SESIMIC BEHAVIOUR OF FOUNDATIONS ON ROCK SLOPES

A DISSERTATION

Submitted in the partial fulfilment of the requirements for the award of the degree of

MASTER OF TECHNOLOGY

in

EARTHQUAKE ENGINEERING

(With specialization in Seismic Vulnerability and Risk Assessment)

by

SHIKHAR PRAKASH

(17553007)

DEPARTMENT OF EARTHQUAKE ENGINEERING
INDIAN INSTITUTE OF TECHNOLOGY ROORKEE
ROORKEE-247667 (INDIA)

JUNE, 2019

CANDIDATE’S DECLARATION

I hereby, declare that the work which is being presented in this dissertation entitled, **“SESIMIC BEHAVIOUR OF FOUNDATIONS ON ROCK SLOPES”**, is submitted in partial fulfilment of the requirements for the award of degree of “Master of Technology” in “Earthquake Engineering” with specialization in Seismic Vulnerability and Risk Assessment, to the Department of Earthquake Engineering, Indian Institute of Technology Roorkee, under the supervision of Dr. Yogendra Singh, Professor, Department of Earthquake Engineering, Indian Institute of Technology Roorkee and co-supervision of Dr. Sanjay Kumar Shukla, Associate Professor, School of Engineering, Edith Cowan University, Australia, is an authentic record of my own work carried out during the period of June 2018 to June 2019.

I declare that I have not submitted the material embodied in this dissertation for the award of any other degree or diploma.

Place: Roorkee
Date:

Shikhar Prakash
17553007

CERTIFICATE

This is to certify that the above statement made by the candidate is correct to the best of my knowledge and belief.

Place: Roorkee
Date:

(Yogendra Singh)
Professor
Department of Earthquake Engineering
Indian Institute of Technology Roorkee

Place: Joondalup
Date:

(Sanjay Kumar Shukla)
Associate Professor
School of Engineering
Edith Cowan University, Joondalup, Australia

ABSTRACT

During a seismic event, the buildings situated on slopes are more vulnerable than those located on flat ground. The foundations placed on the slopes have reduced bearing capacity as compared to the flat surface. This reduced bearing capacity when coupled with the synergistic effect of a seismic event can lead to a drastic drop in the bearing capacity of the foundations. The slopes are themselves made up of a varied geo-material. Rock-mass being a major geo-material in the slopes, it becomes inevitable to model rock-mass in the simulation of slopes for the seismic analysis of foundations.

In general practice, the shallow foundations serve as an economic and reliable solution to support high-rise buildings, bridges and other heavy structures constructed on rock-mass. in flat as well as slope. The seismic bearing capacity of foundations in case of slopes is an important parameter to judge in the overall stability of a structure in the case of a seismic event. In various past earthquakes, even before the failure of a structural element, the failure of foundation due to sliding and overturning has been observed.

Various codes of the world are silent on this aspect of seismic bearing capacity of foundation on rock-slope. Thus a need for the robust guidelines for the seismic bearing capacity estimation is required. Past studies have touched upon this aspect but on a limited scale.

The current study aims to carry out a rigorous parametric analysis using a Finite Element Limit Analysis FELA with both the Upper Bound (UB) and Lower Bound (LB) estimates of the seismic bearing capacity factors for foundations placed on rock slopes.

The objective of the present study is to obtain the seismic bearing capacity factor, $N_{\phi s}$ for a strip footing placed on the top of the slope of rock-mass. The seismic force is considered as pseudo-static force, in terms of horizontal seismic coefficient, α_h , applied on the entire rock mass. The earthquake effect on superstructure is considered as additional horizontal force on foundation. The factor $N_{\phi s}$ is obtained and investigated for the different value of horizontal seismic coefficient, α_h, β, *G.S.I.*, D, γ, m_i, σ_{ci} (where, β is the slope angle; *G.S.I* and D are the in-situ parameters and γ, m_i, σ_{ci} are the laboratory parameters required to define the Hoek-Brown failure criteria)

A rigorous analysis has been carried out to study the effect of all the parameters defining the rock-mass property and the slope geometry. The aim of the thesis is to develop a robust design charts based on the parametric study.

ACKNOWLEDGEMENT

First appreciation goes to my parents, without whose beliefs, pursuing post-graduation would still have been a dream.

Completion of this dissertation would not have been possible without the expertise of my supervisor **Dr. Yogendra Singh,** Professor in Department of Earthquake Engineering, Indian institute of Technology Roorkee. Frequent discussions throughout the dissertation work were extremely fruitful and helped me to overcome hurdles and problems I faced during this work. It was the consistent belief of my supervisor that helped me sail through my research endeavor. I am highly obliged for the personal strength and support extended by him towards me.

I am highly obliged to **Dr. Dhiraj Raj**, Research Associate in Department of Earthquake Engineering, Indian institute of Technology Roorkee whose timely help and vision accelerated the project work. His sincere guidance, encouragement and proper direction gave insight in learning the modelling and understanding the concept in better way.

It will be difficult to mention every friend and family members' name but sincere thanks to **Ms. Bharti M**., Research Scholar, **Mr. Kavan Modha**, Junior Research Fellow, **Mr. Aman Srivastava, Mr. Avanish V.** and **Mr. Praveen Nautiyal,** Master Student in Department of Earthquake Engineering, Indian institute of Technology Roorkee for their constant support and timely help thorough my journey here.

Further, I would like to acknowledge the extremely good computer facilities provided by the department without which this work would not have been possible.

Scholarship given by Ministry of Human Resources and Development, Government of India is highly appreciated for this dissertation work.

Thanks to everyone who have helped me achieve this.

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CHAPTER 1: INTRODUCTION

1.1 General

In the present scenario, as described in Raj et al. (2018), with the rapid urbanization, infrastructure development, and scarcity of flat land in hilly regions are driving the heavy construction on hill slopes. Many times, despite unfavourable conditions for construction, foundations of buildings and bridges are forced to be placed on the slope face. Compared with foundations on flat land, these foundations are more prone to failure due to slope instability.

Further, many of the hilly regions in the world, e.g., the Himalayan region in India, are also susceptible to seismic activity. In case of an earthquake event, a synergistic effect of sloping ground and seismic loading may cause severe stability problems for earth-retaining structures and foundations due to reduction in bearing capacity.

Figure 1.1 Building location and foundation failure on slope

Most current standards and codes of practice primarily focus on the estimation of static bearing capacity of shallow foundations on flat ground. Guidelines for estimating the capacity of foundations on slopes, particularly under the effect of seismic actions, are mostly lacking. On the other hand, ample literature is available for the estimation of the seismic bearing capacity of strip foundations on flat ground.

Further, the seismic analysis of structures and foundations situated on the slopes are majorly restricted to considering the soil as the geo-material existing on the slopes. However, it is not the case usually with many structures located on slopes in the Himalayan region. The slope usually consists of a layered system.

In this layered system of slopes there exist:

i. A top soil layer of varying depth
ii. A layer of highly fractured rock.
iii. A layer of intact rocks with a few pre-defined planes of weakness.

In case of an earthquake, any of the above three layers may be the governing cause of failure of the foundation and the structure resting on it. Thus it is necessary to extend the work done on soil slope to the underlying rock layers as many it may be the case that the highly fractured rock or the pre-defined plane of weakness may lead to the failure of the structure even if the soil strata of the slope is safe. Also, there are cases when the soil does not exist at all and the slope is composed of the rock itself.

As it was understood from the literature review, studies have been done to estimate the seismic bearing capacity of the flat rocks. However, a limited work was found in the domain of rock slopes and a research gap exist where an extensive work can be done in the seismic analysis of the buildings and foundations on the rock slope. Thus all these possibilities lead to this study titled “Seismic behaviour of foundations on rock slope.”

1.2 Rock Slopes

Himalayas being one of the youngest mountain ranges in the world, are also one of the most seismically active mountain ranges in the world. Stability of slopes particularly in case of a seismic event is one of the major concern for the engineering community across the globe. Slope instability can lead to major consequences such as foundation failure, overturning of buildings, landslides and avalanches.

Rock slopes, in particular, behave differently as compared to soil slopes. Unlike soil, if the rock is intact, it is quite rigid, behaves elastically and can offer high compressive strength. Due to these attributes, it is common to place foundations directly upon intact rocks.

However, the rock slopes usually found in the Himalayan ranges are less likely to be found intact on its surface. Rather the rock slopes found in Himalayas are layered system, with the upper layer being the highly fractured rock, followed by the intact rocks with well-defined planes of weaknesses.

In a slope stability analysis, the two major contributors to the instability are the highly fractured rocks that may lead to a plastic flow condition under high stresses or the planes of weakness that may yield, break apart or may slide in case of high stresses.

Thus the stability analysis of a rock slope in its natural state is a precursor before any engineering analysis on the slope. Particularly in case of a seismic event it is necessary to judge the stability of the slope by introducing the seismic forces either pseudo statically or dynamically.

1.3 Buildings and Foundation on Rock Slopes

When a building-foundation system is placed on a slope in a hilly region, various factors such as topographic amplification, slope stability, bearing capacity of foundations on slopes, structural irregularity of hill buildings together contribute to impose a high risk on our existing infrastructural capacity in case of a seismic event.

During a seismic event, the ground motions are affected by the topography, stratigraphy of the slope. This ground motion imposes a broader risk of the slope failure itself, which is even higher when the building load on the slope is high in case of dense urbanisation. There are recommendations for the estimation of slope stability in various codes using both pseudo-static method as well as displacement-based method. However, the complexity due to the presence of building loads has not been addressed.

Further the synergistic effect of sloping ground and seismic loading may cause severe stability problem for foundations and earth retaining structures due to the loss of bearing capacity caused due to the seismic load and the lack of confinement near the edge of the slope. As per the literature review, it has been observed that a robust guideline for the bearing capacity of foundations on rock slope is mostly lacking.

Lastly, it has been a consistent observation that the hill side buildings have irregular structural configuration. Although it has been an evident practice of designing the buildings based on the perspective force based design approach and a special consideration for such hill buildings is mostly lacking. However, in the past decade, due to the development of the performance-based design and seismic fragility development opens up the window for the development of framework for coupled rock slope-building system to investigate the various engineering demand parameters specific to such irregular hill buildings.

1.4 Literature Review

This review incorporates the study of a set of research papers, broadly classified under the following cases:

a) Bearing Capacity of foundation on:
   i. Isotropic Rocks
   ii. Anisotropic Rocks
b) Seismic Bearing Capacity of foundation on flat Rock
c) Seismic Bearing Capacity of foundation on Rock Slope

The following table has been made to enumerate the various researches being done in the past related to the above stated cases as following:

1.4.1 Bearing capacity of foundation on isotropic rocks

Under the above stated heading the following research papers were considered:

Table 1.1 List of works related to bearing capacity of foundations on isotropic rock.

S.N.	Author	Foundation Type	Rock Type	Failure Criteria	Remark
1	Merifield et al.(2006)	Shallow strip footing	Isotropic	The generalised Hoek–Brown failure criterion	Ultimate bearing capacity are obtained by employing finite elements in conjunction with the upper and lower bound limit theorems of classical plasticity.
2	Saada et al.(2007)	Surface strip footing	Isotropic	Modified Hoek–Brown criterion	Ultimate bearing capacity is obtained within the Framework of the kinematic approach of limit analysis theory.
3	Yang & Yin (2005)	Surface strip footing	Isotropic	Modified Hoek–Brown failure criterion	Ultimate bearing capacity is obtained within the Framework of the kinematic approach of limit analysis theory

4	Serrano et al.(2000)	Shallow strip foundation	Extremely fractured isotropic	Modified Hoek Brown criterion	The ultimate bearing capacity of rock mass is calculated using the plasticity theory and the Characteristic method.

In Merrifield et al. (2006) applies numerical limit analyses to evaluate the ultimate bearing capacity of a surface footing resting on a rock mass whose strength can be described by the generalized Hoek–Brown failure criterion. Rigorous bounds on the ultimate bearing capacity are obtained by employing finite elements in conjunction with the upper and lower bound limit theorems of classical plasticity. The bearing capacity of shallow foundations resting on a modified Hoek–Brown rock mass has been investigated within the framework of the kinematic approach of limit analysis theory.

In Yang & Yin (2005) the strength envelope of rock masses is considered to follow a modified Hoek–Brown failure criterion that is a nonlinear failure criterion. Two different kinds of techniques has been used to develop the ultimate bearing capacity in the framework of limit analysis in plasticity.

A consistent theme that can be seen from the above stated papers is that for the rocks (which can be grouped under the category of isotropic rocks i.e. the intact rock or heavily jointed rock masses that can be considered homogeneous and isotropic) is that the strength criteria usually employed is hoek brown failure criteria. In some places the hoek brown criteria was also converted into an equivalent mohr - coulomb criteria for the analysis.

The other consistent theme is that the upper (kinematic) or lower bound limit theorems of classical plasticity were employed to estimate the ultimate bearing capacity obtained from constrained minimization procedure.

For the analysis in the domain of the limit analysis kinematic approach various mechanisms such as generalized prandtl-type failure mechanism, multi wedge translational failure mechanism, etc were used to obtain the lowest ultimate bearing capacity out of all.

1.4.2 Bearing capacity of foundation on anisotropic rocks

The following research items has been studied under the above stated heading:

Table 1.2 List of works related to bearing capacity of foundation on anisotropic rock

S. N.	Author	Foundation Type	Rock Type	Failure Criteria	Remarks
1	Singh & Rao(2005)	Shallow Strip Foundations	Anisotropic Non-Hoek–Brown Rock Masses	A simple parabolic equation is used to define the strength criterion.	The method uses Bell's approach of computing bearing capacity, in which the ultimate bearing capacity is determined as the major principal stress.
2	Bindlish et al. (2012)	Shallow Foundation	Jointed Rock Mass	Experimental studies were carried out to assess failure.	The effect of the joint orientation and interlocking of rock mass on the ultimate bearing capacity of the rock mass has been studied
3	Serrano & Olalla (1998)	Shallow Strip Foundation	Anisotropic Discontinuous Rock Mass.	A non-linear behaviour through the rock mass, defined by the Hoek and Brown model and a linear strength behaviour along the planes of weakness, defined by their cohesion and angle of internal friction	Six different mechanisms of failure were considered under a foundation depending on the boundary conditions and the orientation of the transverse isotropy

Singh and Rao (2005) make use of the mapping of joints in the field and simple laboratory tests on intact specimens of rock.

In bindlish et al. (2012) an experimental study was conducted wherein a rigid footing placed on the top surface of the jointed rock mass was loaded up to the failure. The effect of the joint orientation and interlocking of rock mass on the ultimate bearing capacity of the rock mass has been studied.

Serrano & Olalla (1998) uses six different mechanisms of failure were considered under a foundation depending on the boundary conditions and the orientation of the transverse isotropy. A non-linear behavior through the rock mass, defined by the Hoek and Brown model (parameters m, s and the unconfined compressive strength, respectively) and a linear strength behavior along the planes of weakness, defined by their cohesion and angle of internal friction.

In the above stated papers the bearing capacity of anisotropic rock mass is considered usually by experimental results and mapping of discontinuity. However, if modeled to analyze the intact rock mass were analyzed using hoek brown failure criteria whereas the discontinuity was analyzed using mohr coulomb criteria.

1.4.3 Seismic bearing capacity of foundations on flat rock

The following research items has been studied under the above stated heading:

Table 1.3 List of works related to seismic bearing capacity of foundation on flat rock

S. N.	Author	Foundation Type	Rock Type	Failure Criteria	Remark
1	Keshavarz et al.(2015)	Strip footings	Isotropic	Hoek Brown failure criterion	Stress characteristics or slip line method was used for analysis Seismic effects were incorporated using pseudo static method.
2	Zhou et al. (2015)	Shallow strip footing	Isotropic	non-linear twin shear strength criterion	Kinematic Approach to Limit Analysis Seismic effects were incorporated using pseudo dynamic method.

In Keshavarz et al. (2015) the bearing capacity of strip footings on rock masses has been studied in the seismic case. The stress characteristics or slip line method was used for analysis. The seismic effects were applied as the horizontal and vertical pseudo-static coefficients.

Zhou et al. (2015) analyzes the bearing capacity of shallow foundations resting on rock masses subjected to seismic loads based on limit analysis theory. The non-linear twin shear strength criterion was used to consider the effects of intermediate principal stress on the bearing capacity of shallow foundations. The pseudo-dynamic approach was applied to account for the effects of seismic loads on the bearing capacity of shallow foundations.

In the above two papers the rock mass considered is isotropic and seismic effects are considered using two basic methods pseudo static and pseudo dynamic seismic coefficient methods.

1.4.4 Seismic bearing capacity of foundation on rock slope

The following research items were studied under the above stated heading:

Table 1.4 List of works related to seismic bearing capacity of foundation on rock slope

S. N.	Author	Foundation Type	Rock Type	Failure Criteria	Remark
1	Yang (2009)	Strip footing	Isotropic rock slope	Hoek–Brown failure criterion	Limit analysis framework used. Quasi-static representation of earthquake effects using a seismic coefficient is adopted
2	Saada et al.(2010)	Shallow strip footing	Isotropic rock slope	generalized Hoek–Brown criterion	framework of the kinematic approach of limit analysis theory A pseudo-static approach is adopted to account for the earthquake effects
3	Ausilio&Zimmaro (2015)	shallow strip footing	Isotropic rock slope	“Generalized tangential” technique in which the Hoek–Brown strength is replaced by an “optimal” tangential Mohr–Coulomb domain.	The framework of the kinematic approach of limit analysis theory was adopted. Earthquake-induced displacement-based analysis was used to incorporate seismic effect

The seismic bearing capacity of shallow foundations resting on a modified Hoek–Brown rock mass is investigated within the framework of the kinematic approach of limit analysis theory. A pseudo-static approach is adopted to account for the earthquake effects for the seismic bearing capacity evaluations.

1.5 Objectives

In the present study, the primary focus is to study the behaviour of the foundations placed on slopes made of highly fractured, homogenous and isotropic rock. A rigorous parametric study has been planned to study the effects of rock properties as well as seismic effects so that some robust guidelines for the stability of foundations on rock slopes can be established. The objectives are explicitly illustrated as below:

1. To calibrate a material model in the Finite Element (FE) framework to efficiently model the highly fractured homogenous rock.
2. To study the behaviour of rock slopes and to obtain a set of stable slopes in case of a seismic event.
3. To obtain the seismic bearing capacity factors of the foundations placed on slopes.

1.6 Methodology and Scope of the Research Work

In order to obtain the seismic bearing capacity factor, $N_{\phi s}$ for strip footings placed on rock slopes, 2D plane-strain nonlinear FELA, using the Lower Bound (LB) and Upper Bound (UB) element formulation with Second Order Cone Programming (SOCP). The adaptive meshing technique using the shear dissipation, as offered by OptumG2 (2018) software is used. To simulate the seismic effect, pseudo-static method is used. The seismic horizontal force has been applied on the entire slope in terms of α_h and a horizontal force proportional to the mass is applied on the foundation. The seismic bearing capacity factor obtained has been illustrated with the help of charts to depict its parametric variation with the rock properties (GSI, m_i, σ_c, γ) slope geometry (β) and seismic effect (α_h).

1.7 Organization of Dissertation

The dissertation has been organized in six chapters as follows:

Chapter 1 presents a brief introduction of the research topic, the various risks associated with the seismic event in a hilly region is illustrated. Further the literature review, objectives, methodology and scope of the research work, and organization of the dissertation is sated.

Chapter 2 presents the aspects of numerical modelling of rocks illustrating the methods of analysis available, sensitivity and convergence study involved, material model calibration.

Chapter 3 presents the parametric study to obtain the seismic bearing capacity factor $N_{\sigma s}$ as a function of the the rock properties (GSI, m_i, σ_c, γ) slope geometry (β) and seismic effect (α_h).

Chapter 4 presents the design charts obtained from the numerical results of the parametric study conducted to obtain the seismic bearing capacity factor $N_{\sigma s}$.

Chapter 5 presents the discussions and conclusions derived from the study with the key emphasis on the limitations of the work and the future scope associated with it.

CHAPTER 2: NUMERICAL MODELLING OF ROCKS AND ROCK SLOPES

This particular chapter of the dissertation is focused to illustrate the various steps undertaken in the modelling and analysis of the rock slopes. The chapter has been classified under the following broad categories:

1. A detailed illustration of the various methods of analysis those are available in literature has been presented. Then an argument is also established for the selection of the method of analysis chosen for the work.
2. A detailed illustration of the sensitivity and the convergence study done to ascertain the optimum mesh size and model domain has been presented.
3. A detailed illustration of the efforts made to establish a robust material model to effectively simulate the rock has been presented. Also the efforts made in the calibration of the material model in the available commercial software and its validation is discussed.
4. A detailed illustration of the methods employed in modeling and analysis of seismic bearing capacity of strip footing on flat surface has been presented.
5. A detailed illustration of the methods employed in modelling and analysis of seismic bearing capacity of strip footing on rock slope has also been presented.

2.1 Available Methods of Analysis

As stated earlier, the estimation of bearing capacity of strip foundations on rocks forms an important part of the work. There are various methods available for the estimation of the bearing capacity of foundations on rocks. The utility of a method for an analysis is based on the judgment of the scale and complexity of the problem to be considered in the analysis. Thus a detailed overview of the various available methods is done. The scope and limitations of each method is thoroughly studied so that the method that is most suitable for this particular work can be determined.

Assessment of bearing capacity of shallow foundations has been one of the most common problem in civil engineering. The methods generally used for its estimation mainly fall within one of the four categories.

2.1.1 The limit equilibrium method

It is a method which is being traditionally used to obtain approximate solutions for the stability problems related with the geotechnical domains. The method assumes a failure surface of simple shapes such as plane, circular, log spiral etc. with this assumption each of the be stability or collapse load determination problem is reduced to finding the most critical location of the failure surface which leads to the least estimation of the collapse load. Further assumptions regarding the stress distribution along the failure surface is made such that overall equation of equilibrium is written in terms of stress resultants. It is a simple method that can be solved using simple statics. The method basically gives no consideration to soil kinematics, and equilibrium conditions are satisfied in a limited sense.

2.1.2 The slip-line method

Stress state can be broadly classified into two states

i. Small change in body or surface force will not destroy the equilibrium
ii. Even a small change in body surface forces will cause loss of equilibrium called the limiting stress state. It depends directly on the basic mechanical constants which characterize the resistance of granular media to shear deformation.

Rankine established the idea that the loss of equilibrium occurs by means of the slip of material over certain curvilinear surface. Then after a large part of the research went into two broad streams of thoughts

i. To find the most suitable slip surface to define the loss of equilibrium.
ii. To find the exact solution of the limiting equilibrium so that it was possible to find the complete solution of the various problems and determination of corresponding slip line network. Kötter transferred the set of differential equations of equilibrium and the condition of limiting equilibrium at each point to curvilinear coordinates. Prandtl assumed a weightless granular soil media to obtain a closed form solution.

The slip line method tries to make an effort with the help of numerical methods to solve the set of differential equations and the associated limiting failure equilibrium to obtain a grid of slip surfaces completely defining the sub surface.

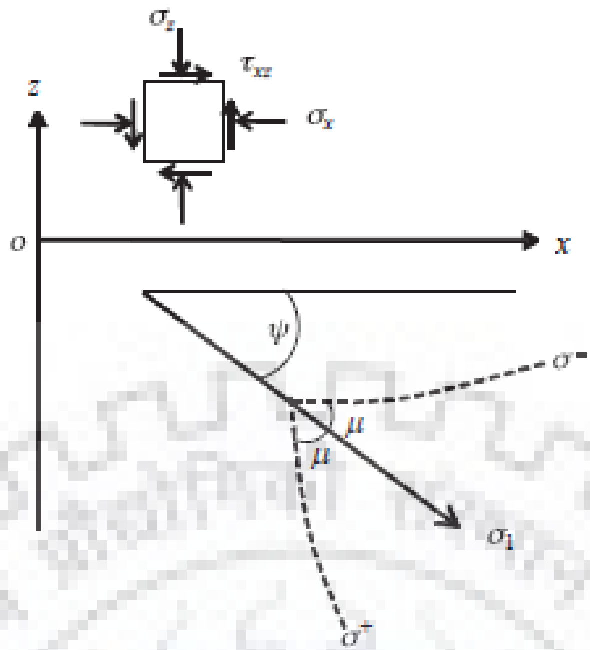


Figure 2.1 The stress components within a rock-mass

Under plane strain conditions in the x-z plane, the unknown stress components at any point in the rock mass are related to the body/ inertial forces through the given equations:

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xz}}{\partial z} = X \qquad (2.1)$$

$$\frac{\partial \sigma_z}{\partial z} + \frac{\partial \tau_{xz}}{\partial x} = Z \qquad (2.2)$$

where, X and Z are the body and/or inertia forces in x and z directions respectively.The stress components can be estimated from the failure criteria (for e.g. if Mohr-Coulomb is used) as follows:

$$\sigma_x = p + R\cos(2\psi) \qquad (2.3)$$

$$\sigma_z = p - R\cos(2\psi) \qquad (2.4)$$

$$\tau_{xz} = R\sin(2\psi) \qquad (2.5)$$

Using the Equations 2.1 and 2.2, the two stress components in characteristic directions can be obtained.

In the stress characteristics method, each point in the medium is described with four parameters: x, z, p and j, where x and z are the coordinates of the point. Writing above equations in finite difference form, the unknown information at any point C can be found from points A and B, where BC is the positive and AC is the negative stress characteristics. The trial and error procedure is used to compute the properties of point C. For the first try, the properties of point Care assumed to be equal to those of points B and A in the positive and negative directions, respectively. Then the new properties are obtained for point C. This procedure is continued until the differences between the calculated properties of point C in the last two-steps are small enough.

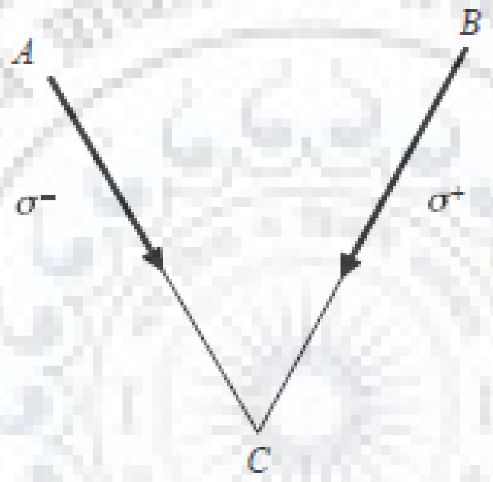


Figure 2.2The unknowns at point C can be found from points A and B where BC is the positive and AC is the negative stress characteristics

2.1.3 The limit analysis method

As understood from Chen and Liu (1990) it deals directly with the estimation of the collapse load bypassing the spreading process of the contained plastic flow. Such a direct determination of collapse load by limit analysis is of great help in obtaining a better understanding of the development of uncontained plastic flow through the contained plastic flow analysis where the FEM solution development becomes difficult.

Collapse load as calculated in limit analysis on an idealized structure (where, strain hardening is not considered) is not equal to the actual plastic collapse load however a good approximation is obtained. An upper and lower bound technique of the limit analysis theorem provides a bound of values for collapse.

Limit load is defined as the plastic collapse load of an idealized body for which the plastic deformation can increase without limit under a constant limit load. This is established under an assumption:

i. Perfectly plastic body
ii. Small deformations

Any structural stability problem is defined by the satisfaction of three basic states:

i. Equilibrium equation
ii. Compatibility equations
iii. Constitutive stress strain relationship

In the limit state analysis problem for lower bound solution

i. Only Equilibrium equations and yield criteria is satisfied.
ii. Statically admissible stress field should:
    a) Satisfy the equation of equilibrium
    b) Satisfy the stress boundary
    c) Nowhere violates the yield criteria.
iii. External load determined is not greater than the actual load thus called a lower bound
iv. It also implies that the unconfined plastic flow will not occur at a load lower than this.

In the limit analysis problem for the upper bound solution

i. Only compatibility equation and flow rule associated with the yield criteria is satisfied.
ii. Kinematically admissible velocity field should
    a) Should satisfy velocity boundary condition.
    b) Should satisfy the strain rate and velocity compatible condition.
iii. External load determined is greater than the actual load thus upper bound.
iv. Unconfined plastic flow must have occurred at a load lower than the obtained.

The implementation of the kinematic approach to limit analysis relies on the following fundamental inequality (Saada, 2011)

$$p_e(U) \leq P_{mr}(U) \tag{2.6}$$

where U is any virtual, kinematically admissible velocity field,P_e(U)denotes the work done by the external forces, andP_{mr}(U) represents the maximum resisting work developed in the failure mechanism. Defined by the π function described below:

$$p_{mr}(U) = \int_{\Omega} \pi\left[d(x)\right] d\Omega + \int_{\Sigma} \pi\left\{v(x);\left[U(x)\right]\right\} d\Sigma \qquad (2.7)$$

Where, d is the strain rate field associated with the velocity field, $[U(x)]$ is the jump in U at a point x when crossing a possible velocity discontinuity surface following its normal, and the Π-functions are the support functions defined by the duality from the strength criteria.

In order to obtain the collapse load various mechanisms are assumed in the analysis of rock structure which can be summarized as follows:

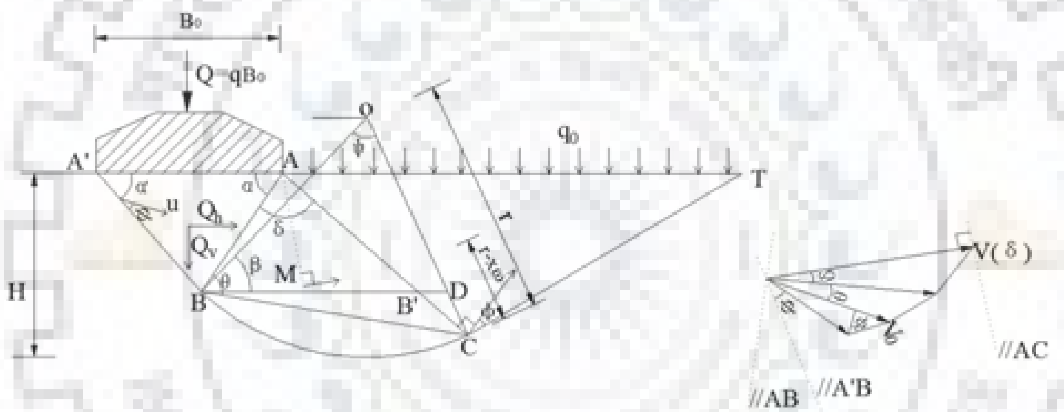


Figure 2.3 Generalized Prandtl-type Failure Mechanism (Saada, 2010)

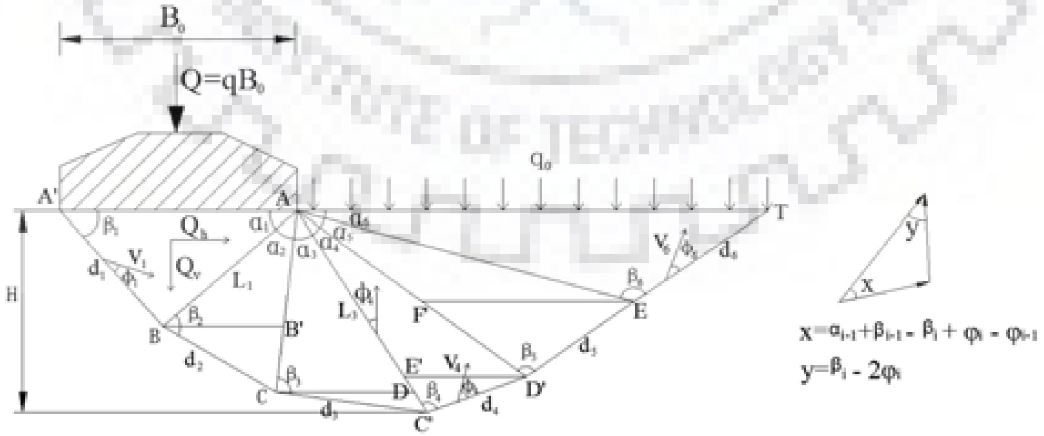


Figure 2.4 Multi-Wedge Translation Failure Mechanism (Saada, 2010)

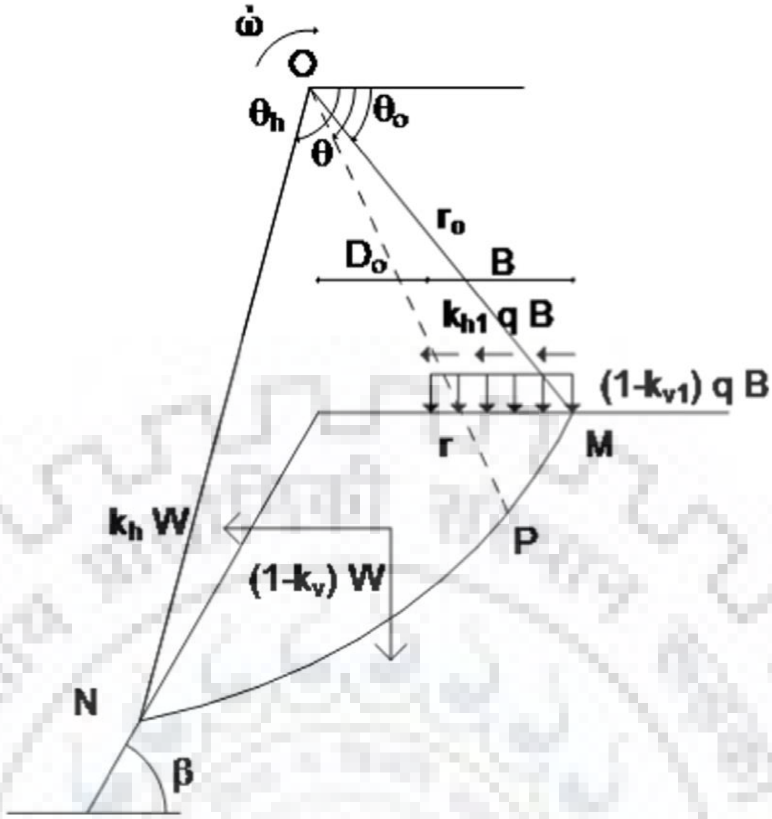


Figure 2.5 A typical mechanism in suitability with the boundary condition of rock or rock slopes

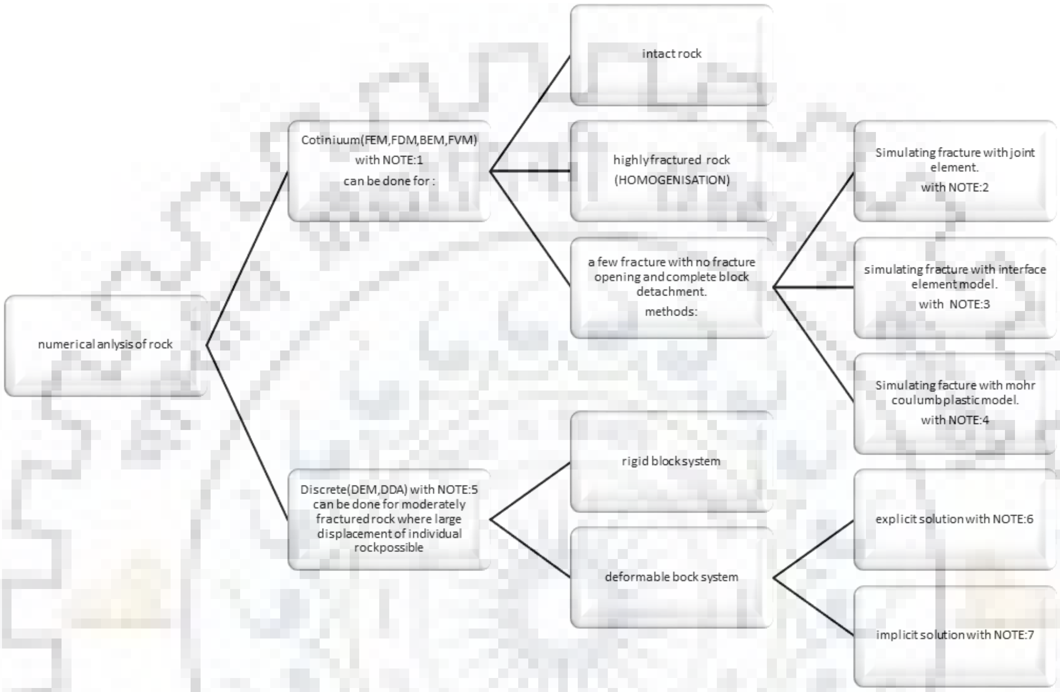
Out of the various mechanisms incorporated in the study, the corresponding ultimate bearing capacity is obtained from the constrained minimization procedure and then further compared to get the least out of all.

2.2 The Numerical Methods

The usage of numerical methods in the domain of bearing capacity determination opens up the space for a very robust simulation of the actual physical characteristics of the geo material existing below the foundation. The choice of the method to be used for analysis depends upon the scale to which the actual characteristics of the geo material are aimed to be simulated in the analysis.

In case of rocks existing as a geo material below the footing there exist a long set of features that requires to be simulated in order to rightly estimate the bearing capacity. A detailed review of the various numerical methods available to rightly simulate the rock characteristic was done with the help of the review of numerical modeling for rock mechanics and rock engineering as published by Jing (2003).

A rigorous study of the various available numerical methods was done to develop an understanding of the scope and shortcomings of each of the methods of numerical modeling of rock. The understanding developed after the thorough study of the methods is represented here with the help of a chart as shown below:



where, abbreviations are as follows:

FEM: Finite Element Method

FDM: Finite Difference Method

BEM: Boundary Element Method

FVM: Finite Volume Method

DEM: Distinct Element Method

DDA: Discrete Displacement Analysis

NOTE 1: Continuum method of analysis of numerical analysis has the following features:

i. Contact pattern between the components of the model do not change.
ii. Stress aligns as per the strain compatibility of the model.
iii. It is incapable of capturing large scale displacements and openings along & across the fractures within the rock mass.

NOTE 2: The above stated method has the following features:

i. A joint element is provided to simulate the fracture in the rocks.
ii. Stiffness parameters both K_N and K_S are specified in the model to incorporate the stiffness along and across the fracture joint elastically.
iii. Large scale opening sliding and complete detachment are not permitted.
iv. Ill condition of the numerical system due to large aspect ratio may occur.

NOTE 3: The above stated method has the following features:

i. The global stiffness may lead to the problem of ill conditioning if the number of fracture elements to be incorporated is increased.

NOTE 4: The above stated method incorporates the elasto plastic mohr coulomb material model to simulate the fracture within the rock.

NOTE 5: discrete methods of numerical analysis have the following features:

i. Contact pattern between the components of model change with time.
ii. Strain compatibility is updated with time.
iii. Capable of simulating large displacements and openings along & across the fractures within the rock mass.

NOTE 6: The above stated method has the following features:

i. The rock mass is considered deformable.
ii. The explicit solution is obtained with the help of FDM/FVM discretization.
iii. No need for solving large scale matrix.
iv. UDEC is the commercial code available for the above method.

NOTE 7: The above stated method has the following features:

i. The rock mass is considered deformable.
ii. The implicit solution is obtained with the help of FEM discretization.
iii. There is a need for solving large scale matrix.

As of now, a homogenous highly fractured rock mass is considered for the various seismic analyses to be done in the work. The scale of complexity of the real physical condition to be incorporated in the modeling is thus fixed in this way.

Owing to the above stated scale of modeling finite element method of the continuum branch of the numerical methods will be used. Continuum numerical methods are found to be suitable to analyze the homogenous set of rock masses. The commercial FEM Code and software used in the present study is ABAQUS (2016).

However, if we consider the bearing capacity estimation part of the work, it is evident from the literature that Finite Element Limit Analysis Method FELA is not only found to be sufficient but also computationally more economical and efficient, particularly in the estimation of the bearing capacities. FELA is a powerful technique that combines the capabilities of FE discretization for handling complex soil properties, loadings, and boundary conditions, with the plastic bound theorems of limit analysis to bracket the exact limit load by upper and lower bound solutions. The theorems of the limit analysis are applicable to a perfectly plastic material with associated flow rule.

This aspect of FELA has been very rigorously explored in Raj et al. (2018). Thus for the bearing capacity estimation FELA will be used. The commercial FELA code OPTUMG2 has been used in the present study.

2.3 Sensitivity and Convergence Study

2.3.1 Sensitivity analysis

For the sensitivity analysis, a simple plane strain 2D Model of a footing was developed in abaqus with a soil mass beneath it. The elasto plastic mohr-coulomb material model was considered. The known material properties of the soil have been assigned to the model so that the results of the study can be validated.

In order to estimate the optimum domain of the model a large number of analyses were run of varying dimensions of soil mass below. The dimension of the footing was fixed and the dimensions of the soil mass were represented as the function of the width of the footing.

On observing the various analyses, the domain size in which the failure pattern of the soil did not hindered with the boundaries of the model was considered optimum. The selected model had a domain width of 15B and depth of 5B.

2.3.2 Convergence Analysis

For the estimation of optimum mesh size different analyses with different mesh sizes were analyzed. The load displacement curves for different models were plotted to obtain the bearing capacity of soil and the mesh size beyond which the results did not varied was considered as the optimum mesh size. The results were further validated with the available bearing capacity formulations (analytical and numerical).

The load displacement curve for different mesh sizes are as given below:

The validation of the model for the estimation of bearing capacity of soil has been done with the soil properties shown in Table 2:

Table 2.1Material Properties

Width of footing, b (m)	2
Cohesion of soil, c (kPa)	50
Angle of fiction, Φ (degrees)	36
Angle of dilation, ψ (degrees)	33
Density of soil (kN/m^3)	18

The results have been validated from the analytical equation given as follows:

$$Q_{us} = cN_c + 0.5\gamma bN_\gamma + \gamma D_f N_q \qquad (2.8)$$

Also the results were validated using the numerical formulation for bearing capacity determination as provided in Raj et al. (2018).

Table 2.2 Comparison of bearing capacity

	Present study	Raj et al.	Analytical
Q_u (kN)	6422.34	6704	6588

2.4 Material Model

The major part of the work is concentrated in the development of a suitable elasto plastic constitutive model to rightly simulate the behavior of rocks. The rock material considered for the analysis is highly fractured homogenous rock mass. Owing to the above stated feature of the rock mass the material model with hoek brown failure criteria was adopted. The various characteristics of the hoek brown failure criteria and the methods used to properly calibrate it in commercial soft wares are discussed in the following sub parts.

2.4.1 Hoek-Brown failure criteria

The Hoek–Brown failure criterion for rock masses was first described in 1980 and has been subsequently updated in 1983, 1988, 1992, 1995, 1997, 2001 and 2002.The latest version that is used here can be written as:

$$\sigma_1^{'} = \sigma_3^{'} + \sigma_{ci}\left(m_b \frac{\sigma_3^{'}}{\sigma_{ci}^{'}} + s\right)^{\alpha} \qquad (2.9)$$

The relationships between m_b/m_i, s and the geological strength index (GSI) are as follows:

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right) \qquad (2.10)$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right) \qquad (2.11)$$

$$\alpha = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right) \qquad (2.12)$$

The GSI was introduced because Bieniawski's rock mass rating (RMR) system and the Q-system were deemed to be unsuitable for poor rock masses. The GSI ranges from about 10, for extremely poor rock masses, to 100 for intact rock. The parameter D is a factor that depends on the degree of disturbance. The suggested value of the disturbance factor D is 0 for undisturbed in situ rock masses and D is 1 for disturbed rock mass properties.

The unconfined strength is obtained as follows:

$$\sigma_c = \sigma_{ci} s^{\alpha} \qquad (2.13)$$

Tensile strength as:

$$\sigma_t = -\frac{s\sigma_{ci}}{m_b} \qquad (2.14)$$

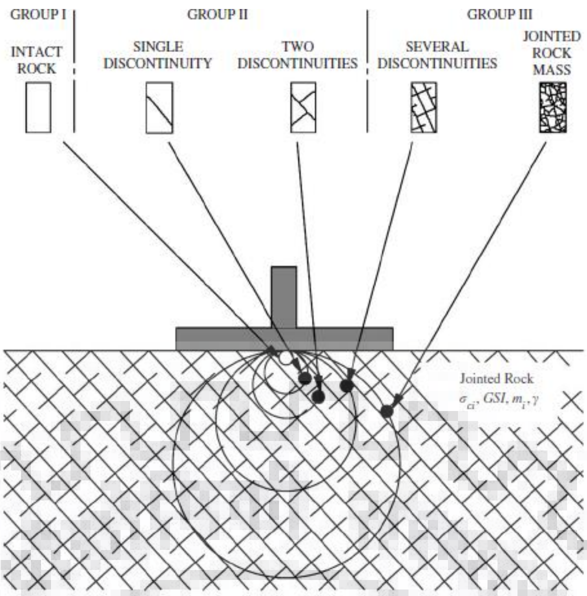


Figure 2.6Applicability of Hoek-Brown failure criterion (Merifield, 2006)

2.4.1.1 Calibration and validation

As it has been discussed above that one of the major problem of implementing the hoek brown failure criteria is the lack of availability of hoek brown material in the various available finite element commercial codes. Thus a large part of the work is focused on calibrating the material model into the available software (In this case ABAQUS). The various methods available for implementing the model can be broadly classified as follows:

i. Using the equivalent Mohr-Coulomb model
ii. Using the tangential linearization of the failure criteria

Using the equivalent Mohr-Coulomb model, under the following section an effort is made to numerically simulate the rock slope using the Hoek Brown failure criteria which is assumed to satisfactorily represent the rock strength as per the studied literature. But a major concern encountered in the process is that the most of the geotechnical soft wares are written in terms of the Mohr-Coulomb failure criterion in which the rock mass strength is defined by the cohesive strength c’ and the angle of friction Φ’. There is no direct correlation between Mohr Coulomb criterion and the non-linear Hoek-Brown criterion. Consequently, determination of the values of c’ and Φ’ for a rock mass that has been evaluated as a Hoek-Brown material is a difficult problem. The difficulty in applying this approach in practice is that most of the geotechnical software currently available provides for constant rather than effective normal stress dependent values of c’ and Φ’.

As suggested in Hoek E, et al (2002) it is necessary to determine equivalent angles of friction and cohesive strengths for each rock mass and stress range. This is done by fitting an average linear relationship of Mohr Coulomb criterion to the curve generated by Hoek Brown criterion for a range of minor principal stress values defined by $\sigma_t < \sigma_3 < \sigma_{3max}$.

$$\phi' = \sin^{-1}\left[\frac{6\alpha m_b\left(s+m_b\sigma'_{3n}\right)^{\alpha-1}}{2(1+\alpha)\left(2+\alpha\right)+6\alpha m_b(s+m_b\sigma'_{3n})^{\alpha-1}}\right] \tag{2.17}$$

$$c' = \frac{\sigma_{ci}\left[(1+2\alpha)s+(1-\alpha)m_b\sigma'_{3n}\right]\left(s+m_b\sigma'_{3n}\right)^{\alpha-1}}{2\left(1+\alpha\right)(2+\alpha)\sqrt{1+(6\alpha m_b(s+m_b\sigma'_{3n})^{\alpha-1})}} \tag{2.18}$$

The Mohr-Coulomb shear strength τ, for a given normal stress σ, is found by substitution of these values of c’ and Φ’ in to the equation: $\tau = c' + \sigma \tan\Phi'$. The equivalent plot, in terms of the major and minor principal stresses, is defined by:

$$\sigma'_1 = \frac{2c'\cos\phi'}{1-\sin\phi'} + \frac{1+\sin\phi'}{1-\sin\phi'}\sigma'_3 \tag{2.19}$$

Relationships between major and minor principal stresses for Hoek-Brown and an equivalent Mohr-Coulomb criterion are given by the following figure:

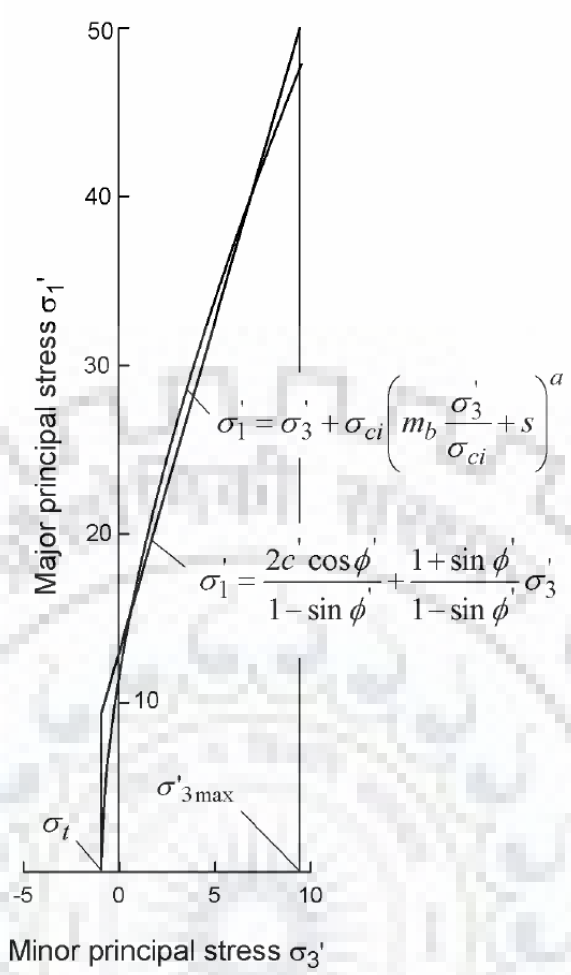


Figure 2.7 Hoek-Brown and an equivalent Mohr-Coulomb criterion

Callibration analysis has been carried out for various varying parameters of rock input properties for Hoek Brown failure criterion (such as GSI, m_i) to better understand how efficiently the Mohr Coulomb criterion curve is fitted with the Hoek Brown criterion using the above stated formula of equivalent *c'* and *Φ'* as suggested by Hoek et al. (2002). The results have been illustrated in the Figure 2.8 to 2.12 for a constant *GSI* and varying m_i and in the Figure 2.13 to 2.21 for a constant m_i and varying *GSI.*

It can be observed, from Figure 2.8 to 2.12 that as the m_i of the rock, which is indicative of the rock type is increasing (i.e. as the rock characteristic improves) the curvature of Hoek-Brown failure criterion curve increases and it loses agreement with the linear Mohr Coulomb failure criterion curve.

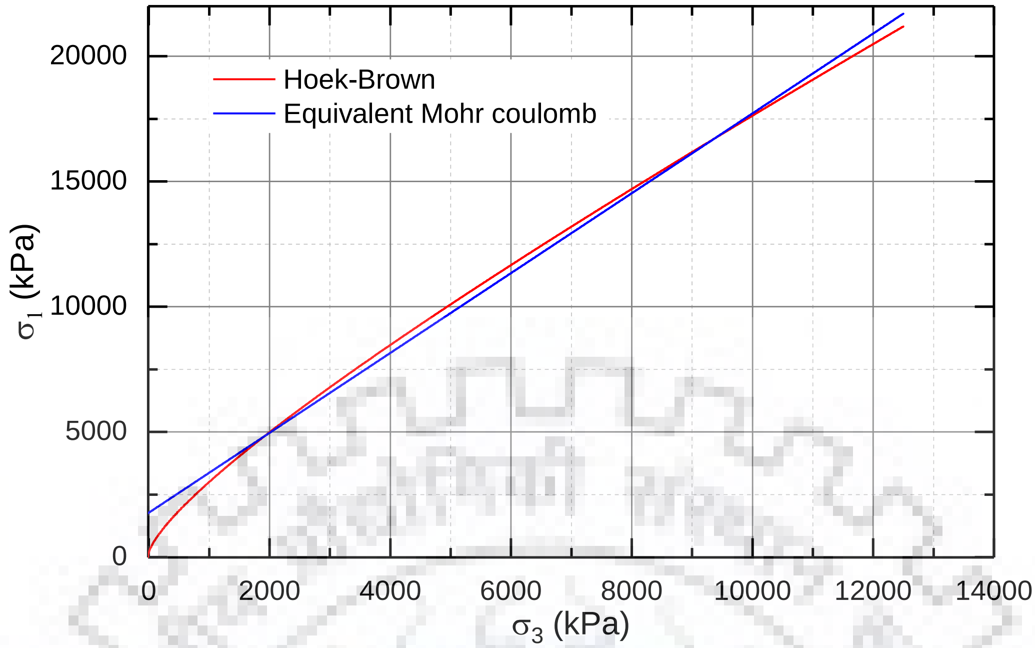


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Figure 2.8 Failure criteria in principle stress plane for GSI = 10 and m_i = 5

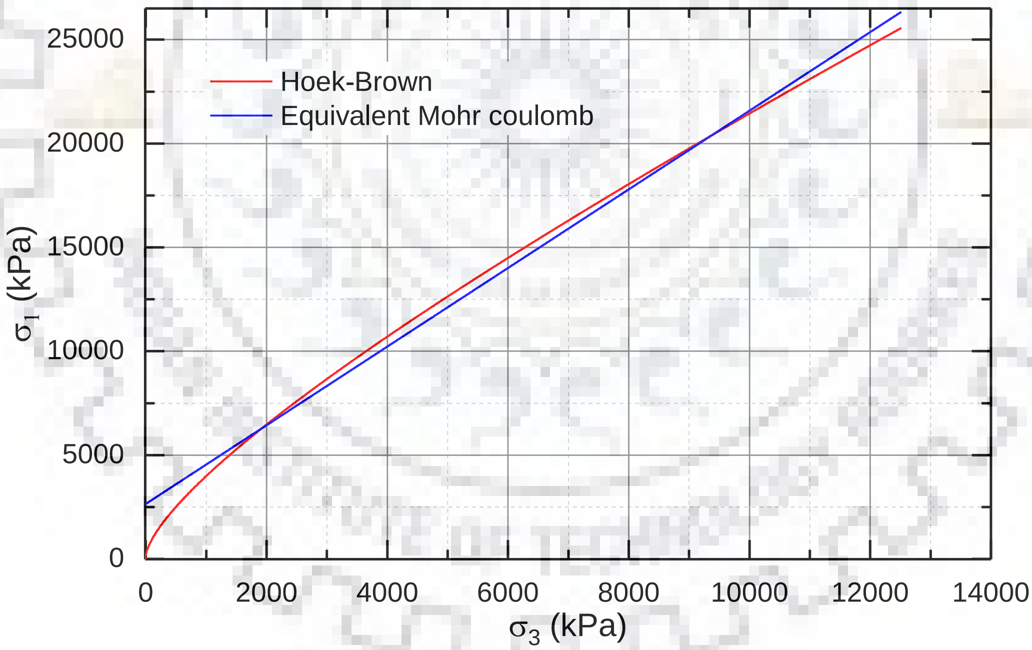

Figure 2.9 Failure criteria in principle stress plane for GSI = 10 and m_i = 10

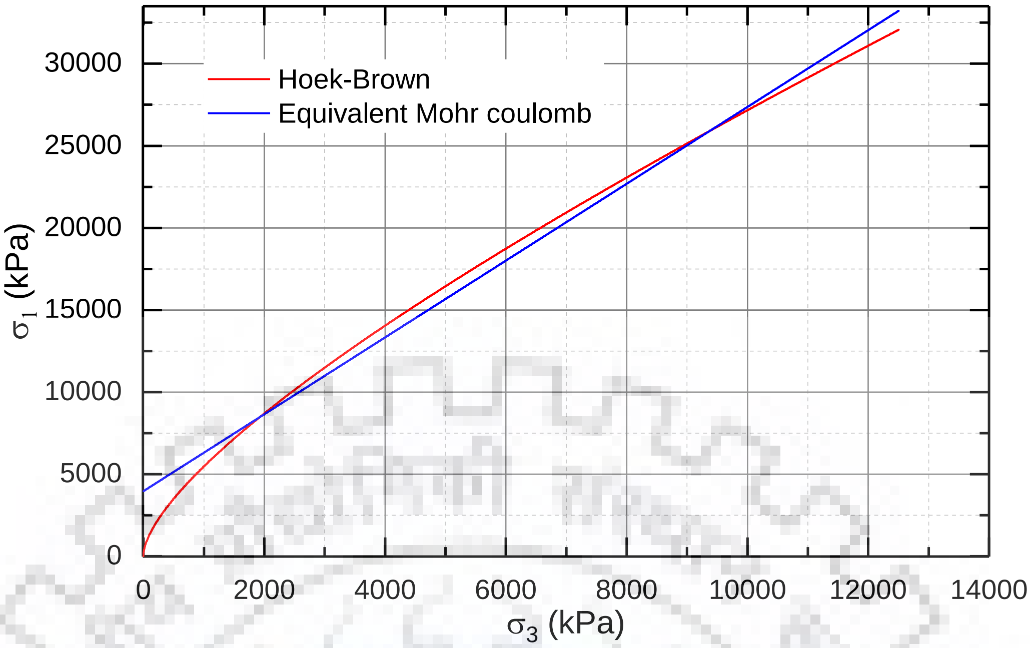

Figure 2.10 Failure criteria in principle stress plane for GSI = 10 and m_i = 20

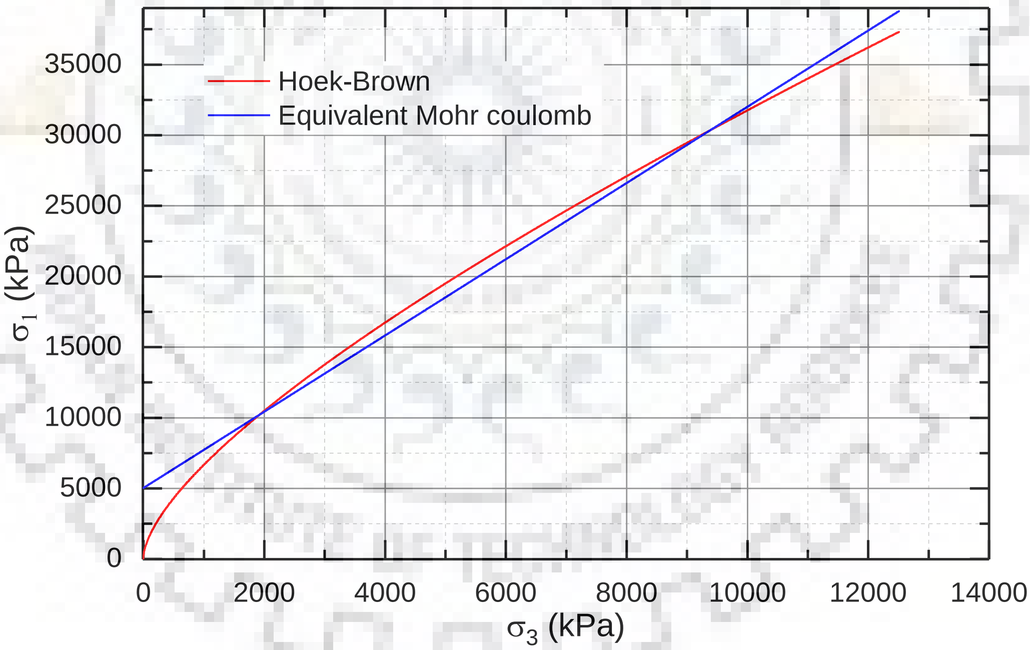

Figure 2.11 Failure criteria in principle stress plane for GSI = 10 and m_i = 30

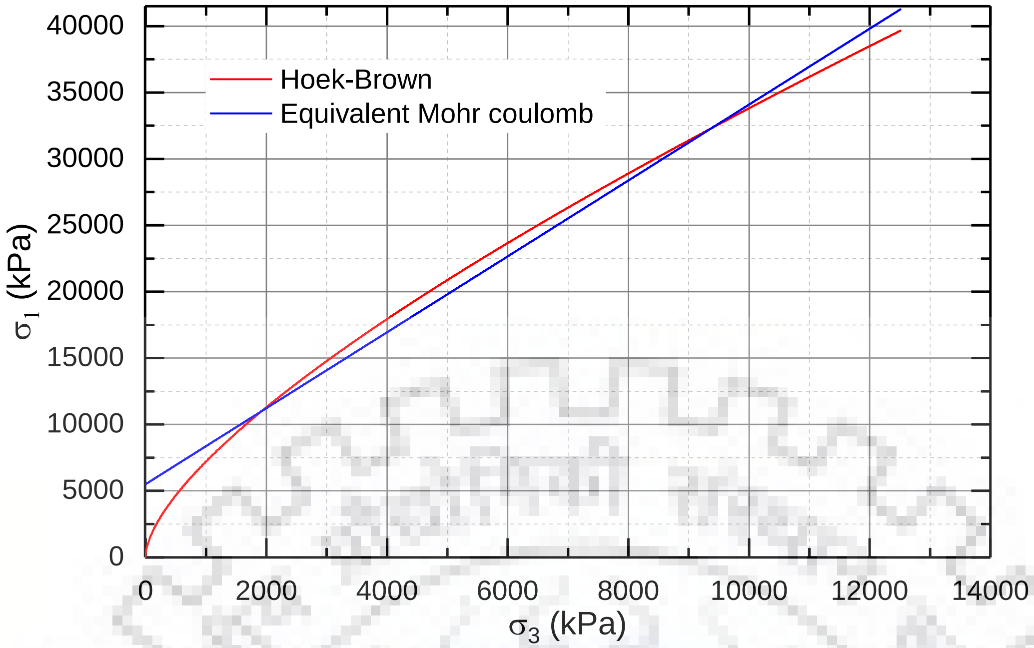

Figure 2.12 Failure criteria in principle stress plane for GSI = 10 and m_i = 35

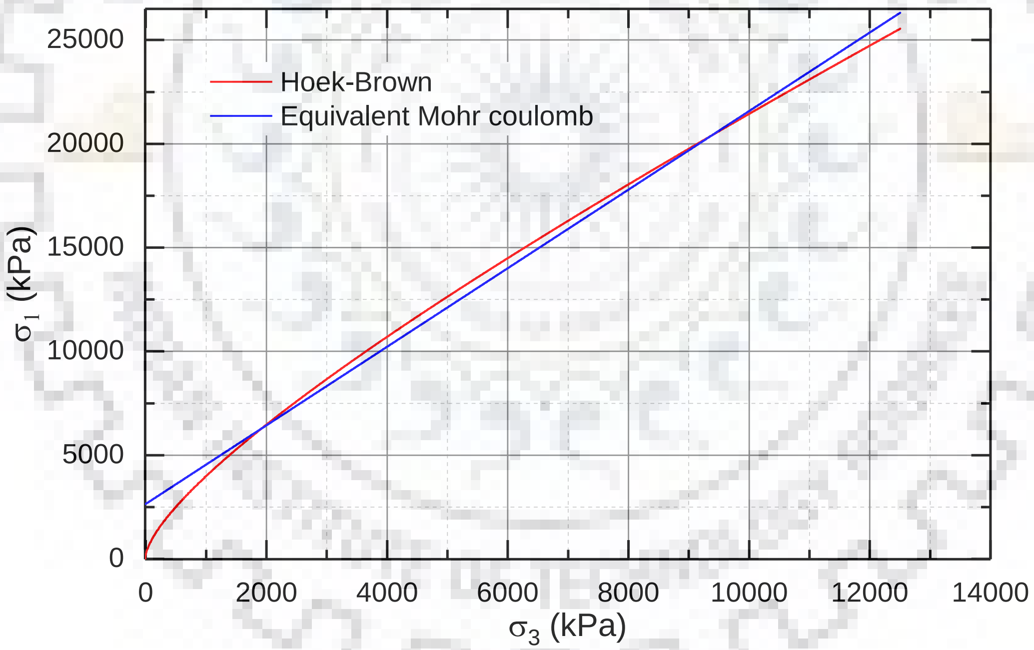

Figure 2.13 Failure criteria in Principle stress plane for m_i = 10 and GSI = 10

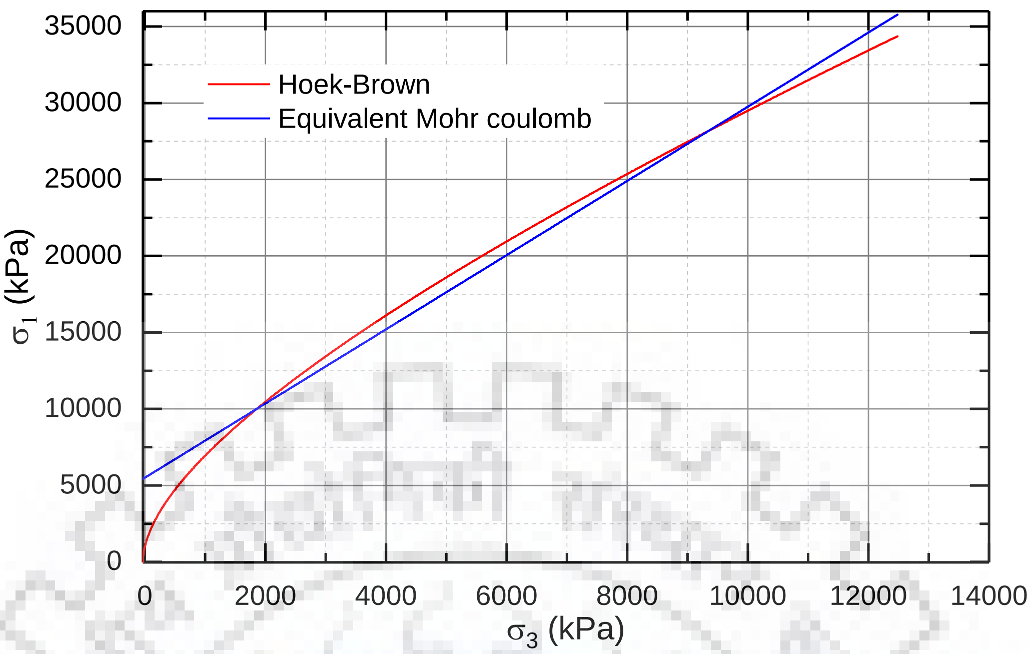

Figure 2.14 Failure criteria in principle stress plane for m_i = 10 and GSI = 20

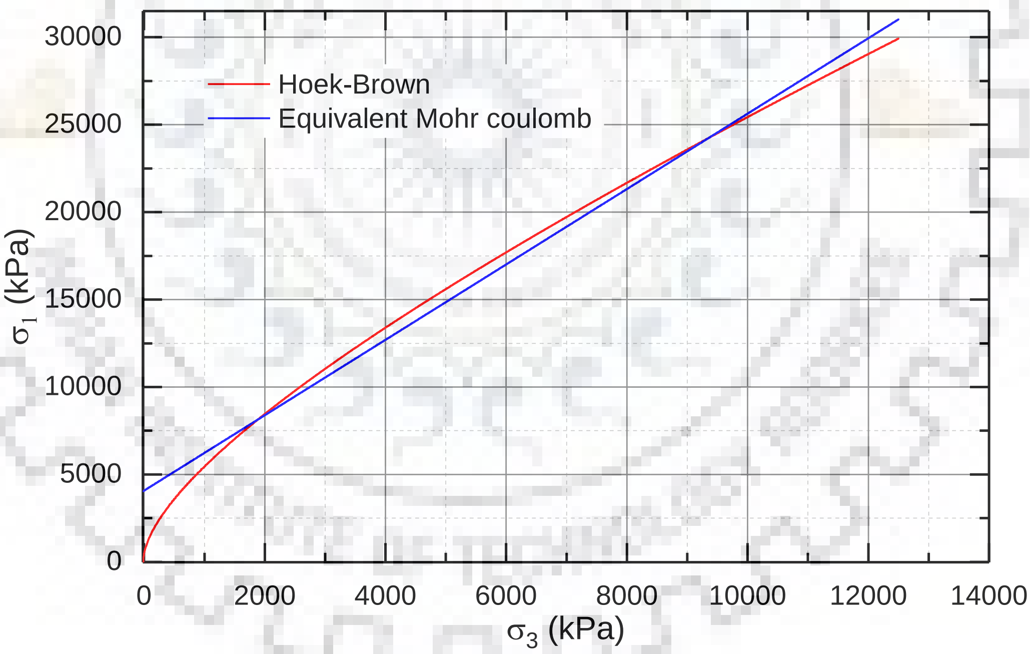

Figure 2.15Failure criteria in Principle stress plane for m_i = 10 and GSI = 30

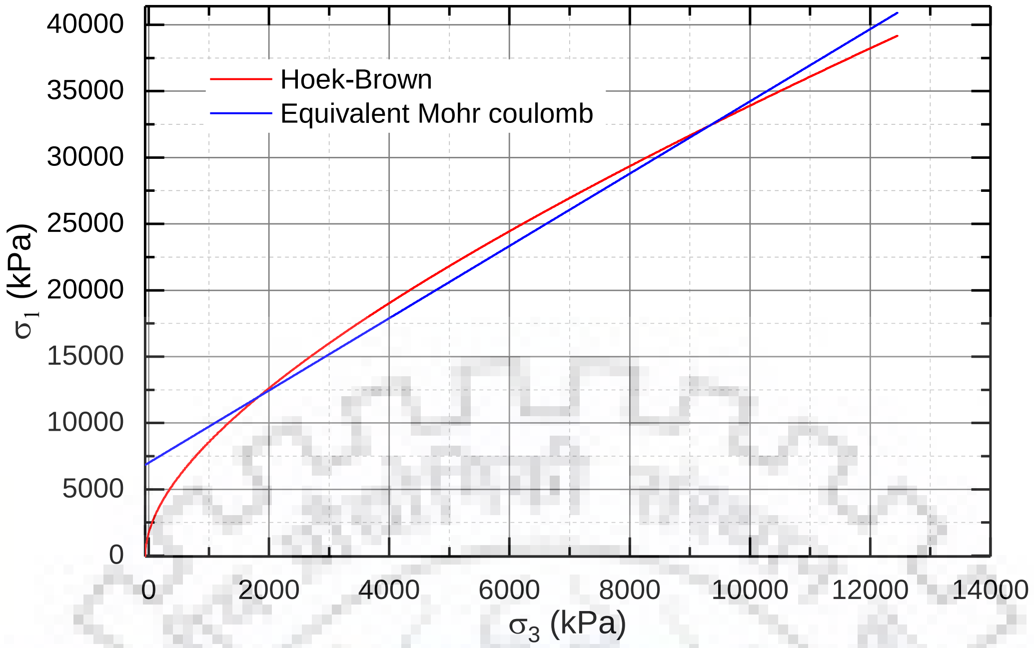

Figure 2.16 Failure criteria in principle stress plane for m_i = 10 and GSI = 40

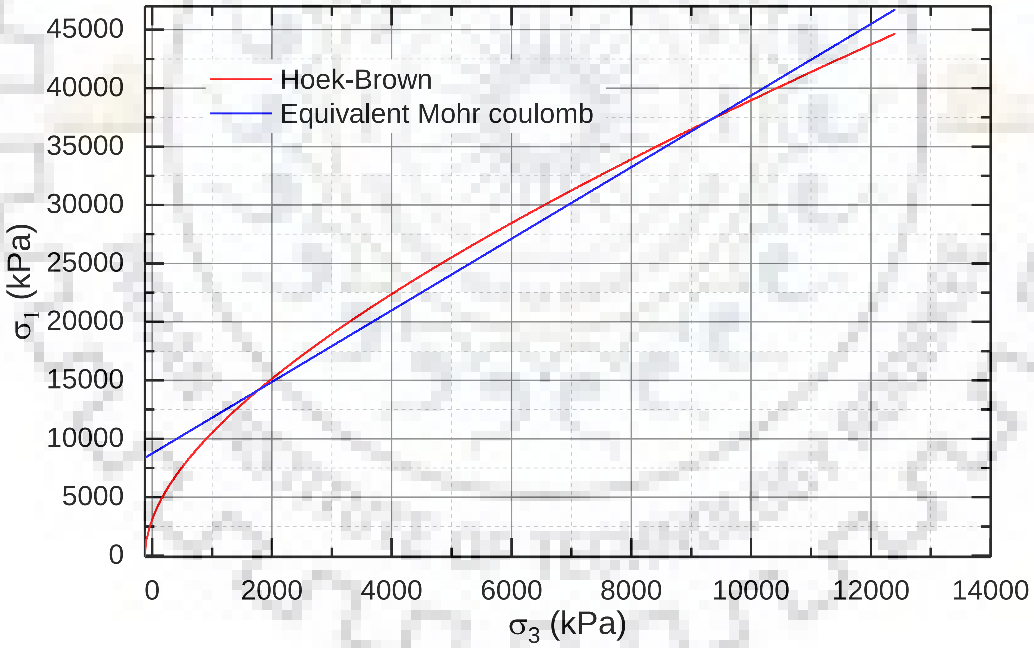

Figure 2.17 Failure criteria in principle stress plane for m_i = 10 and GSI = 50

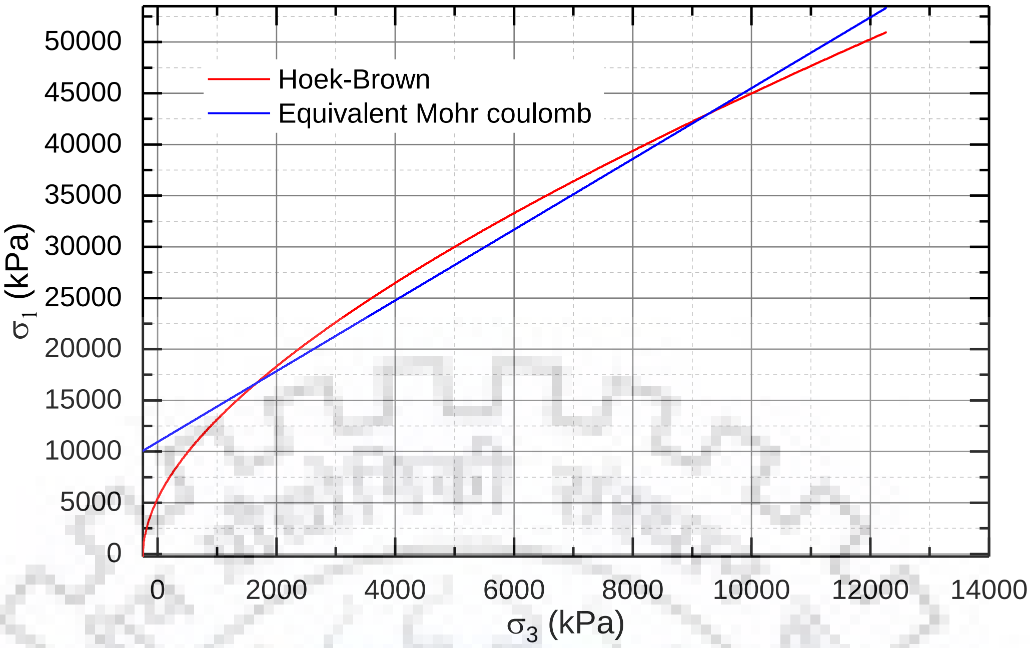

Figure 2.18 Failure criteria in principle stress plane for m_i = 10 and GSI = 60

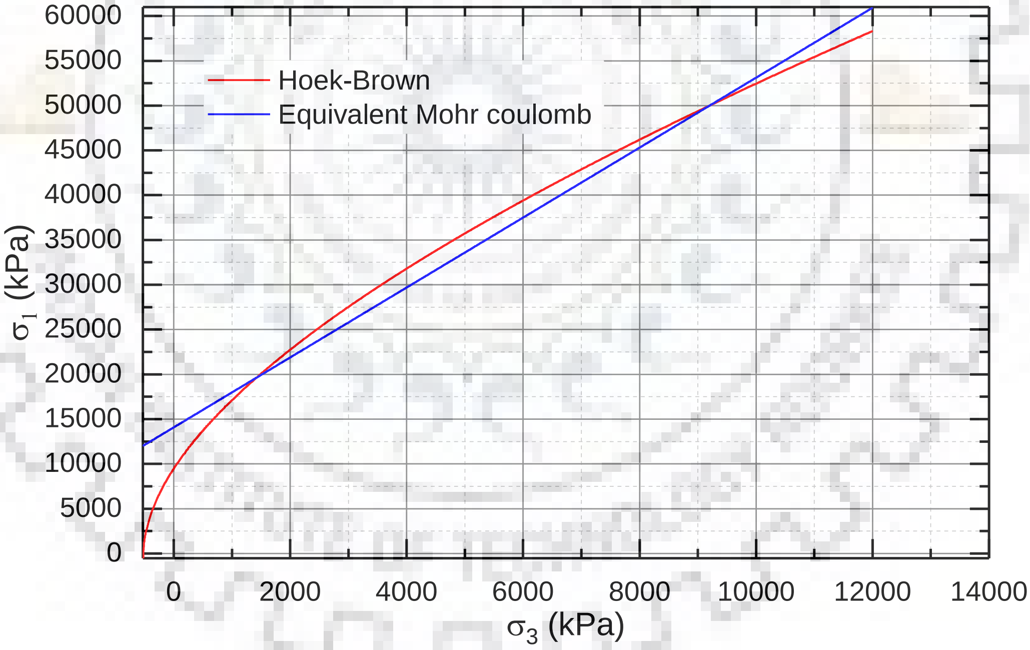

Figure 2.19 Failure criteria in principle stress plane for m_i = 10 and GSI = 70

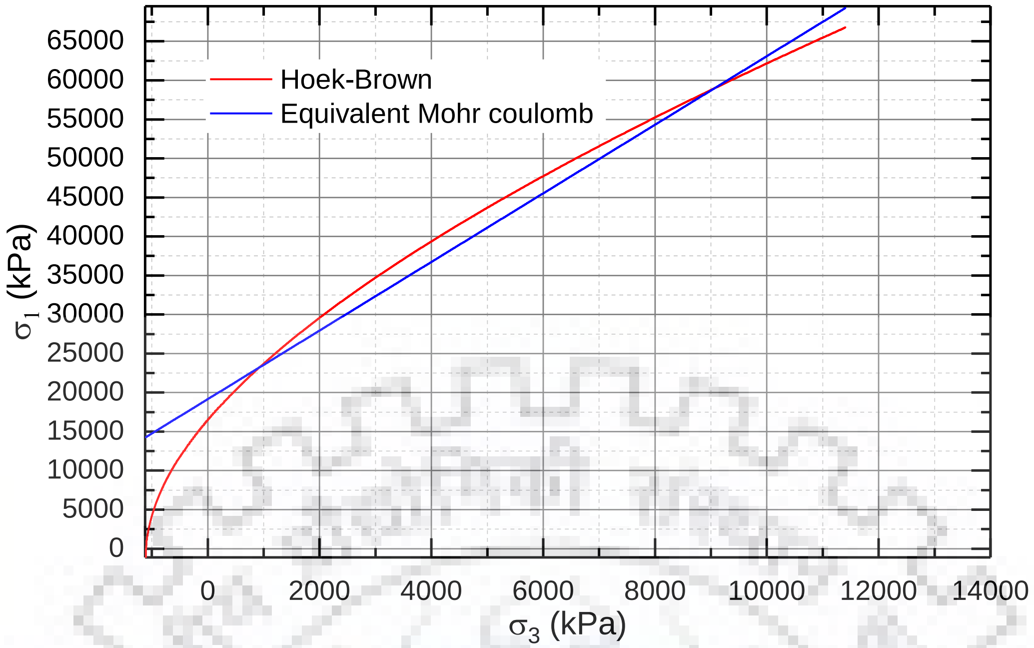

Figure 2.20 Failure criteria in principle stress plane for m_i = 10 and GSI = 80

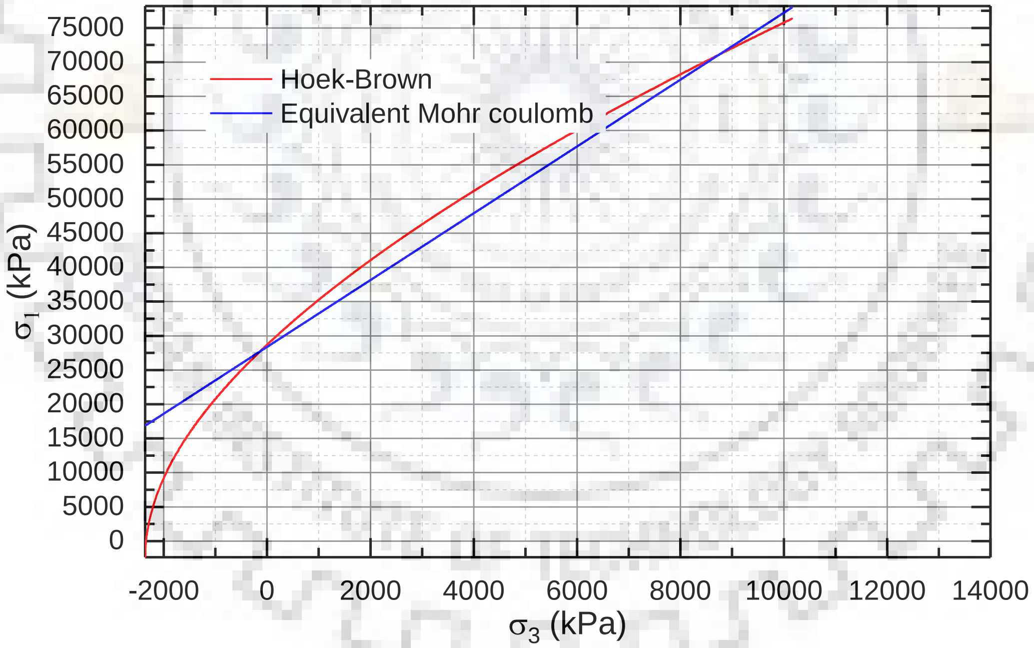

Figure 2.21 Failure criteria in principle stress plane for m_i = 10 and GSI = 90

From the Figure 2.13 to 2.21 it is evident that as the GSI value of the rock increases (i.e. the rock surface characteristics improve) the curvature of the Hoek Brown failure criterion graph increases and the agreement between the two graphs reduces. It can also be seen that with the increasing GSI the tensile strength of the rock also increases.

A concerning issue which can be seen from the above graphs is that in the values of σ_3 less than about 5 MPa (which are usually encountered in slope stability analysis, as the overburden is comparatively less than as compared to tunnel or underground structure) the equivalent Mohr Coulomb criteria overestimates the results by a significant amount. When compared to the results obtained from Hoek-Brown failure criterion. Under such circumstances, it would be prudent to use values of c' and Φ' based on a tangent to the shear strength curve in the range of σ_n where the overestimation is dominant. In this way the c' and Φ' values will be different in the different ranges of stresses. However, it is also a matter of concern as most of the geotechnical FEM software a constant c' and Φ' is entered and a continuously or discreetly varying c' and Φ' input in a geotechnical FEM software is a concept still to be explored.

Further, another concern in using the equivalent mohr-coulomb model is limiting the tensile strength of the equivalent model by using the tension cutoff feature in the various software it is necessary to consider this as the tensile strength of the rock is limited and is given by the equation:

$$\sigma_t = -s\sigma_{ci}/m_b \qquad (2.20)$$

If such a tension cutoff is not provided, then the results will be surely overestimated. The analyses with the provided tension cutoff was done and compared with the analysis results without the tension cutoff and it showed that by ensuring the tension cutoff the results got reduced.

Using the tangential linearization of the failure criteria. Under this section, a large part of the work has been focused on the methods available in the literature to convert the Hoek-Brown failure criteria from the sigma stress plane into the τ σ plane and then using the various tangential methods available to represent the failure criteria into a set of changing c and Φ value. In order to obtain this set of changing c and Φ the various literatures has been studied. Chen and Krakus (2012), Kumar (1998) and Priest (2005) were consulted to obtain the set of changing c and Φ parameters. Once this set of changing *c* and *Φ* is obtained, a computer code is required to be written to implement such a material model. Now analysis for the bearing capacity of rocks has been done under various models on the available commercial software named ABAQUS & OPTUMg2. The results of calibration were validated by the work of Merifield et al. (2006) and illustrated in the graph below:

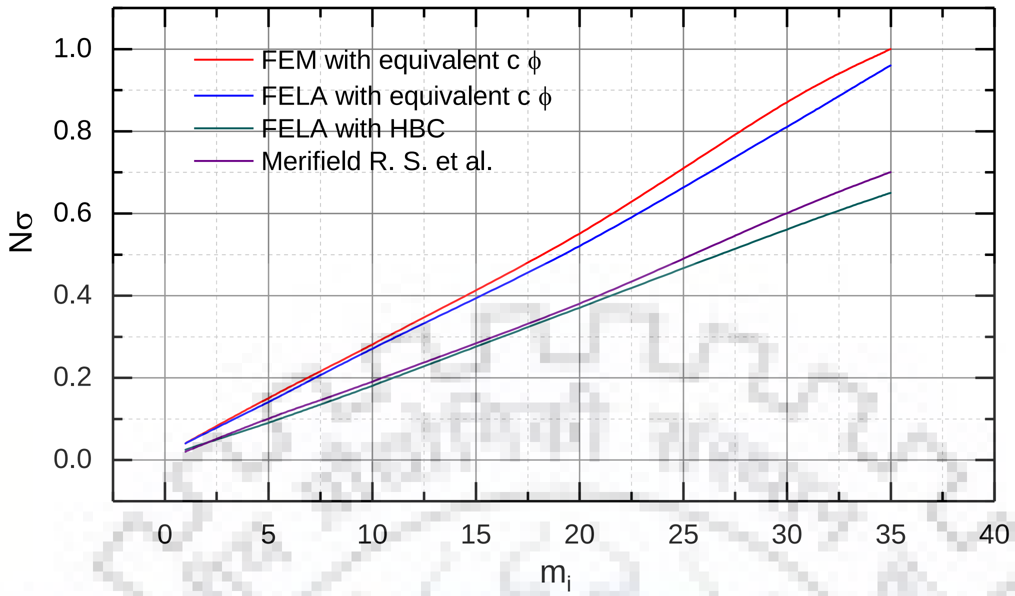

Figure 2.22Validation for the material model

CHAPTER 3: PARAMETRIC STUDY

3.1 Introduction

As it has been illustrated previously, the foundations constructed on hill slopes are more prone to failure as compared to foundations on flat land. Further, it is also observed that many hilly regions are also susceptible to frequent seismic activity.

Severe stability problems with the foundations arise when the reduced bearing capacity due to its position near the slope is coupled with the destabilizing effect of an earthquake. This effect has been widely reported in the past literature by Huang and Chen (2004); Huang (2005) and Huang and Kang (2008) based on post-earthquake site investigation and subsequent stability analysis after Chi-Chi earthquake (1999) in Taiwan. Tatsuoka et al. (1998) reported that during Hyogoken-Nambu (1995) earthquake, the gravity walls failed due to reduced bearing capacity prior to lateral sliding or overturning of the wall.

Currently, most of the standards and codes of practice (IS6403; EN1997-1 2004; NCHRP 2010) are largely silent on the estimation of seismic bearing capacity of foundations situated near the rock slopes. Several researchers (Saada et al. 2006; Merifield et al. 2006; Serrano and ollala 1994) have proposed the estimation of bearing capacity placed on flat rock. Also, there is ample literature for the estimation of bearing capacity of strip foundations on flat rock considering the seismic effects () and the estimation of bearing capacity of strip foundation on rock slopes () under static conditions. However, very limited literature is available for the estimation of bearing capacity of strip foundation located on rock slope (). There is no rigorous parametric study available, which can help in the development of guidelines for the estimation of the bearing capacity of foundation on rock slopes, particularly under the effect of seismic actions.

Most of the earlier studies consider the pseudo-static earthquake body forces within the rock-mass and are primarily based on: (1) the limit equilibrium method (2) the limit analysis method and (3) the method of stress characteristics. A consistent observation from the previous study is that the ultimate bearing capacity decreases quite significantly with an increase in horizontal seismic acceleration coefficient (α_h)

In most of the past studies, considering the foundations on rock slopes, the rock-mass is considered highly fractured, homogenous and isotropic in the analysis. Hoek-Brown failure criteria is widely used to define the yield in the elasto-plastic domain of the analysis. Both the Saada et al. (2010) and Yang (2018) derives the upper bound estimates of the seismic bearing capacity near the rock slopes in the limit analysis framework. In both these works a predefined geometry of failure mechanism has been assumed. Further the range of parameters investigated in these studies are limited. Thus it is difficult to establish a guideline for the practical purposes from them.

To overcome these limitations, the current study utilizes the Lower Bound (LB), Upper Bound (UB) formulations in Finite Element Limit Analysis (FELA) with Second Order Cone Programming (SOCP) and adaptive meshing technique based on shear dissipation, available in OptumG2 (2018) software is used. The results obtained from the numerical study are presented in the form of design charts for the practical usage. The variation in the failure patterns with the different governing parameters has also been explored in this study.

3.2 Problem Statement

The objective of the present study is to obtain the seismic bearing capacity factor, $N_{\phi s}$ for a strip footing placed on the top of the slope of rock-mass. The seismic force is considered as pseudo-static force, in terms of horizontal seismic coefficient, α_h , applied on the entire rock mass. The earthquake effect on superstructure has been considered as additional horizontal force on foundation. The factor $N_{\phi s}$ is obtained and investigated for the different value of horizontal seismic coefficient, α_h, β, $G.S.I.$, D, γ, m_i, σ_{ci} (where, β is the slope angle; $G.S.I$ and D are the in-situ parameters and γ, m_i, σ_{ci} are the laboratory parameters required to define the Hoek-Brown failure criteria)

The schematic representation of the slope foundation system considered in the present study is shown in fig. the magnitude of the collapse load $Q_{us}$is expressed in terms of a non-dimensional factor (Saada et al 2006), as given in equation 3.1

$$q_{us} = \frac{Q_{us}}{B} = \sigma_{ci} N_{\sigma s} \qquad (3.1)$$

Where, Q_{us} is the magnitude of the vertical load for the foundation in presence of pseudo-static horizontal seismic force, $N_{\sigma s}$is the seismic bearing capacity factor considering the effect of the unconfined compressive strength of the rock-mass.

The values of the factor $N_{\sigma s}$ is obtained for the highly fractured, homogenous, and isotropic rock-mass with the selected practical range of governing parameters: β = 10° to 80°; *GSI*= 10 to 90; *D*=0, 0.5, 1; m_i= 7, 10, 15, 17. 25; a dimensionless factor for the scale and density factor $\frac{\sigma_c}{\gamma B}$ = 125, 250, 500, 1000, 2000, 5000, 10000 and ∞; α_h = 0 to 0.5g, where *g* is acceleration due to gravity, have been presented here.

3.3 Finite Element Modelling

In the present study, 2D plane-strain nonlinear Finite Element (FE) model of rock slope with a strip foundation placed on its top has been developed. An elasto-plastic constitutive model based on Hoek-Brown failure criterion and following associated flow rule ($\psi = \phi$, where ψ is the dilation angle) has been used for modelling of rock-mass in FELA. In the present study, the rock-mass has been discretized using triangular elements with LB and UB formulations. The strip foundation has been modelled using 'plate' element. The two-node elastic plate element in plane-strain domain actually acts like a standard Euler-Bernoulli beam element. The strip foundation is considered as a rigid elastic material. The interface material between the foundation and the surrounding rock-mass is considered same as the rock-mass with zero tension cut-off (to simulate gap and uplift) and *R'*=1 (to simulate rough foundation) has been considered.

As shown in the fig., at the base of the FE model, the movement in both the directions (i.e. both X- and Y- displacements) are restrained, while for the side boundaries, only vertical displacement is allowed (i.e. X-displacement is zero). The lateral extent and dimensions of the FE model have been considered after carrying out the sensitivity analysis in such a way that the effect of boundary conditions on the domain of interest is insignificant. The adaptive meshing based on the shear dissipation has been employed. Three iterations of adaptive meshing with number of elements increasing from 8000 to 10000 have been used for all the analysis.

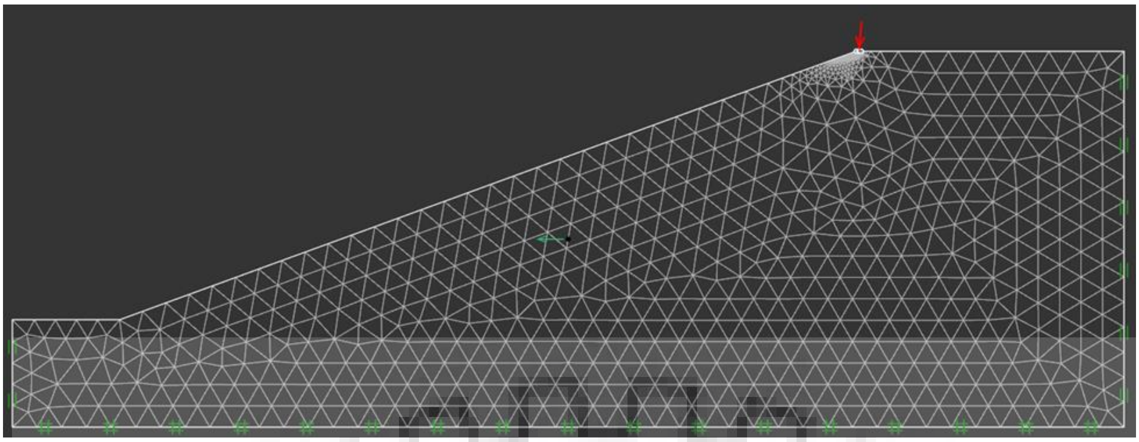

Figure 3.1 Finite element Model (in OptumG2 (2018))

To simulate the seismic effect, a pseudo-static acceleration has been applied on the entire rock-mass, fractioned with the horizontal seismic coefficient, α_h whereas on the foundation an additional horizontal force is applied which is proportional to the vertical load supported by the foundation.

A schematic flowchart of the various parameters to be investigated is shown in fig. 3.2.

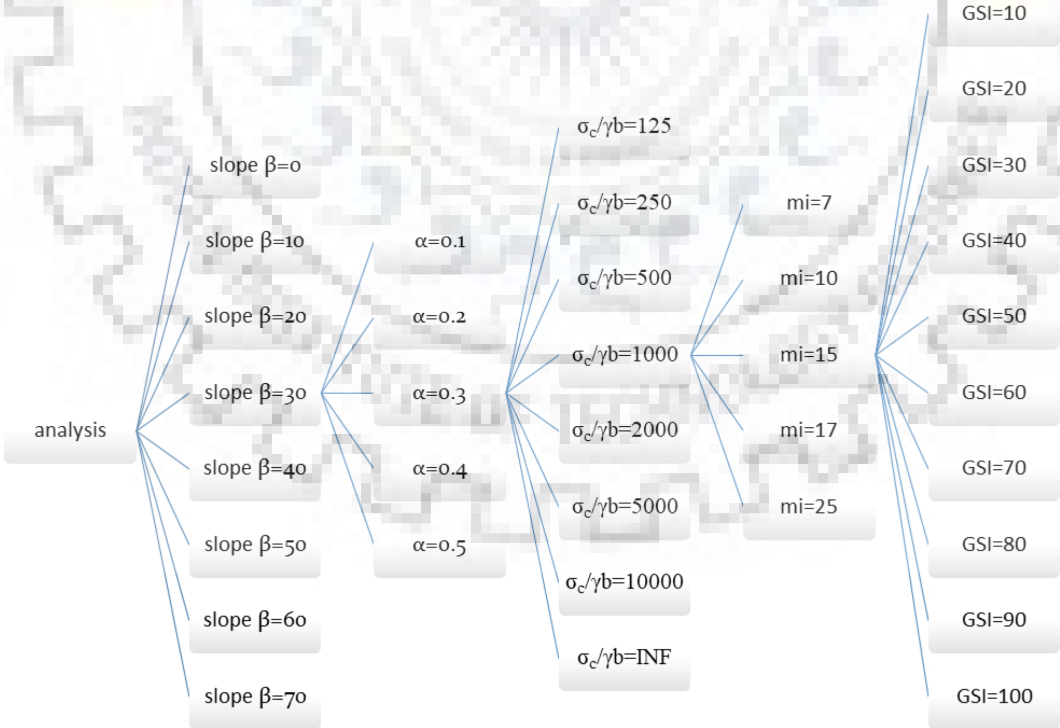

Figure 3.2 Schematic diagram of the parametric study

3.4 Model Validation

In order to validate the plane strain model being used in the analysis in the light of mesh size, type, domain, material constitutive law, etc. a simple model with the footing placed on a flat rock mass is considered. The UB and LB N_σ values are compared with the past studies and the comparison is illustrated with the help of a fig.3.3. In the current analysis both the UB and LB has been estimated and the average of the two is reported as the N_σ value.

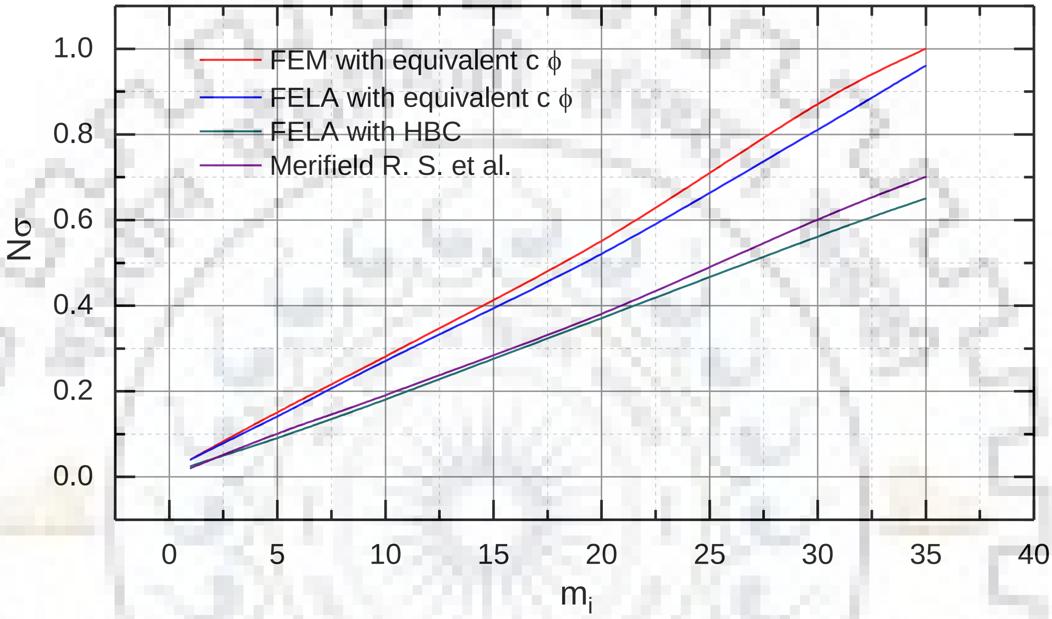

Figure 3.3 Model validation

3.5 Results and Discussion

An extensive numerical study has been performed and a large set of numerical results were available. Thus the results are broadly classified and illustrated in this section so that the effect of the various parameters defining the rock properties (*m_i, GSI, D)* as well as the other parameters incorporating the geometry of the slope (slope angle,β) and the seismic event (Horizontal seismic coefficient,α_h). The results are as follows:

3.5.1 Variation of $N_{\sigma s}$ with the rock type, m_i and disturbance factor, D

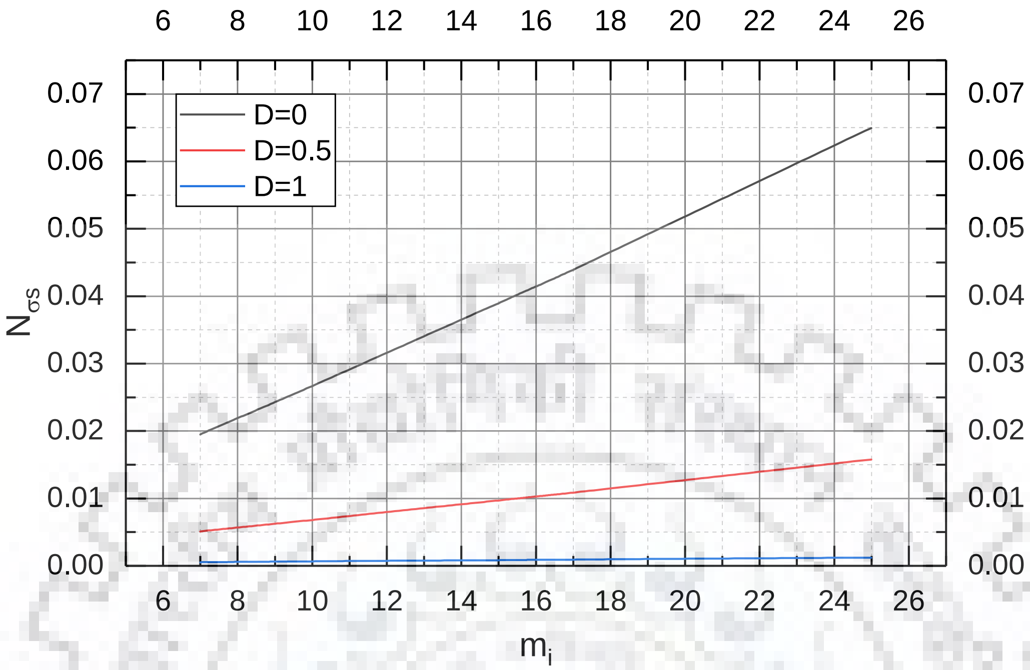

Figure 3.4 Variation of $N_{\sigma s}$ with the rock type, m_i and disturbance factor, D

It is evident from the above graph that the as the material property for the rock-mass increases from being soft rock (e.g. shales, sedimentary rocks), characterised by low value of m_i (such as 5,7,10) to the harder rocks (e.g. igneous rocks), characterised by high value of m_i the seismic bearing capacity factor increases.

This increase in value of the factor can be attributed to the higher collapse load carrying capacity of harder rocks. The harder rocks carry higher collapse load because of the inherent higher failure capacity of such rocks, contributed by the better microstructural arrangement of such rocks.

It is also evident from the graph that, as the disturbance factor for the rock increases its seismic bearing capacity reduces. The disturbance factor is an in-situ parameter for the state of rock at the site due to blasting operations carried on the site in history. The higher value of D is an indicator of greater disturbance in the rock matrix due to the blasting operation on the site.

This disturbance of the matrix due to the blasting operation leads to loss of interlocking strength within the rock-matrix. This leads to a lower failure capacity. thus leading to lower collapse load and hence a lower capacity factor.

3.5.2 Variation of $N_{\sigma s}$ with Geological Strength Index *GSI*

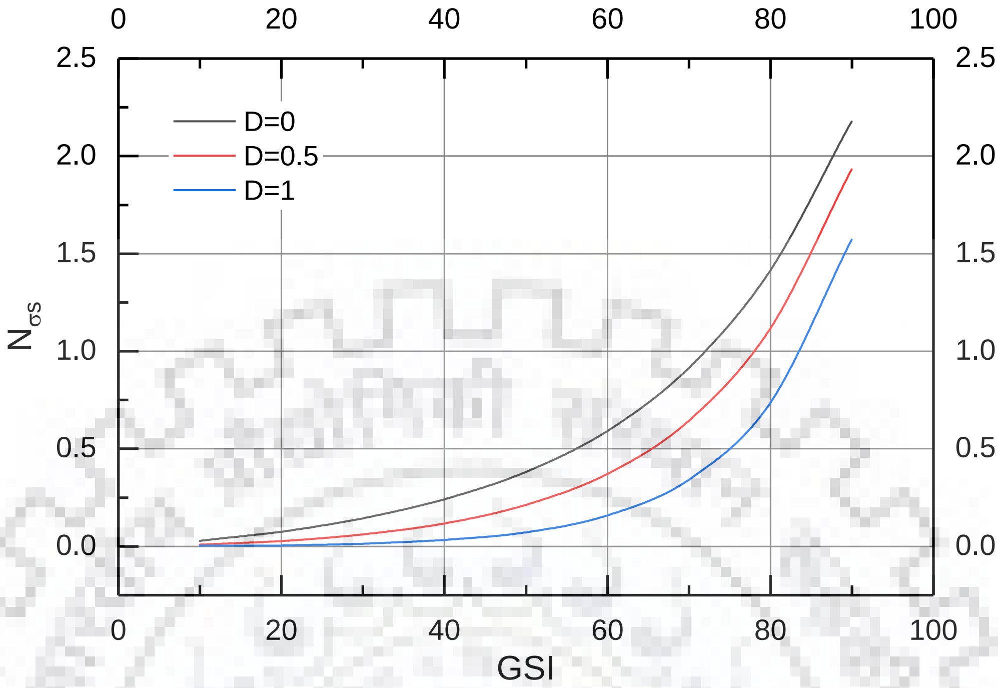

Figure 3.5 Variation of $N_{\sigma s}$ with Geological Strength Index *GSI*

It is evident from the graph that, with the increase in the *GSI* the bearing capacity factor increases for all ranges of *D*. The *GSI* value is also an in-situ parameter depicting the state of rock at the site. The parameter depicts the state of fracture within the rock matrix.

The higher the value of *GSI* the higher is the fracture within the rock-mass. the state of fracture within the rock mass is a major factor defining the strength of the rock-matrix. The higher is the fracture within the rock mass the lower is the failure strength criteria. Thus the collapse load decreases with the increase in the fracture state of the rock.

The *GSI*is different from the disturbance factor in a manner that, it is an indicator of the fracture state due to the natural existence of the rock. Whereas the disturbance factor is the state of disturbance due to the man-made blasting activities carried out at site during tunnelling or road cutting operations.

3.5.3 Variation of $N_{\sigma s}$ with horizontal seismic coefficient α_h and scale factor $\frac{\sigma_c}{\gamma B}$

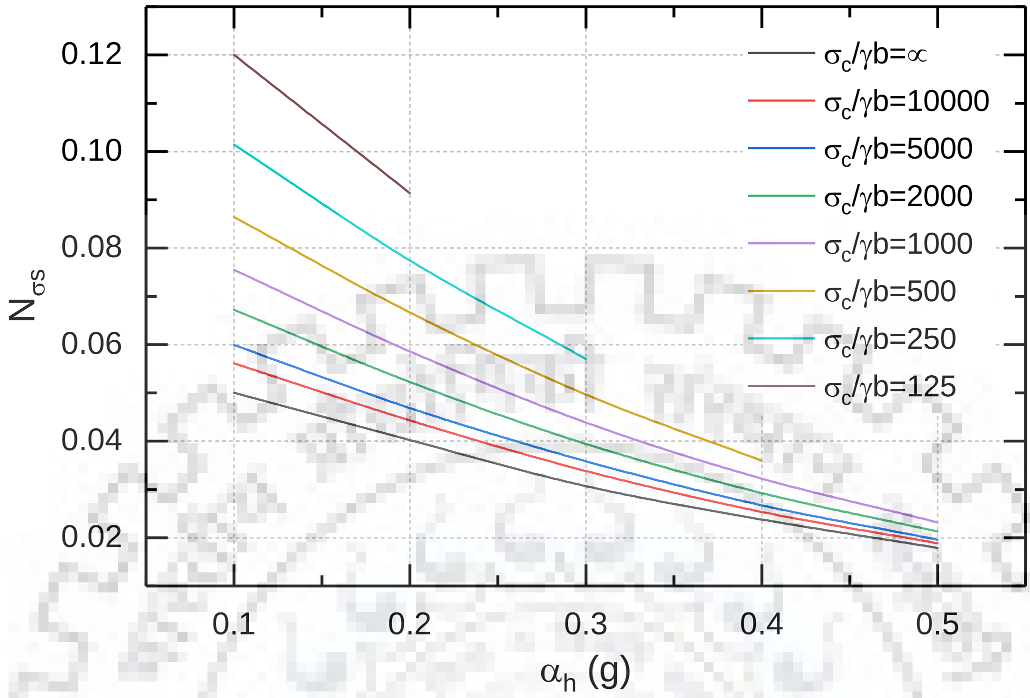

Figure 3.6 Variation of $N_{\sigma s}$ with horizontal seismic coefficient α_h and scale factor $\dfrac{\sigma_c}{\gamma B}$

The scaling factor is a factor which accounts for the area of failure surface and mass of the geo-material being involved in the failure mechanism. The lower is the value of the scale factor the larger is the mass and the area involved in the mechanism.

This suggests the truncation of the bearing capacity charts for the lower value of the scale factor. For lower values since the mass involved in the mechanism is high thus under the effect of horizontal seismic action through pseudo static methods, the slope itself fails.It then becomes a slope stability issue for such low values of the scale factor and thus the bearing capacity calculation in such a case is irrelevant and hence the graph is truncated.

The effect of horizontal seismic coefficient is clearly visible, that by increasing the value of α_h, the factor reduces.

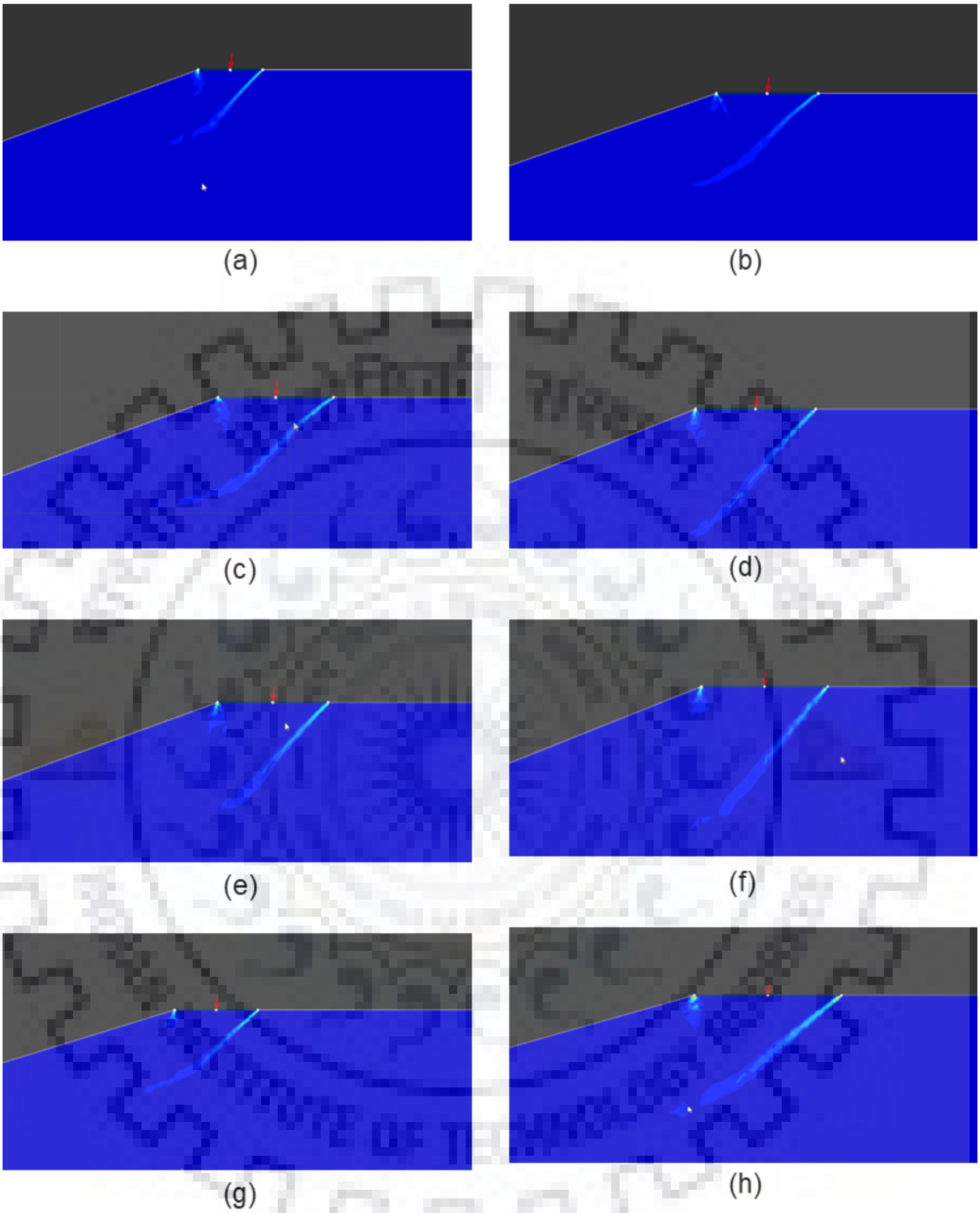

Figure 3.7 Shear dissipation contours

CHAPTER 4: DESIGN CHARTS

In this particular chapter, a sample set of design charts for the estimation of bearing capacity factors for different slopes is provided:

4.1 For 10° slope

In the given section, the charts are developed. For different values of disturbance factor *D* (0, 0.5, 1). For different values of rock type m_i (7, 10, 15, 17, 25) For different *GSI* (10, 20, 30, 40, 50, 60, 70, 80, 90) are given for the estimation of bearing capacity factor for different combinations have been illustrated in the Figure 4.1 to 4.3.

4.2 For 20° slope

In the given section, the charts are developed. For different values of disturbance factor *D* (0, 0.5, 1). For different values of rock type m_i (7, 10, 15, 17, 25) For different *GSI* (10, 20, 30, 40, 50, 60, 70, 80, 90) are given for the estimation of bearing capacity factor for different combinations have been illustrated in the Figure 4.4 to 4.6.

The design charts can be effectively used to estimate the seismic bearing capacity factors. It can prove to be an effective tool for the design engineers.

The result interpretation through graphs and design charts lead to a very large set of document to be investigated in order to achieve the required engineering judgement related with the bearing capacity problem. This can be a cumbersome task and thus to minimize this effort, a better data interpretation technique needs to be employed which is more concise and easy to use.

It was an observation that the rock-mass being a complex geo-material is defined by a large number of input parameters thus the results of the numerical parametric study involving the estimation of seismic bearing capacity factor $N_{\sigma s}$ is returned with a very large set of numerical result data.

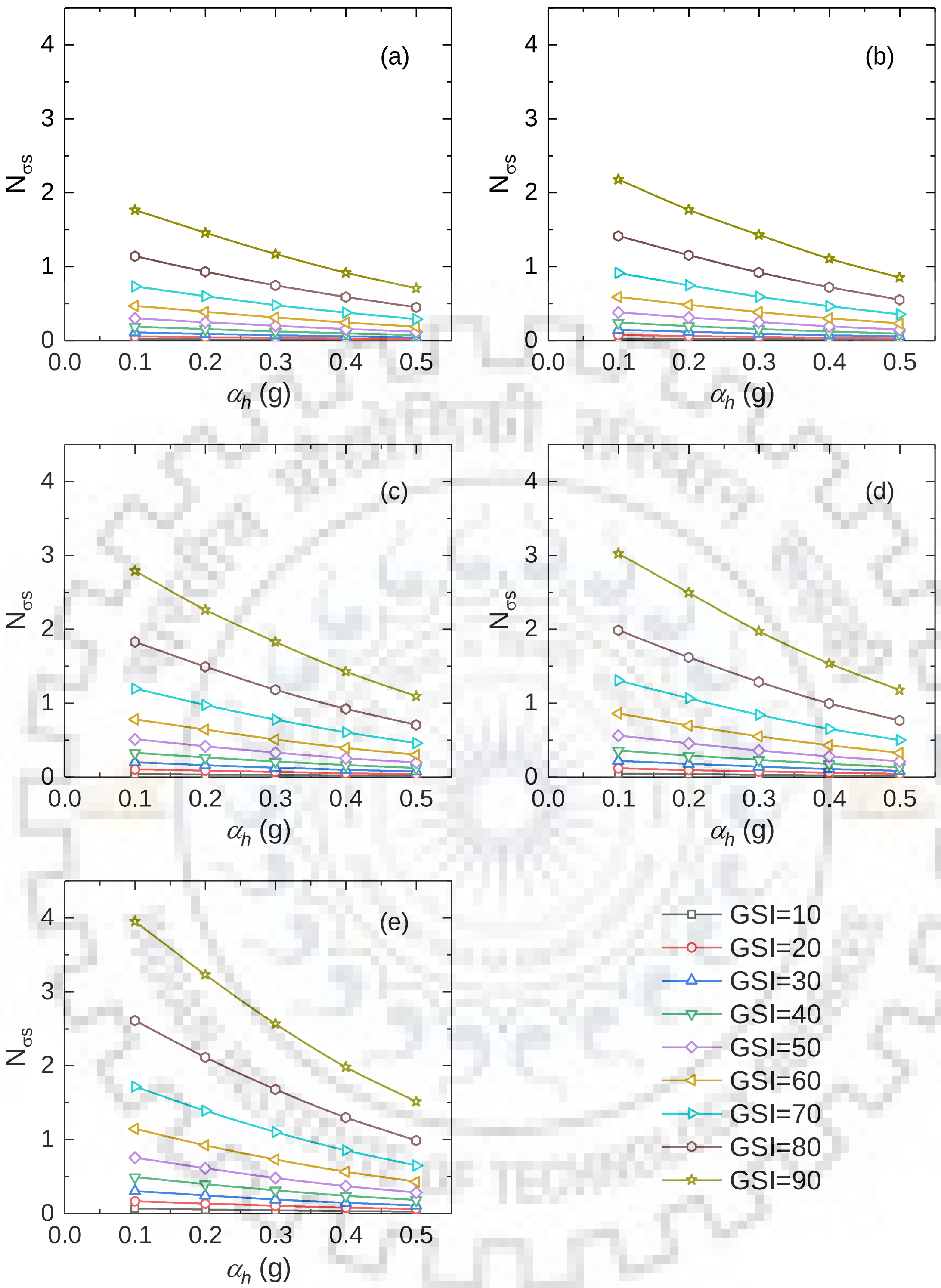

Figure 4.1 Design charts for 10˚slpoe and D=0, (a) m_i=7, (b) m_i=10, (c) m_i=15, (d) m_i=17 and (e) m_i=25

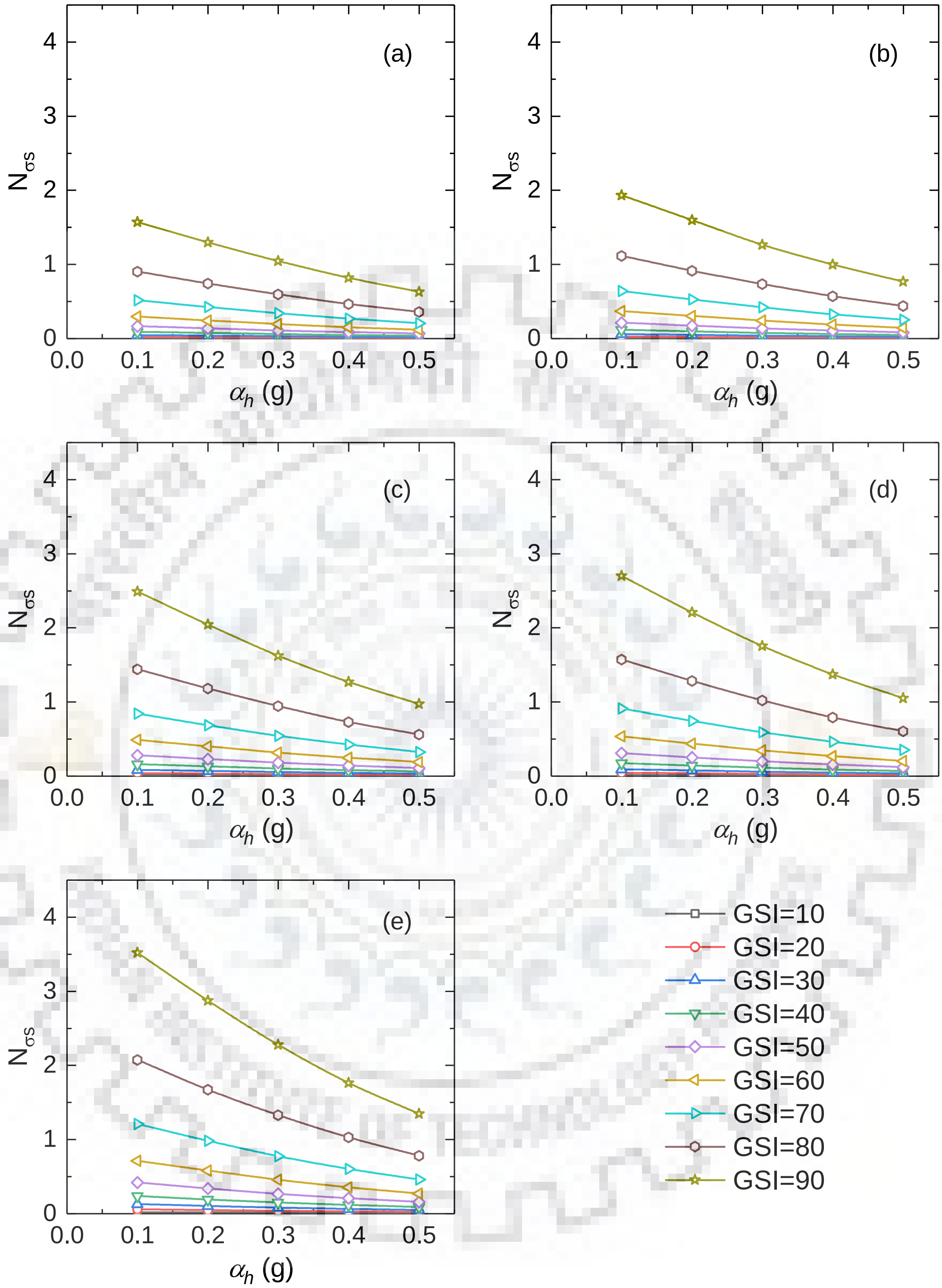

Figure 4.2 Design charts for 10˚slpoe and D=0.5, (a) m_i=7, (b) m_i=10, (c) m_i=15, (d) m_i=17 and (e) m_i=25

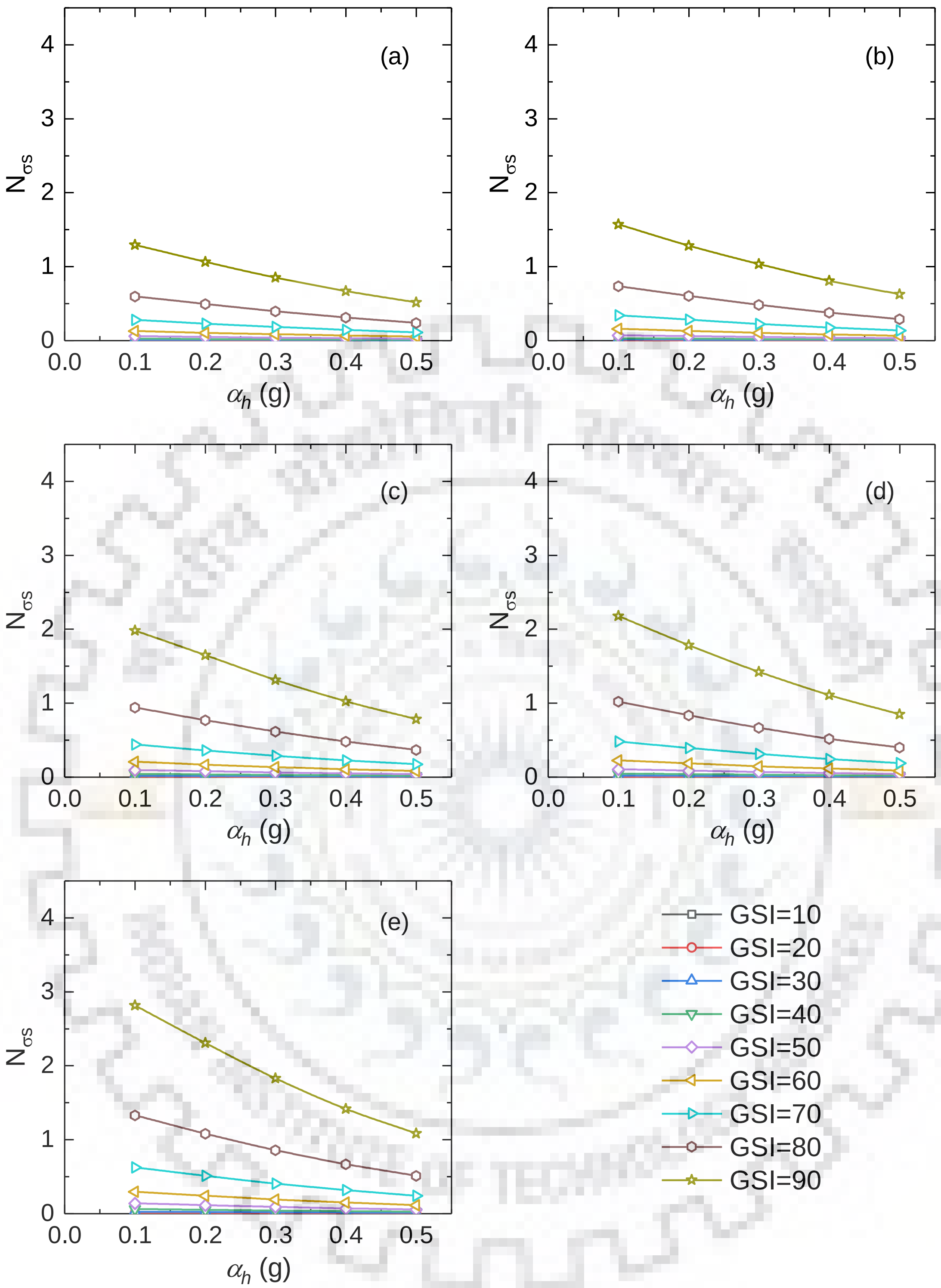

Figure 4.3 Design charts for 10˚slpoe and D=1, (a) m_i=7, (b) m_i=10, (c) m_i=15, (d) m_i=17 and (e) m_i=25

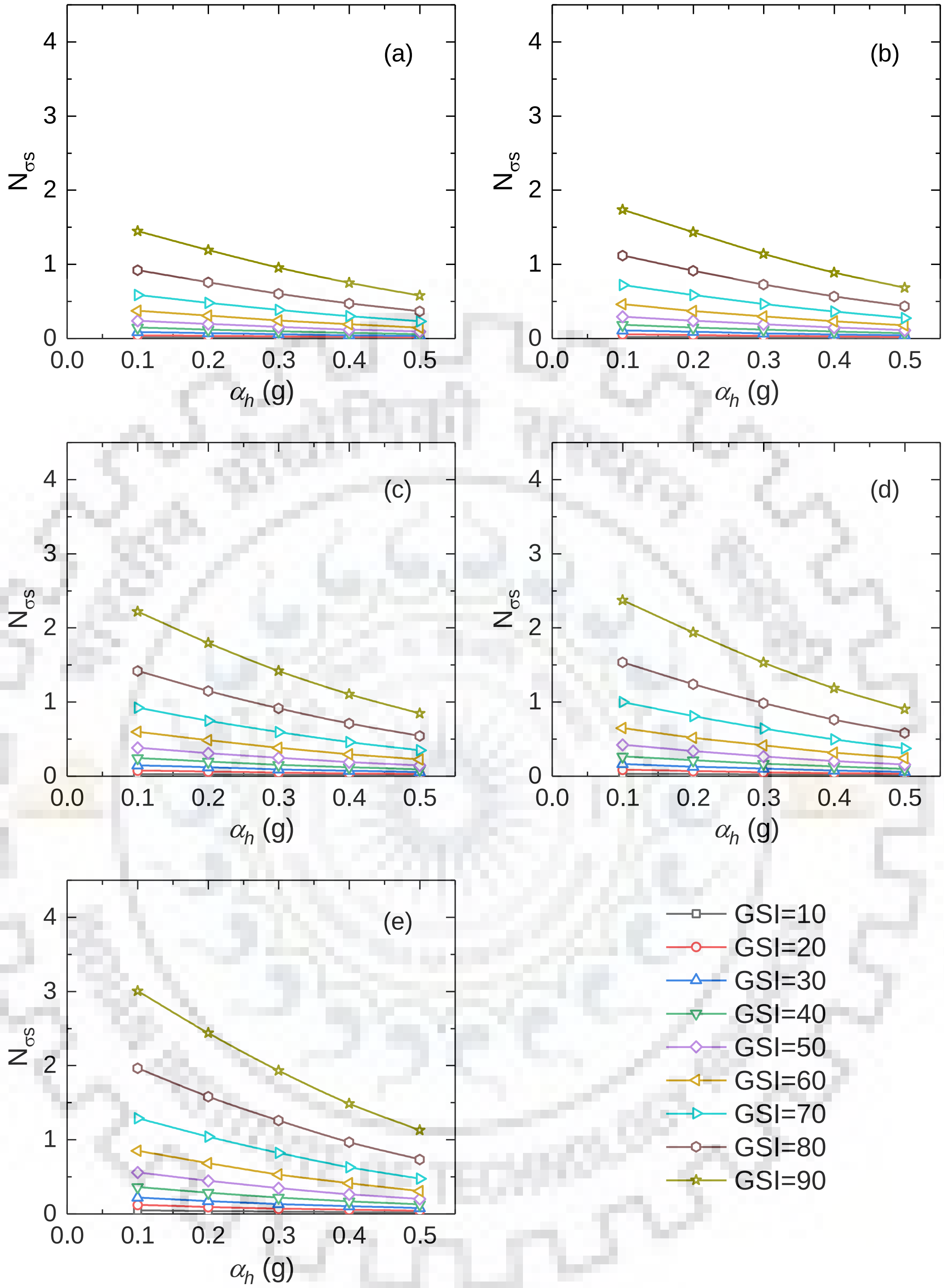

Figure 4.4 Design charts for 20˚slpoe and D=0, (a) m_i=7, (b) m_i=10, (c) m_i=15, (d) m_i=17 and (e) m_i=25

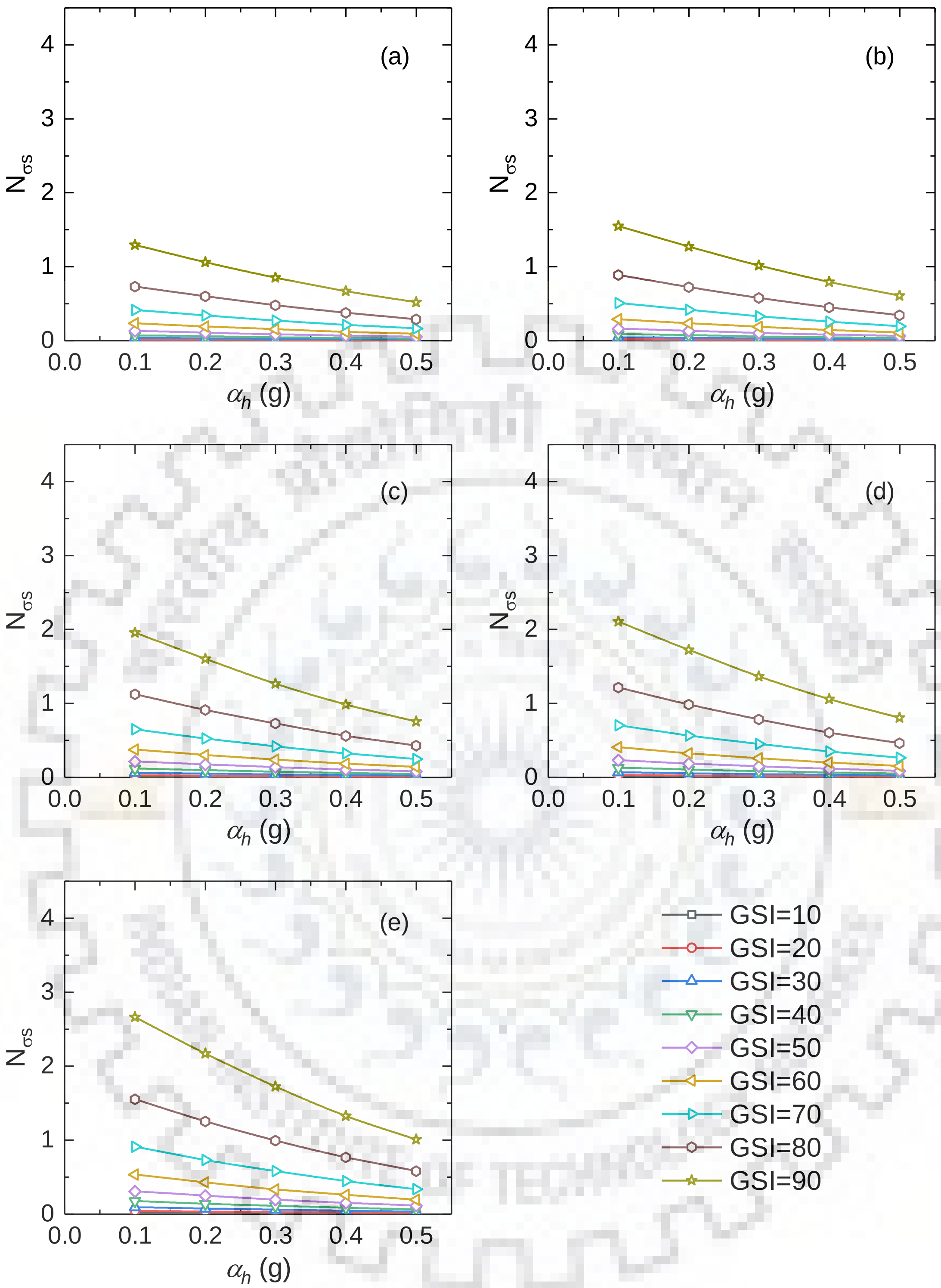

Figure 4.5 Design charts for 20˚slpoe and D=0.5, (a) m_i=7, (b) m_i=10, (c) m_i=15, (d) m_i=17 and (e) m_i=25

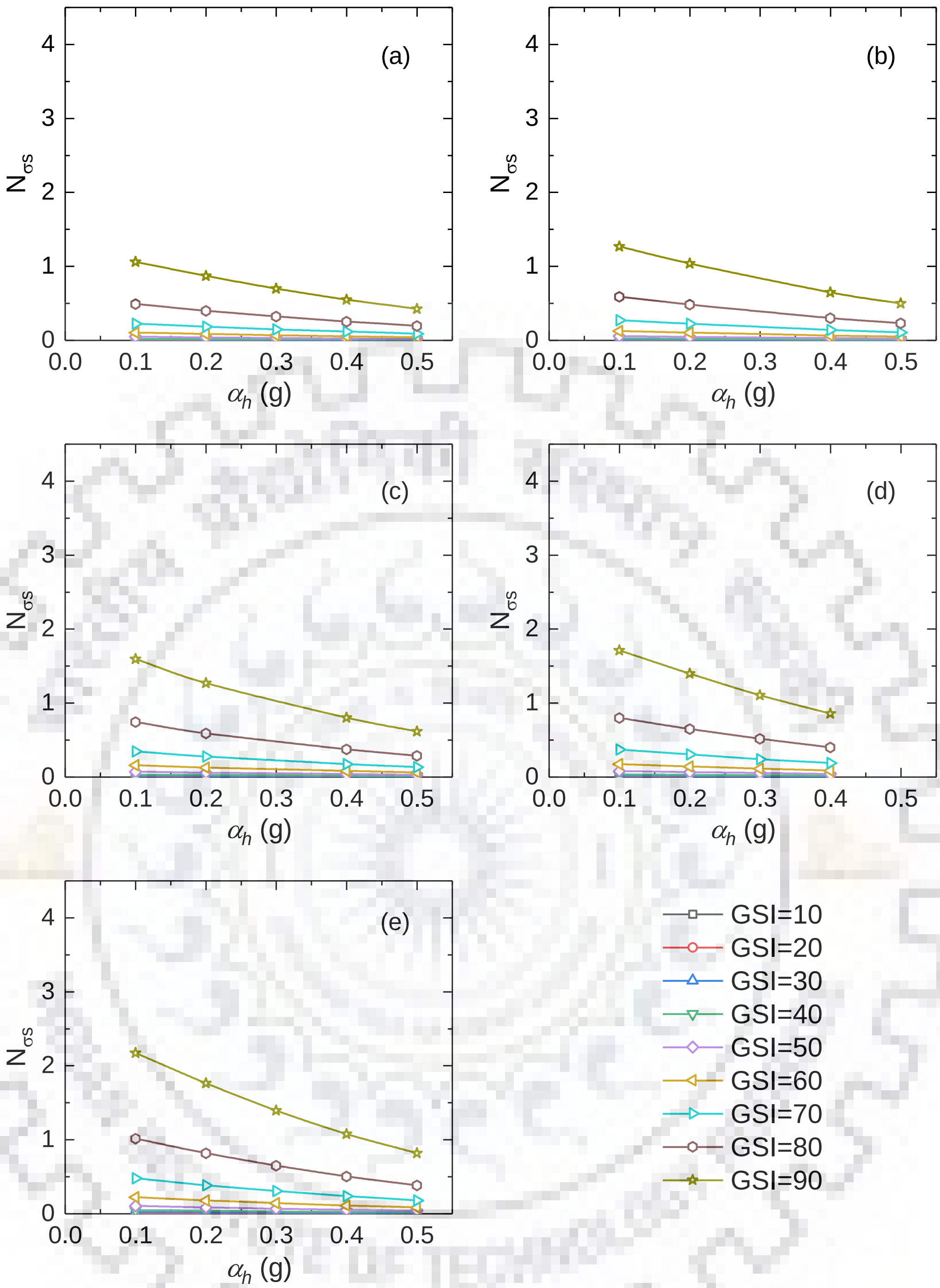

Figure 4.6 Design charts for 20˚slpoe and D=1, (a) m_i=7, (b) m_i=10, (c) m_i=15, (d) m_i=17 and (e) m_i=25

CHAPTER 5: CONCLUSIONS AND FUTURE SCOPE

5.1 Conclusions

The following major conclusions has been drawn from the study reported in earlier chapters:

1. The simulation of rock in any numerical technique scheme requires a much more vigilant approach than any other geo-material. To reliably assess the natural state of rock to be analysed a plethora of experimental techniques (both in-situ and laboratory observations) is required. Depending upon the observed state of rock-mass an appropriate material model is to be selected such that it effectively simulates the actual behaviour of rock.
2. To simulate the highly fractured, homogenous and isotropic rock-mass an elasto-plastic material model based on the widely used Hoek-Brown failure criteria has been found to give satisfactory results. In case of a low-stress range problem, such as bearing capacity problem, the equivalent c-ϕ approach as suggested by Hoek, 2002 has been found to overestimate the results when compared with the results of actual Hoek-Brown material model. Thus it is suggested that this equivalent technique should be restricted to the high stress range problems involving high overburden, such as tunnels.
3. It is an observation that the rock-mass is a complex geo-material defined by a large number of input parameters. Thus the results of the numerical parametric study involving the estimation of seismic bearing capacity factor $N_{\sigma s}$ is returned with a very large set of numerical result data. Here in this work, an effort has been made to illustrate the effects of the characteristic parameters with the help of the design charts and graphs and the observations are discussed. For a large numerical data a more robust and advanced techniques involving probabilistic as well as neural techniques is required.
4. It can be concluded that the FELA method proves to be a very efficient tool in the estimation of a well bracketed collapse load in case of an elasto-plastic analysis. It is a quick and robust tool and thus can be effectively employed to carry out high volume parametric study such as the one in this study. It has its limitation and has been well illustrated in the next section. Further, the of use pseudo-static method to incorporate the seismic event is found to work

satisfactorily in the case of bearing capacity estimation and the slope stability problems. However, its limitation is well illustrated in the next section to follow.

5. From the parametric study it was concluded the seismic bearing capacity varies in the following manner:

   i. Increases with the parameter m_i
   ii. Decreases with the increase in the parameter D
   iii. Increases with the increase in GSI

5.2 Limitations of The Work

In the current work, the limitations associated with the methods of analysis employed are well marked and thus opening a clear window for the recommendations for the future work. The limitations are illustrated as below:

Firstly, the result interpretation through graphs and design charts lead to a very large set of document to be investigated in order to achieve the required engineering judgement related with the bearing capacity problem. This can be a cumbersome task and thus to minimize this effort, a better data interpretation technique needs to be employed which is more concise and easy to use.

Secondly, the utility of FELA in the analysis has its own limitations. The FELA has a limitation that only an associated flow-rule can be implemented thus it is not possible to simulate the dilation features in the rock-mass. Further the FELA is limited to the estimation of collapse load only thus the benefits of a conventional displacement-based finite element methods cannot be utilised. Thus FELA has a limited scope and in order to know the load-displacement curve, carry out the time history analysis, etc. a conventional displacement based finite element framework is required.

Lastly, the pseudo static methods employed to incorporate the seismic effect has its own limitation. It is a conservative method and it may be a case that even if the load exceeds the seismic bearing capacity in case of an earthquake it may not lead to an absolute failure. The case absolute failure or the loss of serviceability is by and large also dependant on the displacement caused in the critical case. Thus it is a necessity in the performance based design framework that certain engineering demands are investigated by carrying out the more robust techniques to incorporate the seismic effect such as time history analysis.

5.3 Recommendations for Future Work

Considering the scope and limitations of the present work the following recommendations are made for the future work:

Firstly, the numerical results need to be presented in a more concise and simple to use way. This may involve development of a simple spreadsheet based tool. Also there is a scope of involving the Artificial Neural Network ANN which can train itself from the data available to it to find the meaningful trends in it. Further in case of rocks, as the probability of presence a particular natural state is not high enough and is largely uncertain. Thus a probabilistic framework can be efficiently employed for predicting the input parameters describing the rock property.

Secondly, since the pseudo-static method is a conservative method and it may be a case that even if the load exceeds the seismic bearing capacity in case of an earthquake it may not lead to an absolute failure. The case absolute failure or the loss of serviceability is by and large also dependant on the displacement caused in the critical case. It is recommended that the time history analysis is to be performed so that better and more realistic estimates can be established.

Thirdly, the vulnerability of the existing infrastructural stock is completely estimated when the entire slope-foundation-structure system is analyzed. For the above purpose, the fragility curve development for the various structural requirements of the structure resting on the foundation is recommended.

Lastly, the risk of the existing infrastructural stock is compounded by the exposure conditions thus it also has to be incorporated in the further work. The domino effect in case of a seismic event is an important consideration which is recommended to be investigated when the total risk is calculated.

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