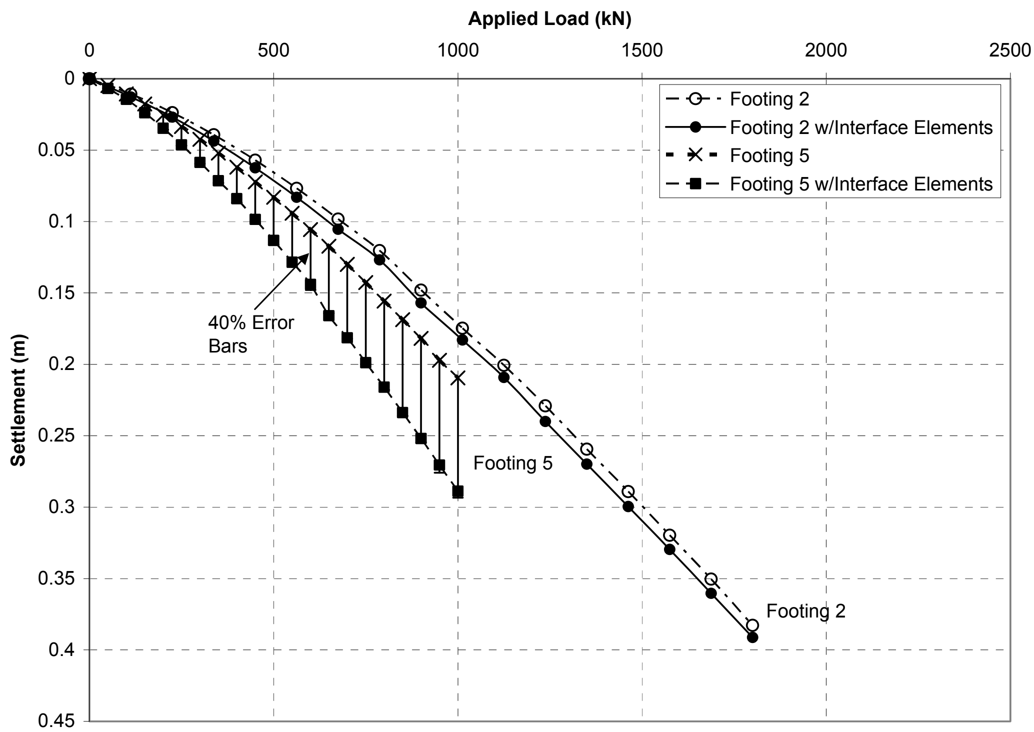


Figure 5.3: Load-settlement predictions with and without interface elements for Footings 2 and 5

The results show that for four of the footings at the Texas A&M Settlement '94 conference, there is minimal influence to the predictions by introducing the interface elements into the models. The average error in the settlement values produced up to 25 mm settlement is 10%, and 5% up to 150 mm settlement. However, when interface elements were used in the model for Footing 5, there was significant deviation between this model and the model without interface elements. The error for this footing was 40%, as shown by the error bars in Figure 5.3. This was inconsistent with the results for the other footings, and the cause of such difference is unknown.

5.3 Effect of Varied Friction Angle

An investigation into the sensitivity of the load-settlement behaviour to the friction angle of the interface was carried out for Footing 1 using friction angles of 10˚, 15˚, 20˚ and 30˚. These results were then compared to the initial prediction as shown in Figure 5.4.

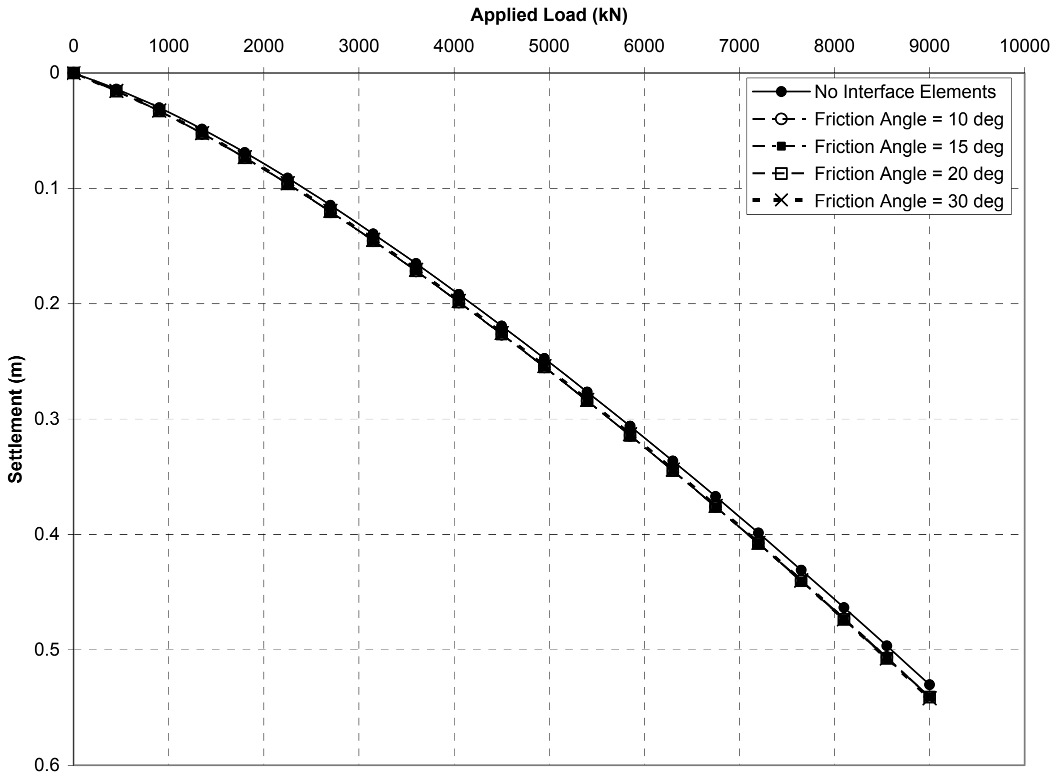


Figure 5.4: Influence of varied interface friction angle on FLAC model for Footing 1

The results show that there is negligible difference between the settlements for each friction angle and indicates that the sensitivity of the load-settlement behaviour to the variation of the interface friction angle is low. Therefore, determining the interface friction angle from shear tests is not necessary and the relationship between the friction angle of the interface and the friction angle of the sand can be confidently used in the model.

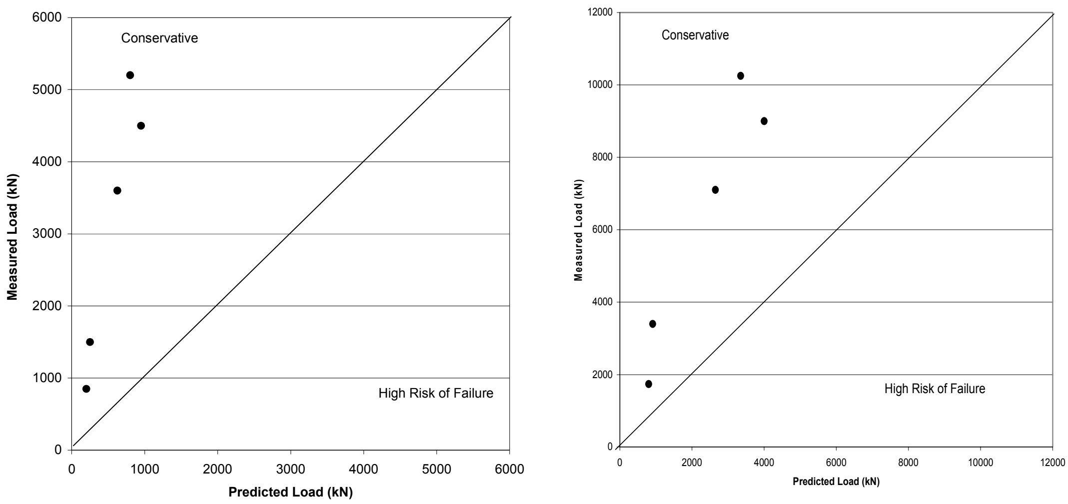
5.4 Conclusions

It was believed that interface elements would introduce slippage effects and therefore produce behaviour which reflected a physical model. Overall (with the exception of Footing 5), the use of interface elements had only a minor influence on the predicted settlements. The variation of the interface friction angle displayed no significant response in the model. Therefore, the predictions made in Chapter 4 will not be modified for comparisons which will be made in the next chapter.

Chapter 6: Comparison of the Results

6.1 Comparing Model Data to Experimental Results

The predictions found in Chapter 4 can be compared to the experimental results for the five footings tested at the conference. It can be seen in Figure 6.1 that the predictions made using the model are extremely conservative. The measured loads for Q_{25} using the FLAC model were between 4 and 6.5 times larger than the corresponding predicted loads for Q_{25}. The predictions of Q_{150} using the FLAC model were slightly more accurate, however the measured loads were still between 2 and 3 time more than the predicted loads for Q_{150}.



(a) (b)

Figure 6.1: Predicted and measured footing loads for (a) 25 mm settlement (Q_{25}), and (b) 150 mm settlement (Q_{150})

The predictions of Q_{25} and Q_{150} using the FLAC model were far too conservative to be used as a design tool. However, predictions made for Q_{150} using the ultimate bearing capacity displayed closer correlation to the measured results. In most of the footings the accuracy was good and was on the conservative side. However, an unsafe prediction was made for Footing 3 whereby the load was over predicted by 1.5 times. This can be seen in Figure 6.2.

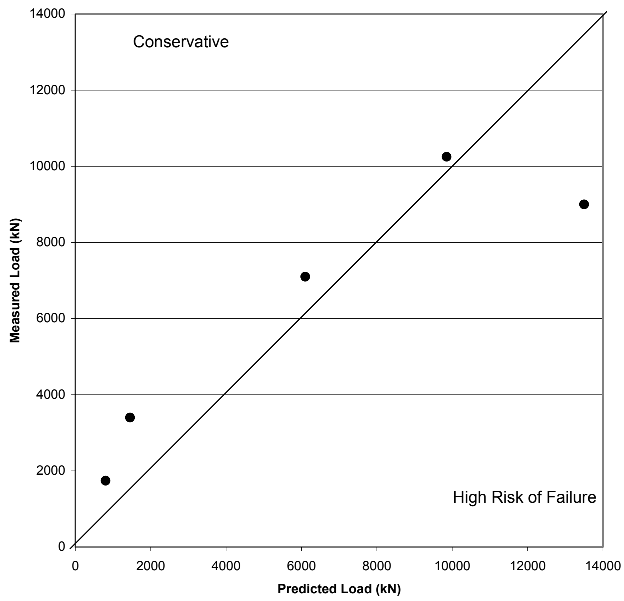


Figure 6.2: Predicted and measured footing loads for 150 mm settlement (Q_{150}) using bearing capacity theory

The above data shows, there were no predictions within ± 20% of the measured value for Q_{25}, and two predictions within ± 20% of the measured value for Q_{150}, but only when 150 mm was considered to be produced by the ultimate bearing capacity of the soil.

Uncertainty in the soil test data and soil variance was evidently a significant cause of the error associated with the prediction. It was assumed that the soil in the model was cohesionless. However, it is possible that there may have been a significant amount of clay within the sandy soil layer. This would have resulted in higher predictions for Q_{25} and Q_{150}. There was significant time difference between soil testing using SPT and construction and testing of the footings. In this time, the height of the water table may have been influenced by seasonal weather. This will have influenced the outcomes of the measured results.

6.2 Comparing Model Data to Predictions by Other Authors

The predictions made by the author using the FLAC and FLAC^{3D} models can be compared against the thirty one predictions made at the Settlement '94 conference and ranked amongst the 'experts'. In total, twenty two different prediction methods were used. Most of the participants used a combination of these methods to obtain their predictions.

The plot below (Figure 6.3) shows the scatter of the predictions made by other participants in the conference. The ratios of measured to predicted loads for Q_{25} over all five footings ranged from 0.3 (unsafe) to 14.5 (very conservative). Most of the predictions made were on the conservative side.

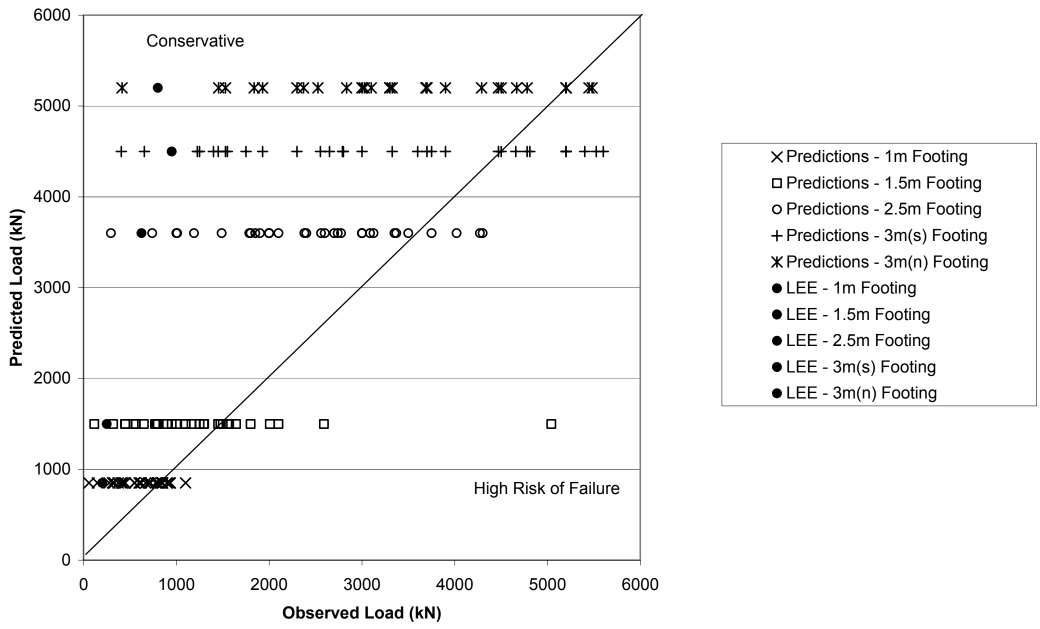


Figure 6.3: Scatter plot for Q_{25} predictions

A scatter plot can similarly be shown for the predictions made for Q_{150} (see Figure 6.4). The ratios of measured to predicted loads for Q_{150} over the five footings ranged from 0.3 (unsafe) to 8.7 (conservative). For this case though, scatter became more widespread as the footing size increased, suggesting that an increase in footing dimensions was not being accounted for in the prediction methods.

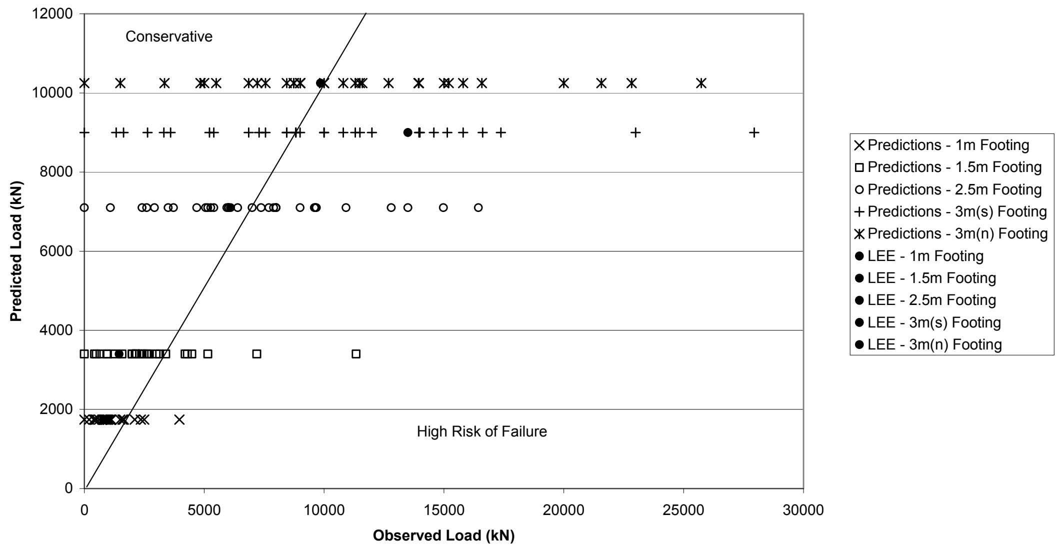


Figure 6.4: Scatter plot for Q_{150} predictions

The initial assumption made in this thesis for the Q_{150} prediction was that it would be reasonable to represent this load as the ultimate bearing capacity of the sand. This assumption proved to be reasonable, and therefore it can be said that the soil experienced plastic deformation. An increase in 'unsafe' predictions made for Q_{150} could be seen as an indication that this behaviour is not very well understood by the profession.

The predicted results were also be compared with each other by representing the data as a series of frequency histograms which show a comparison of the predicted settlement to measured settlement ratios for the Q_{25} and Q_{150} predictions (Figure 6.5 to Figure 6.9). The ratios of the predictions made in this thesis are highlighted in each of the histograms.

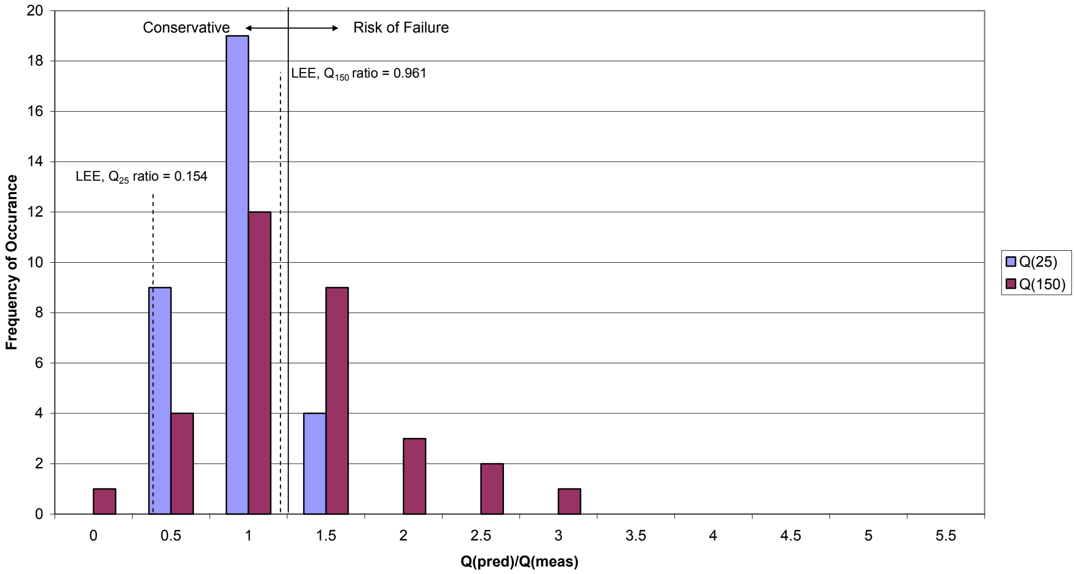


Figure 6.5: Distribution of Q_{PRED}/Q_{MEAS} for Footing 1

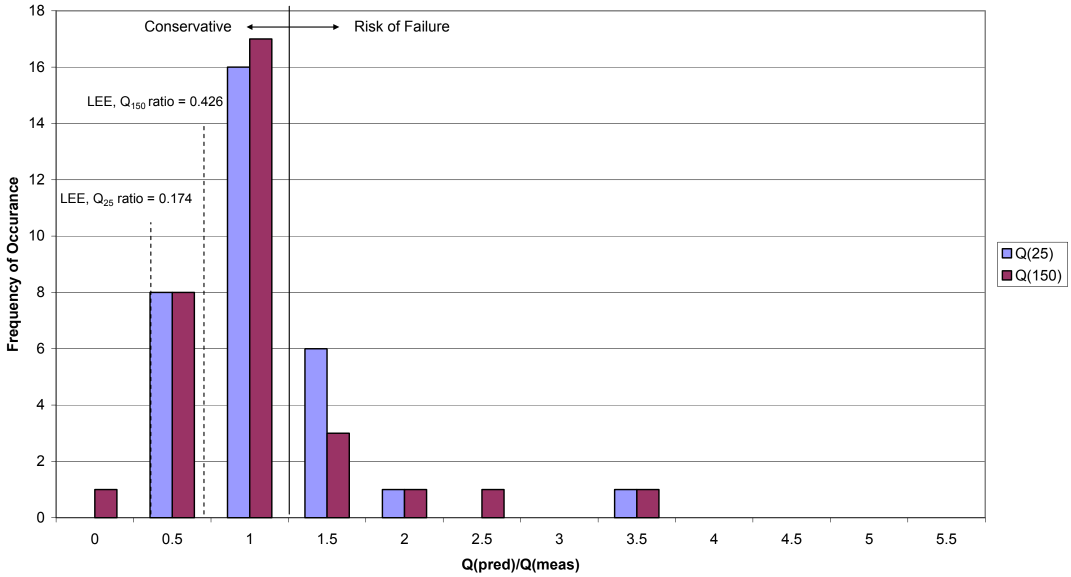


Figure 6.6: Distribution of Q_{PRED}/Q_{MEAS} for Footing 2

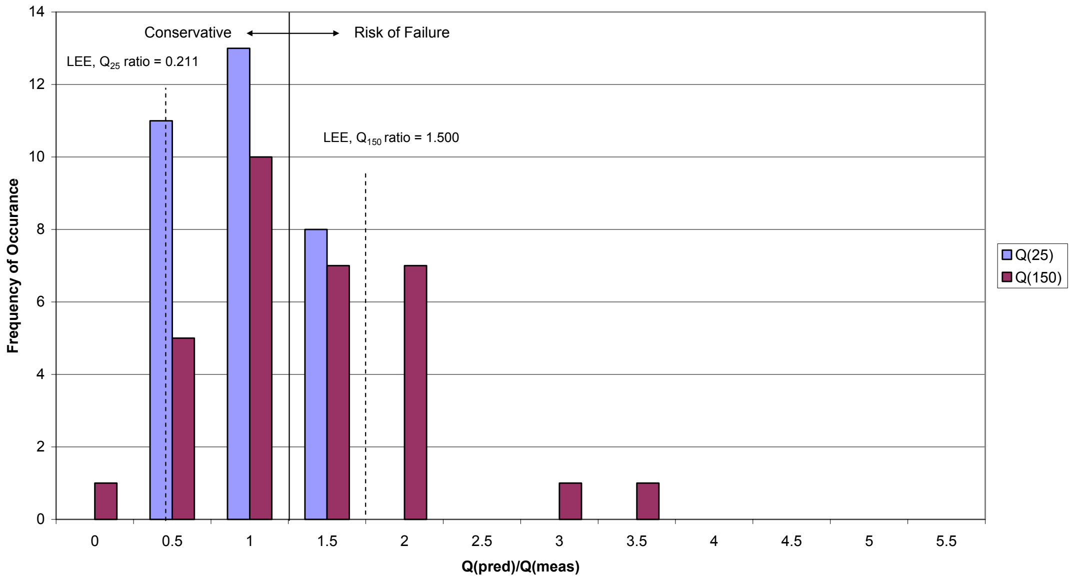


Figure 6.7: Distribution of Q_{PRED}/Q_{MEAS} for Footing 3

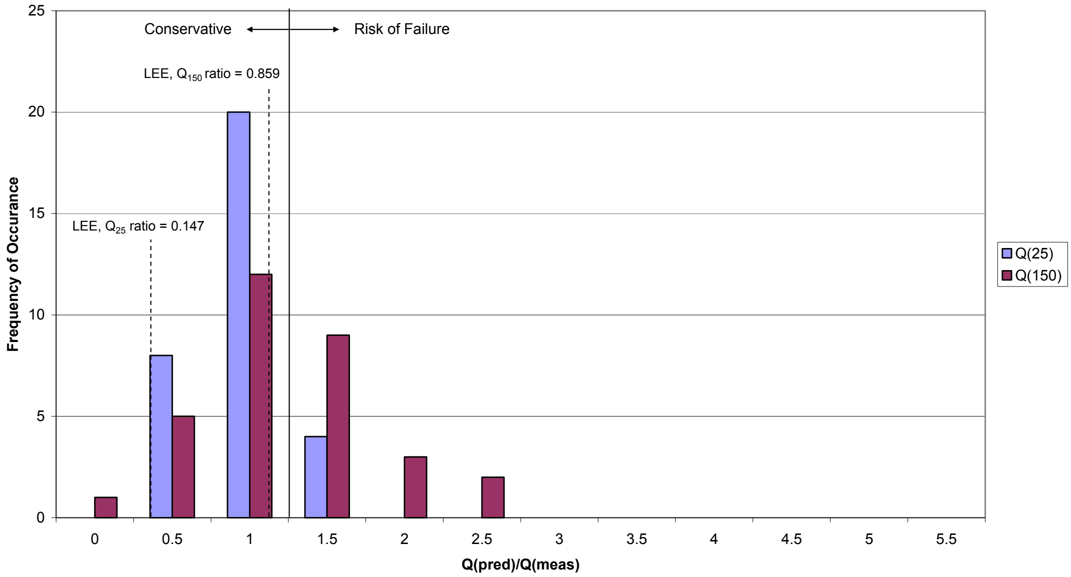


Figure 6.8: Distribution of Q_{PRED}/Q_{MEAS} for Footing 4

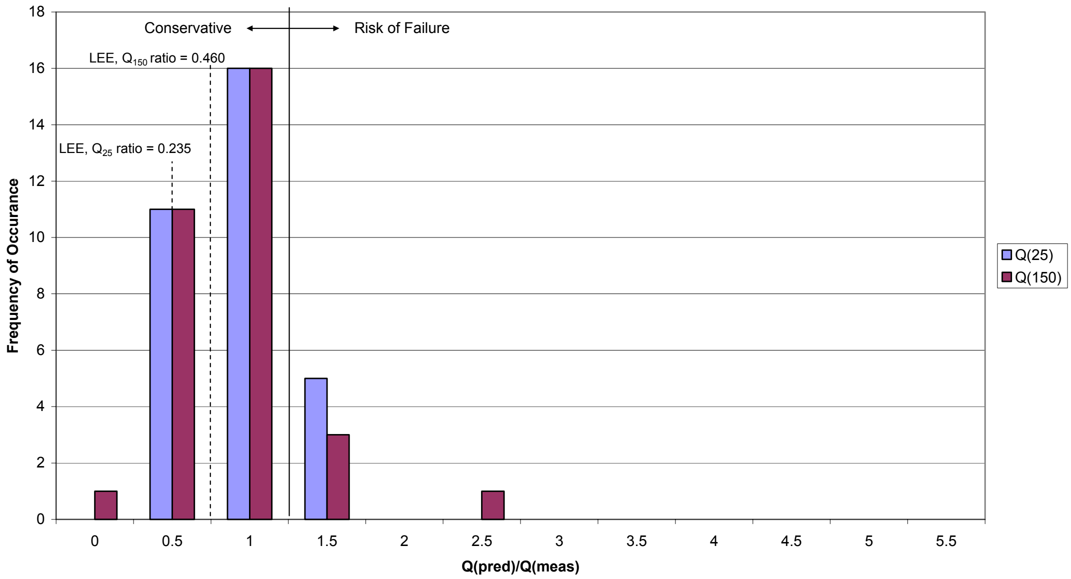


Figure 6.9: Distribution of Q_{PRED}/Q_{MEAS} for Footing 5

A summary of the mean, standard deviation and measured values can be found in Table 6.1. It can be seen in the scatter plot that the spread of the predictions for all of the footings is large, which produces large standard deviations for the predictions. It can also be noted that the mean values of all predictions show reasonable correlations to the measured results.

Table 6.1: Mean and standard deviation of all predictions

Footing	Load Q_{25} for 25 mm Settlement (kN)			Load Q_{150} for 150 mm Settlement (kN)		
	Mean	Std. Dev.	Measured Value	Mean	Std. Dev.	Measured Value
1	3486	1498	5200	11067	5968	10250
2	1227	902	1500	2700	2163	3400
3	3082	1604	4500	10186	6176	9000
4	2397	1062	3600	7026	3846	7100
5	592	263	850	1117	776	1740

The ratios which were plotted in the above histograms were used to rank the predictions made within this thesis against the Settlement '94 conference participants. The ratios of measured results to predicted results are shown in Table 6.2 and are sorted from most conservative to least conservative.

Table 6.2: Ranking of predictions by average ratio of measured results to predicted results

Avg. Q_{meas}/Q_{pred} for Q_{25}			Avg. Q_{meas}/Q_{pred} for Q_{150}	
Siddiquee	12.5	Conservative	Siddiquee	7.3
LEE	5.3	↓	Thomas	3.3
Chang	3.5		Chua	3.1
Shahrour	3.4		Chang	3.0
Chua	3.1		Kuo	2.3
Foshee	2.9		Diyaljee	1.6
Thomas	2.4		Mayne	1.6
Silvestri	2.2		Shahrour	1.5
Kuo	2.0		Silvestri	1.5
Diyaljee	1.9		Tand	1.3
Brahma	1.8		Funegard	1.3
Altaee	1.7		Cooksey	1.3
Bhowmik	1.7		Foshee	1.2
Boone	1.6		Boone	1.2
Cooksey	1.5		LEE	1.2
Abid	1.4		Floess	1.0
Scott	1.4		Surendra	1.0
Wiseman	1.2		Deschamps	1.0
Mayne	1.2		Wiseman	1.0
Mesri	1.2		Brahma	1.0
Tand	1.1		Utah State	1.0
Decourt	1.1		Abid	0.9
Ariemma	1.1		Bhowmik	0.9
Horvath	1.1		Scott	0.9
Funegard	1.0		Altaee	0.8
Utah State	1.0		Gottardi	0.8
Deschamps	1.0		Poulos	0.6
Townsend	1.0		Ariemma	0.6
Floess	0.9	Unsafe	Horvath	0.6
Surendra	0.9		Townsend	0.6
Gottardi	0.8		Decourt	0.5
Poulos	0.8		Mesri	-

The performance of each of the predictions was determined by the number of predictions within 20% of the measured result. This criterion measures the accuracy rather than the reliability of the predictions obtained. Overall, the model ranked poorly, but where engineering judgment was applied, the predictions developed within this thesis ranked as average on comparison with the other participants.

Table 6.3: Performance rankings for all predictions

No.	Author	No. of Predictions within 20% of meas. (max=10)
1	Funegard	8
2	Deschamps	8
3	Tand	7
4	Horvath	6
5	Utah State	6
6	Wiseman	5
7	Floess	5
8	Mayne	5
9	Abid	5
10	Surendra	4
11	Cooksey	4
12	Townsend	4
13	Foshee	4
14	Decourt	4
15	Gottardi	4
16	Mesri	3 of 5
17	Boone	3
18	Altaee	3
19	LEE	2
20	Ariemma	2
21	Brahma	2
22	Poulos	1
23	Scott	1
24	Shahrour	1
25	Diyaljee	1
26	Siddiquee	0
27	Silvestri	0
28	Thomas	0
29	Chang	0
30	Kuo	0
31	Chua	0
32	Bhowmik	0

6.3 Comparing Axisymmetric and FLAC^{3D} Models

A comparison between the load-settlement curves obtained with the FLAC axisymmetric model and the FLAC^{3D} square footing model was performed. This was achieved by considering the load-settlement curves produced using the Terzaghi and Peck validation data (Figure 3.16 to Figure 3.18) and prediction data from the Settlement '94 conference (Figure 4.5 to Figure 4.9). The comparison displayed good correlation for both the validation and prediction data for loads and settlements within the elastic region of the curve. Some deviation can be observed in the validation data when the soil experienced plastic deformation. This information was unavailable for the prediction data due to an error in the

$FLAC^{3D}$ evaluation. However, in most cases it is the elastic deformation (considered to be up to 25 mm settlement) that is of interest to designers.

The equivalent radius for the circular footing used to model the settlement of the square foundation was equal to the width of the square footing. However, when the applied pressure was converted to a load, the pressure was multiplied by the area of the square footing and not the actual area of the circular footing. No other modifications or factors were applied to the dimensions of the circular footing.

Generally the difference between the predictions made by the model ranged from 1% to 30%, depending on the footing being modelled and the points being compared. The error was larger for small loads and deformations, but decreased as the loads and settlements approached the limits of the $FLAC^{3D}$ curves. The exception to the above trend was for Footing 5 from the Settlement '94 conference, which displayed poor correlation between the two-dimensional and three-dimensional model.

The findings of this investigation showed that for footings where B > 1 m and where predictions of small deformations are required, a FLAC axisymmetric model could be confidently used in the place of $FLAC^{3D}$. However, modelling a square footing as a circular footing should be applied with caution for footings where B < 1 m or large deformations are considered.

6.4 Comparison of Typical Prediction Methods to Author's Predictions

Most of the participants used a combination of typical settlement prediction methods to obtain their predictions; therefore it is difficult to use the above data to identify the accuracy of any single method. The most commonly used prediction methods at the conference were Schmertmann (1978), Burland and Burbidge (1985) and Finite Element Methods.

Two prediction methods were chosen to be used as a basis for comparison to the predictions made by the author. The methods chosen for comparison were Terzaghi and Peck's (1948) method, and Schmertmann's simplified (1978) method. To be able to make the comparisons, the same soil test data was used for all methods. Schmertmann's method typically uses CPT tip resistance to derive a soil stiffness value, however for the calculations performed to obtain Q_{25} and Q_{150}, the following correlation was employed:

$$0.8N = 2q_c \quad \text{(MPa)} \tag{6.1}$$

Many designer's would question the applicability of Schmertmann's method when being based on average SPT values, however in this case the results are only being used as a comparison to other predictions, and not in any design capacity. Hand calculations were performed and are summarised in Table 6.4.

Table 6.4: Calculated loads for typical prediction methods

Footing	Load Q_{25} for 25 mm Settlement (kN)				Load Q_{150} for 150 mm Settlement (kN)			
	Terzaghi and Peck (1948)	Schmertmann Modified (1978)	Prediction	Measured	Terzaghi and Peck (1948)	Schmertmann Modified (1978)	Prediction	Measured
1	2340	2405	800	5200	5625	13905	9850	10250
2	450	880	250	1500	1125	5190	1450	3400
3	2700	2935	950	4500	7020	17360	13500	9000
4	2340	1940	625	3600	5625	11540	6100	7100
5	350	775	200	850	610	4615	800	1740

The results show that both of the typical hand calculation methods are conservative and display behaviour which agrees with studies performed by Tan and Duncan (1991) while the soil is in the elastic region. The predictions made using the FLAC model displayed poor accuracy and was more conservative than Terzaghi and Peck's method. Schmertmann's modified method displayed the highest accuracy compared to the three prediction methods used to find the load which results in 25 mm settlement of the foundation. All of the predictions were conservative.

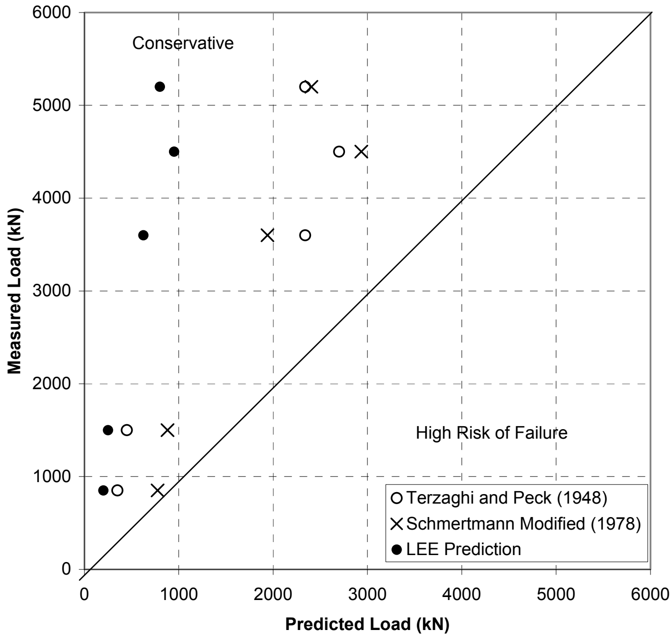


Figure 6.10: Comparison of typical prediction methods for Q_{25}

Terzaghi and Peck's method was the most conservative for the prediction of the Q_{150} load, however displayed good reliability. This could be due to the fact that Terzaghi and Peck's (1948) method is not a prediction method, but a design method (Poulos 1996). Schmertmann's method is based on elastic theory, and therefore it is expected that it would not be valid when the soil undergoes plastic deformation. The results show that for large deformations, the method is extremely unreliable and inaccurate.

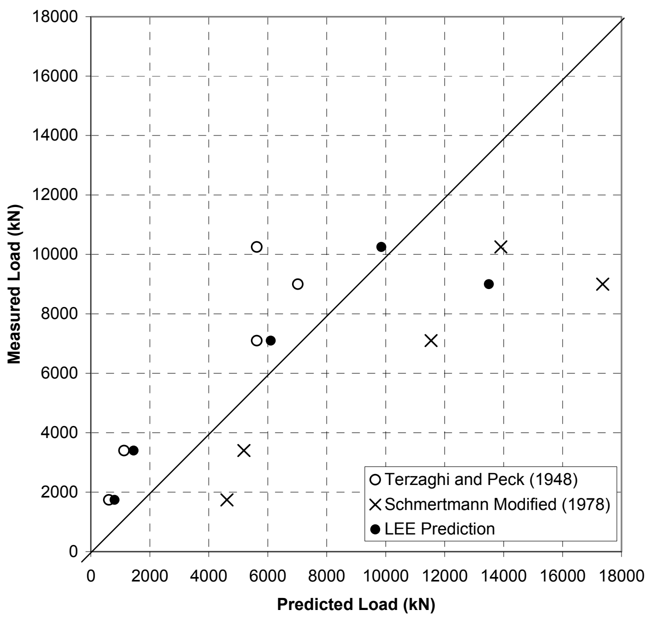


Figure 6.11: Comparison of typical prediction methods for Q_{150}

Chapter 7: Summary, Conclusions and Recommendations

7.1 Summary

The research undertaken in this thesis has resulted in the development of two numerical models which were used to predict the settlement of shallow foundations in granular soils. FLAC and FLAC^{3D} are extremely useful research and analytical tools and were used to develop the models. The models were validated using Terzaghi and Peck's (1948) pressure-settlement experimental data as a basis. A sensitivity analysis was performed, aiding the understanding of the effects of varying soil properties within the model. The influence of interface elements on the models was also investigated. The models were then applied to a specific problem developed as part of the Settlement '94 conference, where five footings were constructed and tested.

Predictions of load-settlement behaviour for all five footings were obtained using the models and compared with the measured results, predictions made by other authors and data obtained using common prediction methods. These comparisons showed that predictions made using the FLAC models were conservative and displayed significant error when compared to the measured results.

An investigation was performed to determine if square footings modelled in FLAC^{3D} could be accurately represented as circular footings using an axisymmetric model. There appeared to be good correlation between the two models within the elastic region of the load-settlement curve, however there was some deviation in the predictions for the plastic region of the curve. However, this deviation was not conclusive and could not be quantified because of a lack of prediction data available for the FLAC^{3D} model for large deformations when applied to the Settlement '94 problems.

7.2 Conclusions

It can be concluded from the literature review performed that there is a lack of understanding within the geotechnical engineering profession of the mechanisms associated with predicting settlements of shallow foundations in granular soils. Actual settlements rarely achieve the limiting value prescribed in the design of the foundation, causing designs to be conservative in nature. There is also a lack of confidence amongst many engineers in the use of the current design methods for shallow foundations, creating the need for additional modelling of settlements of shallow foundations in granular soils.

It was also concluded that the main source of error was due to the uncertainties associated with modelling the underlying soil. These findings support a statement by Robertson (1986), where the correct selection of the appropriate insitu test is governed by geological conditions, project requirements, type of construction and method of analysis intended for design. Large variances in the settlement of the foundation may occur in the predictions depending on the soil data which is used in the model. Assumptions and correlations should be checked for their applicability to the specific problems which are being modelled. It is unreasonable to expect that perfect predictions can be produced from limited soil data.

The predictions made within this thesis ranked moderately amongst the 'experts'. Although FLAC can be used to solve complex problems such as the prediction of settlements in granular soils, the solution obtained will not be realistic if the input data is wrong. Therefore, all predictions dealing with settlements of shallow foundations in granular soils should be treated with caution, and it is often better to choose a conservative method to ensure that failure of the structure does not occur. Notably, the research showed that loads producing large settlements may be approximated by considering the soil to undergo plastic deformation, corresponding to the ultimate bearing capacity of the soil.

Douglas (1986) recommends codification to increase uniformity of design approaches, as in other engineering disciplines. This may only be achieved after extensive research and understanding of mechanisms involved with the settlement of shallow foundations in granular soils. Further modelling using tools such as FLAC may be the key to understanding of the settlement mechanism; which in the author's belief will lead to more accurate predictions of settlements of shallow foundations in granular soil. In spite of the error produced by the model, it is believed that the objectives and goals of this thesis were successfully achieved.

7.3 Recommendations for Further Research

The model constructed as part of this thesis can be used for further investigations regarding shallow foundations in granular soils. Most of the recommendations are associated with eliminating doubt associated with the model, while some are intended to extend on the research completed so far. Suggestions for improving the numerical model include:

- Investigating the effects of varying length to width ratios for rectangular footings;
- Investigating scaling effects for different sized foundations;
- Investigating the effects of varying cohesion in the sand due to clay particles which may be present;

- Investigating the effects of implementing Terzaghi and Peck's (1948) relationship for correction of SPT blow count for fine silty sands below the water table due to dilation of the soil, using the following relationship;

$$N' = 15 + 0.5(N - 15) \tag{7.1}$$

- The vertical stress distributions produced using the FLAC model may be transformed into vertical strain distributions and compared with the work completed by Schmertmann's (1970, 1978) FEM analysis by using the following relationships;

$$\varepsilon_z = \frac{\Delta\sigma_z}{D'}; \qquad \text{where } D' = \frac{(1-\nu)}{(1+\nu)(1-2\nu)} \cdot E' \tag{7.2}$$

The model has been constructed such that the outcomes of future research related to foundation engineering can be readily incorporated within the model. Recommendations for further work in this field are as follows:

- Experimentation and modelling using more direct methods of determining soil stiffness, eliminating error produced by empirical relationships;
- Continued collection of settlement data for shallow foundations to build a comprehensive database to enable comparison of measured field data with analytical models. This data should record pressure-settlement behaviour;
- An investigation into the accuracy of estimates of Young's modulus and soil friction angle based on SPT, CPT and other insitu test methods;
- Conducting plate load tests on various sands using a scale model of a strip footing. The experimentation would be similar to Terzaghi and Peck's (1948) model, but would allow complete validation of plane strain models;
- Further research to confirm the relationship between the FLAC axisymmetric model for circular footings and the FLAC^{3D} model for square foundations for soils experiencing plastic strains;
- Design charts may be produced once an accurate and reliable model has been constructed, where design parameters may include soil test results and foundation geometry.

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Hypertext References

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Appendix A: SPT Results

The following figures show the SPT results which were used to determine the soil properties for the model.

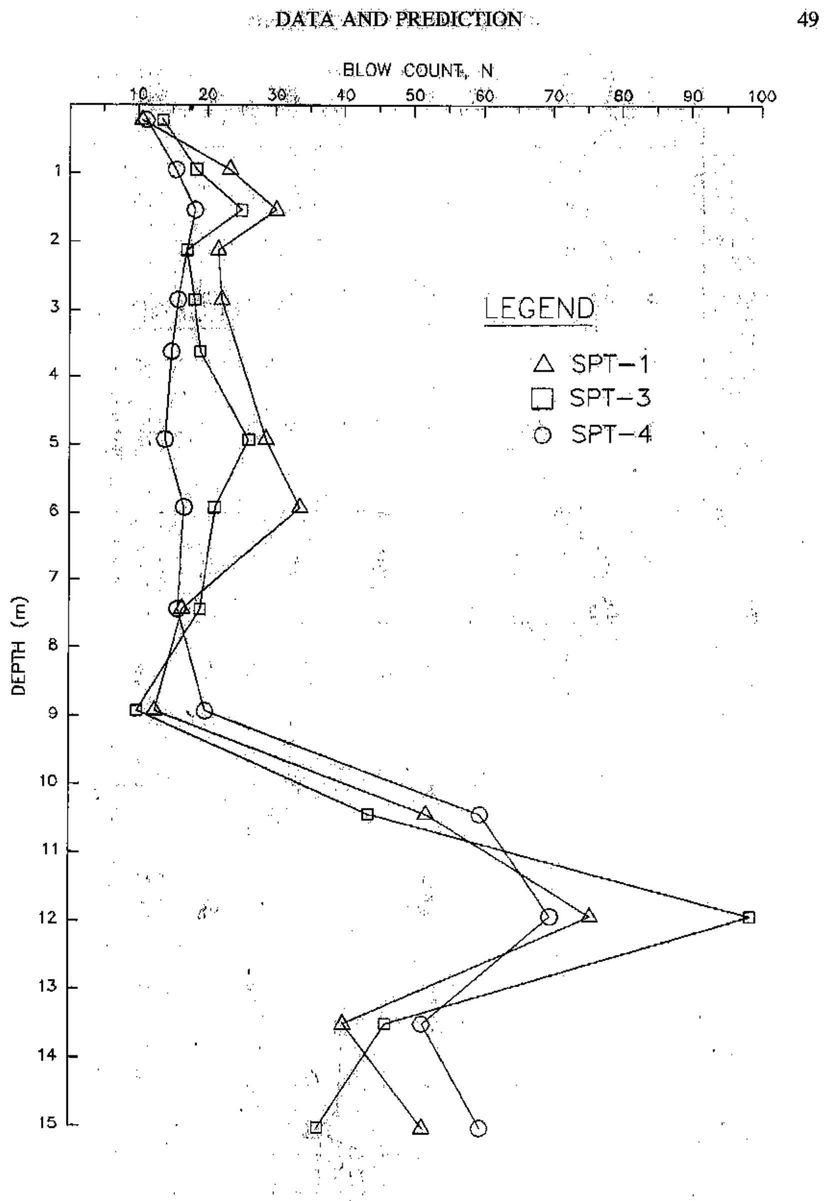

Figure A1: Graph of blow counts (N) versus depth (Briaud and Gibbens 1994)

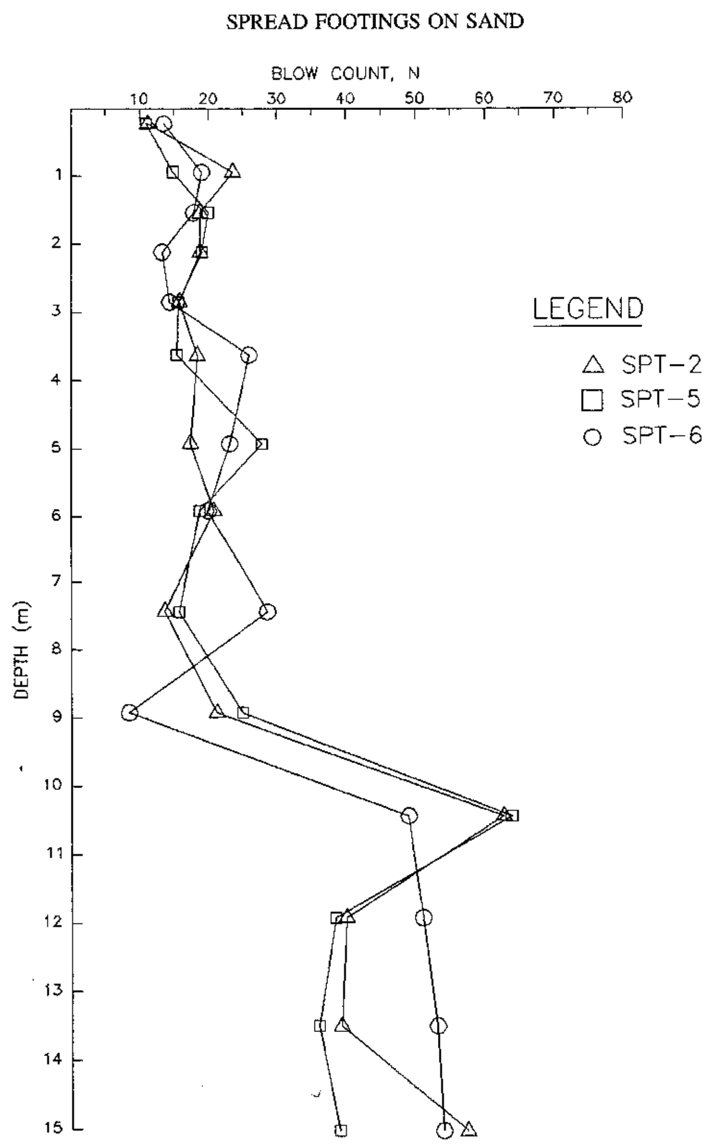

Figure A2: Graph of blow counts (N) versus depth (Briaud and Gibbens 1994)

Appendix B: An Introduction to FLAC^{3D}

By Steven Lee

Most of the background information about FLAC and FLAC^{3D} can be found in *A FLAC Primer*, written by Dr. N. Sivakugan. This guide is intended to be used in conjunction with *A FLAC Primer*. This guide was written to help new users requiring FLAC^{3D} for undergraduate or postgraduate research work.

1. Running FLAC^{3D}

The minimum amount of RAM required to load FLAC^{3D} with the GIIC (Graphical Interface for Itasca Codes) is 3 MB. The memory allocated for a FLAC^{3D} model is adjusted dynamically to take up the required RAM for a model at the time the model is created. If the model requires more memory than is available by the system, Windows will begin to start using virtual RAM by swapping data between the RAM and the hard disk, creating a loss in performance. If a dramatic performance loss or a "Memory Allocation Error" occurs during a model run, too much memory has been allocated for FLAC^{3D}, and it may be necessary to exit and restart FLAC^{3D} to recover this excess memory. It is often a good idea to do this where more than one memory intensive model is run per session. A model containing 144,000 elements should be able to be generated within 224 MB RAM where the material specified is Mohr-Coulomb.

FLAC^{3D} has a partial Graphical User Interface for basic commands for performing plotting, printing and file access. The command driven mode is very similar to that used by other Itasca software products. Most of the commands are the same as or three-dimensional extensions of those used in FLAC.

There are two ways for command input; "interactive" mode where the commands are entered directly into the command prompt and executed line by line, or "batch" mode where scripts are created and modified with a text editor (Microsoft Notepad or similar) and called by FLAC. Files

By default, all files are called, saved and restored from the FLAC^{3D} default system directory. The SET CD NAME command used in FLAC is not recognised by FLAC^{3D}. However, if one wishes to create folders for specific jobs this may still be achieved using File > Call menu. Once a file is selected from a directory in the *Get Call File* dialog box, this directory becomes the working directory for all other file access during the session, and the CALL, RESTORE and SAVE commands can be used from the command line. The menu titles will vary depending on the active window. Commands shown are relative to either the plot window or the command window.

2. Grid Generation

In FLAC^{3D}, elements and nodes can be defined by a vector of i, j and k values. Example 1 shows a simple grid with 4 x 4 x 4 zones (64 zones).

Example 1
```
GEN zone brick size 4 4 4 ratio 1 1 1 &
      p0(0,0,0) p1(50,0,0) p2(0,40,0) p3(0,0,20)
MODEL elastic
PROP bulk = 1e7 shear = 0.3e7 den=1600
PLOT surf yellow
PLOT set rot 20 0 30
```

To set up the initial finite difference grid, use the *GENERATE zone* command to create a mesh. The *size* keyword defines the number of zones in the grid. In the above example the mesh has four zones in the x-direction, four zones in the y-direction and four zones in the z-direction. By default the z-axis is oriented in the vertical direction. Various zone shapes may be used by using keywords such as *brick, wedge, tetra* and *pyramid.* Others may be found in Table 3.2 of the FLAC^{3D} manual. The axes behave in accordance with the right-hand rule. The *ratio* keyword can be used to vary the spacing of the grid lines gradually. In this example, the model boundary dimensions are 50m x 40m x 20m; the boundary coordinates are defined with the *p0, p1,* and *p2* keywords.

As in FLAC, a material must be specified for the elements by the MODEL command. The PROPERTY command specifies the values of the properties for the material selected.

The use of the PLOT command is different from that in FLAC. Here a surface is plotted, and the view of the surface is rotated in perspective. The details of the PLOT command will be discussed later in the document. The plot shown in Figure B1 should appear on your screen.

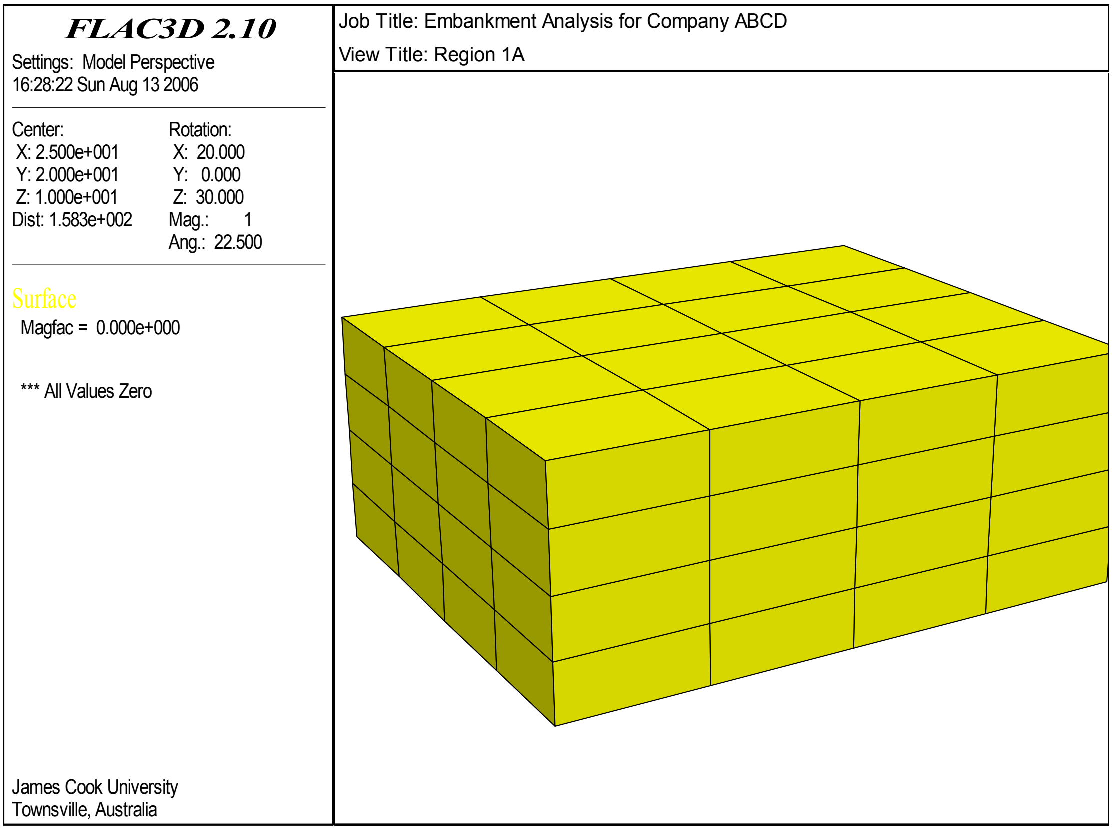

FIGURE B1 – A finite difference mesh in FLAC^{3D}

3. Boundary Conditions

Once the mesh is generated, it is necessary to assign boundary conditions. In Example 1, assuming that the model is being analysed has quarter symmetry.

```
FIX x range x -0.1 0.1
FIX y range y -0.1 0.1
FIX x y z  range z -0.1 0.1
FIX x y z range x 49.9 50.1
FIX x y z range y 39.9 40.1
```

Here it is implied that there are no horizontal deformations along the vertical boundaries on the planes of symmetry, and that there are no deformations in any direction along the bottom horizontal boundary (plane) and the external boundaries (planes) of the model. As in FLAC, the *APPLY* command may be used to include the application of external loads (as stresses) and pore water pressures.

It is important to note that in FLAC^{3D}, element coordinates are not used when specifying boundary conditions (or material properties over a certain region). The dimensions of the region are defined by the x-, y- and z-coordinate range. In the case of boundary fixings, we are interested in fixing the entire plane. Planes are defined by ± 0.1 of the actual coordinate of the plane. Note that external influences (APPLY command) are also applied over a range in a fixed x, y, z space.

Gravitational stresses may be included by assigning a value for gravity.

```
SET grav 0 0 -9.81
```

4. Solving the Problem

The STEP and SOLVE commands are used in the same fashion as FLAC. When the SOLVE command is used, the run will stop when the unbalanced force falls below a limiting value, specified with the SET command:

```
SET mech force = 50
```

A HISTORY command can monitor the change in variables.

```
HIST n=5
HIST unbal
HIST gp zdisp 0,0,19.5
```

The change in variables will be monitored every five calculation steps. By default, it is monitored every 10 steps. Here two variables are monitored: the maximum unbalanced force and the z-displacement at a gridpoint [gp] @ (x=0, y=0, z=19.5). Plotting histories in FLAC^{3D} is the same as plotting histories in FLAC.

5. Output to Screen, Printer and Documents

Inclusion of the following command near the start of the code ensures that the title given will appear on all plots as a job title.

```
TITLE "Embankment Analysis for Company ABCD"
```

For individual plots, view titles may be given as

```
PLOT set title text "Region 1A"
```

The default plot view is identified as `Base/0`, other plot views may be created using the PLOT create command. For this example, the default plot view will be used. *PLOT surf* displays a surface of the plot. This view may then be modified by using the *PLOT set* command followed by the keyword of the property which the user would like to vary. This enables viewing (camera) properties such as magnification, centre coordinates and view rotation to be set. Camera properties may also be adjusted by making the plot window active and selecting Edit > Camera from the menu. Keyboard shortcuts can also be used to control the camera.

Other data may be overlayed on the current plot view by using the *PLOT add* command followed by keywords which will give the user a graphical view of the variable required. Displacements, velocities and stresses may be viewed in this manner. For example, if the vector displacements are required for the example above, this is achieved by adding the following line of code to the end of Example 1.

```
PLOT add disp white
```

Note that the plot display is automatically updated as the problem is being solved if the plot commands are written before the solve or step command. If another plot view is created, the `PLOT show <viewname>` command may need to be used to display the required plot window in the viewport.

When placing the plots into documents, it is better to have a white background as opposed to the default grey background.

```
PLOT set back white
```

`PLOT hard` may be used to send the plot to the printer connected to the computer.
`PLOT clipboard` is a better way to prepare figures for embedding in documents than capturing the screen using Windows Print Screen. The plot shown in Figure 1 was exported to the clipboard as an enhanced windows metafile (.emf).

Similarly to FLAC, data can be tabulated in tables and outputs can be logged and imported into a spreadsheet program such as Microsoft Excel.

```
SET logfile output.log
SET log on
PRINT <variable>
SET log off
```

Appendix C: FLAC and FLAC^{3D} Programs

This code may be used as a basis to extend upon the work completed by myself in this thesis. It should be noted that the supplied code is an example and not necessarily the exact code used to obtain the predictions.

C-1: FLAC Axisymmetric Program

```
new ;  clear all variables

;SPT N value to friction angle and Young's modulus of elasticity correlation

;NOTE: for all friction angle correlations, use N1 (60% efficiency)
;      for all other correlations, use N (60% efficiency)

;      for calculation of N1, assume overburden depth of B

;using the chart by Peck et al.
;x=SPT N, y=friction angle

TABLE 1 insert 8 29.5         ;loose
TABLE 1 insert 10 30          ;loose
TABLE 1 insert 16 32          ;medium dense
TABLE 1 insert 22.5 34
TABLE 1 insert 30 36          ;dense
TABLE 1 insert 38 38
TABLE 1 insert 46 40
TABLE 1 insert 55 42          ;v.dense
TABLE 1 insert 60 43

Title
Settlement of Shallow Foundations on Granular Soil

DEF variables

   b = 3.0 ; footing width, b, m
   h = 4 * b
   w = 4 * b ; ensure that the width has no impact, may reduce at a later stage to 2b
   w1 = b / 2
   h1 = 0.75;  thickness of footing, set this

   w2=w-w1
   h2=h-h1

   N60a = 20 * 0.9 ;  for energy of 53%
   N60b = 18 * 0.9
   N60c = 50 * 0.9

   N1a = N60a*9.78*SQRT(1/(16*(2.5))) ; SPT blow count, corrected for overburden
   N1b = N60b*9.78*SQRT(1/(16*(7.5)))
   N1c = N60c*9.78*SQRT(1/(16*(10.5)))

;This will be used later in the model
   e_mod1 = 8e5*N60a ;  should be within range 5e6 to 50e6 Pa
   e_mod2 = 8e5*N60b
   e_mod3 = 80e6 ; see Journal Entry (25 Jul) for reasoning
   phi1 = table(1,N1a) ; friction angle, deg, derive from table, range 30 to 42
   phi2 = table(1,N1b)
   phi3 = 0 ; undrained clay layer

   p_ratio = 0.2
   shear1 = e_mod1 / (2.0 *(1.0 + p_ratio))  ; shear modulus, Pa
   shear2 = e_mod2 / (2.0 *(1.0 + p_ratio))
   shear3 = e_mod3 / (2.0 *(1.0 + p_ratio))
   bulk1 = e_mod1 / (3.0 *(1.0 - 2.0 * p_ratio))  ; drained (dry) bulk modulus, Pa
   bulk2 = e_mod2 / (3.0 *(1.0 - 2.0 * p_ratio))
   bulk3 = e_mod3 / (3.0 *(1.0 - 2.0 * p_ratio))
```

```
   dens = 1600 ; density of soil, kg/m^3, assume that soil has the same density for
                    all the layers
   coh1 = 0; no cohesion in granular soils
   coh2 = 0
   coh3 = 200e3

end
variables

CONFIG axi; impose axisymmetric loading for circular footing

define grid_dim ; define the grid dimensions
   ga=20        ; no of elements in i dir, ga - grids horizonal, MUST BE EVEN
   gc=25 ; no of element in j dir, gc - grids vertical
   gc1=gc-1
   ga1=5 ; number of grid elements covering the footing and soil beneath the footing
   gaa1=ga1+1
   gcc1=gc1+1
   gaa=ga+1 ; limit of grid, i dir
   gcc=gc+1 ; limit of grid, j dir
end
grid_dim

;define grid
GRID ga gc

;constitutive model and soil parameters - INITAILLY DEFINE MATERIAL AS ELASTIC TO
       PREVENT YIELDING

;Layer 1
MODEL elastic i = 1,ga j=12,25
PROP den=dens bulk=bulk1 shear=shear1 i=1,ga j=12,25

;Layer 2
MODEL elastic i = 1,ga j=5,11
PROP den=dens bulk=bulk2 shear=shear2 i=1,ga j=5,11

;Layer 3
MODEL elastic i = 1,ga j=1,4
PROP den=dens bulk=bulk3 shear=shear3 i=1,ga j=1,4

;generate co-ords for mesh
;need to control the depth of the foundation, therefore create 4 regions
GEN 0,0 0,h2 w1,h2 w1,0 i=1,gaa1 j=1,gc1 rat=1.0,0.95    ; mesh for loaded zone
       beneath footing
GEN w1,0 w1,h2 w,h2 w,0 i=gaa1,gaa j=1,gc1 rat=1.1,0.95   ; mesh for soil adjacent to
       zone beneath footing
GEN 0,h2 0,h w1,h w1,h2 i=1,gaa1 j=gc1,gcc rat=1.0,0.95    ; mesh for footing zone
GEN w1,h2 w1,h w,h w,h2 i=gaa1,gaa j=gc1,gcc rat=1.1,0.95   ; mesh for soil adjacent
       footing

;boundary conditions
;FIX x i=1 ***This becomes redundant in the axisymmetric model
FIX x  i=gaa
FIX y j=1

set grav 9.81
set large

solve ; establish insitu conditions stepping to initial equilibrium
plot hold syy fill grid
;Excavate the soil, solve for this

MODEL null i=1,ga1 j=gc1,gcc
solve
plot hold syy fill grid

;construct the concrete footing, solve for this

;Layer 1
MODEL mohr i = 1,ga j=12,25
PROP den=dens bulk=bulk1 shear=shear1 fric=phi1 coh=coh1 ten=0 i=1,ga j=12,25

;Layer 2
MODEL mohr i = 1,ga j=5,11
PROP den=dens bulk=bulk2 shear=shear2 fric=phi2 coh=coh2 ten=0 i=1,ga j=5,11
```

```
;Layer 3
MODEL mohr i = 1,ga j=1,4
PROP den=dens bulk=bulk3 shear=shear3 fric=phi3 coh=coh3 ten=0 i=1,ga j=1,4


MODEL elastic i=1,ga1 j=gc1,gcc
PROP dens=2380 bulk=1.38e10 shear=1.26e10 i=1,ga1 j=gc1,gcc ; typical values for
concrete
solve
plot hold syy fill grid


;reset the displacements to zero, keeping initial stresses
ini ydisp 0 ;  reset displacements to zero
ini xdisp 0
plot hold model grid

;use FISH so that pressure is incrementally applied

def loadstep
   loop n (1,20) ; plot to 1000 kPa, failure to occur by this point in most soils
        foot_pressure = foot_pressure + stress_inc
        command
           apply pressure = foot_pressure i=1,gaa1 j=gcc
           solve
        end_command

         disp = ydisp(1,gc1)
        ytable(2,n)=foot_pressure
        xtable(2,n)=disp

   end_loop
end
set foot_pressure=0.0 stress_inc=5e4 ; positive downwards (50 kPa)

hist unbal
hist disp
hist foot_pressure

loadstep

save ax_sett1.sav

set log ax_sett94.log
set log on

print table 2

set log off

call ax_sett2.txt
```

C-2: FLAC^{3D} Program

```
new

;define a timer function

def test0
   tim0=clock
end
def test1
   tim=(clock-tim0)/100
end

;SPT N value to friction angle and Young's modulus of elasticity correlation

;NOTE: for all friction angle correlations, use N1 (60% efficiency)
;      for all other correlations, use N (60% efficiency)

;      for calculation of N1, assume overburden depth of half layer thickness

;using the chart by Peck et al.
;x=SPT N, y=friction angle

TABLE 1 insert 8 29.5         ;loose
TABLE 1 insert 10 30          ;loose
TABLE 1 insert 16 32          ;medium dense
TABLE 1 insert 22.5 34
TABLE 1 insert 30 36          ;dense
TABLE 1 insert 38 38
TABLE 1 insert 46 40
TABLE 1 insert 55 42          ;v.dense
TABLE 1 insert 60 43

;plot hold table 1

Title
Settlement of Shallow Foundations on Granular Soil

DEF variables

   b =3.0 ; footing width, b, m
   l =3.0

ga=25 ; grids in the x (from left to right)
gb=25; grids in the y (back into page)
gc=25; grids up the page, vertical
w=4*b ; width across the page
d=4*b ; depth into the page
h=4*b ; height up the page

ga1=5 ; grids for the footing
gb1=5 ;
gc1=3 ;

ga2=ga-ga1 ; grids for remaining soil
gb2=gb-gb1
gc2=gc-gc1

w1=b/2
d1=l/2
h1=0.75 ; thickness of footing, set this

w2=w-w1
d2=d-d1
h2=h-h1

;use the next variables in the loop and for fixings
aa=ga-0.1
aaa=ga+0.1
bb=gb-0.1
bbb=gb+0.1
cc=gc-0.1
ccc=gc+0.1
ww=w-0.1
www=w+0.1
dd=d-0.1
```

```
ddd=d+0.1
hh=h-0.1
hhh=h+0.1

   N60a = 20 * 0.9 ; @ depth of b
   N60b = 18 * 0.9
   N60c = 50 * 0.9

   N1a = N60a*9.78*SQRT(1/(16*(2.5))) ; SPT blow count, corrected for overburden at
        depth B below footing
   N1b = N60b*9.78*SQRT(1/(16*(7.5)))
   N1c = N60c*9.78*SQRT(1/(16*(10.5)))

;This will be used later in the model
   e_mod1 = 8e5*N60a ;  should be within range 5e6 to 50e6 Pa
   e_mod2 = 8e5*N60b
   e_mod3 = 80e6 ; see Journal Entry (25 Jul) for reasoning
   phi1 = table(1,N1a) ; friction angle, deg, derive from table, range 30 to 42
   phi2 = table(1,N1b)
   phi3 = 0 ; undrained clay layer


   p_ratio = 0.2
   shear1 = e_mod1 / (2.0 *(1.0 + p_ratio))  ; shear modulus, Pa
   shear2 = e_mod2 / (2.0 *(1.0 + p_ratio))
   shear3 = e_mod3 / (2.0 *(1.0 + p_ratio))
   bulk1 = e_mod1 / (3.0 *(1.0 - 2.0 * p_ratio))  ; drained (dry) bulk modulus, Pa
   bulk2 = e_mod2 / (3.0 *(1.0 - 2.0 * p_ratio))
   bulk3 = e_mod3 / (3.0 *(1.0 - 2.0 * p_ratio))

   dens = 1600 ; density of soil, kg/m^3
   coh1 = 0; no cohesion in granular soils
   coh2 = 0
   coh3 = 200e3

end
variables

;base soil layer

gen zone brick size ga1 gb1 gc2 rat 1,1,0.9 &
        P0 (0,0,0) P1 (w1,0,0) P2(0,d1,0) P3 (0,0,h2)
gen zone brick size ga2 gb1 gc2 rat 1.1,1,0.9 &
        P0(w1,0,0) p1(w,0,0) p2(w1,d1,0) P3 (w1,0,h2)
gen zone brick size ga2 gb2 gc2 rat 1.1,1.1,0.9 &
        p0(w1,d1,0) p1(w,d1,0) p2(w1,d,0) p3 (w1,d1,h2)
gen zone brick size ga1 ga2 gc2, rat 1,1.1,0.9 &
        p0(0,d1,0) p1(w1,d1,0) p2(0,d,0) p3(0,d1,h2)

;top layer of soil and concrete
gen zone brick size ga1 gb1 gc1 rat 1,1,0.9 &
        P0 (0,0,h2) P1 (w1,0,h2) P2(0,d1,h2) P3 (0,0,h) ;elastic model

gen zone brick size ga2 gb1 gc1 rat 1.1,1,0.9 &
        P0(w1,0,h2) p1(w,0,h2) p2(w1,d1,h2) P3 (w1,0,h)
gen zone brick size ga2 gb2 gc1 rat 1.1,1.1,0.9 &
        p0(w1,d1,h2) p1(w,d1,h2) p2(w1,d,h2) p3 (w1,d1,h)
gen zone brick size ga1 ga2 gc1, rat 1,1.1,0.9 &
        p0(0,d1,h2) p1(w1,d1,h2) p2(0,d,h2) p3(0,d1,h)

MODEL elastic
PROP den=dens bulk=bulk1 shear=shear1 ;fric=phi1 coh=coh1 ten=0

set large

;APPLY BOUNDARY CONDITIONS

fix x range x -0.1 0.1
fix y  range y -0.1 0.1
fix z range z -0.1 0.1
fix x y z range x ww www
fix x y z range y dd ddd

set gravity = 0,0,-9.81

;initialise model, solve for equilibrium, start timer
```

```
set mech force=50
plot surf
plot add axes
plot add con szz
plot set rot 20 0 30

test0 ; timer start
solve

;excavate and construct the foundation

MODEL null range x=0,w1 y=0,d1 z=h2,h
solve

;Layer 1
MODEL mohr range x=0,w y=0,d z=7,25
PROP den=dens bulk=bulk1 shear=shear1 fric=phi1 coh=coh1 range x=0,w y=0,d z=7,25

;Layer 2
MODEL mohr range x=0,w y=0,d z=3,7
PROP den=dens bulk=bulk2 shear=shear2 fric=phi2 coh=coh2 range x=0,w y=0,d z=3,7

;Layer 3
MODEL mohr range x=0,w y=0,d z=0,3
PROP den=dens bulk=bulk3 shear=shear3 fric=phi3 coh=coh3 range x=0,w y=0,d z=0,3

MODEL elastic range x=0,w1 y=0,d1 z=h2,h ; properties of concrete
PROP dens=2380 bulk=1.38e10 shear=1.26e10 range x=0,w1 y=0,d1 z=h2,h
solve

ini xdisp=0 ydisp=0 zdisp=0 ; set all displacements to zero

plot surf
plot add axes
plot set rot 20 0 30
plot add disp white
def loadstep
   loop n (1,6) ;
        foot_pressure = foot_pressure + stress_inc
        command
           apply szz = foot_pressure range x=0,w1 y=0,d1 z=hh,hhh
          solve

        end_command

        pdis0=gp_near(0,0,h2)
         disp = gp_zdisp(pdis0)
        command
          print disp
        end_command
        ytable(2,n)=foot_pressure ; The x and y have been interchanged because of
               interpolation
        xtable(2,n)=disp          ; requirements in FLAC, remember to fix this in
               EXCEL.
        end_loop
   end
set foot_pressure=0.0 stress_inc=-0.5e5 ; negative is downwards (100 kPa)

hist n=10
hist unbal
hist disp
hist foot_pressure
;hist gp_zdisp 0,0,h1 ; record history of disp directly beneath footing

loadstep

test1; stop timer

save 3d_sett1_trial3.sav ; save current run data

set logfile 3d_sett_trial3.log
set log on

print tim
print table 2
set log off
```