University of Southern Queensland

Faculty of Engineering and Surveying

Advanced Analysis of Shallow Foundations Located Near Slopes

A dissertation submitted by

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Abstract

The geotechnical problem of the rigid shallow foundation resting near a slope or cut is a problem that is commonly experienced within engineering practice. Due to the complex nature of sloped soil structures that are subjected to foundation loading, past numerical models have been based on simplified assumptions that propose to produce conservative results for bearing capacity. This project illustrates the use of explicit finite different software (FLAC) to numerically model and analyse the behaviours of slopes under foundation loading at an advanced level. The purpose of this research is to produce a qualitative set of results for the shallow rigid foundation resting near a slope and use them to validate the previous simplified numerical models of the foundation problem.

The advanced FLAC models used to obtain results within this study have been validated against a number of available solutions. These included Explicit Finite Difference, Upper Bound – Lower Bound and physical model solutions. The focus of this study is to produce a weighted foundation and investigate the effects of foundation weight, the interface conditions between the rigid foundation base and underlying soil structure, discontinuous foundation punching into the soft clay material and large strain analysis of the model.

In addition to the studies conducted for the advanced analysis of the shallow rigid foundation problem, analysis of static pseudo seismic foundations was conducted, to investigate the effects of earthquake-induced horizontal forces within the model. Within this section of study comprehensive parametric studies were conducted into the effects of the H/B, D/B and Soil Strength Ratios.

The results obtain from this research project included; that the modelling of the building weight under small strain analysis for a smooth soil structure interface was the most conservative modelling method and the comprehensive parametric study of the static pseudo seismic forces gave an interesting insight into the complex design problem.

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I further certify that the work is original and has not been previously submitted for assessment in any other course of institution, except where specifically stated.

Renee Grace Peters

Student Number: 0080086490

Signature

Date

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Table of Contents

List of Figures

List of Tables

Nomenclature

The principal symbols used are presented in the following list. Locally used notation and modifications, such as by addition of a subscript or superscript, and a symbol that has different meanings in different contexts are defined where used.

B	width of footing.
β	slope angle.
c	soil cohesion.
$c/\gamma B$	soil strength ratio.
D/B	footing distance ratio.
D	distance of footing from slope edge.
E	Young's modulus of elasticity.
F_s	safety factor.
H	height of slope.
H/B	slope height ratio.
N	stability number.
p	averaged pressure below foundation.
$p/\gamma B$	normalised bearing capacity.
q	surcharge pressure.
q_a	allowable bearing capacity.
q_u	ultimate bearing capacity.
$q/\gamma B$	normalised surcharge pressure.
ϕ	friction angle of soil.
γ	unit weight of soil.
k_H	coefficient of horizontal acceleration.
W	weight of soil structure.

Introduction

1

1.1 Outline of the Study

This dissertation endeavours to produce a range of qualitative results that investigate "real life" shallow foundation conditions, for a foundation located near a slope consisting of a pure cohesive soil. The results obtained will be presented in such a way as to be used within a validation process of previously produced design charts and tables for the simplified version of the problem of shallow foundations located near slopes. An explicit finite difference program, Fast Lagrangian Analysis of Continua, will be used throughout this project to produce a series of advanced models. These advanced models will then be used to produce qualitative results for the foundation problem. All results obtained from the explicit finite difference program, will be validated against previous published works on the same problem description.

The advanced models that will be produced, validated and analysed within this dissertation include;

- A soil structure interface model.
- A discontinuous foundation punching model.
- A large strain analysis model.
- A static pseudo seismic model.

From conducting qualitative studies into the above four ‘real life’ conditions this dissertation aims to be a validation tool for previously produced preliminary design charts, produced within past dissertations, for the shallow rigid foundation located near a purely cohesive slope problem. Thus this dissertation differs from past dissertations on this particular geotechnical topic, as it aims to be a qualitative study rather than a quantitative study that can be used to validate whether or not past dissertational findings are accurate.

The static pseudo seismic study that will be presented within this dissertation is a comprehensive study that aims to evaluate the effects of the additional earthquake induced horizontal forces that occur during a seismic event. In order to fully investigate the effect that an earthquake event would have on a shallow rigid foundation located near a slope, a number of parametric studies will be conducted to investigate the effects on the foundation’s ultimate bearing capacity. The parameters that will be investigated include;

$c/\gamma B$	soil strength ratio.
D/B	footing distance ratio.
H/B	slope height ratio.
K_H	coefficient of horizontal acceleration.

Knowledge of these effects will aid in future studies within the area and within future development of seismic bearing capacity preliminary design charts.

1.2 Background Information

Throughout the history of foundations, the problem of the rigid shallow foundation situated near a slope has been a design and construction issue for many engineers, and thus has been the subject of numerous studies. Foundations are an essential component of any structure and have a primary purpose of transferring concentrated loads produced by a structure to the underlying foundation material. Some common examples of foundations include basement excavations for high-rise buildings, bridge abutments and tower footings for electrical transmission lines. When a foundation is constructed near a slope additional design parameters are introduced that are often difficult to evaluate thus making the design process complex and drawn out. To overcome these design difficulties and time issues, past studies have proposed design charts to easily evaluate the capacity of a soil structure under foundation loading. This project has incorporated the use of the explicit finite difference numerical modelling program, FLAC, to investigate "real life" characteristics of rigid shallow foundations located near slopes. This modelling has been considered to be advanced as it takes into consideration "real life" characteristics of foundations that are normally conservative within current design processes. From modelling the advanced characteristics of a foundation, the ultimate bearing capacity that can be applied to the underlying soil structure to induce failure is produced. This ultimate bearing capacity is then used to compare the previously produced design charts and tables with the advanced modelling of the problem, to validate whether or not the charts and tables could be used within a preliminary design process for a foundation situated near a slope. Overall the project focuses on defining the ultimate bearing capacity of the soil structure so that it can be compared with previously proposed design methods.

In addition to this advanced foundation characteristic modelling, the FLAC program will be also used to model the effects that static pseudo seismic forces would have on a foundation located near a slope, during an earthquake event. As this project focuses on obtaining the ultimate bearing capacity of a sloped soil structure under foundation loading, the results may be limited by foundation bearing capacity or slope stability. But due to the scope of this project only the foundation bearing capacity will be considered, thus for the purpose of this project all slopes are assumed to be marginally stable, thus having a factor of safety equal to one, and should not be subjected to further loading.

1.2.1 Foundations

A foundation is a structural component that is situated below ground level that transfers the load from the structure above ground level into the underlying soil structure. Due to soil being a relatively weak material the load is required to be transferred at an increased volume and area in order to prevent over settlement within the soil structure or gross failure. There are two main types of foundations; shallow foundations and deep foundations, but due to the scope of this project only shallow foundations will be discussed. When designing a shallow foundation it is very important to obtain sufficient values for the allowable bearing capacity, to calculate a suitable factor of safety that will minimise settlement within a structure. There are four main types of shallow foundations; isolated spread footings, combined footings, strip footings and mat footings, but the most common for a building structure is spread footing. Overall the design of a footing is based on the allowable bearing capacity which is the maximum pressure that a soil structure can be subjected to by a foundation before overstressing and failure occurs.

1.2.2 Ultimate Bearing Capacity

Ultimate bearing capacity, symbolised as q_u, is the limiting load that a foundation cannot exceed without causing shear failure within a soil structure. Evaluation of this ultimate bearing capacity is a difficult process as it is difficult to evaluate the shear strength parameters within the underlying soil structure. When a soil structure is subjected to loading from a foundation, the load per unit area will gradually increase and the foundation will undergo a certain level of settlement. It is important when designing the foundation to take into consideration the level of settlement that will occur with different foundation areas and weights, in order to minimise this effect.

When a foundation is designed there are three types of failure mechanisms that could occur when the ultimate bearing capacity is exceeded. Depending on which failure mechanism occurs will determine the ultimate bearing capacity and settlement occur, as each mechanism varies and magnitude and depth. The three failure mechanisms for a pad footing include; general shear failure, local shear failure and punching shear failure. Each of the three failure types has been discussed below in more detail.

1.2.3 General Shear Failure

General shear failure can be defined as a diagonal slipe surface movement of a well-defined wedge beneath a foundation that initially forces the side edges of the footing downwards into the soil structure, followed by an upwards movement to the ground surface. This causes the soil structure adjacent to the footing to bulge or hump above ground level. In addition to the footing being displaced the footing can also be subjected to a certain level of tilting, but this is dependent on foundation restraint. General shear failure will typically occur within soils that posses a brittle-type of stress-strain relationship. Figure 2.1 depicts a foundation undergoing general shear failure and a load verse settlement plot of the failure.

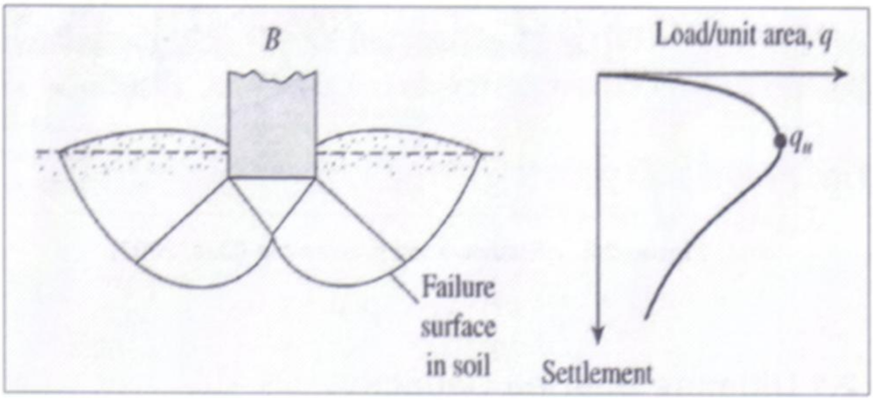

Figure 1-1: General Shear Failure (Das, 2007)

2.2.4 Local Shear Failure

Local shear failure can be defines as a well-defined wedge of soil below a foundation being subjected diagonally downwards like general shear failure, but the depth of the downward movement is increased, thus the slip surfaces within the soil structure beyond the foundation edges fade before they are seen at ground level. Only very slight bulging of the ground surface is the result of this failure mode, thus it can go undetected. Due to this behaviour of high soil compression directly below the foundation and the movement of the foundation upwards, this failure mode represents a transitional failure mode between general and punching shear failure. This type of failure is most common within soil structures that possess a plastic stress-strain relationship. Figure 2.2 depicts a foundation undergoing local shear failure and a load verse settlement plot of the failure.

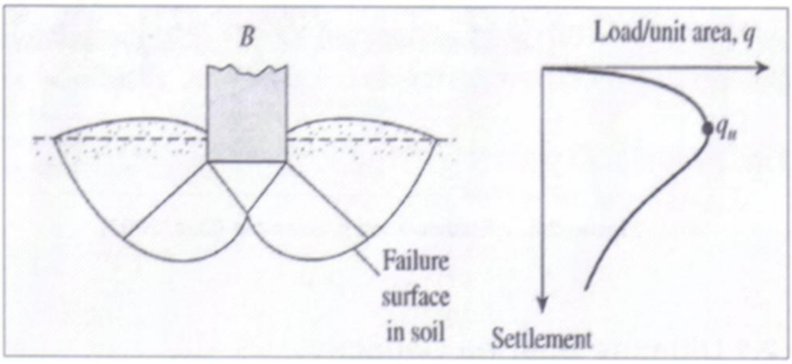

Figure 1-2: Local Shear Failure (Das, 2007)

2.2.5 Punching Shear Failure

Punching shear failure can be defined as a well-defined wedge of soil below a foundation being subjected to a significant level of compression as well as vertical shearing beneath the foundation. The soil structure either side of the foundation only undergoes minimal affect during this failure mechanism and thus only very minimal surface bulging is present at the soil surface, which is general undetected. This type of failure is common within soil structures that possess a very plastic stress-strain relationship, with very poorly defined shearing planes. Figure 2.3 depicts a foundation undergoing punching shear failure and a load verse settlement plot of the failure.

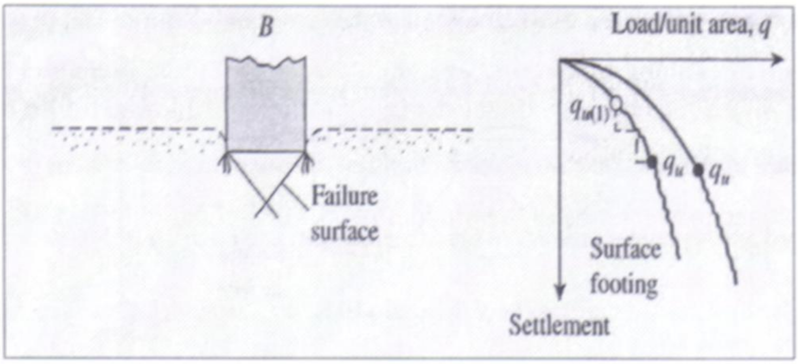

Figure 1-3: Punching Shear Failure (Das, 2007)

1.3 Objectives of Research

The modelling and analysis of a shallow foundation located near a slope can be quite a complex problem, as there are many different parameters and conditions that need to be taken into consideration to fully evaluate the ultimate bearing capacity of a foundation. As this project aims to produce an advanced model for a shallow foundation located near a purely cohesive slope, only the following four "real life" foundation conditions will be considered; the effect of the soil structure interface, the effect of the discontinuous foundation punching into the soft clay soil structure, the effect of large strain analysis and the effect of static pseudo seismic forces on the soil structure. A general problem description for the project has been presented within Figure 4.1.

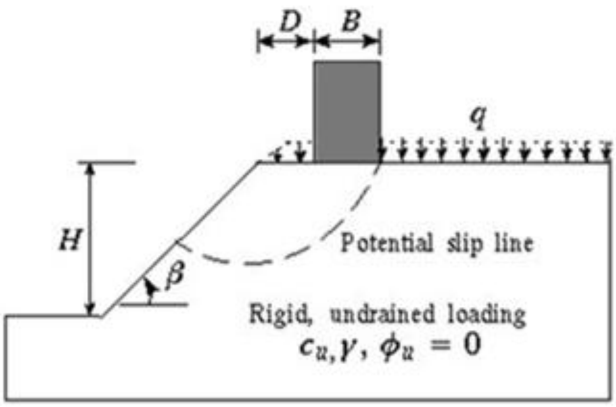

Figure 1-4. Problem notation and potential failure mechanism for advanced study.

The objective of this project is to use the finite difference modelling software package, FLAC, to model the four advanced conditions, previously mentioned, in order to produce a qualitative set of ultimate bearing capacity results for the soil structure under shallow foundation loading. In modelling this advanced model, an actual foundation will be modelled within the FLAC mesh, in order to evaluate the elements of the foundation; this has not previously been done, previously only velocities have been applied at an imaginary footing location. All results from this advanced model will be validated against existing works with the same problem description. On completion of this project the qualitative set of results for the ultimate bearing capacity produced will be used as a validation tool for previously produced design charts and methods that propose to be conservative methods of obtaining the ultimate bearing capacity. Thus it is proposed that the results obtained from this study, will be less conservative, but will be a more realistic representation of rigid shallow foundations located near slopes, as certain real life foundation characteristics, such as; interfaces and static pseudo seismic forces have been taken into consideration.

1.4 Process

The project has been broken down into the several manageable components to ensure that the project is successfully completed. These project components are as follows;

1. Research background information for the project.
2. Develop FLAC programming skills.
3. Produce the advanced FLAC model for a horizontal interface between foundation and soil structure.
4. Validate this soil structure interface FLAC model and conduct advanced studies.
5. Produce the advanced FLAC model for the discontinuous foundation punching effect.

6. Validate the discontinuous foundation punching FLAC model and conduct advanced studies.
7. Conduct research into static pseudo seismic forces and foundations.
8. Produce the FLAC model for a static pseudo seismic situation.
9. Validate the static pseudo seismic FLAC model.
10. Conduct a parametric study using the static pseudo seismic FLAC model.
11. Produce a series of design charts for the static pseudo seismic model.
12. Conclude the dissertation and discuss any future work.
13. Complete and submit dissertation.

1.5 Overview of Chapters

This dissertation presents a series of models for advanced analysis of the foundation located near a slope problem. The topics presented within this dissertation are; an introduction and background information into the project, a literature review of past findings, including past dissertation FLAC modelling, the development, validation and an advance study into the role the interface between the base of the foundation and the underlying soil structure plays on the ultimate bearing capacity of the soil, a repeated process for the vertical interface between the foundation corners and soil structure, a study into static pseudo seismic foundations and the effect the conditions have on the soil structure and finally a parametric study into the static pseudo seismic model, with a series of design charts and tables produced. Outlined below is a brief description of each chapter.

1.5.1 Chapter 1 - Introduction

This chapter presents the outline of the study, an introduction into the problem along with the essential background information for the problem and a discussion of the project's objectives and methodology.

1.5.2 Chapter 2 - Literature Review

This chapter will present a literature review of all past studies into the bearing capacity problems for foundations to introduce the project and give a background into why this study is required. Included within the literature review will be findings of past researchers, results from past dissertational FLAC modelling of the problem and finally an overview of the current available texts on the subject matter of shallow rigid foundations located on or near slopes.

1.5.3 Chapter 3 – Introduction to FLAC Analysis

This chapter will present a brief introduction into the software, FLAC that was used throughout this project. It will present the capabilities of the program along with the methods of modelling the project problem within the program and the analysis of editing of the results obtained from the program.

1.5.4 Chapter 4 – The Advanced Modelling of the Soil Structure Interface

This chapter will present the advanced modelling of the horizontal interface that is present between the soil structure and the base of the foundation. Within this chapter a validation of the advanced model will be conducted, along with the use of the advanced model in the analysis of the effects of extremely smooth interfaces and extremely rough interfaces for a foundation that has a density of 2000kg/m^3, for a range of different footing distance ratios.

1.5.5 Chapter 5 – The Advanced Modelling of the Discontinuous Foundation Punching

This chapter will present the advanced modelling of the discontinuous foundation punching and separation that occurs when the load is released on the soft clay soil structure. Within this chapter a validation of the advanced model will be presented along with the use of the model in the analysis of smooth and rough interfaces, different vertical interface lengths and for a range of different footing distance ratios.

1.5.6 Chapter 6 – The Advanced Modelling of Large Strain Analysis

This chapter presents the advanced modelling of the large strain analysis for the previously presented models within chapters four and five. Within this chapter the large strain analysis results will be compared with those obtained for the small strain analysis conducted within chapters four and five.

1.5.7 Chapter 7 – The Advanced Modelling of Static Pseudo Seismic Forces

This chapter present the advanced modelling of the static pseudo seismic forces on a rigid shallow foundation located near a slope. Within this chapter will be a brief introduction to Pseudo Seismic Forces, the preparation of the model, the validation of the model the use of the model to investigate the following three design parameters; footing distance ratio, slope height ratio and soil strength ratio.

1.5.8 Chapter 8 – Conclusion

This chapter will present the overall findings from each of the four advanced modelling studies presented within chapters four, five, six and seven, In addition this chapter will make a final conclusion on the status of previous studies that proposed to have constructed design charts and tables that conservatively calculated the ultimate bearing capacity for a rigid shallow foundation located near a slope, that can easily be used within preliminary foundation designs.

1.6 Summary

The objective of this chapter was to give the dissertation reader an introduction and a basic understanding of the content of the studies that are presented within this dissertation. From this chapter it is evident that there are many aspects that require consideration throughout the duration of this project. The following chapter presents the literature review of past studies that have been conducted within this project topic.

Literature Review

2

2.1 Introduction

This section of the report presents a summary of the previous research that has been conducted and published within geotechnical textbooks and journal papers, on the subject of ultimate bearing capacity for a footing on both flat ground conditions and on sloped conditions. Unfortunately it was determined from this literature review that the research previously conducted within this area of study has not been extensive, with majority of the works surrounding the footing on a flat ground surface case. All the previously published work on the subject matter adopted a wide range of methodologies to evaluate the effect of the ultimate bearing capacity of a footing on foundation material, some of the methods that have been noted to be adopted are; slip-line, equilibrium, finite element, and upper bound-lower bound methods. The aim of this project is to conduct further advanced finite difference analysis of the problem and produce qualitative less conservative set of solutions for more typical "real life" foundation conditions.

Throughout the years there have been a number of researchers that have conducted studies into the problem of foundations and their ultimate bearing capacity. A number of these theories have been reproduced in a number of different geotechnical textbooks. This literature will be presented in a way to firstly present the different theories of for the foundation on flat and sloped grounds that have been developed throughout the years and then present a summary of the theories, on the subject, that have been presented within a number of geotechnical textbooks, that are from both the foundation and soil mechanics repertoire.

2.2 Past Theories of Footings

Throughout the research of the footings being built on slopes there has been a number of different methods and theories suggested to evaluate the ultimate bearing capacity within the soil structure, but most of this research has been based on Terzaghi's flat ground bearing capacity theory. Due to the vast number of parameters that require consideration when evaluating the ultimate bearing capacity for a foundation, some of the theories presented have had some limitations that have restricted and often excluded their use in current footing design methods. Some of the researchers that have conducted studies into the footings include; Terzaghi (1943), Meyerhof's (1957, 1963), Hansen's (1970), Vesic's (1973), Kusakabe et al. (1981), Narita and Yamaguchi (1990), Georgiadis et al. (2008) and Shiau et al (2007). In addition to this research there has also been some dissertations presented on the subject matter by three previous University of Southern Queensland students; Catherine Smith (2006), Joshua Watson (2008) and Nathan Lyle (2009). These theories have been presented below.

2.2.1 Terzaghi's (1943) Flat Ground Bearing Capacity Theory

The first comprehensive theory for the ultimate bearing capacity on flat ground was presented by Terzaghi (1943). Terzaghi developed the general equation for a strip footing that considered the following factors; soil cohesion, internal friction, foundation size, soil weight and surcharge effects. Terzaghi's equation utilized non-dimensional bear capacity factors, that had values that were functions of supporting soils shear. Terzaghi's theory was based on the theory of plasticity, which was a slight modification of a previous theory presented by Prandtl (ca. 1920), to analyse the punching effect of a rigid base into a softer soil material. The original equation that Terzaghi presented for the Bearing Capacity on a flat ground has been presented bellow in Equation (2.1)

$$q_{ult} = c'N_c + \gamma_1 D_f N_q + \frac{1}{2} B\gamma_2 N_\gamma \dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots\dots Equation\ (2.1)$$

Where:

q_{ult}	soil bearing pressure (kPa).
c'	Cohesion of soil below foundation (kPa).
D_f	depth of footing (meters).
γ_1	unit weight of soil above foundation level (kN/m^3).
γ_2	unit weight of soil below foundation level (kN/m^3).
B	width of footing (meters).
N_c, N_q, N_γ	Non-dimensional bearing capacity factors.

The evaluation of the non-dimensional bearing capacity factors, have been previously evaluated and presented by other researchers. Reissner (1924) presented an equation to obtain N_q, Prandtl (1921) presented an equation to obtain the value of N_c, and both Caquot and Kerisel (1953) and Vesic (1973) presented an equation to obtain N_γ.

In addition to Terzaghi's general equation for the ultimate bearing capacity for continuous and strip foundations, he also made some alterations to the equation presented within Equation (2.1), to determine the ultimate bearing capacity for square and circular foundations.

2.2.2 Meyerhof's (1963) Bearing Capacity Theory

Meyerhof (1963) produced an additional equation for obtaining the bearing capacity for foundations on flat ground. Meyerhof determined that the earlier equation presented by Terzaghi (1943) neglected to take into consideration two important factors; the effect of shear resistance along the failure surface in the soil situated above the foundation and the effect an inclined foundation loading would have on the bearing capacity. Thus he produced his equation for the bearing capacity of foundations that has been presented within Equation (2.2). Presented within this formula are some additional factors for; shape, depth and inclination.

$$q_u = c'^{N_c F_{cs} F_{cd} F_{ci}} + qN_q F_{qs} F_{qd} F_{qi} + \frac{1}{2}\gamma B N_\gamma F_{\gamma s} F_{\gamma d} F_{\gamma i} \ldots\ldots\ldots\ldots \quad .Equation\ (2.2)$$

Where:

q_{ult}	Soil bearing pressure (kPa).
c’	Cohesion of soil below foundation (kPa).
B	Width of footing (meters).
N_c, N_q, N_γ	Non-dimensional bearing capacity factors.
F_{cs}, F_{qs}, $F_{\gamma s}$	Shape Factors.
F_{cd}, F_{qd}, $F_{\gamma d}$	Depth Factors.
F_{ci}, F_{qi}, $F_{\gamma i}$	Inclination Factors.

2.2.3 Hansen's (1970) and Vesic's (1973) Bearing Capacity Theories

Hansen (1970) further developed Meyerhof's (1963) equation for the bearing capacity, by including additions factors such as; base factors for situations where the footing may be tilted from the horizontal.

Vesic (1973) developed his own bearing capacity theory, but it was basically the same as Hansen (1970). The major difference between the two theories lied in the calculation of one of the bearing capacity factors and the inclination, base and ground factors.

2.2.5 Meyerhof’s (1957) Sloped Ground Bearing Capacity Theory

Meyerhof (1957) developed a theoretical relationship for the ultimate bearing capacity of shallow rigid foundations located on top of a slope. His theoretical relationship for the ultimate bearing capacity was a minor variation of Terzaghi's (1943) flat ground bearing capacity theory. Meyerhof's (1957) equation for a continuous foundation has been presented within Equation (2.3).

$$q_u = c' N_{cq} + \frac{1}{2} B \gamma N_{\gamma q} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ Equation\ (2.3)$$

Variations to this general equation have been made for purely cohesive soils and purely granular soils, where the equation has been simplified with respect to the level of cohesion and friction angle. Meyerhof also developed design charts for obtaining the value for the bearing capacity factors; N_{cq} and $N_{\gamma q}$.

2.2.6 Kusakabe et al. (1981)

Kusakabe et al. (1981) presented an upper bound plasticity solution to the vertical loading of footing on slopes. This method produced an understanding of the soil strength relationship within the slope and was the first to introduce the concept of the soil strength ratio into the model. There were some limitations to the upper bound method presented by Kusakabe et al. as the results produced were less than those of a physical modelling of the problem. It was concluded that this difference was due to the lack of considerations made within the model for the friction between the footing and foundation material, thus producing an overly conservative result of the ultimate bearing capacity.

2.2.7 Narita and Yamaguchi (1990)

Narita and Yamaguchi (1990) presented their research into the bearing capacity factor for footings on slopes that adopted the method of log-spiral solution. These researches made use of the previously established soil strength ratio and the normalised bearing capacity to evaluate the slopes behaviour and bearing capacity. Validation of this log-spiral solution method was conducted against actual physical modelling of the problem and Bishops results. The scope of parameters researched within this method was very limited and the main finding from this research was that the values obtained for the bearing capacity were overestimates in comparison to Bishop's results.

2.2.8 Georgiadis et al. (2008)

Georgiadis et al. (2008) presents a finite element analysis of a strip footing near or on undrained soil slopes. This study was conducted as many of the available methods of evaluating the bearing capacity; equilibrium methods and upper bound plasticity calculations, failed to take into consideration undrained bearing capacity factor, footing distance ratios footing height ratios, the slope height and the soil properties. From this finite element research comprehensive design charts and tables were produced. The major findings from this research conducted by Georgiadis et al. (2008) were that the design charts and tables previously produced by limit equilibrium and upper bound methods were less conservative then those produced within this research.

2.2.6 Shiau et al (2007)

Shiau et al (2007) conducted a study and produced a research paper on undrained stability of footings on slopes. His research presents a series of plasticity solutions for the ultimate bearing capacity of footings located on purely cohesive slopes. The methodology applied within this research paper was a finite element numerical upper and lower bound bearing capacity estimates for strip footings located on purely cohesive slopes. Presented within the research are a number of parametric studies for the problem, this studies include; the effect of the interface between footing and foundation material, the effect of the dimensionless strength ratio, the effect of the slope angle, the effect of the footing distance to the crest, the effect of the surcharge and the effect of footing height ratio. The results obtained from the research were presented within the paper in terms of normalised bearing capacity. It was determined from the study that the effect of the strength ratio has a significant impact on the failure mechanism that will occur, whether it; bearing capacity failure or slope failure. The evaluation of this critical strength ratio has proven to be an important parameter that needs to be carefully considered when designing a foundation near a slope.

2.2.7 Catherine Smith (2006)

Catherine Smith (2006) produced a dissertation paper that tested the reliability of a two-dimensional explicit difference program, called Fast Lagrangian Analysis of Continua or FLAC. This study was conducted for numerous geotechnical problems including the problem of a foundation located near a slope problem for a number of different parameters. All results obtained from the research where validated against existing solutions for the problem. The parametric studies conducted within this research were conducted on a cohesionless soil and parameters considered were; the slope angle, footing distance ratio and the dimensionless strength ratio. The main findings from this research were that the numerical modelling program, FLAC, was producing acceptable results with respect to theoretical bearing capacity values, when the mesh size used to model the problem was reduced. The findings from research has proven to be significantly beneficial, as this software program is the basis of this project, as it is the main software program that will be used.

2.2.8 Joshua Watson (2008)

Joshua Watson (2008) conducted a study using the numerical modelling program FLAC, to investigate the effects of several of non-dimensional parameters and different modelling techniques for the bearing capacity for the problem of the shallow foundation situated near purely cohesive slope. Within this research studies have been conducted into the effect of the footing distance ratio, the footing height ratio, footing length ratio, the effect of the interface between foundation and foundation material and an analysis of large deformation with respect to small deformation. From this research numerous design charts and tables were produced that could be used by consulting engineers to conservatively obtain a value for situation specific ultimate bearing capacities, more effectively than previous methods. Throughout the course of the study there were some issues encountered with the FLAC program and the modelling of the problem, which effected the students overall reliability of results.

2.2.9 Nathan Lyle (2009)

Nathan Lyle (2009) further developed the research conducted by Joshua Watson (2008), by conducting more comprehensive studies into the shallow foundation located near a purely cohesive slope problem. Again the software program FLAC was used and all modelling issues previously encountered were corrected. Nathan conducted a wider range of parametric studies for analysing the ultimate bearing capacity of the shallow foundation near a slope, these parameters include the effect of; footing distance ratio, footing height ratio, strength ratio, surcharge loading and stability number. From these studies a comprehensive set of design charts and tables were produced, with again the endeavour to produce an easy method that a consulting engineer can use, with confidence, to obtain the ultimate bearing capacity of situation specific cases. All result were validation against either the results obtained from Shiau et. al. (2007), available Upper Bound – Lower Bound results and against results obtained from previous physical modelling of the problem. It was determined that the results obtained within this project were approximately 10 percent higher than the upper bound solutions produced by Shiau et. al. (2007), thus the accuracy was within an acceptable range. In addition to the parametric studies some preliminary studies were conducted into the effect of the interface texture between the foundation and foundation material, the main conclusion was that Shiau et. al (2007) findings were accurate and the smooth interface is of a more conservative level than a rough interface.

2.3 Summary of Geotechnical Textbooks

Throughout the years there has been a large variety of geotechnical textbooks produced some of which are the genera of foundation design textbooks and others are purely soil mechanics textbooks. The purpose of this section of the literature review is to establish what methods and theories are being presented within textbooks and validate that there is a need for more research to be conducted into the area of shallow rigid foundations located near slopes, thus validating the relevance of this project. Within the genera of foundation design geotechnical textbooks the following books will be analysed and summarised; The Design and Construction of Engineering Foundations (Henry, 1986), Foundation Analysis and Design (Bowles, 1996), Principles of Foundation Engineering (Das, 2007), and Essentials of Soil Mechanics and Foundations (McCarthy, 2007). The geotechnical books from the soil mechanics genera that have been summarised are; Foundations of Soil Mechanics (Taylor, 1948), Soil: Mechanics and Engineering (Schoustra, 1968) and Engineering Geology Principles and Practices (Price, 2009).

From these geotechnical textbooks, particularly within the foundation design textbooks, there was distinct repetition within the theories and methodologies presented, and the soil mechanics genera neglected to take into consideration the effect of shallow foundations near a slope problem. Thus this summarisation of geotechnical textbooks has reinforced the need for further study to be conducted into the problem of the foundation located near a slope.

2.3.1 The Design and Construction of Engineering Foundations

Henry (1986) presents within his textbook a chapter on Stability Problems in Foundation Engineering. Within this chapter many researchers and their theories have been presented some in which were relevant to this project and some in which were not. These researchers and their theorise on obtaining the ultimate bearing capacity for a foundation have been presented below.

- Firstly Terzaghi’s (1943) initial theory for a shallow foundation’s overburden at the sides of the foundation could be treated as a surcharge, thus the relationship for the ultimate gross base bearing capacity for a strip foundation was produced, this equation has been presented previously within Equation (2.1), and commonly termed Terzahi’s Flat Ground Bearing

Capacity Theory. This theory presents some limitations as is neglects to consider the effect of shear resistance along the failure surface in the soil situated above the foundation and the effect an inclined foundation loading.

- Next Meyerhof's (1952) research was presented, this theory was a general theory for shallow foundation and deep foundations and a series of charts were produced for a number of bearing capacity values. But after further research it was discovered that the depth of the shallow foundation, greatly overestimated the value for the bearing capacity. Thus in 1963, Meyerhof presented a revised set of results for the bearing capacity, in which he used the equation presented within Equation (2.2) of this appreciation. In order to obtain the factors for this equation, many equations and tables have been produced, by a number of different researchers.

- Skempton (1951) conducted an extensive research into foundations on pure clay; from his research he produced a set of design graphs from the bearing capacity of the foundation on clays. Unfortunately these plots had limitations as well as slope stability was not fully investigated.

Unfortunately there were no methods presented within the text to obtain a value for the ultimate bearing capacity for foundations located near or on slopes, and all theories were based on the flat ground conditions.

2.3.2 Foundation Analysis and Design

Bowles (1996) presents within his textbook similar theories of calculating the bearing capacity of shallow foundations, as Henry (1986). Again he presents Terzaghi's (1943) flat ground bearing capacity theory and Meyerhof's (1963) adaptation for Terzaghi's theory to take into consideration the shear resistance along failure surfaces and the effect of inclined foundation loading. But theories by Hansen (1970) and Vesic (1973, 1975), have been also presented. Hansen (1970) presented a theory much like Meyerhof's (1951) work, but considered two additional factors, base factors and ground factors. Tables of values of bearing capacity factors to be used within his equation are the same as Meyerhof's. Vesic's (1973, 1975) theory presented within the text for calculating the bearing capacity on flat ground is very much similar to Hansen (1961). Vesic uses the same equation as Hansen, but adopts a different equation for calculating the bearing capacity factor $N\gamma$.

In addition to the flat ground theories, Bowles' (1996), text presents a section on calculating the bearing capacity of footings on slopes. But within this chapter no theories have been presented, and only a guide to using the additional software to this text has been provided for this problem. Thus again this text neglects to go into depth on the effects of shallow foundation near or on slopes, and vastly presents the footing on flat ground case.

2.3.3 Principles of Foundation Engineering

Das (2007) presents a geotechnical textbook that has provided similar theories as those of the two previous textbooks, but has gone into more depth for the theories that evaluate the bearing capacity of shallow foundation located on slopes. Again Terzaghi's (1943) flat ground bearing capacity is presented along with some variations of Equation (2.1) to take into consideration square and circular foundations. Das also presents a table of values for the bearing capacity factors for Terzaghi's equation. The next theory that was presented was Meyerhof (1963), which was the development of Terzaghi's flat ground theory; again a table of bearing capacity factors for Meyerhof's equation has been presented along with equations to calculate these values. The text also presents a number of different adaptations to the equations used within the bearing capacity equations that have been made by various researchers throughout the years.

The theory presented within Das (2007) on the bearing capacity of foundation on top of slopes, was Meyerhof's (1957) method where he produced the equation previously presented within this appreciation as, Equation (2.3). Two charts have been presented to obtain the Meyerhof's bearing capacity factors for granular soil and purely cohesive soil. Limitations to this formula have been found as it has been proven to overestimate the value for the bearing capacity of the shallow foundations and there is uncertainty if slope stability has been taken into consideration. The text then progresses to present the theories of Meyerhof for bearing capacity of foundation on a slope, but as this is not relevant to this dissertation it will not be discussed further.

2.3.4 Essentials of Soil Mechanics and Foundations

McCarthy (2007) presents a geotechnical textbook that has presented the most extensive information, out of the four summarised textbooks. It presents the following theories;

- That Terzaghi's (1943) theory was based on works completed by Prandtl and Reissner. It presents the formation of the formula along with a chart and table of the values to obtain the bearing capacity factors for the equation. The book also goes onto explain why even though Terzaghi's method has been refined over the years, it has been kept due to its practicality.

- Although not stated within the book, but footings on slopes theory produced by Meyerhof is also presented with the text. The Equation (2.3) of this appreciation and the charts to obtain the bearing capacity factors for purely cohesive and granular soils is also presented. Interestingly there is an additional chart that presents the relationship between the cohesive soils bearing capacity factor and the slope stability factor.

- The text then goes into the effects of Seismic events on bearing capacity of spread footing foundations. This topic is directly related to this project. The equation that has been presented for seismic bearing capacity was presented by Richards et al. (1993) and is;

$$q_{ultE} = cN_{CE} + \frac{1}{2}B\gamma_1 N_{\gamma E} + \gamma_2 D_f N_{qE} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots Equation\ (2.4)$$

Where N_{CE}, $N_{\gamma E}$ and N_{qE} are the bearing capacity factors for earth quake conditions obtained from a series of charts and gamma on and two are the soil weight above and below the foundation and D_f is the depth of the foundation.

From the above text books it is evident that there has been some extensive research been conducted into the area of shallow foundations on flat ground and only minimal research into foundations on or near slopes, thus there is a requirement for more research into this are of study, hence the purpose of this project.

2.3.5 Soil Mechanics Geotechnical Textbooks

Out of the three geotechnical soil mechanics textbooks studied, two of them; Fundamentals of Soil Mechanics (Taylor, 1948) and Soil Mechanics and Engineering (Schoustra, 1968) were mainly focused on presenting the literature on the fundamentals of soil, but did contribute small sections of the basic slope stability and the actions of shallow foundations and bearing capacity. All the information present was very basic summaries of what was presented in the foundation design textbooks. This may be due to the age of the text's selected as the theories and knowledge on foundations and bearing capacities were still being developed. As for the more recent text book, Engineering Geology Principles and Practices (Price 2009), no mention of foundations was made.

2.3.6 Conclusions from Textbook Summary

From the literature review of the current published geotechnical textbooks and soil mechanics textbooks commercially available, it was determined that most theories were based on past studies conducted by reaches such as Terzaghi and Meyerhof, for a flat ground foundation situation. However Meyerhof did present some literature and research into foundation located near inclined land, but the relevance to this study was in adequate. As for the evaluation of seismic foundations there was very little published works, and most researched gathered for this topic was sourced from published research papers. Therefore from the literature presented within the textbooks reviewed it was apparent that most of the theories and method of foundation analysis were either outdated or irrelevant to the problem of the shallow rigid foundation located near a slope.

Through additional research of published papers on the topic of shallow foundations located near slopes it was determined that significant amounts of modelling and analysis of the shallow foundation problem has been conducted within recent years. The availability of the research highlights the need for textbook reviews to ensure that the geotechnical engineering discourse is up to date on foundation design methods.

This literature review of text books has highlighted the need for this study as the results presented within this dissertation are prepared with the aspiration of validating some current modelling methods presented within published research papers. Within this process it is anticipated that accurate research papers be used within updating older textbook design methods.

2.4 Project Resource Requirements

As this is project is based on software analysis techniques there is primarily one software package that will be used; FLAC a numerical analysis program.

2.4.1 Fast Lagrangian Analysis of Continua

FLAC is the major piece of software that will be used throughout the duration of this project. FLAC stands for Fast Lagrangian Analysis of Continua and is a two dimensional explicit finite difference program. Its capabilities include the ability to stimulate and model the behaviour of various structures built on rock, soil or similar materials. A linear or nonlinear stress/strain relationship can be used to describe the behaviour of the pre-described elements of a structure. This finite difference software adopts the use of explicit methods rather than implicit methods, which were commonly adopted within finite element analysis. The benefits of using the explicit method include; the reduced time required for the program to produce a result when analysing a non-linear problem, and the reduced memory requirements that are needed. Depending on the program users skills the software can be either command driven or GUI mode. Unfortunately this software is not user friendly to beginners and a high level of skill within the program is required to accurately model a problem, to ensure accurate results are obtained.

This software is among many other packages that essentially do the same thing, but due the availability of this FLAC at the University of Southern Queensland, it was selected. It is unknown whether the results obtained from FLAC are of the same accuracy level as other software packages, without making a comparison between them, which is outside the scope of this assignment. Thus FLAC will be used throughout this project, to model and analysis the shallow foundation located near a purely cohesive slope problem.

Introduction to FLAC Analysis and Advanced Modelling

3

3.1 Introduction

This chapter is an overview of the geotechnical numerical modelling program FLAC or Fast Lagrangian Analysis of Continua that was used throughout this dissertation to model and analysis the geotechnical problem of a shallow foundation located near a purely cohesive slope. Presented within this chapter is an explanation of the program, the major features of the program, thus the reasoning for its selection and an explanation of the use of the program to model the four advanced analysis models presented within this dissertation, along with example model inputs and outputs.

3.2 Fast Lagrangian Analysis of Continua

“FLAC is a two-dimensional explicit finite difference program for engineering mechanics computation” (Itasca, 2002) The program has the capability of modelling engineering structures on various geotechnical soil structure materials, such as soil, rock or similar material, to investigate the behavioural effects of plastic flow within the material after a yield limit has been reached. As FLAC is an explicit finite difference program, the problems being modelled are solved using a time stepping procedure rather than forming a stiffness matrix like finite element solutions.

There are a number of different versions of FLAC currently available, with the most current version being 6.0. However for the purpose of this dissertation the version that has been adopted is FLAC 4.0, due to its availability. Nathan Lyle (2009) established within his dissertation that the difference between the two versions was only marginal, with the major difference being the newer version, version 6.0, contained a number of different speed improvements. Thus the use of version 4.0 over version 6.0 would not compromise the accuracy of all advanced modelling solutions presented within this dissertation.

FLAC version 4.0 allows for the program to be command driven or GUI mode, thus providing alternative methodologies for different programming requirements and operator skill levels. In addition FLAC contains a robust built-in programming language called FISH, that is stored within a text file, that allows the program to be command driven, thus reducing the repetitive tasks that would be required within GUI mode. As the problem of the shallow foundation located near a slope contains a number of different parameter changes, the command driven mode and the storage of the FISH code within a text file, allows for easy editing of the code outside of the software program.

3.2.1 Major Features of FLAC

After reviewing the software program FLAC, it was established from the program creator, Itasca's program explanation, that the program had a number of major features that could be utilised within this project. These features listed by Itasca include;

1. Large-Strain simulation of continua, with the optional interface option that simulated distinct planes along which slip and/or separation can occur.
2. Explicit solution scheme, giving stable solutions to unstable physical processes.

3. Groundwater flow, with full coupling to mechanical calculations (including negative pore pressure, unsaturated flow and phreatic surface calculations).
4. Convenient specifications of general boundary conditions.
5. Library of material models (e.g. Mohr-Coulomb plasticity, ubiquitous joint, double-yield, strain-softening, modified Cam-Clay and Hoek-Brown).
6. Automatic re-meshing during the solution process in large strain simulations.
7. Pre-defined database of material properties; users may add and save their own material properties specifications to the database.
8. Statistical distribution of any property with extensive facility for generating plots of virtually any problem variable.

3.2.2 Reasoning for the Selection of FLAC

Due to the major features presented within section 3.2.1, its availability at the University of Southern Queensland and its previous use within past related dissertational studies, FLAC has been selected to model and analysis the advanced models that have been presented within this dissertation.

3.3 Producing Advanced Models within FLAC

Presented within this dissertation are the modelling and analysis of four advanced foundation characteristics within FLAC. These advanced foundation characteristics being modelled include; the soil structure interface, the discontinuous punching of the foundation, large strain analysis and a foundation subjected to static pseudo seismic forces. Each of the advanced models was based on the adaptations of a simplified code produced by previous studies conducted within this area of study.

Presented within Figure 3-1 is an example of an adapted base model of a FLAC script that has been used within the advanced analysis of the geotechnical problem. Presented below are the basic steps that have been undertaken within the FLAC model, to analysis the shallow foundation located near a slope problem;

1. The first step is to define various input variables for the model, this step can be seen within the example shown within Figure 3-1.
2. The second step is to specify the magnitude of the gravity and its angle of magnitude, for the model.
3. The third step is to define the properties of the soil structure mesh and the foundation structure mesh.
4. The fourth step is to set up the extents and boundaries of the model, by excavating the building mesh.
5. The fifth step is to apply the initial velocities at the base of the foundation to signify the presence of the foundation, and to investigate the effect it has on the soil structure.
6. Then the final step is to save the graphical and numerical output data that is produced during the solution phase of the FLAC model into a specified project folder.

```
; ------This file is for solving 2D bearing capacity of footing on slope ------
; --------Interface and concrete elements are included----------
;-----Soil conditions consist of purely cohesive clay soil----------------------
        new
        set echo on
        set log on
        config extra=10
        ;set cust1 'Footing on Slope'
        ;set cust2 'USQ Geomechanics Group'
        set legend on; "off" to switch of the jpg frame
        set overwrite on
        set replot on; "off" to add to the existing plot

; -----------Set up all the parameters of the model-----------
; -This is the place you can change ther problem geometry and material property-

def para_meter

;;;;;;;;;;;;CHANGE the following parameters to suit your need;;;;;;;;;
        pathname = 'F:\Rough_0.0\NoBuild\HB3SR5DB1' ; need to change for each case
        Beta = 90
        HonB = 3
        DonB = 1
        SR = 5
        FrictionAngle = 0
        DilationAngle = 0
        qrB=0;surcharge loading ratio q/rB
        Roughness = 0 ; Key in "0", "0.33", "0.5", "0.67" and "1"
;;;;;;;;;;;;CHANGE the above parameters to suit your need;;;;;;;;;

;;;;;;;;;;;;DO NOT CHANGE the following parameters unless you know exactly how/what.----
        Y_vel = 1e-5            ; Footing velocity
        stepping_no = 25000
        XElementSize = 0.1 ; element size;divide equally with B
        YElementSize = 0.1 ; for coarse- use "0.2"; fine - "0.1"
        largeorsmall = 'small'; Key in "large" or "small"
        building = 'allow_x'; put "allow_x' for movement in x, otherwise "not_allow_x"
        movieon = 'off';  put "off" for no movie
        B = 1 ; always keep B=1 for convenience
        H3=1; this is building height
        Mstepping_no = stepping_no/10
        MatDensity = 2000
        MatDensity1 = 0.1; density for building
        MatGravity = 9.81

        iface_nstiff=1e9; normal stiffness of iface
        iface_sstiff=1e9; shear stiffness of iface
;;;;;;;;;;;;DO NOT CHANGE the above parameters unless you know exactly how/what.----
```

Figure 3-1: Sample FISH Input Script

3.3.1 Typical FLAC Input Variables

The typical input variables that would be required to obtain a solution from the FLAC script, include;

- Y_Velocity (Footing Velocity)
- Number of Steppings (Iterations)
- X_Element size (Width of Element)
- Y_Element size (Height of Element)
- Strain (Small or Large)
- Footing Roughness (Smooth of Rough)

- Mesh Angle (Vertical or Inclined)
- Model Extents and Boundaries
- Footing Distance Ratio
- Height Distance Ratio
- Strength Ratio
- Angle of Slope

In order to accurately model and evaluate the geotechnical problem of the shallow foundation located near a slope problem, it is essential that all of these input variables are correctly evaluated and entered within the script.

3.3.2 Typical FLAC Output Variables

A number of graphical and textual outputs can be produced by the FLAC program and saved to a specified folder for future viewing and analysis, some of the typical output values include;

- Xvel.jpg (also in textual form)
- Yvel.jpg (also in textual form)
- Grid.jpg
- Vel_vector.jpg
- Dip_Vector.jpg
- Deform_shape.jpg
- Load.jpg (also in textual form)
- Normalised_load.jpg (also in textual form)
- Unbal.jpg (also in textual form)
- Central_Disp.jpg (also in textual form)

3.4 Data Extraction from Result Files

The extraction of useful data from the output solutions that have been listed within section 3.3.2, has been achieved through the adoption of a standard methodology that has been used throughout the result obtaining duration of the project, to ensure that all results were accurate and plausible.

The general procedure used to obtain these results from output files was;

1. Prepare a script file with all accurate input variables.
2. Create a list of the script files the script files that require analysis.
3. Run each of these script files within FLAC.
4. Physically analysis the results obtain to verify the plausibility of the results.
5. Export the numerical data within the text files produced by FLAC into an excel spread sheet and analysis and presented the results in the required result presentation format.

As this process is a simplified method it allows for physical verification of result accuracy as once the data is exported into excel and plotted, plot outliers or inaccuracies in results can be visually determined. Overall this methodology adopted has been successful within the result gathering stage of the project.

3.5 Chapter Summary

This chapter has been provided to present the dissertation reader a brief introduction to the FLAC analysis program and its main features, thus the rational for its selection as the main software program for the analysis of the shallow foundation located near a slope problem. Also presented within this chapter was an introduction to the four advanced models that will be presented in more detail throughout consecutive chapters of this dissertation, a general example of a basic script, typical inputs that are required by the FLAC program to obtain results for the foundation problem and the typical outputs that would be achieved from FLAC modelling, along with a methodology adopted throughout this project to analysis these output results.

The Soil Structure Interface

4

4.1 Introduction

This chapter presents the first advanced model presented within this dissertation, the soil structure interface model for a rigid shallow foundation located near a 90^{o} slope consisting of homogenous clay soils. Within this model actual incorporation of a foundation within the FLAC mesh is produced, thus investigating the effects of frictional forces between the rigid foundation base and the underlying soil structure. This study is an advancement of the models produced by past dissertational works, as these models were simplified and only investigated a smooth model with applied velocities at a proposed foundation location, whereas this model represents the foundation as an actual element. For the purpose of this study two interface types will be investigated; a smooth interface and a rough interface and the effect of building loading will also be investigated. It is important to investigate the effect that interface conditions and building loadings have on the failure mechanism of foundations located near slopes and resulting effect on the ultimate bearing capacity of these foundations. It is important to keep in mind that the results produced within this chapter are qualitative rather than quantitative, as the model produced is aimed to be a more realistic representation of actual foundations located near slopes.

To insure the advanced model presented within this chapter is producing qualitative results it will be validated against previous published FLAC models for this shallow foundation problem. After validation of this advanced model, it will be used within a validation process of design charts and tables for ultimate bearing capacity, that were produced within past quantitative dissertational studies conducted by Nathan Lyle (2009).

The parameters that are relevant to this chapter include:

- c/γB soil strength ratio.
- H/B slope height ratio.
- D/B footing distance ratio.
- p/γB normalised bearing capacity.

The statement of the problem including the horizontal interface location is shown in Figure 3-2. The two interface types included for this problem are; smooth (c_a=0) and Rough (c_a =c). For the purpose of this study presented within this chapter only small strain analysis will be considered.

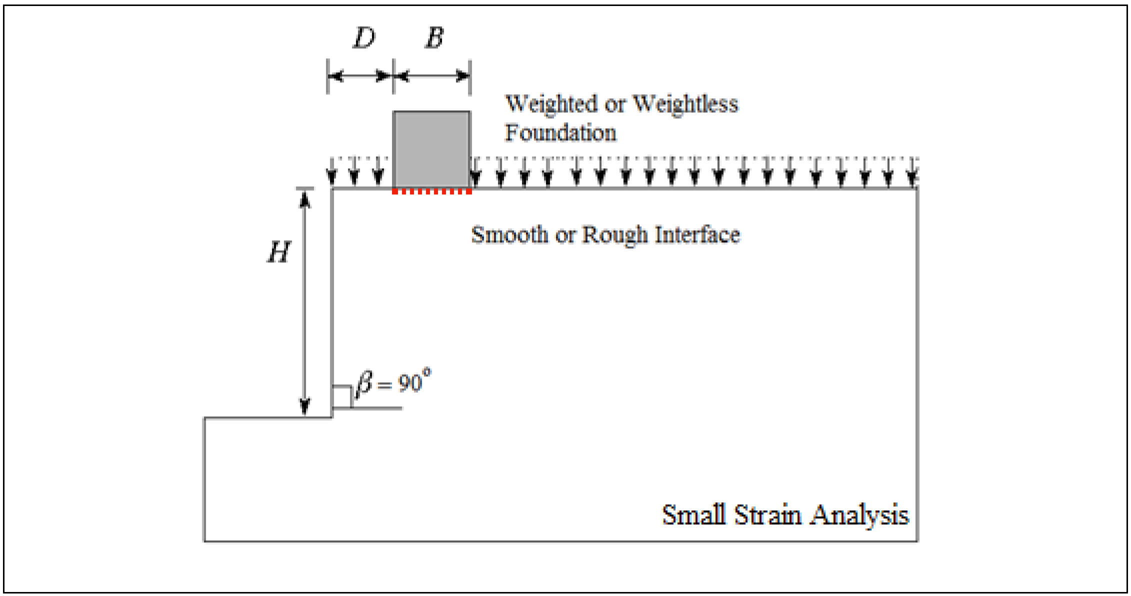

Figure 4-1 Chapter Problem Description (Including Interface)

4.2 The Model Development

This chapter focuses on developing a fully validated advanced model for the investigation of the effect of building loading and the interface condition between the base of the rigid foundation and soil structure. The first step in developing this model was to include the presence of foundation within the mesh and creating the horizontal interface between the foundation and soil structure. This required there to be two material types and a frictional interface between them. As previously mentioned both smooth and rough interface conditions were studied within this chapter thus the next step was to create the different two interface conditions within the FLAC model. This was done within FLAC through the application of cohesion within the interface boundary and through permitting slippage within the boundary. For smooth interface conditions the level of cohesion within the interface condition was equated to zero, while for rough interface conditions the level of cohesion was equated to the level of cohesion within the soil structure, which equalled soil strength ratio multiplied by the soil unit weight and foundation width. Slippage within the interface boundary was permitted within both smooth and rough interface conditions to investigate the failure of the foundation. Foundation weight was altered within the model through the application of different material densities for the foundation mesh.

4.3 The Model Validation

The validation of this model was an important step within this chapter as it ensured the quality of the results being obtained. Due to this model eventually being used as a validation tool for past dissertational work presented by Nathan Lyle (2009) it was important to ensure that the weightless advanced model presented within this chapter produced results that were similar to those of the simplified 'imaginary' foundation problem presented by Lyle, before a weighted foundation condition could be considered. Thus for validation purposes a building material density of 0.1kg/m^3 was applied within the model to signify weightless conditions. The validation of the model was conducted for smooth and rough interface conditions to eliminate any uncertainties within later studies presented within this chapter.

Table 4-1 presents the validation between the imaginary foundation model and the weightless foundation model. The results presented within this table are ultimate bearing capacities for smooth and rough interface conditions between the rigid foundation base and soil structure interface, for a range of different D/B ratios. Also presented within this table are the percentage differences between the two modelling methods.

The obvious trend within these results is the reduced ultimate bearing capacity that occurs when the foundation element within FLAC is modelled. This occurrence is the case for both the smooth and rough interface conditions, thus it can be concluded that the slippage that is allowed within the interface boundary between the foundation elements and the soil structure elements within FLAC is causing failure to occur at lower capacities than the imaginary foundation model. Also observed was the compatibility between the two models with respect to increased capacities when rough interface conditions between the foundation base and soil structure are considered. Thus it can be concluded that the rough interface modelling off the weighted foundation is producing reasonable results. Finally it was observed that the percentage difference between the two foundation modelling methods was greatest at D/B ratios between 0 and 4, for both smooth and rough interface conditions. This observation would be the result of the allowable slippage within the weightless foundation interface boundary, causing instability. As the foundation transitions from local shear failure (unsymmetrical) to general shear failure (symmetrical), at D/B ratios greater than 4, the slip surface reaches ground level, thus achieving flat ground failure mechanisms. It is evident at flat ground failure that minimal to zero difference between modelling methods of the foundation, whether it an element foundation or applied velocities at a proposed foundation location, is occurring.

Therefore from these validation results it can be concluded, with reasonable certainty, that the advanced soil structure interface model will produce results that are of reasonable qualitative standard and thus can be used within validation of previous simplified numerical models of the shallow rigid foundation resting near a slope problem.

Table 4-1. Comparison of Ultimate Bearing Capacity between the Imaginary Foundation Model and the Weightless Foundation Model for Range of D/B Ratios.

D/B	"Imaginary" Foundation		Weightless Foundation		Percentage Difference Smooth (%)	Percentage Difference Rough (%)
	Smooth	Rough	Smooth	Rough		
0	10.94	13.94	10.36	10.51	5.30	24.61
1	18.13	20.07	16.66	16.78	8.11	16.39
2	22.30	23.69	20.39	20.49	8.57	13.51
3	25.41	26.27	23.59	23.68	7.16	9.86
4	27.60	28.18	26.33	26.44	4.60	6.17
5	27.60	28.47	27.60	28.17	0	1.05
6	27.60	28.47	27.60	28.17	0	1.05

From dissertational work completed by Lyle (2009), it was determined that the most appropriate value for an applied velocity within the model was $1e^{-5}$ meters/iterations, as it was determined that applied velocities above this value greatly increased the computer processing unit time, but only minimally affected the accuracy. Lyle (2009) came to this conclusion through a variety of trial runs with a range of different applied velocities, to ensure that the FLAC model produced real life situations. Thus the applied velocity adopted within this chapter of study, based on Lyle's (2009) conclusions, was set at $1e^{-5}$ meters/iterations.

4.4 Investigation of Building Weight

The investigation of the building weight involved applying an increased material density to the foundation mesh, within FLAC. For the purpose of this study the density selected was 2000kg/m^3 as this value would produce realistic representations of actual foundation weights. Within the investigation of the effect of building weight on the soil structure interface, smooth and rough interface conditions have been considered. Comparative results for weighted and weightless foundations have been included below within respect to the interface condition. At the end of this section a comparison has been presented between the smooth and rough interface bearing capacity results to evaluate the effect of increasing the frictional forces within the interface boundary.

4.4.1 Smooth Soil Structure Interface

The smooth soil structure interface condition is achieved by equating the level of cohesion within the interface boundary between the rigid foundation base and soil structure to zero and allowing slippage to occur. Figure 4-2 present the results for change in normalised bearing capacity with D/B ratio, for a weighted and weightless foundation. The main observation made from this figure is the reduction in the bearing capacity for the weighted foundation. This occurrence would be the resultant effect of the extra force that is produced with foundation weight, thus the weight of the foundation is increasing the momentum of slippage thus reducing the bearing capacity. Thus it can be concluded that within modelling the weight of the foundation with respect to soil structure interface the results produced for the ultimate bearing capacity are more conservative. Therefore a weighted foundation should be used within the validation of the Lyle's simplified imaginary foundation for smooth interface conditions.

A secondary observation was the increase of normalised bearing capacity with increased D/B ratio and at a D/B ratio equal to 5, both the weighted foundation model and weightless foundation model reach equilibrium suggesting that the transition from local shear failure to general shear failure is complete. Figure 4-3 depicts this failure mechanism transition through the use of shear strain rate plots.

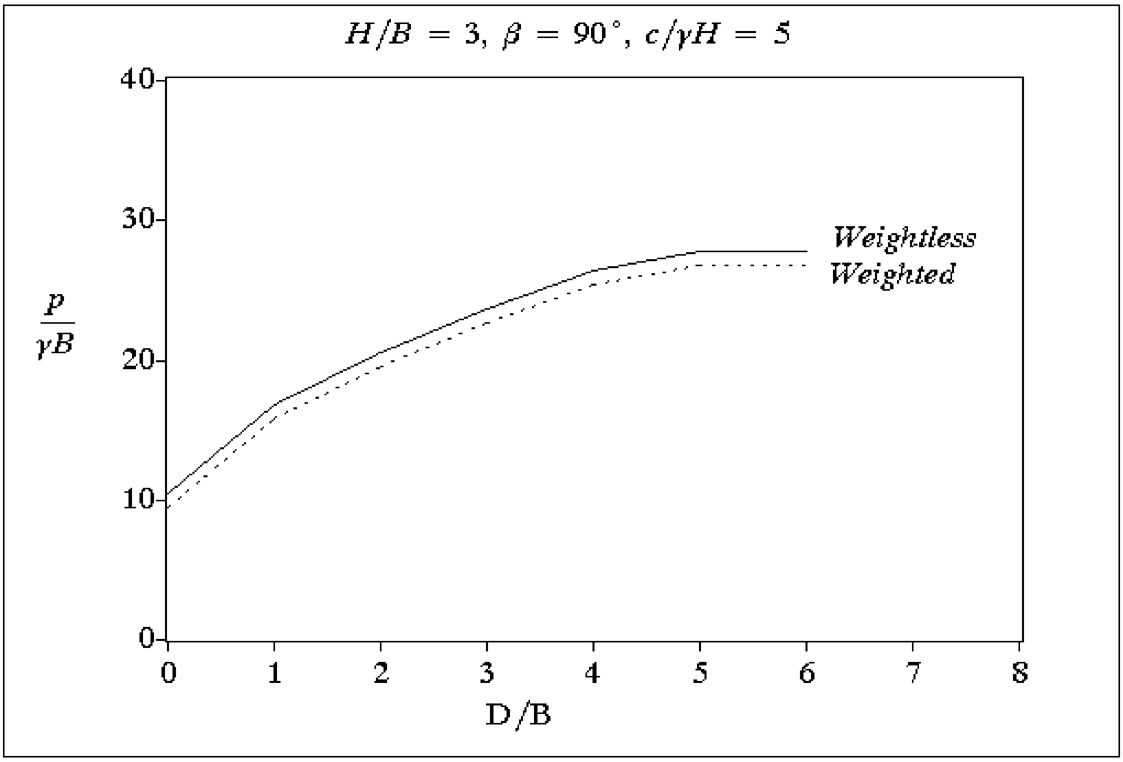

Figure 4-2. Comparison of Normalised Bearing Capacity with Footing Distance Ratio.

The results shown in Figure 4-3 show the change in bearing capacity and failure mechanism with changing D/B ratio for a weighted foundation with H/B ratio of 3, and a soil strength ratio of 5. It can be seen that for these conditions the bearing capacity is increasing with D/B ratio, until a D/B ratio of 3. The shear strain rate for a D/B ratio of 4 appears to be symmetrical indicating that the transition from local shear failure to general shear failure is almost complete, but it isn't until a D/B ratio of 5 that the foundation has reached full general shear failure as after this point the ultimate bearing capacity and shear strain rate figures stay constant. This change in behaviour indicates that flat ground failure is occurring thus the slip surface has reached ground surface and heaving of soil is occurring.

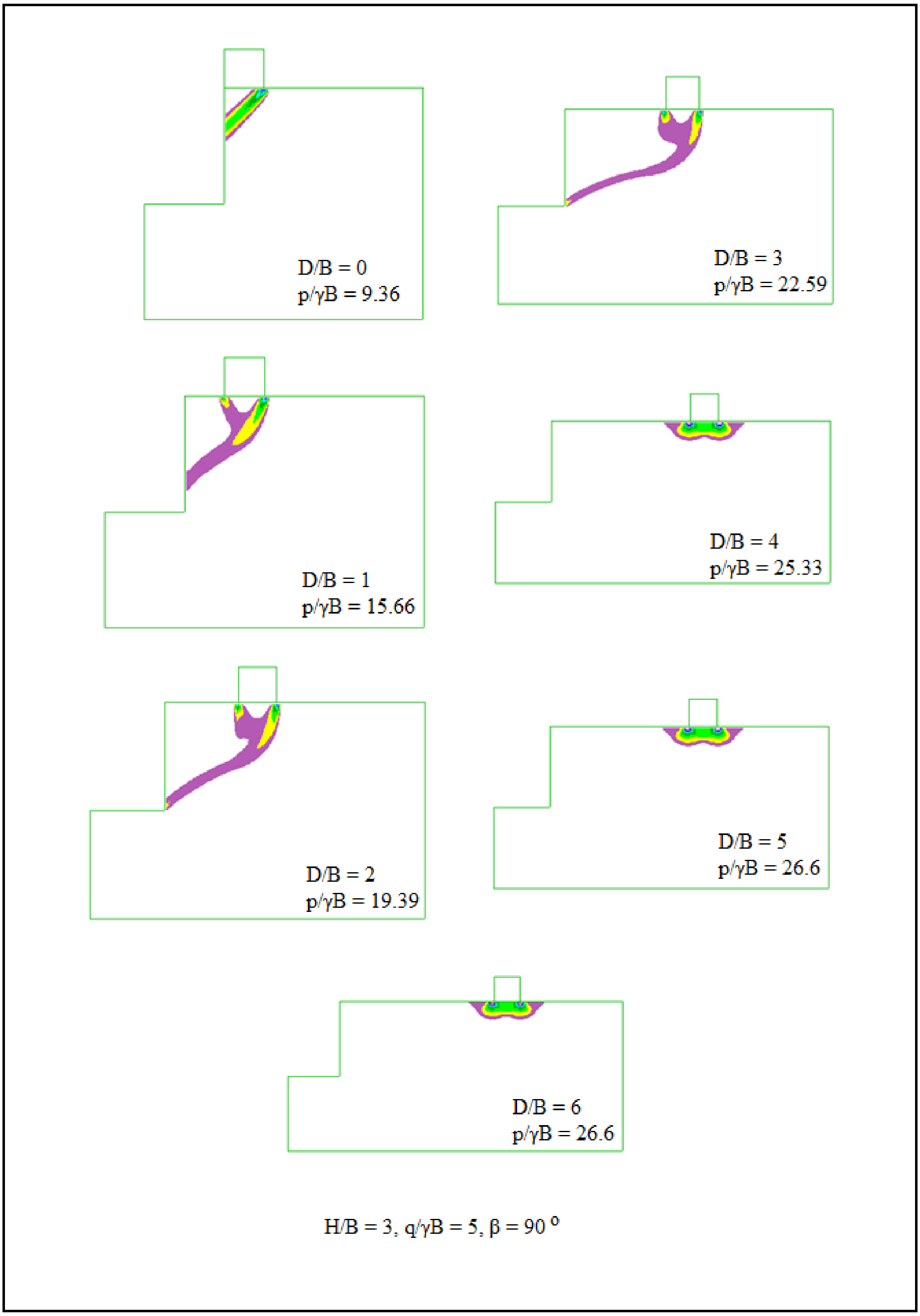

Figure 4-3. Change in normalised bearing capacity with D/B ratio.

Therefore from the weighted results for a smooth soil structure interface model it can be concluded that the inclusion of the foundation weight will reduce the foundation ultimate bearing capacity. Thus the weighted foundation should be used in the validation of Lyle's simplified model for smooth interface conditions. As for the failure mechanism results for the weighted foundation under smooth interface conditions, it isn't until a D/B ratio of 5 that general shear failure occurs.

4.4.2 Rough Soil Structure Interface

The rough soil structure interface condition is achieved by equating the level of cohesion within the interface boundary between the rigid foundation base and soil structure to level of cohesion within the soil structure mesh and allowing slippage to occur. Figure 4-4 present the results for change in normalised bearing capacity with D/B ratio, for a weighted and weightless foundation, under rough interface conditions. It again can be concluded from this graph that the weighted foundation for rough interface conditions again produces reduced normalised bearing capacities when compared to the weightless foundation. Again this occurrence can be the result of additional force provided by the weight of the foundation, providing the slippage motion with more momentum. Therefore it can be concluded that when validating the simplified imaginary foundation problem, proposed by Lyle, for both smooth and rough soil structure interface conditions, a weighted foundation should be used as this modally method yields the more conservative capacity values.

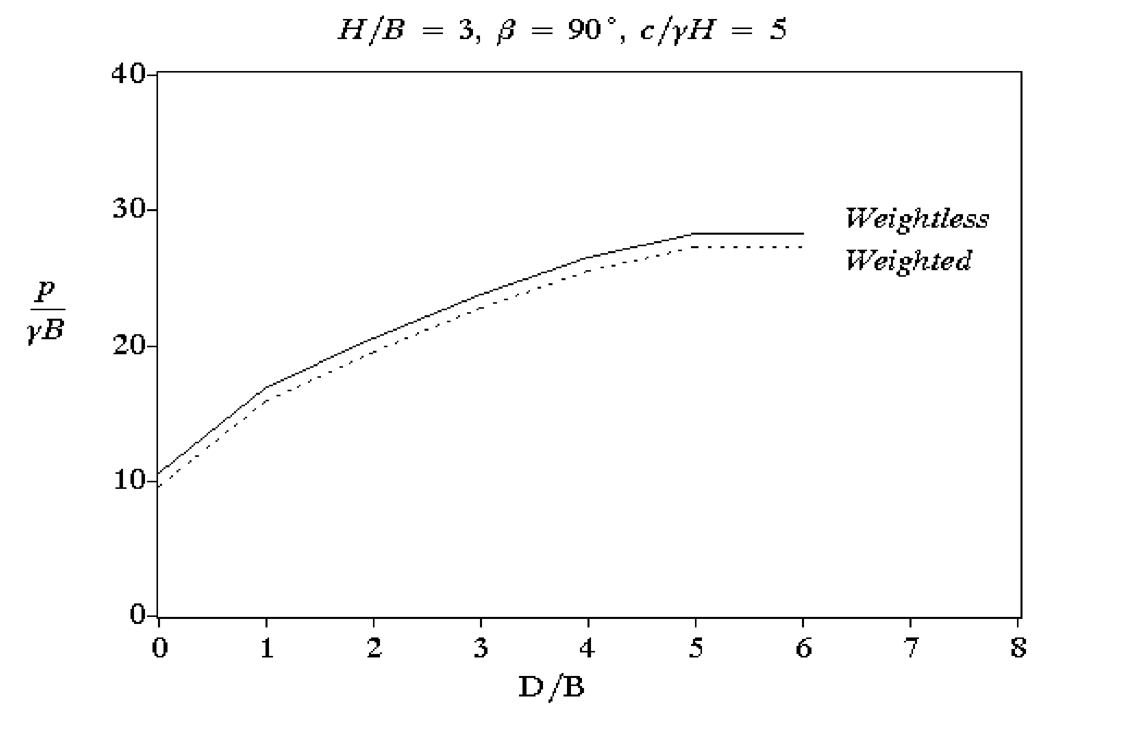

Figure 4-4. The change in normalised bearing capacity with D/B ratio for a rough soil structure interface.

The results shown in Figure 4-5 show the change in bearing capacity and failure mechanism with changing D/B ratio for a weighted foundation with H/B ratio of 3, and a soil strength ratio of 5, under rough soil structure interface conditions. It can be seen that for these conditions the bearing capacity is increasing with D/B ratio, until a D/B ratio of 4, after this D/B ratio the bearing capacity and shear strain rate figures stay constant. Therefore for a weighted foundation model with a rough soil structure interface general shear failure (flat ground failure) occurs at a D/B ratio of 5. This change in behaviour indicates that flat ground failure has occurred after a D/B ratio of 5 and the slipe surface has reached the ground surface resulting in soil heaving at either side of the foundation.

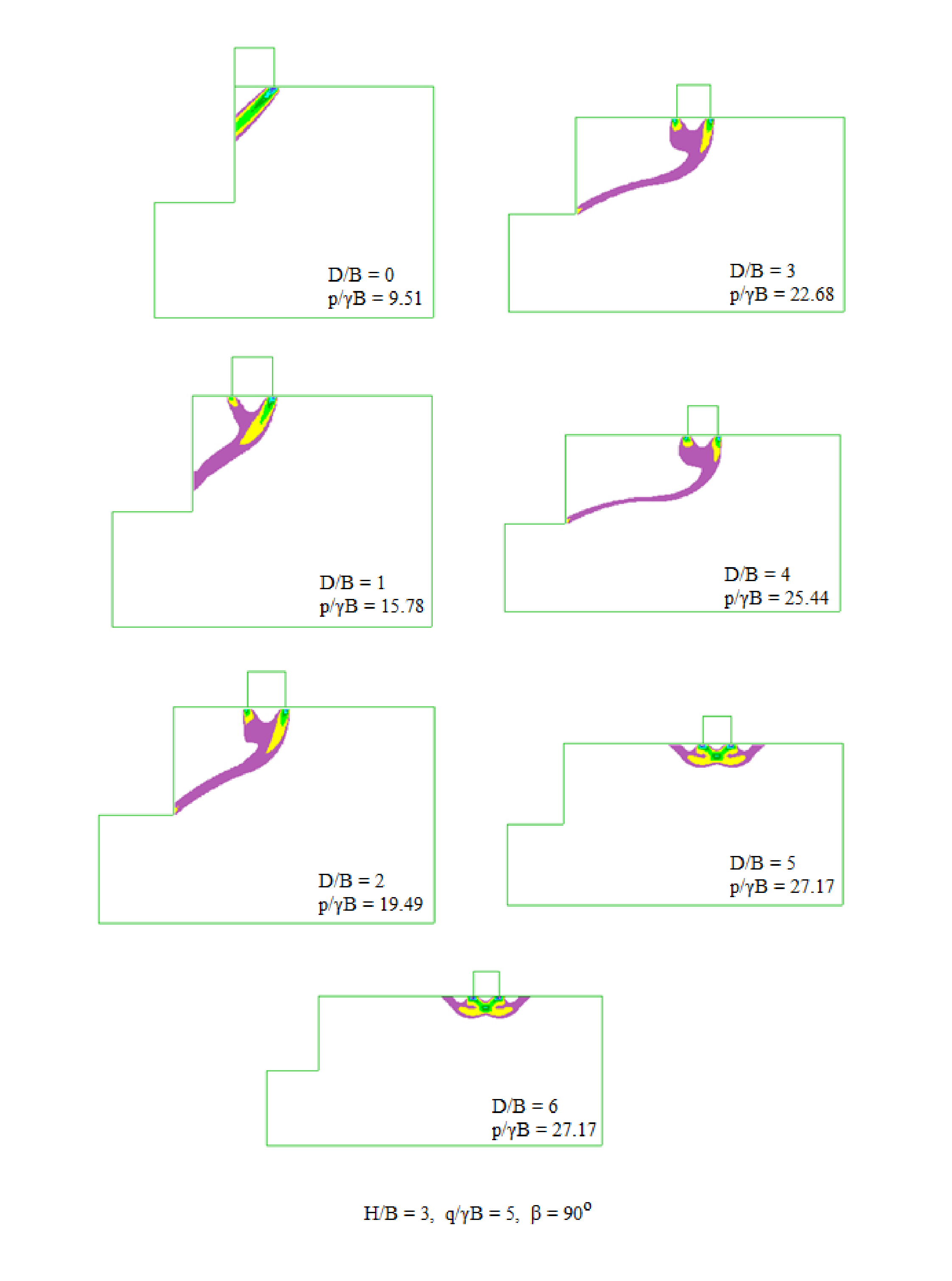

Figure 4-5. Change in normalised bearing capacity with D/B ratio.

4.4.3 Comparison of Interface Conditions

To fully evaluate the weighted foundation condition and to determine the more conservative advanced modelling method, to be used within the validation of Lyle's simplified foundation model, it is important to compare and analysis the difference in ultimate bearing capacities and failure mechanisms between smooth and rough soil structure interfaces. For the purpose of this investigation Figure 5-6 was prepared. Presented within Figure 5-6 is the change in normalised bearing capacity with D/B ratio for weighted foundations under smooth and rough interface conditions, with H/B ratios of 3 and soil strength ratios of 5. The most obvious trend within the graph is the minimal difference between the different interface conditions at D/B ratios between 0 and 4. It isn't until a D/B ratio of 5 that the difference between the two models is noticeable, with the rough interface producing the larger capacities.

Overall these results are interesting but if the shear strain plots for the smooth and rough interface conditions presented within Figures 4-3 and 4-5, respectively are compared, it can be seen that the smooth interface condition achieves slip failure at the ground surface at a D/B ratio of 4, while the rough interface condition does not produce this failure effect until a D/B ratio of 5. Therefore due to the additional frictional forces that are present within the rough interface conditions the capacity is increased thus taking a longer time to reach equilibrium that occurs with general shear failure.

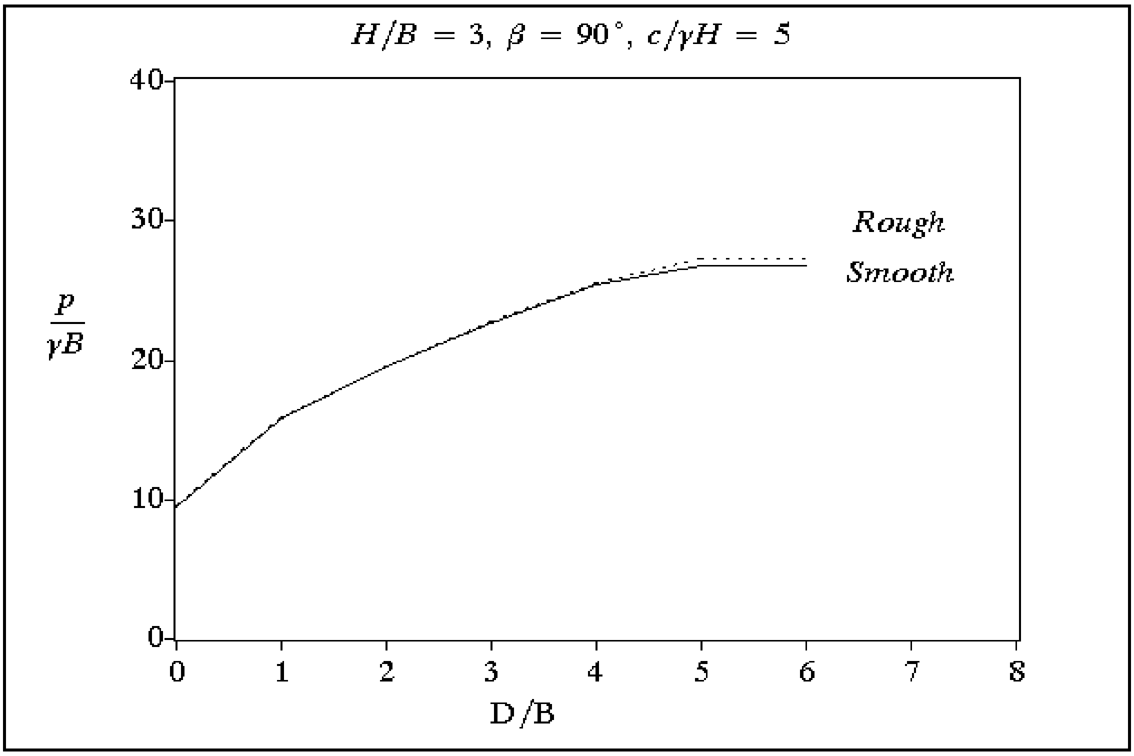

Figure 4-6. The comparison of normalised bearing capacity with D/B ratio for smooth and rough soil structure interfaces.

Therefore from the comparison of the smooth and rough soil structure interfaces for a weighted foundation it was concluded that the smooth soil structure interface model produced slip failure at ground surface at a D/B ratio of 4, whereas the rough soil structure interface required a D/B ratio of 5 to induce soil heaving. Therefore the smooth interface condition produced a reduced capacity when compared to the rough interface, thus it can be concluded that the smooth interface with respect to the advanced modelling of the soil structure interface is the more conservative modelling method. Therefore the smooth weighted foundation will be used within the final validation of Lyle's simplified model when the advanced modelling of the soil structure interface is considered.

4.5 Validation of the Simplified Model

Figure 4-7 presents the change in normalised bearing capacity with D/B ratio for the comparison of Lyle's simplified smooth interface imaginary foundation model and the advanced smooth soil structure model with a weighted foundation. It can be concluded from this figure that although the simplified imaginary foundation is reaching general shear failure at a smaller D/B ratio, the weighted foundation is producing bearing capacities less than the smooth imaginary foundation model, thus it can be concluded that the smooth weighted foundation is more conservative than the smooth imaginary foundation model proposed by Lyle, with respect to advanced modelling of the soil structure interface. From this conclusion it can be said that previous design charts produced by Lyle should be revised.

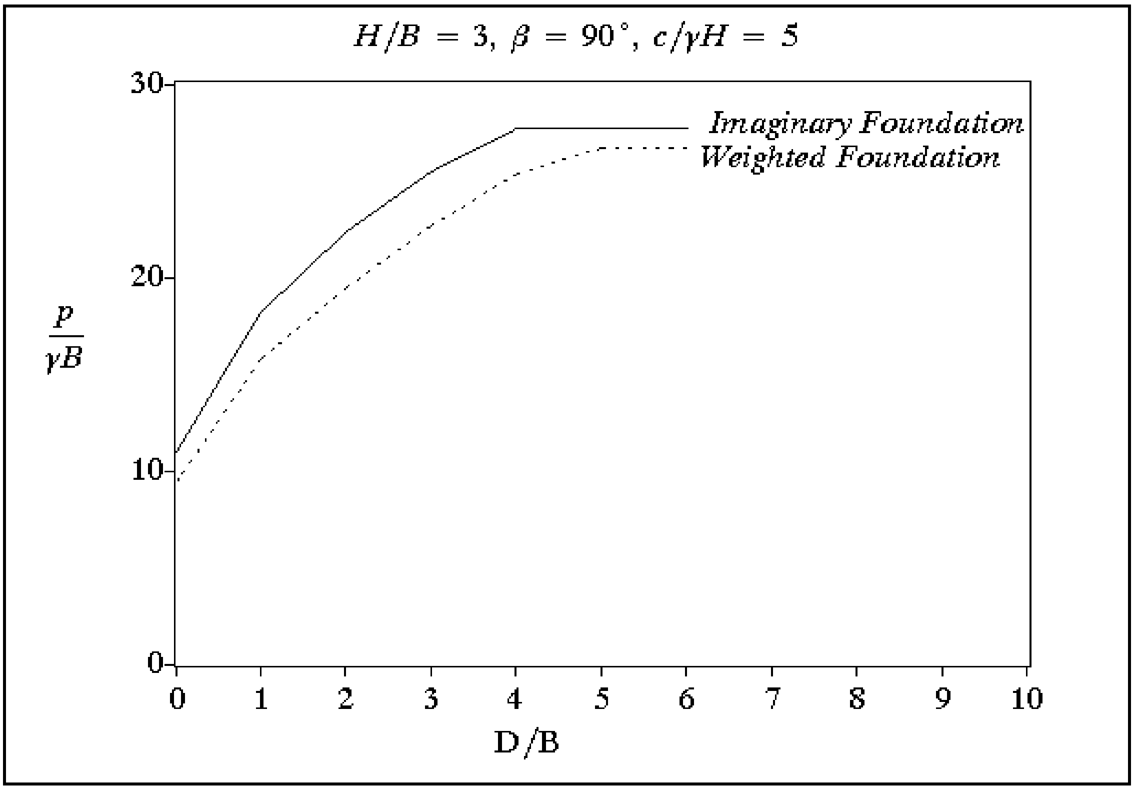

Figure 4-7. Change in normalised bearing capacity with D/B ratio for the imaginary foundation and the weighted foundation.

4.6 Conclusion

From the study presented within this chapter a number of conclusions were drawn from the results produced. The first conclusion drawn was that the weighted foundation produced more conservative ultimate bearing capacities than the weightless foundation model. It was concluded that this result was due to the additional momentum that comes with weight, during a slippage motion. Thus the weighted foundation was used to analysis the frictional forces that occur within the interface boundary when the interface was varied from smooth to rough.

The second finding was that the smooth interface between the soil structure and the weighted foundation produced bearing capacities less than the rough interface condition. This was concluded to be due to the increased friction between the rough foundation and the soil structure increasing the strength of the foundation by resisting the slippage motion. Thus from this finding it was concluded that a smooth interface between the soil structure and the weighted foundation would be the final model adopted, with respect to the advance modelling of the soil structure interface, to validate the simplified model produced by past studies of Lyle, to determine whether or not the design charts produced within this past study are conservative.

The third and final conclusion made within this chapter was that the smooth soil structure interface model with weighted foundations produced smaller ultimate bearing capacity results than Lyle’s simplified numerical FLAC model. Therefore the main finding from this chapter was that the model used within Lyle’s studies to produce conservative design charts for obtaining bearing capacities should be revised as the modelling of a smooth soil structure interface for a weighted foundation produced more conservative bearing capacities.

4.7 Future Work

When analysing the interface effects between a foundation and the foundation material, it is incomplete to only model just a horizontal interface, because according to physical modelling there is also two additional interface that act vertically downwards from the corners of the foundation, into the foundation material. Thus future work that can be conducted within this advance modelling of the interface effects is the addition of these two vertical interfaces into the soil structure, also termed discontinuous foundation punching modelling. Within chapter five of this dissertation, considerations have been made within an additional advance model for these two discontinuous foundation punching interfaces.

Discontinuous Foundation Punching

5

5.1 Introduction

This chapter is a development of the advanced model presented within chapter four, of this dissertation. In addition to just modelling the horizontal interface between the foundation and the soil structure, this chapter models and analyses the two vertical interfaces that occur between the foundation corners and the soil structure when the load of the foundation punches into the clay soil, foundation material. The addition of this discontinuous punching modelling was essential within this advanced modelling, as this is a real life characteristic of foundations, and is termed punching shear failure when failure is induced. Thus the results produced within this chapter will theoretically be more accurate representations of actual final bearing capacities for foundations located near slopes. Thus the results achieved within this chapter will be used in an analysis and comparison process for the proposed conservative results achieved by Lyle's (2009) dissertation. It is the aspiration that the results obtained within this chapter of advanced modelling will be less conservative than the results obtained from Lyle's (2009) dissertation, thus will ultimately yield a greater ultimate bearing capacity, at failure.

The content covered within this chapter will include; a validation of the advanced model to ensure quality of output results, once validated the model will be used to analysis a range of different foundation characteristic that will then be used to confirm the proposed conservative status of the results produced by the dissertational work of Lyle. The results presented within this chapter will be both numerical and visual results to provide the reader ease of understanding the complex behaviours and failure mechanisms that occur within advanced numerical modelling of the discontinuous foundation punching.

All of the modelling and analysis presented within this chapter has been conducted for small strain analysis and due to time constraints only a weightless foundation has been considered, under both smooth and rough conditions.

The parameters which are relevant to this chapter include;

- $c/\gamma B$ Soil strength ratio.
- D/B Footing distance ratio.
- $p/\gamma B$ Normalised bearing capacity.

The values of the other essential parameters used throughout this study have also been presented below:

- H/B = 3;
- D/B = 0, 1, 2, 3, 4
- $c/\gamma B = 5$
- B = 1 m
- $\phi = 0^{o}$ (this dissertation only considers clay foundation material)
- γ = 1.962kN/m^3
- $\beta = 90^{o}$
- q = 0kN/m

The problem statement that has been investigated within this chapter has been included within Figure 5-1. The interface type for this problem is either Rough (c_a =

c) or Smooth (c_a=0) and the foundation that has been modelled will have a weightless foundation density (foundation density = 0.1kg/m^3).

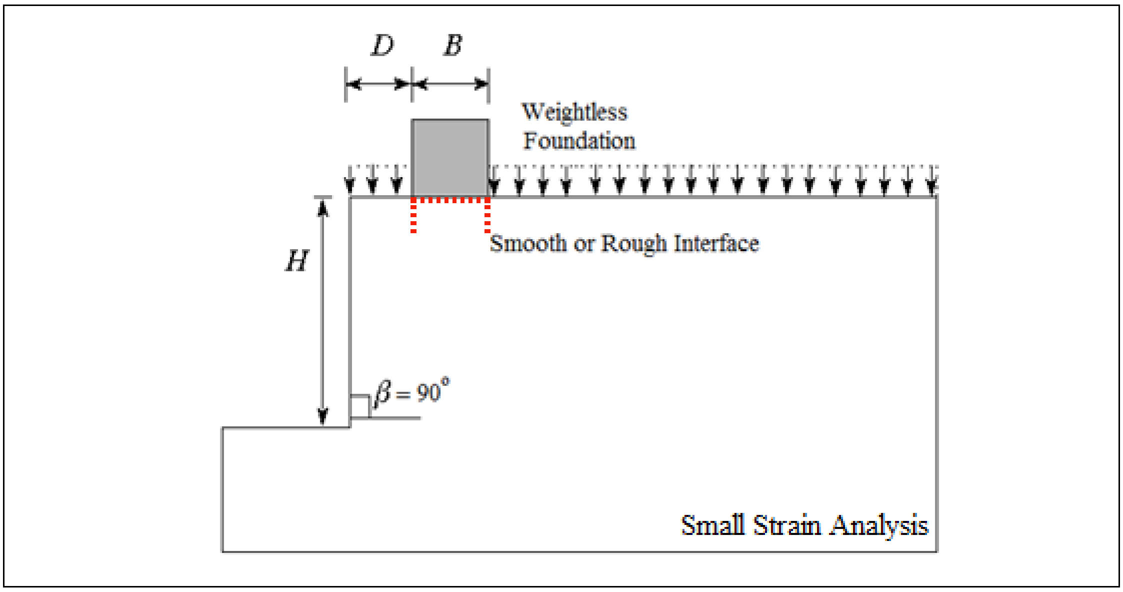

Figure 5-1. Problem notation for discontinuous foundation punching

5.2 The Model

The advanced model presented within this chapter is a numerical model that aims to investigate the effect of the interface between the clay foundation material, the base of the foundation and the edges of the foundation. To ensure that the numerical FLAC model produces results that resembled physical behaviours of foundations located near slope, the models code was constructed to include realistic physical properties of a foundation located near a slope. From previous physical modelling of the foundation problem, conducted by Shiau et al. (2006) it was determined that when a foundation material is under a continual loading from a foundation, a secondary interface forms, as a result of the foundation punching into the soft clay. Thus when modelling the full interface effects of a shallow foundation located near a slope it is essential to model the horizontal interface between the foundation and the soil structure as well as the vertical interface that forms between the edges of the foundation and the soil.

Within theory a foundation problem like the problem being physically modelled within Figure 5-2, should depict some clay soil rotation, but as it can be seen from Figure 5-2, this is not the case. Watson (2008) concluded that the physical results obtained from studies conducted by Shiau et al, indicated that either the actual rotation of the footing about the base is minimal regardless of the interface properties or the vertical interface acts as a brace against rotation. Therefore this would suggest that the rotation of clay soils subjected to a significant punching failure load, like the advanced model presented within this chapter, will be minimal, thus will have minimal effect to the advanced numerical model.

Therefore in addition to the basic model discussed within chapter three of this dissertation, the advanced discontinuous foundation punching model, presented within this chapter, consists of two major components; the modelling of the horizontal interface between the soil structure and he base of the rigid foundation and the vertical interface between the edges of the rigid foundation and the soil. From this advanced model an investigation of the effect of interface type and loading was conducted.

5.2.1 Development of the Horizontal Interface

The first step within this advanced model was to incorporate the horizontal interface between the soil structure and the foundation base. This modelling was conducted within chapter four and was just adapted for use within the advanced model presented within this chapter. The basic concept of the horizontal interface was to firstly establish two different materials; the foundation and the soil structure, and set each different material with their respected properties. This can be modelled within FLAC by removing the mesh coordinates that were common to the two materials and then rejoining the materials, thus achieving different coordinate points for the different materials, thus creating an interface boundary and allowing differentiation of material properties.

The next step was to establish a level of friction between the two materials, for the purpose of this study only two friction cases have been taken into consideration; an extremely smooth interface and an extremely rough interface. Within FLAC these interface friction levels were modelled through the allocation of interface cohesion in stress units, thus the completely smooth and completely rough interface levels of cohesion were set at 0KPa and 117.12KPa, respectively, to signify a theoretical smooth interface cases and rough interfaced material such as concrete.

5.2.2 Development of the Vertical Interface

The second step within producing this advanced model was to model the vertical interface that occurs between the edges of the foundation and the soil, as a result of the foundation punching into the soft clay material. The basis of FLAC is the continuous analysis of continua, meaning the program is constructed to analysis continuous media and does not allow for discontinuities. This produces difficulties when attempting to model the discontinuous punching effect on the soil structure, as the separation of the mesh that is required by the soil punching mechanism is difficult to model without errors within the software, because this process was not the intention of FLAC. Previous interface studies have used a number of methods to try and rectify these software issues, the method adopted within this dissertation involved separating the mesh the entire length of the soil structure and then rejoining it back together and applying a vertical interface at the edges of the building to a certain depth below the soil structure interface. This separating and rejoining process allows for individualised material coordinates at the proposed punching interface locations.

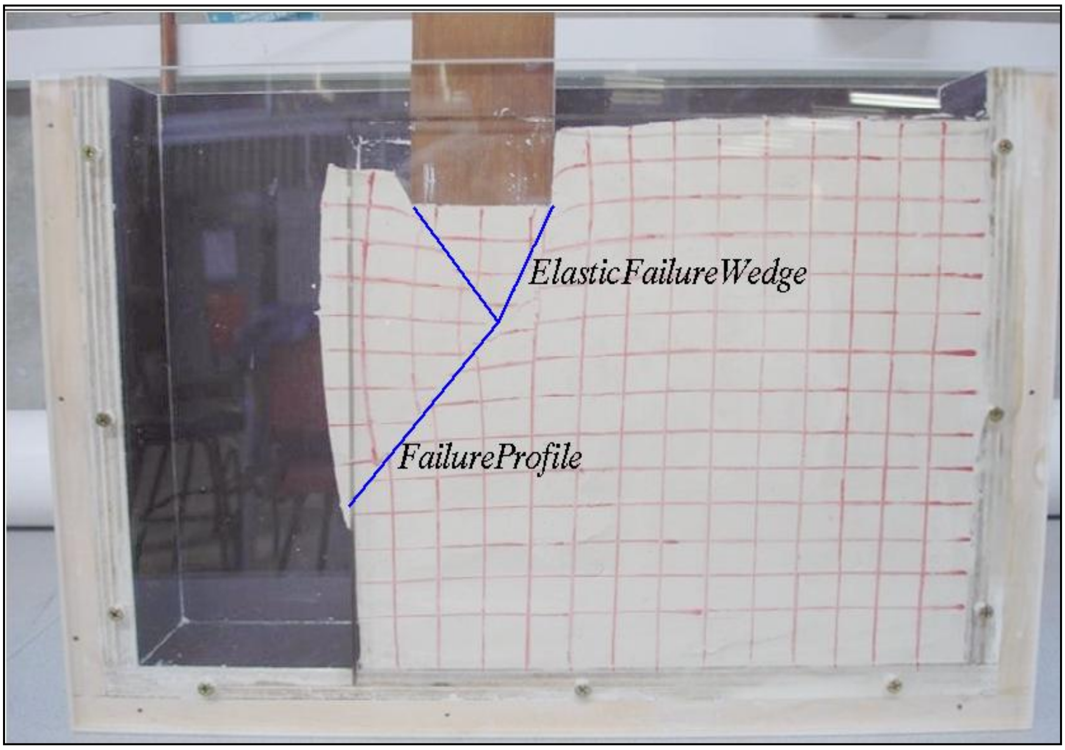

Figure 5-2. Physical modelling results produced by studies conducted by Shiau et al. (2006)

For the purpose of this study two different interface lengths were investigated and two different interface conditions were investigated. The two interface lengths investigated within this chapter were; 0.5 meters (10 elements) and 1 meter (20 elements) below the soil structure interface level, these values were selected on the basis of pure investigation. The two interface conditions that were investigated within this chapter include; a completely smooth interface and a completely rough interface. The investigation of these two extreme interface conditions was conducted to evaluate the effect that real life interface conditions have on the value of the ultimate bearing capacity for the foundation and the induced failure mechanism. Thus the rough interface was included to symbolise a real life foundation interface condition and the smooth interface was included to symbolise a theoretically conservative foundation. The methodology of modelling the full foundation-soil interface within FLAC involved; defining interface boundaries within the mesh, applying a level of cohesion for the horizontal interface between the soil structure and the foundation base, applying a tensile strength within the vertical boundaries and allowing slippage at the bonded vertical interface level. To induce a smooth interface and rough interface conditions within the model, the cohesion level for the soil structure interface was set to; zero and equated to the tensile strength within the vertical interface boundaries, respectively.

5.3 Model Validation

The validation of the advanced model presented within this chapter was conducted at two different levels, through visual inspection of the results obtained from the numerical modelling within FLAC and the physical results obtained from studies conducted by Shiau et al. and through the comparison with the imaginary foundation model results obtained from studies conducted by Lyle.

Presented within Figure 5-3 are some preliminary graphical results of the mesh deformation output from the numerical modelling of the discontinuous punching of the foundation into the soft clay material, from FLAC. It can be seen from this figure that the mesh deformation results obtained from the FLAC model illustrate similar behaviours with respect to failure profile, the elastic wedge failure that occurred directly under the foundation and the separation within the mesh at the edge of the foundation on the slope side. From this visual validation it can be concluded that the advanced model is producing physical behaviour results that resemble those of the physical results obtained from studies conducted by Shiau et al.

The validation of the model with respect to the results obtained by Lyle's imaginary foundation model have been presented within sections 5.4.1 and 5.4.2 for the smooth interface model and the rough interface models, respectively.

Figure 5-3. Example Output of a Smooth Interface for Visual Validation Purposes

The magnitude of the initial velocity that was applied at the base of the foundation to signify the real life foundation loading on the underlying clay soil structure was set at $1x10^{-5}$ units/iterations. This velocity was determined from previous studies conducted by Lyle, in accordance with the accuracy of the ultimate bearing capacity achieved by the model, the number of iterations and the computer processing time required to achieve these results and the overall stability of the solution.

5.4 Interface Analysis

The interface of the shallow foundation located near a purely cohesive clay slope, was analysed for two different foundations conditions, a smooth interface and a rough interface, where the smooth interface was expected to produce lower ultimate bearing capacities than the rough interface case, thus producing a more conservative model. Presented within this section, is an analysis of the two interface types, along with a range of qualitative results obtained from modelling within FLAC and finally a comparison of the two interface types and the previously obtained results for the smooth imaginary foundation model.

5.4.1 Smooth Interface

The smooth interface model was simply the modelling of the foundation problem when zero frictional forces are present between the two different mesh types; the soil structure and the foundation base, within the FLAC model. There were two primary purposes for modelling a smooth interface situation, firstly as a validation tool to ensure that the results being obtained by this, discontinuous foundation punching model, were of a reasonable accuracy and secondly to provide an evaluation tool, for the rough interface, to ensure that the assumption of the rough interface being less conservative, is correct.

To ensure accurate results were being obtained from this advanced model the ultimate bearing capacity produced by the FLAC model were compared with the results obtained from previous studies conducted by Lyle for the proposed conservative, smooth, imaginary foundation located near a slope model. Presented within Table 5-1 are the results obtained for the ultimate bearing capacity, from each FLAC model, for a range of different footing distance ratios.

Table 5-1. Comparison of the Ultimate Bearing Capacity for a smooth interface.

D/B	"Imaginary" Foundation *(Model produced by Lyle(2009))*	"Weightless" Foundation *(Advanced model produced by this dissertation)*	Percentage Difference (%)
0	10.94	11.14	-1.80
1	18.13	16.79	7.39
2	22.3	21.32	4.39
3	25.41	24.35	4.17
4	27.6	24.35	11.78

From these results, excluding the results for the footing distance ratio of zero, it can be seen that the ultimate bearing capacities produced by the "weightless" foundation, yields a lower capacity, than Lyle's "imaginary" foundation model. This occurrence is a common result with the previous advanced modelling of the soil structure interface, presented within chapter four. It was concluded within that chapter and again within this chapter that the inclusion of the interface boundary within the weightless foundation has reduced the capacity, as slippage within this interface boundary is allowed. The interface condition for the imaginary foundation involved certain fixation of shared nodal points between the imaginary foundation and the soil structure, to induce a interface condition. The effect of allowing slippage within the two vertical interfaces at the edge of the foundations is causing the shear strain rate to be reduced thus reducing the final capacity of the underlying soil structure.

The reasoning for the result obtained for the D/B ratio of 0 is unknown at this stage of study and requires further investigation. Due to the requirements of this dissertation time did not permit further investigations into this result and it was assumed that this result was just an outlier for this case and would have minimal effect on the use of the model throughout further analysis of foundation interface effects.

In addition to comparing the weightless model's outputs with the imaginary foundation outputs, a comparison of the smooth interface results obtained from the weightless model was made with previous upper bound lower bound studies conducted for the foundation located near a slope problem. This comparison has been presented within Figure 5-4. The two dotted lines represent the upper and lower bound results and the dashed black line and solid blue line represent the results obtained from the FLAC models for the weightless foundation and the imaginary foundation, respectively. It can be seen from these results that the FLAC solutions follow a similar trend to the existing solutions. The FLAC solutions can be most closely compared to the upper bound solution, although they give a consistently higher result than this method. But from the similarities present within the two numerical methods it was concluded that the FLAC model was producing adequate solutions for the foundation problem.

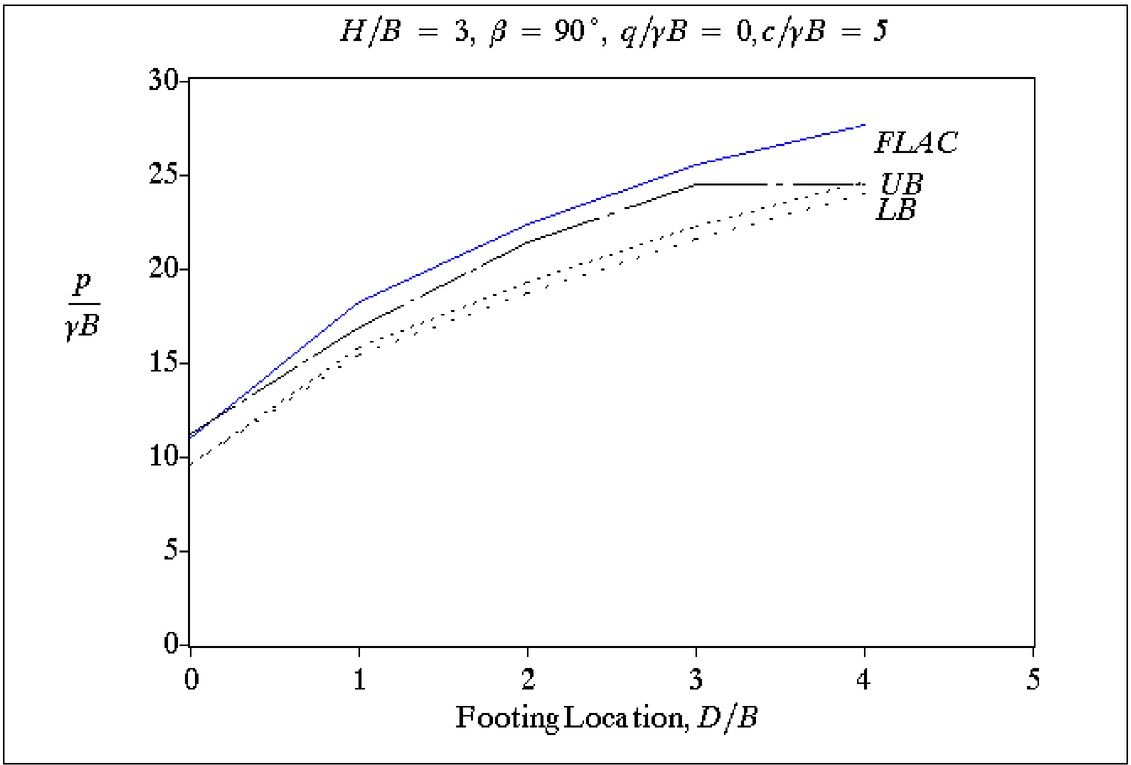

Figure 5-4. Comparison of Ultimate Bearing Capacity with Foundation Location

Figure 5-5 shows the shear-strain rate plots of the soil for the smooth interface case, for a range of different footing distance ratios. It can be seen from this figure that as the footing distance ratio is increased, the ultimate bearing capacity increases. This is due to the gradual transition from local shear slope failure at foundation locations close to the slope to general shear failure for foundation locations further from the slope. For the D/B ratios equal to or less than 2 it can be seen that failure is due to the slope. As the D/B ratio is increased to above three the shear strength ratio plot is symmetrical and the results obtained for the ultimate bearing capacity were equal to those of the flat ground failure situation. Therefore as the distance between the foundation and the slope is increased the strength within the soil structure is increased, due to the decreasing influence of the slope, thus yielding an increased ultimate bearing capacity.

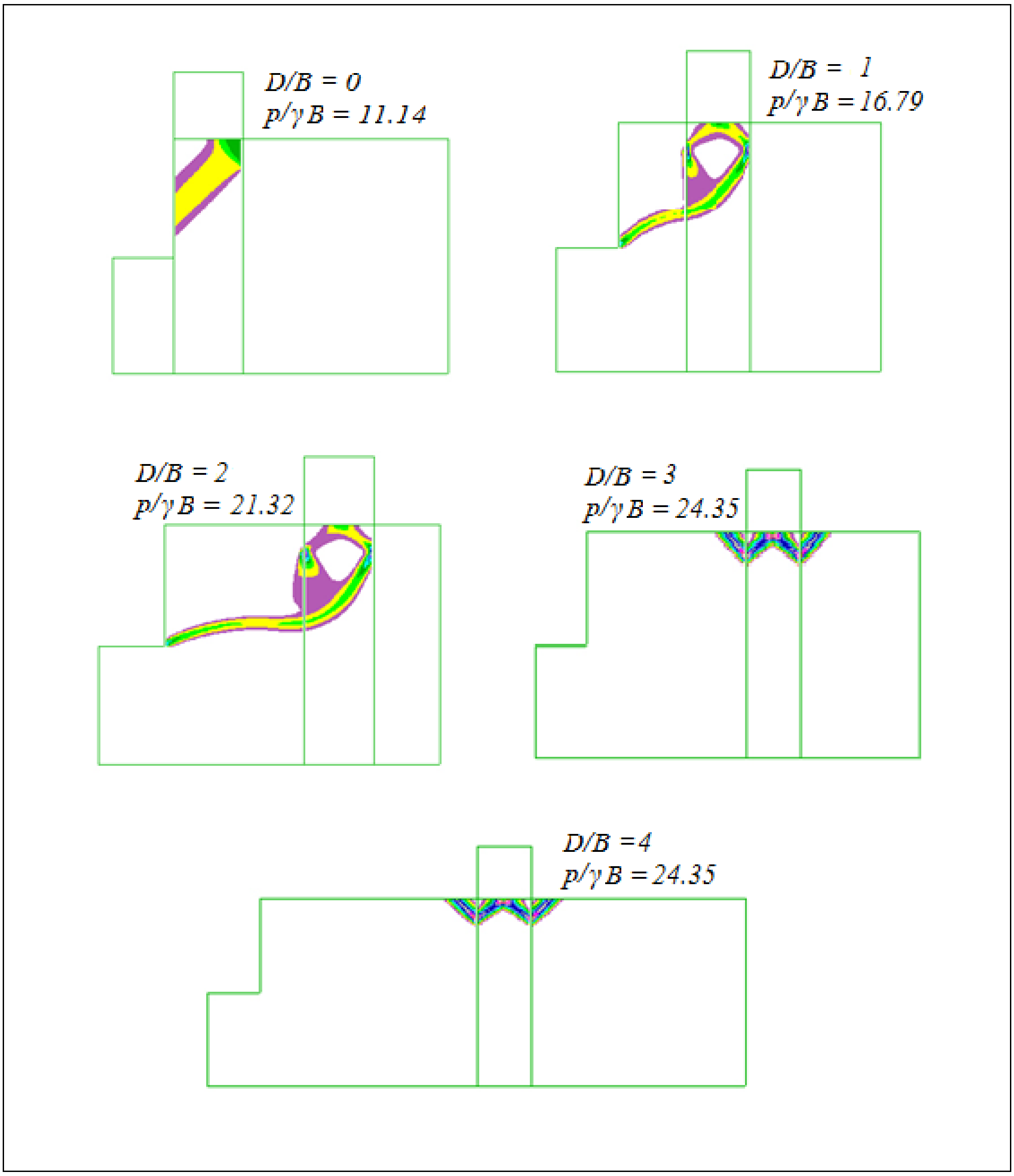

Figure 5-5. Change in normalised capacity with footing location for a smooth weightless model with considerations made for interfaces.

In addition to the increased capacity with increased D/B ratio, the shear strength ratio plot produced for the flat ground failure cases, (D/B = 3, 4), the uplift forces are shown quite clearly and when compared with plots produced by the imaginary foundation (Figure 5-6) it can be seen that when the interface condition was taken into consideration the behaviour of the model under failure condition was more realistic.

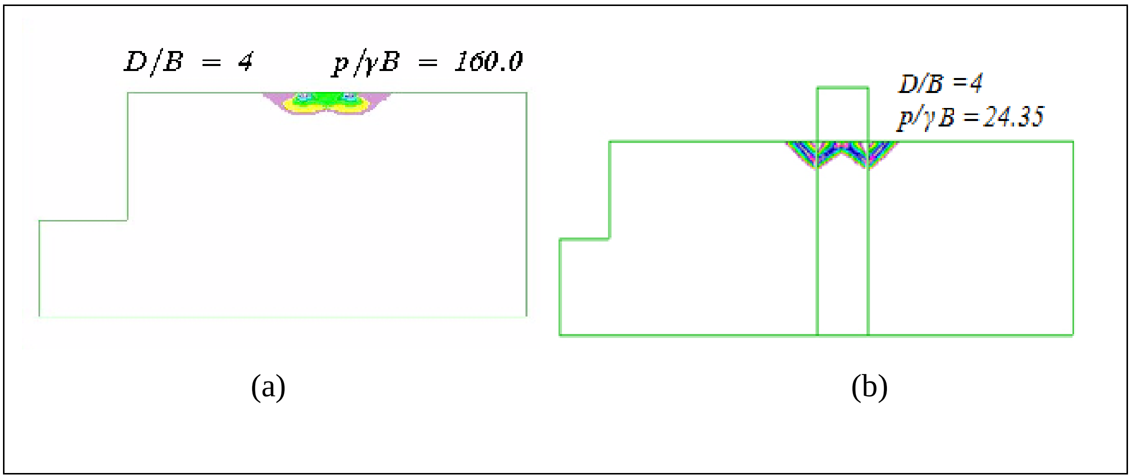

Figure 5-6. Shear Strain Ratio Plots (a) imaginary, smooth case with no interface considerations. (b) weightless foundation, smooth with interface considerations

Therefore from the comparison of the smooth imaginary foundation model and the smooth weightless foundation model and the results obtained for upper and lower bound limit state modelling, it can be seen that the inclusion of the horizontal interface between the foundation and the soil structure and the two vertical foundations at the edges of the foundation, have produced physical behaviour results that were more realistic, but due to the modelling of the vertical interfaces and the permittances of slippage within them, the capacity results obtained for the advanced, smooth, weightless discontinous foundation punching model, were reduced, as the shear-strain ratio within the soil structure was reduced.

5.4.2 Rough Interface

The rough interface model was simply the modelling of a weightless foundation problem with the cohesion level within the interface boundary equated to the soil structure cohesion, and slippage allowed within the two vertical interfaces, at either side of the foundation. The importance of modelling this interface condition was to evaluate actual foundation conditions as this case is most symbolic of the real life case of a shallow rigid foundation located near a slope.

Table 5-2 presents the validation results for the comparison of normalised bearing capacity with D/B ratio for the rough imaginary foundation proposed by Lyle and the rough weightless discontinuous foundation punching model, presented within this chapter.

Table 5-2. The comparison of ultimate bearing capacities.

D/B	"Imaginary" Foundation Proposed by Lyle (Smooth)	"Weightless" Advanced Discontinuous Foundation Punching Model (Rough)	Percentage Difference (%)
0	13.94	12.81	8.1
1	20.07	18.49	7.87
2	23.69	22.56	4.78
3	26.27	25.91	1.37
4	28.18	27.06	3.97

The main observation from these results was the similarity between the models. The percentage difference calculated, between the two models, was less than 10 % for all cases of D/B ratio investigated. It was also observed that again the weightless foundation produced capacities less than the simplified imaginary foundation model. Therefore it can be concluded from the comparison of the simplified model that the results obtained from weightless rough interfaced discontinuous foundation model were of an accurate standard, thus their use within further analysis of the discontinuous foundation punching modelling was warranted.

Figure 5-7 presents the comparison of the results obtained from; the imaginary smooth interface foundation model, the weightless rough interface model and the Upper Bound and Lower Bound limit results. The dotted lines are the Upper and Lower Bound limit, while the dashed black line represents the FLAC results for the rough weightless model and the blue solid line represents the results for the FLAC results for the smooth imaginary foundation model. From this graph it can be seen that the rough imaginary foundation model is producing larger capacities than the weightless foundation model, thus indicating the inclusion of the weighted foundation reduces the bearing capacity. The comparison of the Upper and Lower Bound limits and the FLAC models show the similar trend in the solutions obtained. Again the FLAC solutions can be most closely compared to the upper bound solutions, with again the FLAC model producing consistently higher bearing capacities.

Therefore from these results it can be concluded that the advanced FLAC model presented within this chapter is producing accurate representations of an actual foundation with respect to previous upper bound solutions of the problem.

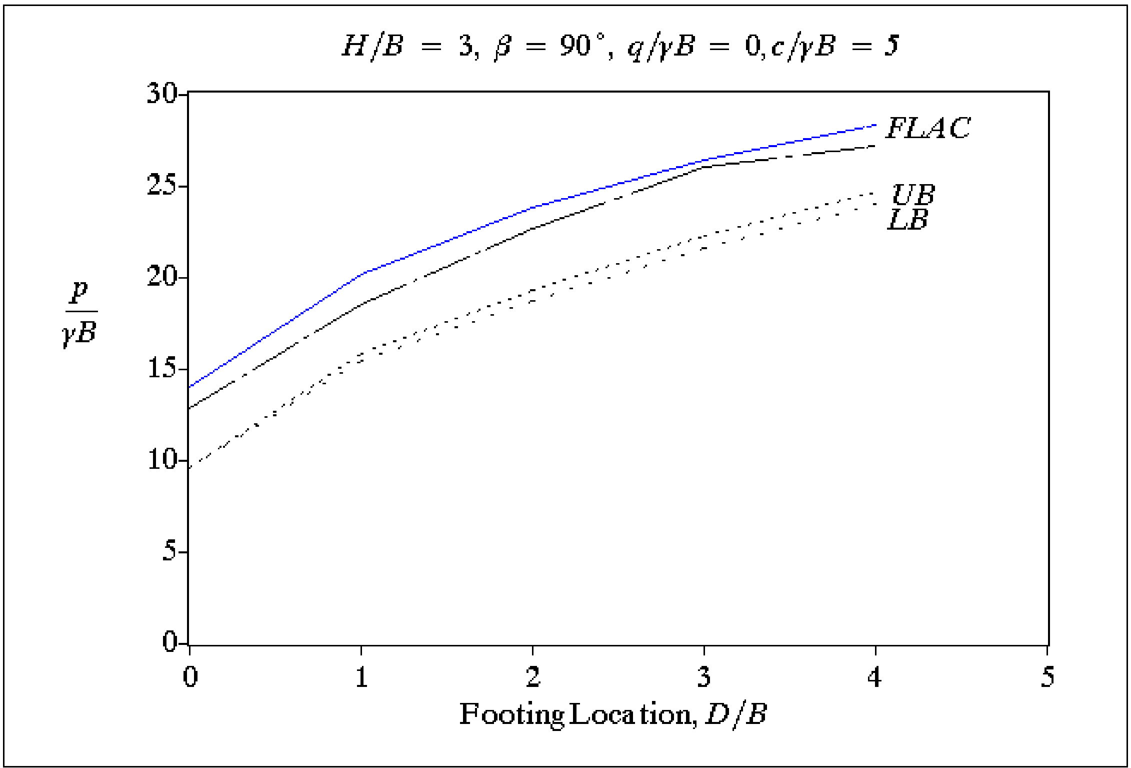

Figure 5-7. Change in normalised bearing capacity with footing location.

Figure 5-8 presents the shear-strain rate plots for the rough interface conditions for the purpose of investigating the failure mechanisms at different D/B ratios and the changes in normalised bearing capacity. The main observation within these results was the interesting behaviour of the slip surface, under rough interface conditions. At D/B ratios between 0 and 2 the slip surface was evident only at the slope surface and the normalised bearing capacity was still increasing with D/B ratio. At a D/B ratio of 3 there was presence of slip failure at both the slope surface and at the ground surface, thus indicating the presence of uplifting forces at the ground surface. This uplift force was the result of two factors; the transition between local shear failure to general shear failure and due to insufficient provisions within the model for vertical interface lengths. Within section 5.5 an investigation into the effects of this vertical interface length has been presented.

At a D/B ratio of 4 the slip surface is only evident at the ground surface thus indicating that flat ground failure has been induced within the model. It is interesting to note at this D/B ratio the separation wedges that are forming to either side of the foundation, this behaviour is symbolic of the physical modelling of the problem produced by Shiau et al, previously presented within Figure 5-2. Therefore it can be concluded from the results presented within this section that as the D/B ratio is increased the normalised bearing capacity increases until the failure mechanism reaches general shear failure, which occurs at a D/B ratio of 4, for the rough interface foundation punching model. It was also concluded from these results the need for an investigation of the required vertical foundation length, from visual inspection of the shear strain rate plot for a D/B ratio of 3.

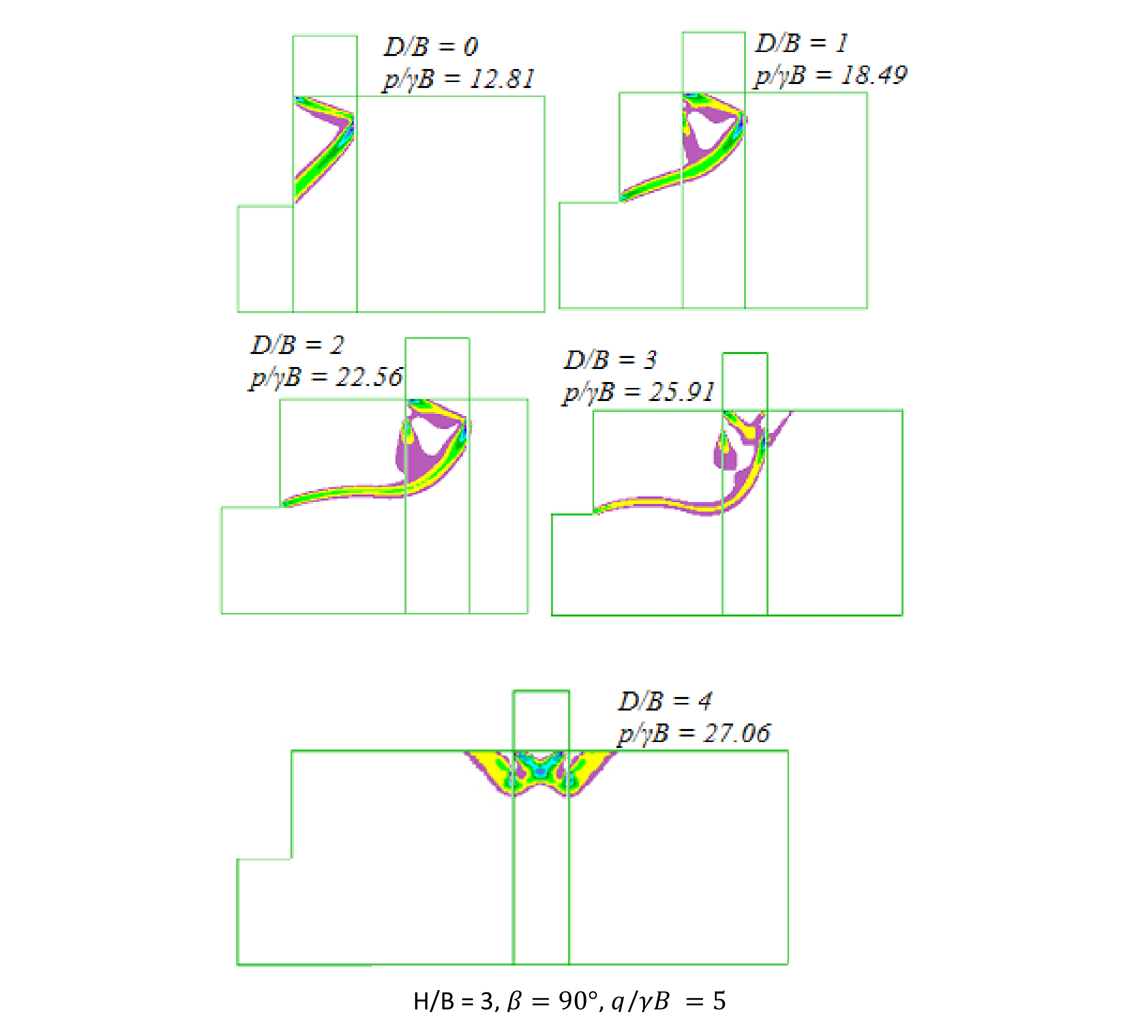

Figure 5-8. Change in normalised capacity with footing location for a rough weightless model with considerations made for interfaces.

5.4.3 Comparison

The comparison of the smooth and rough interface results obtained from the advanced modelling and analysis of the discontinuous foundation punching, is a crucial step within this section of study as it determines which method of interface modelling produces the more conservative ultimate bearing capacity. Table 5-4 presents the comparison of the change in ultimate bearing capacity with D/B ratio, for smooth and rough interface conditions, along with a calculation of the percentage difference between the two interface models. The main obvious trend within these results was the increase in the bearing capacity when the rough foundation condition was considered. From this observation it can be concluded that the smooth interface model produces more conservative values for ultimate bearing capacity, thus this model has been used within the further investigation of the interface length and with the validation of the previous simplified model produced by Lyle, for the imaginary foundation, under smooth interface conditions. Overall the results obtained from both interface face models were within relatively close proximity indicating that the inclusion of the interface friction level has minimal effect on the overall ultimate bearing capacity achieved.

Table 5-3. Comparison of Ultimate Bearing Capacity for Smooth and Rough Interface Conditions.

D/B	Smooth Interface	Rough Interface	Percentage Difference (%)
0	11.14	12.81	13.04
1	16.79	18.49	9.19
2	21.32	22.56	5.50
3	24.35	25.91	6.02
4	24.35	27.06	10.01

Figure 5-9 presents the ultimate bearing capacities that were previously presented within Table 5-3. From this figure it can be seen that the smooth interface case has arrived at general shear failure, thus achieved equilibrium within the ultimate bearing capacity produced, while the capacity of the rough interface case is still increasing, indicating general shear failure is yet to be reached. From visual comparison of the shear strain rate plots presented within Figures 5-5 and 5-8, for smooth and rough interface conditions, respectively, it can be clearly seen that D/B ratio required to induce general shear failure within the smooth interface model is less than the required D/B ratio for the rough interface. Thus concluding through visual analysis, the different distance required to transition the two models from local shear failure to general shear failure.

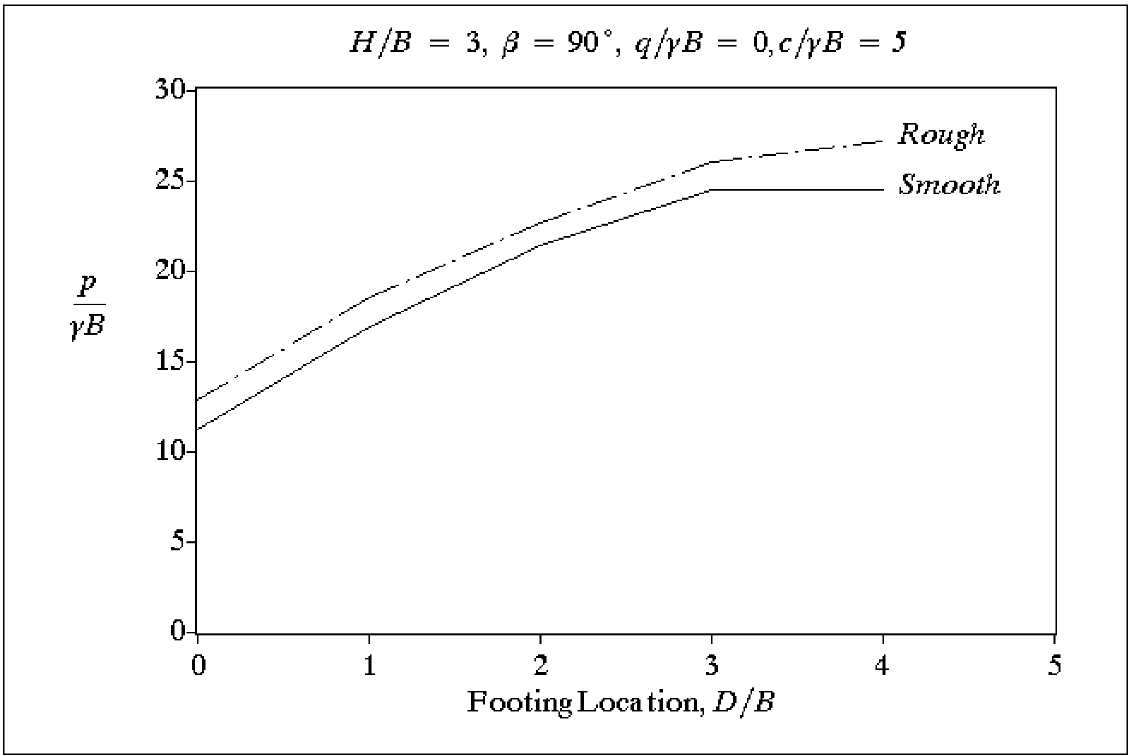

Figure 5-9. The Comparison of Ultimate Bearing Capacities for Smooth and Rough Interface Conditions.

From these results it can be concluded that the smooth interface case is yielding the lowest values for the ultimate bearing capacity, thus producing results that are more conservative. Whereas the rough interface model is producing capacities approximately on average 8.8% higher than the smooth interface model. These results confirm the assumption that the smooth interface is more conservative than the rough interface case. Therefore the use of the smooth interface model within the validation of Lyle's simplified imaginary foundation model has been adopted.

5.5 Interface Length Analysis

The interface length analysis involves the investigation of the effects the vertical interface length has on the failure mechanisms and the ultimate bearing capacity produced within the foundation model. For this vertical interface length investigation, two difference lengths were trialled; 0.5 meters (10 elements) and 1 meter (20 elements). Within previous interface investigations presented within this chapter the 0.5 meter length was used, thus this chapter presents the results for the 1 meter length interface, along with a comparison of the two lengths. From the previous results presented within this chapter it was determined that the smooth interface condition produced the more conservative values thus for the purpose of this investigation only a smooth interface was considered.

The first set of results presented for this investigation has been presented within Figure 5-10, which presents the change in normalised bearing capacity with D/B ratio for the lengths of smooth vertical interface. The main trend within these results was the reduction of the normalised bearing capacity with increased interface length. Thus indicating that the length of the interface is a significant parameter because ultimately it will affect the final ultimate bearing capacities achieved from a discontinuous foundation punching model.

Another interesting observation was the degree of difference between the two interface length capacities; initially at a D/B ratio of 0 there was minimal difference between the two models, but for D/B ratios between 1 and 3 the difference increases, until the 0.5 meter long interface reaches general shear failure at a D/B ratio of 3. At a D/B ratio of 4 the normalised bearing capacity for the 1 meter interface length model appears to still be increasing, thus indicating that general shear failure is yet to occur, however further investigations of this have been presented within Figure 5-11.

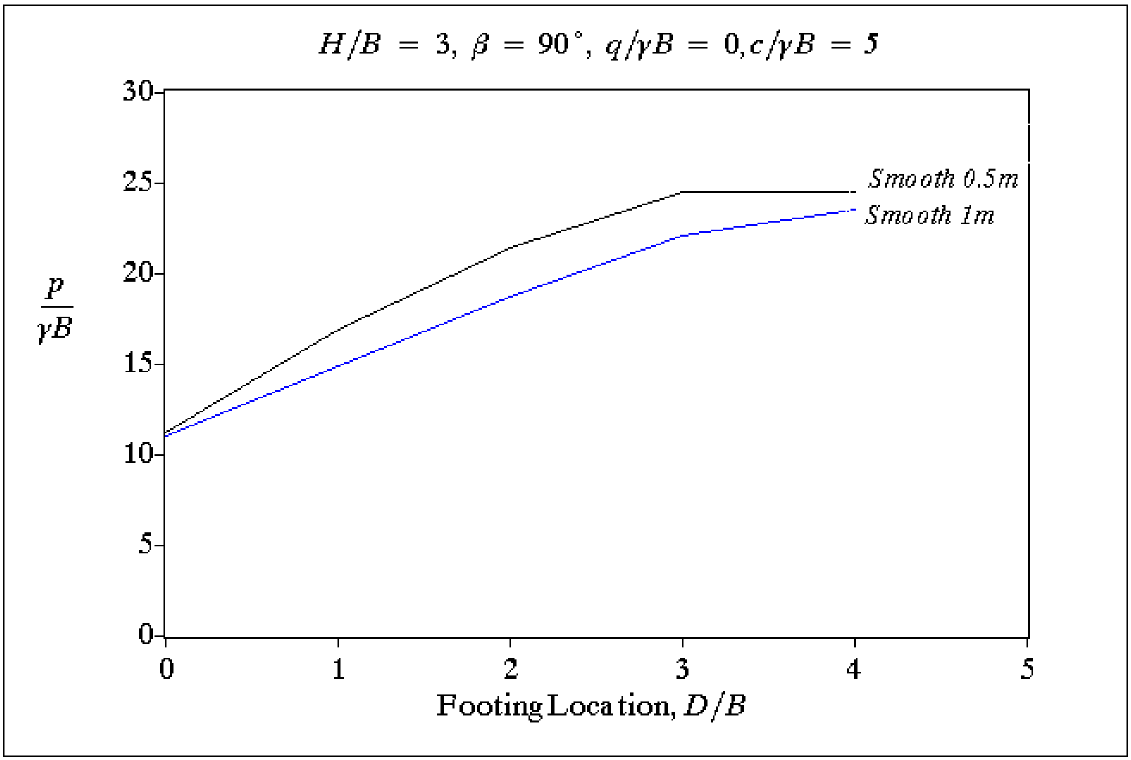

Figure 5-10. Comparison of Ultimate Bearing Capacities for Different Vertical Interface Lengths for Smooth and Rough Interface Conditions.

Figure 5-11 presents the stress strain rate plots for the change in normalised bearing capacity with D/B ratio for a vertical interface length of 1 meter. It can be seen from this figure that the ultimate bearing capacity is constantly increasing with D/B ratio and for D/B ratios less than or equal to 3 the slip surface is evident at the slope surface, but at a D/B ratio of 4 the slip surface has reached the ground surface resulting in uplift forces to either side of the foundation are present. This result indicates that the 1 meter long interface length requires a D/B ratio of 4 to induce general shear failure within the foundation. When Figure 5-11 is compared with Figure 5-5, the 0.5 meter interface model results, it can be seen that with increased interface length comes increased D/B ratio to induce general shear failure. Therefore it can be concluded that the bearing capacity of a foundation is reduced with increasing vertical interface depth. Thus for the purpose of the validation of the simplified model conducted within the next section of this chapter the interface length of 1 meter will be used, as this interface model produced the more conservative value for ultimate bearing capacity.

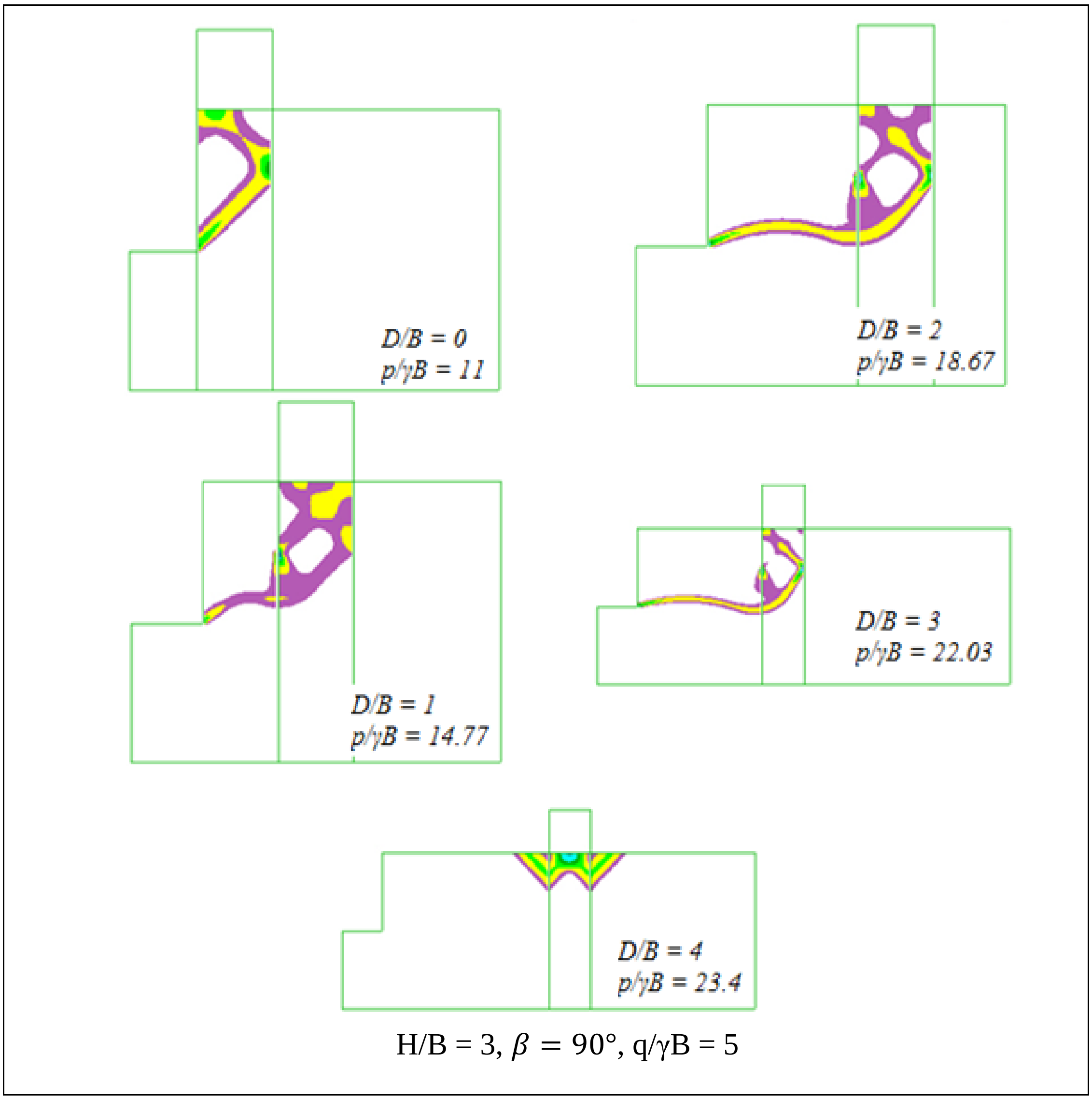

Figure 5-11. Comparison of ultimate bearing capacity with D/B ratio for a vertical interface length of 1 meter.

5.6 Validation of Simplified Model

Figure 5-12 presents the change in normalised bearing capacity with D/B ratio for the comparison of Lyle's simplified smooth interface imaginary foundation model and the advanced discontinuous foundation punching model with a weightless foundation. It can be concluded from this figure that the advanced discontinuous foundation modelling produces bearing capacities less than the imaginary foundation proposed by Lyle's studies. Thus it can be concluded from the results presented within this chapter that the advanced modelling of the interface at the soil structure and the interface that occurs due to the discontinuous foundation punching, produces the more conservative capacities. This is due to the advanced modelling of foundation within the model. Therefore before use of the preliminary design charts presented within Lyle's dissertation revision of the methodology of modelling the mesh should be revised as the values presented within this design charts are less conservative than those produced within this chapter of study.

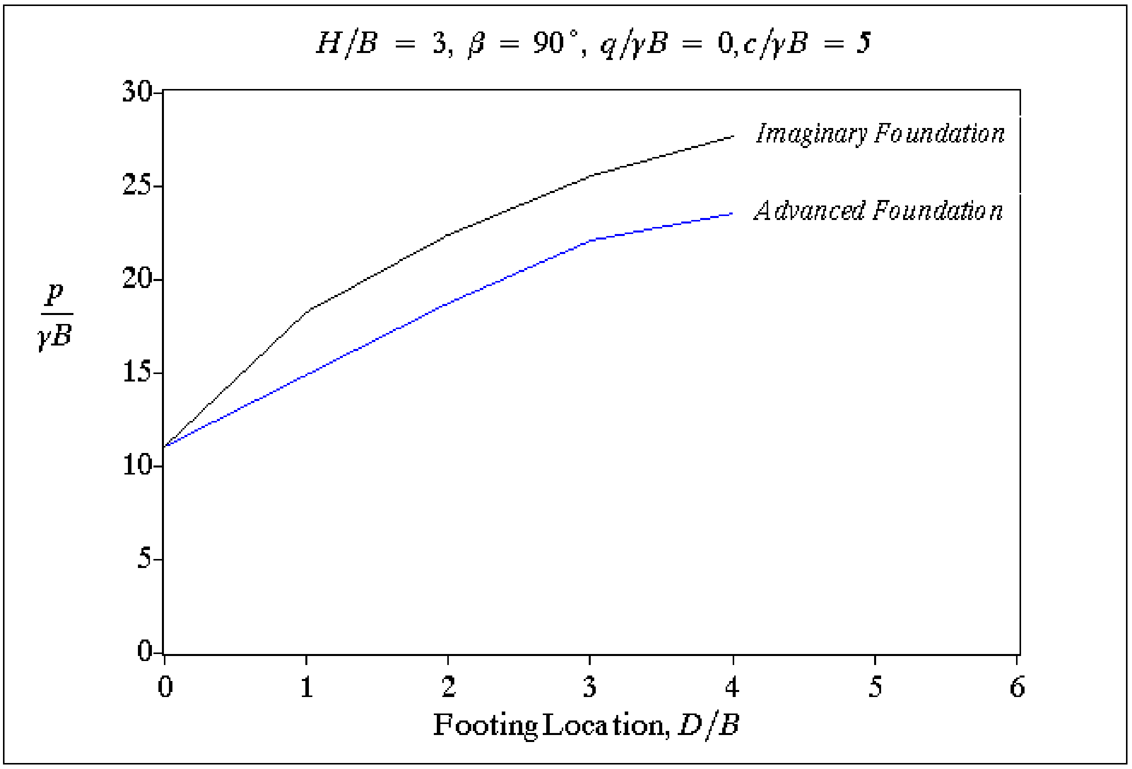

Figure 5-12. Change in normalised bearing capacity with D/B ratio for the imaginary foundation and the weightless foundation with discontinuous modelling the foundation punching.

5.7 Chapter Summary

From the study presented within this chapter a number of conclusions were drawn from the results produced. The first conclusion drawn was that the inclusion of the weightless foundation reduced the overall capacity of the foundation. It was concluded that this was due to the method of modelling the interface boundary and the allowed slippage within the boundary.

The second finding was that the smooth interface discontinuous foundation punching model produced bearing capacities less than the rough interface condition. This was concluded to be due to the increased friction between the rough foundation, thus the

increased resistance to movement that comes within friction. Thus for further investigations within this chapter the smooth interface case was adopted.
The third conclusion made within this chapter was with respect to the vertical interface length, within the model. It was concluded that increases in the modelled vertical interface reduced the ultimate bearing capacity. Therefore to ensure the most conservative model was used within the validation of Lyle's simplified imaginary foundation model, the longer interface length investigated was used.

The fourth and final conclusion made within this chapter was that the advanced modelling of the discontinuous foundation punching with a smooth soil structure interface and vertical interface lengths of 1 meter produced ultimate bearing capacities that were less than Lyle's imaginary foundation model. Therefore the advanced model presented within this chapter produced more conservative evaluations for ultimate bearing capacity. Thus the main recommendation from this section of study for advance modelling and analysis of the shallow rigid foundation located near a 90^{o} slope problem, is that the design charts based on the simplified model produced by Lyle should be revised to ensure overestimation of foundation capacities within preliminary foundation designs does not occur.

5.8 Future Work

One particular area for future development within this study would be the inclusion of the building weight and the investigation of how this inclusion affects the failure mechanism and ultimate bearing capacity of the foundation. Unfortunately due to the scope of this assignment time was not permitted to continue this investigation further.

Large Strain Analysis

6

6.1 Introduction

This chapter presents the analysis and evaluation of large strain analysis for the advanced modelling of the shallow rigid foundation located near a purely cohesive clay soil structure slope, within FLAC. Large strain analysis within FLAC means the geometry of the model is continuously updated throughout the loading process to take into account the effects of the additional moments caused by the moving load. The incorporation of large strain analysis within the model will provide more realistic solutions for the shallow foundation problem located near a slope, due to the effect of displacement being taken into consideration.

The models that have been investigated for large strain analysis within this chapter include;

- A smooth soil structure interface model with weighted foundations.
- A rough soil structure interface model with weighted foundations.
- A smooth discontinuous foundation punching model with weightless foundations.
- A rough discontinuous foundation punching model with weightless foundations.

Therefore this chapter will further develop the previously presented advanced models within chapters four and five to investigate and compare the effects that both small and large strain analysis has on the ultimate bearing capacity of the foundation problem.

The parameters which are relevant to this chapter include;

- $c/\gamma B$ soil strength ratio.
- D/B footing distance ratio.
- H/B slope height ratio.
- $p/\gamma B$ normalised bearing capacity.

The values of other essential parameters used throughout this study have been kept constant with respect to the parameters set within chapters four and five.

The problem statement for the large strain analysis of the soil structure interface has been included within Figure 6-1. The interface type for this problem is either rough (c_a = c) or smooth (c_a = 0). The foundation that has been modelled was weighted thus the foundation density was set to 2000kg/m^3.

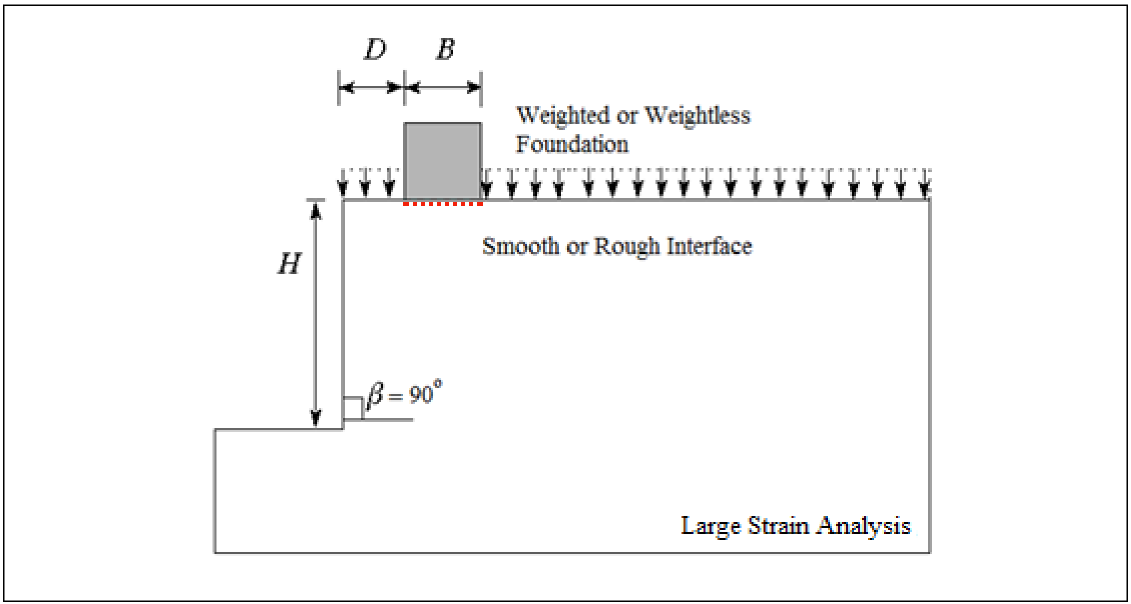

Figure 6-1. Problem Description for Large Strain Analysis of the Soil Structure Interface

The problem statement for the large strain analysis of the discontinuous foundation punching interface has been included within Figure 6-2. The interface type for this problem was either rough (c_a = c) or smooth (c_a = 0). The foundation was only considered to be weightless, thus the foundation density was 0.1 kg/m^3.

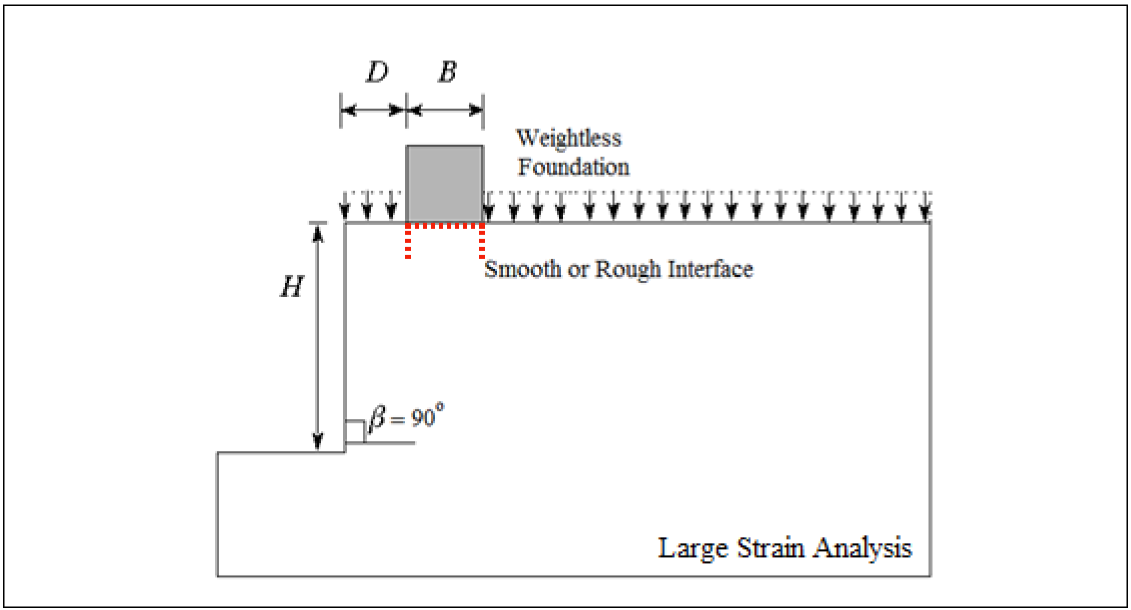

Figure 6-2. Problem Description for Large Strain Analysis of the Discontinuous Foundation Punching Interface.

6.2 Large Strain Analysis of the Soil Structure Interface

The first model that was investigated within this chapter was the large strain analysis of the soil structure interface model that was previously presented within chapter 4, under small strain analysis. It was concluded from the small strain analysis that the inclusion of the foundation weight produced capacities that were more conservative than the model that did not include the foundation weight, thus for the purpose of this study only a weighted foundation will be modelled and analysed for large strain analysis. It was also concluded within chapter 4 that the modelling of the smooth soil structure interface produced ultimate bearing capacities that were more conservative than the rough soil structure interface, for the purpose of this study however, both the smooth and rough soil structure interfaces have been model and analysed to determine whether the introduction of the large strain analysis has any affect on the final conclusions made within chapter 4. This study of large strain analysis for the smooth and rough soil structure interface model is an essential step within the advanced modelling of the shallow rigid foundation situated near a slope problem as large strain analysis of the problem will produce the continual displacement that is occurring during the loading phase of the foundation material, thus the results obtained within this study will be more accurate representations of actual shallow foundations located near slopes.

The relevant parameters to this large strain analysis study include;

- D/B = 0, 1, 2, 3, 4, 5, 6 (footing distance ratio)
- H/B = 3 (slope height ratio)
- $c/\gamma B$ = 5 (soil strength ratio)
- $B = 1$ (width of footing)
- β = 90^o (slope angle)
- ϕ =0^o (friction angle of soil)
- γ = 1.962 kN/^{m3} (unit weight of soil)
- q = 0km/m (surcharge pressure)

6.2.1 Large Strain Analysis of the Smooth Soil Structure Interface

The first set of results for the large strain analysis of the smooth interfaced soil structure interface model for a weighted foundation condition has been presented within Figure 6-3. Presented within this figure is the change in normalised bearing capacity with D/B ratio for small and large strain analysis of the model. The most obvious trend within this graph was the increased capacity for the large strain analysis of the model, when compared to the small strain analysis of the model. When the difference between the capacities produced by both analysis models was considered, it was seen that the difference between them increased as the D/B ratio increased. This result indicated the mesh regeneration that occurred within large strain analysis, resulting in an increased overall capacity of the foundation problem.

Another obvious observation from this figure was the behaviour of each analysis method with respect to failure mechanisms. At a D/B ratio of 4 the small strain analysis of the model had reached general shear failure, after this D/B ratio the capacity produced is constant. But when the large strain analysis of the model was considered it can be seen that the capacity was still increasing at a D/B ratio of 6, thus indicating that the failure mechanism is still local shear failure. From this aspect of the results it can concluded that the presence of the slope within the model affects the ultimate bearing capacity of the foundation at greater distances from the slope under large strain analysis of the problem. This is an interesting finding as the ultimate bearing capacity of the foundation is greater than the small strain analysis.

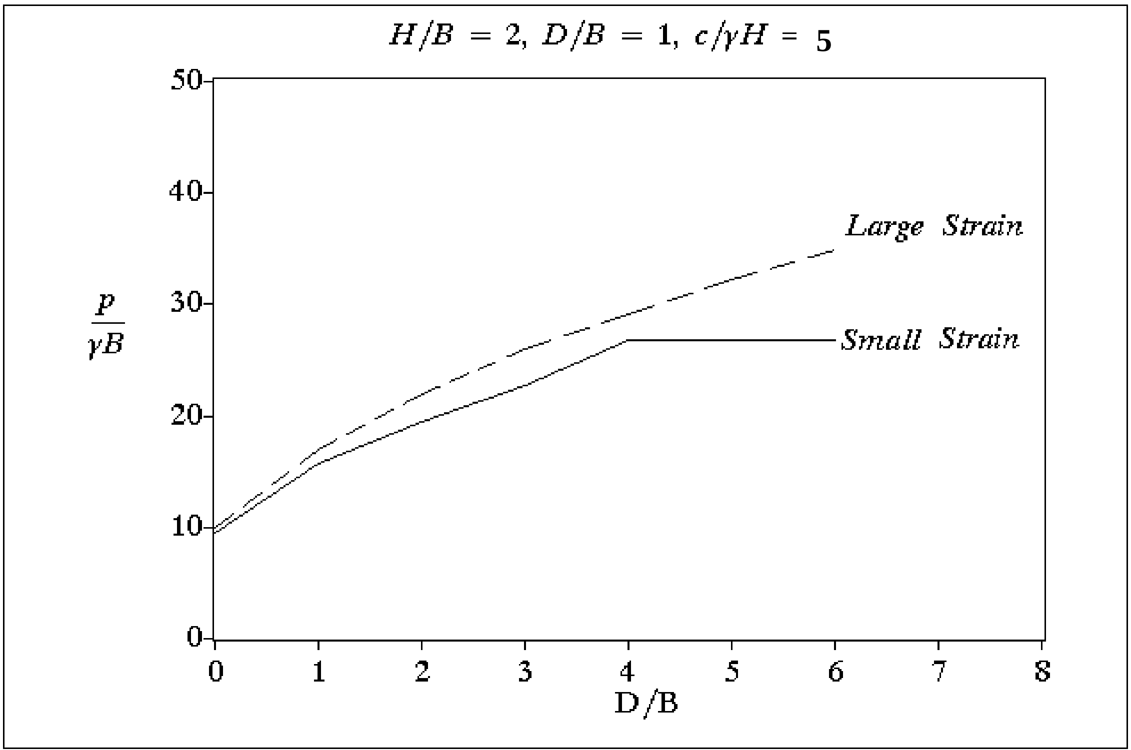

Figure 6-3. Comparison of Small Strain and Large Strain Analysis Ultimate Bearing Capacity Results for Varying Footing Distance Ratios.

Table 6-1 represents the results presented within Figure 6-3 along with actual calculations of the percentage differences at different footing distance ratio locations for small and large strain analysis. It was clear from these percentage differences that the difference between the two models, increased as the D/B ratio increased, except for at a D/B ratio of four, where the small strain analysis modelling of the foundation reached flat ground failure.

Table 6-1. The Comparison of Ultimate Bearing Capacities for Small and Large Strain Analysis, for the Weighted Foundation under Smooth Interface Conditions.

D/B	Small Strain Analysis	Large Strain Analysis	Percentage Difference %
0	9.36	9.86	5.07
1	15.66	16.91	7.4
2	19.39	21.87	11.34
3	22.59	25.78	12.37
4	26.60	29.02	8.34
5	26.60	32.04	16.98
6	26.60	34.63	23.19

Figure 6-4 and continued on Figure 6-5, presents the change in normalised bearing capacity with D/B ratio for large strain analysis of the smooth soils structure interface with a weighted foundation. The results presented within these figures are the shear-strain rate plots and the mesh deformations for the range of footing distance ratios investigated for the smooth weighted foundation model. When the results were compared with the results presented within chapter four, for the smooth soil structure interfaced model it was seen that the deformation of the mesh and stress strain plots was greater, this was due to FLAC continuously updating the geometry of the problem, to allow further mesh deformation to occur. Within this increased deformation there was also a degree of separation occurring between the foundation and the soil structure, as the load punched into the soft clay soil. This separation behaviour within the model presents more realistic failure behaviour for the foundation, than those presented within chapter four. Thus it could be assumed the results produced by the large strain analysis of the smooth soil structure interface model are more realistic representations of actual foundation capacities.

Another important observation made from the figures, was the presence of the uplifting of the soil that occurred at D/B ratios equal to and greater than 4. This was previously not seen within small strain analysis stress-strain plots, until flat ground failure had occurred. From the figures it can be observed that the large strain analysis model is yet to reach flat ground failure as stability within ultimate bearing capacities is still to be achieved at a footing distance ratio of 6 and the stress strain plot is not yet completely symmetrical, which is a common indication of the flat ground failure mechanism.

Figure 6-4. Change in normalised bearing capacity with D/B ratio for large strain analysis of the weighted foundation subjected to smooth soil structure interface conditions.

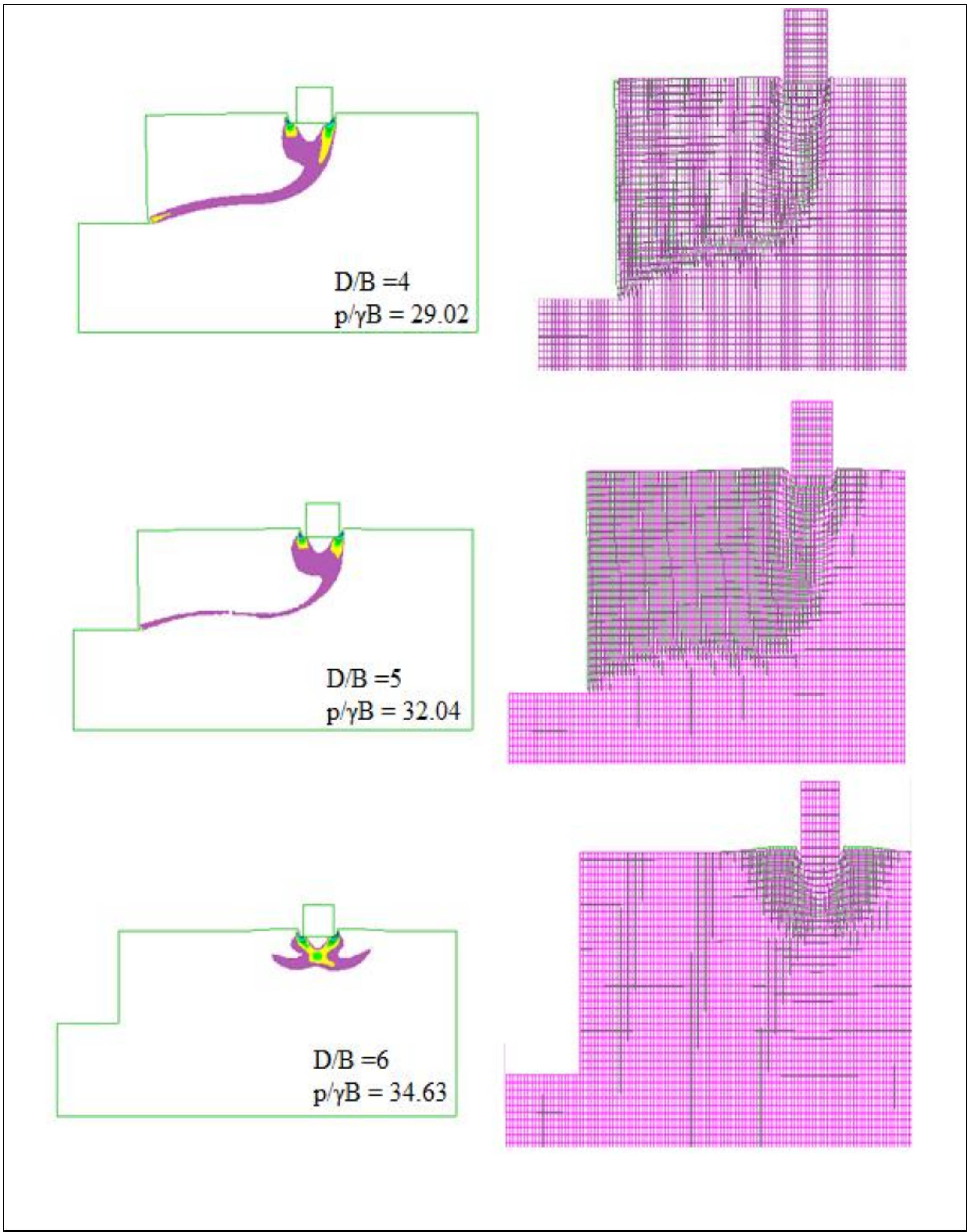

Figure 6-5. Change in normalised bearing capacity with D/B ratio for large strain analysis of the weighted foundation subjected to smooth soil structure interface conditions (continued).

6.2.2 Large Strain Analysis of the Rough Soil Structure Interface

Also presented within chapter four was the small strain analysis of a shallow foundation that was weighted, thus had a set density within FLAC as 2000kg/m^3, with a rough interface condition. Meaning the boundary between the foundation base and the soil structure had an applied cohesion and allowed for slippage to occur. This model was again investigated within this chapter with the addition of large strain analysis. Presented below are the results and conclusion from the large strain analysis investigation for the weighted foundation model under rough interface conditions.

Figure 6-6 presents the ultimate bearing capacity results for both small strain and large strain analysis of the weighted rough soil structure interface model. Small strain has been included to compare and evaluate the overall effect of large strain analysis to this particular model.

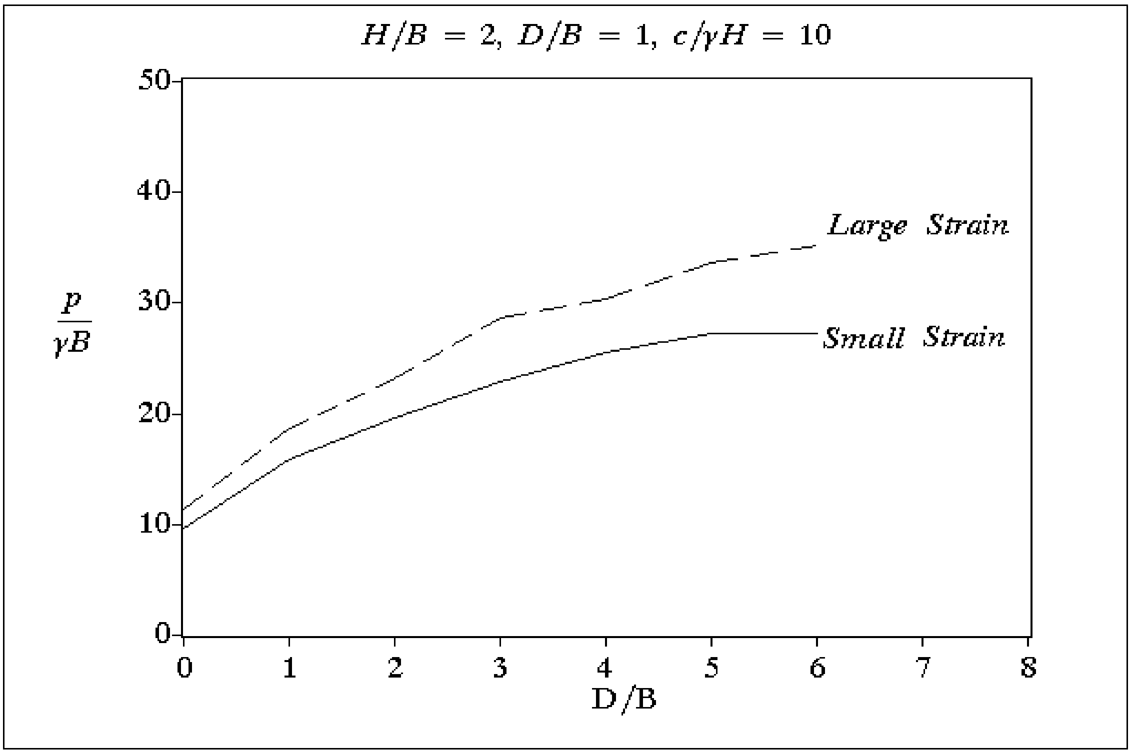

Figure 6-6. The Comparison of Ultimate Bearing Capacities for Small and Large Strain Analysis, for the Weighted Foundation under Rough Interface Conditions.

The results presented within Figure 6-6 have also been presented within Table 6-2, along with the percentage differences between the capacities produced by each strain analysis method.

Table 6-2. Comparison of Small and Large Strain Analysis Ultimate Bearing Capacity Results for the Rough Soil Structure Interface model, with Varying Footing Distance Ratios.

D/B	Small Strain Analysis	Large Strain Analysis	Percentage Difference %
0	9.51	11.21	15.17
1	15.78	18.48	14.61
2	19.49	23.11	15.7
3	22.68	28.47	20.34
4	25.44	30.27	15.96
5	27.17	33.46	18.8
6	27.17	34.98	22.33

It is evident from visual inspection of the results presented within Figure 6-6 and Table 6-2 that the ultimate bearing capacities being produced by large strain analysis of the rough soil structure interface model are greater than those produced by small strain analysis. This characteristic is common to large strain analysis of both smooth and rough soil structure interface models, therefore it can be concluded that the large strain analysis of this soil structure interface model, will always yield larger ultimate bearing capacities than small strain analysis of the problem, due to continual geometry updating that is occurring within large strain analysis.

The percentage difference between the two strain analysis methods, presented within Table 6-2, like for the smooth interface condition, has a general trend, with the exception of the footing distance ratio of 5, to increase as the footing distance ratio increases. This is due to the ability of continuous model geometry within FLAC for large strain analysis. Thus as the distance between the foundation and the slope is increased the area of deformation that can occur directly underneath and in closely surrounding soils to the foundation is increased due to the increased soil surface area, that results from moving the foundation further away from the slope. This behaviour is also evident in the transitional phase between local shear slope failure and flat ground failure that occurs, from the results it can be seen that the rough soil structure interface model was behaving in the same way as the smooth soil structure interface model presented within the previous section, with the large strain analysis model not yet reaching flat ground failure at a D/B ratio of 4, which occurred at a D/B ratio of 3 for the small strain analysis of this model. This transitional behaviour between local shear slope failure and flat ground failure was investigate within this section with the presentation of the shear-strain rate plots and the general FLAC mesh deformation that occurred within the model under large strain analysis.

Figures 6-7 and 6-8 present the change in normalised bearing capacity with D/B ratio for the large strain analysis of a weighted foundation subjected to a rough soil structure interface condition. Within these figures are the stress strain rate plots and mesh deformation outputs for the weighted foundation under rough soil structure interface conditions. Presented to the left of this figure are the stress strain plots for a range of footing distance ratios and to the right are the respected mesh deformation outputs for the different footing distance ratios.

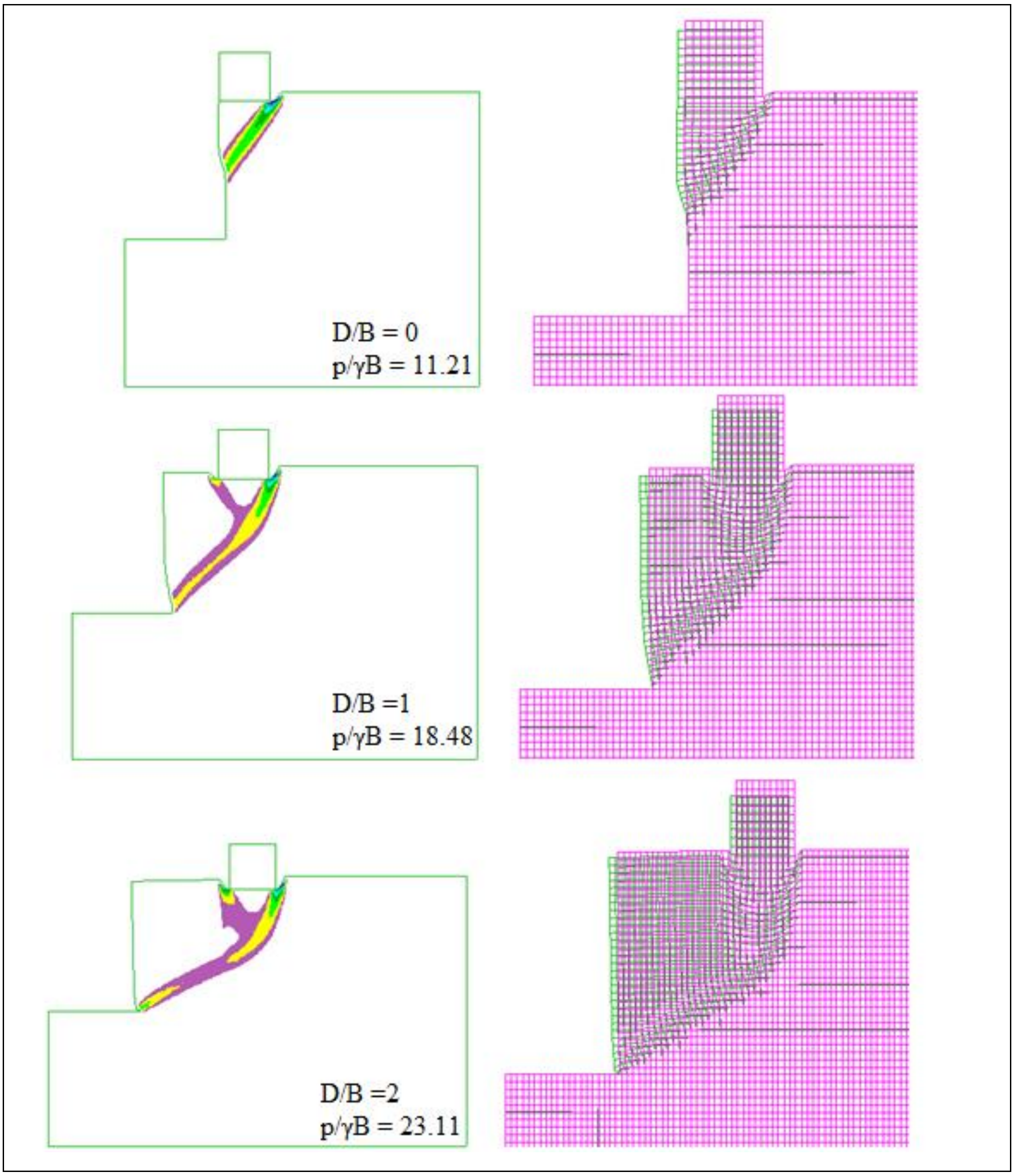

Figure 6-7. Change in normalised bearing capacity with D/B ratio for large strain analysis of the weighted foundation subjected to rough soil structure interface conditions.

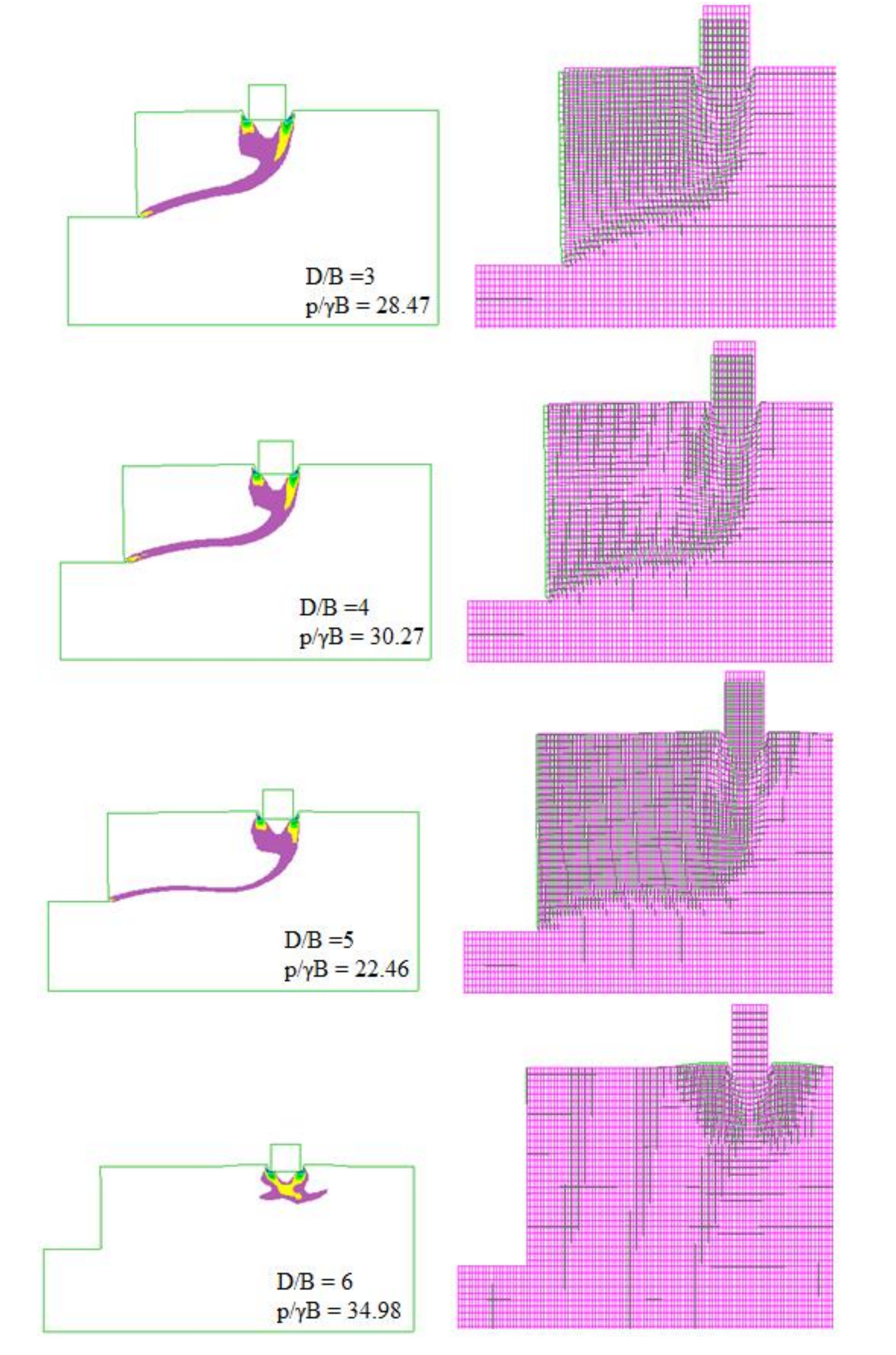

Figure 6-8. Change in normalised bearing capacity with D/B ratio for large strain analysis of the weighted foundation subjected to rough soil structure interface conditions. (continued)

From the two plots presented above it was initially noted from the mesh deformation plots that there was forward movement of the deformed foundation between the D/B ratios 0 to 5, and at a D/B ratio 6, clear foundation punching was evident. This forward movement of the foundation would be the result of the additional friction forces that result from the rough interface conditions and the foundation load and stiffness acting downwards on the soil structure. In addition to the movement of the foundation there also appeared to be some bulging of the soil structure on the slope side for D/B ratios between 0 and 4 and for D/B ratios 5 and 6, uplifting of the soil appeared at either side of the foundation. This bulging and uplift behaviour would be the result of the foundation load. When this model is compared with actual shallow foundations constructed near slopes, the characteristics of the soil presented within the large strain analysis model, would be evident at failure. Thus the results presented by large strain are again considered to be more realistic representations of a shallow foundation problem situated near a clay soil slope, due to the failure mechanisms that are presented within Figures 6-7 and 6-8.

When the ultimate bearing capacities were compared with the smooth interface case it was noted that the large strain analysis of the rough soil structure interface model for a weighted foundation, yielded larger capacities. This result coincides with the findings from chapter five, that rough interface conditions are less conservative, thus produce larger vales for the normalised bearing capacity of the foundation.

Therefore from the results presented within this section for the weighted foundation modelled with a rough horizontal soil structure interface, it can be concluded that large strain analysis produced more realistic results for the foundation problem with respect to mesh deformation and stress strain rates within the underlying soil structure. However the ultimate bearing capacities obtained from large strain analysis were significantly higher than the small strain analysis, thus for the purpose of constructing preliminary design charts for foundation capacities, the use of large strain analysis would be insufficient as overestimation of a foundations capacity would result.

6.3 Large Strain Analysis of Discontinuous Foundation Punching

The second model that was investigated within this chapter was the large strain analysis of the discontinuous foundation punching model that was previously presented within chapter 5, under small strain analysis. Within this section only the modelling of the weightless foundation will be investigated under large strain analysis, due to this model being the final model presented within chapter 5 that yielded the most conservative capacities for the foundation problem. It was concluded within chapter 5 that the modelling of the smooth soil structure interface produced ultimate bearing capacities that were more conservative than the rough soil structure interface, for the purpose of this study however, both the smooth and rough soil structure interfaces have been modelled and analysed to determine whether the introduction of the large strain analysis has any affect on the final conclusions made within chapter 5. This study of large strain analysis for the smooth and rough discontinuous foundation punching model is an essential step within the advanced modelling of the shallow rigid foundation situated near a slope problem as large strain analysis of the problem will analysis the continual displacement occurring during the loading phase of the foundation material, thus the results obtained within this study will be more accurate representations of actual shallow foundations located near slopes.

The relevant parameters to this large strain analysis study include;

- D/B = 0, 1, 2, 3, 4, 5, 6 (footing distance ratio)
- H/B = 3 (slope height ratio)
- $c/\gamma B$ = 5 (soil strength ratio)
- $B = 1$ (width of footing)
- β = 90^{o} (slope angle)
- ϕ =0^{o} (friction angle of soil)
- γ = 1.962 kN/^{m3} (unit weight of soil)
- q = 0km/m (surcharge pressure)

6.3.1 Large Strain Analysis of the Smooth Interface

The first set of results for the large strain analysis of the smooth interfaced discontinuous foundation punching model for a weightless foundation condition has been presented within Figure 6-9. Presented within this figure are the changes in normalised bearing capacity with D/B ratio for small and large strain analysis of the model.

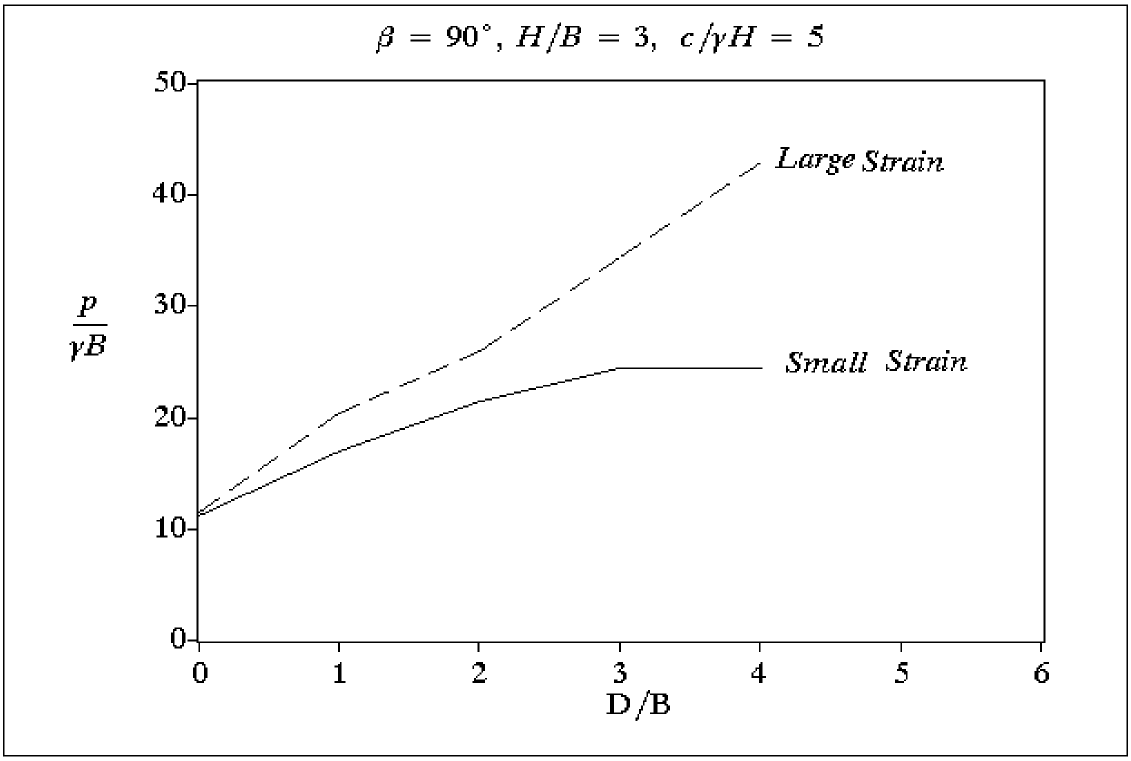

Figure 6-9. The Comparison of Ultimate Bearing Capacities for Small and Large Strain Analysis, for the Weightless Foundation under Smooth Interface Conditions.

Presented within Table 6-3 are the ultimate bearing capacities for the smooth interfaced weightless foundation that were previously presented graphically within Figure 6-9. Along with the capacities achieved, the percentage difference between the two has been included.

Table 6-3. The Comparison of Ultimate Bearing Capacities for Small and Large Strain Analysis, for the Weightless Foundation under Smooth Interface Conditions.

D/B	Small Strain Analysis	Large Strain Analysis	Percentage Difference %
0	11.14	11.36	1.94
1	16.79	20.31	17.33
2	21.32	25.91	17.72
3	24.35	34.28	28.97
4	24.35	42.81	43.12

It is evident from initial inspection of these results presented within Figure 6-9 and Table 6-3, that large strain analysis, again produced ultimate bearing capacities that were larger than those achieved from small strain analysis of the same smooth interfaced weightless foundation model presented within this section. This result coincides with the findings that were presented for the soil structure interface model, presented within section 6.2 of this chapter. The significant increases in the ultimate bearing capacity is a result of the continuous geometry updating that occurs within large strain analysis that allows the displacement of the geotechnical problem to be measured. It is thus expected that the results obtained from large strain analysis would be more realistic representations of actual foundation ultimate bearing capacities, this will be investigated further within the analysis of stress strain rate plots and mesh deformation figures for the model.

Like the smooth and rough soil structure interface models presented within section 6.2, the percentage difference that is occurring within this model is also increasing as the footing distance ratio is increased, again it can be assumed that this is due to large strain analysis not yet achieving flat ground failure, whereas the small strain analysis achieved flat ground failure at a D/B ratio of 3. It can be concluded that the flat ground failure within the large strain analysis is yet to be achieved due to the increased volume of soil that is directly below and surrounding the foundation that can be deformed, as a result of the slippage allowed within the horizontal soil structure interface and the two vertical discontinuous foundation punching interfaces.

Figure 6-10 presents the change in normalised bearing capacity with D/B ratio in the form of stress strain rate plots and mesh deformation figures for the large strain analysis of the smooth interfaced discontinuous foundation punching model. Within this figure a number of observations can be noted, the first being the bulging of the slope for a D/B ratio of 0. When the D/B ratio is increased to 1 the bulging of the soil structure is not evident but there is a certain notable degree of separation occurring between the foundation and the soil structure on the right hand side (the side of the slope). This separation was present within small strain analysis of the model, but not to the degree that is presented within the large strain analysis. Thus it can be concluded that the large strain analysis increases produces large deformation with respect to soil bulging and separation, due to the continual displacement updating that occurs within large strain analysis.

Another observation made from these results was the increase in separation and deformation within the mesh as the D/B ratio was increased. This result indicates that the surface area surrounding the foundation problem has increased thus permitting further deformation to occur. At D/B ratios of 3 and 4 there are significant soil bulging at the ground surface and mesh separation present within the deformation plot. This result signifies the transition from local shear failure to general shear failure (flat ground failure), thus the transition of the slip surface from the slope edge to the ground surface.

From the results presented within this section it can be concluded that the inclusion of the large strain analysis, increases the deformation within the FLAC mesh, which in turn increases the ultimate bearing capacity. It can also be concluded that the degree of separation and surface soil bulging produced within large strain analysis of the problem are more realistic representations of actual shallow foundations, but as this study is concerned with determining the most conservative method of modelling the shallow foundation problem this finding was insignificant, with respect to the validation of the simplified model.

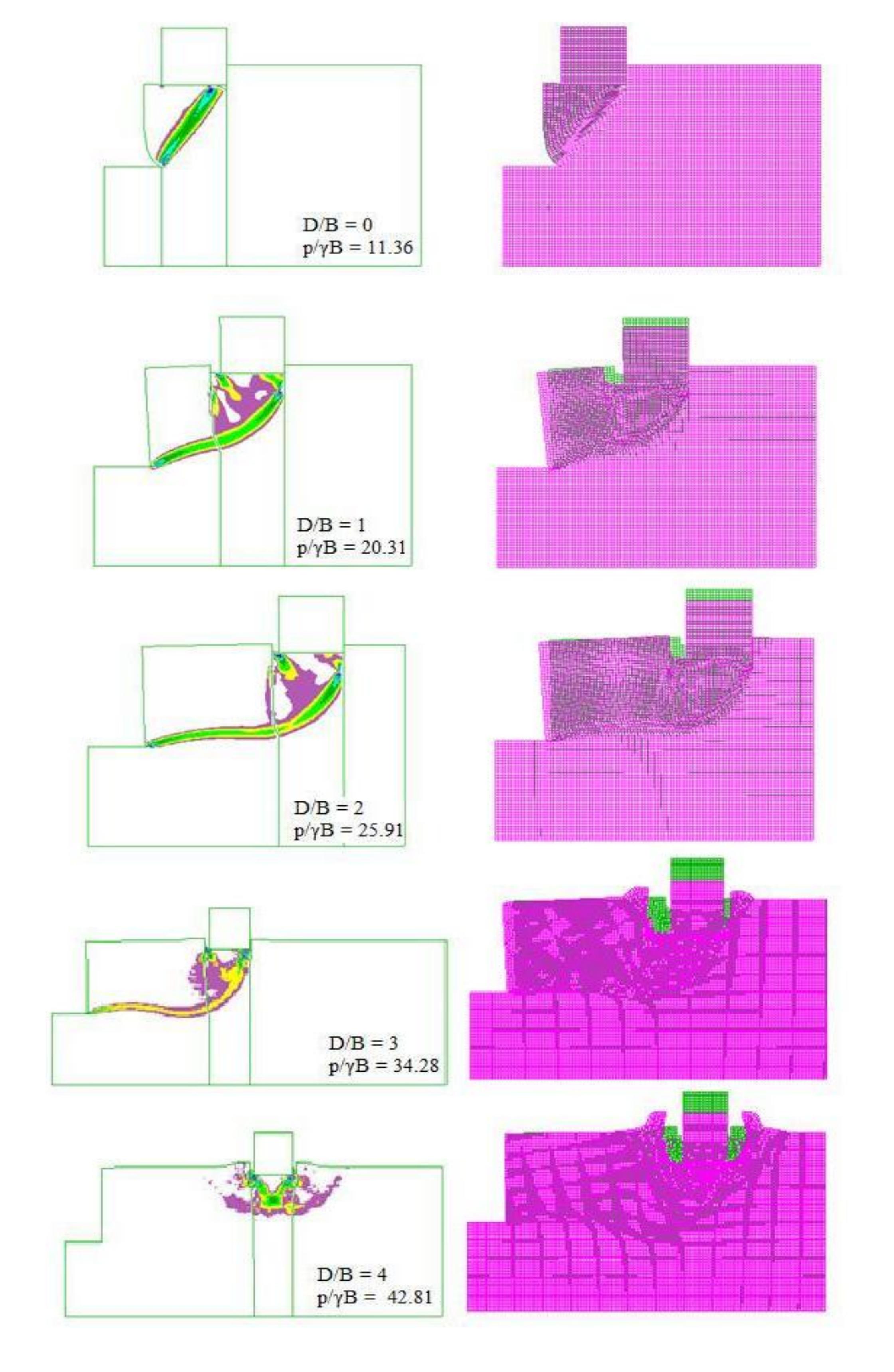

Figure 6-10. The Stress Strain Rate Plots and Mesh Deformations for Large Strain Analysis of the Smooth Interfaced Weightless Model.

6.3.2 Rough Interface

The large strain analysis of the rough interfaced foundation punching interface model for a weightless foundation, involved investigating the continual mesh regeneration for a rough soil structure interface. Within this model the horizontal soil structure interface was modelled with the cohesion within the boundaries equated to the soil structure and slippage was permitted within the horizontal interface boundary and within the vertical discontinuous foundation punching interface boundaries.

The first set of results for this large strain analysis has been presented within Figure 6-11, which presents the change in bearing capacity with D/B ratio for the comparison of small and large strain analysis. In addition to this figure, Table 6-4 presents the percentage differences between the changes in bearing capacity with D/B ratio for small and large strain analysis.

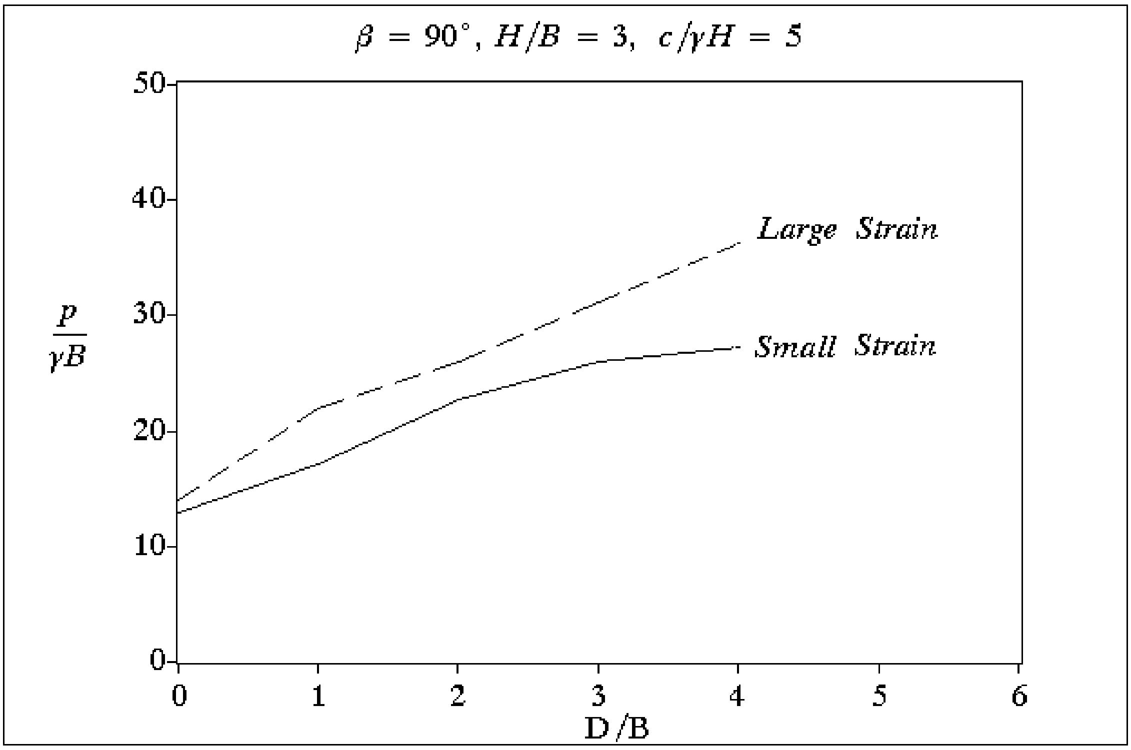

Figure 6-11. The Comparison of Ultimate Bearing Capacities for Small and Large Strain Analysis for the Weightless Foundation Under Rough Interface Conditions.

Table 6-4. The Comparison of Ultimate Bearing Capacities for Small and Large Strain Analysis, for the Weightless Foundation under Rough Interface Conditions.

D/B	Small Strain Analysis	Large Strain Analysis	Percentage Difference %
0	12.81	13.96	8.24
1	16.97	21.83	22.26
2	22.56	25.79	12.52
3	25.91	30.99	16.39
4	27.06	36.09	25.02

The main observation from these results was the reduced percentage difference between the two analysis models when compared with the large strain analysis of this model with smooth interface conditions. Also observed was the reduction in the ultimate bearing capacity with the large strain analysis of the rough interface, with respect to the large strain analysis of the smooth interface. This result indicates that the large strain analysis of the rough interfaced discontinuous foundation punching model is more conservative than the smooth interfaced model. But when the result were compared with the small strain of the smooth soil structure interfaced model weighted foundation, the proven conservative model, it can be seen that the capacity is significantly higher thus less conservative and would not be used within the preparation of preliminary design charts.

However for the purpose of full advanced analysis study of large strain analysis Figure 6-12 presents the stress strain plots and mesh deformations for the large strain analysis of the rough discontinuous foundation punching model. The deformation trends occurring within this figure are similar to those of the smooth interface case, but the scale is reduced significantly. Soil bulging is evident at a D/B ratio of 0 and at D/B ratios 1 to 4 the slope slips forward due to the degree of separation occurring between the foundation and soil structure at the punching interface. At D/B ratios of 3 and 4 again significant uplift forces are occurring at the ground surface resulting in soil bulging. It is also note worthy that the failure mechanism for this model is still at local shear failure for a D/B ratio of 4, but the slip surface was evident at both the slope surface and the ground surface. From these results it can be concluded that the large strain analysis of this rough interfaced model produced failure mechanisms that were more realistic representations of actual shallow foundations, therefore it could be assumed that the capacities produced by this model were more conservative representations.

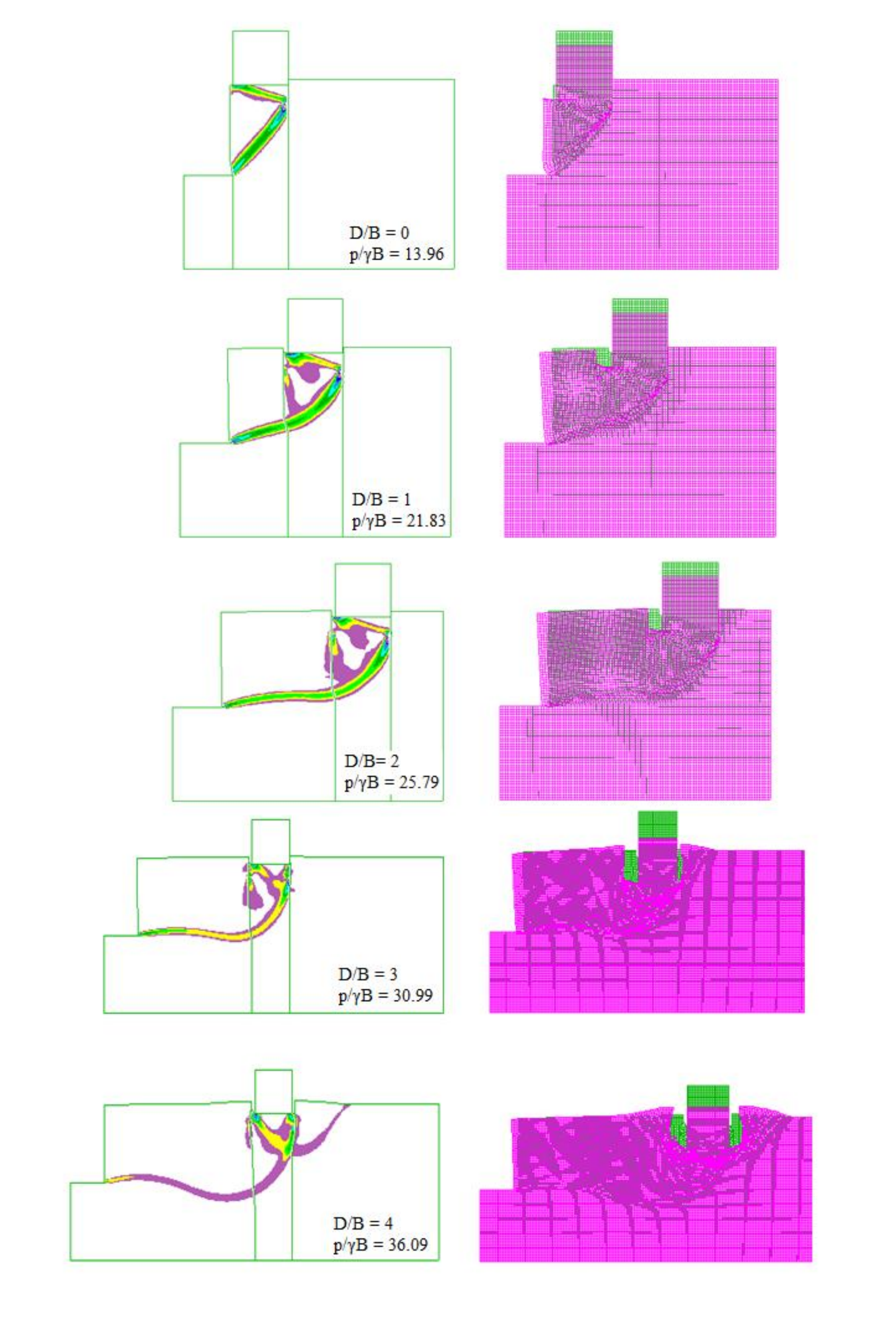

Figure 6-12. The Stress Strain Rate Plots and Mesh Deformations for Large Strain Analysis of the Rough Interfaced Weightless Model.

6.4 Validation of Simplified Model

It was determined from this study that the large strain analysis of the two models previously presented within chapters 4 and 5 produced deformation and failure mechanisms that were more realistic representations of actual foundations. But for the purpose of validating whether the ultimate bearing capacities being produced by the small strain analysis of the simplified model it can be concluded that the small strain analysis is more conservative as all the results presented within this chapter exceed the capacities produced in previous small strain analysis results. Therefore with respect to the simplified model using small strain modelling it can be concluded that this method of analysis is a conservative method of obtaining capacities for preliminary design charts.

6.5 Conclusion

The main conclusion drawn from this chapter was that large strain analysis produced failure mechanisms and mesh deformations that were more realistic representations of actual foundations located near slopes at failure, as the continued mesh regeneration that occurs within large strain analysis was depicting the deformations occurring more accurately, due to displacement. However the purpose also included the evaluation of the ultimate bearing capacity for the purpose of validating whether or not large strain analysis produced more conservative results than small strain analysis. From the results presented within this chapter it was clearly evident that for all four cases; smooth soil structure interface model, rough soil structure model, smooth discontinuous foundation punching model and the rough discontinuous foundation punching model that the large strain analysis produced significantly higher results for ultimate bearing capacity. Thus it can be concluded from this chapter that the modelling of small strain analysis is the most conservative method and the use of this method within preparation of preliminary foundation design charts is warranted.

Static Pseudo Seismic Modelling

7

7.1 Introduction

This chapter models and investigates shallow foundations situated near slopes that are subjected to additional earthquake-induced horizontal forces, also termed static pseudo seismic forces. The importance of this study was to evaluate the effects that these additional horizontal static pseudo seismic forces have on the ultimate bearing capacity of shallow foundations located near slopes and to evaluate the failure mechanism trends for different earthquake magnitudes. Within the geotechnical engineering discourse, there have been a number of different studies conducted to investigate the problem of seismic bearing capacities, along with a number of studies for seismic bearing capacities for shallow foundations located near slopes. To introduce the reader to the seismic bearing capacity problem a brief review of past studies within the area has been included within section 7.1.

This chapter will present the development of the static pseudo seismic model, the validation of the model and the use of the validated model within comprehensive parametric studies. The parameters that will be investigated within the parametric study include; footing distance ratio, slope height ratio and soil strength ratio. These different parameters will be investigated for a number of different earthquake magnitudes.

The parameter notations respected values, relevant within this chapter include;

$\beta = 90°$	slope angle
$D/B\ = 0, 1, 2, 3, 4, 5, 6$	footing distance ratio.
$H/B\ = 0, 1, 2, 3, 4, 5$	slope height ratio.
$c/\gamma B\ = 0.5, 5, 10, 20, 30$	soil strength ratio.
$W = 9.81$	gravity within soil structure
$k_h = 0.1, 0.2, 0.3, 0.4$	coefficient of horizontal acceleration.

The problem notation for the seismic bearing capacity of footings located near slopes is presented graphically within Figure 7.1. The foundation density that has been considered within this problem is a weightless foundation (footing density = 0.1kg/m^3) and the interface type that has been considered for this problem is either smooth (c_a = 0) or rough (c_a = c).

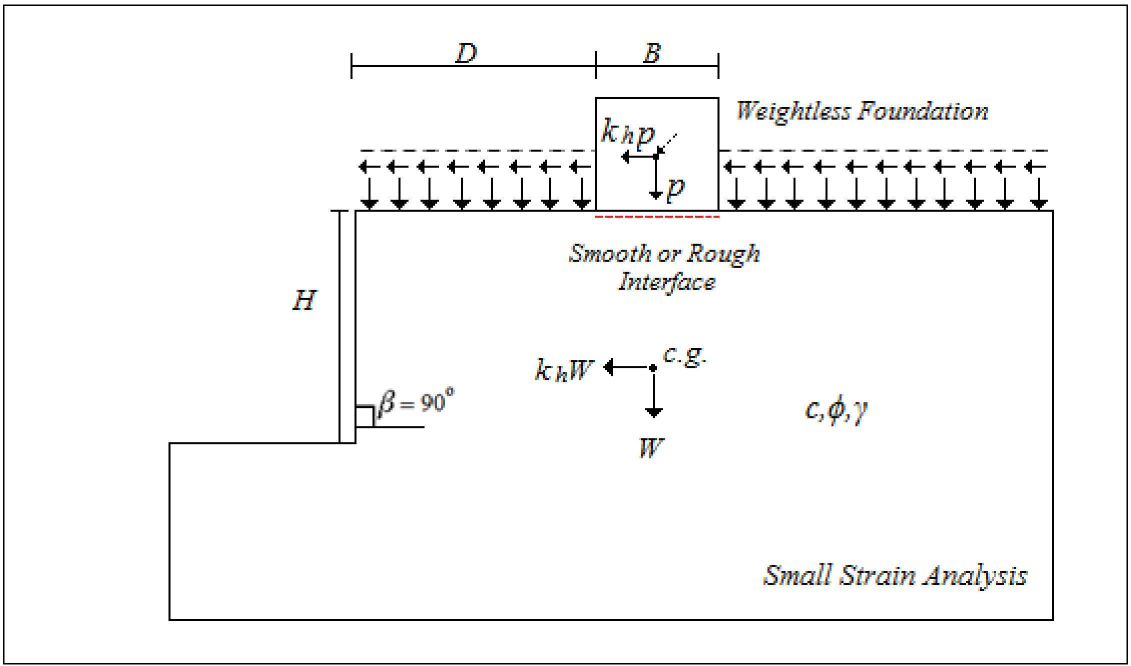

Figure 7-1. Problem notation for seismic bearing capacity of foundations located near slopes.

7.2 Previous Studies and Modelling Methods

Throughout the development of foundation within recent years there has been a number of studies conducted into seismic bearing capacities for foundation situated in seismic zones. The studies conducted involve a number of different numerical modelling methods to analysis the effects of seismic forces on the ultimate bearing capacity of the foundation. Some of these numerical modelling methods include; Upper Bound Lower Bound limits, limit equilibrium methods and the method of stress characteristics. These three modelling methods have been discussed briefly below along with the main findings from these evaluation methods.

7.2.1 Shiau et al., Sloan S and Lyamin A. (2006)

Shiau J, Sloan S and Lyamin A (2006) presented a study into seismic bearing capacities, based on the classic limit theorems of upper and lower bounds. Through the evaluation of the coefficients of horizontal and vertical acceleration it was concluded that the upper and lower bound limit results obtained from this study were adequate validation tools for any available method, modelling the same conditions, as the results obtained typically bracket the true solution for the foundation problem within $\pm$ 10%. Therefore this study provided a model to validate the seismic model presented within this chapter.

7.2.2 Kumar J & Kumar N (2003)

Kumar J & Kumar N (2003) presented a study into the foundation placed on horizontal ground surfaces, incorporating the effects of earthquake body forces, under limit equilibrium methods. The main conclusion presented within this study was the increases of the coefficient of horizontal acceleration reduced the overall bearing capacity factors. This study highlighted the need for a parametric study into different magnitudes of horizontal acceleration.

7.2.3 Kumar J & Mohan Rao, V.B.K. (2003)

Kumar J & Mohan Rao, V.B.K. (2003) presented a study into the analysis of the seismic bearing capacity of a foundation, using the method of stress characteristics. The main conclusions drawn from this study was again that increases in the magnitude of the coefficient of horizontal acceleration reduced the overall capacity of the foundation. It was also determined from this study that the magnitude of the bearing capacity factors decrease further with increases in ground inclination. This study highlighted the need for an investigation of a foundation located near a 90^{o} slope, subjected to seismic forces.

7.3 FLAC Model Development

The development of the seismic bearing capacity model for the shallow rigid foundation was developed in three distinct stages;

1. Firstly the model was simplified and an imaginary foundation condition was considered with inclined gravity applied within the soil structure.
2. Secondly a weightless foundation was introduced to the model and inclined gravity within the soil structure was applied.
3. Finally a horizontal initial velocity at the soil structure foundation interface was introduced to the weightless foundation model, in addition to the applied inclined gravity within the soil structure.

This method of model development was essential to ensure that each component was coded correctly and produced reasonable results that produced acceptable failure behaviour at ultimate bearing capacity. Discussed below in more detail are the three modelling steps followed in the development of the finial static pseudo seismic model.

7.3.1 First Step of Model Development

The first model developed within this chapter of seismic investigations was based on the simplified 'imaginary' foundation model produced by Lyle (2009). This model involved initially specifying a footing location and the fixity of the nodal elements that represent the footing, and then applying an initial velocity at this footing location. The footing location was then used to find the resistive forces at the "imaginary" foundation nodal points that were then divided by the area to find the average pressure. The average pressure was then normalised by dividing by the specific gravity of the soil structure multiplied by the width of the imaginary foundation, to produce the ultimate bearing capacity.

The seismic inclusions within this initial model involved applying the earthquake induced horizontal acceleration within the soil structure. This modelling within FLAC involved finding the resultant of the horizontal acceleration and vertical acceleration and applying it as the gravity within the soil structure at an angle of theta in which the resultant acts. The vertical acceleration was simply equated to gravity, 9.81 m/s, the horizontal acceleration on the other hand was calculated as gravity multiplied by a coefficient of horizontal acceleration, K_H. For the purpose of this dissertation four coefficients of horizontal acceleration were considered; 0.1, 0.2, 0.3 and 0.4. Table 7-1 presents the respected resultant gravities and the theta angles for each coefficient of horizontal acceleration that has been included within this initial seismic model.

Table 7-1. The applied gravities and applied angles for the seismic forces within the soil structure for the initial seismic model.

K_H	Horizontal Acceleration, $K_H W$	Resultant Gravity, W_R	Applied Angle Theta, θ
0.1	0.981m/s	9.86m/s	-5.739^o
0.2	1.962m/s	10.004m/s	-11.30^o
0.3	2.943m/s	10.24m/s	-16.7^o
0.4	3.924m/s	10.57m/s	-21.86^o

7.3.2 Second Step of Model Development

The second step of modelling the seismic forces was the inclusion of the building, but with weightless conditions, thus this involved applying the process as presented within section 7.2.1 to the advanced soil structure model previously presented within chapter 4 of this dissertation. Therefore the resultant gravity and angle of actions presented within Table 7-1 were then applied within the soil structure of this model. This step in modelling produced reduced capacities from the first step as the presence of the foundation element reduced the ultimate bearing capacity.

7.3.3 Third Step of Model Development

The third and final step was the modelling of the seismic forces within the weightless building model, but with the addition of an initial horizontal velocity within the foundation structure, as well as the inclined gravity within the soil structure. The inclusion of the initial horizontal velocity within the foundation structure was calculated with respect to the angle of gravity within the soil structure. It was assumed for the purpose of this study that the angle within the inclined loading in the soil structure would be equal to the inclined loading at the foundation level, thus simple trigonometry was then used to determine the vertical and horizontal velocities from an assumed inclined velocity and the angle of theta with respect to the magnitude of earthquake being investigated.

The final analysis results presented by FLAC for this model were thus presented within normalised loads for the x and y direction loading, thus from this step the resultant was calculated within excel to determine the overall combined inclined ultimate bearing capacity produced by this model.

7.4 Model Validation

To ensure that the results produced by the seismic bearing capacity model were accurate a validation was conducted with previous published paper produced by Lyamin, Sloan and Shiau et al. (2006). Within this paper a study into the use of the classic limit theorems of upper and lower bound numerical modelling to evaluate the seismic bearing capacity of footings located near slopes was conducted. The selection of this paper for the validation of the explicit finite difference model presented within this chapter, was based on the relevance to the problem of the rigid shallow foundation located near a slope and the proven accuracy of the results presented within the paper.

To ensure consistency between the finite difference model and the validating upper and lower bound limits model the soil strength ratio, friction angle, slope angle and footing distance ratio were equated within each model. The values used for these parameters were;

$c/\gamma B = 1$ soil strength ratio.
$\phi = 40°$ friction angle.
$\beta = 90°$ slope angle.
$D/B = 0$ footing distance ratio.

Presented within Table 7-1 are the seismic bearing capacities produced by the upper and lower bound limits and explicit finite difference modelling of the rigid shallow foundation located near a slope, for a range of earthquake magnitudes (variance in k_h). The explicit finite difference modelling has incorporated both a smooth and rough soil structure foundation interface, to evaluate which modelling method is more accurate, with respect to the upper bound limits model.

Table 7-2. The validation of model with upper bound limits results.

K_h	Ultimate Seismic Bearing Capacity			Percentage Difference Smooth (%)	Percentage Difference Rough (%)
	Upper Bound Limits	Explicit Finite Difference			
		Smooth Interface	Rough Interface		
0.0	3.28	2.678	2.526	18.25	22.99
0.1	2.58	2.609	2.517	1.11	2.44
0.2	2.03	2.426	2.052	16.323	1.07
0.3	1.6	2.254	1.748	25.9	8.47
0.4	1.19	2.112	1.526	43.66	22.02

It is evident within Table 7.1 that the rough interface condition between the foundation and soil structure produced seismic bearing capacities closer to those of the upper bound limit state, the average percentage difference for the rough interface explicit finite difference model and the upper bound limits model was calculated as 11.40%. Whereas the average percentage difference between the smooth interface explicit finite difference model and the upper bound limits model was calculated as 21.05%.

Figure 7-2, presents the results presented within Table 7-1 graphically for ease of comparison.

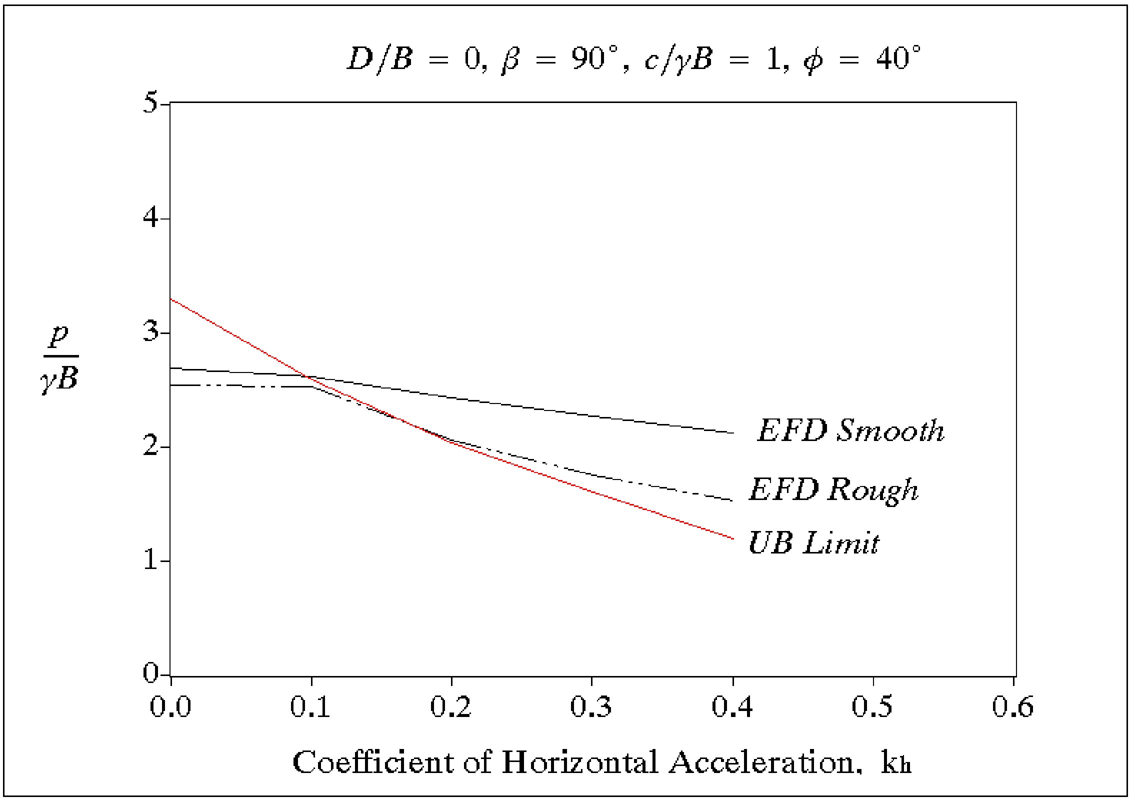

Figure 7-2. Validation of Seismic Bearing Capacity Model.

From visual inspection it can be seen that the trends in the upper bound limit modelling and the FLAC modelling are slightly different. This difference has been considered and from analysis it has been concluded that the final results obtained from the FLAC model would be of suitable accuracy. As for the difference within the smooth soil structure foundation interface or a rough soil structure foundation interface FLAC models, the model that resembles the upper and lower bound limit states was adopted as the modelling method throughout the parametric study presented within this chapter.

Therefore from the validation results presented above it was concluded that a rough interface between the soil structure and foundation base would be adopted for the modelling and analysis of the seismic bearing capacity problem for a shallow rigid foundation located near a slope, as it produced results closer to the upper bound limits model. The rough interface modelling method was also selected as the modelling method as this interface condition proved to produce more conservative results for the ultimate bearing capacity, than the smooth interface case.

7.5 Parametric Study

Presented within this section of the chapter is a comprehensive parametric study of a number of the major geometrical and material factors that would affect the ultimate bearing capacity of the static pseudo seismic model, presented within section 7.2.3 and validated within section 7.3. This study is important as an understanding of these parameters is essential in the design shallow foundations that are situated near a slope within seismic areas. For the purpose of this study only a 90^{o} slope was considered due to the project scope and as previously mentioned the soil structure interface condition that was be modelled was rough due to the validation results presented within section 7.3.

7.5.1 Effect of D/B Ratio

The D/B ratio represents the relationship between the foundation width and the distance from the slope edge. It was established from the studies conducted in previous chapters that as the D/B ratio is increased the stability of the foundation increased and the failure mechanism eventually resulted as flat ground failure at significant distances from the slope. The actual distance required to induce flat ground failure is dependent on both the steepness of the slope and the strength of the soil, thus for the purpose of this study the worst case scenario for slope angle has been considered with the slope angle being applied as 90°. Discussed within this section is an analysis of the effect that changes in D/B ratio has on the ultimate bearing capacity when horizontal earthquake induced forces are modelled. For the purpose of this parameter study the H/B and soil strength ratios have been modelled as 3 and 5, respectively. Further investigations into the optimum values for these parameters will be investigated within later sections of this parametric study.

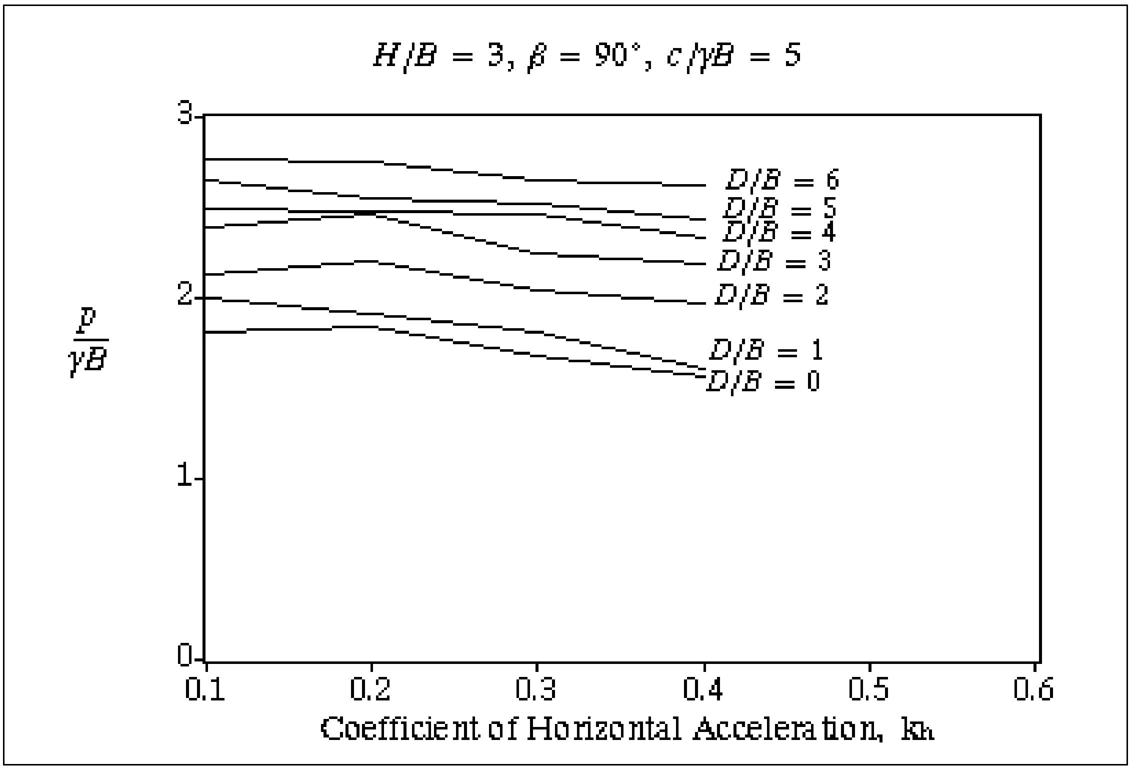

Figure 7-3. Change in normalised bearing capacity with horizontal coefficient of acceleration.

Figure 7-3 presents the change in inclined normalised bearing capacity with horizontal coefficient of acceleration for a range of different D/B ratios. The main trend evident within this graph was the increase in combined normalised bearing capacity with increased D/B ratio. This result is expected as there is a certain degree of stability increase with greater distances from the slope, regardless of whether it is an earthquake event. It was also noted from the graph that the as the coefficient of horizontal acceleration increased the normalised bearing capacity reduced, regardless of the D/B ratio. This result indicates that the D/B ratio and the coefficient of horizontal acceleration are independent of parameters and that any increase in horizontal acceleration will reduce the capacity.

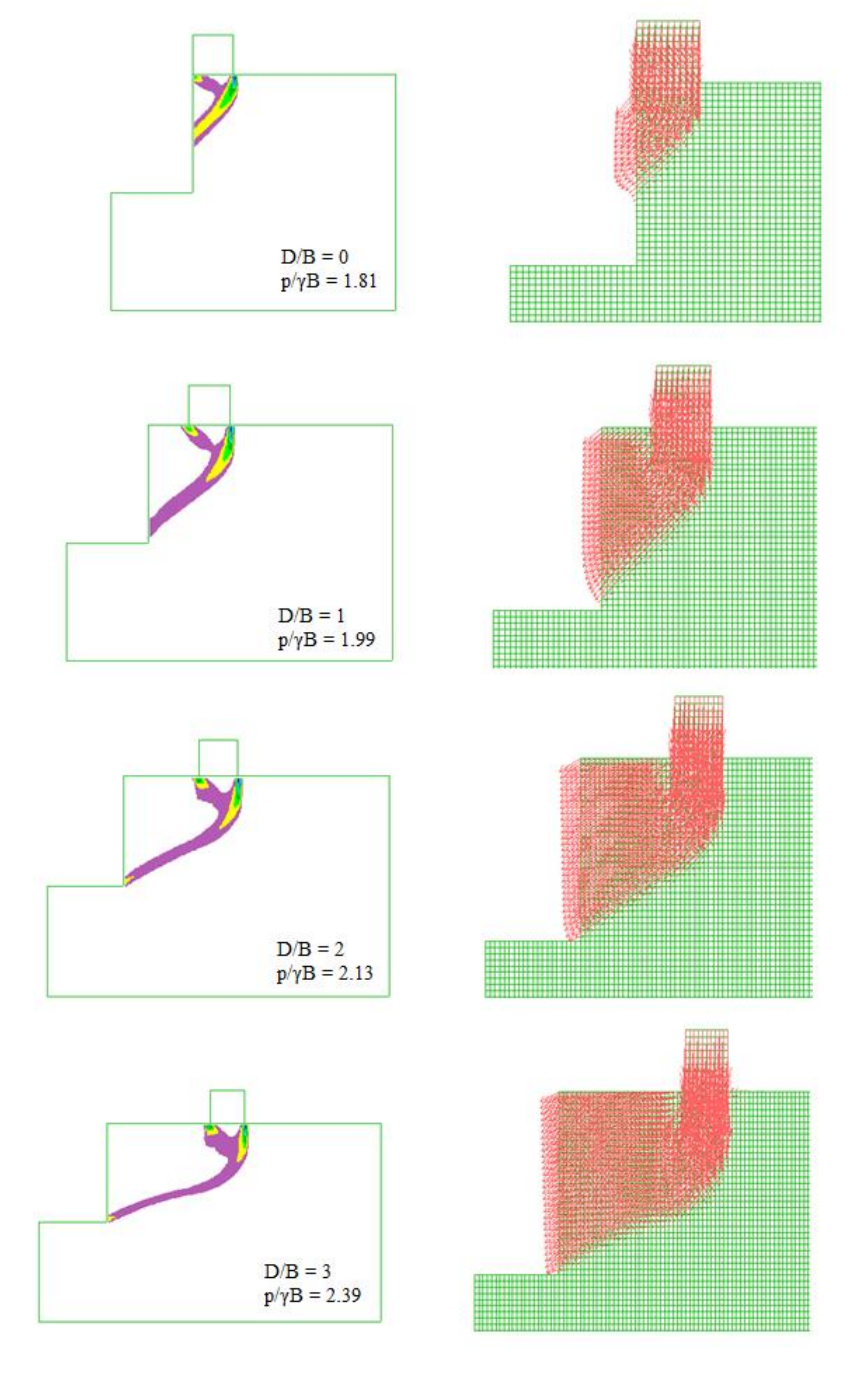

Figure 7-4 The change in inclined normalised bearing capacity with D/B ratio for Kh=0.1.

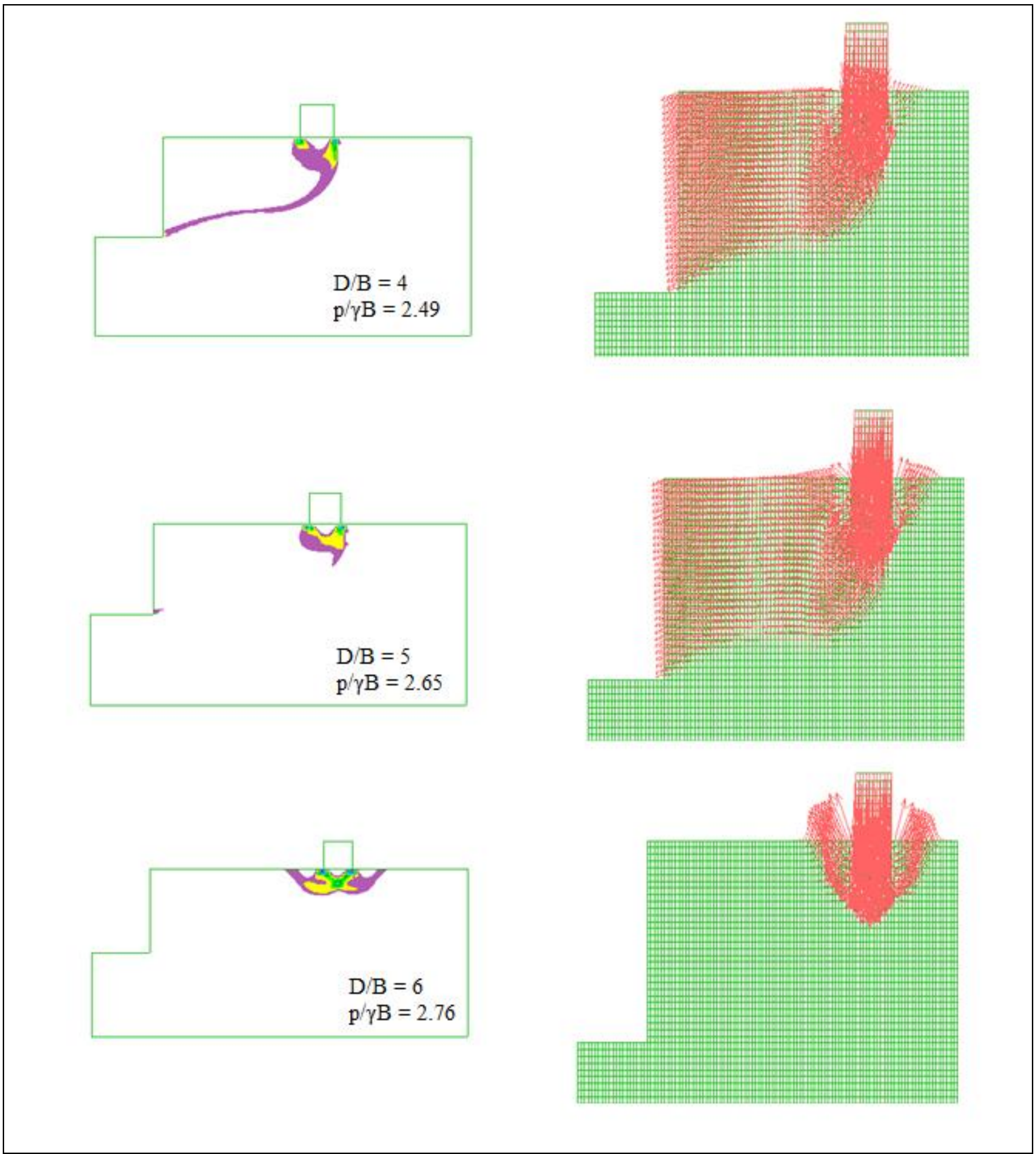

Figure 7-5. The change in inclined normalised bearing capacity with D/B ratio for Kh=0.1 (continued).

Figure 7-4 and 7-5 presents the change in inclined normalised bearing capacity with D/B ratio, in the form of shear strain rate and velocity vector plots, for a coefficient of horizontal acceleration equal to 0.1. The obvious trend within these plots was the increase in the proportion of velocity fields that occur as the D/B ratio is increased and the foundation is moved further from the slope surface. This result is due to the increased slip surface that comes with increased D/B ratios. Another observation was the failure mechanism that occurs. At a D/B ratios of 0 and 1 the failure mechanism occurring was local shear failure above the toe, but as the D/B ratio increased to 2 the failure mechanism changed to local shear failure about the toe. At a D/B ratio of 6, the failure mechanism induced was general shear failure or flat ground failure (symmetrical shear strain plot). At this point it is expected that the bearing capacity will remain constant.

Figures 7-6 and 7-7 presents the change in inclined normalised bearing capacity with D/B ratio, in the form of shear strain rate and velocity vector plots, for a coefficient of horizontal acceleration equal to 0.2. The obvious change between this case and the previous earthquake magnitude case was the increased presence of the local shear failure at the toe of the slope. Within the previous earthquake magnitude results failure due to the slope stopped at a D/B ratio of 4, whereas for the earthquake magnitude of 0.2w the failure due to the slope was still present at a D/B ratio of 5. This result coincides with the reduction in the capacities produced with a 0.2w magnitude consideration. Therefore the main conclusion drawn from this set of results is as the earthquake magnitude is increased the presence of the slope affects the ultimate bearing capacity for greater distances from the slope edge.

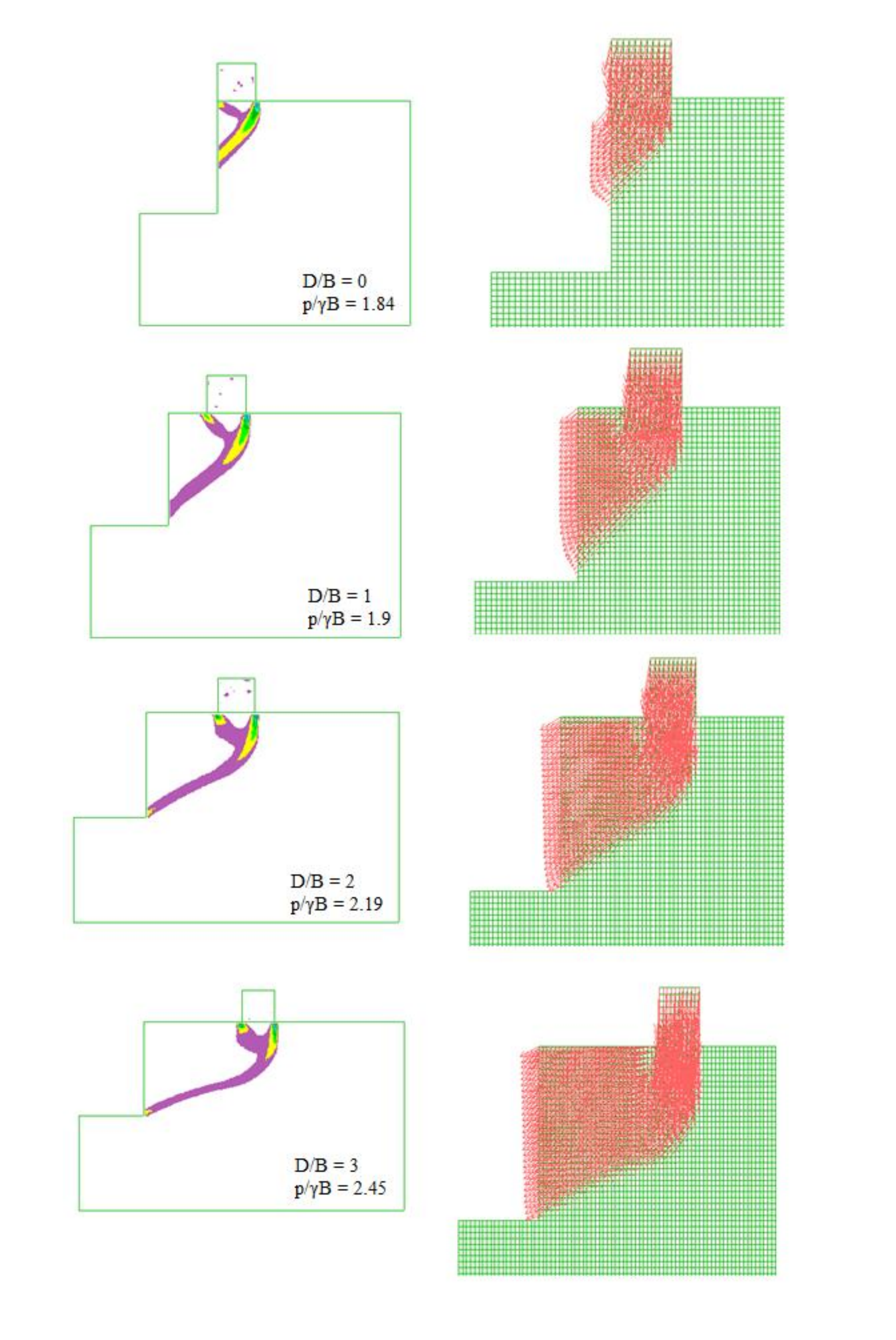

Figure 7-6 The change in inclined normalised bearing capacity with D/B ratio for Kh=0.2.

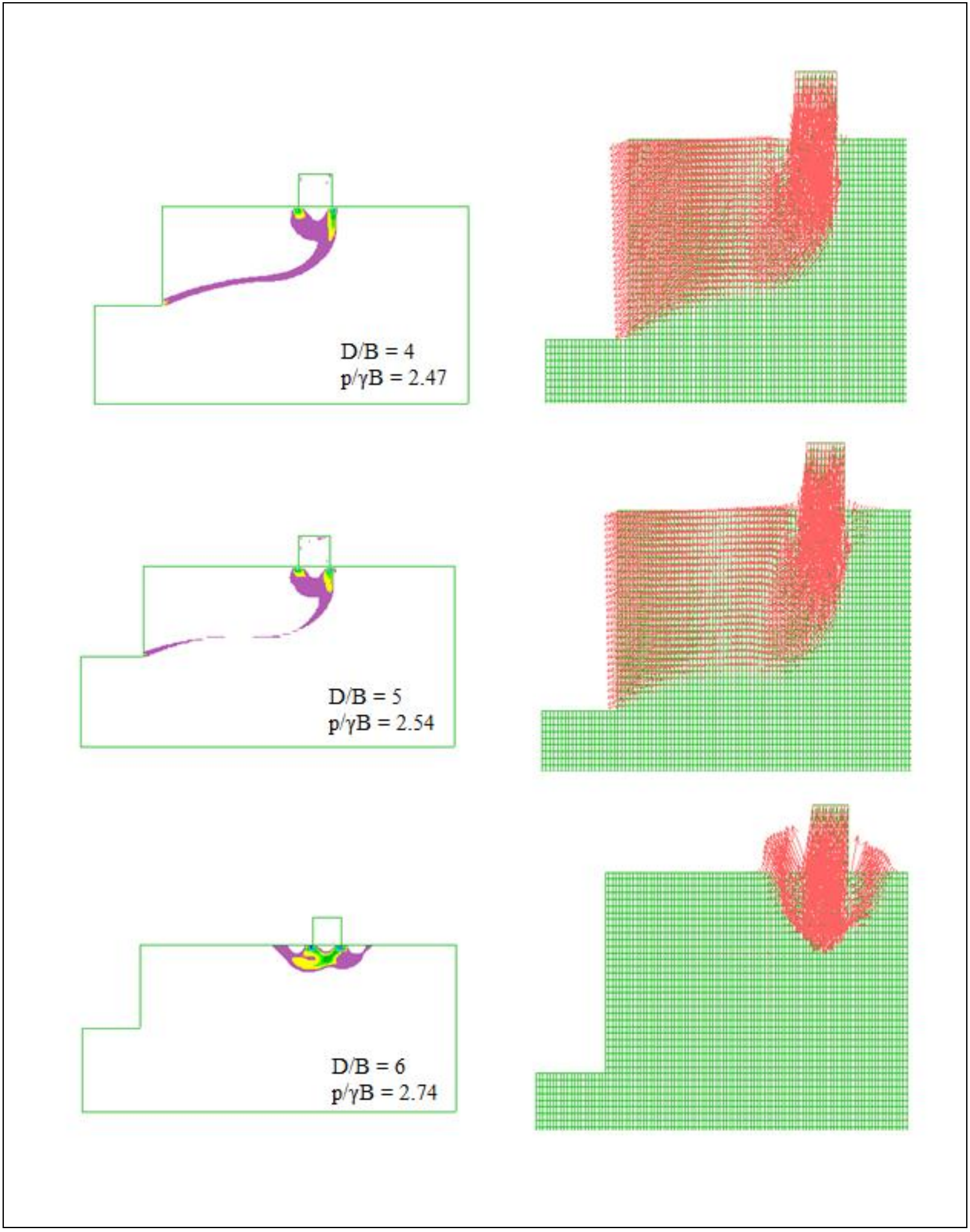

Figure 7-7. The change in inclined normalised bearing capacity with D/B ratio for Kh=0.2 (continued).

Figures 7-8 and 7-9 present the change in inclined normalised bearing capacity with D/B ratio, in the form of shear strain rate and velocity vector plots, for a coefficient of horizontal acceleration equal to 0.3. Again the obvious trend was the increased effect of the slope presence at greater distances from the slope edge, with increased earthquake magnitude. This result is evident through the more dominant slip surface that is shown within the shear strain rate plots for a D/B ratio of 5 and the unsymmetrical results that occurred for a D/B ratio of 6. Previously within smaller magnitude earthquake magnitudes the failure mechanism at D/B 6 was symmetrical, thus flat ground failure. Therefore the main conclusion drawn within the previous results is again drawn within these results. As the earthquake magnitude is increased the presence of the slope has a prolonged affect on the ultimate bearing capacity with greater distances from the slope edge, thus resulting in a reduction in the overall foundation capacity.

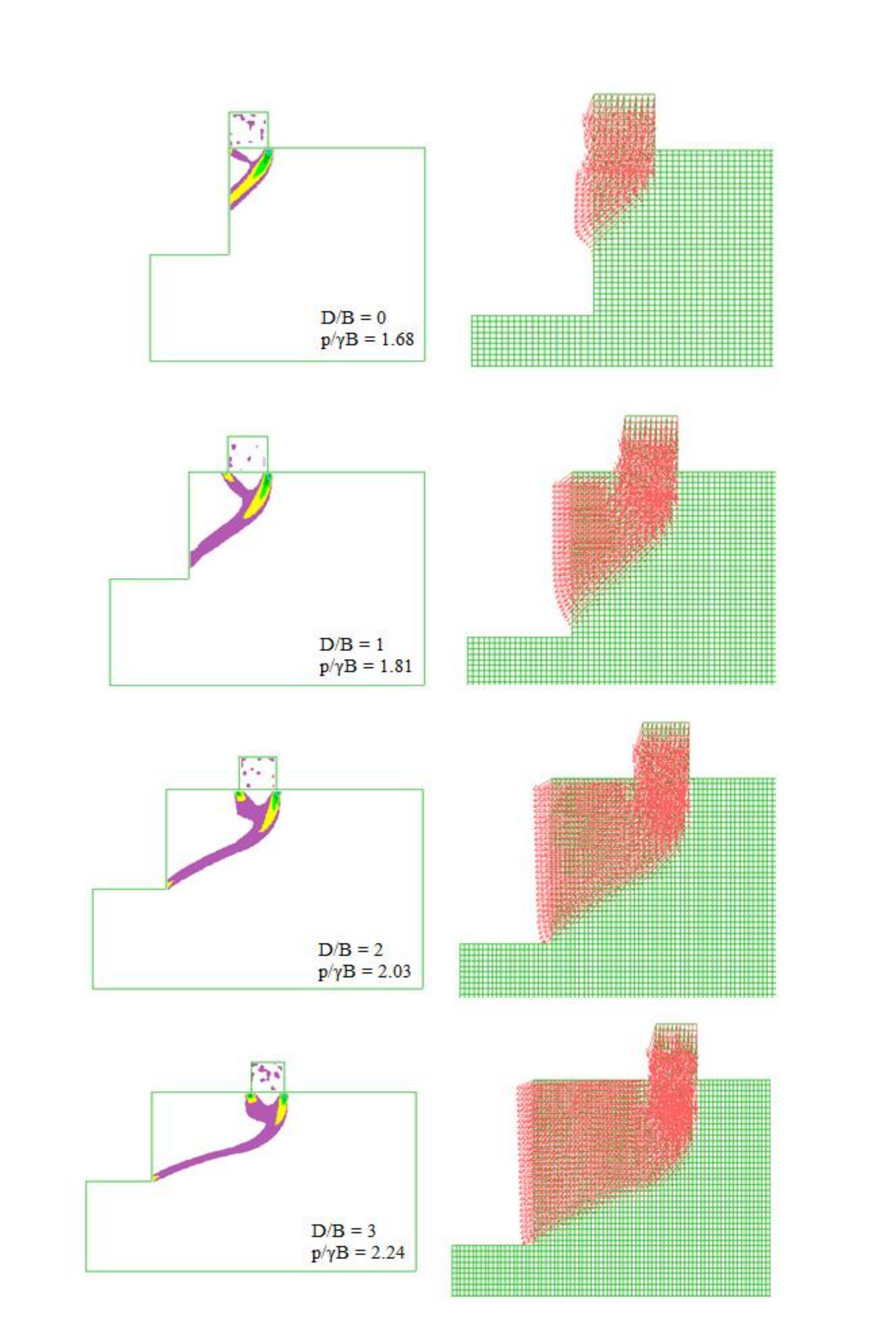

Figure 7-8. The change in inclined normalised bearing capacity with D/B ratio for Kh=0.3

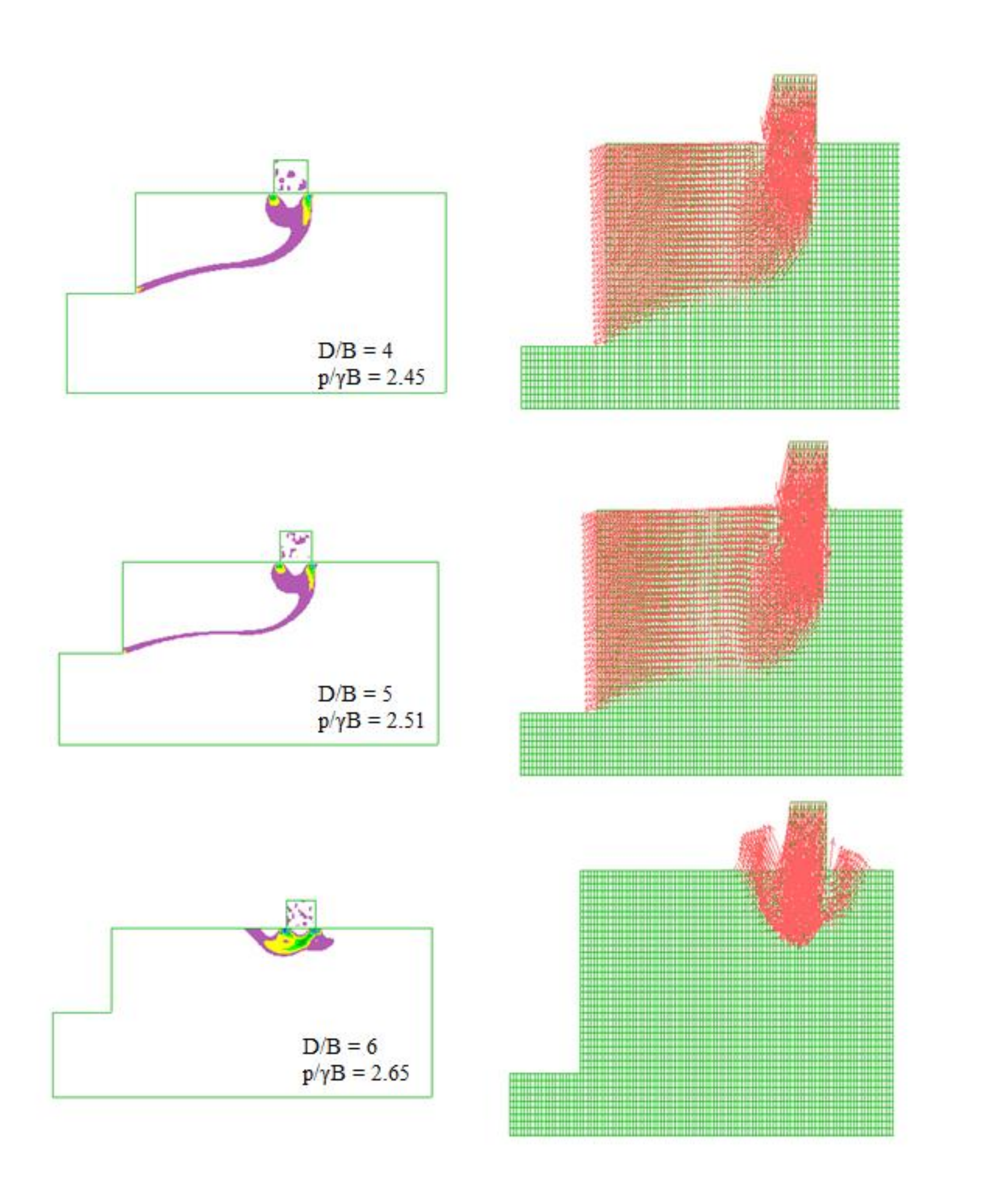

Figure 7-9. The change in inclined normalised bearing capacity with D/B ratio for Kh=0.3 (continued).

Figures 7-10 and 7-11 present the change in inclined normalised bearing capacity with D/B ratio, in the form of shear strain rate and velocity vector plots, for a coefficient of horizontal acceleration equal to 0.4. These results further develop the other findings presented within this parameter study, with respect to increased affects of the presence of the slope with increased earthquake magnitude. This conclusion can be seen even clear within Figure 7-11 for a D/B ratio of 6. At this distance a clear slip surface is starting to be created towards the slope. Although this slip surface is still at ground level at further increased earthquake magnitudes it could be assumed that this slip surface would eventually reach the slope surface.

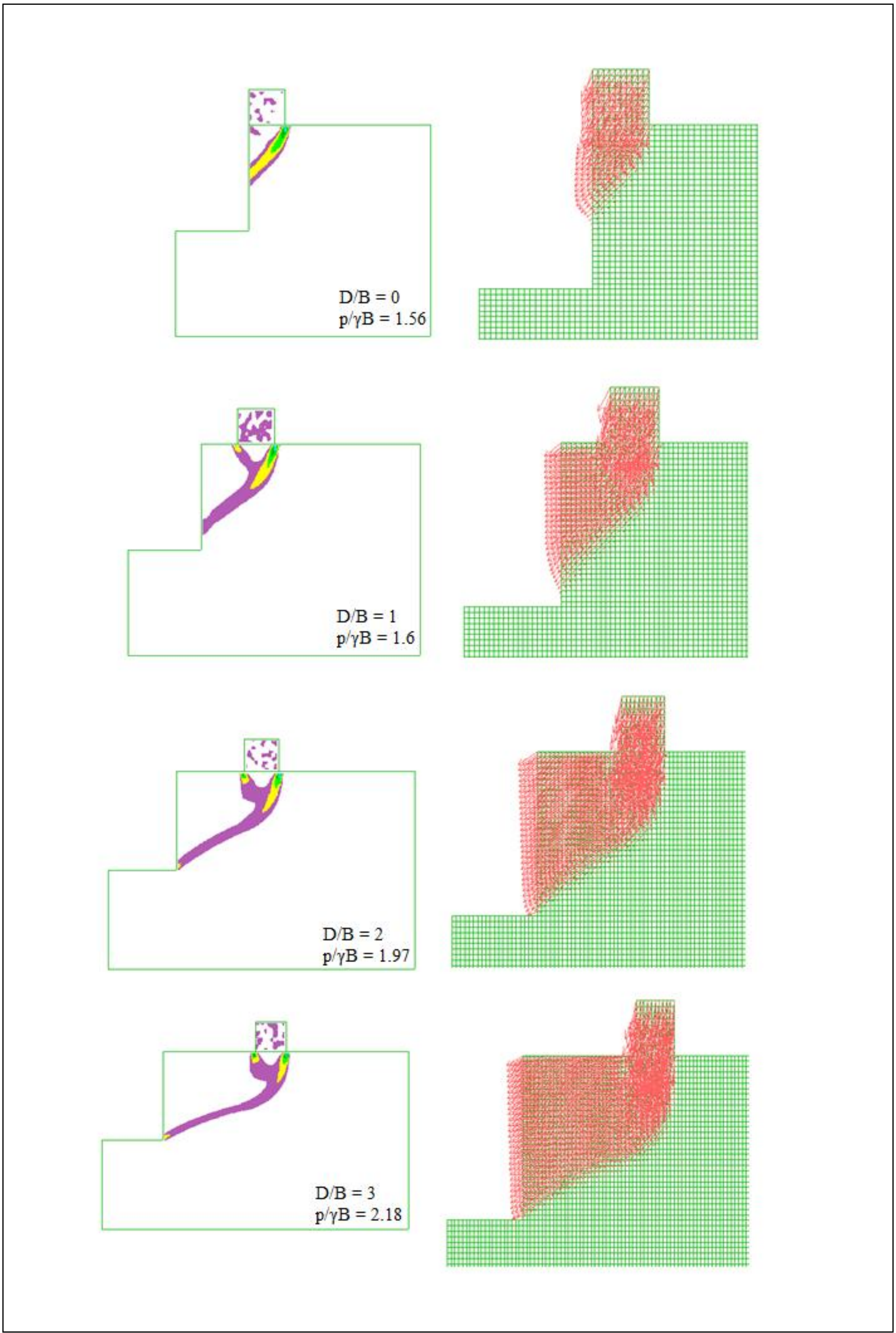

Figure 7-10. The change in inclined normalised bearing capacity with D/B ratio for Kh=0.4 (continued).

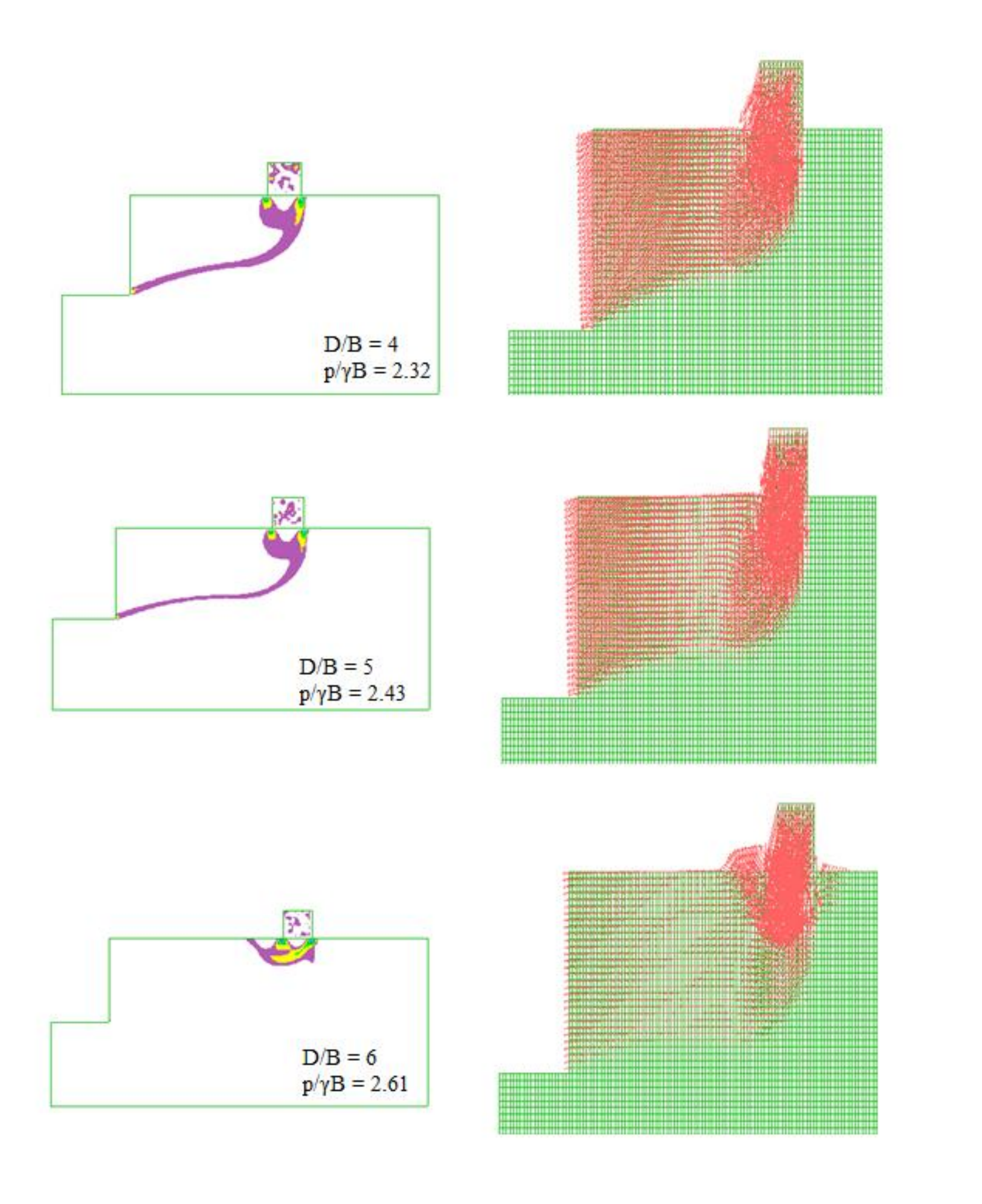

Figure 7-11. The change in inclined normalised bearing capacity with D/B ratio for Kh=0.4 (continued).

7.5.2 D/B Ratio Conclusions

From this parameter study on the effects of footing distance ratio D/B it can be seen that the positioning of a foundation has a considerable effect on the ultimate bearing capacity of a foundation. But when earthquake induced horizontal accelerations are taken into consideration the effect of increasing the D/B ratio to increase the bearing capacity is reduced, as the presence of the slope affects the failure mechanism for greater D/B ratios. Therefore it can be concluded that earthquakes reduce the overall capacity of foundations and as the magnitude of the earthquake is increased the presence of the slope affects the ultimate bearing capacity for greater distances from the slope edge.

7.5.3 Effect of H/B Ratio

The H/B ratio represents the relationship between the height of the slope and the foundation width. This ratio is an important parameter as it can alter the failure mechanism that are induced by the slope and therefore change the inclined ultimate bearing capacity of the slope. The failure mechanisms that occur with varied H/B ratio are; below the toe failure and above the toe failure. As the slope height is increased the transition from below to above toe failure is induced. From investigations of past studies it has been a general trend that as the H/B ratio is increased the ultimate hearing capacity is reduced until above toe failure is reached, where at this point equilibrium within the capacity is reached and the value stays constant. Discussed within this section is an analysis of the effect of H/B ratio on the inclined seismic bearing capacity, for a range of different earthquake magnitudes. For the purpose of this study a D/B ratio of 2 was adopted and a soil strength ratio of 5 was used.

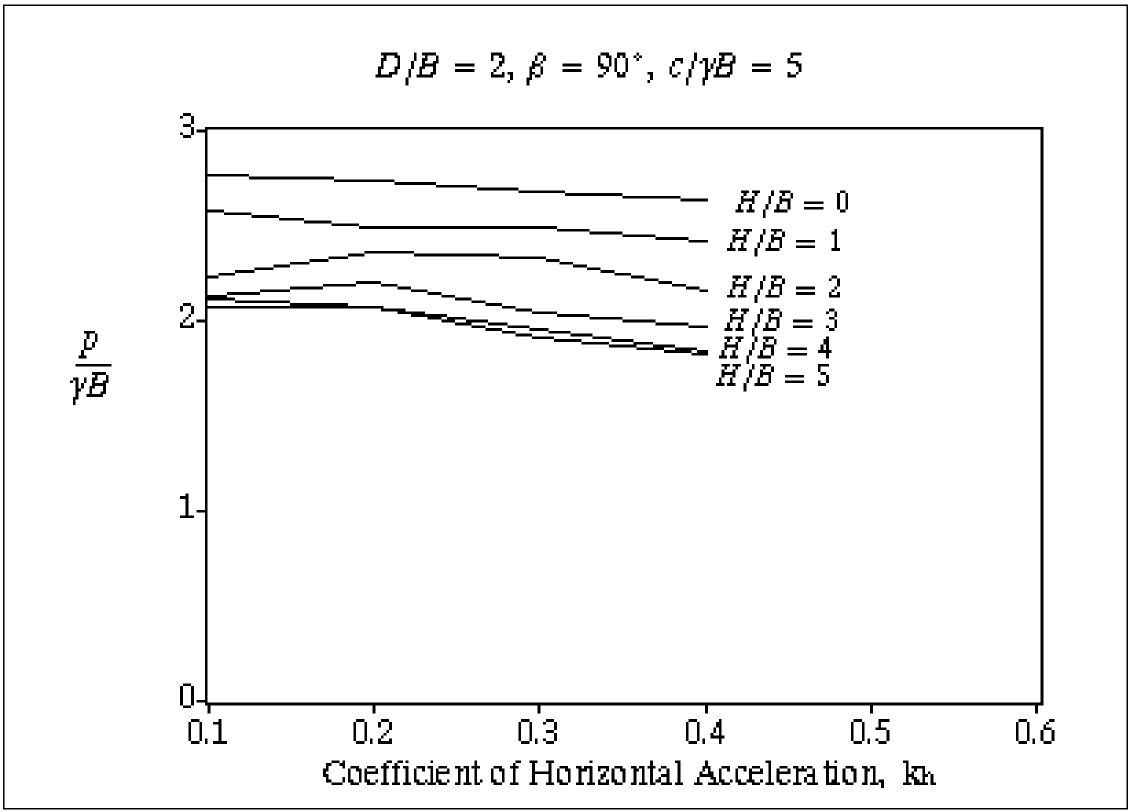

Figure 7-12. The change in inclined normalised bearing capacity with coefficient of horizontal acceleration.

Presented within Figure 7-12 is the change of inclined normalised bearing capacity with varied coefficient of horizontal accelerations, for a range of H/B rations. The main trend observed from this graph is the reduction of the inclined normalised bearing capacity, with increased H/B ratio. This reduction is due to the foundation transitioning from flat ground failure to local shear failure due to the slope, as the slope height is increased. Another observation is again the reduction of the capacity with increased coefficient of horizontal acceleration. Thus indicating that the two parameters H/B ratio and the coefficient of horizontal acceleration are independent of each other. Another note worthy observation is the gradual convergence of capacities at H/B ratios of 4 and 5. This result indicates that the transition from at toe to above toe failure is occurring within the model. But overall the capacities are still reducing with increased H/B ratio indicating that full above the toe failure is yet to be induced within the model.

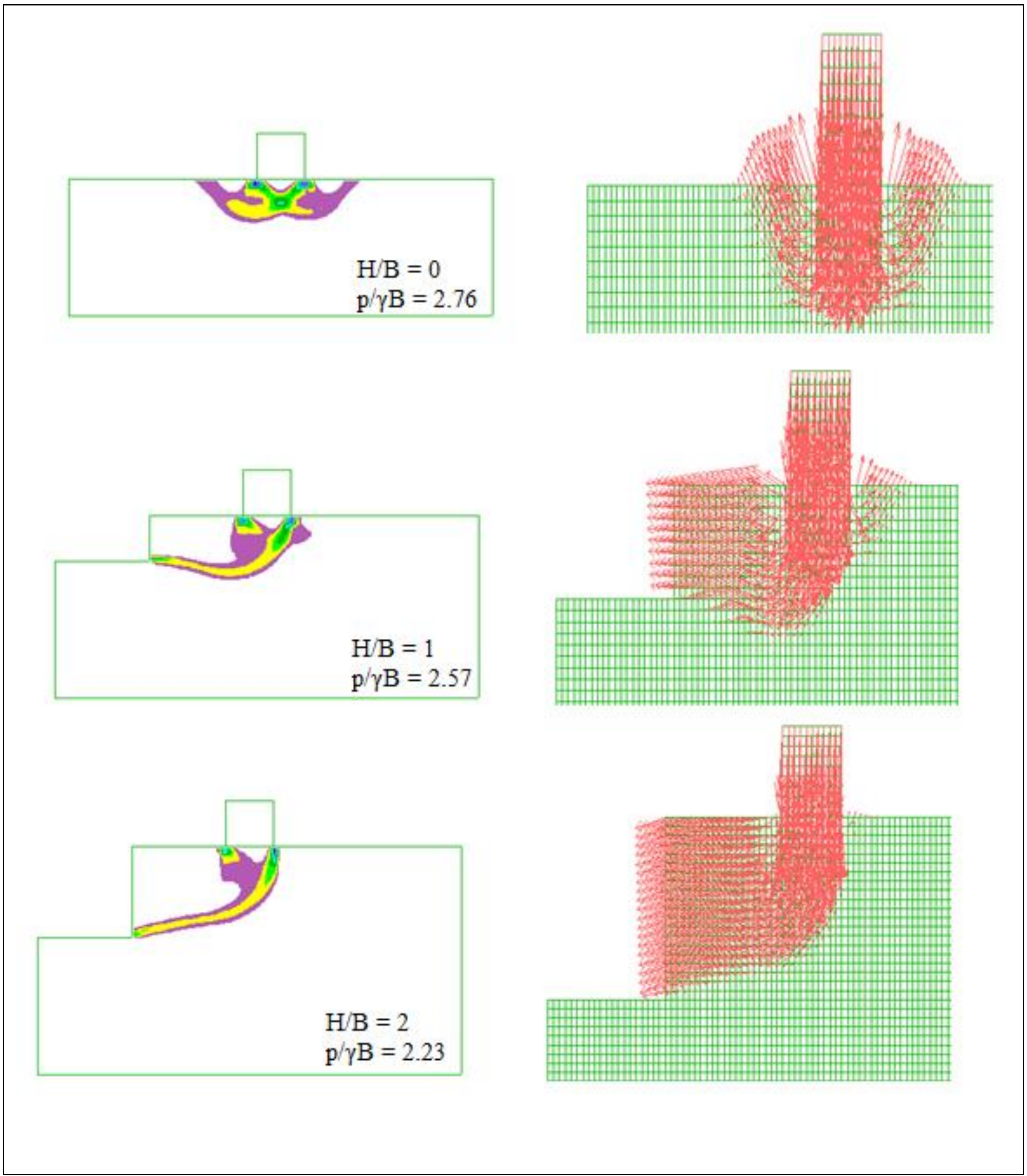

Figure7-13. The change in inclined normalised bearing capacity with H/B ratio for Kh=0.1

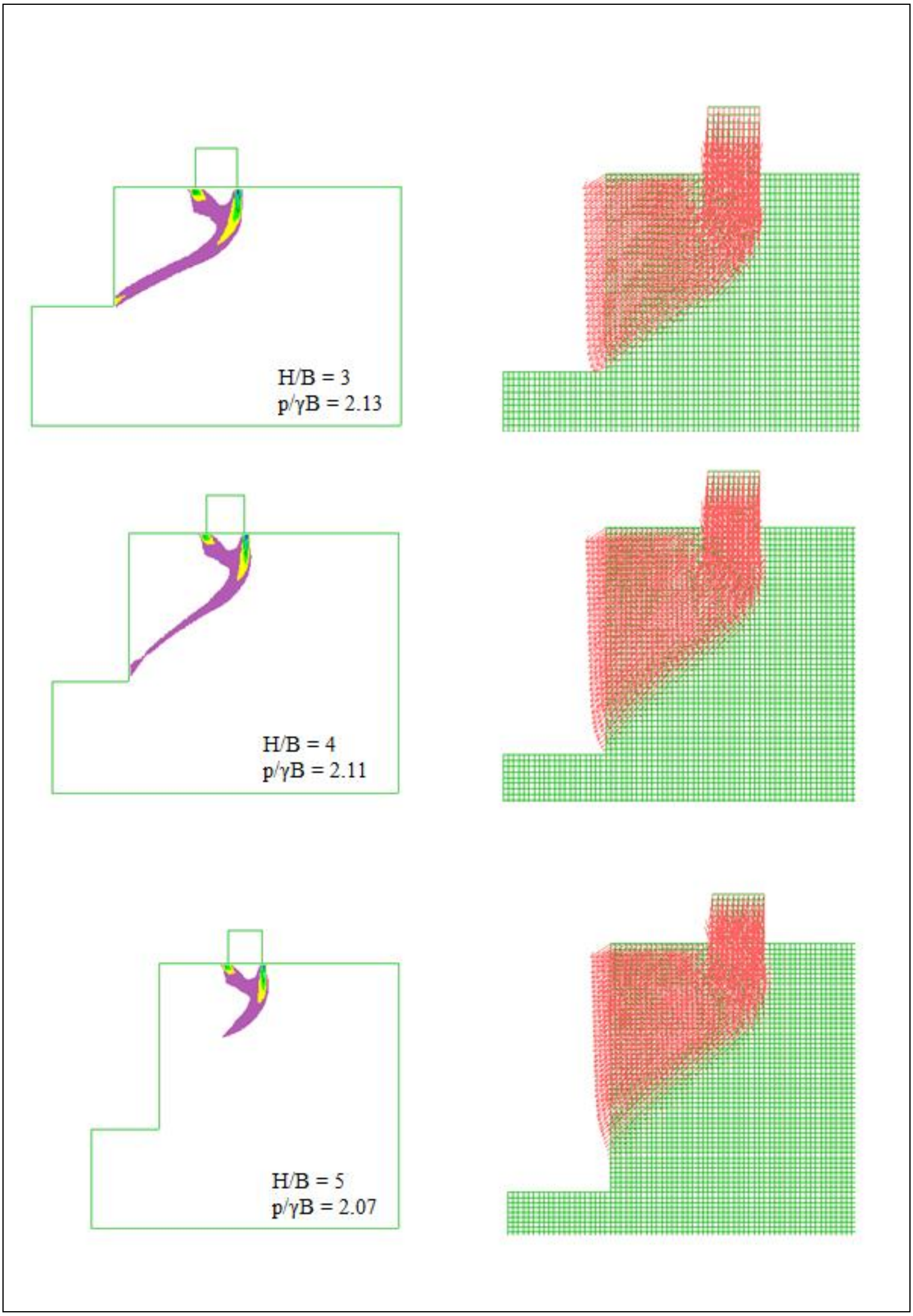

Figure 7-14. The change in inclined normalised bearing capacity with H/B ratio for Kh=0.1 (continued)

Presented within Figures 7-13 and 7-14 are the shear strain rate and velocity vector plots for change in inclined normalised bearing capacity with H/B ratio, for a coefficient of horizontal acceleration 0.1. The most evident observation from these plots was the transition from flat ground failure (symmetrical) to local shear failure (unsymmetrical) that occurred between H/B ratios of 0 and 1. After an H/B ratio of 1 the capacity of the foundation gradual reduced with increased slope height. The failure mechanism at H/B ratios between 1 and 3 where all at toe failures, while at an H/B ratio of 4 the failure was above the toe and at an H/B ratio of 5 the slip surface was no longer evident at the slope surface. These results imply that as the H/B ratio is increased less uplifting forces are present at ground surface and the presence of the horizontal acceleration increases the required H/B ratio to achieve above the toe failure.

The change in direction of the velocity vectors presented within Figures 7-13 and 7-14 indicate that the slope is transitioning from flat ground failure to local shear failure. As the H/B ratio increases the evidence of the transitional phase of at toe to above toe failure can be clearly seen between H/B ratios 3 and 4 within the velocity vector plots.

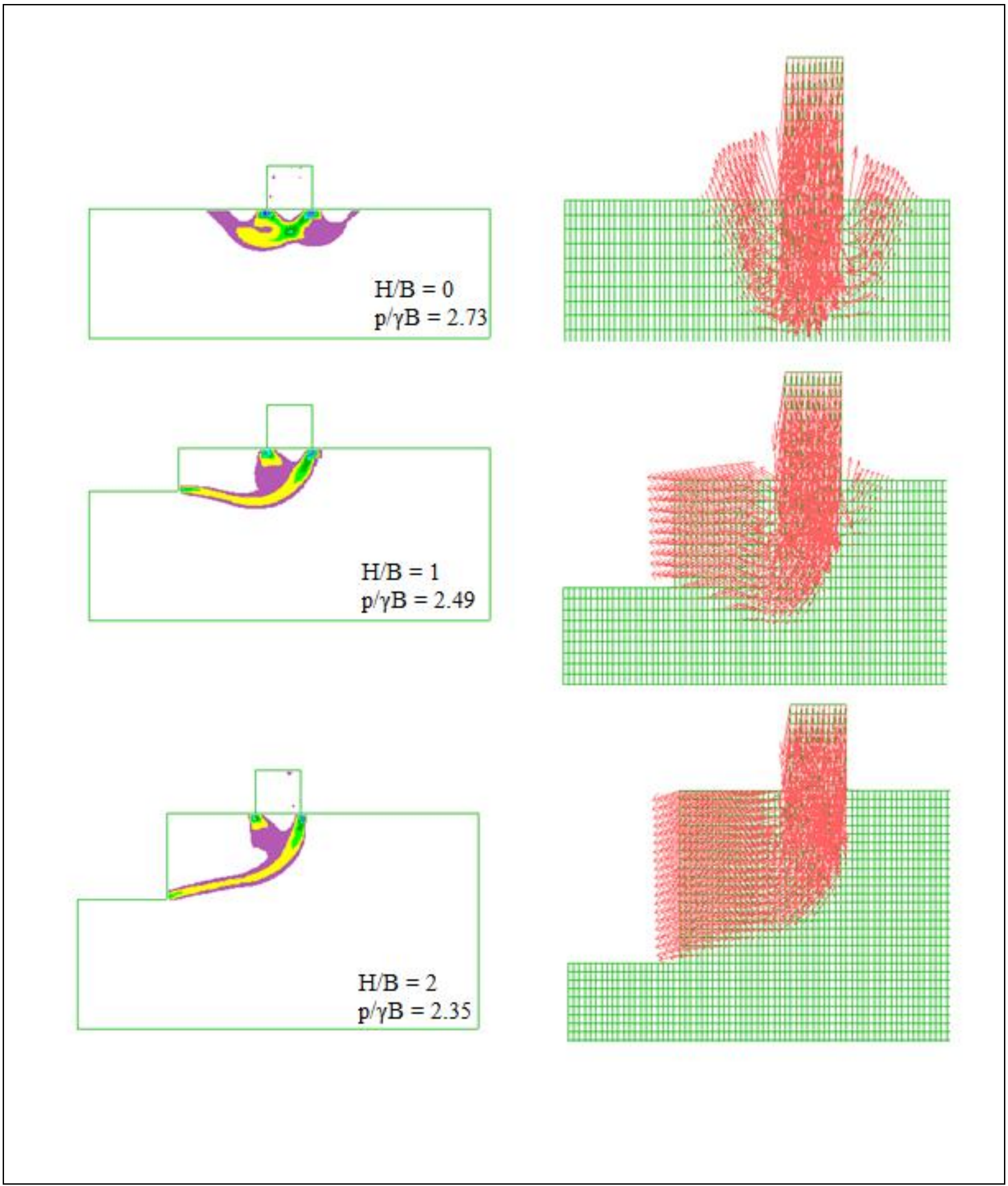

Figure7-15. The change in inclined normalised bearing capacity with H/B ratio for Kh=0.2.

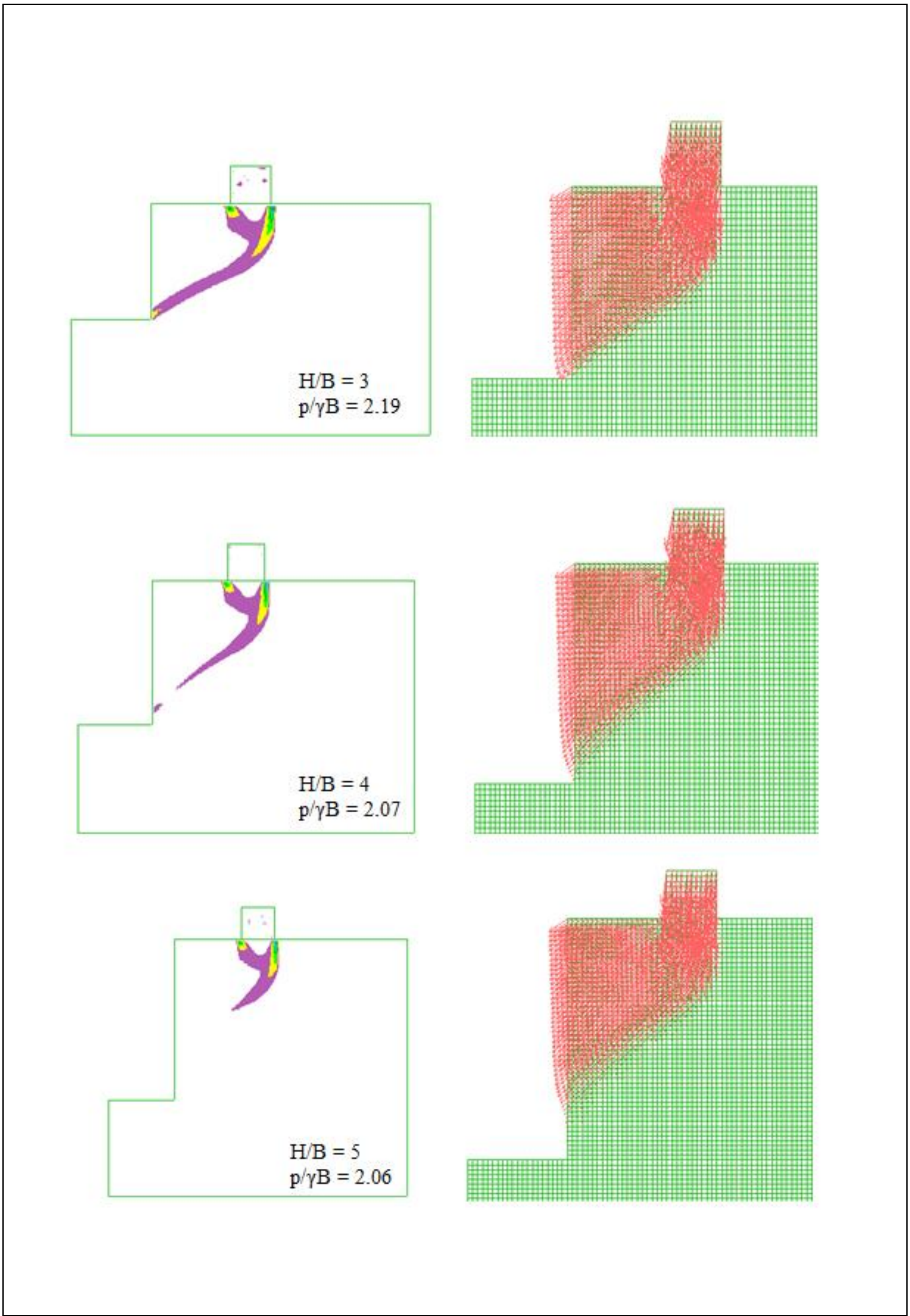

Figure 7-16. The change in inclined normalised bearing capacity with H/B ratio for Kh=0.2 (continued)

Presented within Figures 7-15 and 7-16 are the shear strain rate and velocity vector plots for change in inclined normalised bearing capacity with H/B ratio, for a coefficient of horizontal acceleration 0.2. The most evident observation from these plots was again the transition from flat ground failure (symmetrical) to local shear failure (unsymmetrical) that occurred between H/B ratios of 0 and 1. The transition from at toe failure to above toe failure again occurred at a H/B ratio of 4 thus indicating that the increase of the coefficient of horizontal acceleration from 0.1 to 0.2 had minimal effect on the failure mechanism however it did slightly reduce the inclined normalised bearing capacity. The velocity vectors showed minimal changes between the different earthquake magnitudes.

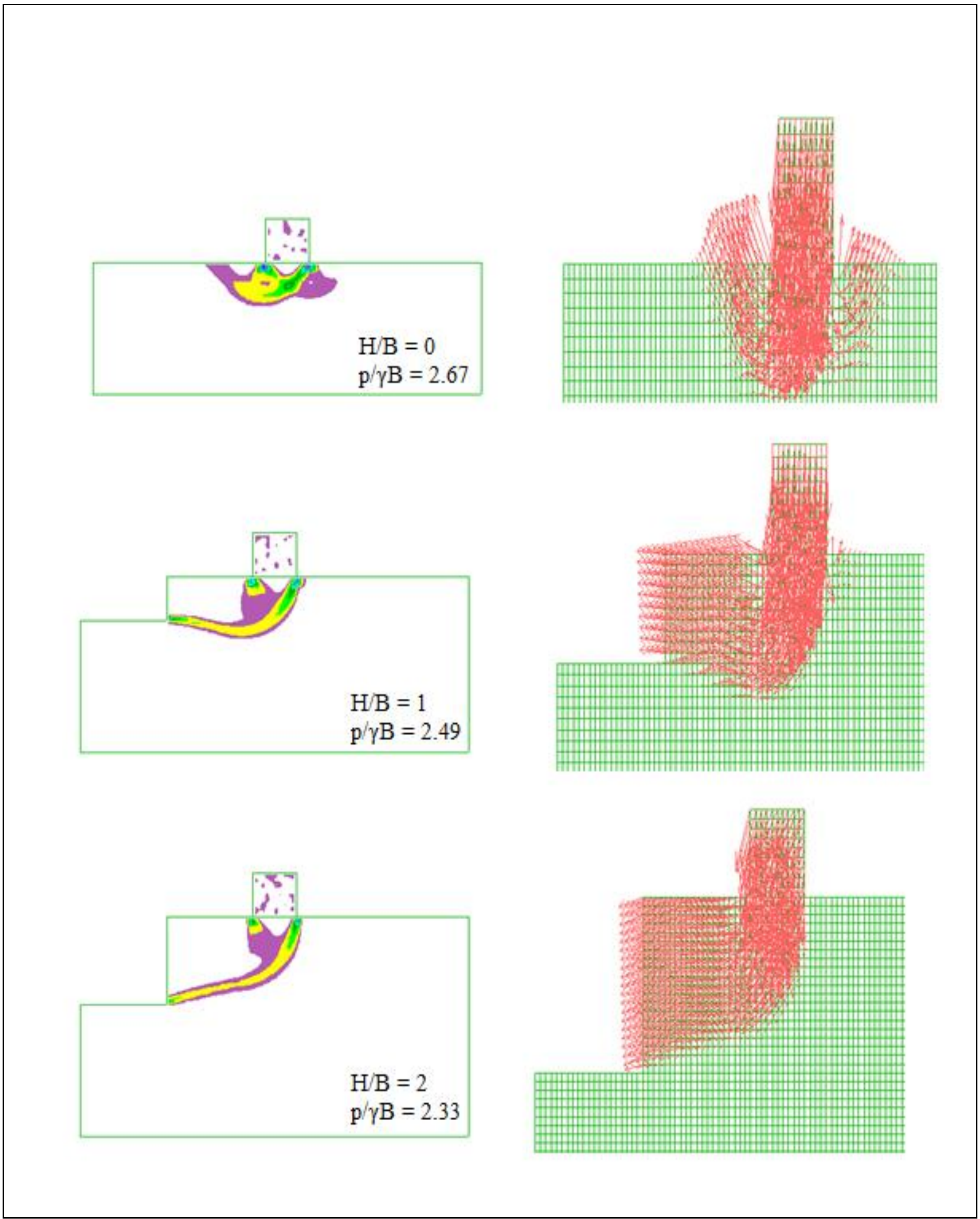

Figure 7-17. The change in inclined normalised bearing capacity with H/B ratio for Kh=0.3.

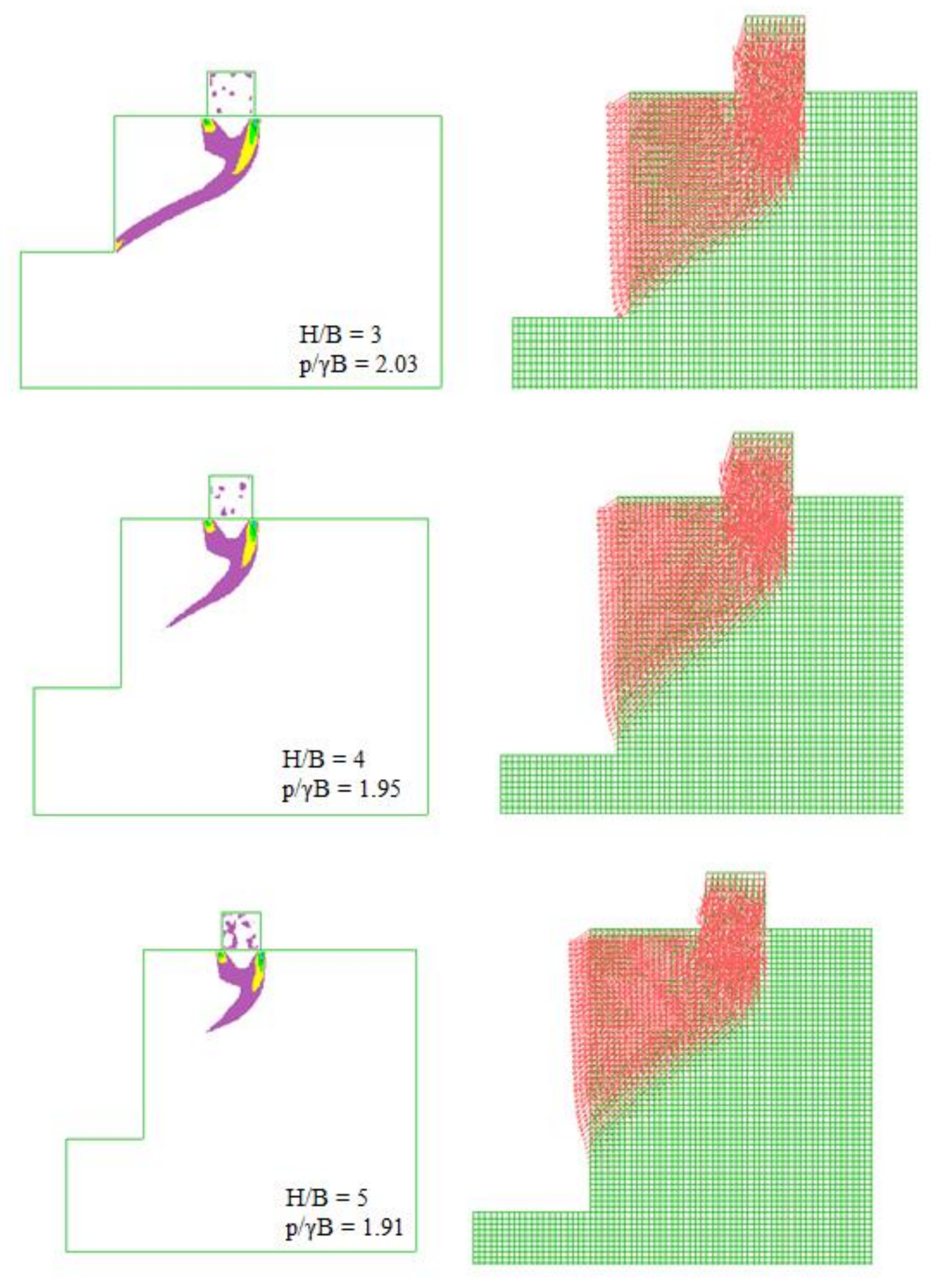

Figure 7-18. The change in inclined normalised bearing capacity with H/B ratio for Kh=0.3 (continued)

Presented within Figures 7-17 and 7-18 are the shear strain rate and velocity vector plots for change in inclined normalised bearing capacity with H/B ratio, for a coefficient of horizontal acceleration 0.3. A noticeable difference between the 0.3 magnitude earthquake and the 0.2 earthquake was the failure slope at a H/B ratio of 4. Previously at this slope height the failure mechanism transitioned from at toe failure to above toe failure, but for an earthquake magnitude of 0.3 the slip surface is no longer evident at the slope surface. This indicates with the presence of increased slope height the effect of the slip surface is reduced. This occurrence has clearly been depicted within the velocity vectors for H/B ratios of 4 and 5. Between these two slope heights the proportion of velocity fields tends to decrease, thus resulting in a reduction of inclined normalised bearing capacity. Therefore with increased acceleration of 0.2w to 0.3w a variation of failure mechanism is present resulting in a reduced bearing capacity.

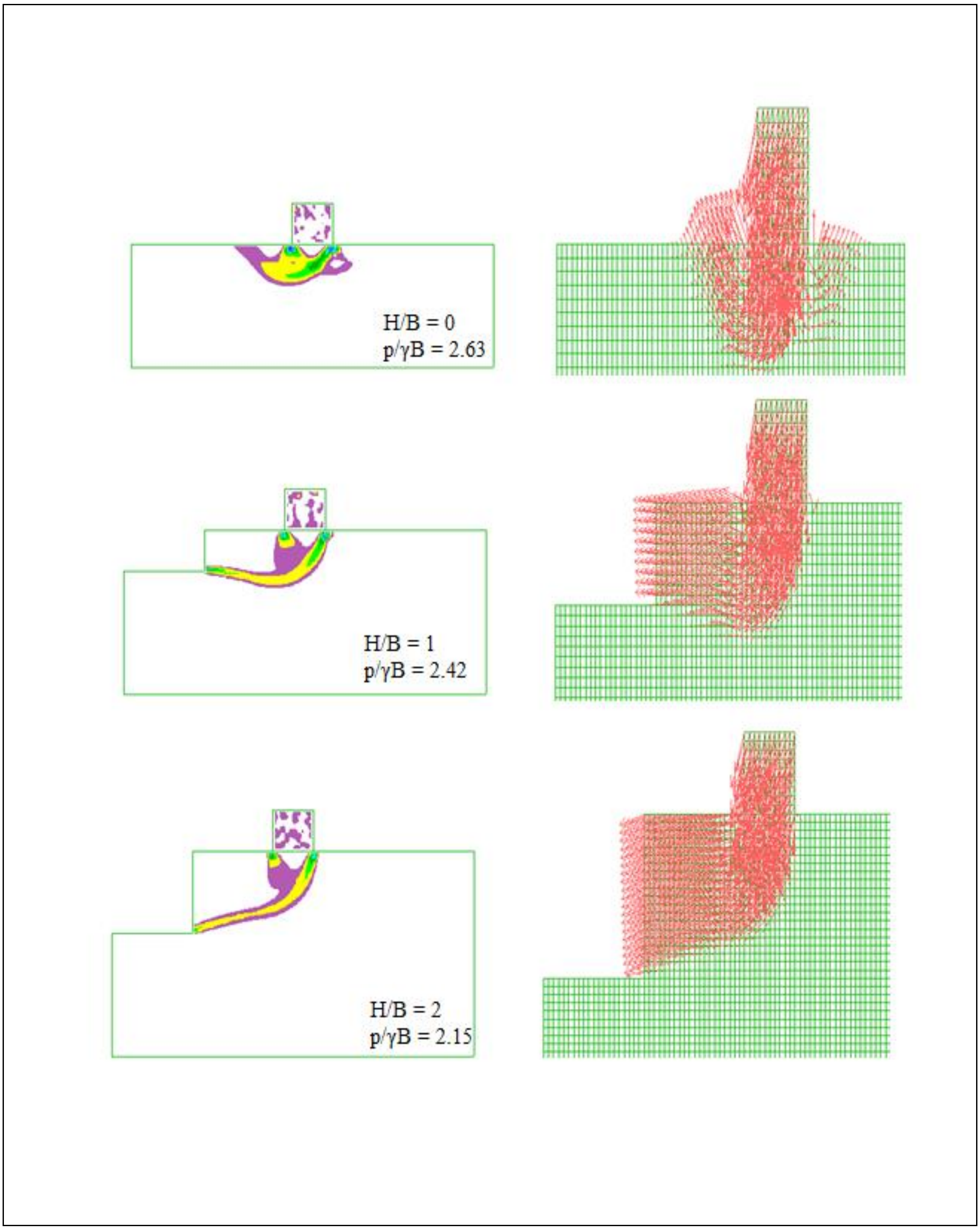

Figure 7-19. The change in inclined normalised bearing capacity with H/B ratio for Kh=0.4.

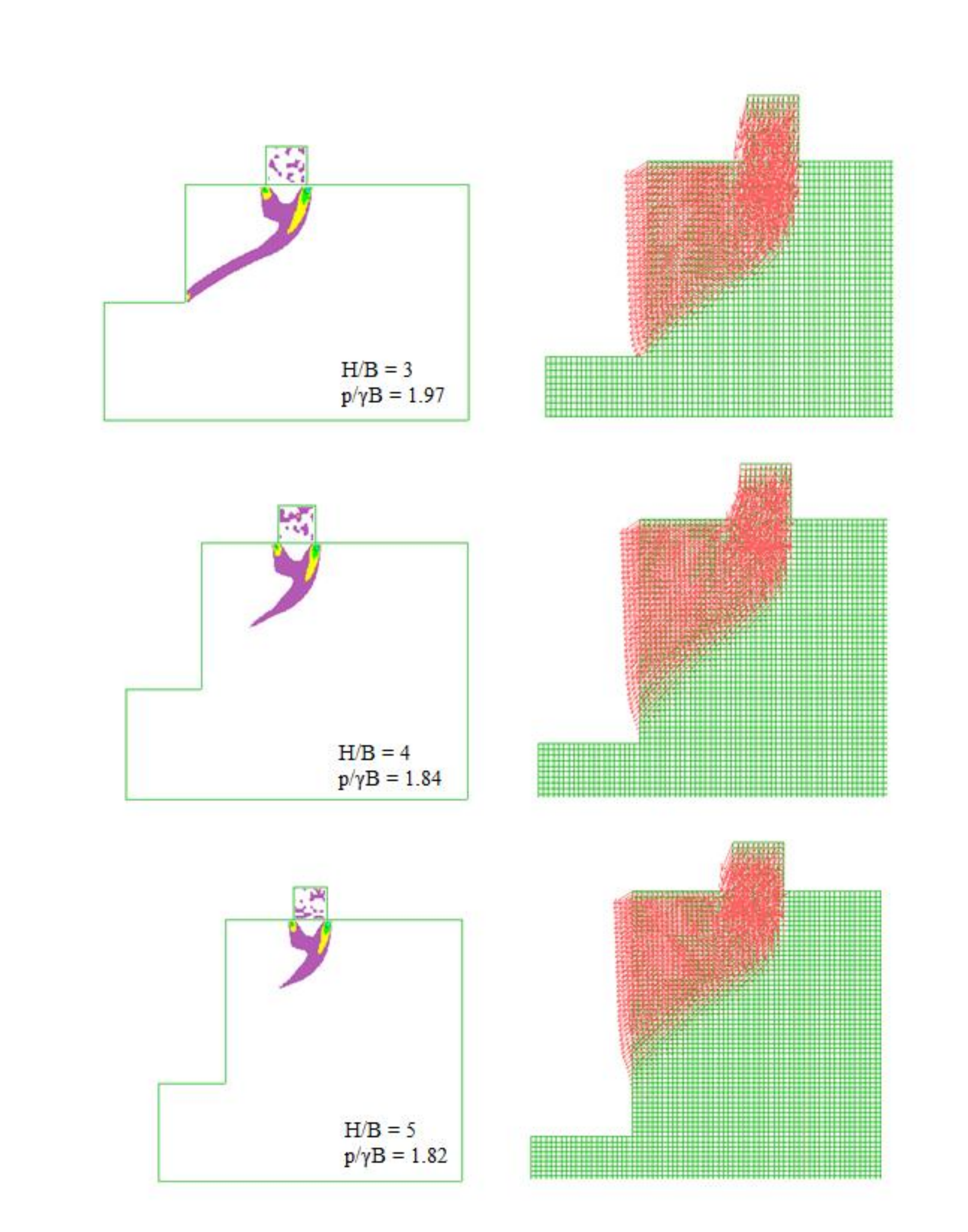

Figure 7-20. The change in inclined normalised bearing capacity with H/B ratio for Kh=0.4 (continued)

Presented within Figures 7-19 and 7-20 are the shear strain rate and velocity vector plots for change in inclined normalised bearing capacity with H/B ratio, for a coefficient of horizontal acceleration 0.4. The major difference observed between the previous earthquake magnitude and this earthquake magnitude is the slight reduction in the proportion of the velocity fields as the earthquake magnitude increases. This difference can be seen more clearly within the shear strain plots, with the gradual reduction of the slip surface within the soil structure towards the slope.

7.5.3 H/B Ratio Conclusions

The first conclusion made was the effect of H/B ratio was found to be crucial at smaller heights. It was found that as the slope height was increased the capacity of the foundation reduced, with the most significant reductions in capacity occurring at H/B ratios between 0 and 1, when the foundation failure mechanism transitioned from flat ground failure (symmetrical shear strain plot) to local shear failure (unsymmetrical shear strain plot).

The second conclusion was that the presence of the earthquake induced horizontal acceleration only marginally affected the overall capacity of the foundation, with increases in magnitude decreasing the capacity slightly. The most significant change within failure mechanisms occurred at H/B ratio of 4 between earthquake magnitudes of 0.2w and 0.3w, where the slip failure reduced. This result was clearly presented within the shear strain rate plots.

From this study it is suggested that further studies be conducted into the effect of the D/B ratio on these different H/B ratio results to determine whether at toe and above toe failure is altered. This was not covered within this study as time did not permit due to the scope of the project.

7.5.4 Effect of Soil Strength Ratio

The soil strength ratio of soil is measured by the cohesion of a soil structure. For purely cohesive soils the non-dimensional soil strength ratio is an important parameter as it directly affects the cohesion of a soil. It is expected that a linear relationship between the normalised bearing capacity and soil strength ratio is induced under varied foundation and soil conditions, due to the proportionality of the two parameters. Thus it is expected from this study that any increase in soil strength ratio will proportionally increase the normalised bearing capacity.

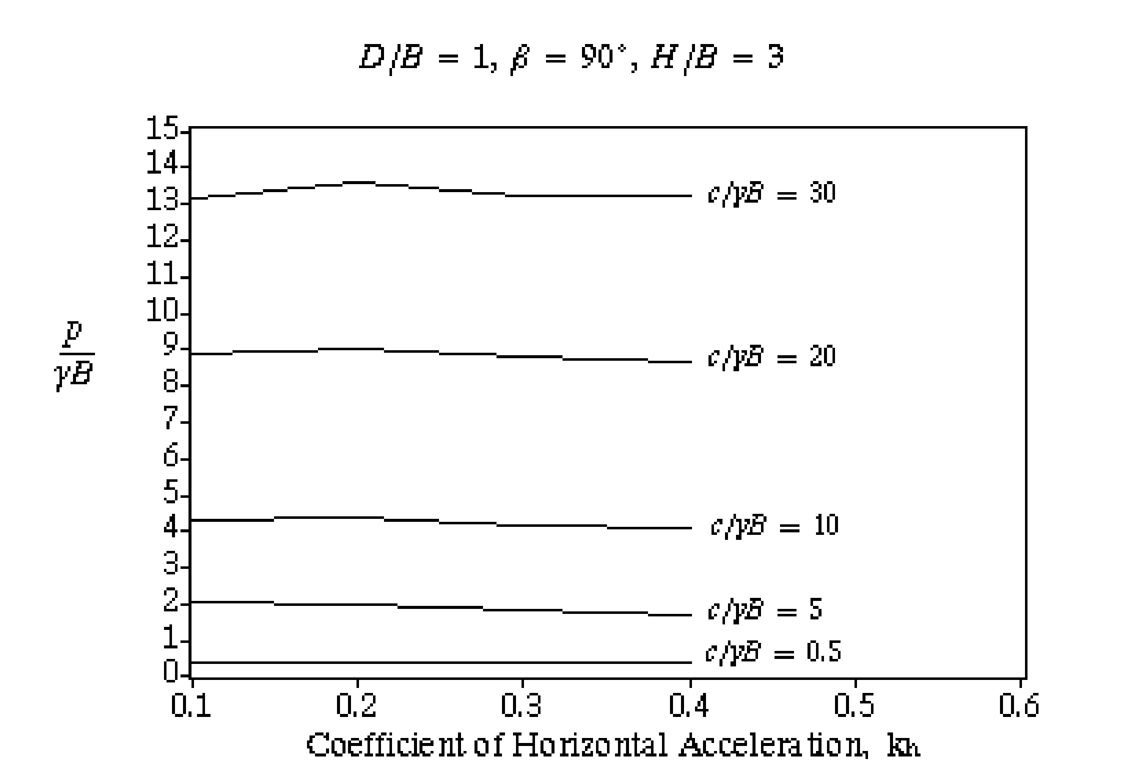

Figure 7-21. Change in inclined normalised bearing capacity with coefficient of horizontal acceleration.

Figure 7-21 presents the change in inclined normalised bearing capacity with the coefficient of horizontal acceleration for a range of different soil strength ratios. From this graph it is evident that increases in the soil strength ratio significantly increase the inclined normalised bearing capacity of the foundation. It is also evident that as the coefficient of horizontal acceleration is included only very minimal reductions in capacity occur, with the differences reducing further with increased soil strength ratio. Therefore it can be concluded that there is a relationship between the soil strength and earthquake magnitudes.

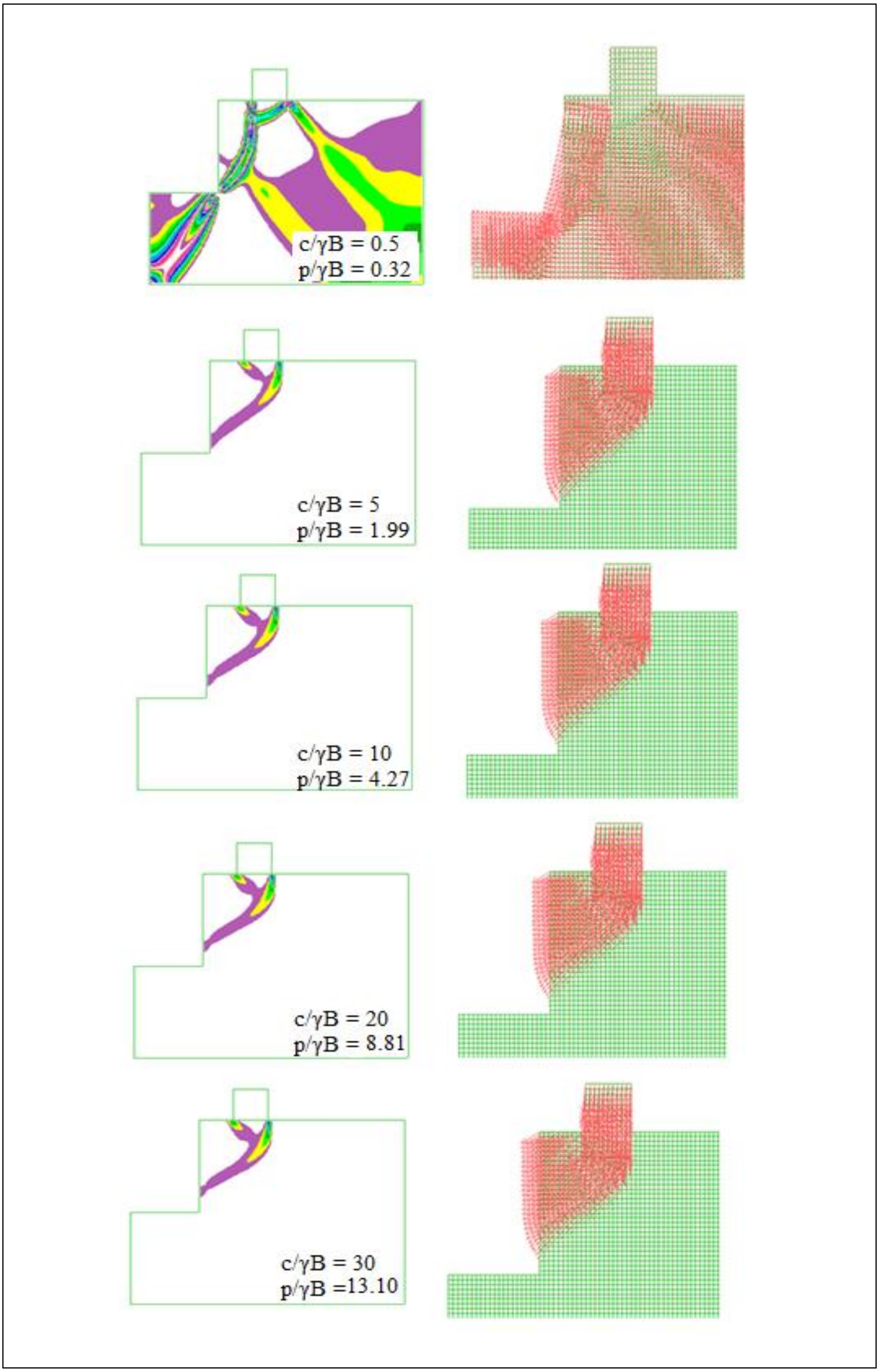

Figure 7-22. The change in inclined normalised bearing capacity with q/γB ratio for Kh=0.1

Figure 7-22 presents the change in inclined normalised bearing capacity with soil strength ratio for a coefficient of horizontal acceleration equal to 0.1. Within this figure are the shear strain rate plots and the velocity vector plots. The main conclusion drawn from this is the change in stability when the foundation soil strength ratio is increased to above 5. For soil strength ratios between 5 and 30 the failure mechanism is above the toe failure thus, and the capacity is increasing significantly. Within the velocity vectors there is no real trend except for a reduction in the slip surface as the soil strength ratio increases, indicating increases in bearing capacity within the model.

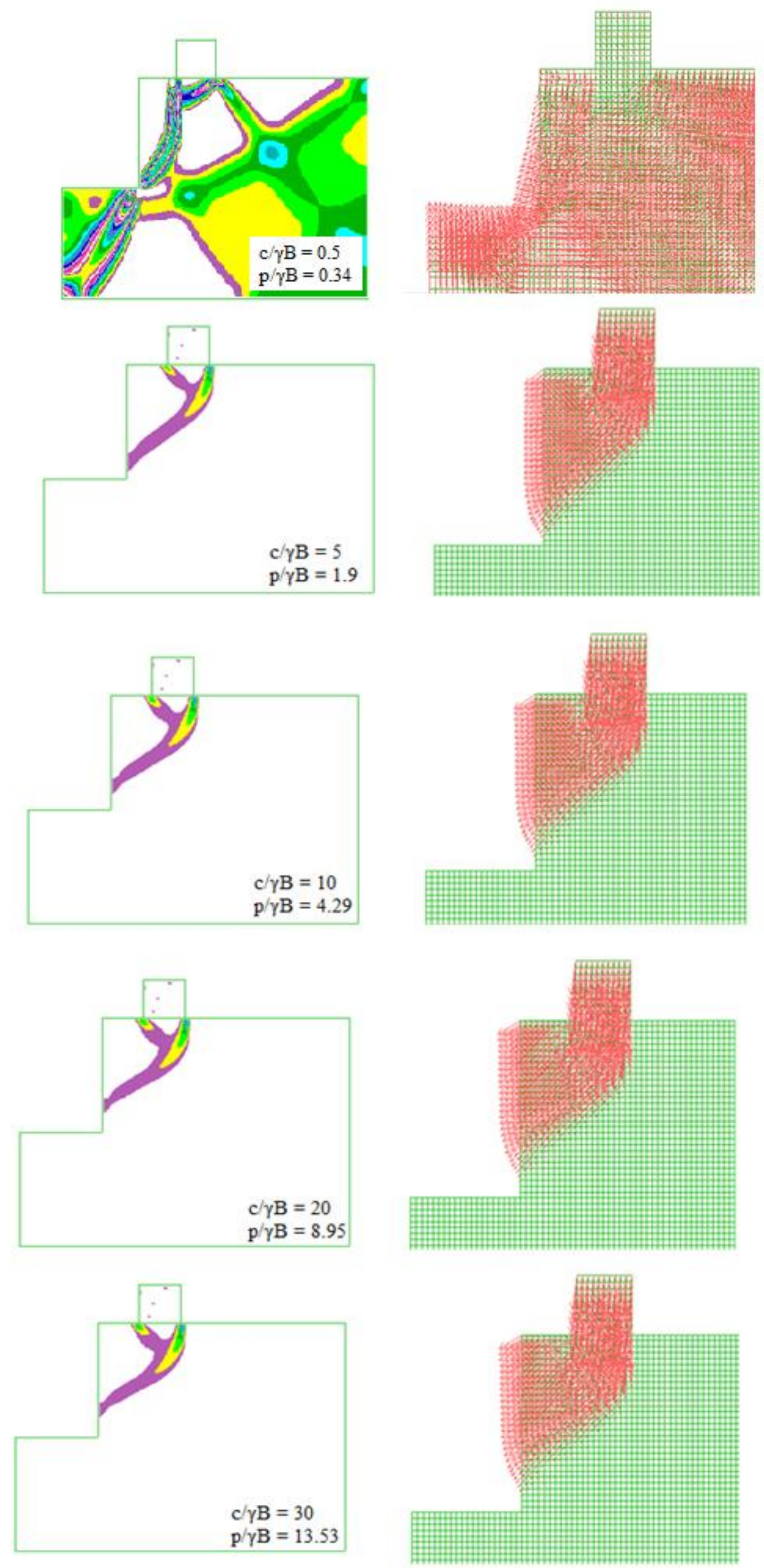

Figure 7-23. The change in inclined normalised bearing capacity with q/γB ratio for Kh=0.2

Figure 7-23 presents the change in inclined normalised bearing capacity with soil strength ratio for a coefficient of horizontal acceleration equal to 0.2. The results presented within this figure are very similar to the results presented within Figure 7-22 for the earthquake magnitude 0.1w. The soil strength 0.5 is incapable of supporting the foundation load, while at soil strengths equal to and greater than 5 the failure induced in local shear failure above the toe of the slope. Again as the soil strength increases the capacity proportional increases, indicating the relationship between capacity and soil strength.

Figure 7-24 and 7-25 present the changes in inclined normalised bearing capacity with soil strength ratio for a coefficient of horizontal accelerations equal to 0.3 and 0.4, respectively. Again it is evident that there is minimal change in the failure slopes produced, from past earthquake magnitudes. The constant trend within the results is the increasing capacity as the soil strength ratio increases, and the local shear failure above the slope toe failure that is occurring for soil strength ratios between 5 and 30.

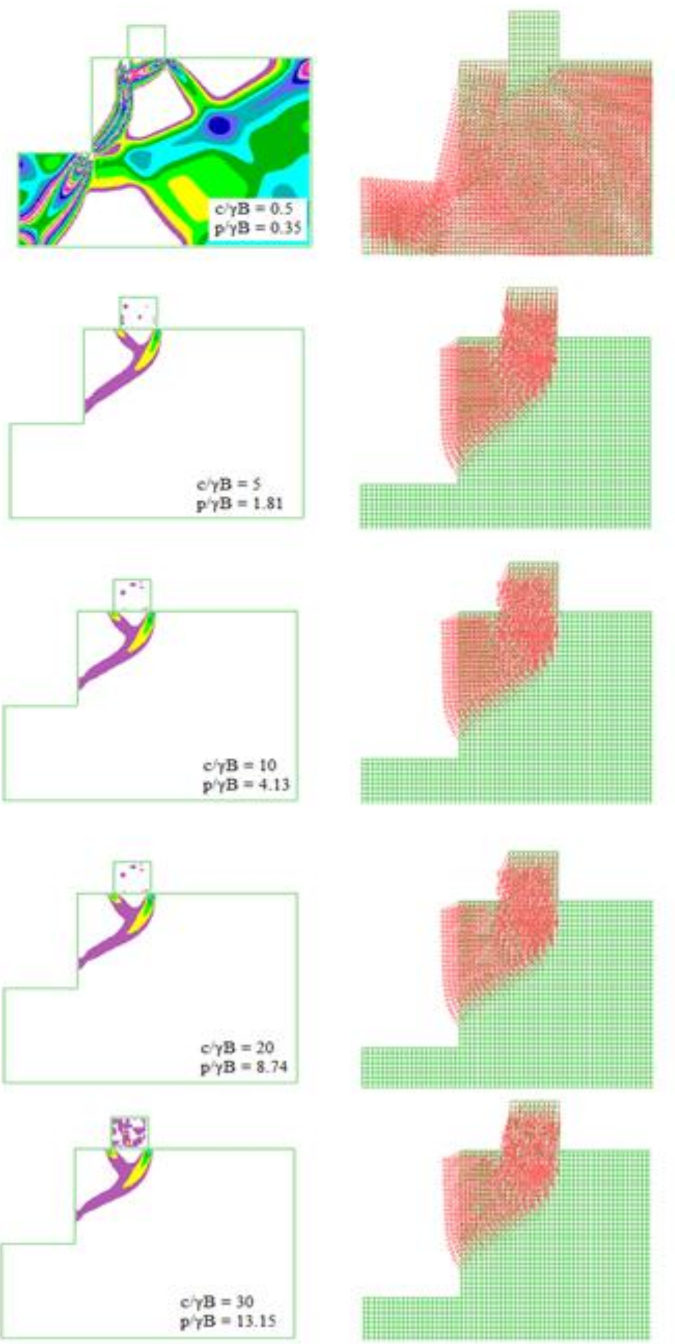

Figure 7-24. The change in inclined normalised bearing capacity with q/γB ratio for Kh=0.3

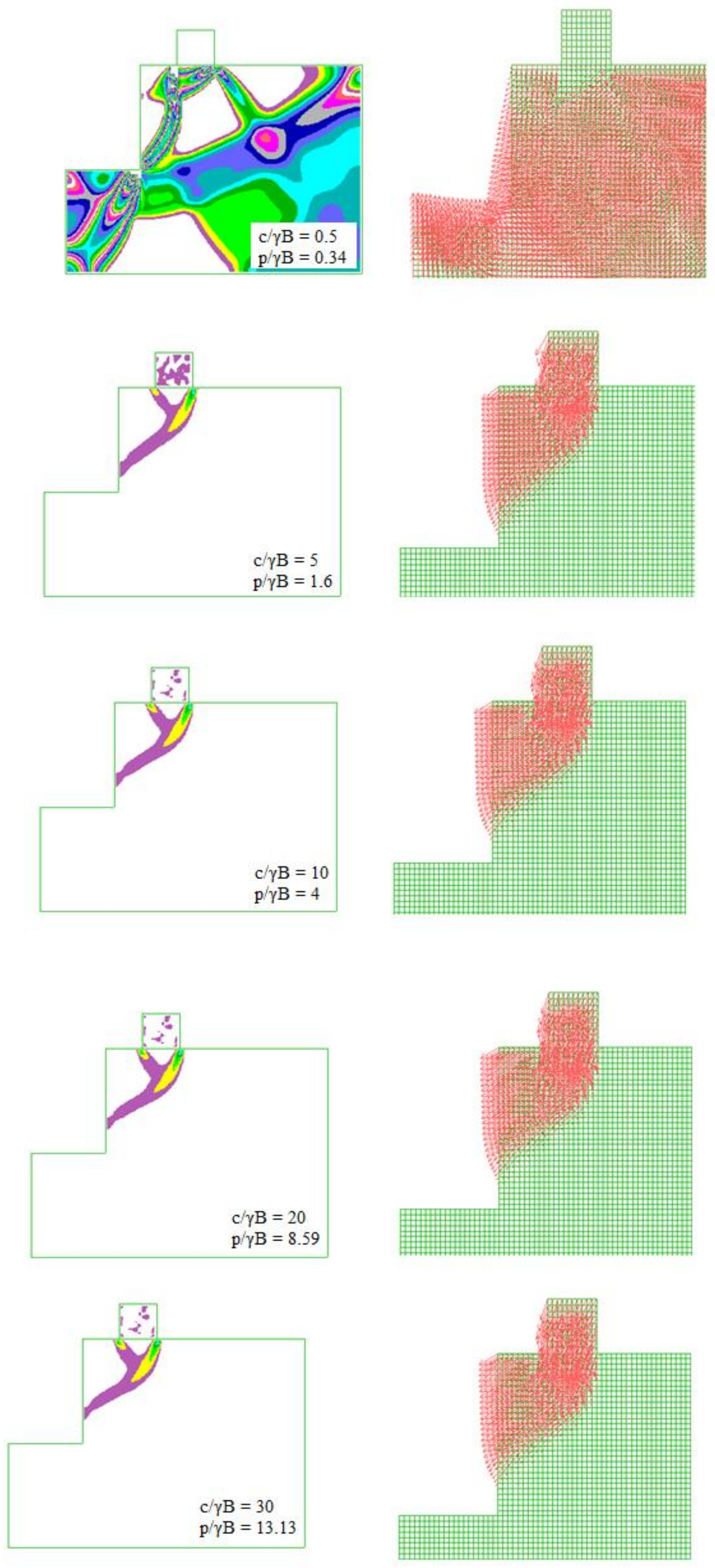

Figure 7-25 The change in inclined normalised bearing capacity with q/γB ratio for Kh=0.4

7.5.5 Soil Strength Ratio Conclusions

The conclusions drawn from this parametric study of the soil strength ratio, were minimal as there was insignificant changes in capacities for increases in the coefficient of horizontal acceleration. But the main finding was that linear relationship between the soil strength ratio and the ultimate bearing capacity of the foundation. It was determined throughout the results that as the soil strength increased the bearing capacity of the foundation increased. Therefore it was concluded from this chapter that the level of cohesion within a clayey soil played an important part in determining the normalised bearing capacity of a slope.

Conclusion

8.1 Summary of Findings

This dissertation has illustrated the use of the explicit finite difference software program FLAC to produce advanced models of the shallow foundation located near a purely cohesive 90^o slope. The purpose of this study was to produce qualitative results that would then be used to validate whether or not past FLAC models were adequately and conservatively producing ultimate bearing capacities for the foundation problem. This chapter presents the overall findings and achievement of this dissertation along with a brief explanation of further studies that could be conducted within this topic. From review of the major project goals set within the project specifications, it was determined that this dissertation achieved all project goals sufficiently with adequate results being obtained for all aspects of the project.

8.2 Conclusions

The problem of the rigid shallow foundation resting near a slope or cut is commonly experienced design problem encountered within engineering practice. Due to this there have been a number of different numerical modelling studies conducted for the foundation problem, some in which have resulted in the preparation of ultimate bearing capacity design charts. The major focus of this study was to conduct advanced modelling and analysis of the foundation model, whilst taking in real life foundation characteristic, to develop a qualitative set of results that could be used within the validation of previous simplified numerical models.

The analysis method adopted within this project was Elasto-Plastic, Mohr Coulomb failure criterion, and this method was used to obtain bearing capacities of the shallow foundation built hear a slope. This analysis was done through the use of the explicit finite difference modelling of the FLAC software. All numerical models produced by developed models within this dissertation have been validated against, available works in the form of past dissertations, published research papers and physical modelling of the problem.

Within the advanced analysis of the shallow rigid foundation, there were four different models developed; a soils structure interface model, a discontinuous foundation punching model, a large strain analysis model and a static pseudo seismic model.

Within the soil structure interface model a weighted foundation model was prepared with considerations made for both smooth and rough interface conditions. The main conclusions drawn from this model were;

- The introduction of the foundation weight produced ultimate bearing capacities less than the model that only considered applied velocities at an imaginary foundation base. The reduction in capacity was concluded to be due to the method of modelling the interface boundary and allowing slippage within it and due to the additional momentum during slippage, that was a result of the foundation weight.
- The modelling of the smooth conditions between the interface boundary of the foundation and soil structure, produced capacities less than the rough interface modelling of the interface. This result was concluded to be due to the additional frictional forces present within the rough interface model increasing the bondage strength of the foundation problem. Thus it was concluded that smooth interface modelling was the most conservative

interface modelling method, with respect to soil structure interface modelling.

- The final conclusion made from this chapter was that the smooth soil structure interface model, with a weight foundation, produced more conservative values for the ultimate bearing capacity of the foundation, than the models produced within previously studies. This was concluded to be due to the inclusion of the foundation weight.

Within the discontinuous foundation punching model a weightless foundation model was prepared with considerations made for both smooth and rough interface conditions. The main conclusions drawn from this model were;

- The introduction of the two vertical interfaces at the location of the foundation punching increased the ultimate bearing capacity of the foundation from the soil structure interface model. Thus it was less conservative than the soil structure interface model.
- A secondary conclusion drawn from the investigation was that increasing the vertical interface lengths reduced the bearing capacities produced. Thus it was concluded that the smooth interfaced model with increased vertical foundations lengths produced the most conservative results with respect to models produced by previous studies and the smooth soil structure interface model investigated within chapter 4 of this dissertation.

Within the large strain analysis of the shallow foundation model to models were analysed; the soil structure interface model for smooth and rough conditions and the discontinuous foundation punching model for smooth and rough conditions. The main conclusions drawn from this model were;

- The results obtained from the large strain analysis of both models proved to be greater than the capacities produced within the small strain analysis of the models. Thus it was concluded that the small strain analysis of the foundation is most conservative modelling method.

- However it was concluded the failure mechanisms and mesh deformations produced from large strain analysis, were more realistic representation of actual foundations situated near slopes. Due to the scope of this project this was however not taken consideration.

Within the static pseudo seismic modelling a weightless foundation with soil structure interface modelling was prepared. Considerations within the model were made for smooth and rough interface conditions. The main conclusions drawn from this model were;

- The rough interface model presented the more accurate modelling method with respect to available results for upper bound limit modelling of the problem. Thus from this conclusion on rough interface considerations were considered within the parametric study.
- The conclusions from the study of D/B were that increased earthquake magnitude reduces the capacity, while increasing the footing distance ratio D/B increases the capacity. It was also conclude that the modelling of the seismic forces increased the presence of the slope for greater foundation distances from the slope.
- The conclusions from the H/B ratio study were that again increased earthquake magnitude reduced the foundation capacities. But increases of H/B ratio reduced the capacity as the foundation transitioned from flat ground failure to local shear failure at the slope toe.
- The soil strength ratio study proved not very beneficial, with respect to investigating earthquake magnitudes as there was minimal if any differences within capacities between foundation magnitudes. However it was concluded that soil strength ratio has a linear relationship with normalised bearing capacity, thus as the soil strength was increased the ultimate bearing capacity within the foundation increased.

From the above conclusions drawn it was determined that the findings from this dissertation were adequate in satisfying the project aims presented prior to project commencement within the project specification.

8.3 Recommendations for Future Work

Through conducting the research presented within this dissertation a number of areas where further work could be done to increase the value of the results obtained from the project were highlighted. Some of these topics include;

1. Investigation of Dilation Angle Effects.
2. 3D Footing Effect Modelling.
3. Comprehensive Design Charts for Foundation Materials other the purely cohesive clays.
4. Further actual physical modelling of the problem.
5. Investigations into the Effects of Slope Angle with Respect to Pseudo Seismic Modelling.
6. Modelling of the Foundation Under Construction Conditions (soil consolidation taken into consideration).
7. Investigation of inclined foundation loads, such as wind loads.
8. Develop the Discontinuous Foundation Punching Model Through the Modelling of the Building Weight.
9. Further Develop the Parametric Studies conducted for Seismic Modelling.
10. Investigate the Seismic model for different foundation materials.

Within this topic of study there are endless avenues and directions that could be taken in the advanced modelling of the shallow foundation situated near slopes. But due to the scope and time constraints presented for this project investigation of these additional factors was not achievable. Thus they have been recommended for the future study within future student dissertations.

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9

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Project Specification

A.1 Project Specification

University of Southern Queensland

FACULTY OF ENGINEERING AND SURVEYING

ENG4111/4112 Research Project

PROJECT SPECIFICATION

FOR: **RENEE PETERS**

TOPIC: Advanced Analyses of Shallow Foundations Located near Slopes

SUPERVISOR: Dr. Jim Shiau

ENROLMENT: ENG4111 – S1, D, 2011;

ENG4112 – S2, D, 2011

PROJECT AIM: Throughout the development of civil construction there has been a continuous problem of footings on slopes, thus this project aims to create a comprehensive and qualitative set of design charts and tables that are user friendly and that could aid in the preparation of preliminary designs for the age old problem of footings on or near a slope, for a homogenous clay soil. There is also a secondary aim of providing enough qualitative research to provide information for appropriate text book revision, as footings on a slope is not an extensively researched area. In order to produce such results, modelling and investigation into a range of complex conditions and scenarios will be conducted. The geomechanic software package, FLAC, will be used throughout the problem study, to model and produce a range of design charts and tables for the investigation of complex conditions and scenarios that could possibly be encountered within the design of footings on slopes. To ensure the results obtained from the FLAC software are accurate and relevant, validation with previous workings on the problem will be conducted.

SPONSORSHIP: University of Southern Queensland, Faculty of Engineering and Surveying

PROGRAMME: **Issue A, 22nd of March 2011**

1. Conduct research into previous workings into the footings on a slope problem, through the research of published text books available and published research papers.

2. Validate the current code used within the FLAC analysis and verify the results being obtained are accurate, by comparing with past solutions.

3. Model and investigate the condition of interface affects between the footing and underlying clay soil, for an extreme smooth case and an extreme rough case, for a small strain condition.

4. Model and investigate the condition of the interface affects whilst including the condition of vertical interface separation.

5. Model and investigate the affects of large and small strain condition, for a smooth interface condition, and yield a concluding result of which parameter is the most conservative.

6. Investigate and model the scenario of Pseudo Seismic conditions for a footing on a slope, while considering two cases of a building and no building case, with an inclined velocity due to the vertical and horizontal movement within Pseudo Seismic conditions.

AGREED________________(student) ____________________(supervisor)

Date: / /2011 Date: / / 2011

Assistant Examiner: