

**INFLUENCE OF FOOTING SHAPE ON THE SEISMIC
BEARING CAPACITY OF FOUNDATIONS LOCATED
NEAR TO COHESIVE SLOPES USING FINITE
ELEMENT METHOD 3D ANALYSIS**

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INFLUENCE OF FOOTING SHAPE ON THE SEISMIC BEARING CAPACITY OF FOUNDATIONS LOCATED NEAR TO COHESIVE SLOPES USING FINITE ELEMENT METHOD 3D ANALYSIS

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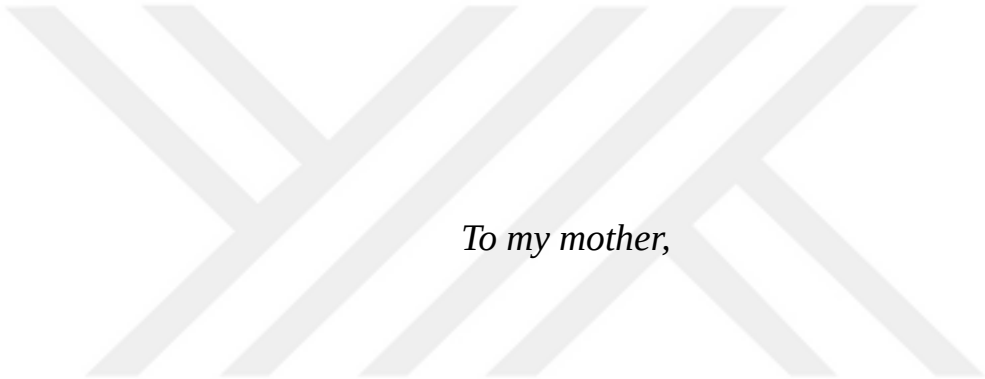
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To my mother,

ABSTRACT

This thesis focuses on assessing the impact of different footing shapes on the undrained seismic bearing capacity of surficial foundations located near slopes. The study employs the PLAXIS 3D finite element software to conduct a comparative analysis, aiming to provide valuable insights for geotechnical engineers involved in foundation design in such scenarios. For this purpose, numerical models are created to simulate the behaviour of surficial foundations under seismic loading. Various footing shapes are examined during the analysis, considering the undrained soil properties, slope characteristics, distance between the slope and footing, and seismic loading parameters. The research findings highlight the substantial influence of footing shape on the undrained seismic bearing capacity of surficial foundations near slopes. As a result of the study, design charts specific to each footing shape are developed, offering engineers a tool to estimate the bearing capacity in similar scenarios. These design charts provide practical guidance for optimal foundation design and enhance the understanding of how footing shape impacts the seismic performance of surficial foundations near slopes.

ÖZETÇE

Bu makale, şevlerin yakınında bulunan yüzeysel temellerin drenajsız sismik taşıma kapasitesi üzerindeki farklı temel şekillerinin etkisinin değerlendirilmesine odaklanmaktadır. Çalışma, karşılaştırmalı bir analiz yapmak için PLAXIS 3D sonlu elemanlar yazılımını kullanır ve bu tür senaryolarda temel tasarımında yer alan jeoteknik mühendisleri için değerli bilgiler sağlamayı amaçlar. Bu amaçla, yüzeysel temellerin sismik yükleme altındaki davranışını simüle etmek için sayısal modeller oluşturulmuştur. Drenajsız zemin özellikleri, şev özellikleri, şev ve temel arasındaki mesafe ve sismik yükleme parametreleri dikkate alınarak çeşitli temel şekilleri analiz sırasında incelenir. Araştırma bulguları, temel şeklinin, şevlerin yakınındaki yüzey temellerin drenajsız sismik taşıma kapasitesi üzerindeki önemli etkisini vurgulamaktadır. Çalışma sonucunda, her bir temel şekline özel tasarım çizelgeleri geliştirilerek, mühendislere benzer senaryolarda taşıma gücünü tahmin etme aracı sunulmaktadır. Bu tasarım çizelgeleri, optimum temel tasarımı için pratik rehberlik sağlar ve temel şeklinin şevlerin yakınındaki yüzeysel temellerin sismik performansını nasıl etkilediğinin anlaşılmasını geliştirir.

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CHAPTER I

INTRODUCTION

The recent earthquake in Kahramanmaraş (February 2023), Turkey, serves as a stark reminder of the vulnerability of structures in seismic-prone regions. With Turkey's sloping terrain and high seismic activity, it becomes imperative to have a comprehensive understanding of the seismic behaviour and bearing capacity of foundations located near cohesive slopes. A key aspect in ensuring the safety and stability of structures under seismic loading conditions is to determine the influence of footing shape on the seismic bearing capacity.

The seismic bearing capacity of foundations plays a pivotal role in safeguarding structures during earthquakes. Accurate assessment of the bearing capacity is essential to prevent potential failures and mitigate risks to human life and infrastructure. Thanks to advancements in numerical modelling techniques, such as the Finite Element Method (FEM), it is now possible to investigate the effects of various parameters on the seismic behaviour of foundations near cohesive slopes. FEM, widely employed for geotechnical analyses, provides a powerful tool for simulating the complex interaction between foundations, soil, and slopes. By utilizing FEM-based software, such as PLAXIS 3D, researchers can conduct detailed investigations into the influence of footing shape on the seismic bearing capacity. This approach enables a comprehensive understanding of the response and stability of foundations in cohesive slope environments subjected to seismic forces.

The objective of this thesis is to explore the influence of footing shape on the seismic bearing capacity of foundations located near cohesive slopes using FEM-based PLAXIS 3D analysis. Through systematic study of different footing shapes and their interaction with cohesive slopes, valuable insights can be gained regarding their impact on the seismic performance and stability of foundations. The outcomes of this research will contribute to the development of improved design guidelines and recommendations for foundations in seismic-prone areas near cohesive slopes.

In the following chapters, this thesis will review relevant literature related to seismic behaviour and bearing capacity of foundations located near cohesive slopes, discuss the methodology and analytical techniques employed in the research, present the numerical simulations conducted using FEM-based PLAXIS 3D, and analyse the obtained results, comparing them with findings from previous studies. Finally, a conclusion chapter will summarize the key findings of this research, highlight their implications, and propose avenues for future research.

By comprehensively investigating the influence of footing shape on the seismic bearing capacity using FEM-based PLAXIS 3D analysis, this study aims to contribute to the existing body of knowledge in geotechnical engineering. The findings will aid engineers and designers in making informed decisions regarding foundation design, leading to improved safety and resilience of structures during seismic events.



Figure 1: Kahramanmaraş earthquake (February 2023), Türkiye.

CHAPTER II

LITERATURE REVIEW

A thorough literature review is an integral part of any thesis as it allows for an in-depth exploration of previous studies conducted by researchers on similar topics. These studies serve as a foundation for discussion and comparison with the findings of the current thesis. The literature review in this study comprises three main parts, which are outlined in the following.

Influence of Footing Shape on Bearing Capacity: The first part of the literature review, covered in the initial chapter, focuses on presenting and discussing studies that investigate the impact of footing shape on bearing capacity. However, due to the lack of articles considering seismic conditions to examine the influence of shape, only studies related to static conditions will be considered.

Seismic Bearing Capacity Estimation for Footings Near Cohesive Slopes: In this section, studies pertaining to the estimation of seismic bearing capacity for footings located near slopes will be examined. The numerical results obtained in this thesis study will be compared with previous findings to assess their consistency and identify any significant differences.

Static and Seismic Stability of Homogeneous Cohesive Slopes: The final part of the literature review delves into proposed methods and analyses concerning the stability of cohesive slopes under static and seismic conditions. The objective is to examine the alignment of critical parameters leading to slope instability, as identified in previous

studies, with those discovered in this thesis, with the exclusion of applied load influence.

By undertaking this comprehensive literature review, the thesis aims to build upon and contribute to the existing body of knowledge while ensuring a robust comparison of findings and enhancing the understanding of the subject matter.

2.1 Influence of Footing Shape on Bearing Capacity

The literature contains a limited number of studies addressing the influence of footing shapes on soil bearing capacity. However, most of these studies have primarily focused on the case of footings located on the ground surface under static conditions. Unfortunately, there are only a few studies that have considered the effect of footing shapes on the bearing capacity of foundations situated on or near slopes under seismic conditions. Consequently, the following section will present and discuss studies related to the influence of footing shapes on bearing capacity, specifically in cases where the footings are located on the ground surface or near slopes under static conditions.

Golder [1] examined the bearing capacity of rectangular footings on both sand and clay soils through experimental means. The author performed loading tests using plastic soft clay soil for two different footing shapes: rectangular and square. The dimensions of the rectangular and square footings were 18 inches by 3 inches and 3 inches by 3 inches, respectively. The purpose of the tests was to determine the ultimate load that would lead to failure of the soil sample. The clay soil was placed in a wooden box measuring 2 feet 6 inches by 1 foot 6 inches and 8 inches deep, and it was compacted in layers using wooden hammers measuring 2 inches square. The top surface was levelled carefully, and the footing was then centrally placed and pushed into the

clay at a constant rate using a screw-driven testing machine. Load-settlement graphs were plotted for each test, and subsequently, the same soil samples were taken to determine their shear strength characteristics and moisture contents. The ultimate bearing capacity values for each test were determined by dividing the ultimate load by the footing area. The results of the tests are presented in Table 1.

Table 1: Results of loading tests for different footing shapes.

Test No.	1.	2.	3.	4.	5.	6.
Size of footing : inches .	3×3	18×3	3×3	18×3	18×3	3×3
Ultimate pressure : lb. per square inch	12.76	9.86	12.50	9.64	9.71	12.30

The average ultimate bearing capacity for the square footing in Golder's [1] study was found to be 12.5 lb. per square inch, while for the rectangular footing, it was 9.7 lb. per square inch. The test results indicate that the ultimate bearing capacity can vary depending on the shape of the footing. In this experiment, the ultimate pressure was 30% higher for the square footing compared to the rectangular footing with the same width.

These findings were compared to previous studies on the bearing capacity of strip foundations located on plastic soils. Prandtl [2] determined that the ultimate bearing capacity in this case can be determined solely based on the shear strength, as shown in Equation (1). Wilson [3] also derived a mathematical solution (Equation (2)) to calculate the ultimate pressure for the same scenario,

$$q_u = (2 + \pi)s_u \quad (1)$$

$$q_u = 5.41 s_u \quad (2)$$

where s_u is the undrained soil shear strength. When considering the long rectangular footing (18x3 inches) as a strip footing and dividing the obtained ultimate pressure by the average shear strength obtained from shear strength tests (1.88 lb. per square inch), a value of 5.15 was obtained. This value aligns with the findings of both Prandtl [2] and Wilson [3] regarding strip footings.

Overall, the results of Golder's [1] study highlight the influence of footing shape on the ultimate bearing capacity, with the square footing exhibiting higher ultimate pressure compared to the rectangular footing. Additionally, the comparison to previous studies demonstrates the consistency in the findings regarding the bearing capacity of strip footings on plastic soils.

Later, Terzaghi [4] proposed that in the case of strip foundations, soil particles would move perpendicularly to a plane that is perpendicular to the footing's centreline. However, for square or circular footings, the soil particles would move radially. Terzaghi suggested that the influence of footing shape could be incorporated into a simplified bearing pressure equation by introducing shape factors, denoted as n_c , n_q , and n_γ . These shape factors were found to depend solely on the friction angle ϕ and were independent of the footing length. This relationship is expressed in Equation (3).

$$q_u = cN_cn_c + qN_qn_q + \gamma BN_\gamma s_\gamma \quad (3)$$

At that time, numerical studies for calculating shape factors were not available. Hence, Terzaghi [4] relied on limited experimental results obtained by Golder [1], Skempton [5], and unpublished test data. For circular footings, the most unfavourable test results conducted by Skempton [5] were considered, leading to shape factors of $n_c = 1.3$, $n_q = 1$, and $n_\gamma = 0.6$. For square footings, Terzaghi utilized the test results provided

by Golder [1] and determined the n_γ shape factor to be 0.8, while the values of the other two factors remained unchanged.

Meyerhof [6] conducted a significant study on the bearing capacity of strip foundations located on the face and top of slopes. This research was highly influential and became widely adopted in subsequent design manuals. Meyerhof's [6] study involved combining the bearing capacity theory for footings on the ground level with slope stability theory, leading to the development of Equation (4),

$$q_u = cN_{cq} + 0.5\gamma BN_{\gamma q} \quad (4)$$

where $N_{\gamma q}$ and N_{cq} are factors that can be obtained from design charts proposed in the study. These factors depend on the inclination angle β , friction angle ϕ , and the distance from the edge of the slope and the soil properties. The author specifically stated that Equation 2.4 should be used for calculating the bearing capacity of strip foundations. However, no specific formulas were provided for calculating the shape factors for rectangular or circular foundations. The author suggested that empirical evidence and future studies would be necessary to determine the shape factors for these other footing shapes.

Meyerhof [7] introduced estimated formulas for shape factors specifically designed for rectangular foundations with varying B/L ratios, where B is the width and L is the length of the foundation. These formulas were derived by interpolating between the factors for strips and circles in direct proportion to the B/L ratio. The resulting formulas are presented in Equations (5), (6), and (7),

$$s_c = 1 + 0.2 \tan^2 \left(45 + \frac{\phi}{2} \right) \frac{B}{L} \quad (5)$$

$$s_q = s_\gamma = 1, \text{ when } \phi = 0 \quad (6)$$

$$s_q = s_\gamma = 1 + 0.1 \tan^2 \left(45 + \frac{\phi}{2} \right) \frac{B}{L} \quad (7)$$

where s_c , s_q and s_γ are shape factors, ϕ is the soil friction angle, and B/L is foundation aspect ratio. These equations provide a means to calculate the shape factors for rectangular foundations based on the B/L ratio. The value of s_c is determined by the equation involving the tangent function, while s_q and s_γ are assigned values of 1 when the friction angle ϕ is zero. Figure 2 illustrates the variation of N_c , N_q , and N_γ influenced by these shape factors, demonstrating the impact of foundation shape on these factors. However, the author did not provide any information regarding the applicability of these shape factors for foundations located on or near slopes in Equation (4).

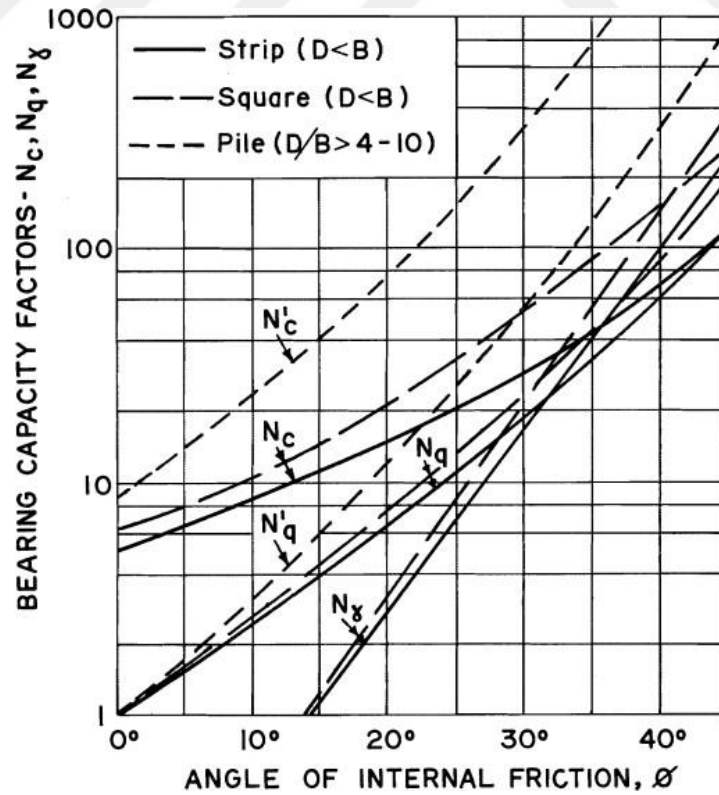


Figure 2: The effect of shape factors on the bearing capacity factors. (Meyerhof, 1963)

Debeer [8] proposed formulas to estimate the shape factors of footings located on sand soils by conducting multiple loading tests with different footing shapes. The tests were conducted on a small scale, with footings ranging in width or diameter from 36 to 150 mm. The shape of the footings was characterized by the B/L ratio, which represents the width-to-length ratio (Width/Length). Two types of footings were used: square and circular footings with $B/L = 1$, and rectangular footings with $B/L = 1/6$. The base of all the footings was covered with rough material to simulate a perfectly rough foundation.

Two types of tests were performed: surface footings tests, where the footings were located on the soil surface, and embedded footings tests, where the footings were embedded in the soil to consider overburden pressures. A total of 662 tests were conducted, with 350 tests performed on surface footings and 312 tests with overburden pressure.

To determine the shearing resistance angles for the soil samples, normal triaxial tests were performed. These tests were conducted at various relative densities, as shown in Figure 3. It should be noted that the shearing strength of cohesionless materials cannot be defined solely based on the values obtained from these tests, as the actual relationship between shearing strength and density is curved. Additionally, different realistic relationships were found for each state of strain. For this reason, a significant number of triaxial tests were conducted with different densities and mean normal pressures to determine the secant angle of the sand samples. Figure 4 illustrates the variation of the secant angle ϕ with relative density, considering the value of the normal stress σ_g on the shearing plane.

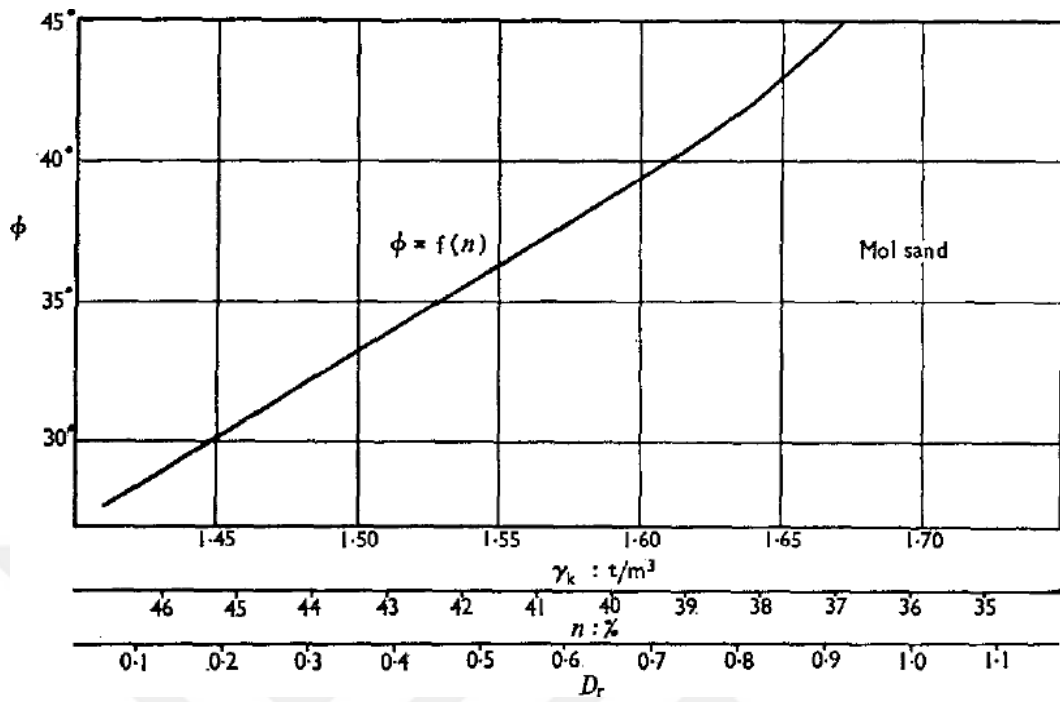


Figure 3: Normal triaxial test results. (Debeer, 1970)

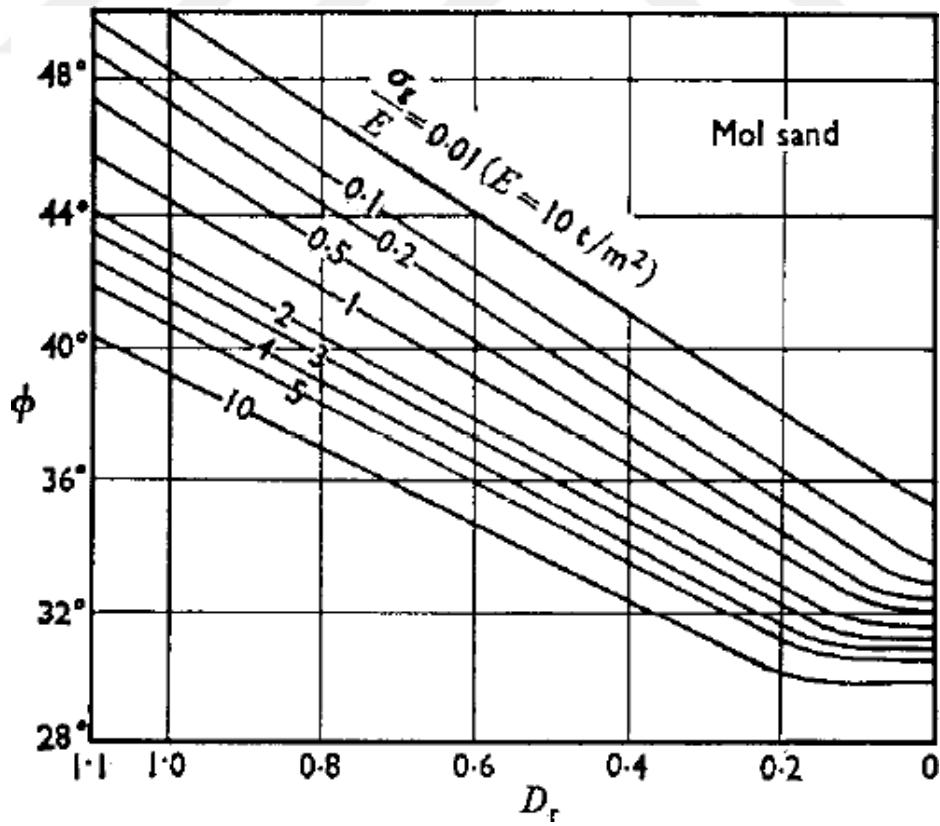


Figure 4: The variation of secant shear angle with relative density. (Debeer, 1970)

To calculate the mean normal stress underneath the footing, Equation (8) proposed by Meyerhof [6] was used:

$$\sigma_{g.M} = \frac{1}{4}(P_r + 3q)(1 - \sin\phi) \quad (8)$$

In Figure 4, the secant angle values were determined for different relative densities and stress conditions. These values were used to estimate the $s_\gamma N_\gamma/2$ factors using the bearing capacity equations proposed by Hansen [9] (Equations 0):

$$\begin{cases} P_r = N_q d_q s_q \gamma_k w_r + 0.5 N_\gamma s_\gamma \gamma_k b \\ \frac{s_\gamma N_\gamma}{2} = \frac{P_r}{\gamma_k b} - \frac{N_q d_q s_q w_r}{b} \\ N_q = \frac{P_r}{\gamma_k b \left(\frac{d_q s_q w_r}{b} + 0.5 \frac{s_\gamma N_\gamma}{N_q} \right)} \end{cases} \quad (9)$$

Figure 5 illustrates the experimental points corresponding to different footing shapes. Rectangular footings with $B/L = 1/6$ are represented by small rectangles, square footings by squares, and circular footings with $B/L = 1$ by circles. The curve AB represents the mean curve obtained for all circular footings, regardless of their size, while the curve CD represents the mean curve for all rectangular footings with $B/L = 6$, regardless of their size. The ratio between AB and CD can be used to express the shape factor using Equation (10):

$$s_\gamma = 1 - 0.4 \frac{B}{L} \quad (10)$$

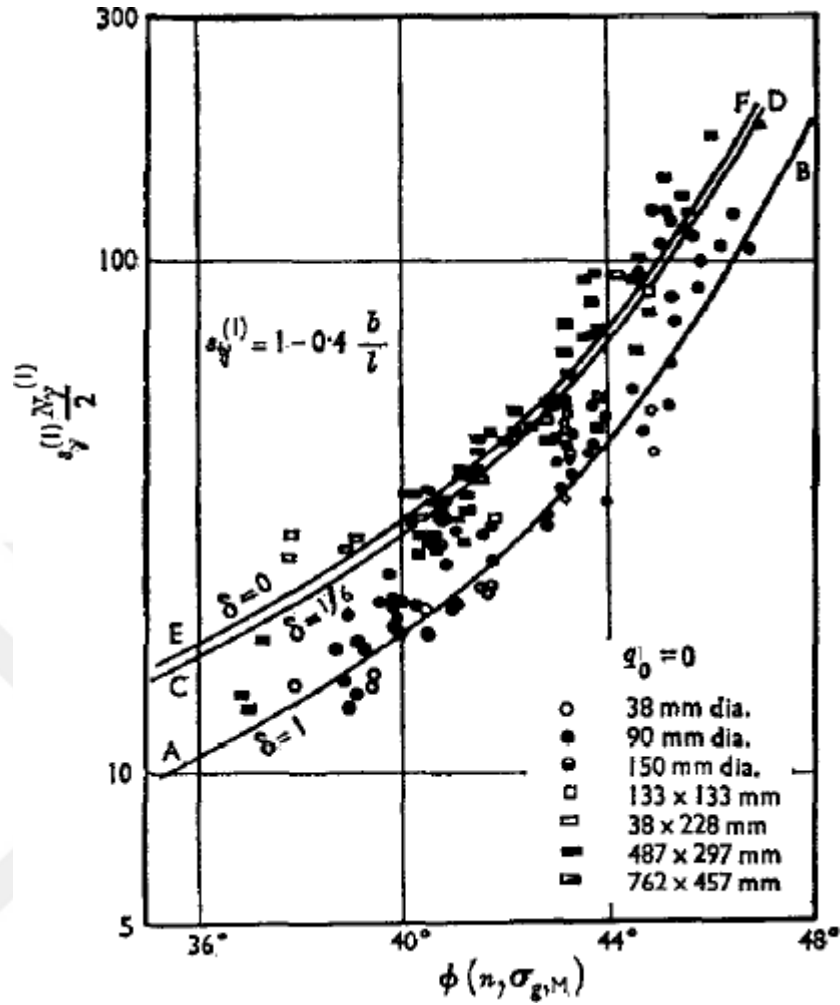


Figure 5: The variation of shape factor s_q with respect to secant angle values.

(Debeer, 1970)

To determine the shape factor s_q , the author examined the relationship between the secant angle ϕ values and the factor $N_q s_q$ by utilizing the equations proposed by Hansen [9] (Equation (11)). In this process, the author considered the experimental results conducted under overburden pressure q_0 . Equation (8) was employed to calculate the $\sigma_{g,M}$ values, and Figure 4 was utilized to obtain the secant angle values. Figure 6 displays the variation of the $N_q s_q$ factor in relation to the secant angle ϕ for both circular and rectangular footings.

$$P_r = N_q d_q s_q (q_0 + \gamma_k w_r) + 0.5 N_\gamma s_\gamma \gamma_k b \quad (11)$$

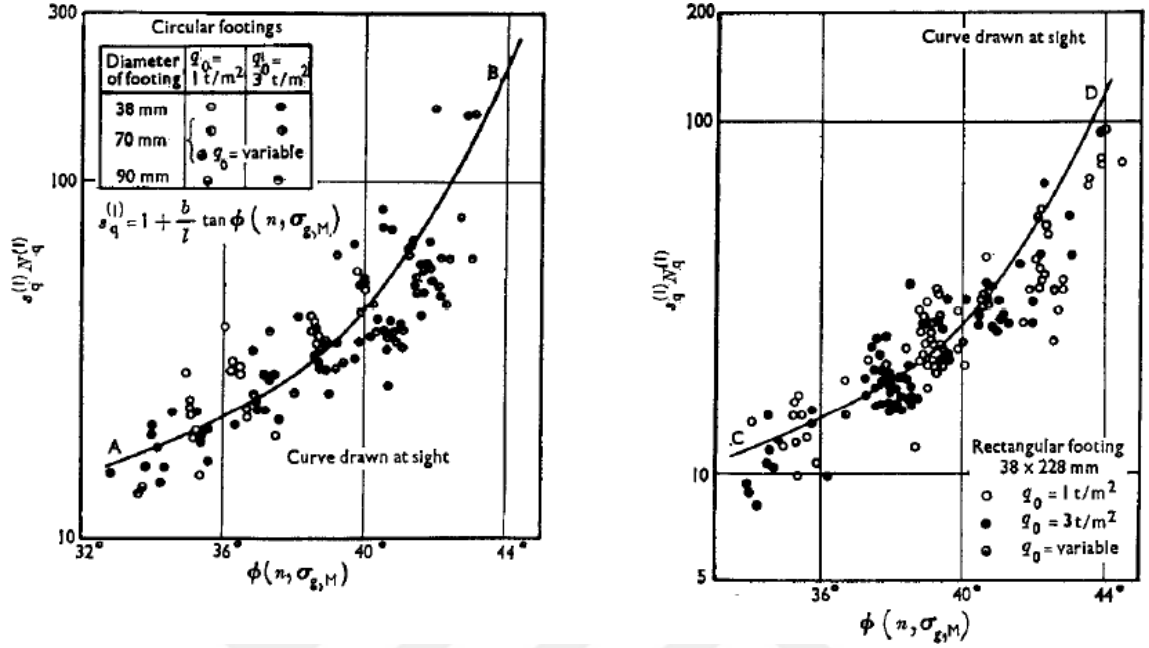


Figure 6: The variation of $N_q s_q$ factor with respect to secant angle ϕ . (Debeer, 1970)

The curves AB and CD in Figure 6 represent the circular and rectangular footings, respectively. By examining the ratio of the ordinate values of these two curves, Equation (12) is derived to express the shape factor s_q .

$$s_q = 1 + \frac{B}{L} \sin \phi \quad (12)$$

The author utilized the expression for the s_c factor in Equation (13) proposed by Hansen [9], along with the experimental N_q values and the s_q factor from Equation (12), to propose Equation (14).

$$s_c = \frac{s_q N_q - 1}{N_q - 1} \quad (13)$$

$$s_c = 1 + \frac{B}{L} \frac{N_q}{N_q - 1} \tan \phi \quad (14)$$

Zhu and Michalowski [10] conducted a numerical study using the finite element system ABAQUS to analyse the impact of footing shape on the bearing capacity of an elasto-plastic medium soil.

In their study, a 3D model was employed to represent rectangular footings with varying L/B ratios (length to width ratio). To simplify the calculations and take advantage of symmetry, only one-fourth of the footing was considered, as depicted in Figure 7. The soil directly beneath the footing was discretized into 18 elements, and the mesh in the surrounding area was refined. The distance between the boundaries parallel to the footing length was maintained at least 15 times the width of the footing, and the depth of the model was set to half of that distance to minimize the influence of boundary conditions. Although the author performed calculations with finer models, the effect of increased refinement was found to be negligible. This is because the shape factors are determined by comparing the bearing capacity factors of rectangular footings with those of strip footings.

During the loading process, the load was incrementally increased until failure of the model occurred. The bearing capacity was then determined by dividing the maximum vertical force obtained from the calculations by the area of the footing.

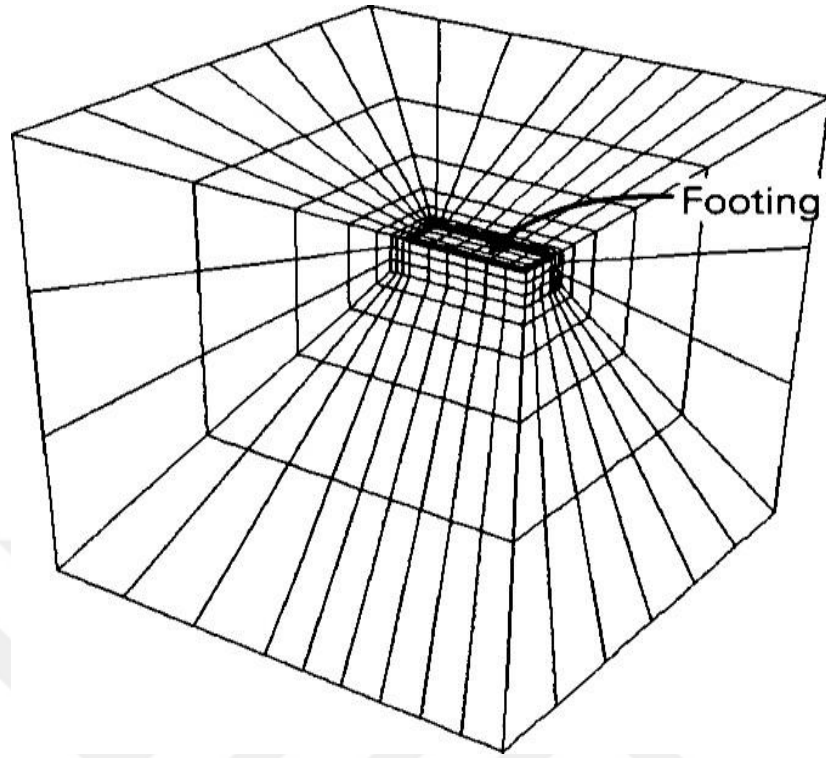


Figure 7: Finite Element Mesh. (Zhu and Michalowski, 2005)

The soil was modelled as an isotropic elasto-perfectly plastic continuum, following the Mohr-Coulomb function, with a Poisson's ratio of $\nu = 0.3$ and a Young's modulus of $E = 20 \text{ MPa}$.

The shape factors were determined under various conditions depending on the type of factor. To calculate the s_c factor, the bearing capacity factor N_c was obtained by assuming weightless soil ($\gamma = 0$) and no overburden pressure ($q = 0$),

$$q_u = cN'_c = s_c N_c c \quad (15)$$

$$s_c = \frac{N'_c}{N_c} \quad (16)$$

where q_u is the ultimate bearing capacity, c is the cohesion value, N'_c is the bearing capacity factor for rectangular shape footing, and s_c is shape factor.

Similarly, the s_q factor can be determined using Equation (18). Additionally, Equation (18)(17), proposed by Caquot [11], can be utilized to calculate the value of N_q' if N_c' is known.

$$N_q' = N_c' \tan \phi + 1 \quad (17)$$

$$s_q = \frac{N_q'}{N_q} = \frac{N_c' \tan \phi + 1}{N_c \tan \phi + 1} \quad (18)$$

The s_γ factor was determined for $q=0$ and $\gamma > 0$ using Equation (19). A small amount of cohesion was assumed to prevent numerical difficulties that could arise in purely frictional materials. Subsequently, this cohesion was removed by subtracting the value cN_c' .

$$s_\gamma = \frac{N_\gamma'}{N_\gamma} \quad (19)$$

The numerical analysis results are depicted in Figure 8 for the shape factors s_γ , s_q , and s_c . The author proposed approximation formulae for each factor represented by Equations (20), (21) and (22).

$$s_\gamma = 1 + (0.6 \tan^2 \phi - 0.25) \frac{B}{L} \quad \text{for } \phi < 30^\circ$$

$$s_\gamma = 1 + (1.3 \tan^2 \phi - 0.5) \frac{B}{L} \quad \text{for } \phi > 30^\circ \quad (20)$$

$$s_q = 1 + 0.9 \tan^2 \phi \sqrt{\frac{B}{L}} \quad (21)$$

$$s_c = 1 + (1.8 \tan^2 \phi + 0.1) \sqrt{\frac{B}{L}} \quad (22)$$

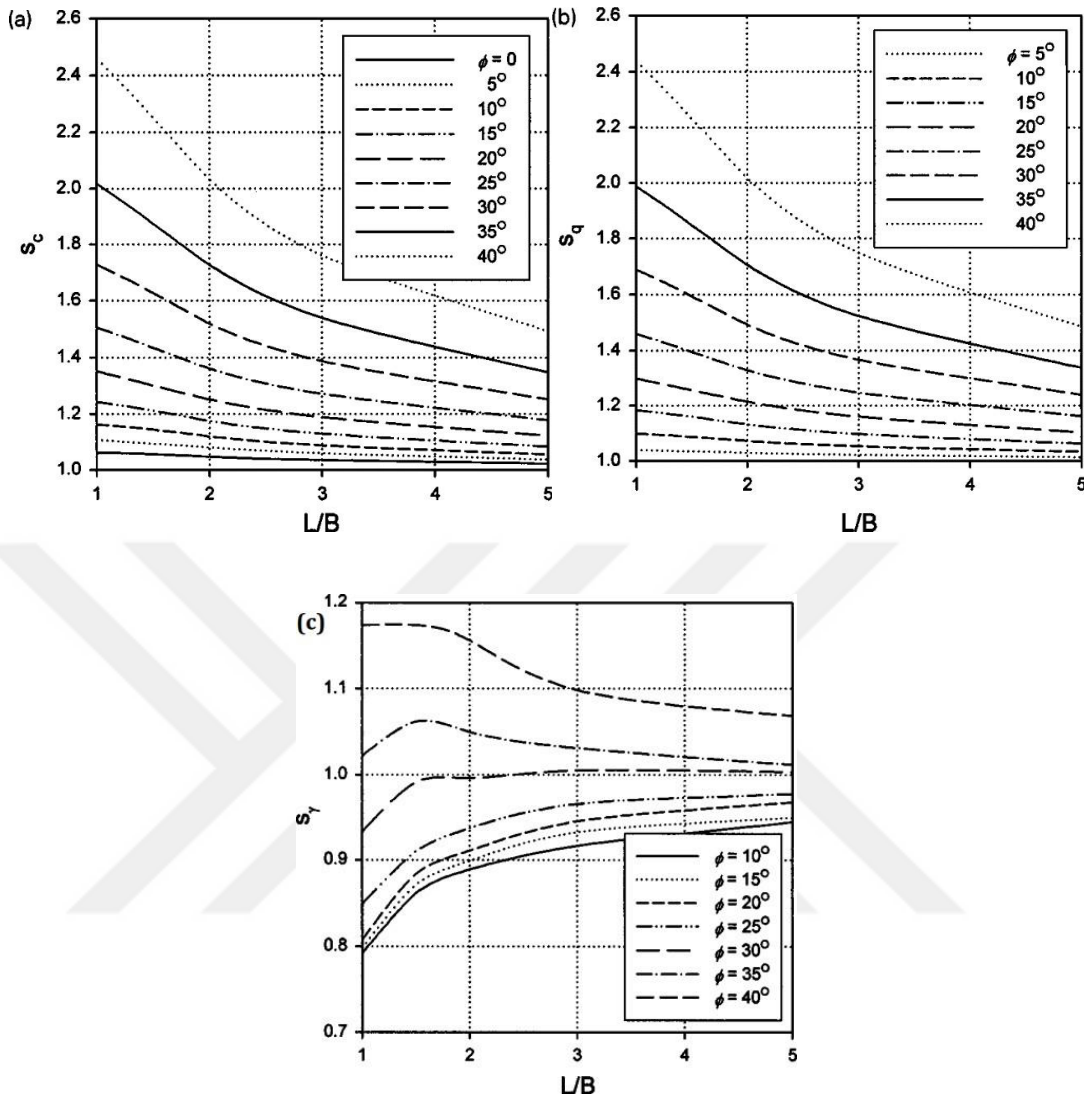


Figure 8: Shape factors (a) s_c , (b) s_q and (c) s_γ for FEM as function of B/L. (Zhu and Michalowski, 2005)

Gourvenec *et al.* [12] conducted a study on the bearing capacity of square and rectangular footings resting on homogeneous undrained clay using finite element analysis with ABAQUS software.

A 3D model was created to represent rectangular footings with six different L/B ratios (1, 1.5, 2, 3, 10). To simplify the calculations, only half of the footing was considered due to symmetry, as shown in Figure 9. For longer footings, more elements

were used to maintain a uniform element size across the models. The mesh extended $3B$ from the footing edges and $2.5B$ beneath the footing, with zero-displacement boundary conditions to prevent out-of-plane displacements. A 2D plane-strain mesh with the same dimensions was also constructed to obtain the bearing capacity factors for strip footings. A bearing capacity factor N_c of 5.31 was obtained, which showed a 3% overprediction compared to the exact value. Further calculations were performed using a finer model with 1500 elements for plane strain, resulting in an N_c value of 5.27 with only a 0.5% increase in accuracy.

In the loading process, the load was incrementally increased until the model failed. Bearing capacity was determined by dividing the maximum vertical force obtained at a point directly under the midpoint of the footing base by the area of the footing.

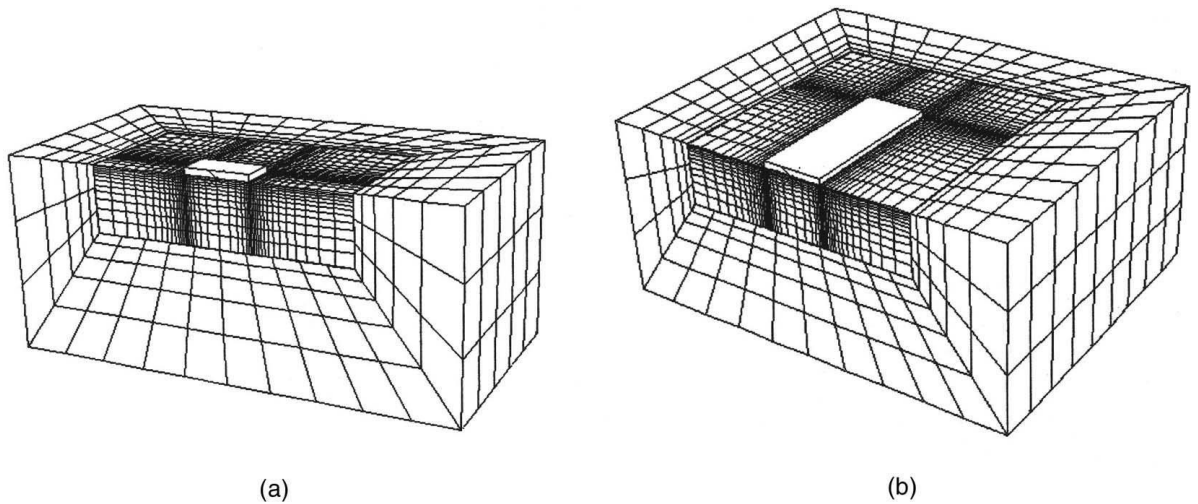


Figure 9: Meshes for (a)square footing, and (b)rectangular footing with $L/B=5$.

(Gourvenec *et al.*,2006)

The soil material was modelled as a simple soil with an isotropic linear elastic-perfectly plastic behaviour following the Tresca flow rule and having a uniform

undrained shear strength value s_u . The material had a constant stiffness index of $E_u/s_u = 1,000$, a Poisson's ratio of $\nu = 0.49$, and a unit weight of $\gamma = 8 \text{ kN/m}^3$.

The obtained N_c values from the finite element analysis with rough interface were compared to other values obtained analytically and empirically, as presented in Table 2. The s_c shape factor value was determined using the empirically obtained N_c value of 5.14 for strip foundations by Skempton [5].

Table 2: Undrained Bearing Capacity and shape factors for rough-based rectangular footings. (Gourvenec *et al.*, 2006)

L/B	Skempton (1951)		Michalowski (2001)	Salgado et al. (2004)			This study		
	N_c	s_c	N_c	N_c (LB)	N_c (UB)	N_c (average)	N_c (UB)	N_c (FE)	s_c^a (FE)
1	6.17	1.20	6.56	5.52	6.22	5.87	6.41	5.91	1.15
1.5	5.83	1.13	6.19	—	—	—	6.03	5.78	1.12
2	5.65	1.10	6.06	5.36	6.02	5.57	5.83	5.69	1.11
3	5.48	1.07	5.64	5.25	5.89	5.51	5.62	5.58	1.09
5	5.35	1.04	5.49	5.17	5.78	5.47	5.44	5.49	1.07
10	5.24	1.02	5.20	—	—	—	5.30	5.41	1.05
∞	5.14	1.00	5.14	5.13	5.20	5.17	5.14	5.31	1.03

^a s_c calculated assuming plane strain N_c of 5.14.

An overprediction can be observed in the obtained s_c values, particularly for the strip value ($s_c=1.03$), which should ideally be 1. To address this issue, the finite element results can be adjusted to eliminate the gradual overprediction as the footing approaches a strip shape. This adjustment involves reducing the s_c values by a factor of $0.032(1-B/L)$. By applying this correction, the corrected shape factors are obtained, as presented in Table 3.

Table 3: Corrected Undrained Bearing Capacity and shape factors for rough-based footings. (Gourvenec *et al.*,2006)

L/B	1	1.5	2	3	5	10	∞
s_c	1.15	1.11	1.09	1.06	1.04	1.02	1.00
N_c	5.91	5.72	5.61	5.47	5.36	5.26	5.14

The relationship between the shape factor and the footing aspect ratio, obtained from the corrected finite element results, can be expressed by Equation (23). Figure 10 illustrates both the raw and corrected shape factor values.

$$s_c = 1 + 0.214 \frac{B}{L} - 0.067 \left(\frac{B}{L} \right)^2 \quad (23)$$

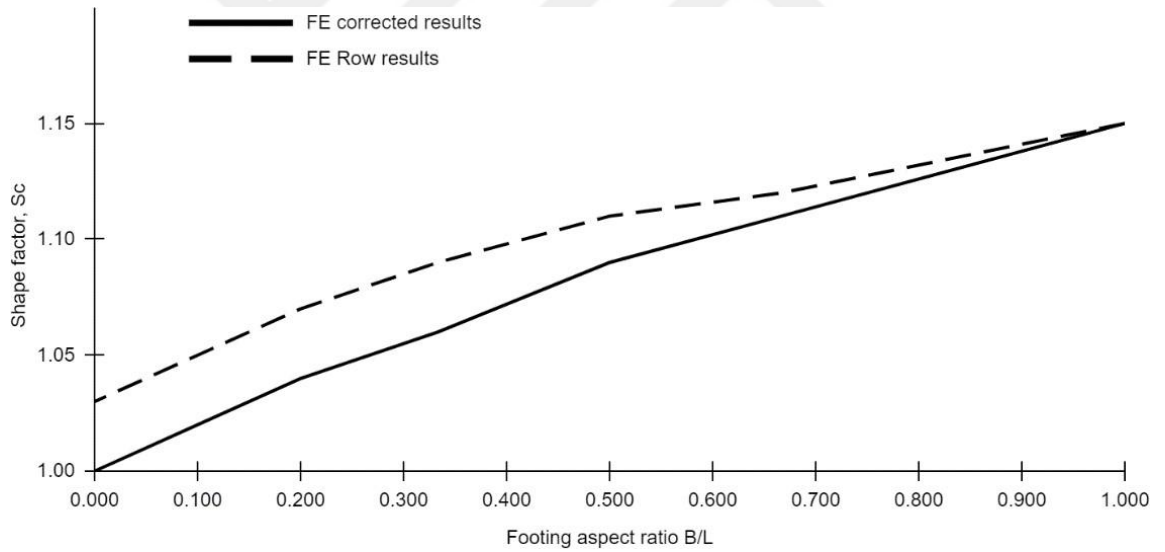


Figure 10: Compared row and corrected FE results for sc shape factor. (Gourvenec *et al.*,2006)

Recently, Yang *et al.* [13] focused on the shape factors of footings located on or near slopes, considering only the static condition. The authors conducted finite element limit analysis to determine the bearing capacity of the footings. Two cases were analysed in this study, as depicted in Figure 11: (a) when the long direction of the

footing is parallel to the slope, and (b) when it is orthogonal to the slope. The distance between the slope and the footing edge was denoted as λM , where M is the minimum of the footing's width (B) and length (L), ensuring consistency of the distance for aspect ratios greater than unity.

Regarding the boundary conditions, the distances between the footing and the front, rear, bottom, and side boundaries were set as $12M + D/2$, $8M + D/2$, $7.5M + D/2$, and $8M + D/2$, respectively. The front, rear, and two side boundaries were fixed in the normal direction. At the bottom boundary, both normal and tangential degrees of freedom were fixed. The footing itself was modelled as a rigid weightless body placed on the slope crest, while the slope was represented by homogeneous and isotropic soil adhering to the Mohr-Coulomb failure criterion and an associated flow rule.

The material properties used in the analysis were as follows: the stiffness E was 30 MPa, the Poisson's ratio ν was 0.29, and the earth pressure coefficient at rest (K_0) was $1 - \sin\phi$, where ϕ denotes the internal friction angle.

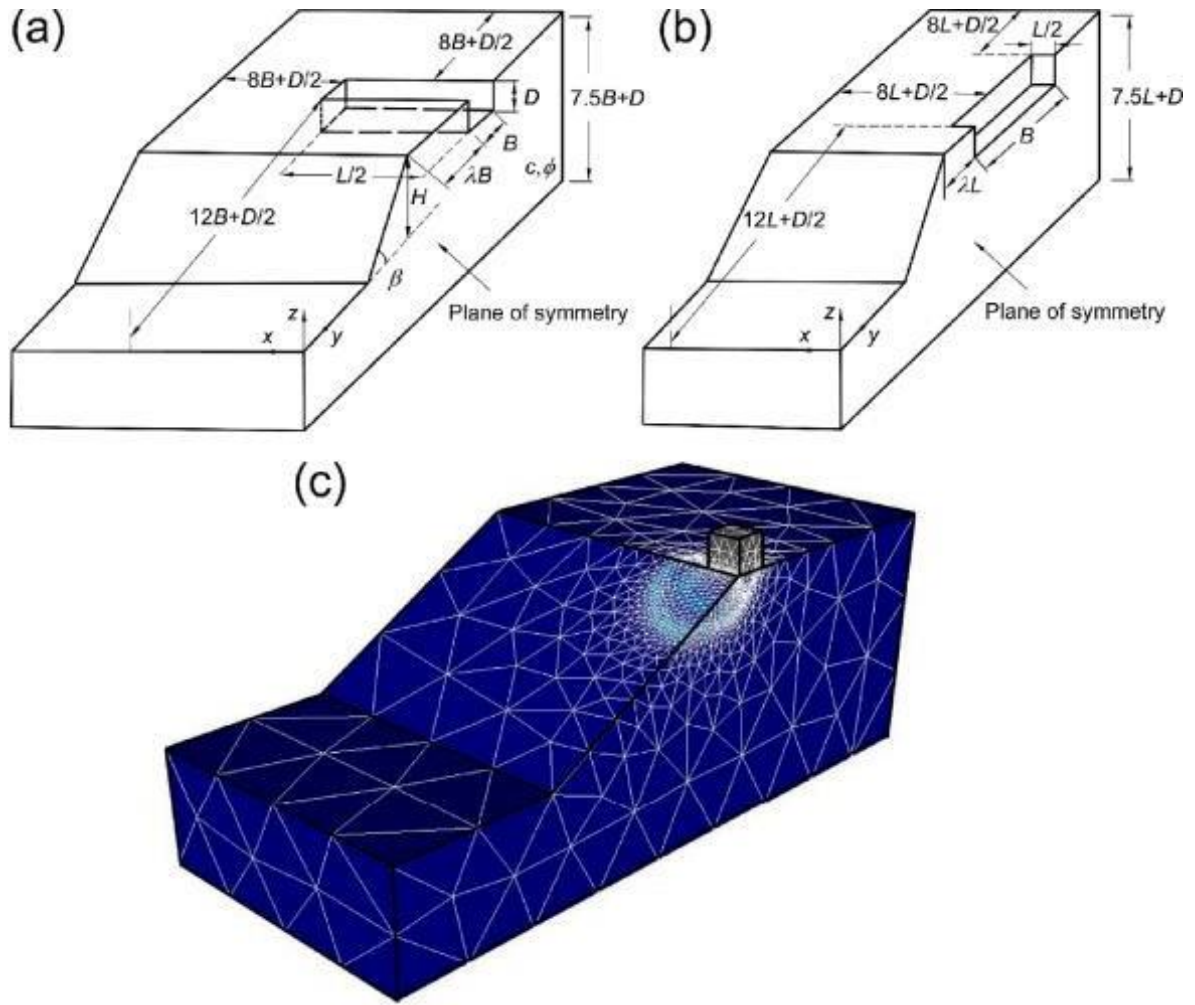


Figure 11: Schematic of 3D numerical model for (a) parallel and (b) orthogonal, longitudinal directions of footings and slopes, and (c) mesh with adaptive iteration.

(Yang et al., 2021)

The study conducted by the author involved multiple finite element limit analysis (FELA using OptumG3) calculations using different parameter values, as summarized in Table 4. Rather than directly determining the shape factors, the author chose to focus on finding the ratio Q_{3D}/Q_{2D} . This ratio represents the bearing capacity of the rectangular footing divided by the bearing capacity of a strip footing with identical slope and soil properties, as illustrated in Figure 12.

Table 4: Parameters used in the FELA.

Parameter	Value
β	$10^\circ, 30^\circ$
φ	$0^\circ, 15^\circ, 30^\circ$
$c/\gamma M$	1
M/H	0.25, 2
B/L	0–10
λ	0, 1
D/M	0, 0.5, 1

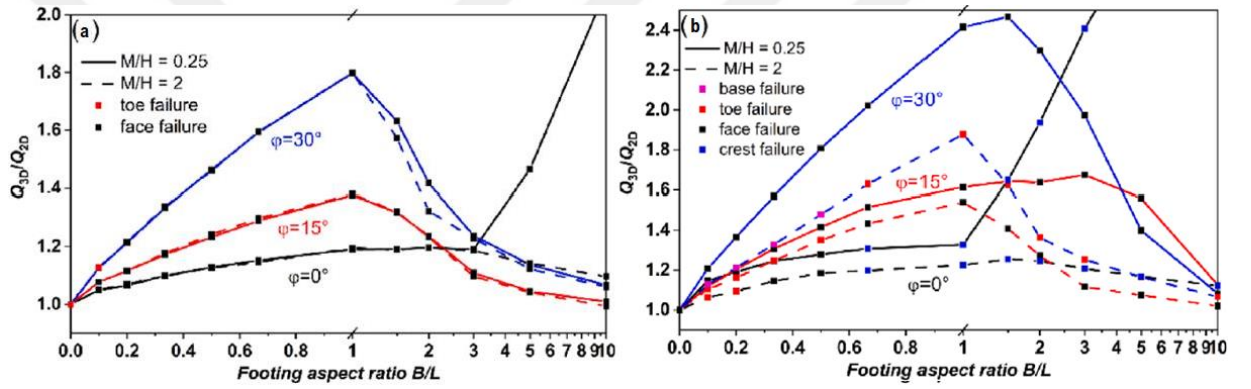


Figure 12: Ratio of Q_{3D}/Q_{2D} for: (a) $\lambda = 0, \beta = 10^\circ$ and (b) $\lambda = 1, \beta = 30^\circ$. (Yang et al., 2021)

Upon examining Figure 12 for cases (a) and (b), it is evident that the bearing capacity of rectangular footings increases as the aspect ratio B/L increases until it reaches unity. However, when the aspect ratio exceeds unity, the bearing capacity decreases, indicating that square footings exhibit the maximum bearing capacity. Nevertheless, for larger values of φ , the square footing may not necessarily yield the maximum bearing capacity, as depicted in Figure 12(b). Additionally, when the ratio M/H is small, it is observed that the bearing capacity of the slope-orthogonal case ($B/L > 1$) increases, suggesting that utilizing a plain-strain 2D bearing capacity estimation may not be conservative.

The author also conducted calculations for embedded footings in both rough and smooth walls, while maintaining the same slope and soil properties. Figure 13 illustrates the results of the ratio Q_{3D}/Q_{2D} with various aspect ratios. Notably, the aspect ratio has a more pronounced effect on the bearing capacity when significant embedment occurs in conjunction with rough walls.

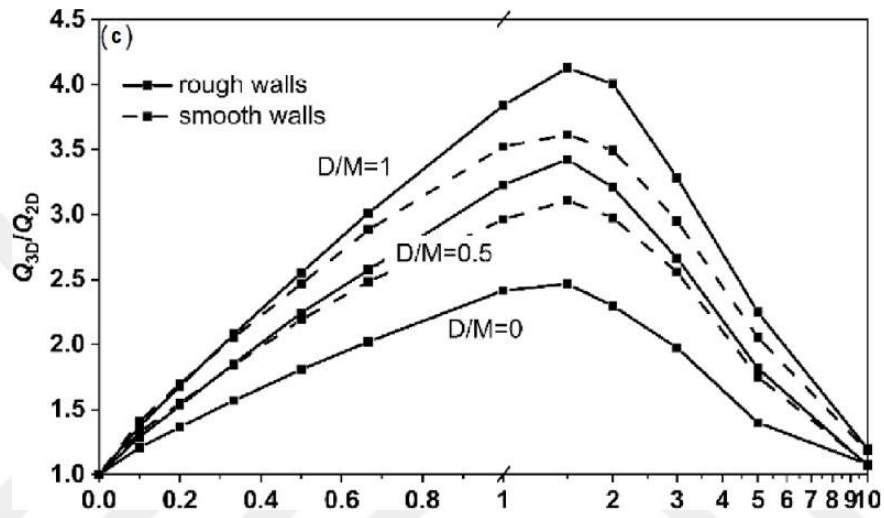


Figure 13: Ratio of Q_{3D}/Q_{2D} for $\lambda = 1$, $\beta = 30^\circ$, $\phi = 30^\circ$ and $M/H = 0.25$. (Yang et al., 2021)

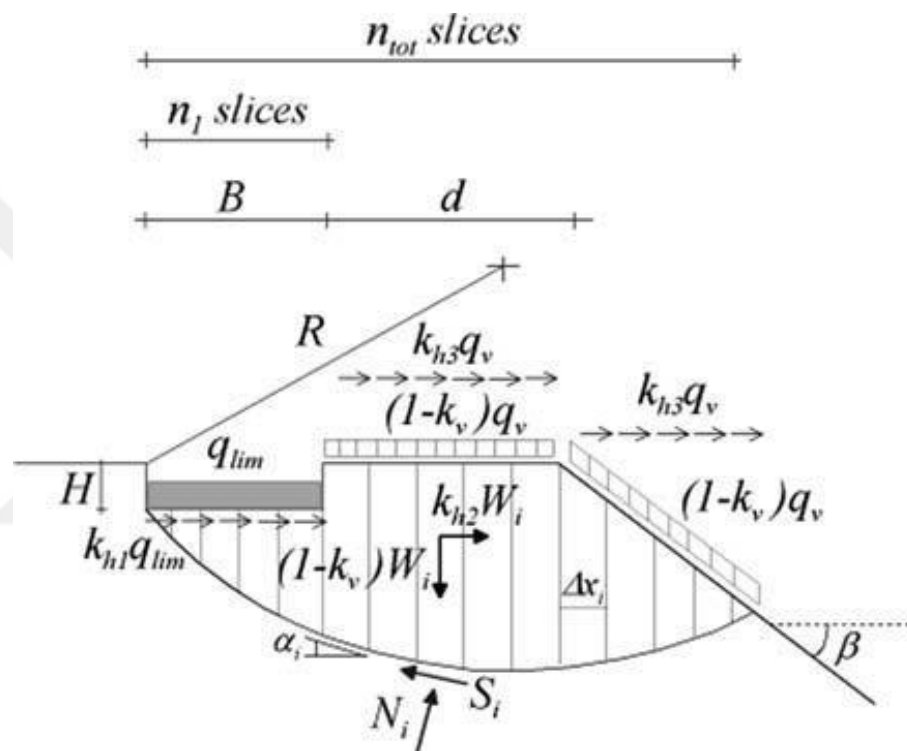
2.2 Seismic Bearing Capacity Estimation for Footings Near Cohesive Slopes

This chapter focuses on the examination of studies found in the literature regarding the seismic bearing capacity of footings situated near slopes. Prior to 2010, the number of studies conducted on this topic was limited. However, in subsequent years, researchers began to address this subject matter by employing various numerical and analytical methods to find solutions. Several of these studies will be reviewed in the following sections and later discussed to compare their findings with the previous solutions.

Unfortunately, no study has been discovered in the literature that specifically considers the seismic conditions to assess the influence of footing shape on the bearing capacity of footings located on slopes. Consequently, the subsequent studies will solely pertain to the case of strip footings.

Castelli and Motta [14] conducted a study on the bearing capacity of footings situated on granular and cohesive soil under both static and seismic conditions. The analysis was performed using limit equilibrium analysis. The authors considered a failure mechanism where a circular surface initiates from the edge of the foundation and extends towards the slope until it reaches the sloping ground. By applying the moment equilibrium principle, they calculated the maximum load value, q_{lim} , which leads to failure, and considered it as the bearing capacity of the system, as illustrated in Figure 14. The q_{lim} value corresponds to the most critical failure circle.

To incorporate the seismic effect, the authors employed a pseudo-static approach, considering an inertial force acting on both the footing and the soil. For the static analyses, the ground acceleration values were assumed to be zero. The study



and Motta, 2010)

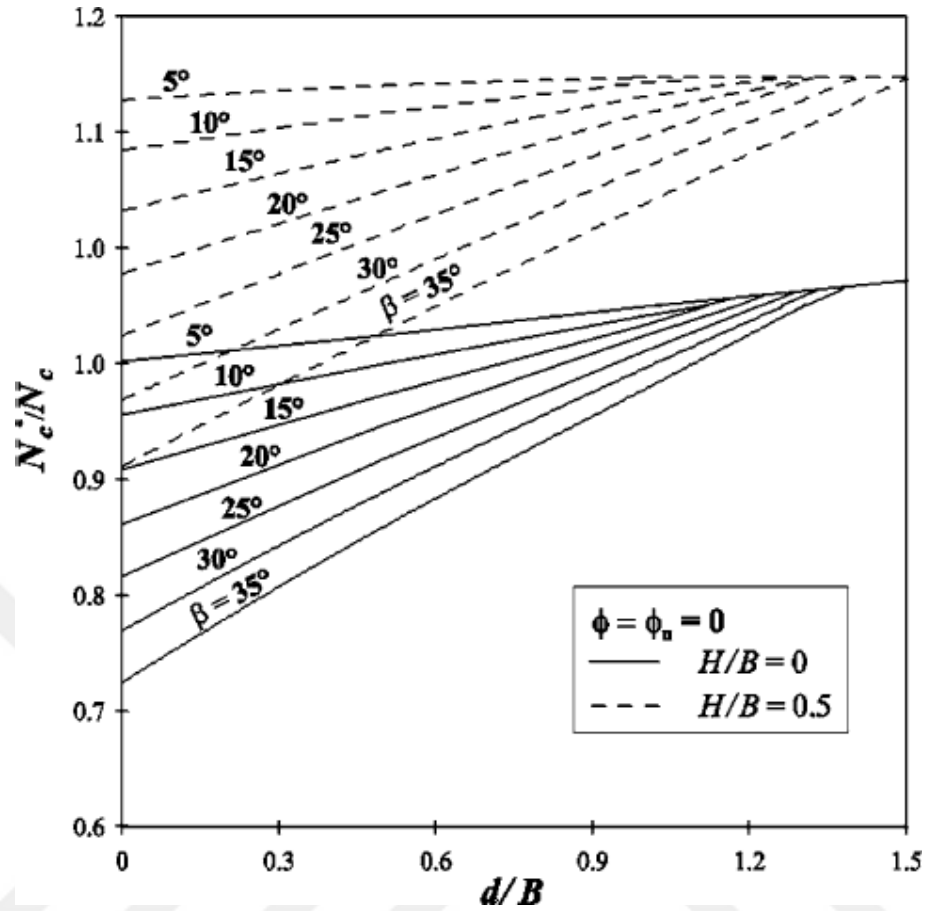


Figure 15: The variation of (N_c^*/N_c) factor with the distance from the edge of slope. (Castelli and Motta, 2010)

It is evident that the bearing capacity increases as the distance from the slope edge increases, and the ratio N_c^*/N_c tends to approach unity, indicating that the failure surface is not influenced by the slope properties. However, it is noteworthy that embedding the footing leads to higher bearing capacity values compared to superficial footings. These values even surpass those of ground footings when the d/B value exceeds 0.45 for an embedment factor H/B of 0.5.

For the seismic analyses, the same parameters were considered to determine the variation of N_c^*/N_c . Additionally, a pseudo-static approach was employed, and the values of $k_{h1}=k_{h3}$ and k_{h2} were assumed to be 0.1, 0.2 and 0.2, 0.4, respectively. This

choice was made in accordance with Eurocode 8, which allows for the reduction of seismic action based on the behaviour factor and the ductility classification of structures.

Since the inertia force of the structure is the most influential factor in the seismic bearing capacity under undrained conditions, only k_{h1} was considered in the undrained seismic analyses. The results are illustrated in Figure 16. Similarly, to the static analysis results, it can be observed that the bearing capacity increases as the distance from the slope edge increases, eventually reaching a point where the effect of the slope becomes negligible and only the seismic force affects the bearing capacity.

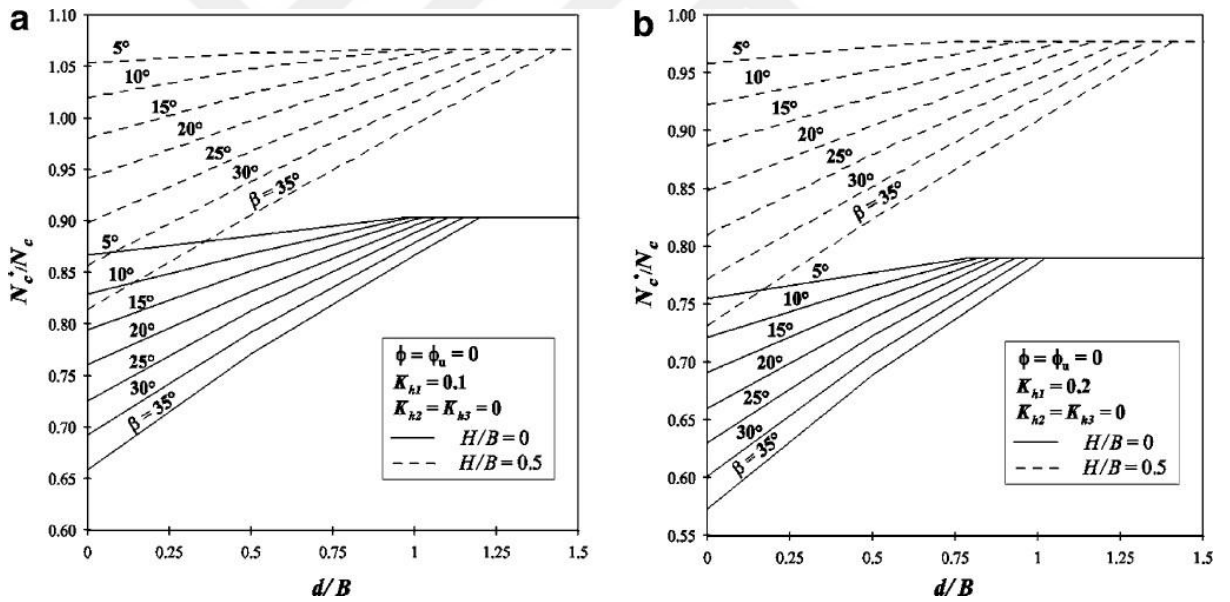


Figure 16: Variation of N_c^*/N_c with distance from the slope edge under undrained seismic condition. (Castelli and Motta, 2010)

Farzaneh and Mofidi [15] investigated the seismic bearing capacity of strip footings adjacent to purely cohesive slopes using a finite element lower bound method combined with linear programming. The lower bound analysis provides conservative results that are always on the safe side. The model used to determine the seismic

bearing capacity of the strip footing near the purely cohesive slope is depicted in Figure 17. The analysis considered several influential parameters, including the slope angle, distance from the slope edge, footing width, and slope height denoted by β , a , B , and H , respectively. The soil was modelled using the Tresca yield criterion with the associated flow rule, with undrained shear strength c_u and unit weight γ . The foundation was assumed to have a rough base. The horizontal acceleration coefficient of the earthquake (k_h) was considered to act outward from the slope, which is the more critical direction, while the vertical acceleration of the earthquake (k_v) was disregarded in the analyses. The obtained bearing capacity value p was normalized by the value of γB . The equation for the bearing capacity in this case is given by Equation (24).

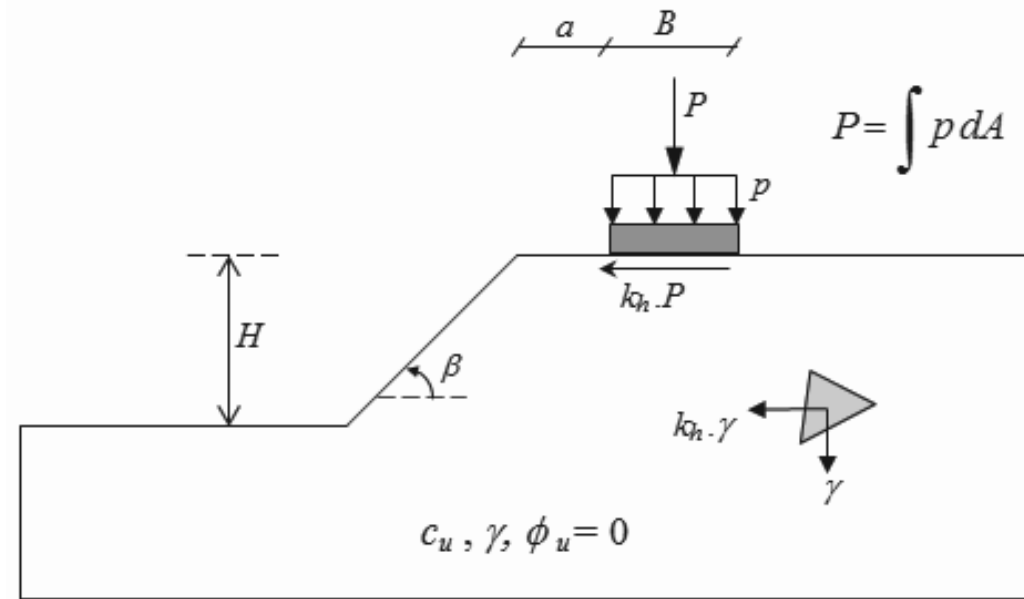


Figure 17: Schematic of the parameters used in analyses. (Farzaneh and Mofidi, 2013)

$$\frac{p}{\gamma B} = f \left(\beta, \frac{a}{B}, \frac{H}{B}, \frac{c_u}{\gamma B}, k_h \right) \quad (24)$$

The analyses were performed for various values of the parameters, where the inclination angle β assumed as 30° , 60° and 90° , while the horizontal acceleration coefficient k_h is considered as 0.1, 0.15, 0.2, 0.25, 0.3, and various ratios of a/B and $c_u/\gamma B$ were considered. The value of $H/B=4$ which ensures the bearing capacity failure mode (Figure 18 (a)) is constant for the whole analysis.

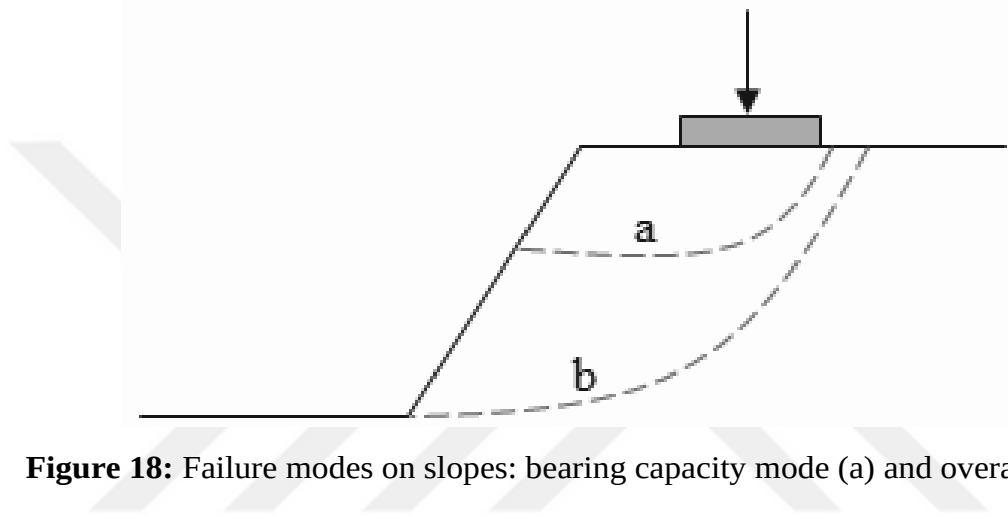


Figure 18: Failure modes on slopes: bearing capacity mode (a) and overall slope failure (b). (Farzaneh and Mofidi, 2013)

It is evident that an increase in the k_h value results in a decrease in the normalized bearing capacity value. This decrease is more significant for k_h values less than 0.1 compared to k_h values greater than 0.1. Additionally, the influence of the inclination angle β decreases for the latter case, as depicted in Figure 19.

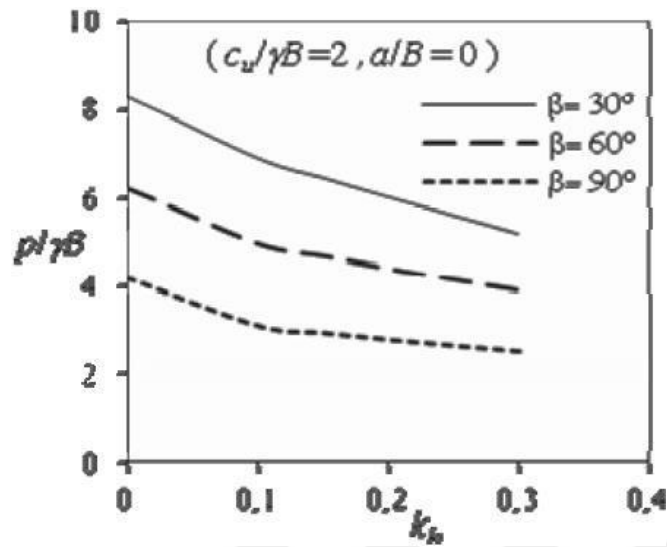


Figure 19: Variation of dimensionless bearing capacity factor with seismic coefficient k_h . (Farzaneh and Mofidi, 2013)

As the distance from the slope edge increases, the bearing capacity initially increases until it reaches a constant value. Beyond this point, the effect of the slope diminishes, and the failure of the footing becomes solely dependent on the properties of the soil. Additionally, slopes with lower inclination angles reach the constant bearing capacity value more quickly, as shown in Figure 20.

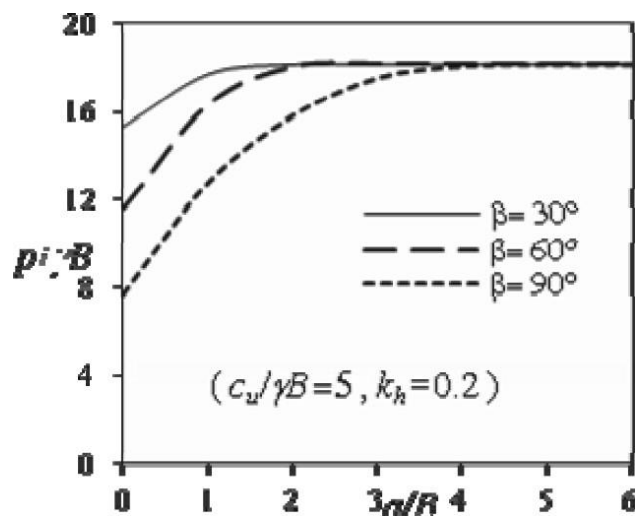


Figure 20: Variation of seismic bearing capacity with a/B . (Farzaneh and Mofidi, 2013)

Surprisingly, Figure 21 demonstrates that the normalized bearing capacity value exhibits a linear variation within a specific range of $c_u/\gamma B$ values. Earlier investigations conducted by previous studies have established that this relationship is non-linear when dealing with lower $c_u/\gamma B$ values. It has been observed that during this range, slope stability deteriorates rapidly, leading to overall slope failure. It is important to note that the present study exclusively focuses on the bearing capacity failure mode, which elucidates the rationale behind the linear relationship depicted in Figure 21.

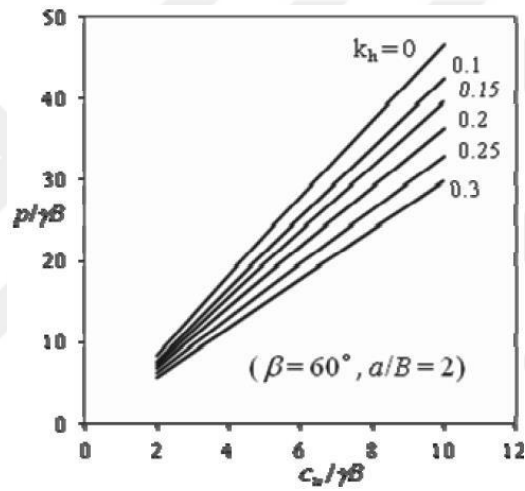


Figure 21: Variation of seismic bearing capacity with $c_u/\gamma B$. (Farzaneh and Mofidi, 2013)

Design charts that provide lower bound estimations for the seismic bearing capacity of strip footings near cohesive slopes are presented in Figure 22. As previously discussed, the linear segment observed in these design charts can be attributed to the sole consideration of bearing capacity failure during the analysis.

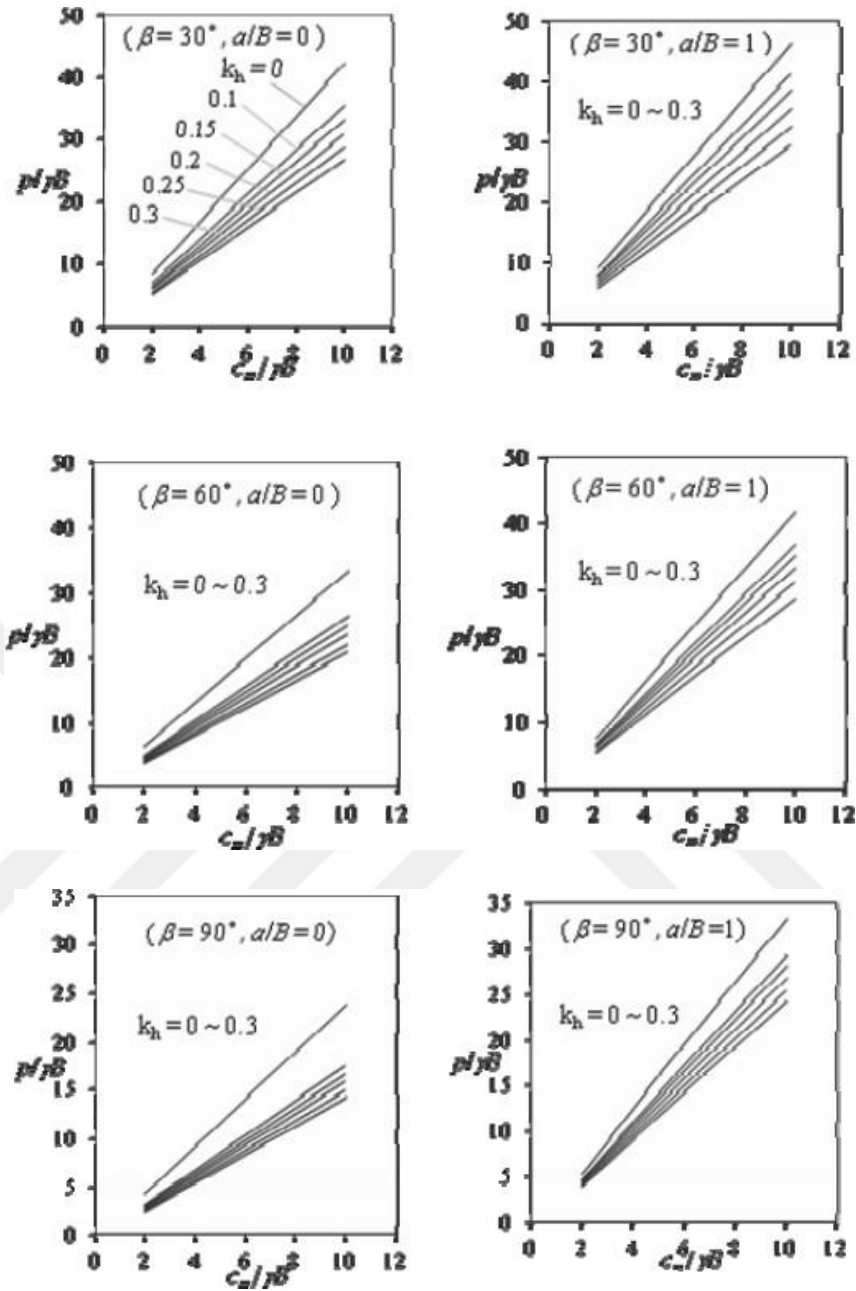


Figure 22: Design charts for seismic bearing capacity of strip footing located near cohesive slopes. (Farzaneh and Mofidi, 2013)

Cinicioglu and Erkli [16] employed the finite element method analysis using Plaxis 2D software to estimate the undrained seismic bearing capacity of surficial strip footings near purely cohesive slopes. The advantage of the finite element method lies in its ability to avoid assumptions regarding the failure surface, which is particularly

crucial in sloping areas where the failure mode can vary based on geometric properties, soil characteristics, and seismic accelerations. Figure 23 illustrates the model utilized in their analysis.

The analysis incorporated several parameters, including the normalized slope height in relation to footing width (H/B), the inclination angle (β), the normalized distance from the slope crest (λ), the normalized undrained shear strength factor ($c_u/\gamma B$), and the horizontal acceleration coefficient (k_h), as presented in Equation (25). To describe the seismic behaviour, a pseudo-static approach was adopted, considering the inertia force acting on the footing in the horizontal direction towards the slope body, which corresponds to the most critical scenario. The vertical component was disregarded due to its negligible impact on the seismic bearing capacity. The soil was modelled using a Tresca failure criterion as an elastic-perfectly plastic material, while the footing base was assumed to be rough.

$$N_{cse} = f \left(\beta, \lambda, \frac{H}{B}, \frac{c_u}{\gamma B}, k_h \right) \quad (25)$$

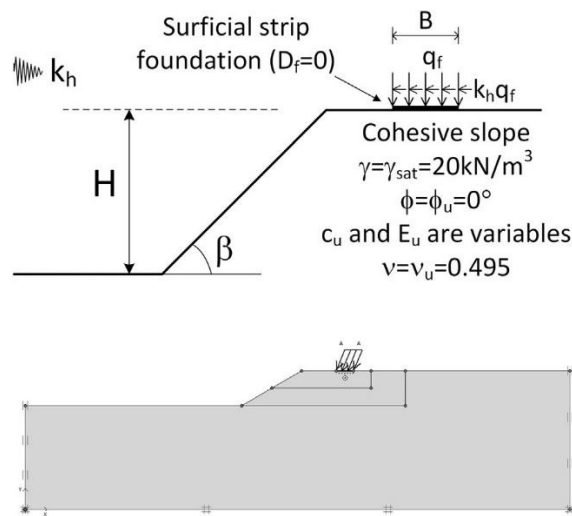


Figure 23: Schematic of the model and parameters used in the analyses. (Cinicioglu and Erkli, 2018)

The model in this study is subjected to a vertical load, denoted as q , which is incrementally increased until the model experiences failure. The resulting load value represents the bearing capacity, which is normalized to c_u to obtain a dimensionless bearing capacity factor N_{cse} . More detailed information regarding the model and the stages utilized in Plaxis 2D software can be found in Erkli (2015). The analyses yield four distinct failure modes, each of which will be discussed in the following section.

The first failure mode is overall slope failure, which occurs due to the instability of the defined geometry and the soil characteristics under gravitational or pseudo-static accelerations. In this case, increasing the geometric properties only expands the failure area, as the overall system becomes unstable under the applied accelerations.

The second failure mode is foundation failure or bearing capacity failure. In this scenario, the failure surface may extend to the slope surface or terminate at the upper ground surface, depending on the distance of the footing from the slope crest. This represents cases where the failure is not influenced by slope stability.

The third failure mode is associated with slope instability caused by the applied load on the foundation. The system remains stable under the imposed accelerations but fails due to the applied load. The failure surface extends beyond the toe of the slope.

The final failure mode is foundation sliding. The foundation slides when subjected to a horizontal acceleration value k_h greater than 0.36, even if the slope itself is stable. This occurs when the horizontal component of seismic bearing capacity $q_{ult,se}$ is equal to the undrained soil strength c_u value, resulting in $N_{cse} k_h$ being unity. The failure modes are illustrated in Figure 24.

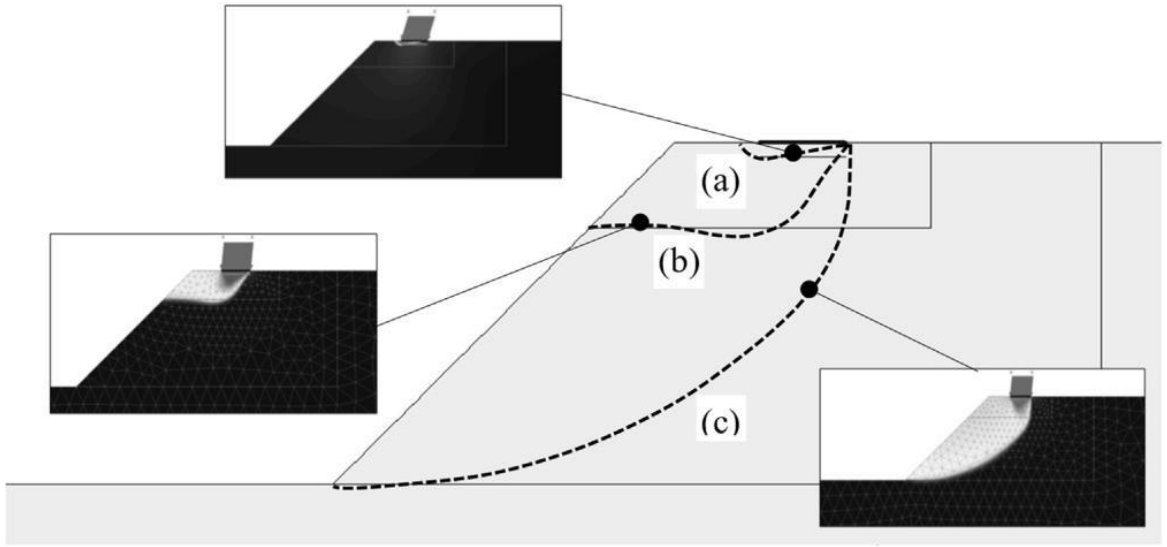


Figure 24: Failure modes obtained by FEM for different models. (Cinicioglu and Erkli, 2018)

The author introduced design charts for estimating the bearing capacity factor N_{cse} considering the influential parameters mentioned earlier in Equation (25). These parameters, along with the undrained shear strength value (c_u), which can be determined experimentally, allow for the estimation of the seismic bearing capacity of strip footings near cohesive slopes using Equation (26). The design charts, specifically the λ - N_{cse} relationship, are illustrated in Figure 25. Additional charts can be found in Erkli [17].

$$q_{ult,se} = c_u N_{cse} \quad (26)$$

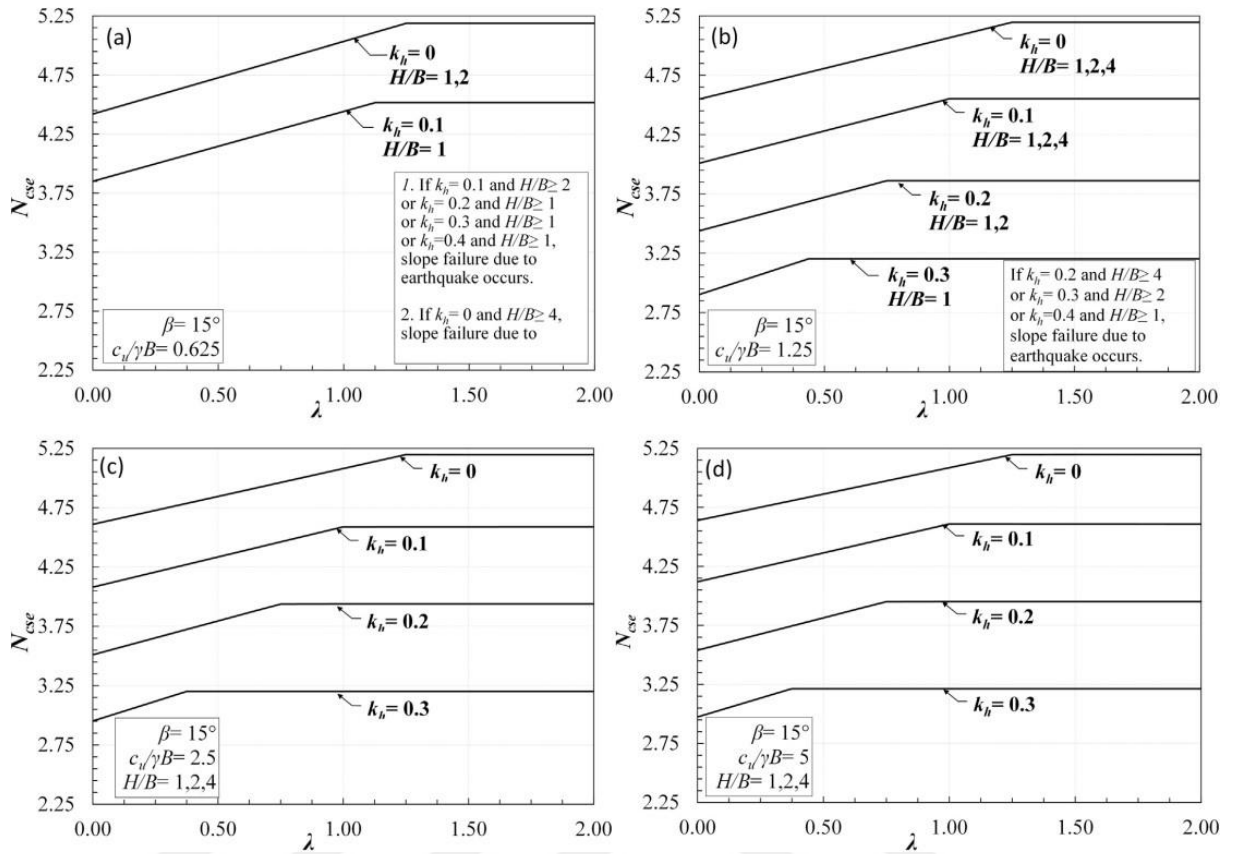


Figure 25: Design charts for seismic bearing capacity factor of strip footing near cohesive slopes with $\beta=15^\circ$. (Cinicioglu and Erkli, 2018)

The observation is clear that the N_{cse} value exhibits a linear increase as λ (the normalized distance) rises until reaching a threshold value λ_o , beyond which the failure mechanism becomes independent of the slope. However, for specific combinations of influential parameters, the relationship between N_{cse} and λ appears to differ, as depicted in Figure 26. In these cases, the seismic bearing capacity factor experiences a rapid initial increase followed by a slight drop. This behaviour can be attributed to the slopes having a barely stable soil characterization. As λ increases, the failure mode transitions from bearing capacity failure to foundation cased instability, resulting in a higher expected threshold value λ_o due to the slope's stability.

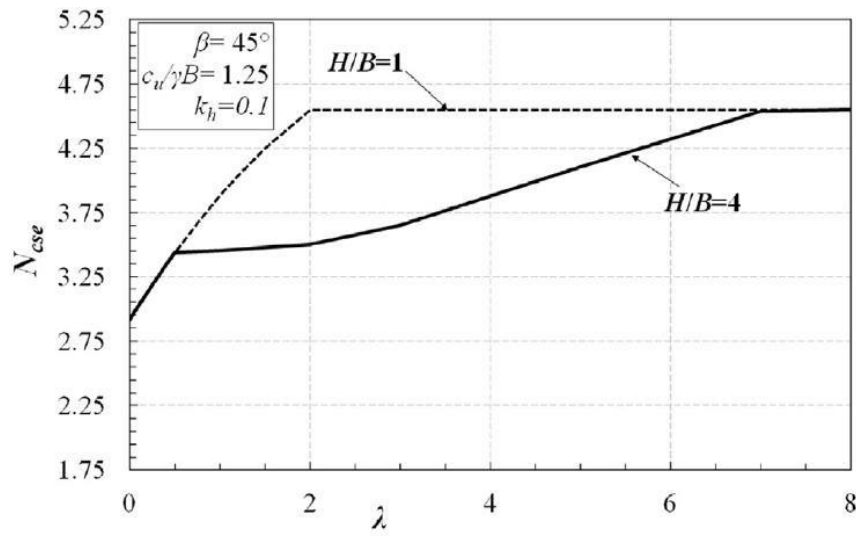


Figure 26: N_{cse} - λ relationship for barely stable slopes. (Cinicioglu and Erkli, 2018)

Luo et al. [18] conducted finite element limit analysis to estimate the undrained seismic bearing capacity of surficial strip footings near purely cohesive slopes. They utilized the AFELA program, developed by the authors, to obtain both upper bound (UB) and lower bound (LB) results for this specific problem. The influential parameters considered in their analysis were largely similar to those considered by Cinicioglu and Erkli [16], as well as the soil material. To describe the seismic behaviour in the horizontal direction, a pseudo-static approach was employed with the horizontal component represented by k_h , while the vertical component k_v was disregarded due to its negligible influence on the seismic bearing capacity. The model utilized in the analysis is depicted in Figure 27.

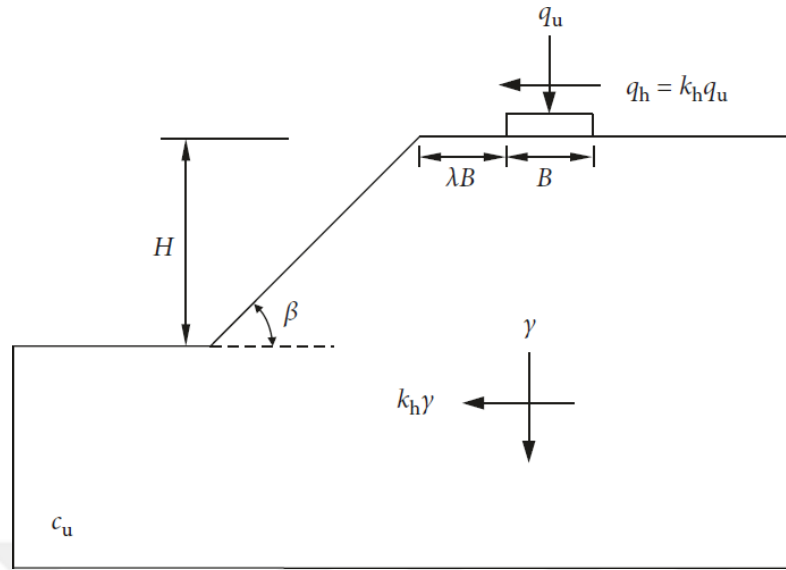


Figure 27: Schematic diagram of the model. (Luo et al., 2019)

The failure modes observed in this study generally align with those observed by Cinicioglu and Erkli [16]. However, the author categorizes the failure modes into four types: bearing capacity failure with the surface line reaching the slope surface, bearing capacity failure with the surface line extending to the slope toe, bearing capacity failure unaffected by the slope, and overall slope failure. It is important to note that the study does not consider failure due to foundation sliding under seismic conditions, as previously explained.

The upper and lower bound analyses yielded results with an average relative error of 3%, indicating a high level of accuracy in the numerical results. This low error prompted the author to use the average value of the upper and lower bound results to generate the design charts.

In Figure 28, as expected, an increase in the λ factor leads to an increase in the seismic bearing capacity factor N_{ce} , until a critical point (λ_{cr}) is reached, beyond which the influence of the slope diminishes. This critical value of λ_{cr} increases with a higher

inclination angle β . Conversely, as shown in Figure 29, an increase in the $c_u/\gamma B$ factor also results in an increase in the seismic bearing capacity factor N_{ce} , until a critical value is attained, beyond which the N_{ce} is unaffected by the soil properties. This critical value increases with an increasing inclination angle β . Generally, the seismic bearing capacity factor N_{ce} exhibits minimal influence when $c_u/\gamma B > 5$. It is worth noting that this relationship can be considered somewhat linear when $c_u/\gamma B > 2$, which aligns with the findings of Farzaneh and Mofidi [15]. Additionally, Figure 29 represents a linear relationship between N_{ce} and k_h , considering various values of β , $c_u/\gamma B$, and H/B . From this, one can conclude that the H/B factor does not significantly impact the seismic bearing capacity.

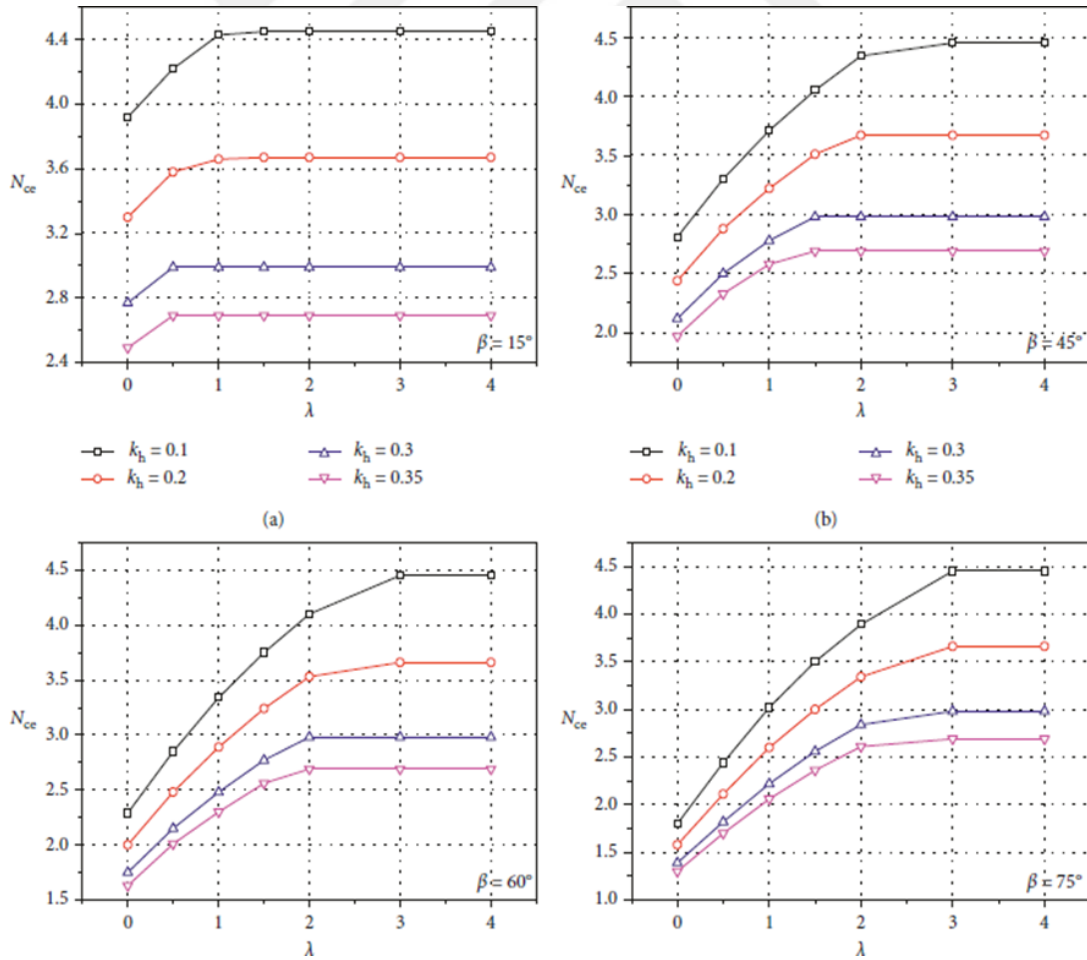


Figure 28: $N_{ce} - \lambda$ relationship for $c_u/\gamma B = 1$ and $H/B = 1$. (Luo et al., 2019)

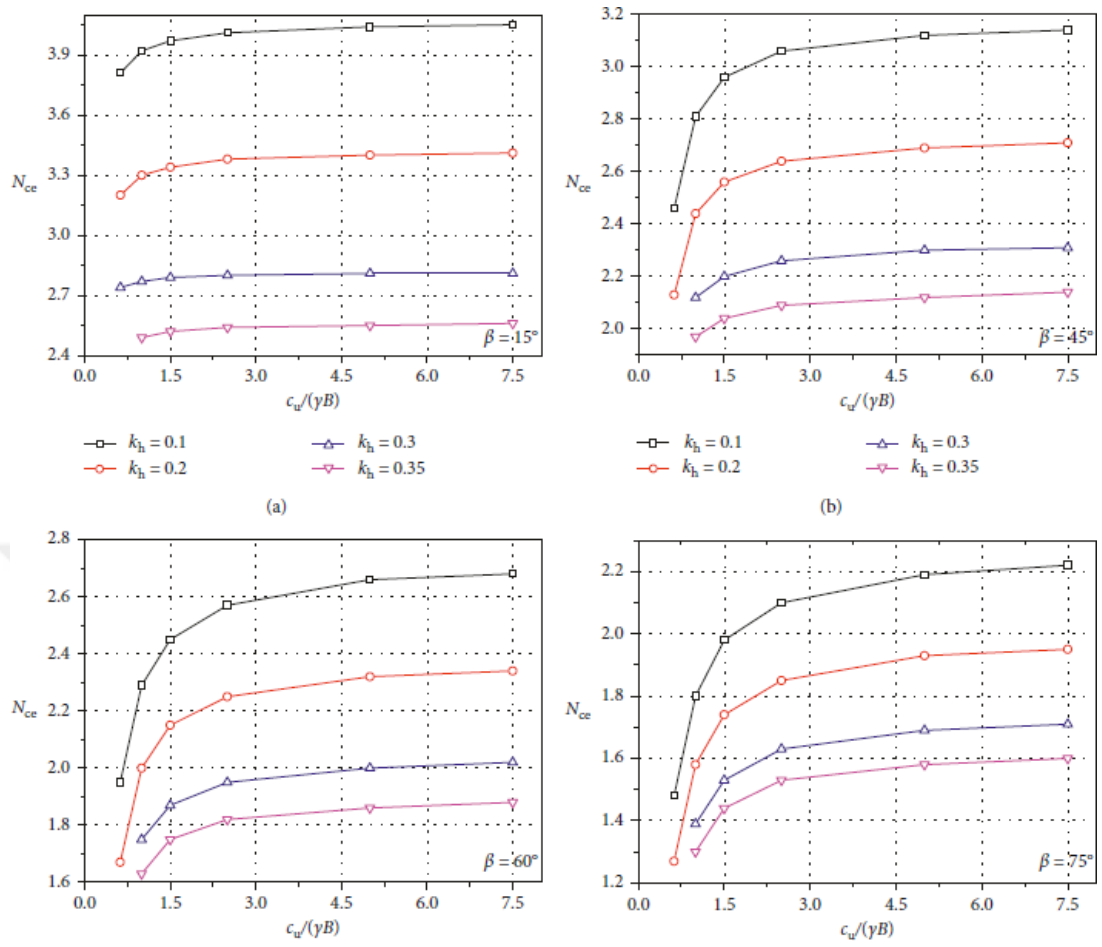


Figure 29: $N_{ce} - c_u/\gamma B$ relationship for $\lambda = 0$ and $H/B = 1$. (Luo et al., 2019)

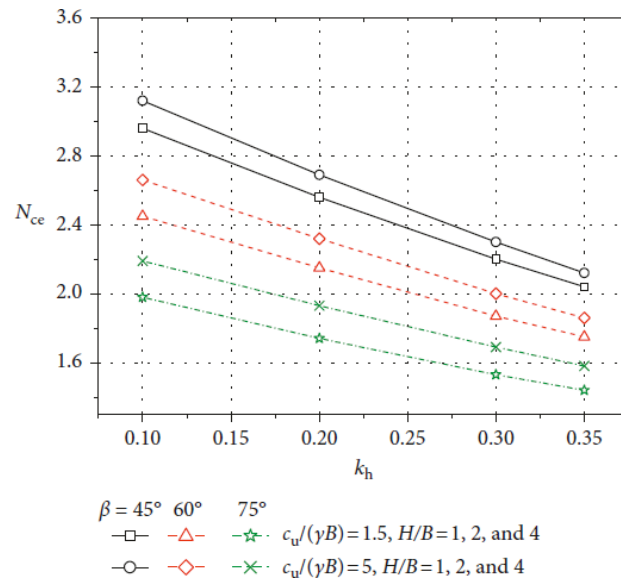


Figure 30: $N_{ce} - k_h$ relationship with various β , $c_u/\gamma B$ and H/B values.

2.3 Static and Seismic Stability of Homogeneous Cohesive Slopes

This chapter focuses on examining the stability of homogeneous cohesive slopes under both static and seismic conditions, discussing various methods and studies. Homogeneous cohesive slopes can experience instability solely under their own loads, independent of any foundation loads, or they can fail due to the inertia force of an earthquake. In this regard, Taylor's method [19] and Koppula's method [20] will be employed to compare the stability of slopes under static and seismic conditions, respectively. The objective is to assess the consistency of their results with the stability outcomes obtained through Finite Element Method (FEM) analysis conducted in this study.

Taylor [19] provided analytical solution for the static stability of cohesive slopes, assuming the circular failure mechanism. Taylor [19] proposed Equation (27) which is used to find the equilibrium case of cohesive slope along the failure surface,

$$c_u = m\gamma H_{cr} \quad (27)$$

where c_u is the undrained shear strength, m is the unitless stability number that can be found using the stability chart provided by Taylor [19], γ is soil unit weight and H_{cr} is the critical slope height.

As represented in Figure 31, when the slope angle $\beta < 53^\circ$, the stability number m is a function of both β and the normalised stratum depth D ($D = \text{stratum depth/slope height}$). Whereas, when $\beta > 53^\circ$ the stability number m changes depending on β only.

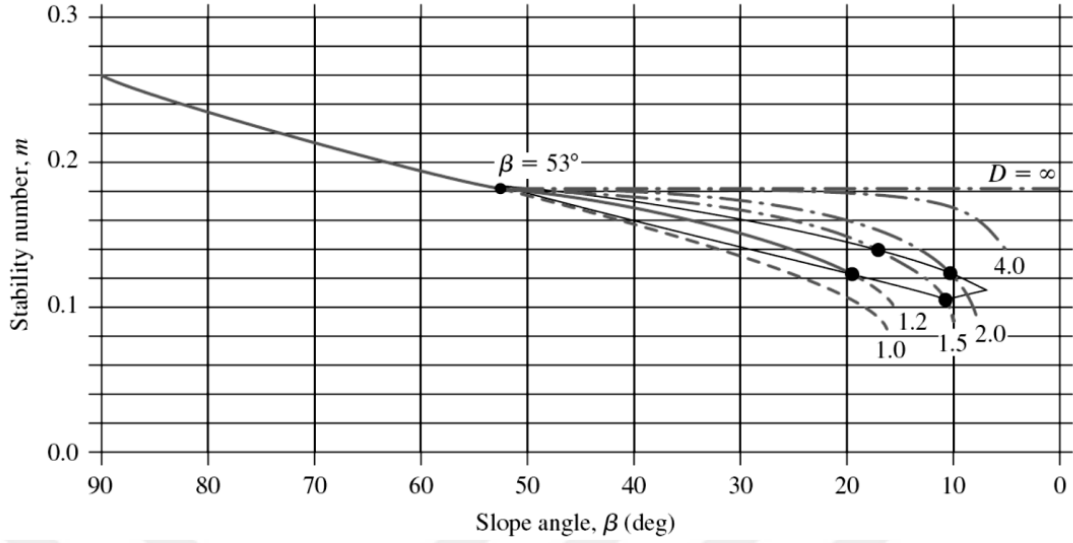


Figure 31: Variation of stability number m with slope angle β . (Taylor, 1937)

(Adapted from Das, 2009)

Koppula [20] studied the seismic stability of cohesive slopes considering the case where the soil shear strength changes with depth. Equation (28) represents a solution to find the factor of safety of such a case,

$$F_s = \frac{a_0}{\gamma} N_1 + \frac{c_0}{\gamma H} N_2 \quad (28)$$

where a_0 is the change in shear strength with depth, γ is soil unit weight, c_0 is the initial soil shear strength, H is the slope height and N_1 and N_2 are stability number can be read by the stability charts provided by Koppula [20].

In this study, only the homogenous purely cohesive soil case, where the soil shear strength is constant, will be considered. Equation (28) was revised assuming that a_0 is zero, c_0 is c_u and F_s is unity to represent the case investigated in this study, resulting in the Equation (29),

$$H_{cr} = \frac{c_u}{\gamma} N_2 \quad (29)$$

The same with Taylor's [19] findings, when the slope angle $\beta < 53^\circ$, the stability number N_2 is a function of both β and the normalised stratum depth D ($D = \text{stratum depth/slope height}$). Whereas, when $\beta > 53^\circ$ the stability number N_2 changes depending on β only, as shown in Figures 32 and 33.

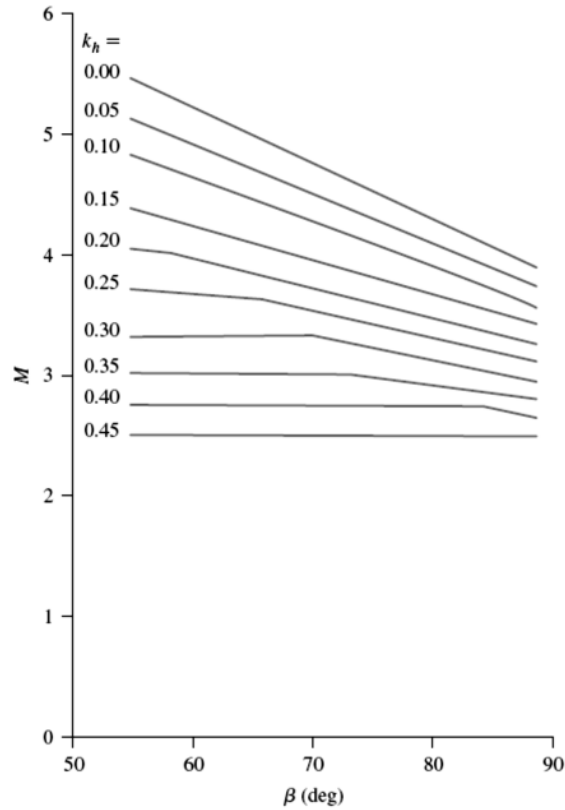


Figure 32: Variation of N_2 and β with different k_h for $\beta > 53^\circ$. (Koppula, 1984)

(Adapted from Das and Ramana, 2010)

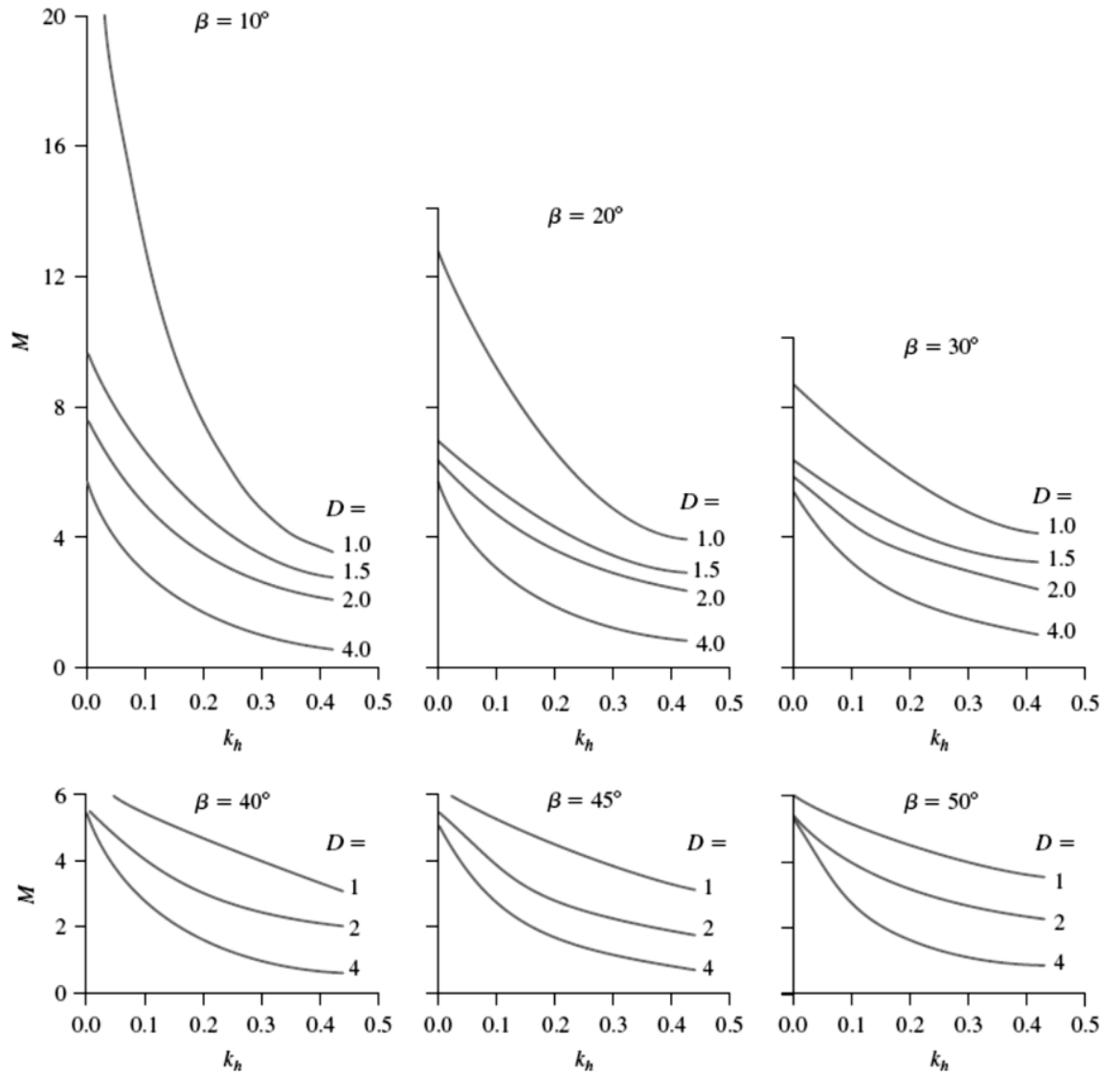


Figure 33: Variation of N_2 and β with different k_h for $\beta < 53^\circ$. (Koppula, 1984)

(Adapted from Das and Ramana, 2010)

CHAPTER III

ANALYSIS METHODOLOGY

This chapter will provide information about the analysis method used in this study (FEM) using Plaxis 3D and Plaxis 2D softwares. The first part provides brief information about the use of FEM in geotechnical engineering problems with emphasis on the use of the commercial geotechnical engineering FEM programs Plaxis 3D and 2D to tackle slope stability and bearing capacity problems which are the focus of this study. Later, the problem that will be studied in this thesis will be defined and the assumptions related to the problem, will be explained in detail. Depending on the assumptions, previous studies that investigated the bearing capacity problem will be used to propose a simplified equation to solve the problem. Following, details of the parametric study will be summarized explaining the impact of each parameter on seismic bearing capacity. Additionally, the data inputted into the Plaxis 3D software, including model geometry, meshing, construction stages, and output evaluation, will also be thoroughly explained.

The last part of this chapter will include a comparison between this study's results with the results obtained by previous researchers in the same field in order to compare the consistency between both results. However, the comparison will include the level ground case (Skempton [5], Michalowski [10], Gouvernec [12] and Salgado et al. [21]) and slope case under static conditions only (Yang et al. [13]), as no studies exist in the literature that studied the seismic bearing capacity of triangular footings located near slopes. This part will also include a comparison of the slope stability results

and failure mechanisms obtained by Plaxis 3D to those obtained by previous studies in the literature.

3.1 The Use of FEM In Geotechnical Engineering and Plaxis 2D and 3D

3.1.1 Finite Element Method and Geotechnical Engineering

Finite Element Method (FEM) is a numerical technique widely used in geotechnical engineering to analyse and solve complex problems related to soil and rock mechanics. It provides a systematic and efficient approach to simulate the behaviour of geotechnical systems under various loading and boundary conditions. The FEM divides the entire domain of interest into smaller elements, enabling the analysis of local behaviour and the aggregation of these elements to understand the overall response of the system.

The use of FEM in geotechnical engineering problems can be summarized as below:

Problem Formulation: The first step in using FEM is to define the problem. This includes identifying the geometry and dimensions of the geotechnical system, specifying the material properties of the soil or rock, and defining the boundary and loading conditions. The problem can range from simple 2D plane strain or axisymmetric problems to complex 3D problems involving non-linear soil behaviour.

Mesh Generation: In FEM, the continuous domain is discretized into smaller finite elements. The choice of element shape (triangular or quadrilateral in 2D, tetrahedral or hexahedral in 3D) depends on the complexity of the problem and the desired accuracy. The mesh should be refined in areas where the stress or strain

gradients are expected to be high, such as near foundations, slopes, or areas of concentrated loads.

Element Properties and Material Behaviour: Each finite element has specific properties associated with it, such as geometric properties (area, volume), material properties (elastic modulus, Poisson's ratio), and integration points. The material behaviour is typically represented using constitutive models that describe the stress-strain relationships of soils or rocks, accounting for factors like strain hardening, softening, and non-linearity. Different elements within the mesh can have different material properties, allowing for the representation of complex geological conditions.

Discretization and Equation Formulation: The governing equations, such as equilibrium equations and compatibility equations, are discretized using variational methods, such as the principle of minimum potential energy. This involves converting the differential equations into algebraic equations using shape functions, which interpolate the unknown variables (e.g., displacements, pore pressures) within each element. The equations are then assembled into a global system of equations.

Boundary and Loading Conditions: The boundary conditions define the constraints applied to the geotechnical system, while the loading conditions represent external loads acting on the system. Examples of boundary conditions include fixed supports, roller supports, or prescribed displacements. Loading conditions can include static loads, dynamic loads, water pressure, temperature changes, and more. These conditions are applied to the appropriate nodes or elements in the finite element model.

Solution and Analysis: Once the global system of equations is formulated, it is solved iteratively using numerical methods like the matrix inversion method or iterative solvers. The unknown variables, such as displacements and pore pressures, are obtained for each element and node within the mesh. The solution provides information about the behaviour of the geotechnical system, such as deformations, stresses, strains, and pore pressures, under the specified conditions.

Post-processing and Interpretation: After obtaining the solution, post-processing techniques are employed to analyse and interpret the results. This can involve generating contour plots, stress distribution diagrams, deformation profiles, and failure surfaces. The outputs are compared with design criteria, safety factors, and geotechnical standards to assess the performance and stability of the system. Post-processing also helps in making decisions regarding reinforcement requirements, slope stability, foundation design, and other geotechnical considerations.

The use of FEM in geotechnical engineering offers several advantages. It allows for realistic modelling of complex soil-structure interactions, provides insights into localized behaviour, enables the analysis of non-linear and time-dependent soil behaviour.

3.1.2 Plaxis 2D and Plaxis 3D

Plaxis is a software package that is widely used for two-dimensional and three-dimensional FEM analysis in geotechnical engineering. The software provides a simple interface which allows the user to create 2D and 3D models of soil and structures. The software has a large library of pre-defined soil and structural elements, and it allows the user to define custom elements as well. Plaxis also provides advanced features such as

automatic mesh generation, time-dependent material behaviour, and soil-structure interaction analysis.

FEM-based geotechnical analysis using Plaxis 2D and 3D has been used to analyse a wide range of geotechnical problems. For example, FEM analysis has been used to analyse the stability of slopes, the behaviour of retaining walls, the settlement of foundations, and the interaction between soil and underground structures. FEM analysis has also been used to analyse the behaviour of soil under dynamic loading conditions such as earthquakes.

3.2 Definition of The Problem

To investigate the impact of footing shape on the undrained seismic bearing capacity of surficial foundations located near slopes, a parametric study with various variables was conducted using Plaxis 3D, a Finite Element method-based software. The Finite Element method was chosen over the Limit equilibrium method because it provides a solution without presumptions about the failure surface and does not require assumptions about slice forces.

The assumptions related to the problem of this study can be summarised as below:

- The footing is located on the surface of the soil to represent the most critical case.
- The foundation is represented by a rectangular rigid plate with a constant width (B) and varied length (L) to cover different foundation shapes.

- The soil behaviour is modelled using an elastic-perfectly plastic model adopting Tresca failure criterion.
- The soil is uniform with no variation in properties with depth.
- For seismic activity, a pseudo-static approach is employed, with the horizontal component of seismic acceleration (k_h) being the only significant factor, as the vertical component (k_v) has negligible influence on the seismic bearing capacity (Kramer [22]).
- The inertia force of the structure is considered to act at the foundation base.

In the process of constructing the Plaxis 3D model, the slope edge line is positioned parallel to the y-axis and perpendicular to the x-axis. Additionally, the slope surface is oriented perpendicular to the x-z plane and forms an angle (β) with the x-y plane, as shown in Figure 34. Since this study focuses on the most critical case, the horizontal seismic acceleration component should be set through x-axis only, as a seismic acceleration through y-axis will not examine the slope influence. As a result, seismic acceleration component k_h through the y-axis is set to zero.

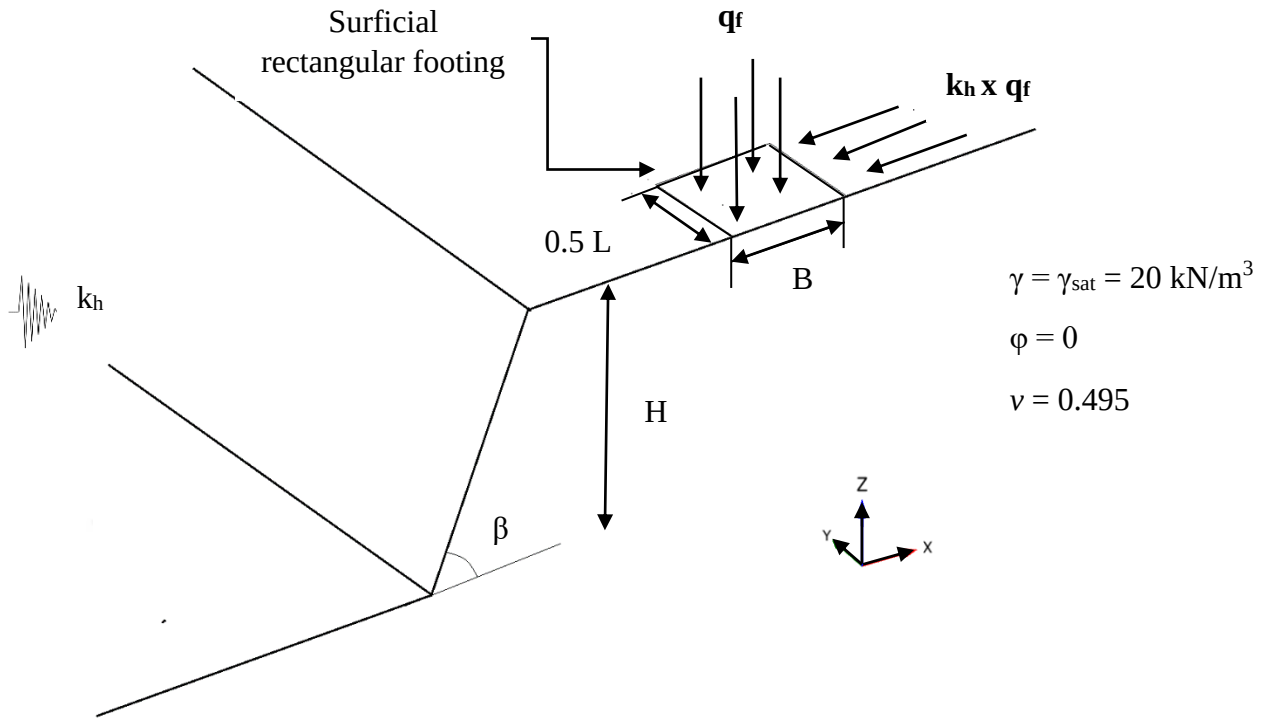


Figure 34: Schematic of the problem investigated in this study.

3.3 Solution Method

To solve the problem mentioned in Chapter 3.2, a staged analysis using Plaxis 3D software should be conducted to determine the seismic bearing capacity of each model based on the assumed input values. The model should be subjected to incrementally increased loads until failure occurs, which can be identified as the ultimate bearing capacity of the model. Ultimate bearing capacity (q_u) of surficial foundations located on homogeneous undrained soils is a direct function of undrained shear strength (c_u) as described by Skempton [5]:

$$q_u = c_u N_{cs} \quad (30)$$

where N_{cs} is the coefficient of proportionality referred to as seismic bearing capacity factor.

The main objective of this study is to determine and quantify undrained seismic bearing capacity factor N_{cs} for different problem geometries and soil properties. The focus is on investigating the impact of footing shape on the seismic bearing capacity of foundations situated near cohesive slopes. The outcomes of this study may serve as a basis for developing design charts that engineers can employ to estimate the seismic bearing capacity in similar scenarios. To achieve this, a comprehensive parametric study with varied parameter values will be conducted, as detailed in the subsequent chapter, to thoroughly investigate the influencing factors.

3.4 Parametric Study

Rectangular surficial footings with rough bases will be subjected to parametric analyses. The short side (B), which is the foundation width as illustrated in Figure 34, was fixed at two meters while the long side (L) was chosen as $L = 0.5, 2, 4, \text{ and } 8 \text{ m}$, corresponding to $B/L = 0, 0.25, 0.5, 1 \text{ and } 4$ used as the dimensionless parameter to define variations in dimensional properties. The value of $B/L = 0$ corresponds to the continuous footing case, as indicated by Cinicioglu and Erkli [16].

For all the analyses, the Mohr-Coulomb elastic-perfectly plastic constitutive soil model with $\phi = 0$ and $\gamma = \gamma_{sat} = 20 \text{ kN/m}^3$ was used. Adoption of $\phi = 0$ corresponds to the assumption of undrained behaviour. The values of soil strength c_u were varied as 25, 50, 100, and 200 kN/m^2 to test the response of soft, stiff, and hard soils. The undrained stiffness, which has no practical influence over the stability problem, was determined using the equation $E_u = 200c_u$, and the undrained Poisson's ratio was assumed to be 0.495 to avoid singularity in the stiffness matrix.

The footing was placed at various distances s from the slope crest, and this distance was normalized by footing width B , which was defined as $\lambda = s/B = 0, 0.5, 1$, and 2 . The height of slope H was normalized by the footing width B as $H/B = 1, 2$, and 4 , while the slope angle values β were $15^\circ, 30^\circ, 45^\circ$, and 60° to define the slope shape. Moreover, analyses with β equals zero will be conducted to examine the level ground case.

For the horizontal seismic acceleration component, $k_h = 0.1, 0.2$ and 0.3 values were used in this study. The reason of not using $k_h = 0.4$ is that all the models fail under this value as indicated by Cinicioglu and Erkli [16]. Additionally, a set of analyses with $k_h = 0$ values were conducted to represent the static conditions response with varying foundation shapes and provide a basis of comparison to interpret the influence of seismic activity. Table 5 presents the values of the parameters used in this study.

Table 5: Parameters used in this study.

Parameter	Value	Explanation
H/B	0, 1, 2, 4	Slope height normalized by footing width.
B/L	0, 0.25, 0.5, 1, 4	The footing length.
B	$0^\circ, 15^\circ, 30^\circ, 45^\circ, 60^\circ$	Slope angle.
λ	0, 0.5, 1, 2	The distance between the footing and the slope crest normalized by footing width.
$C_u/\gamma B$	0.625, 1.25, 2.5, 5	Undrained shear strength normalized by the production of the soil unit weight and footing width.
k_h	0, 0.1, 0.2, 0.3	Horizontal seismic acceleration component.

3.5 Plaxis 3D Data Input

This chapter will provide an overview of the project properties employed in this study to create the models that will be utilized in the analyses, using Plaxis 3D software. The explanation will encompass the key settings and configurations employed in generating the models for subsequent analysis.

3.5.1 Project Properties

The Project Properties window in Plaxis 3D software consists of three tabs: "Project", "Model", and "Cloud Services" as shown in Figure 35, 36, and 37, respectively. The "Project" tab is used to define the project title, file name, company name and logo, as well as adding comments to the project. The "Model" tab has four options. In the "Type" option, the model is set to "full" and the elements are set to "10-noded" (further explanation about 10-noded settings will be provided later in this chapter). In the "Gravity" option, the default earth gravity is set to 9.810 m/s^2 , while the unit weight of water is set to 10 kN/m^3 , which is not critical as this study focuses on the undrained case. The units of length, force, and time are selected as m, kN, and day, respectively. In the "Contour" option, the dimensions of each model are entered based on the input values. The last tab, "Cloud Services" is a storage cloud that is provided by the owner company of Plaxis 3D.

The soil volume in Plaxis 3D software is represented using 10-node tetrahedral elements, which are created in the 3D mesh procedure. These elements utilize second-order interpolation of displacements. Each tetrahedral element has three local coordinates (ζ , η , and ξ). Detailed explanations about the interpolation functions, their derivatives, and the numerical integration for this type of element can be found in the

Plaxis 3D - Scientific Manual. The local numbering and positioning of nodes and integration points for a 10-noded tetrahedral element are illustrated in Figure 38.

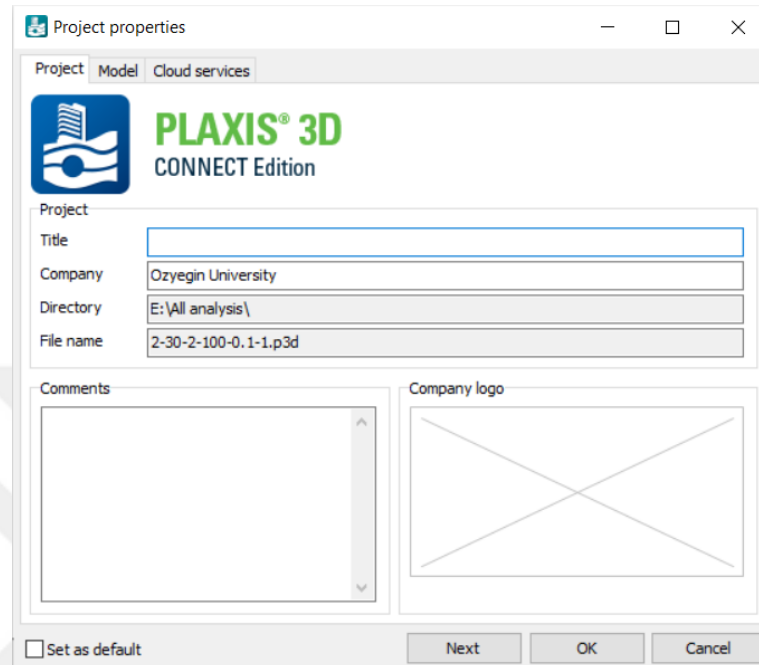


Figure 35: "Project" tab of Project Properties window.

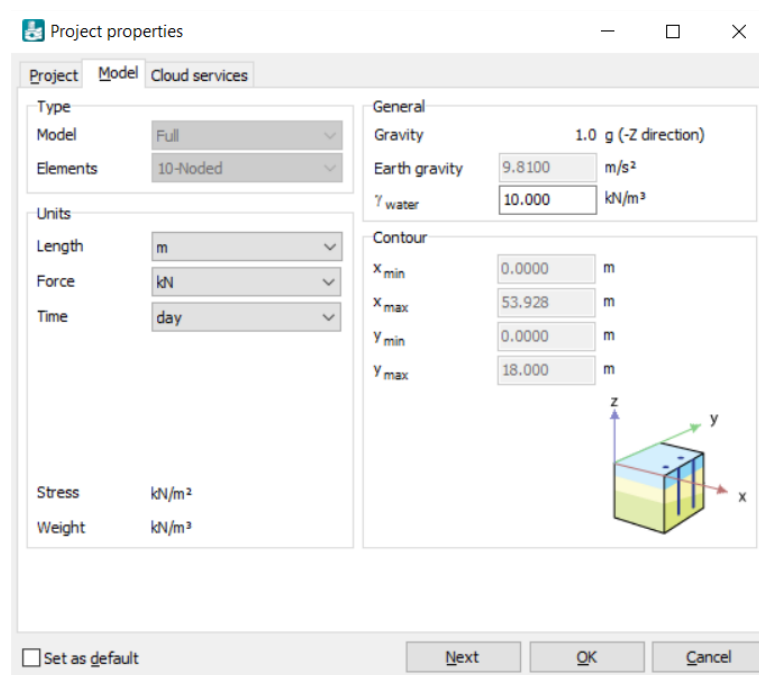


Figure 36: "Model" tab of Project Properties window.

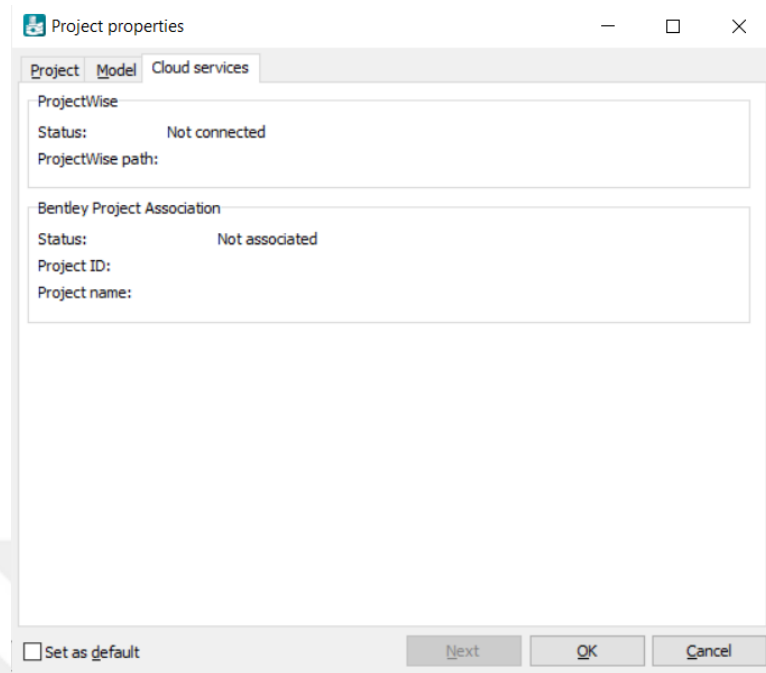


Figure 37: "Cloud Services" tab of Project Properties window.

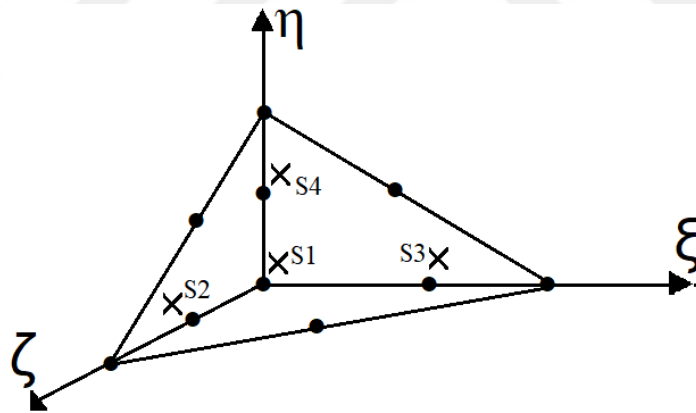


Figure 38: The local numbering and positioning of nodes and integration points for a 10-noded tetrahedral element.

3.5.2 Model Geometry

The selection of an appropriate model geometry is of utmost importance in determining the ultimate load for a given problem, as it ensures that the calculated results are not influenced by the assumed boundaries in the model. In this study, the model geometry

proposed by Erkli [17] for plane-strain is adopted and modified for three-dimensional problems. As illustrated in Figure 39, the boundary along the x-axis is located at a distance of $8B$ from the edge of the footing, while the boundary along the y-axis is positioned at $8L$ from the edge of the footing. To eliminate any possibility of size effects, the front and bottom boundaries are positioned at $12.5B$ and $6B$ from the slope toe, respectively. Additionally, the model is divided into two halves with a symmetric surface along the xz plane passing through the centre of both the slope and the rectangular footing. Only one half of the model is utilized during the analysis, taking advantage of the symmetry to optimize the calculation time. The FEM model is represented in Figure 39.

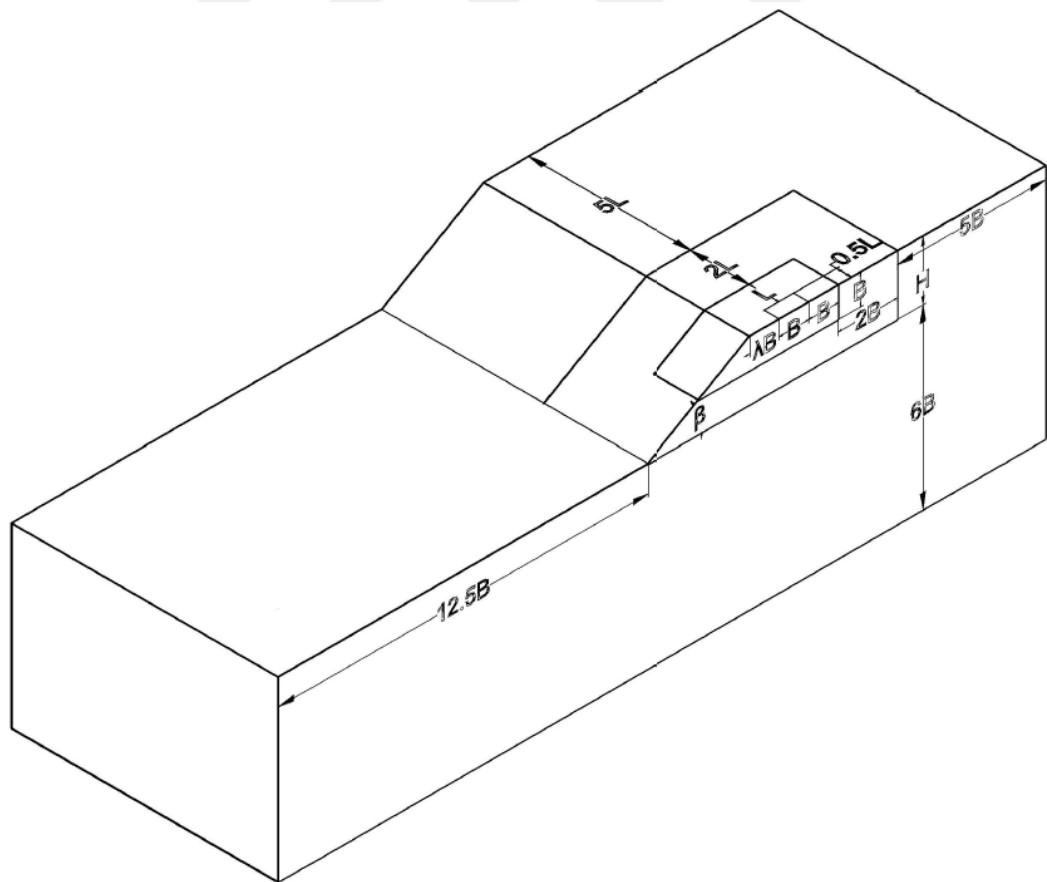


Figure 39: Schematic of FEM model used in this study.

3.5.3 Properties of Surficial Footing

Surficial rectangular footing with a constant width $B=2\text{m}$ and various lengths L represented by a weightless elastic rigid plate is used in this thesis. The surficial footing is assumed to be fully rough in all the analysis. Accordingly, the interface strength reduction factor R_{inter} value is set to a unity. The plate is set to isotropic type. The mechanical properties of the plate are represented in Table 6.

Table 6: Mechanical properties of the plate.

Material Type	Young's modulus, $E1$ (kN/m ²)	Poisson's ratio, $\nu12$ (Unitless)	Plate thickness, d (m)	In-plane shear modulus, $G12$ (kN/m ²)	Unit weight, γ (kN/m ³)
Elastic, isotropic	1.5×10^7	0.2	1	6.25×10^6	0

3.5.4 Boundary Conditions

Plaxis 3D software automatically applies standard fixities to the boundaries of the geometry model. Specifically, the vertical model boundaries in the x-direction are fixed ($u_x=0$), while being free in the y- and z-directions. Similarly, the vertical model boundaries in the y-direction are fixed ($u_y=0$), while being free in the x- and z-directions. The vertical model boundaries in the z-direction are fixed ($u_z=0$), while being free in the x- and y-directions. The model bottom boundary is fixed in all directions ($u_x= u_y= u_z=0$), while the ground surface remains free in all directions.

3.5.5 Loads

In seismic conditions, the loads acting on the rectangular surficial footing can be classified into two categories: vertical load and horizontal load. The former denotes the load that is transferred from the superstructure onto the foundation (q_f), whereas the

latter represents the inertia force that affects the foundation ($k_h \times q_f$) along the x-axis. To ensure the overall stability of the slope, the vertical load applied after the initial stress generation is set to unity in the z-direction, as depicted in Figure 40. In order to achieve the most critical scenario, the inertia force should act outward from the slope. To accomplish this, a positive value of "PseudoStatic" is set over the x-direction in the "Model conditions" list, under the "Staged construction" tab. In the final phase, the magnitude of the vertical stress in the z-direction (q_f) is incrementally increased to a very high value (-10,000 kN/m²), which has the potential to induce failure in the entire model. The horizontal stress in the x-direction that represents the inertia force is $k_h \times q_f$, depending on the value of k_h for the model. Figure 41 illustrates the applied stresses in the last phase.

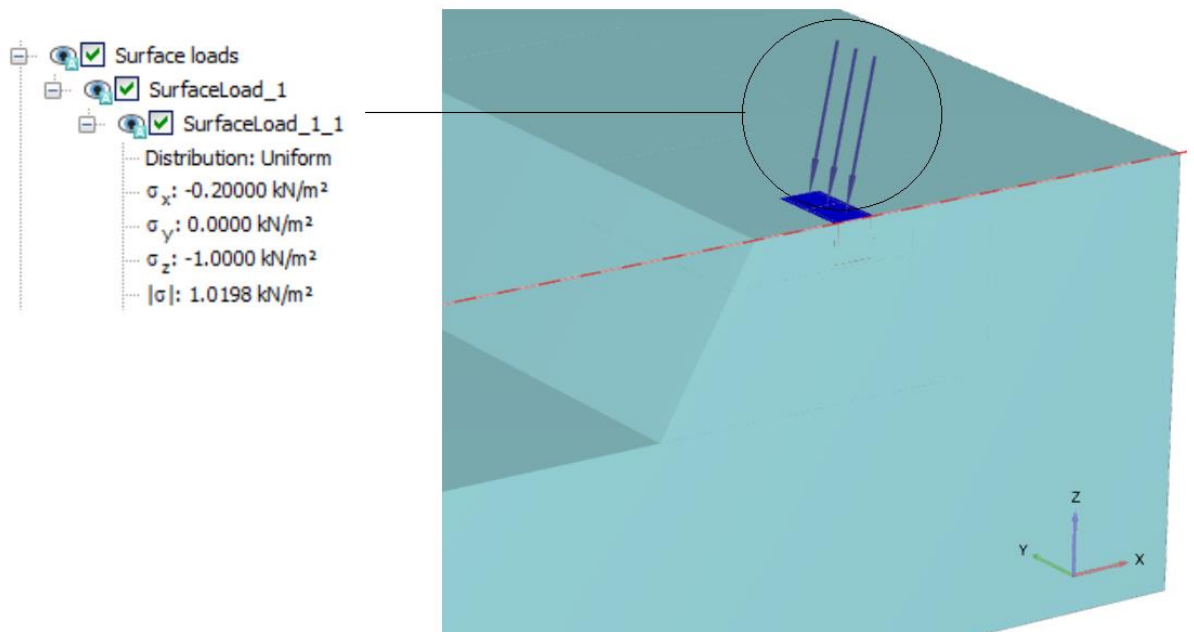


Figure 40: Loads applied after the initial stress generation ($k_h=0.2$).

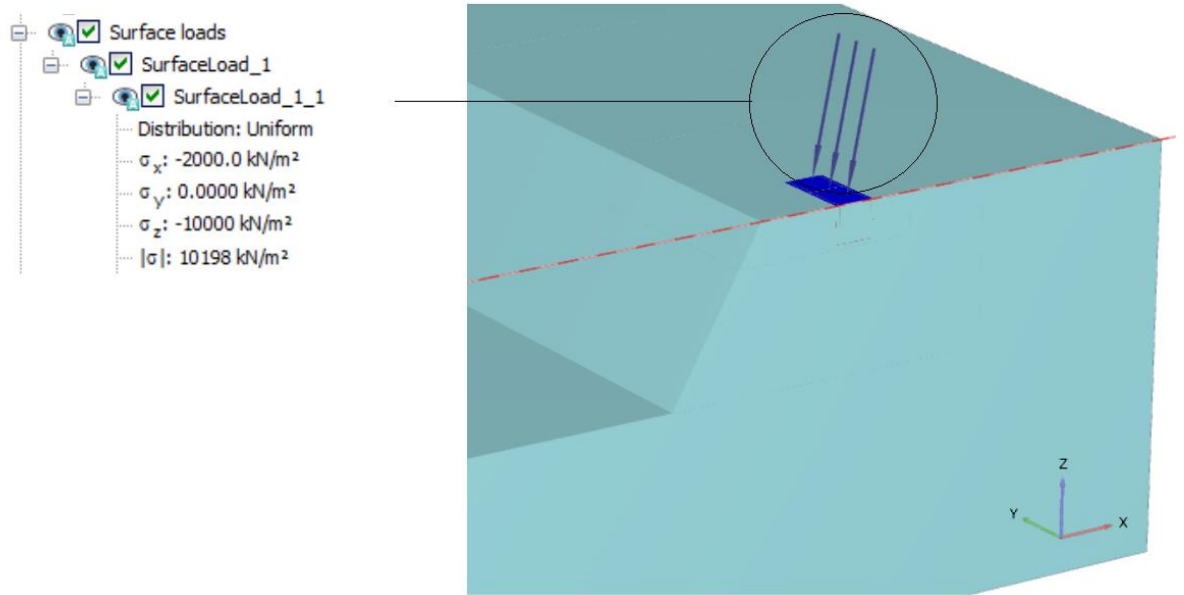


Figure 41: Loads applied at the last phase ($k_h=0.2$).

3.5.6 Material Properties

Plaxis 3D software provides three options (Undrained A, Undrained B, and Undrained C) for modelling soil material in the undrained condition. For this study, "Undrained C" was found to be the most suitable option as it defines stiffness and strength in terms of undrained values. The analysis was carried out using total stresses, and no excess pore pressures or effective stresses were explicitly calculated. Therefore, only two parameters, undrained elasticity modulus ($E_{u,ref}$), and undrained shear strength (c_u), were required to model the undrained behaviour using the Mohr-Coulomb model. The value of undrained elasticity modulus ($E_{u,ref}$) was defined by the equation ($E_u = 200 c_u$) for all models, making only undrained shear strength (c_u) necessary for modelling the undrained behaviour using the Mohr-Coulomb model.

In this study, Tresca failure criterion is assumed. So, in addition to undrained elasticity modulus ($E_{u,ref}$) and undrained shear strength (c_u), parameters such as

undrained Poisson ratio (v_u), undrained internal friction angle (ϕ_u), and dilation angle (Ψ) must also be defined. When "Undrained C" is chosen, Plaxis 3D automatically sets the undrained internal friction angle (ϕ_u) and dilation angle (Ψ) to zero. Theoretically, undrained Poisson ratio (v_u) should have a value of 0.5 to avoid volume change in undrained conditions. However, defining a value between 0.495 and 0.499 is more convenient to avoid stiffness matrix singularity. Therefore, a value of 0.495 was chosen for undrained Poisson ratio (v_u) in all finite element analyses.

The parametric study used four soil data sets corresponding to different cohesive soil stiffness (soft, medium, stiff, hard), and their stiffness and strength parameters are presented in Table 7. Additionally, the unsaturated unit weight of soil (γ_{unsat}) was defined as constant (20 kN/m³) for all soil data sets. As the material behaviour type was chosen as "Undrained C," the Plaxis 3D software automatically deactivated the γ_{sat} data box, and the constant value of γ_{unsat} was input in its place. The interface strength reduction factor (R_{inter}) was defined as one for all soil data sets due to the rough footing assumption.

Table 7: Material properties of soils used in this study.

Parameter			Soil Stiffness			
			Soft	Medium	Stiff	Hard
	Name	Unit	Value			
Material model	Soil model	-	Mohr-Columb	Mohr-Columb	Mohr-Columb	Mohr-Columb
Undrained condition	Drainage type	-	Undrained C	Undrained C	Undrained C	Undrained C
Soil unit weight above phreatic level	γ_{unsat}	kN/m ³	20	20	20	20
Soil unit weight below phreatic level	γ_{sat}	kN/m ³	-	-	-	-

Permeability in x-direction	k_x	m/day	0	0	0	0
Permeability in y-direction	k_y	m/day	0	0	0	0
Permeability in z-direction	k_z	m/day	0	0	0	0
Undrained elasticity modulus	$E_{u,ref}$	kN/m ²	5000	10000	20000	40000
Undrained Poisson ratio	ν_u	-	0.495	0.495	0.495	0.495
Undrained shear strength	$s_{u,ref}$	kN/m ²	25	50	100	200
Undrained friction angle	ϕ_u (phi)	°	0	0	0	0
Undrained dilatancy angle	Ψ (psi)	°	0	0	0	0
Interface strength reduction factor	R_{inter}	-	1	1	1	1

3.5.7 Mesh Generation

In the course of modelling studies, efforts were undertaken by Erkli [17] to identify an optimal procedure for creating a finite element mesh that would ensure sufficient accuracy in problem-solving while maintaining acceptable levels of computer memory usage and computational time. This was deemed crucial in view of the considerable number of numerical models developed as part of the effort to establish a comprehensive methodology. The mesh creation procedure suggested by Erkli [17] have been employed in this study, along with the inclusion of the third dimension and certain modifications, which will be elucidated in the subsequent sections.

According to Yamamoto's [23] findings, the failure surface is located at shallower depths beneath the ground surface as the horizontal acceleration coefficient (k_h) increases. Therefore, studying a problem under static conditions would be most appropriate to determine the possible extent of high stress and stiffness regions. For that

reason, problem of strip footings on level ground was employed in both 2D and 3D scenarios, as it has an exact solution studied by Prandtl [2].

In order to obtain a mesh creation procedure that could provide a bearing capacity factor (N_c) value close to the exact solution of the problem ($\pi+2= 5.14$), various procedures were tested using both Plaxis 2D and Plaxis 3D software. The results obtained from Plaxis 2D and Plaxis 3D yielded values of 5.14 and 5.27 with error percentages of 0% and 2.5%, respectively. Additionally, the failure mechanism obtained by Plaxis 3D is consistent with the one proposed by Prandtl [2], as depicted in Figure 42.

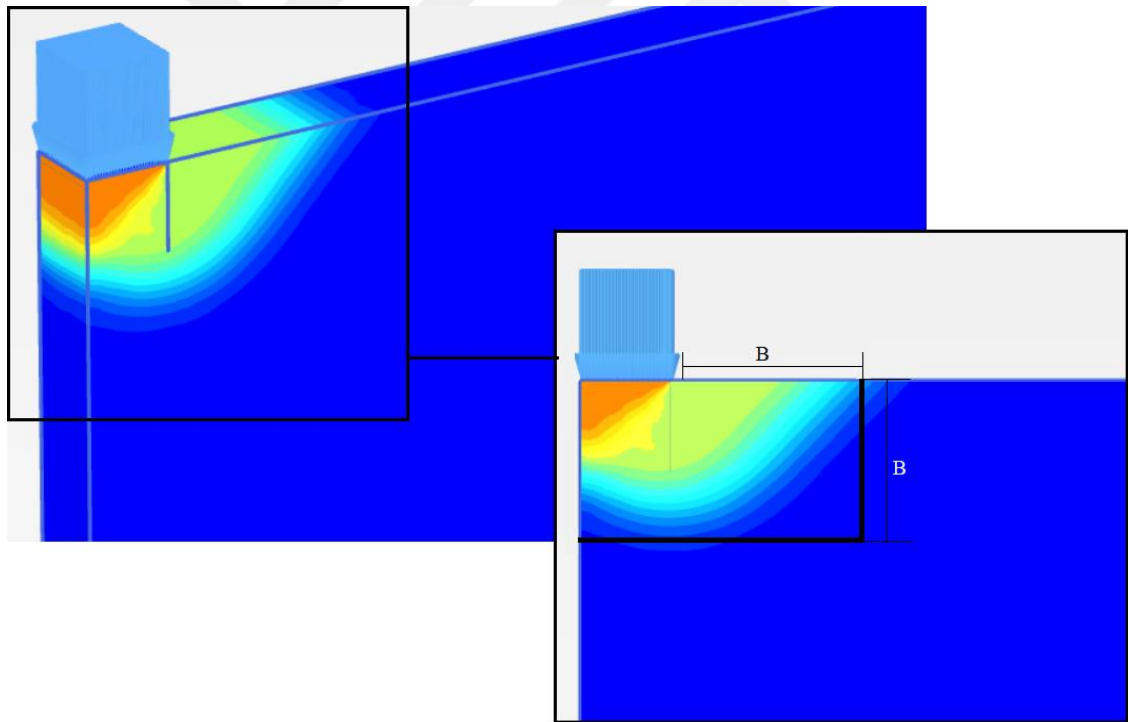


Figure 42: Failure mechanism of strip foundation rested on surface ground obtained by Plaxis 3D.

Various tests were conducted using Plaxis 3D to develop a meshing technique suitable for the entire model, in addition to the meshing procedure for the area beneath

the foundation. As suggested in Plaxis 3D Validation - Bearing capacity of strip footing (2018), the addition of a vertical surface with a depth equals to half of the footing width (0.5B) below each edge of the footing enables local mesh control, as shown in Figure 44. To improve accuracy, positive surfaces are assigned to each vertical surface to increase node density, especially in regions prone to deformation. Positive surface is a technique used in finite element analysis to increase the accuracy of the solution by increasing the density of nodes on the surface with the highest potential for deformation, which can enhance the accuracy of the solution and reduce the probability of analysis errors. The final meshing technique used in this study is explained as four steps:

- The coarseness factor of the footing is set to 0.125, while the global coarseness of each edge is set to 0.5.
- The coarseness factor of the vertical surfaces under the footing is set to 0.035355.
- The coarseness factor of the regions 1, 2 and 3 is set to 0.35355, 0.70711 and 1, respectively, as shown in Figure 43.
- The mesh generation procedure is done with “Very fine” element distribution, and enhanced mesh refinements with global scale factor of 1.2 and minimum element size factor of 5×10^{-3} .

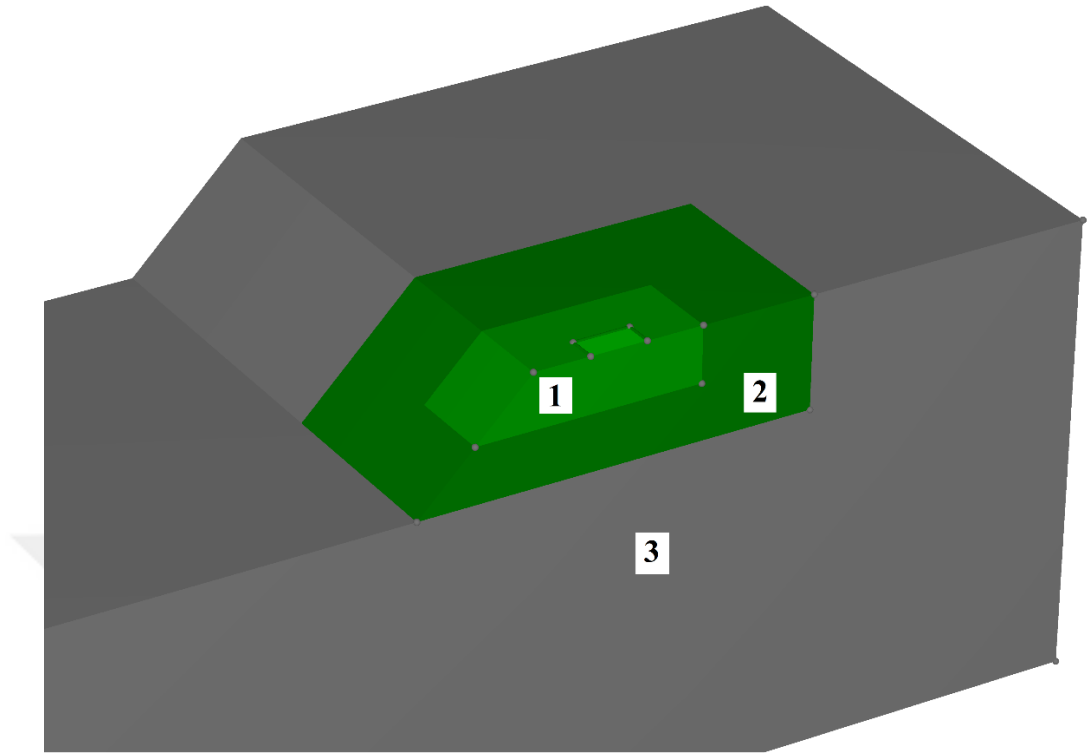


Figure 43: Meshing regions of the model used in this study.

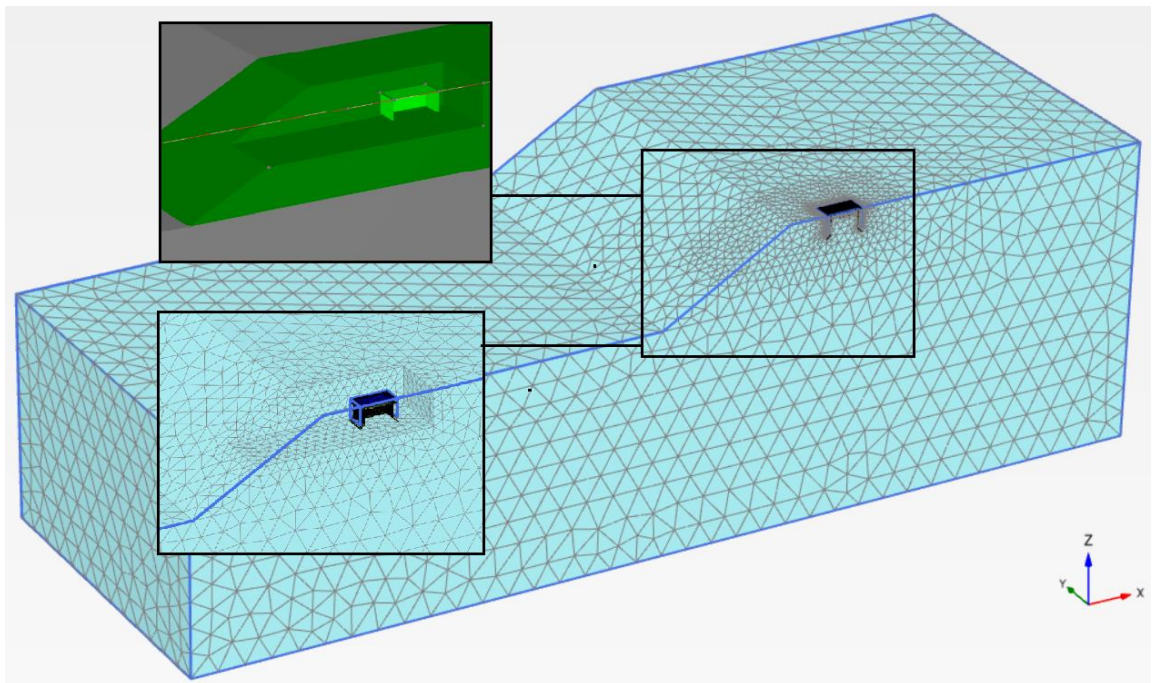


Figure 44: Typical connectivity plot obtained by applying mesh generation for

$\beta=30$, $H/B=2$, $B/L=1$ and $\lambda=1$.

To assess the efficiency of the mesh creation procedure, a specific problem was selected and analysed to investigate how the seismic bearing capacity factor (N_{cs}) varied with the number of triangular elements used in the model. It was found that using a mesh procedure with approximately 46,000 elements resulted in an N_{cs} value of 4.478, while a model with approximately 56,000 elements resulted in an N_{cs} value of 4.476, with an error percentage of %0.04, which is a neglectable value. The previous results demonstrate that approximately the same value of the seismic bearing capacity factor is obtained when the number of elements is more than that generated using the meshing procedure proposed in this study, as presented in Figure 45. These findings suggest that the developed mesh generation technique is acceptable.

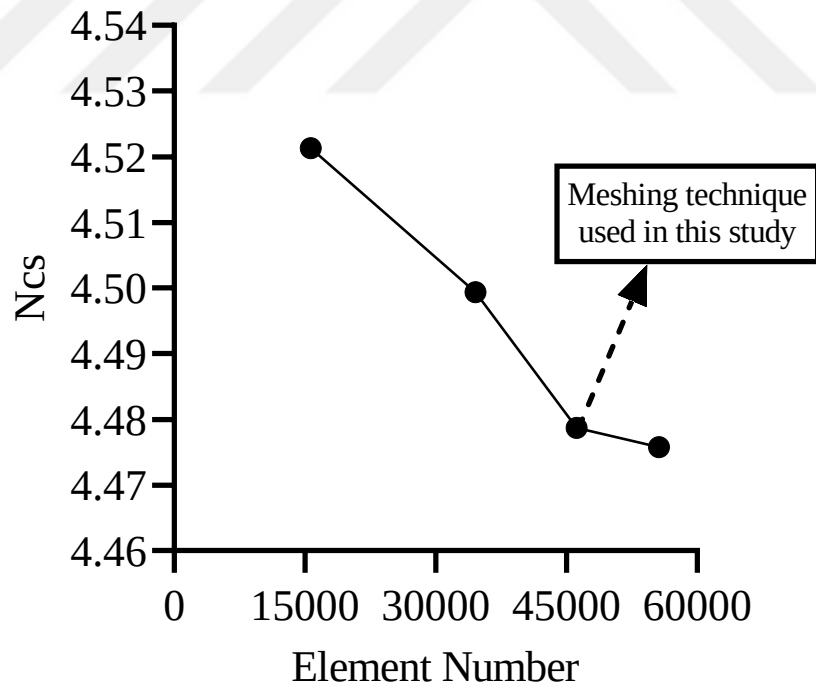


Figure 45: Variation of N_{cs} with the number of triangular elements ($\beta= 30^\circ$, $H/B= 2$, $\lambda= 0$, $c_u= 100\text{kPa}$, $k_h= 0.1$, $B/L=1$).

3.5.8 Initial Stresses

Typically, the initial stresses are generated during the first phase of staged construction in Plaxis 3D using various calculation methods. In situations where there are non-horizontal layers, such as slopes, the "Gravity loading" calculation type is utilized. As such, for the case investigated in this study, the initial stresses in the first stage were generated using the "Gravity loading" option, which was deemed appropriate.

3.6 Construction Stages and Calculation of Seismic Bearing Capacity

Factor N_{cs} Using the Results Obtained by FEM Analysis

The aim of the analysis is to find the seismic bearing capacity and the seismic bearing capacity factor. In this chapter, detailed information about staged construction phases, input values and the settings used in Plaxis 3D software will be explained. As represented in Figure 46 and Table 8, there are four calculation stages employed to find the problem solution, these calculation stages are,

- Initial_phase: to generate the initial stresses by “Gravity loading” option.
- Phase_1: activation of foundation and surface load system.
- Phase_2: activation of pseudo static load acting on soil clusters.
- Phase_3: increasing the value of surface load system to a high value that causes a failure.

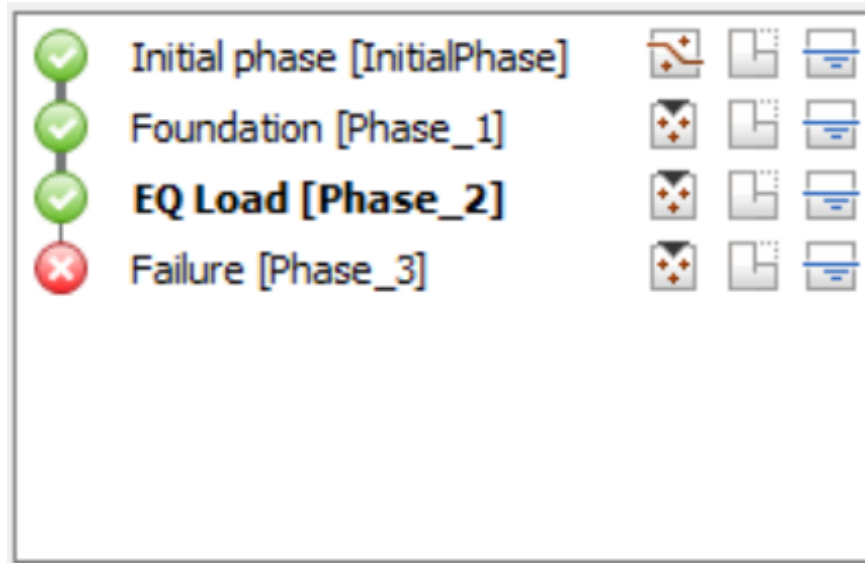


Figure 46: Calculation phases defined to the staged construction window.

Table 8: Settings and loads definition over the phases.

Phase Number	Definition	Settings	Loads																						
0	Initial stresses generation	<table><tr><td>ID</td><td>Initial phase [InitialPhase]</td></tr><tr><td>Calculation type</td><td> Gravity loading</td></tr><tr><td>Loading type</td><td> Staged construction</td></tr><tr><td>ΣM_{weight}</td><td>1.0000</td></tr><tr><td>Pore pressure calculation type</td><td> Phreatic</td></tr><tr><td colspan="2">Deformation control parameters</td></tr><tr><td>Ignore undr. behaviour (A,B)</td><td>☒</td></tr><tr><td>Ignore suction</td><td>☒</td></tr></table>	ID	Initial phase [InitialPhase]	Calculation type	Gravity loading	Loading type	Staged construction	ΣM_{weight}	1.0000	Pore pressure calculation type	Phreatic	Deformation control parameters		Ignore undr. behaviour (A,B)	☒	Ignore suction	☒							
ID	Initial phase [InitialPhase]																								
Calculation type	Gravity loading																								
Loading type	Staged construction																								
ΣM_{weight}	1.0000																								
Pore pressure calculation type	Phreatic																								
Deformation control parameters																									
Ignore undr. behaviour (A,B)	☒																								
Ignore suction	☒																								
1	Activation of foundation and surface load system.	<table><tr><td>ID</td><td>Foundation [Phase_1]</td></tr><tr><td>Start from phase</td><td>Initial phase</td></tr><tr><td>Calculation type</td><td> Plastic</td></tr><tr><td>Loading type</td><td> Staged construction</td></tr><tr><td>ΣM_{stage}</td><td>1.0000</td></tr><tr><td>ΣM_{weight}</td><td>1.0000</td></tr><tr><td>Pore pressure calculation type</td><td> Phreatic</td></tr><tr><td colspan="2">Deformation control parameters</td></tr><tr><td>Ignore undr. behaviour (A,B)</td><td>☐</td></tr><tr><td>Reset displacements to zero</td><td>☒</td></tr><tr><td>Reset small strain</td><td>☒</td></tr></table>	ID	Foundation [Phase_1]	Start from phase	Initial phase	Calculation type	Plastic	Loading type	Staged construction	ΣM_{stage}	1.0000	ΣM_{weight}	1.0000	Pore pressure calculation type	Phreatic	Deformation control parameters		Ignore undr. behaviour (A,B)	☐	Reset displacements to zero	☒	Reset small strain	☒	
ID	Foundation [Phase_1]																								
Start from phase	Initial phase																								
Calculation type	Plastic																								
Loading type	Staged construction																								
ΣM_{stage}	1.0000																								
ΣM_{weight}	1.0000																								
Pore pressure calculation type	Phreatic																								
Deformation control parameters																									
Ignore undr. behaviour (A,B)	☐																								
Reset displacements to zero	☒																								
Reset small strain	☒																								

2

Activation of pseudo static load acting on soil clusters.

ID	EQ Load [Phase_2]
Start from phase	Foundation
Calculation type	Plastic
Loading type	Staged construction
ΣM_{stage}	1.0000
ΣM_{weight}	1.0000
Deformation control parameters	
Ignore undr. behaviour (A,B)	☐
Reset displacements to zero	☐
Reset small strain	☐

Surface loads
SurfaceLoad_1
SurfaceLoad_1_1
Distribution: Uniform
σ_x : -0.10000 kN/m ²
σ_y : 0.0000 kN/m ²
σ_z : -1.0000 kN/m ²
$ \sigma $: 1.0050 kN/m ²
DynSurfaceLoad_1_1
PseudoStatic
Acceleration _x : 0.10000 g
Acceleration _y : 0.0000 g
Acceleration _z : 0.0000 g

3

Increasing the value of surface load system to a high value that causes a failure.

ID	Failure [Phase_3]
Start from phase	EQ Load
Calculation type	Plastic
Loading type	Staged construction
ΣM_{stage}	1.0000
ΣM_{weight}	1.0000
Deformation control parameters	
Ignore undr. behaviour (A,B)	☐
Reset displacements to zero	☐
Reset small strain	☐

Surface loads
SurfaceLoad_1
SurfaceLoad_1_1
Distribution: Uniform
σ_x : -1000.0 kN/m ²
σ_y : 0.0000 kN/m ²
σ_z : -10000 kN/m ²
$ \sigma $: 10050 kN/m ²
DynSurfaceLoad_1_1
PseudoStatic
Acceleration _x : 0.10000 g
Acceleration _y : 0.0000 g
Acceleration _z : 0.0000 g

In addition to the parameters defined in Table 8, the solver type of the numerical problem was set to "Paradiso (multicore direct)" as it is a direct solver that utilizes multi-core processors to solve the system of equations in parallel. This solver type is widely recognized for its robustness in effectively solving equations. However, it is important to mention that the Paradiso solver tends to have higher memory consumption compared to other methods (Schank and Gartner (2006)). Picos (multicore iterative) solver is not preferred due to its iterative solving approach. While the iterative solver can reduce calculation time, it may also compromise the solution accuracy. Hence, for the purposes of this study, Paradiso (multicore direct) is chosen that prioritizes solution accuracy over computational speed. To prevent any solution errors related to the step number, the Max steps number kwass was increased to 10,000 after some analyses were noted to reach the last step without reaching a solution.

Upon completion of the analyses, it was observed that the model may fail in the initial, second, and third phases. The first phase failure denotes the instability of the slope under gravity loads, whereas in the second phase, pseudo-static acceleration is applied under which failure is attributed to the seismic instability of the slope in the absence of foundation loading. In the third phase, foundation loading that includes inertia component is gradually increased until failure. Therefore, the last phase in essence involves bearing capacity failure. Further details regarding the failure types and mechanisms will be provided in the following chapters.

The seismic bearing capacity factor N_{cs} is determined using the ΣM_{stage} parameter, which is a calculation result parameter in Plaxis 3D. This parameter signifies the percentage value of the applied load that caused a failure in the model. To examine the determination of the bearing capacity factor N_{cs} , an example model with values of $\beta = 15^\circ$, $H/B = 2$, $\lambda = 2$, $c_u = 100$ kPa, $k_h = 0.1$, and $B/L = 0.5$ was used, as illustrated in Figure 47.

Reached values	
Reached total time	0.0000 day
CSP - Relative stiffness	6.3187E-6
ForceX - Reached total force X	0.0000 kN
ForceY - Reached total force Y	0.0000 kN
ForceZ - Reached total force Z	0.0000 kN
Pmax - Reached max pp	0.0000 kN/m ²
ΣM_{stage} - Reached phase proportion	0.051698
ΣM_{weight} - Reached weight proportion	1.0000
ΣM_{sf} - Reached safety factor	1.0000

Figure 47: Reached maximum ΣM_{stage} value obtained by Plaxis 3D analysis.

The ultimate seismic bearing capacity (q_u) of the system is calculated as below,

$$q_u = \text{The magnitude of applied load} \times \Sigma M_{stage}$$

$$q_u = 10,000 \times 0.051698 = 516.98 \text{ kN/m}^2$$

Using Equation (30), one can calculate the seismic bearing capacity factor N_{cs} as below,

$$N_{cs} = \frac{q_u}{c_u} = \frac{516.98}{100} = 5.17$$

3.7 Comparison of the Results with Previous Studies

Due to the absence of studies available in the literature that investigate the effect of footing shape on the seismic bearing capacity of foundations located near slopes, this comparison will only be conducted for the static case. The static case comparison will be divided into two parts: the first part will focus on the effect of footing shape on the bearing capacity of foundations situated on level ground, while the second part will compare the results of previous studies that investigated the influence of footing shape on the bearing capacity of foundations located near slopes with the findings of this study. Additionally, this chapter will also include a comparison of the overall slope stability under static and seismic conditions used in this study with the previous research conducted in the same field by other researchers.

3.7.1 Comparison of Level Ground Case

There have been numerous studies in the literature investigating the impact of footing shape on bearing capacity, particularly in cases involving level ground. Various methods have been utilized, such as empirical approaches (Skempton [5]), numerical finite element analysis (Michalowski [10] and Gouvernec [12]), and finite element limit analysis (Salgado et al. [21] and Yang et al. [13]).

As presented in Figure 48, the variation of $s_c N_c$ values, that indicates the bearing capacity factor multiplied by the shape factor which represents the N_{cs} value calculated in this study, with B/L values is compared to the results obtained by other researchers. It can be observed that the results obtained by this study compared with those obtained in previous studies are consistent. FE results obtained by Gouvernec [12] for the level ground case are almost identical to those obtained by this study, except for the $B/L=0$ case. The reason for this difference is that this study used 2D plain-strain conditions to represent the strip foundation case, which provides more realistic results than those found by Gouvernec [12] using a 3D model.

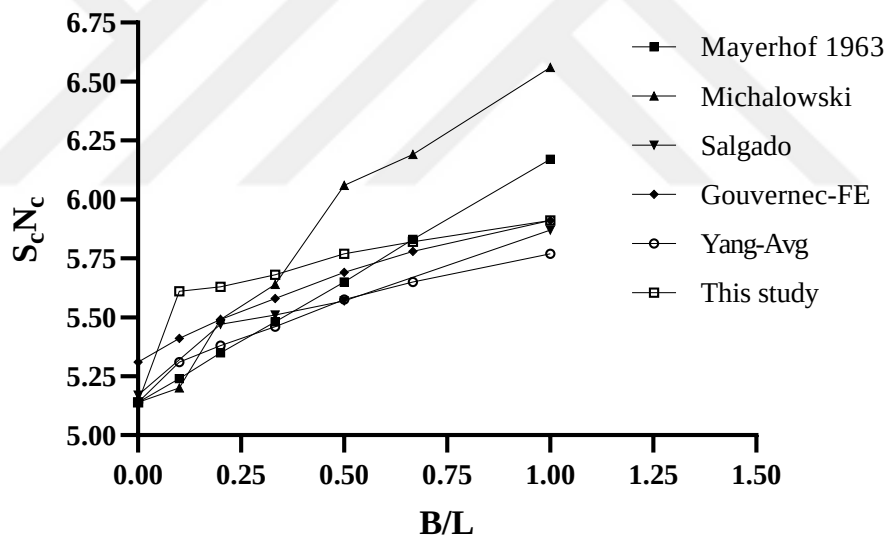


Figure 48: Variation of $S_c N_c$ with B/L of this study compared to previous studies.

3.7.2 Comparison of the Sloping Ground Solutions under Static Conditions

At the start of this chapter, it was noted that no previous studies had examined the impact of footing shape on bearing capacity under seismic conditions, although Yang *et*

al. [13] had studied this factor under static conditions for foundations located near slopes. In this study, the influence of footing shape on bearing capacity under seismic conditions was examined and compared with Yang *et al.*'s [13] findings, as illustrated in Figure 49. The case ($\lambda = 1$, $c_u = 100 \text{ kN/m}^2$, $\beta = 30^\circ$ and $k_h = 0$) is considered to be compared with this study's findings. To compare the results, the bearing capacity factor N_{cs} for rectangular footings, obtained through 3D analysis in this study, was divided by the N_{cs} value obtained for a strip foundation case with the same width using 2D analysis, resulting in the shape factor parameter s_c . The results of this study and Yang *et al.*'s study are remarkably consistent for this case. It was found that as the B/L value increases, the shape factor s_c increases until reaching a critical point where the shape factor s_c begins to decline. This can be explained by the fact that as the B/L value increases, the foundation reaches the square-shaped footing when B/L is unity, which has the highest bearing capacity value. However, with increasing B/L value over than a unity, the foundation is more likely to reach the strip foundation shape, resulting in a decrease in the bearing capacity value.

It is important to note that in the study conducted by Yang *et al.* [13], the B value was kept constant while the L value was decreased to achieve B/L values ranging from zero to one. On the other hand, when aiming for B/L values greater than one, the L value was kept constant while the B value was increased. Consequently, for B/L values less than one, the 2D analysis employed strip foundations with a constant width (B value), whereas for B/L values greater than one, strip foundations with larger width (B value) were utilized. In their analysis, Yang *et al.* [13] observed a significant decrease in the bearing capacity values of strip foundations, especially when $H/B = 4$, when the B value was increased in order to obtain B/L values greater than unity. This decrease

explains the increase in the shape factor s_c reported by Yang et al. [13] when B/L exceeds one, as depicted in Figure 49. However, it is worth mentioning that in this study, a constant B value was employed for all B/L values to maintain a constant bearing capacity for the strip foundations. The s_c values for cases with B/L greater than one, as found by Yang et al. [13], were recalculated using the bearing capacity value obtained from strip foundations with a constant B value when B/L is less than one. Subsequently, a new graph was generated to compare these findings with the results of this study, as presented in Figure 50. Both studies exhibit a high level of consistency, demonstrating that the s_c value increases as the B/L ratio rises until it reaches its peak at $B/L = 1$. However, for B/L values greater than one, the shape factor s_c starts to decline.

According to the findings of Yang et al. [13], the influence of H/B (the ratio of foundation height to foundation width) on the bearing capacity is generally insignificant for undrained cases ($\phi = 0$), except for the specific scenario ($B/L = 0$, $\beta = 30^\circ$, $\lambda = 1$, $c_u = 100 \text{ kN/m}^2$, $k_h = 0$) involving a strip foundation. It was observed that the bearing capacity of the strip foundation in the compared case is higher when $H/B = 0.5$ compared to when $H/B = 4$. This behaviour can be attributed to the transition from face failure (where the failure line passes through the slope face) to toe failure (where the failure line extends to the slope toe) when moving from a rectangular foundation to a strip foundation with $H/B = 4$. This transition explains the decrease in the bearing capacity of the strip foundation. This explains the observed trend of increasing H/B values leading to higher s_c values, which is consistent with the findings of this study. It is important to note that neither Yang et al. [13] nor this study observed this specific behaviour for the case involving ($\lambda = 0$, $c_u = 100 \text{ kN/m}^2$, $\beta = 15^\circ$, $k_h = 0$), where the

failure mechanism always involves face failure for all combinations of H/B and B/L values.

Yang et al. [13] did not specifically focus on investigating the influence of the normalized distance parameter (λ) extensively. They only examined two cases: one with $\lambda = 0$ and $\beta = 15^\circ$, and another with $\lambda = 1$ and $\beta = 30^\circ$. Comparing the influence of λ becomes challenging since the slope angle (β) differs between these two cases. However, it can be observed that the s_c values obtained when $\lambda = 1$ are greater than those obtained when $\lambda = 0$. It is important to note that it is insufficient to conclude that decreasing the λ values will directly lead to a decrease in the bearing capacity, primarily due to the difference in the slope angle (β) values between the cases studied.

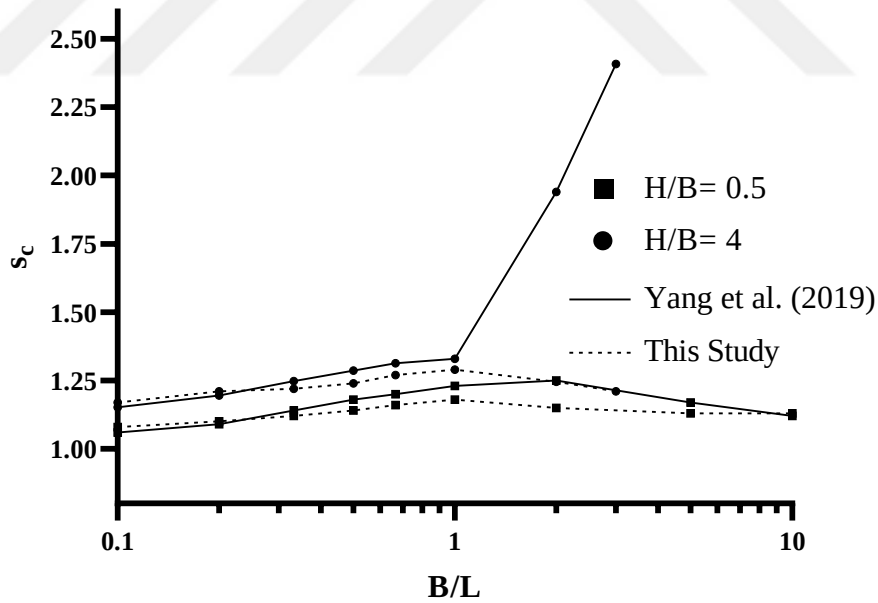


Figure 49: Variation of s_c with B/L of this study and Yang et. al. (2019) ($\lambda = 1$, $c_u = 100 \text{ kN/m}^2$, $\beta = 30^\circ$ and $k_h = 0$) (B increases when $B/L > 1$).

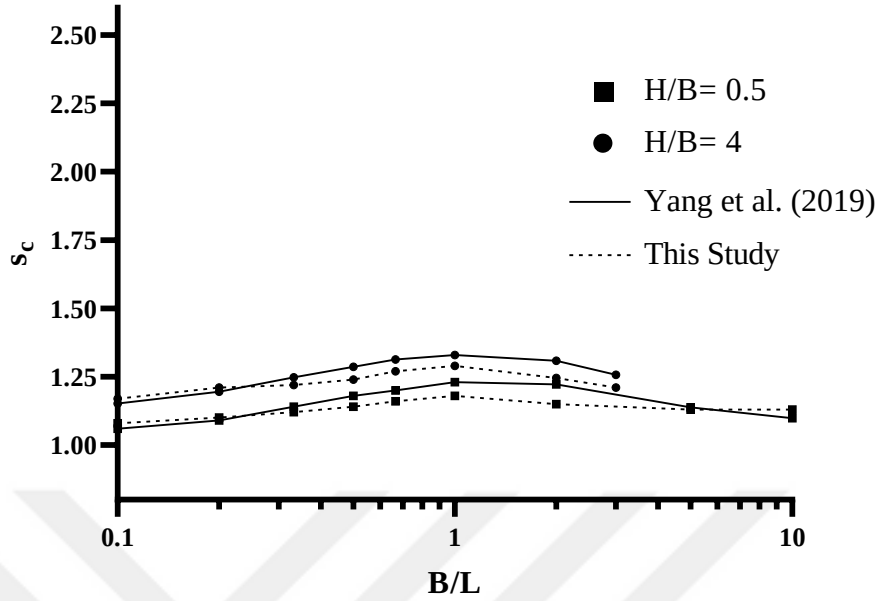


Figure 50: Variation of s_c with B/L of this study and Yang et. al. (2019) ($\lambda = 1$, $c_u = 100 \text{ kN/m}^2$, $\beta = 30^\circ$ and $k_h = 0$) (B is constant).

3.7.3 Cohesive Slope Stability Assessment Under Both Static and Seismic Conditions

During the analyses, it was observed that the slope can fail before it reaches the last calculation phase, which corresponds to a slope instability problem under its own gravity load or under the seismic load applied to the soil. As mentioned before in Chapter 3.6, the numerical model can fail at the first, third or fourth phases, as the first two represents slope instability problems under slope's own gravity loads or due to the applied seismic load, respectively. In this chapter, the results of the stability of the slopes used in this study, that obtained by Plaxis 3D, will be compared to previous studies that investigated the slope stability under static and seismic conditions. The purpose of this comparison is to assess the consistency between the different studies.

Taylor [19] proposed a method for determining the maximum height (H_{cr}) that a cohesive slope can withstand without experiencing any loss of stability, which depends on the slope's geometry and soil strength. Table 9 provides information on the geometric and strength properties of unstable cohesive slopes under the influence of self-weight, which were calculated using the stability charts provided in Chapter 2.3.

Table 9: H_{cr} value and stability check using Taylor's method.

H (m)	D	β (°)	m	γ (kN/m³)	c_u (kPa)	H_{cr} (m)	Stability Check
2	7	15	0.182	20	200	54.9	Stable
2	7	30	0.182	20	200	54.9	Stable
2	7	45	0.182	20	200	54.9	Stable
2	7	60	0.19157	20	200	52.20	Stable
2	7	15	0.182	20	100	27.5	Stable
2	7	30	0.182	20	100	27.5	Stable
2	7	45	0.182	20	100	27.5	Stable
2	7	60	0.19157	20	100	26.10	Stable
2	7	15	0.182	20	50	13.7	Stable
2	7	30	0.182	20	50	13.7	Stable
2	7	45	0.182	20	50	13.7	Stable
2	7	60	0.19157	20	50	13.05	Stable
2	7	15	0.182	20	25	6.9	Stable
2	7	30	0.182	20	25	6.9	Stable
2	7	45	0.182	20	25	6.9	Stable
2	7	60	0.19157	20	25	6.53	Stable
4	4	15	0.176	20	200	56.8	Stable
4	4	30	0.18	20	200	55.6	Stable
4	4	45	0.18	20	200	55.6	Stable
4	4	60	0.19157	20	200	52.20	Stable
4	4	15	0.176	20	100	28.4	Stable
4	4	30	0.18	20	100	27.8	Stable
4	4	45	0.18	20	100	27.8	Stable
4	4	60	0.19157	20	100	26.10	Stable
4	4	15	0.176	20	50	14.2	Stable
4	4	30	0.18	20	50	13.9	Stable
4	4	45	0.18	20	50	13.9	Stable

4	4	60	0.19157	20	50	13.05	Stable
4	4	15	0.176	20	25	7.1	Stable
4	4	30	0.18	20	25	6.9	Stable
4	4	45	0.18	20	25	6.9	Stable
4	4	60	0.19157	20	25	6.53	Stable
8	2.5	15	0.154	20	200	64.9	Stable
8	2.5	30	0.175	20	200	57.1	Stable
8	2.5	45	0.18	20	200	55.6	Stable
8	2.5	60	0.19157	20	200	52.20	Stable
8	2.5	15	0.154	20	100	32.5	Stable
8	2.5	30	0.175	20	100	28.6	Stable
8	2.5	45	0.18	20	100	27.8	Stable
8	2.5	60	0.19157	20	100	26.10	Stable
8	2.5	15	0.154	20	50	16.2	Stable
8	2.5	30	0.175	20	50	14.3	Stable
8	2.5	45	0.18	20	50	13.9	Stable
8	2.5	60	0.19157	20	50	13.05	Stable
8	2.5	15	0.154	20	25	8.1	Unstable
8	2.5	30	0.175	20	25	7.1	Unstable
8	2.5	45	0.18	20	25	6.9	Unstable
8	2.5	60	0.19157	20	25	6.53	Unstable

It can be observed that cohesive slopes are unstable for all slope angles considered in this study when the values of c_u and H are 25 kPa and 8.0 m, respectively. Although H_{cr} was calculated as 8.1 m for a specific set of parameters ($\beta = 15^\circ$, $c_u = 25$ kPa and $H = 8.0$ m), the difference between the height values (H and H_{cr}) is insignificant. Furthermore, the corresponding numerical model (depicted in Figure 51) indicated base failure. The results obtained from the numerical model in static conditions are in full agreement with the previous findings of Taylor [19]. The failure mechanisms obtained by Plaxis 3D for the four sets of parameters defined as "Unstable" in Table 9 are shown in Figures 51, 52, 53 and 54.

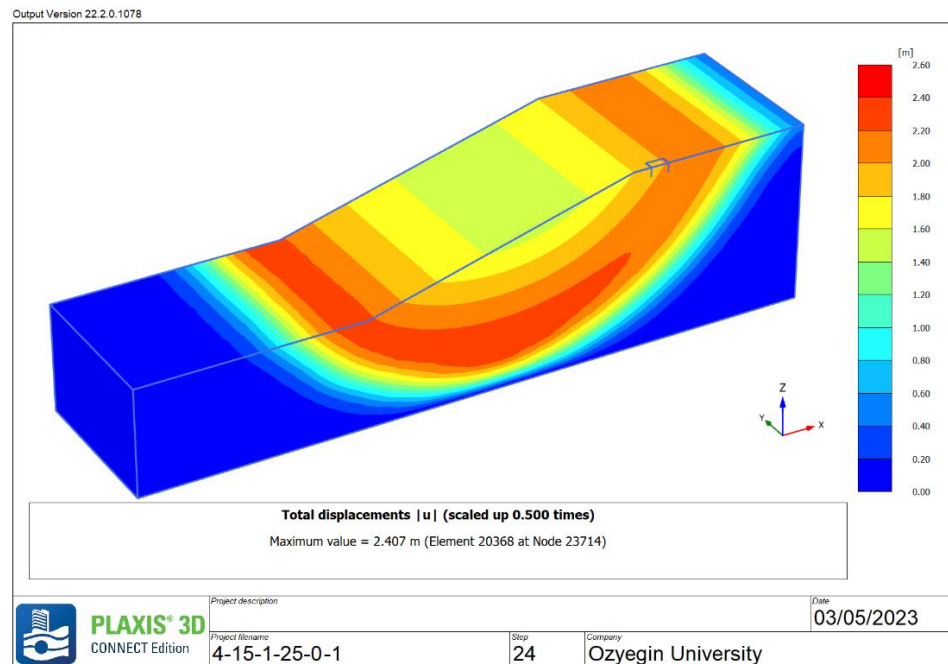


Figure 51: Failure mechanism of unstable cohesive slope in static condition ($\beta=15^\circ$, $H=8.0$ m and $c_u=25$ kPa).

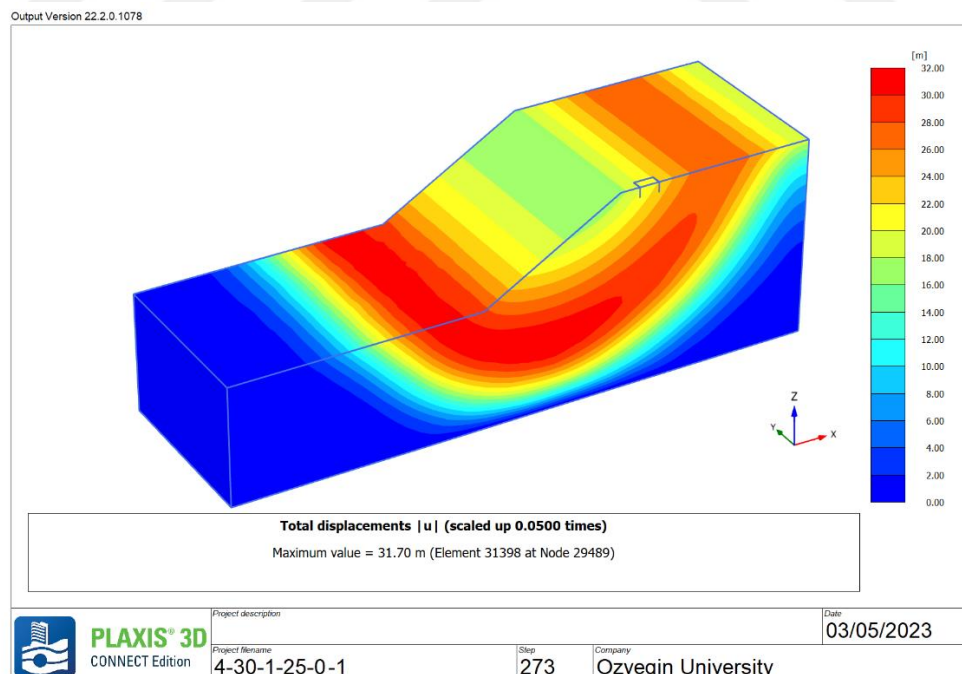


Figure 52: Failure mechanism of unstable cohesive slope in static condition ($\beta=30^\circ$, $H=8.0$ m and $c_u=25$ kPa).

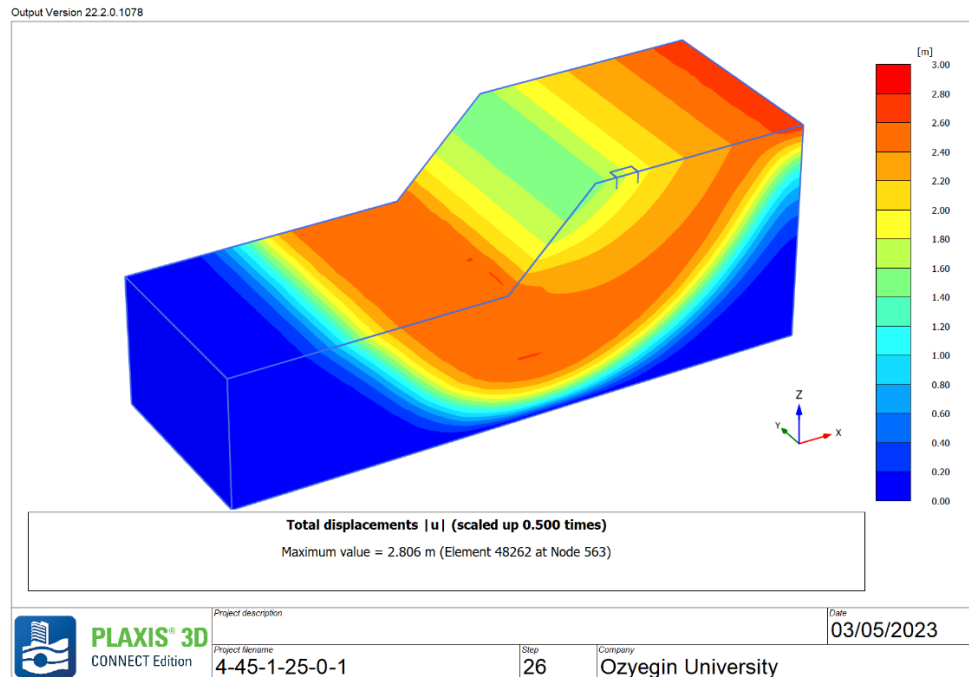


Figure 53: Failure mechanism of unstable cohesive slope in static condition ($\beta=45^\circ$, $H=8.0$ m and $c_u=25$ kPa).

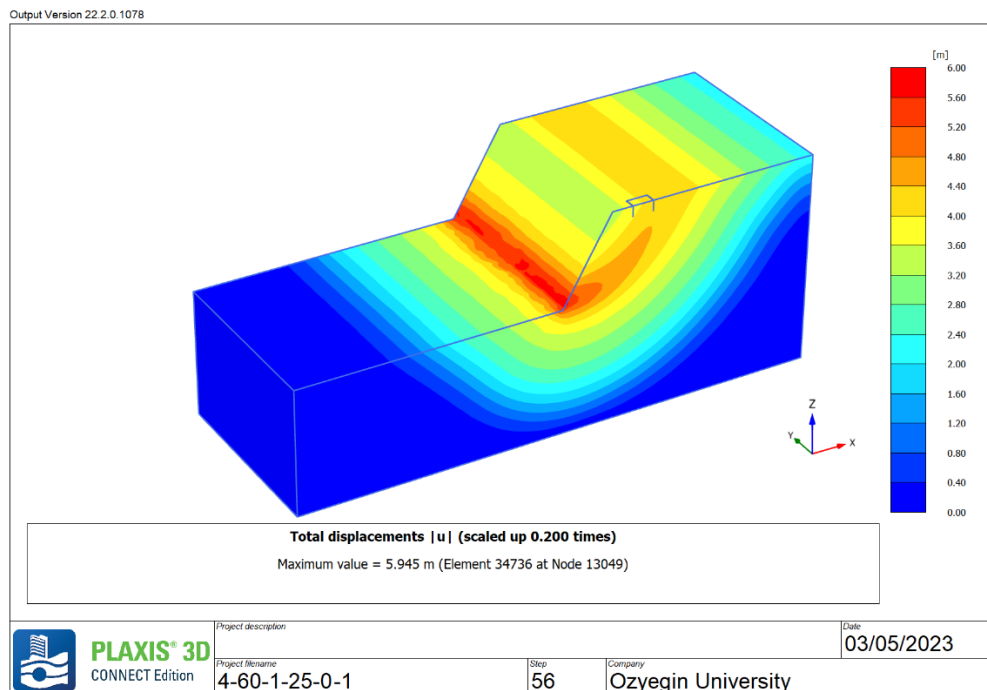


Figure 54: Failure mechanism of unstable cohesive slope in static condition ($\beta=60^\circ$, $H=8.0$ m and $c_u=25$ kPa).

Koppula's method [20] was employed to calculate the critical height H_{cr} that a cohesive slope can sustain without failing under seismic conditions. H_{cr} varies based on the slope's geometry, soil strength, and seismic load, up to a slope angle of $\beta = 53^\circ$, beyond which the critical height remains independent of the slope geometry. Table 10 displays the critical values calculated for $H = 8$ m, which corresponds to the most critical case. The calculation for the other heights is provided in Appendix-I.

Table 10: H_{cr} value and stability check using Koppula's method.

H (m)	D	β (°)	k_h	N2	γ (kN/m ³)	c_u (kPa)	H_{cr} (m)	Stability Check
8	2.5	30	0.1	4.12	20	200	41.2	Stable
8	2.5	30	0.2	3.16	20	200	31.6	Stable
8	2.5	30	0.3	2.59	20	200	25.9	Stable
8	2.5	45	0.1	3.58	20	200	35.8	Stable
8	2.5	45	0.2	2.51	20	200	25.1	Stable
8	2.5	45	0.3	1.97	20	200	19.7	Stable
8	2.5	60	0.1	4.583	20	200	45.8	Stable
8	2.5	60	0.2	3.975	20	200	39.8	Stable
8	2.5	60	0.3	3.278	20	200	32.8	Stable
8	2.5	30	0.1	4.12	20	100	20.6	Stable
8	2.5	30	0.2	3.16	20	100	15.8	Stable
8	2.5	30	0.3	2.59	20	100	13.0	Stable
8	2.5	45	0.1	3.58	20	100	17.9	Stable
8	2.5	45	0.2	2.51	20	100	12.6	Stable
8	2.5	45	0.3	1.97	20	100	9.9	Stable
8	2.5	60	0.1	4.583	20	100	22.9	Stable
8	2.5	60	0.2	3.975	20	100	19.9	Stable
8	2.5	60	0.3	3.278	20	100	16.4	Stable
8	2.5	30	0.1	4.12	20	50	10.3	Stable
8	2.5	30	0.2	3.16	20	50	7.9	Unstable
8	2.5	30	0.3	2.59	20	50	6.5	Unstable
8	2.5	45	0.1	3.58	20	50	9.0	Stable
8	2.5	45	0.2	2.51	20	50	6.3	Unstable
8	2.5	45	0.3	1.97	20	50	4.9	Unstable
8	2.5	60	0.1	4.583	20	50	11.5	Stable
8	2.5	60	0.2	3.975	20	50	9.9	Unstable
8	2.5	60	0.3	3.278	20	50	8.2	Unstable

8	2.5	30	0.1	4.12	20	25	5.2	Unstable
8	2.5	30	0.2	3.16	20	25	4.0	Unstable
8	2.5	30	0.3	2.59	20	25	3.2	Unstable
8	2.5	45	0.1	3.58	20	25	4.5	Unstable
8	2.5	45	0.2	2.51	20	25	3.1	Unstable
8	2.5	45	0.3	1.97	20	25	2.5	Unstable
8	2.5	60	0.1	4.583	20	25	5.7	Unstable
8	2.5	60	0.2	3.975	20	25	5.0	Unstable
8	2.5	60	0.3	3.278	20	25	4.1	Unstable

It can be observed that cohesive slopes are unstable for all slope angles considered in this study when the values of c_u and H are 25 kPa and 8.0 m, respectively. However, when H and c_u values are 8.0 m and 50 kPa, respectively, only the slopes imposed to a seismic acceleration value of 0.1 remain stable, more than this seismic acceleration value will cause a failure. Although H_{cr} values were calculated as 8.1 m and 9.9 m for specific set of parameters ($\beta = 60^\circ$, $c_u = 50$ kPa, $H = 8.0$ m and $k_h = 0.3$) and ($\beta = 60^\circ$, $c_u = 50$ kPa, $H = 8.0$ m and $k_h = 0.2$), respectively, the difference between the height values (H and H_{cr}) is insignificant. The results obtained from the numerical model in seismic conditions have a great agreement with the previous findings of Koppula [20]. The failure mechanisms obtained by Plaxis 3D for some sets of parameters defined as "Unstable" in Table 10 are shown in Figures 55 to 60.

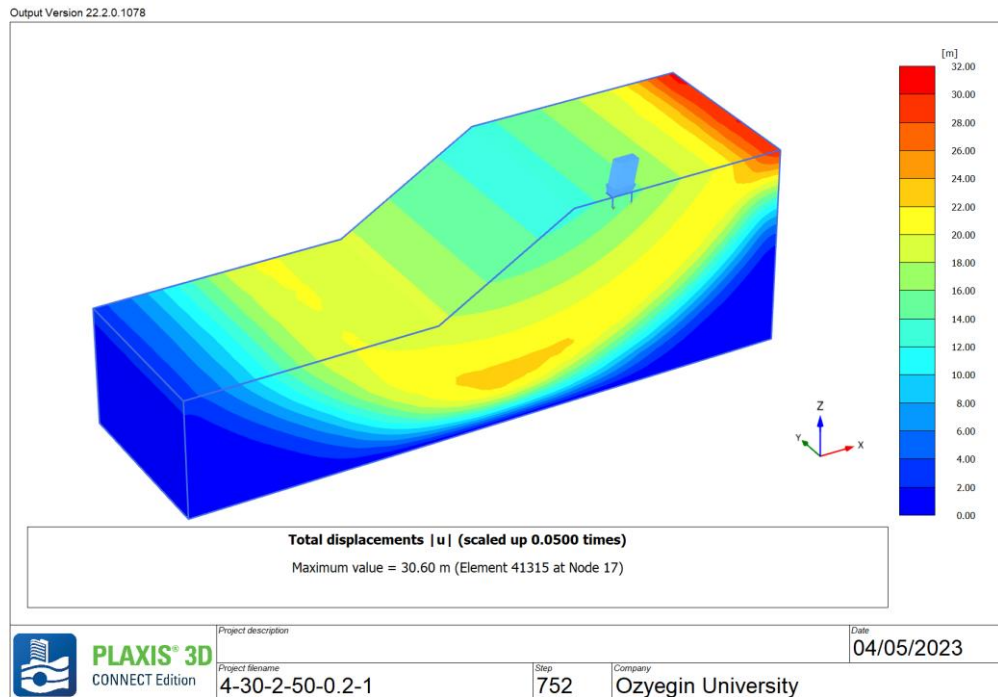


Figure 55: Failure mechanism of unstable cohesive slope under seismic condition

($\beta = 30^\circ$, $H = 8.0$ m, $c_u = 50$ kPa, $k_h = 0.2$).

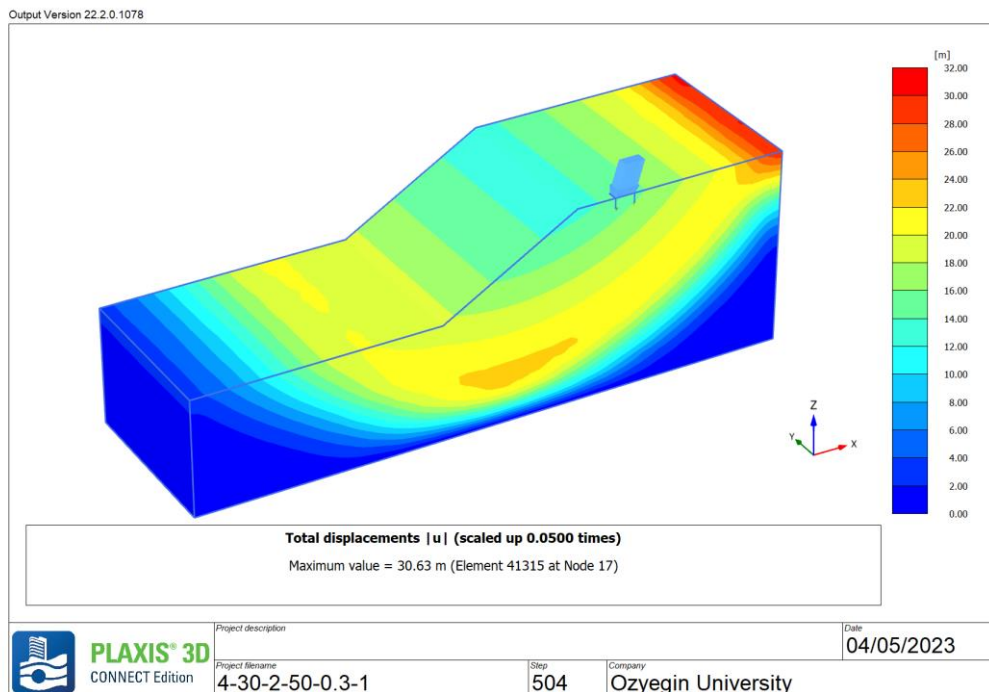


Figure 56: Failure mechanism of unstable cohesive slope under seismic condition

($\beta = 30^\circ$, $H = 8.0$ m, $c_u = 50$ kPa, $k_h = 0.3$).

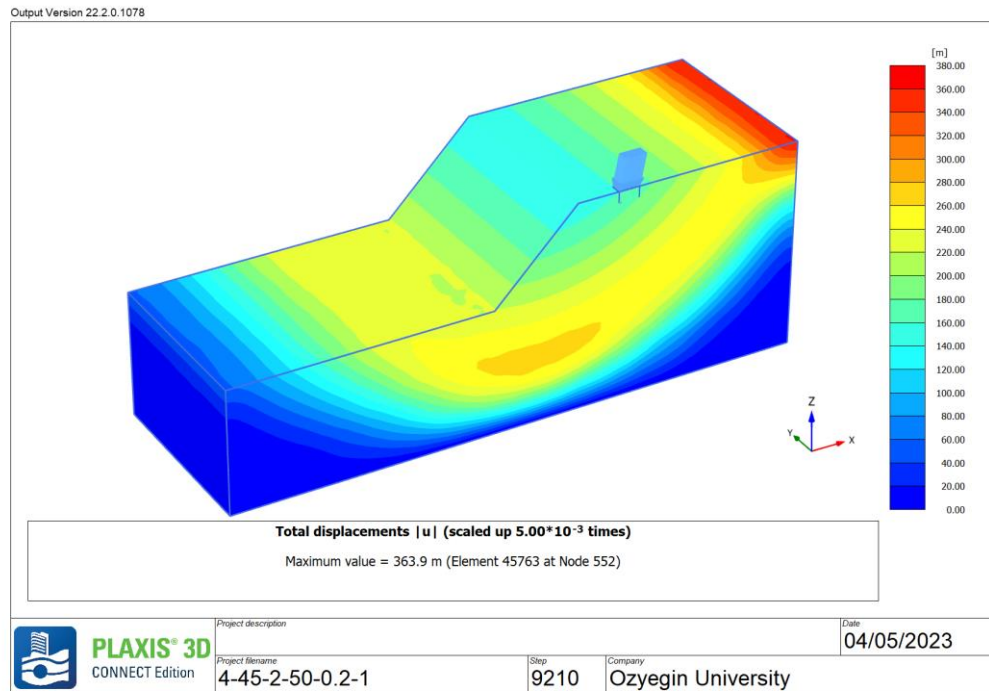


Figure 57: Failure mechanism of unstable cohesive slope under seismic condition

($\beta = 45^\circ$, $H = 8.0$ m, $c_u = 50$ kPa, $k_h = 0.2$).

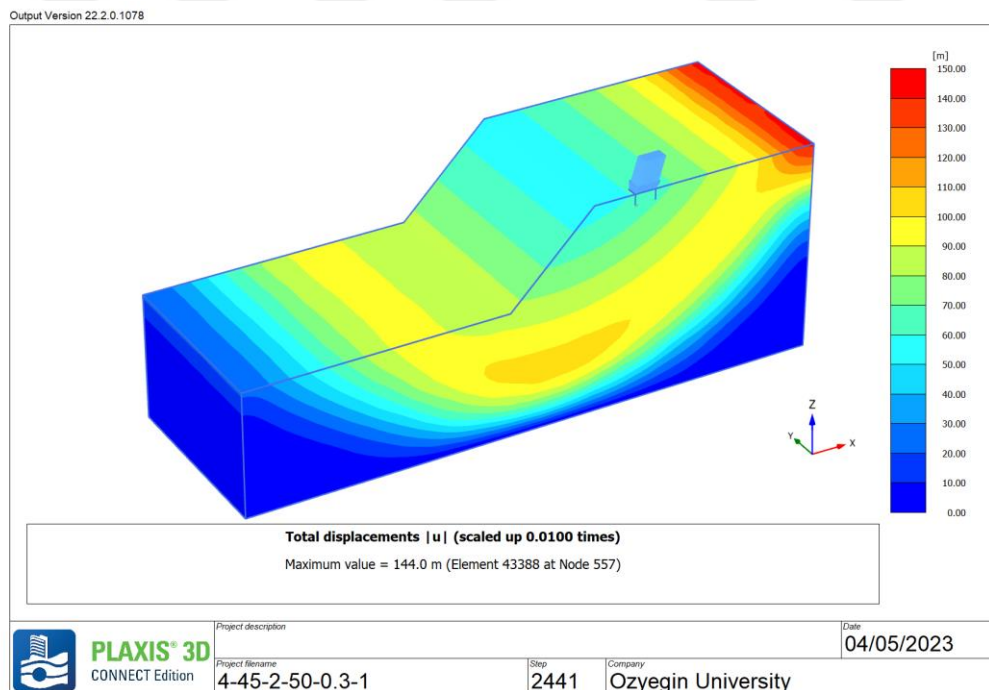


Figure 58: Failure mechanism of unstable cohesive slope under seismic condition

($\beta = 45^\circ$, $H = 8.0$ m, $c_u = 50$ kPa, $k_h = 0.3$).

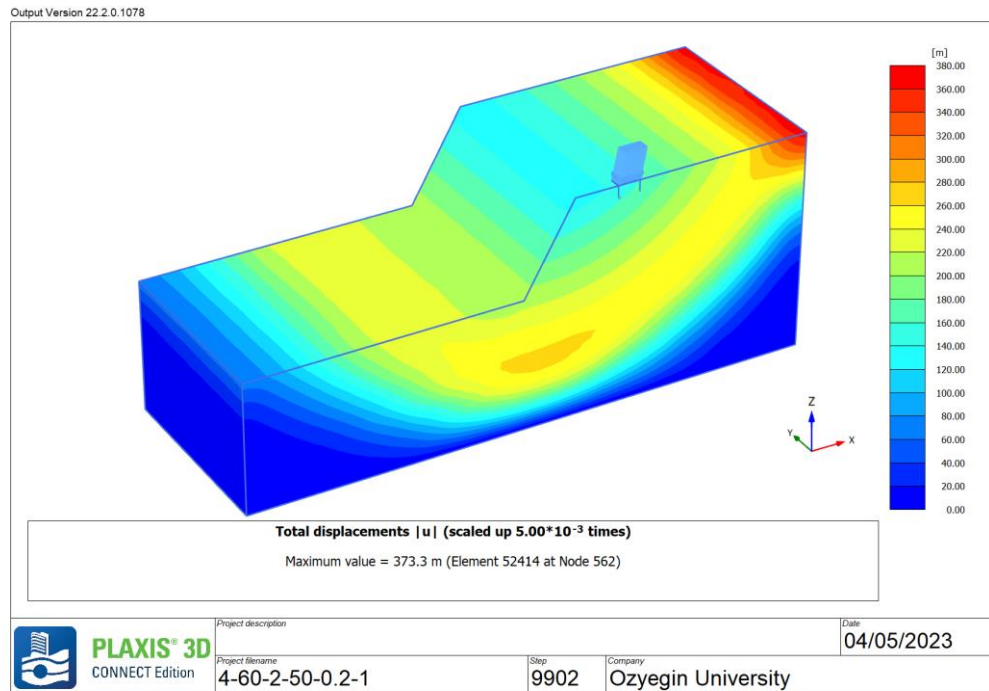


Figure 59: Failure mechanism of unstable cohesive slope under seismic condition

($\beta = 60^\circ$, $H = 8.0$ m, $c_u = 50$ kPa, $k_h = 0.2$).

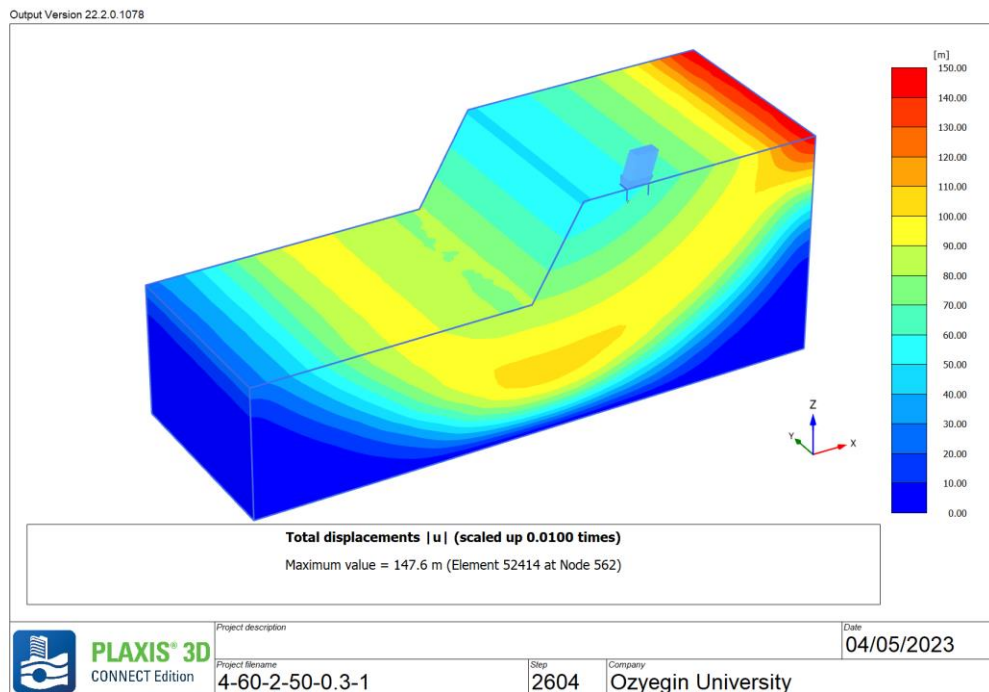


Figure 60: Failure mechanism of unstable cohesive slope under seismic condition

($\beta = 60^\circ$, $H = 8.0$ m, $c_u = 50$ kPa, $k_h = 0.3$).

CHAPTER IV

RESULTS AND DISCUSSIONS

4.1 Influential Parameters

This chapter will focus on determination of the effect of each parameter, used during the analyses, on the seismic bearing capacity in term of footing shape which is represented by the parameter B/L . Each parameter will be investigated separately in the following part.

4.1.1 Effect of Footing Shape B/L

In Figure 61, it is evident that the seismic bearing capacity factor N_{cs} increases as the footing aspect ratio B/L increases. When B is held constant, an increase in B/L implies a reduction in the area affected by the applied load. Equation (31) demonstrates that decreasing the area (A) under a constant force (F) will lead to a decrease in the applicable load q_u (bearing capacity). However, it is also observed that the N_{cs} value decreases when B/L exceeds unity in static conditions ($k_h=0$). This behaviour can be attributed to the fact that as B/L increases, the failure mechanism tends to occur parallel to the slope, thereby increasing the bearing capacity. However, when B/L exceeds unity, the footing becomes slope-orthogonal and approaches the case of a strip foundation, resulting in a decrease in the bearing capacity. This behaviour is not evident when $\lambda = 0$ (as shown in Figure 62), where the failure mechanism always encompasses the slope.

Interestingly, at $k_h = 0.3$, the influence of footing aspect ratio B/L on N_{cs} is negligible, as the bearing factor N_{cs} reaches a specific value ($N_{cs} = 3.33$) and remains constant across various B/L values, particularly when λ is greater than zero. This

behaviour can be attributed to the sliding failure mechanism, as explained by Richards et al. [24]. In this mechanism, a foundation slides when the horizontal component of the bearing pressure at failure equals the undrained strength of the soil c_u . Consequently, when N_{cs} multiplied by k_h equals unity, sliding failure occurs, resulting in a sliding N_{cs} value of 3.33 for $k_h = 0.3$. When $\lambda = 0$, the failure mechanism affects the entire slope in the model, which explains why sliding failure does not occur when the foundation is positioned on the crest. However, when $\lambda = 0.5$, the failure mechanism occurs at a shallow depth without impacting the slope.

$$q_u = \frac{F}{A} \quad (31)$$

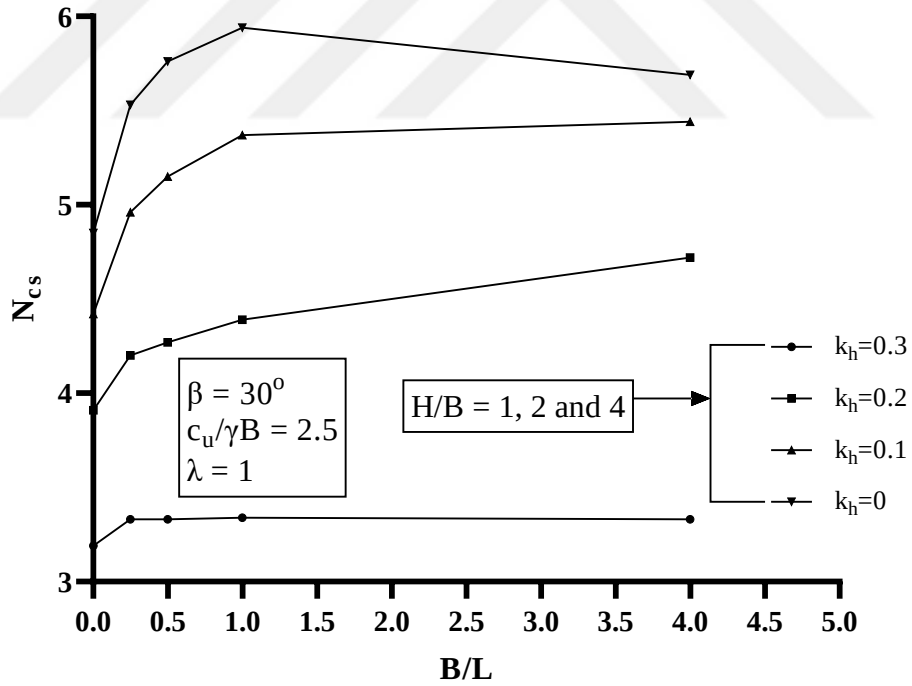


Figure 61: The effect of B/L on the seismic bearing capacity factor N_{cs} ($\beta = 30^\circ$, $c_u/\gamma B = 2.5$, $\lambda = 1$).

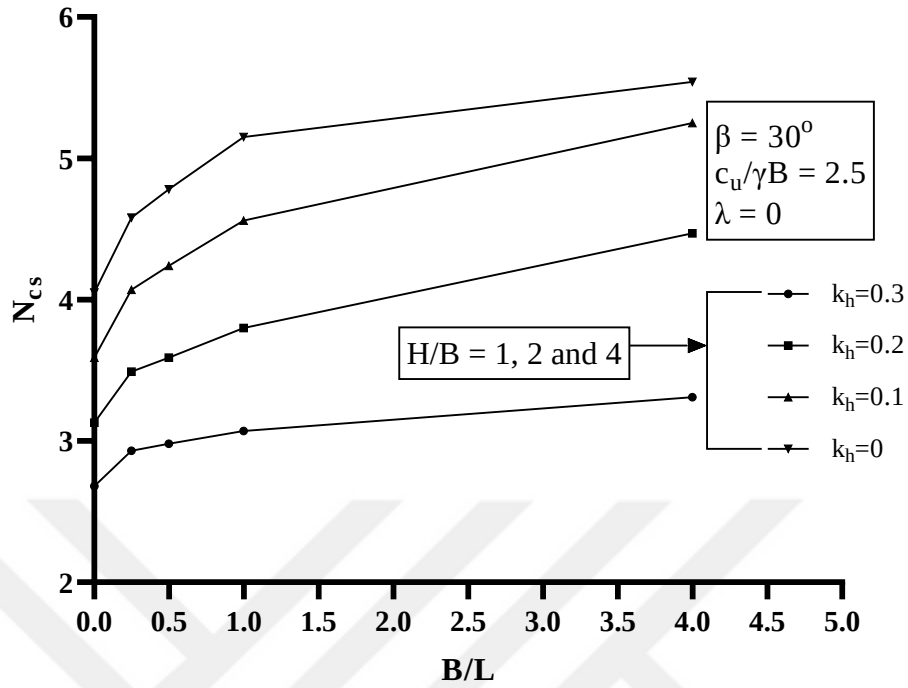


Figure 62: The effect of B/L on the seismic bearing capacity factor N_{cs} ($\beta = 30^\circ$, $c_u/\gamma B = 2.5$, $\lambda = 0$).

4.1.2 Effect of Horizontal Seismic Acceleration Coefficient k_h

Overall, a trend can be observed where the seismic bearing capacity factor N_{cs} generally decreases as the horizontal seismic acceleration coefficient k_h increases for all the footing shapes employed in this investigation, as shown in Figure 63. However, it is important to note that when the aspect ratio (B/L) is equal to 4, the curve initiates from a lower N_{cs} value. This phenomenon can be attributed to the explanation provided in Chapter 4.1.1.

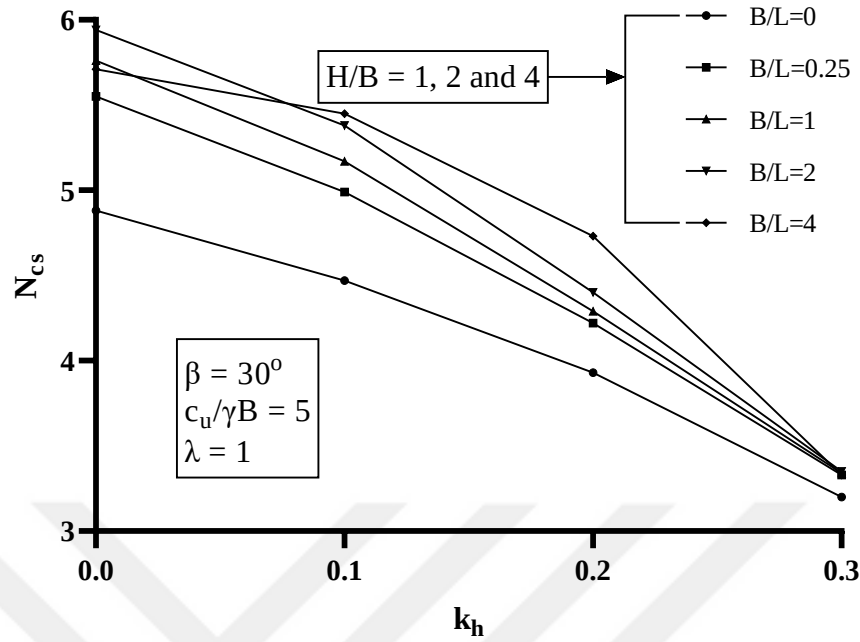


Figure 63: The effect of k_h on the seismic bearing capacity factor N_{cs} for all the footing shapes ($\beta = 30^\circ$, $c_u/\gamma B = 5$, $\lambda = 1$).

4.1.3 Effect of Soil Strength $c_u/\gamma B$

The impact of soil strength, as indicated by the normalized parameter $c_u/\gamma B$, is shown in Figures 64 and 65. It is clear that $c_u/\gamma B$ has a minimal effect on the seismic bearing capacity factor N_{cs} , particularly when $c_u/\gamma B$ exceeds 1.25. The introduction of a higher horizontal acceleration coefficient k_h leads to failure in models with low soil strength values, which explains the absence of some data points on the graphs. This trend appears almost similar for all aspect ratios (B/L) considered in this study.

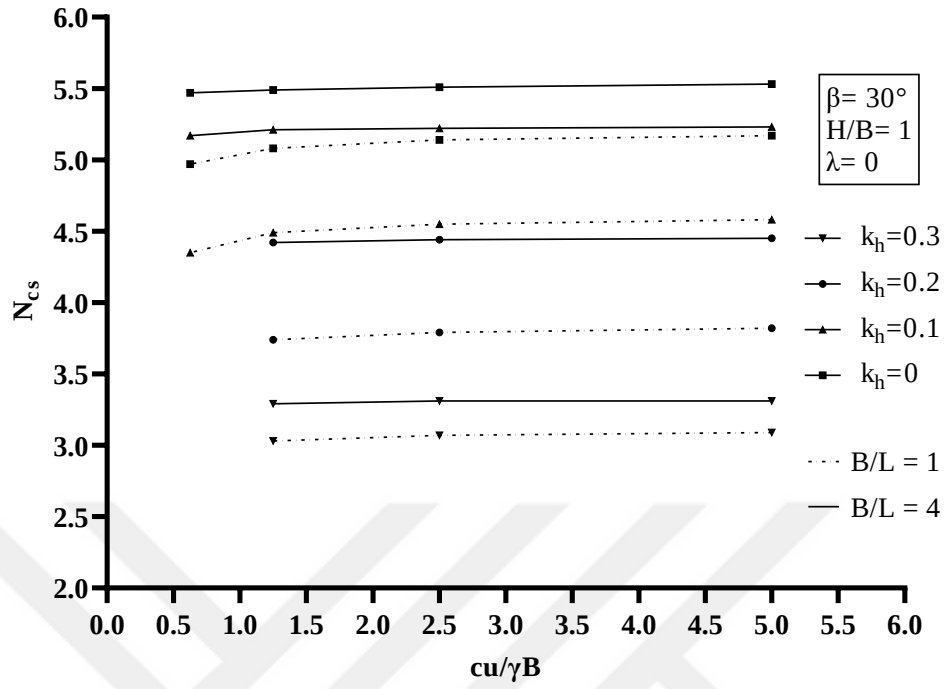


Figure 64: The effect of $c_u/\gamma B$ on N_{cs} for $B/L=1$ and $B/L=4$ ($\beta = 30^\circ$, $H/B = 1$, $\lambda = 0$).

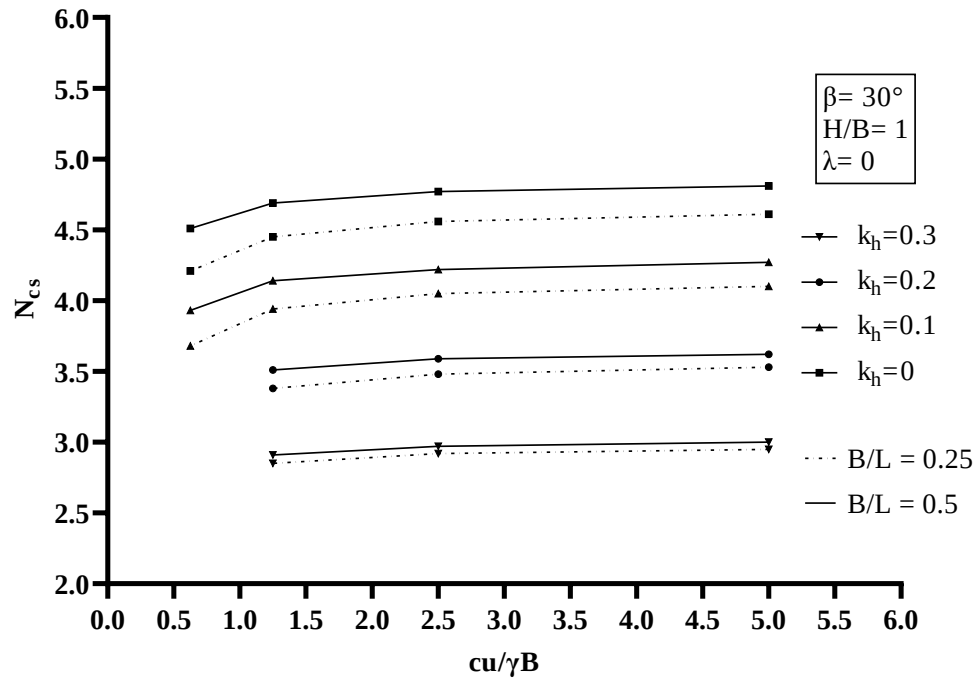


Figure 65: The effect of $c_u/\gamma B$ on N_{cs} for $B/L=0.25$ and $B/L=0.5$ ($\beta = 30^\circ$, $H/B = 1$, $\lambda = 0$).

4.1.4 Effect of Slope Angle β

Increasing the slope angle β often leads to a decrease in the seismic bearing capacity factor N_{cs} , as a result of the loss of stability as the slope becomes steeper. Figure 66 illustrates a linear trend when the footing is positioned on the slope crest ($\lambda = 0$), where the failure mechanism is consistently affected by the slope. As λ value increases and the footing moves away from the slope, the impact of slope angle β gradually diminishes, as the failure mechanism tends to occur independently of the slope. When $\lambda = 2$, the effect of slope angle β becomes negligible since the footing is sufficiently distant from the slope, and failure transpires completely unrelated to the slope. Similarly, in the case represented in Figure 67, the same behaviour is observed, but the effect of slope angle β becomes insignificant at $\lambda = 1$, due to the reason explained in Chapter 4.1.1. By increasing the B/L value, the foundation's tendency to fail parallel to the slope leads to failure occurring independently of the slope at lower λ values.

It's observed that the slope height, indicated by the normalized parameter H/B , generally does not affect the seismic bearing capacity factor N_{cs} , except in specific cases when the slope angle β is 60° . In this scenario, increasing the slope height results in an increased seismic bearing capacity factor N_{cs} . The reason behind this finding is that when $\beta = 60^\circ$ and the slope height is low, a larger portion of the slope is involved in the deformation mechanism, potentially leading to a failure mechanism. This emphasizes the significance of slope stability as the dominant factor. However, at higher slope heights, only a small portion of the slope is engaged in the deformation mechanism, resulting in a higher seismic bearing capacity factor N_{cs} . A comprehensive explanation of this behaviour will be provided later in the investigation of failure mechanisms.

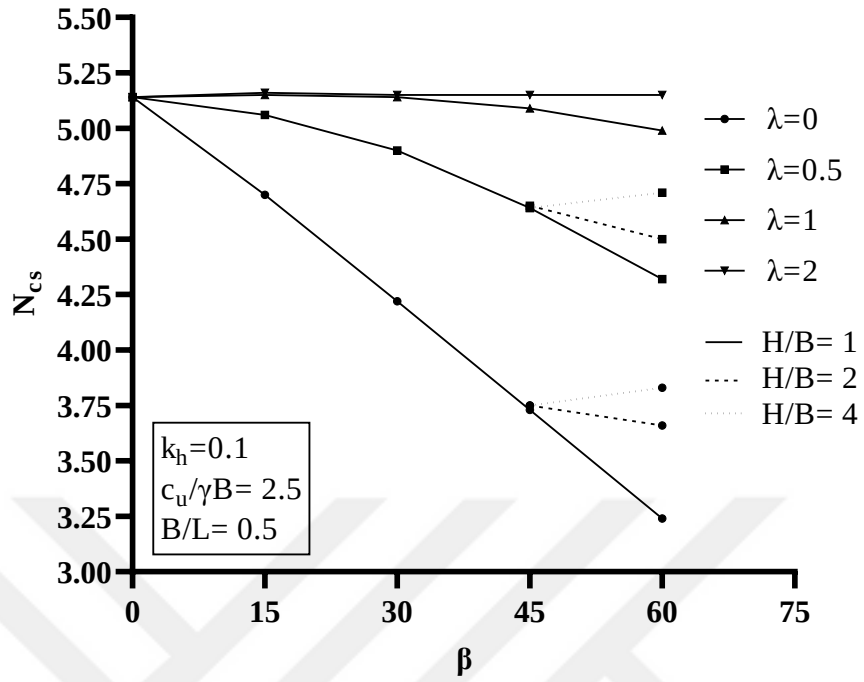


Figure 66: The effect of β on N_{cs} for different λ values ($k_h = 0.1$, $c_u/\gamma B = 2.5$, $B/L = 0.5$).

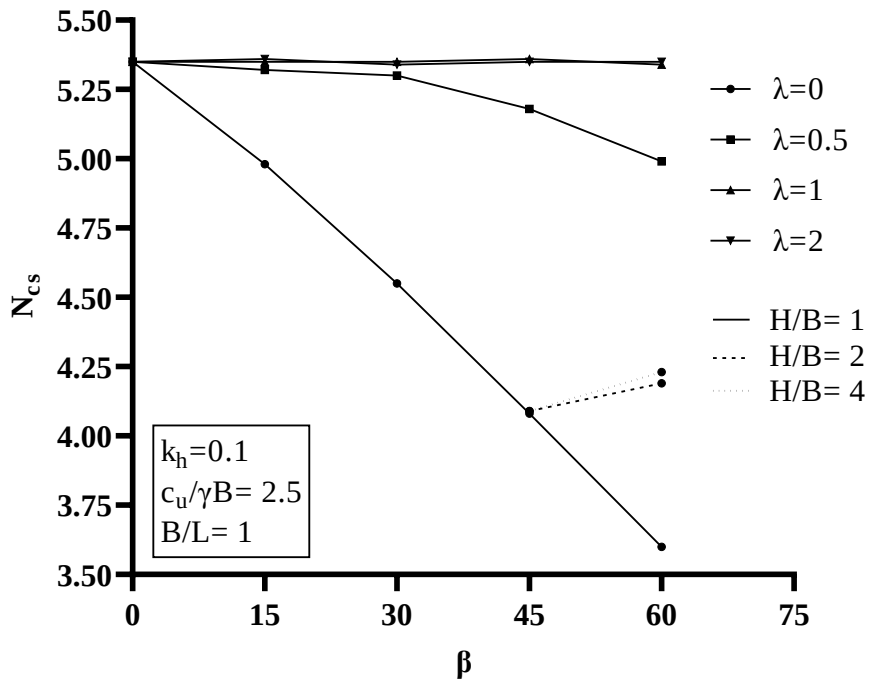


Figure 67: The effect of β on N_{cs} for different λ values ($k_h = 0.1$, $c_u/\gamma B = 2.5$, $B/L = 1$).

4.1.5 Effect of Normalised Footing Distance λ

The normalized footing distance λ plays a crucial role in this study. Upon examining Figures 68 and 69, it becomes evident that as the footing moves farther away from the edge of the slope, the seismic bearing capacity factor N_{cs} increases. This behaviour can be attributed to the gradual diminishing of the slope's influence as the footing distances itself from the slope edge. Additionally, it is observed that the increase in bearing capacity factor N_{cs} continues until reaching a threshold value ($\lambda_{\text{threshold}}$), beyond which it remains constant regardless of the distance. At this threshold point, the model fails entirely, irrespective of the slope, indicating that the slope no longer affects the bearing capacity. Moreover, an increase in the horizontal acceleration component k_h leads to a decrease in the $\lambda_{\text{threshold}}$, as the failure mechanism becomes shallower. Similarly, an increase in the aspect ratio of the footing B/L results in a decrease in the $\lambda_{\text{threshold}}$, as the failure mechanism aligns parallel to the slope edge.

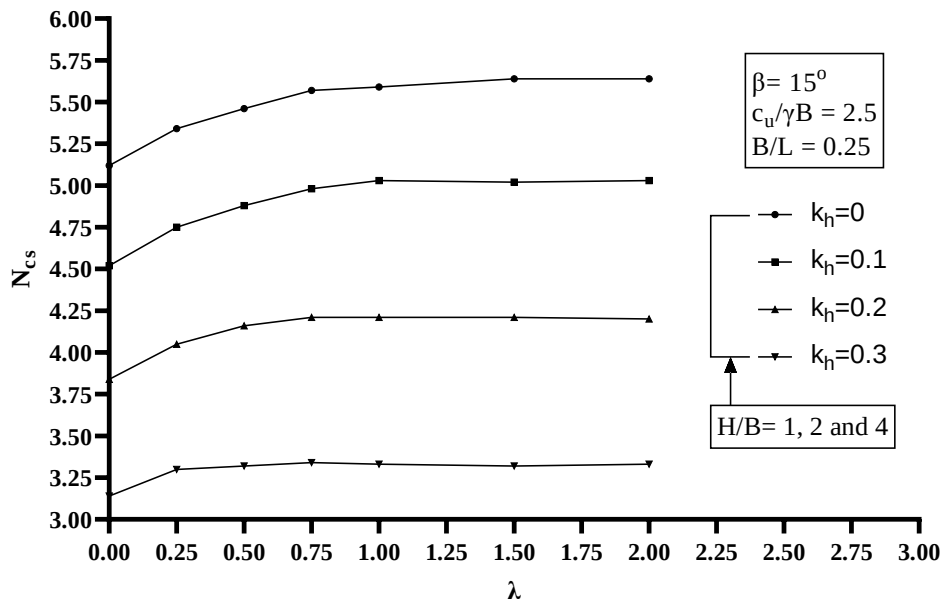


Figure 68: The effect of λ on N_{cs} for different k_h values ($\beta = 15^\circ$, $c_u/\gamma B = 2.5$, $B/L = 0.25$).

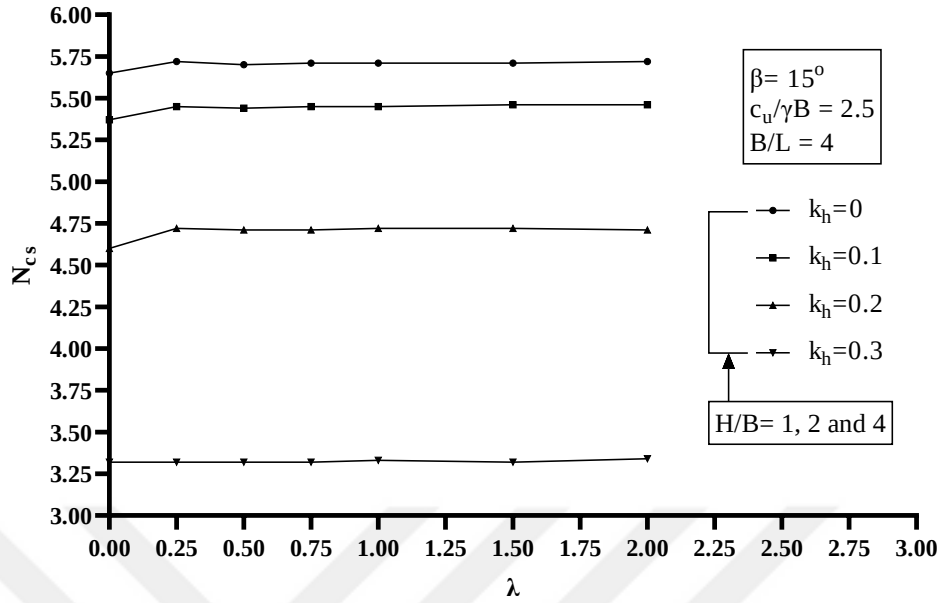


Figure 69: The effect of λ on N_{cs} for different k_h values ($\beta = 15^\circ$, $c_u/\gamma B = 2.5$, $B/L = 4$).

4.1.6 Effect of slope height H/B

As explained in Chapter 4.1.4, the slope height, indicated by the normalized parameter H/B , generally does not affect the seismic bearing capacity factor N_{cs} , except in specific cases when the slope angle β is 60° . In addition to the case explained before, it's also observed that the slope height affects the failure type when the slope has a low soil strength value c_u , as the increase of slope height will cause the slope to fail due to seismic loads firstly, then due its own loads at greater heights. It was also noted that the slope height affects the seismic bearing capacity factor N_{cs} in some cases where the slope is barely stable.

4.2 Failure Mechanisms

As previously mentioned, the model in this study can encounter three different failure modes. The first involves slope instability resulting from gravity load, as the Plaxis

model fails at the initial phase, as illustrated in Figure 70. The second mode pertains to slope instability due to seismic inertia force, where the Plaxis model fails at the third phase, as shown in Figure 71. The final failure mode concerns bearing capacity failure caused by reaching the ultimate load, when the model fails at the last phase.

Regarding the bearing capacity failure, where the model fails at the last phase, the failure mechanism occurs beneath the footing. The failure mechanism initiates at the footing edge, extends across the slope body, and terminates at the slope line or reaches the slope toe. When the footing is positioned at a significant distance from the slope edge, the failure mechanism may occur regardless of the slope's presence. Figure 72 illustrates various failure modes observed in this study. It's important to mention that the failure modes observed in this study align with those identified by Meyerhof [6].

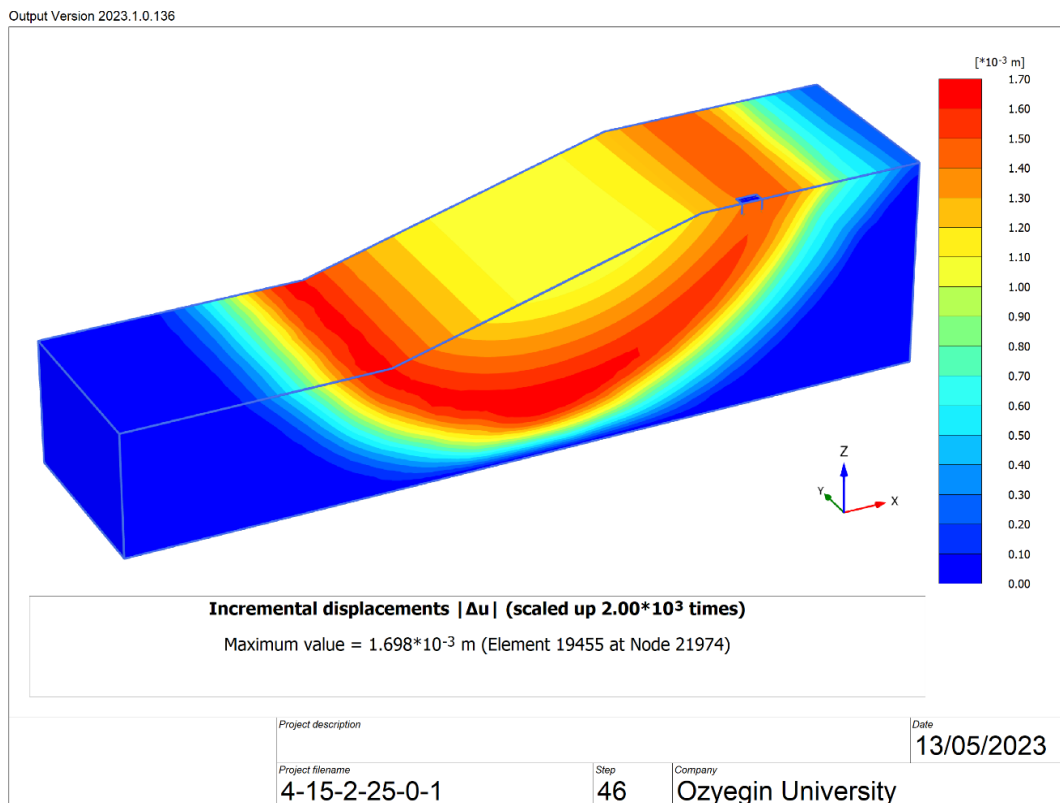


Figure 70: Failure mechanism due to gravity load ($\beta = 15^\circ$, $H/B = 4$, $c_u/\gamma B = 0.625$).

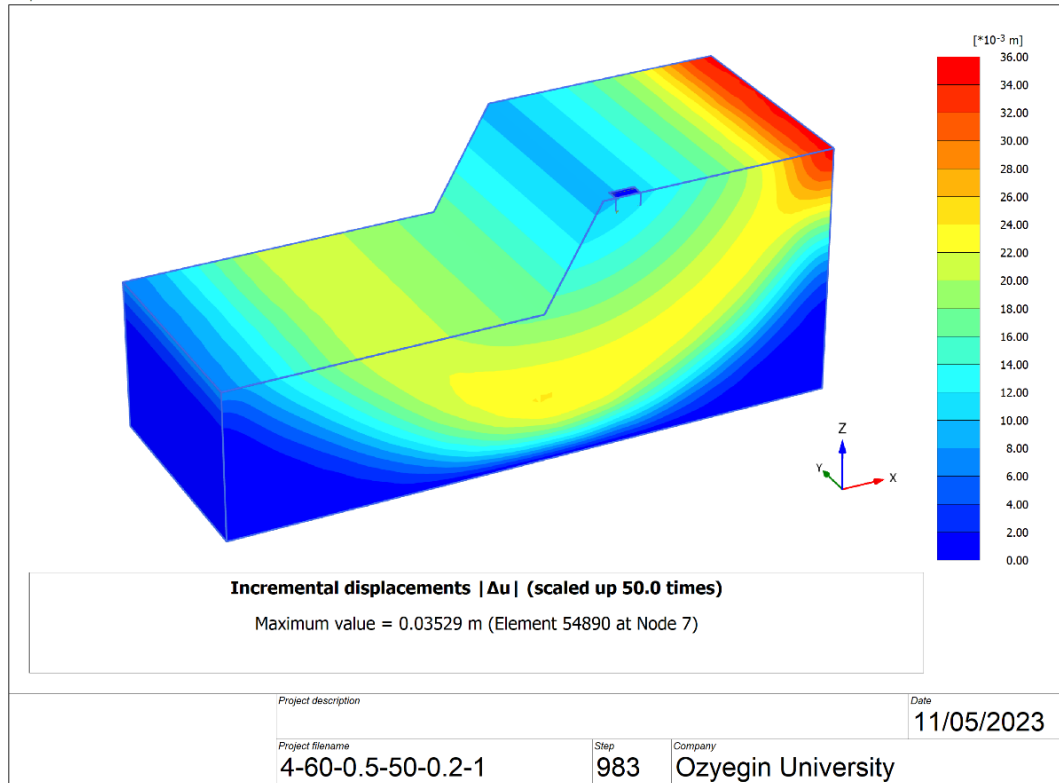


Figure 71: Failure mechanism due to seismic inertia force ($\beta = 60^\circ$, $H/B = 4$, $c_u/\gamma B = 1.25$, $k_h = 0.2$).

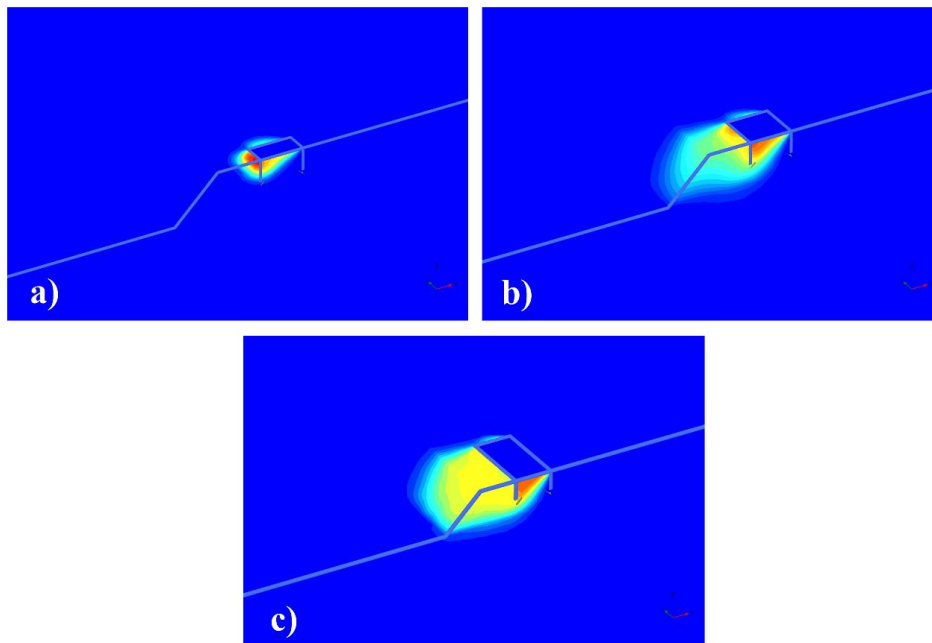


Figure 72: Bearing capacity failure modes for $H/B = 1$, $k_h = 0.1$, $\beta = 45^\circ$, $c_u = 25$ kPa and a) $B/L = 1$; b) $B/L = 0.5$; c) $B/L = 0.25$.

In addition to the previously discussed bearing capacity failure modes, it was observed that the model can experience failure prior to reaching its ultimate bearing capacity due to sliding behaviour, as detailed in Chapter 4.1.1. Considering Richards et al. [25], it was suggested that a sliding failure mechanism occurs when the failure surface line aligns with the foundation width, as shown in Figure 73. Specifically, sliding occurs when the horizontal component of the bearing pressure at failure reaches the undrained strength of the soil c_u . Consequently, sliding failure is triggered when the product of the seismic bearing capacity factor N_{cs} and the horizontal seismic acceleration coefficient k_h equals unity. This relationship can be expressed using Equation (30), as shown below:

$$q_u = N_{cs} c_u$$

by multiplying both sides with k_h ,

$$q_u k_h = N_{cs} c_u k_h \quad (32)$$

The left side of the equation becomes the horizontal component of bearing capacity, which is equal to the soil strength c_u , accordingly the equation becomes as below,

$$N_{cs} k_h = 1 \quad (33)$$

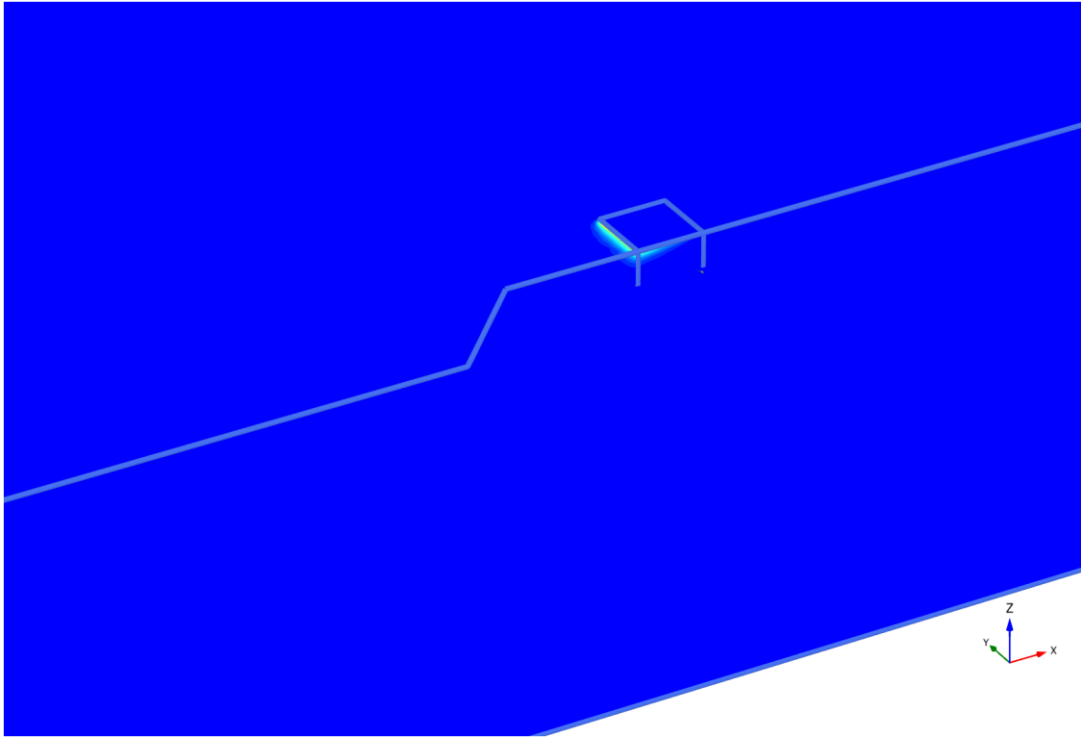


Figure 73: Sliding failure mechanism for ($H/B= 1$, $k_h = 0.3$, $\beta = 60^\circ$, $c_u = 100$ kPa, $\lambda= 2$, $B/L= 0.5$).

The sliding failure mode in this study was observed exclusively when the horizontal seismic acceleration coefficient k_h is 0.3, in conjunction with a seismic bearing capacity factor N_{cs} of 3.33. A meticulous investigation was undertaken to ascertain the threshold value of k_h that induces sliding failure for each B/L value. To accomplish this, the problem was examined on level ground, as the sliding failure mechanism primarily occurs at a shallow depth, unaffected by the slope's influence. The investigation revealed that, except for the B/L value of 4, a threshold k_h of 0.3 was determined for all B/L values employed in this study. However, when B/L is 4, the threshold k_h value leading to sliding failure was found to be 0.26. This implies that any k_h value exceeding 0.26 will result in sliding failure for the model. Detailed information regarding these findings is presented in Table 11.

Table 11: The effect of k_h on N_{cs} for level ground case. ($B/L = 4$)

k_h	N_{cs}	$k_h N_{cs}$
0	5.66	0
0.1	5.41	0.541
0.2	4.70	0.94
0.22	4.36	0.96
0.23	4.22	0.97
0.24	4.08	0.98
0.25	3.96	0.99
0.26	3.85	1
0.27	3.70	1
0.3	3.33	1
0.4	2.50	1
0.5	2.00	1
0.6	1.67	1
0.7	1.43	1
0.8	1.25	1
0.9	1.11	1
1	1.00	1

4.3 Design Charts

After conducting a comprehensive study on the impact of various parameters on the seismic bearing capacity of a foundation situated near a cohesive slope, it became necessary to develop a design methodology that could assist geotechnical engineers in predicting the seismic bearing capacity for such scenarios. To achieve this, design charts were created, incorporating the relationship between N_{cs} and λ for each footing shape considered in this study. As described in Chapter 4.1, the N_{cs} value initially increases with λ until it reaches a critical point, λ_o . Beyond this critical point, the N_{cs} remains constant since the failure mechanism is no longer affected by the slope.

By integrating these findings, the design charts offer a practical tool for geotechnical engineers. They provide a graphical representation that enables engineers

to estimate the seismic bearing capacity based on the values of λ and B/L , while considering the effects of other influential parameters. These charts serve as a valuable resource for designing foundations near cohesive slopes, facilitating informed decision-making and ensuring the structural integrity of such foundations under seismic conditions.

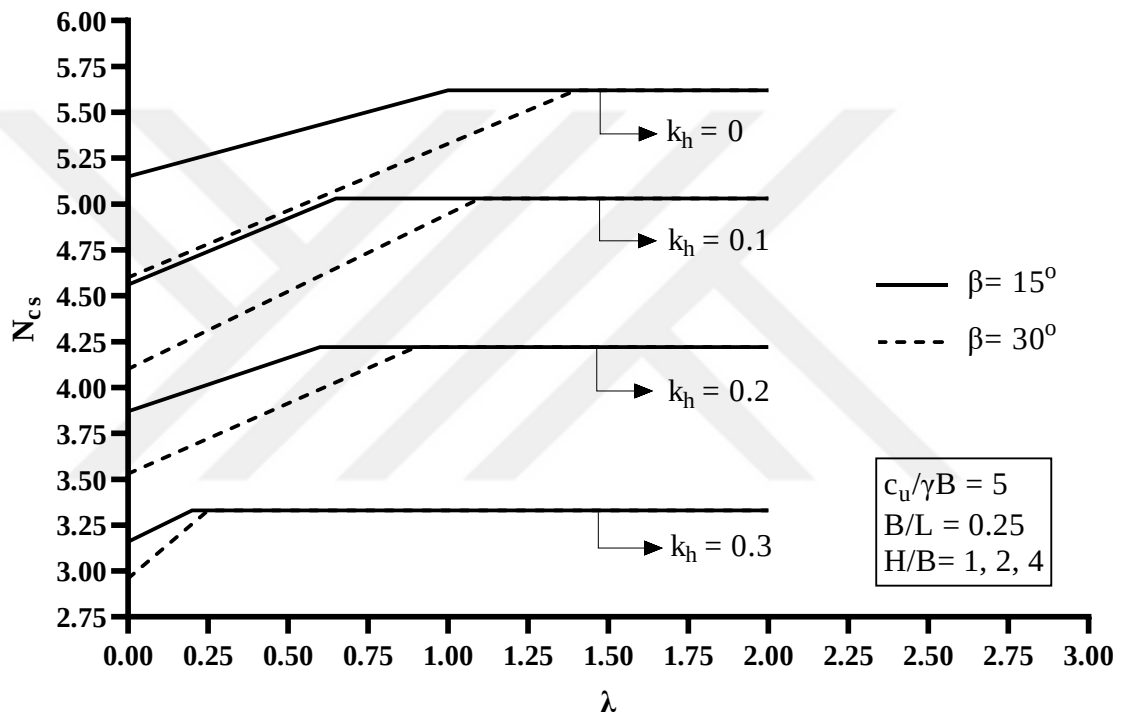


Figure 74: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 5$, $H/B = 1, 2, 4$, $B/L = 0.25$).

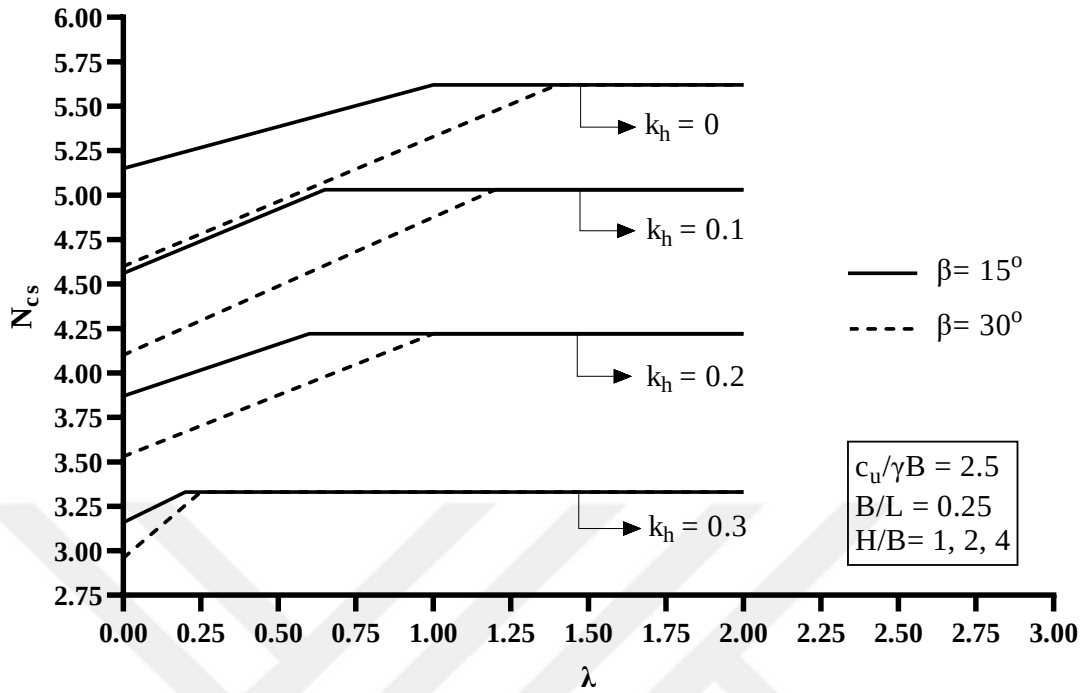


Figure 75: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 2.5$, $H/B = 1, 2, 4$, $B/L = 0.25$).

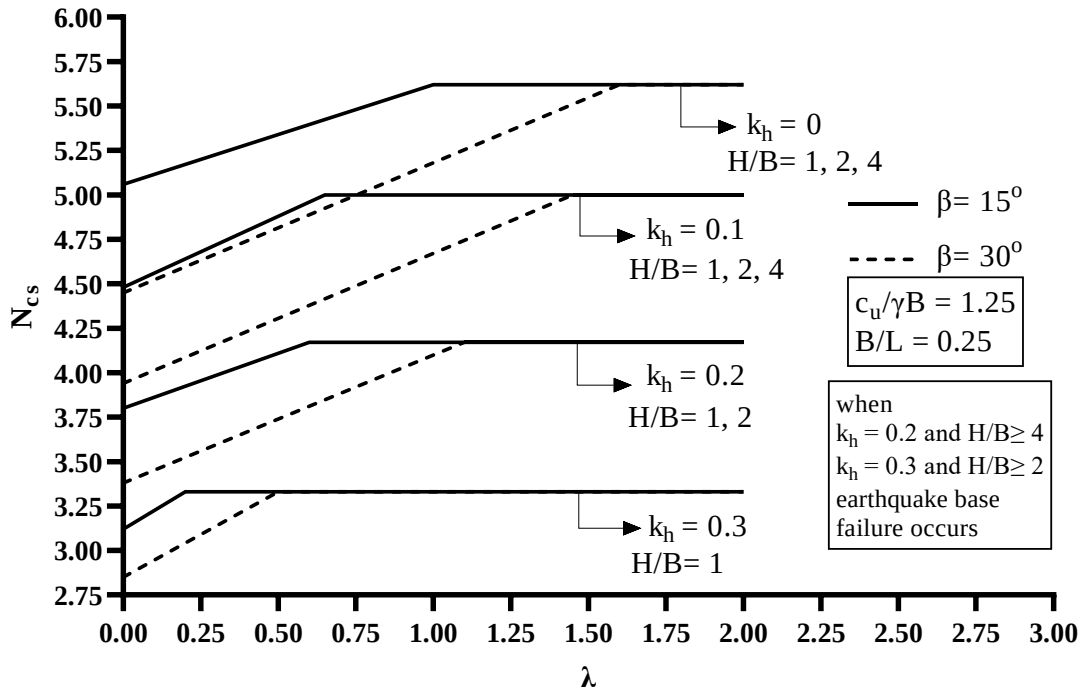


Figure 76: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 1.25$, $H/B = 1, 2, 4$, $B/L = 0.25$).

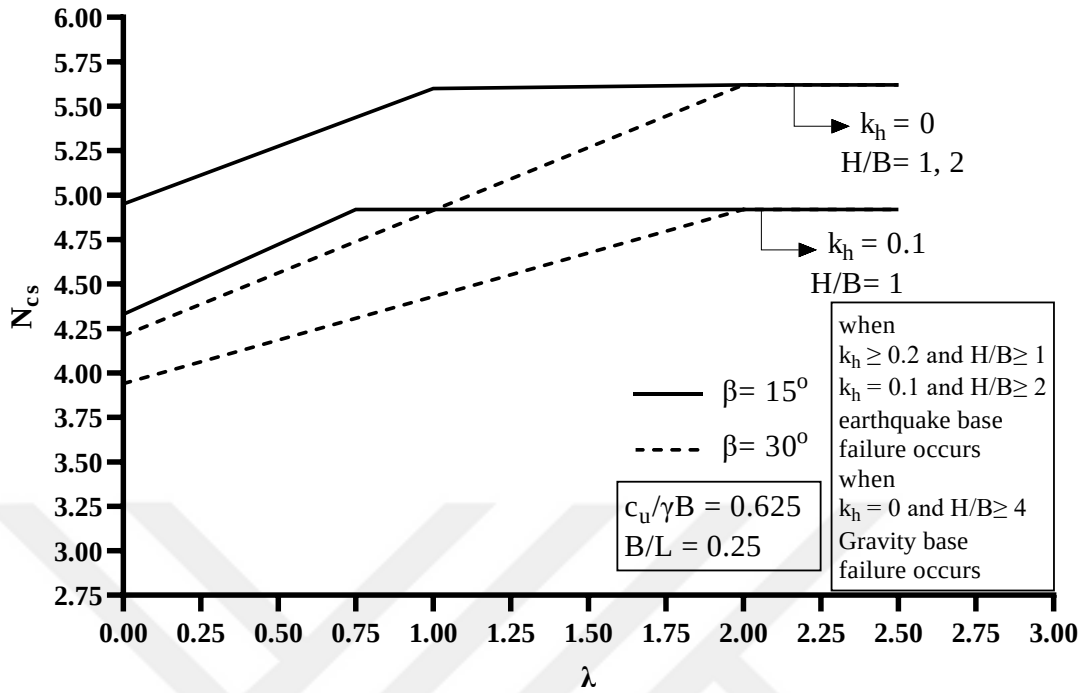


Figure 77: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 0.625$, $H/B = 1, 2$, $B/L = 0.25$).

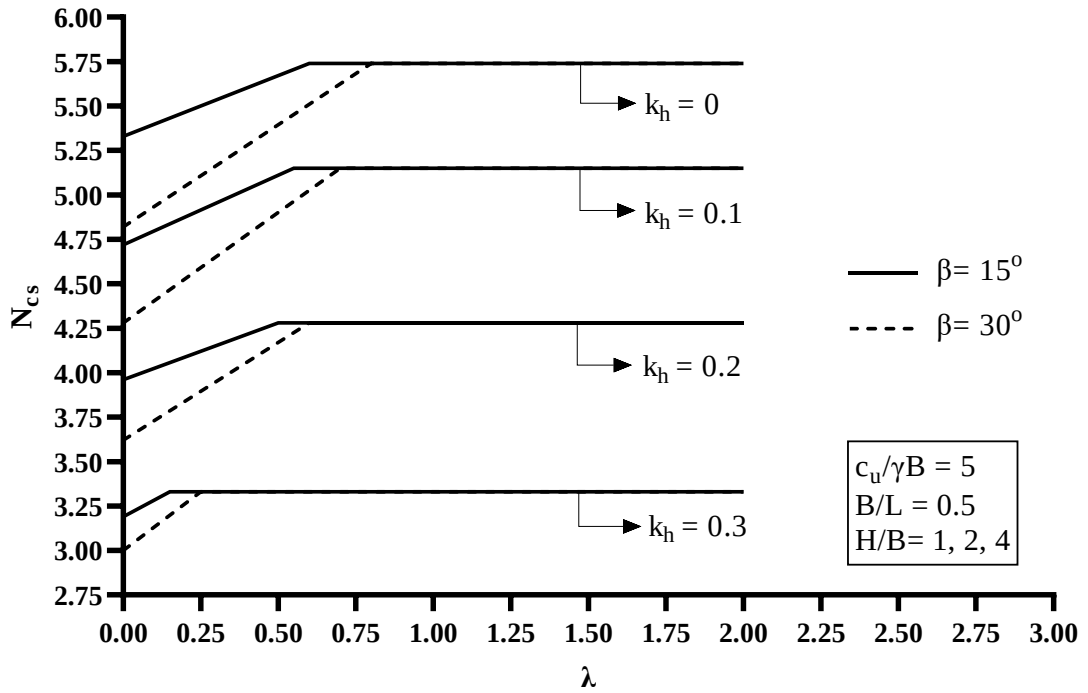


Figure 78: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 5$, $H/B = 1, 2, 4$, $B/L = 0.5$).

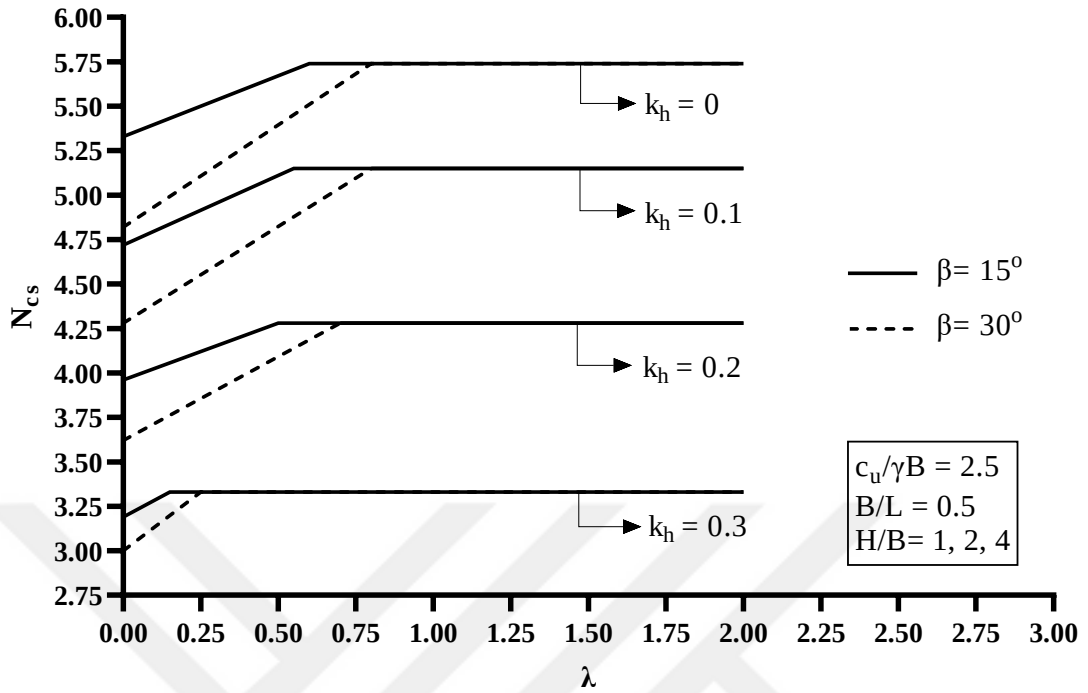


Figure 79: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 2.5$, $H/B = 1, 2, 4$, $B/L = 0.5$).

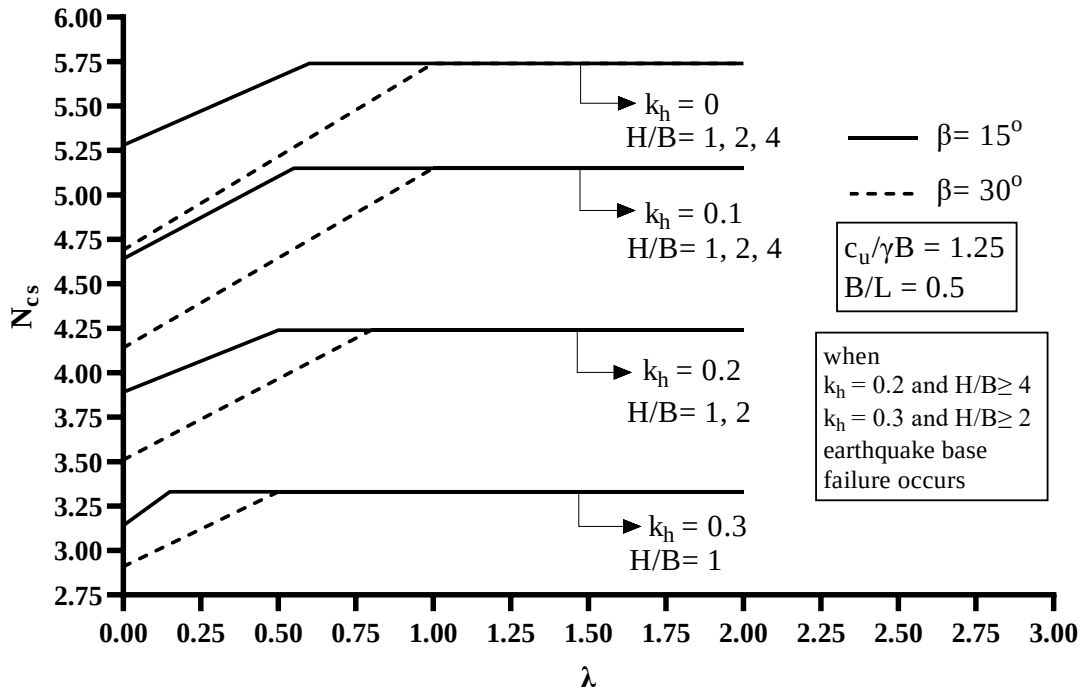


Figure 80: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 1.25$, $H/B = 1, 2, 4$, $B/L = 0.5$).

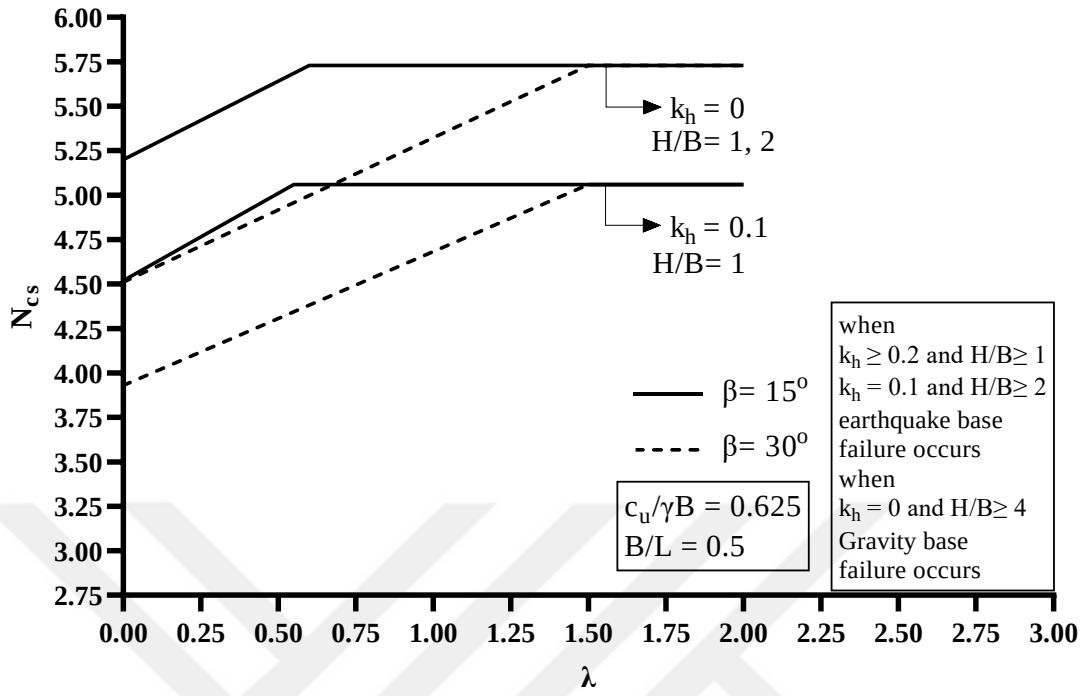


Figure 81: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 0.625$, $H/B = 1, 2$, $B/L = 0.5$).

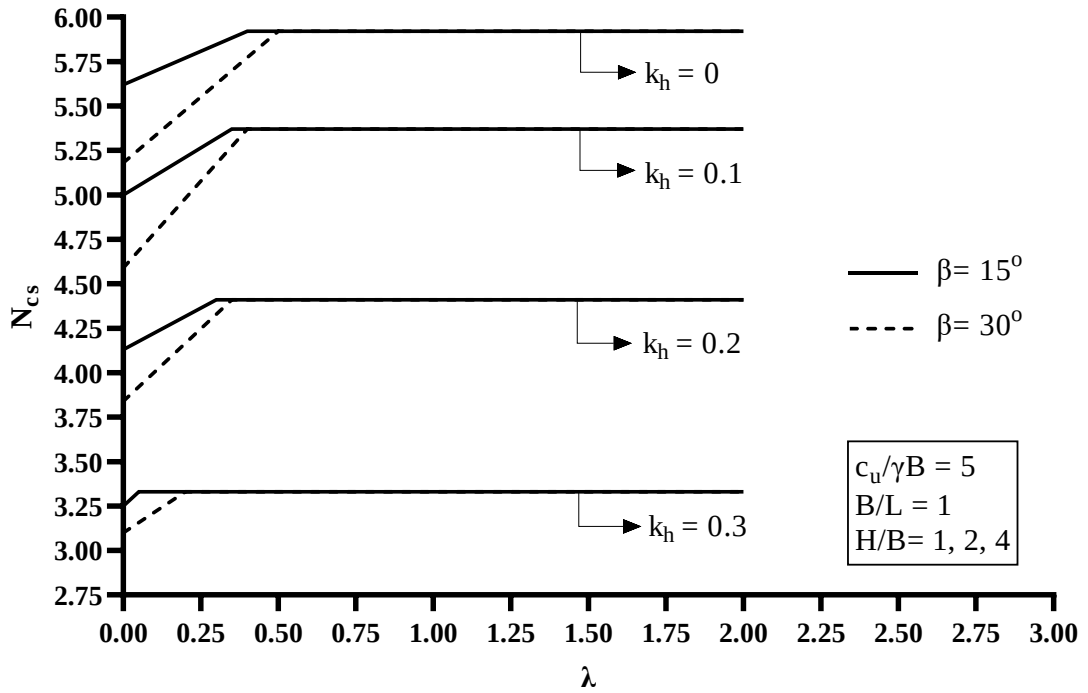


Figure 82: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 5$, $H/B = 1, 2, 4$, $B/L = 1$).

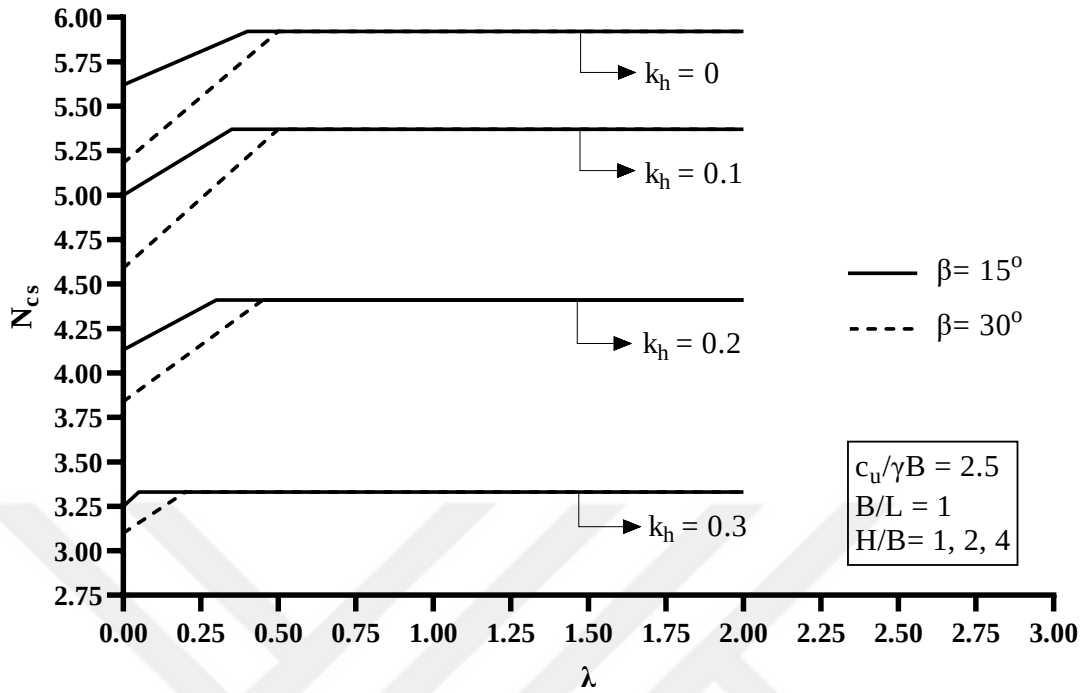


Figure 83: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 2.5$, $H/B = 1, 2, 4$, $B/L = 1$).

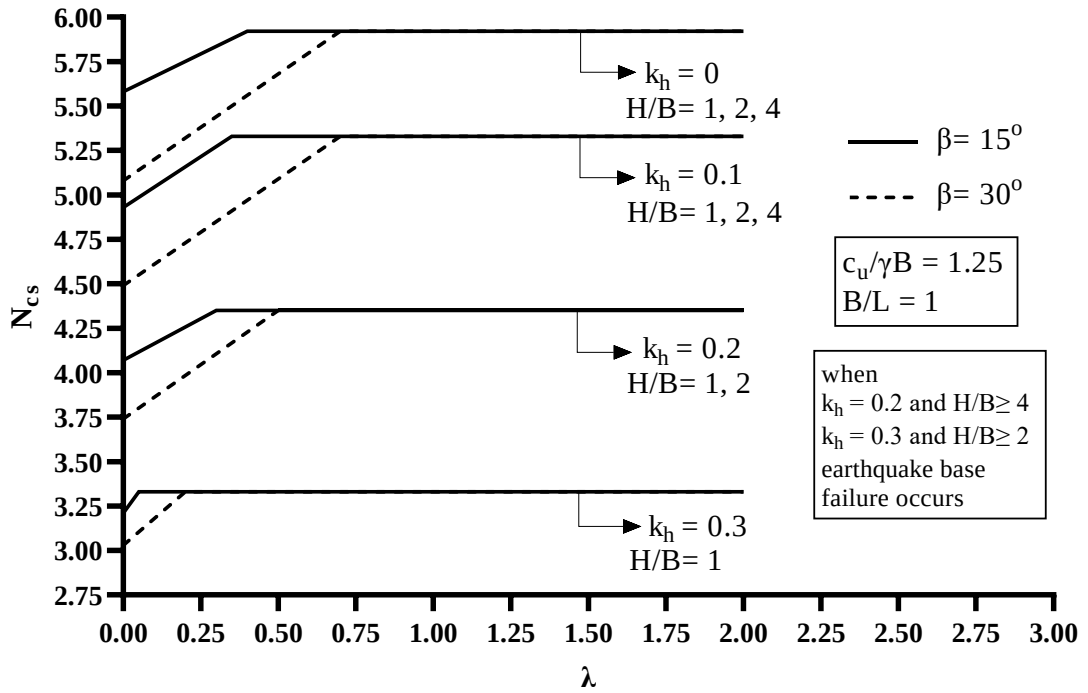


Figure 84: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 1.25$, $H/B = 1, 2, 4$, $B/L = 1$).

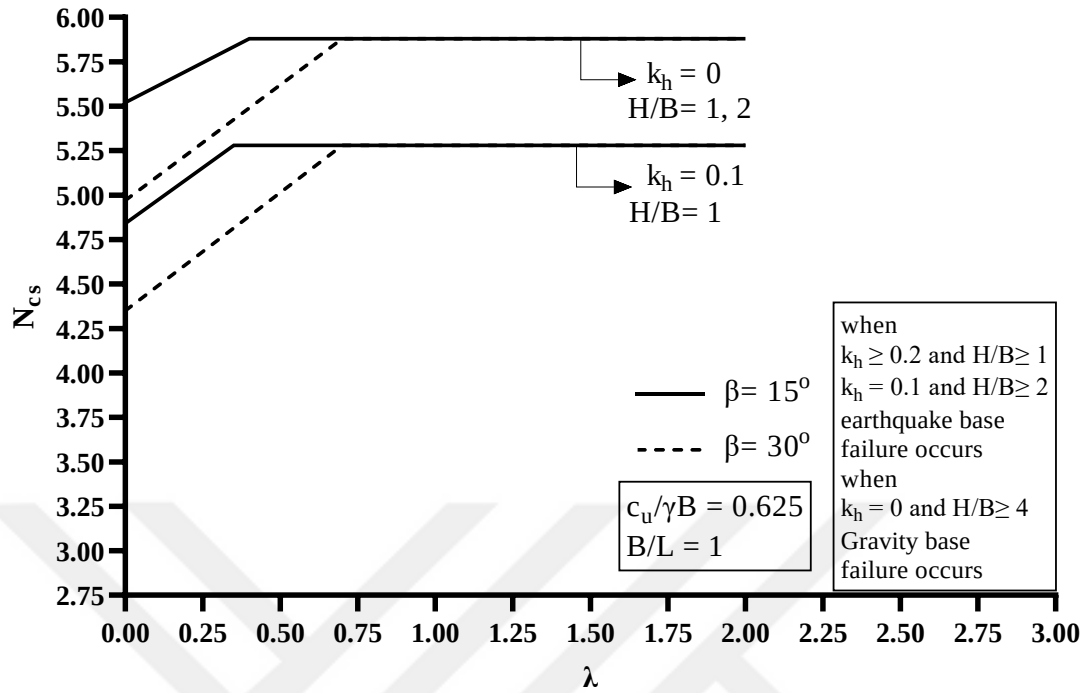


Figure 85: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 0.625$, $H/B = 1, 2$, $B/L = 1$).

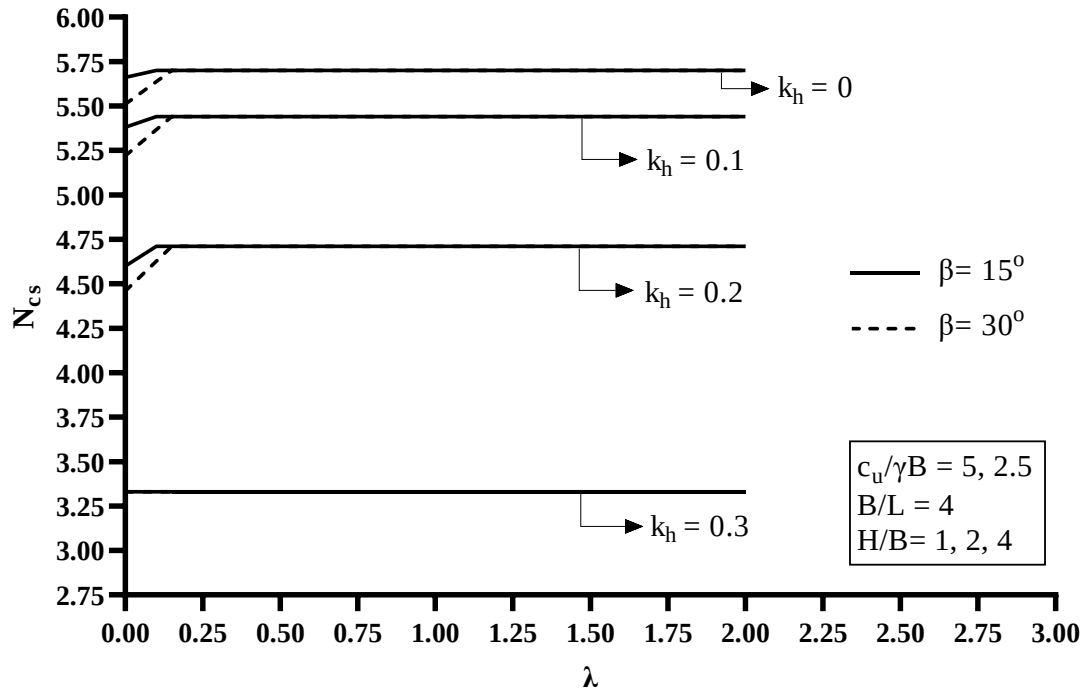


Figure 86: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 5, 2.5$, $H/B = 1, 2, 4$, $B/L = 4$).

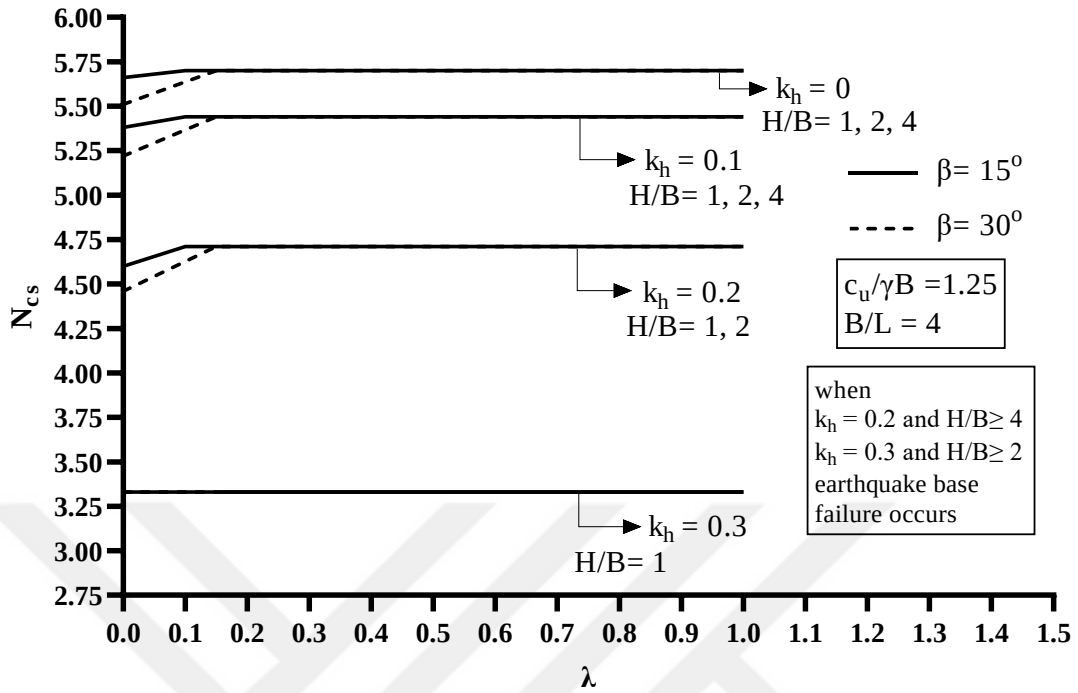


Figure 87: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 1.25$, $H/B = 1, 2, 4$, $B/L = 4$).

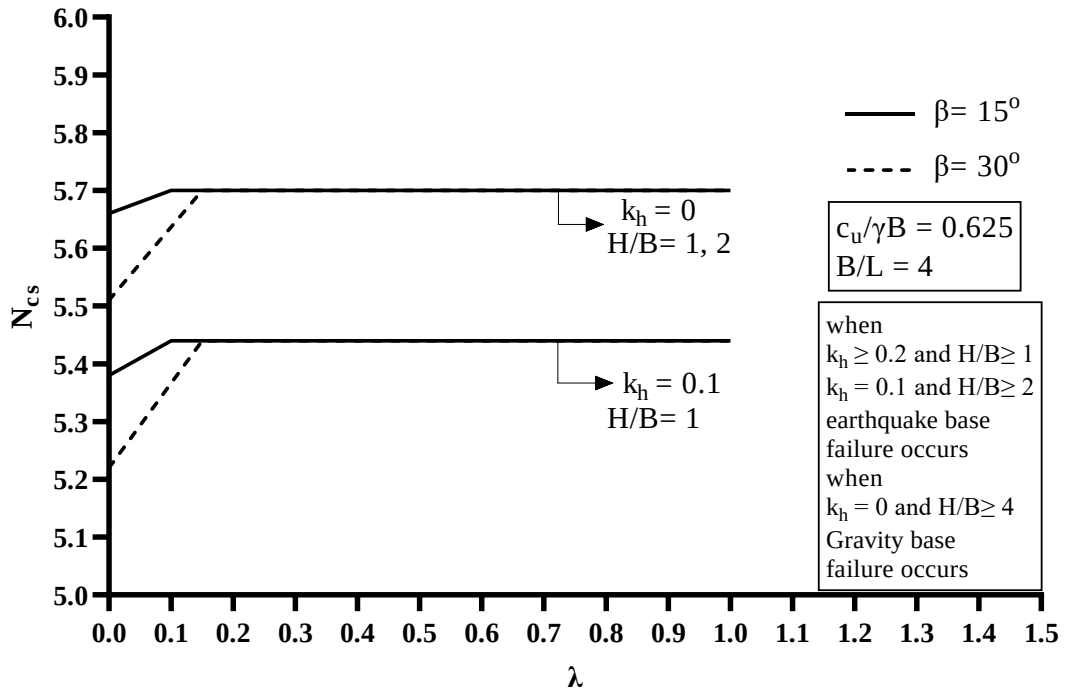


Figure 88: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 15^\circ, 30^\circ$, $c_u/\gamma B = 0.625$, $H/B = 1, 2$, $B/L = 4$).

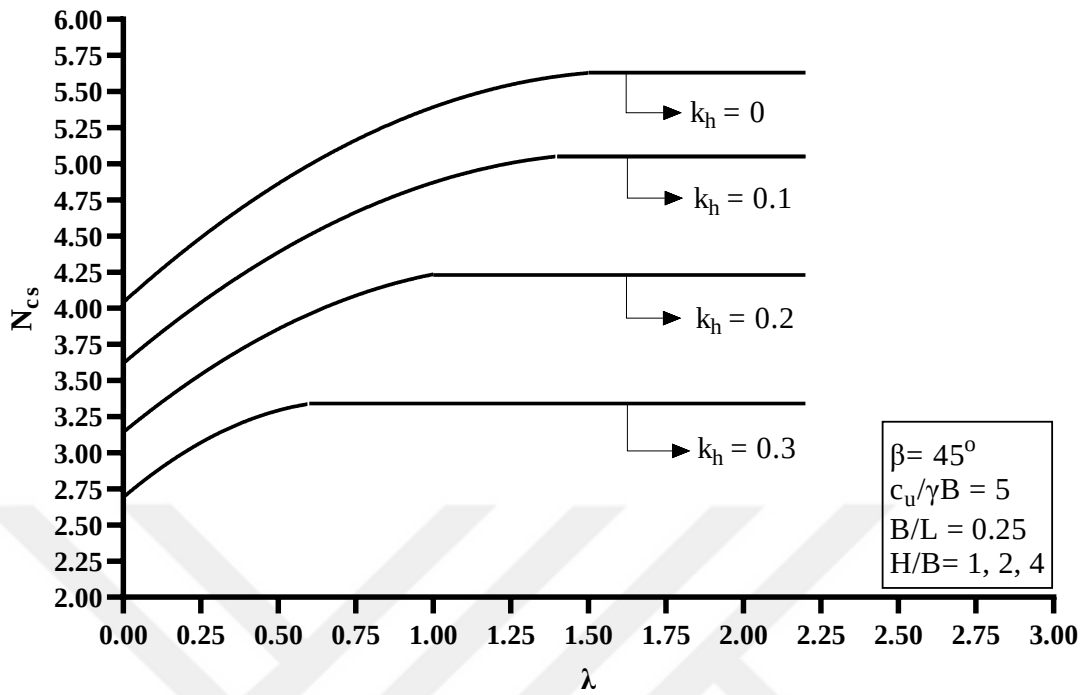


Figure 89: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 5$, $H/B = 1, 2, 4$, $B/L = 0.25$).

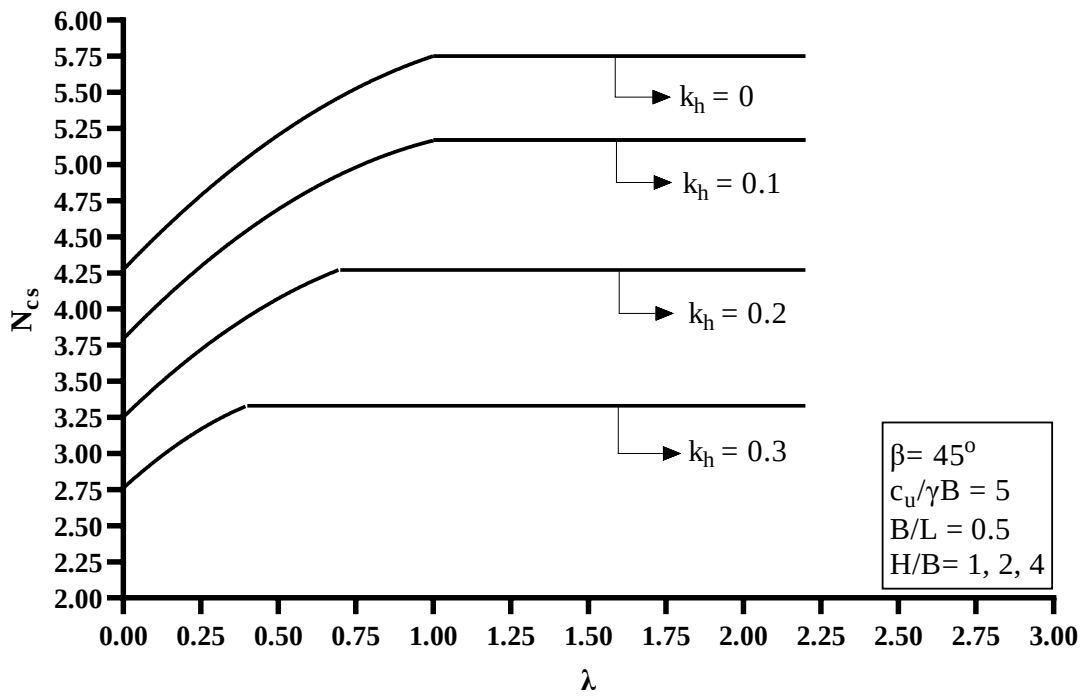


Figure 90: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 5$, $H/B = 1, 2, 4$, $B/L = 0.5$).

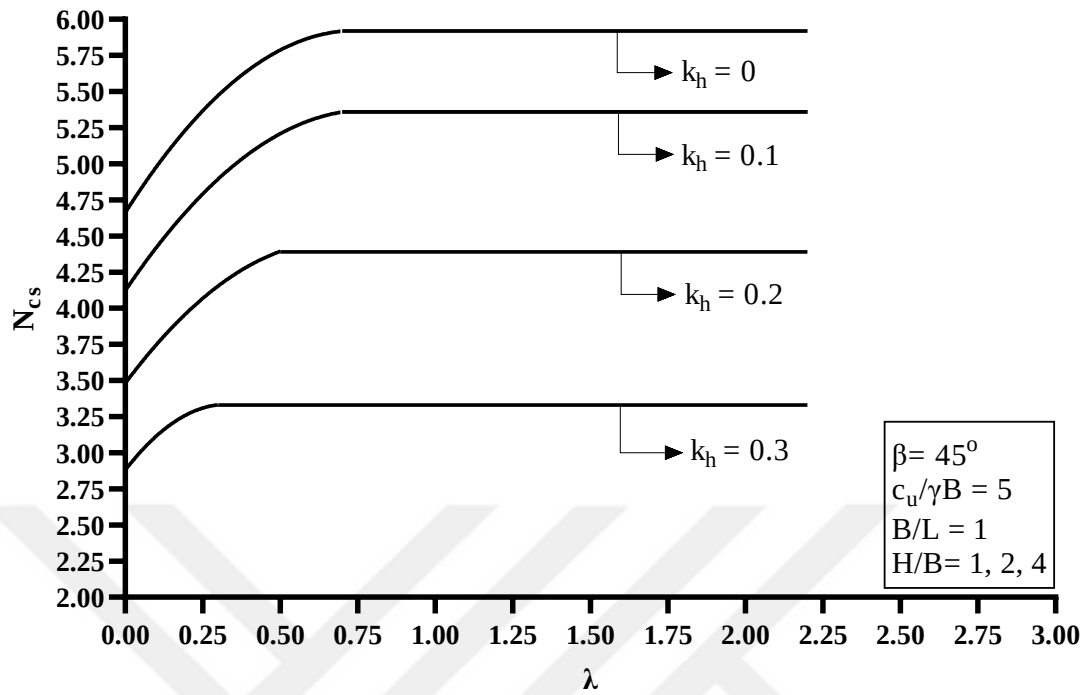


Figure 91: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 5$, $H/B = 1, 2, 4$, $B/L = 1$).

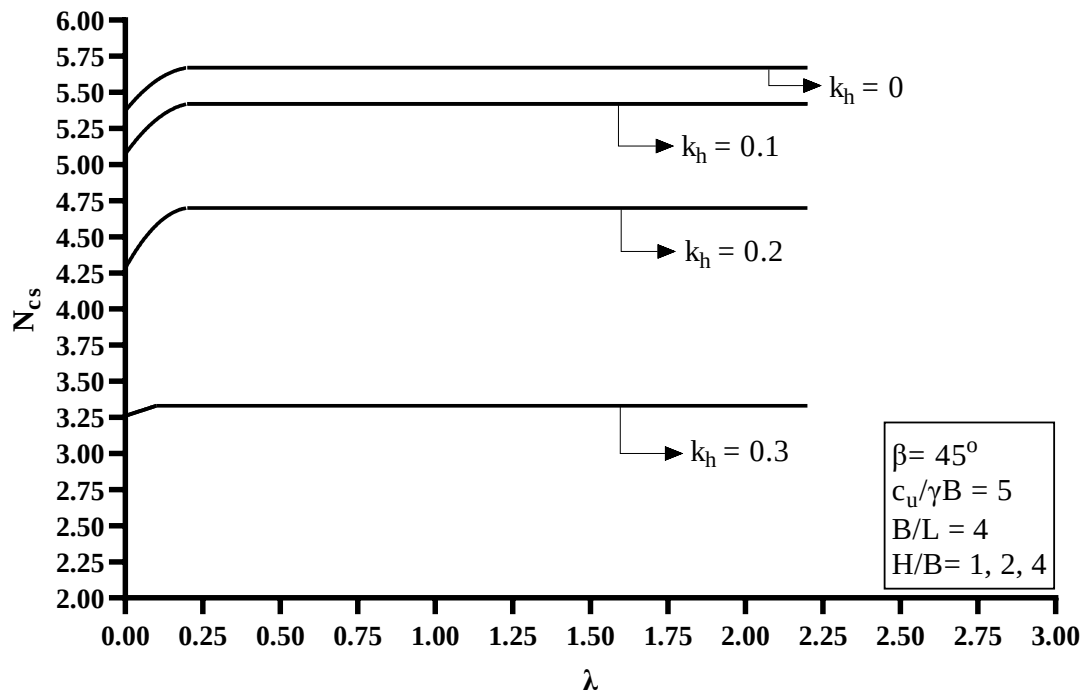


Figure 92: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 5$, $H/B = 1, 2, 4$, $B/L = 4$).

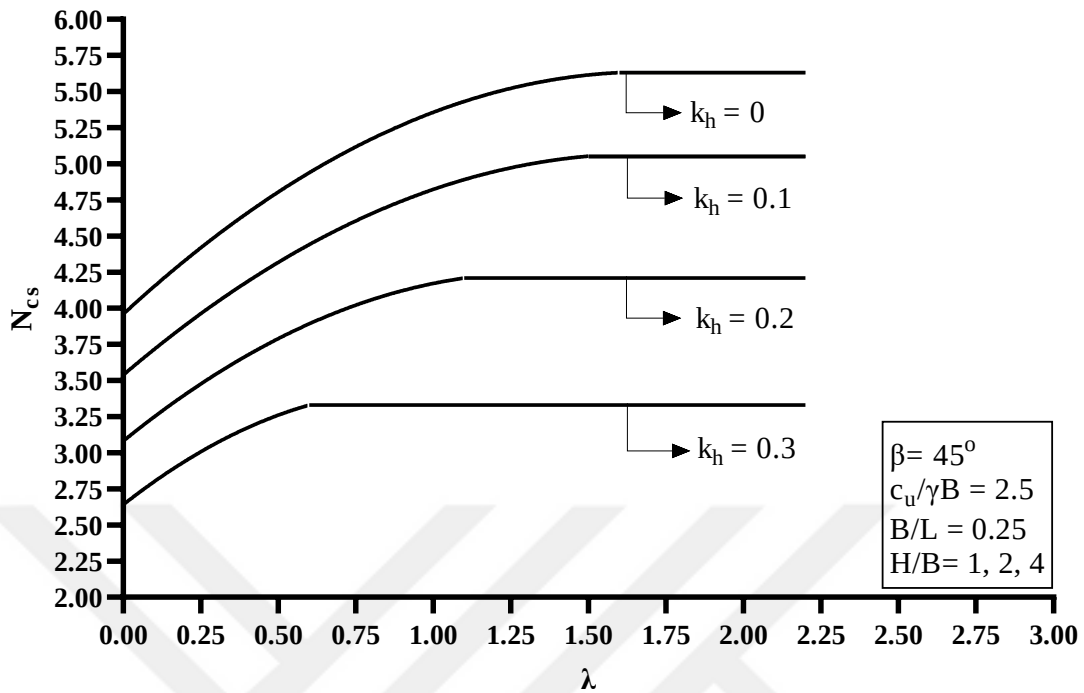


Figure 93: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 2.5$, $H/B = 1, 2, 4$, $B/L = 0.25$).

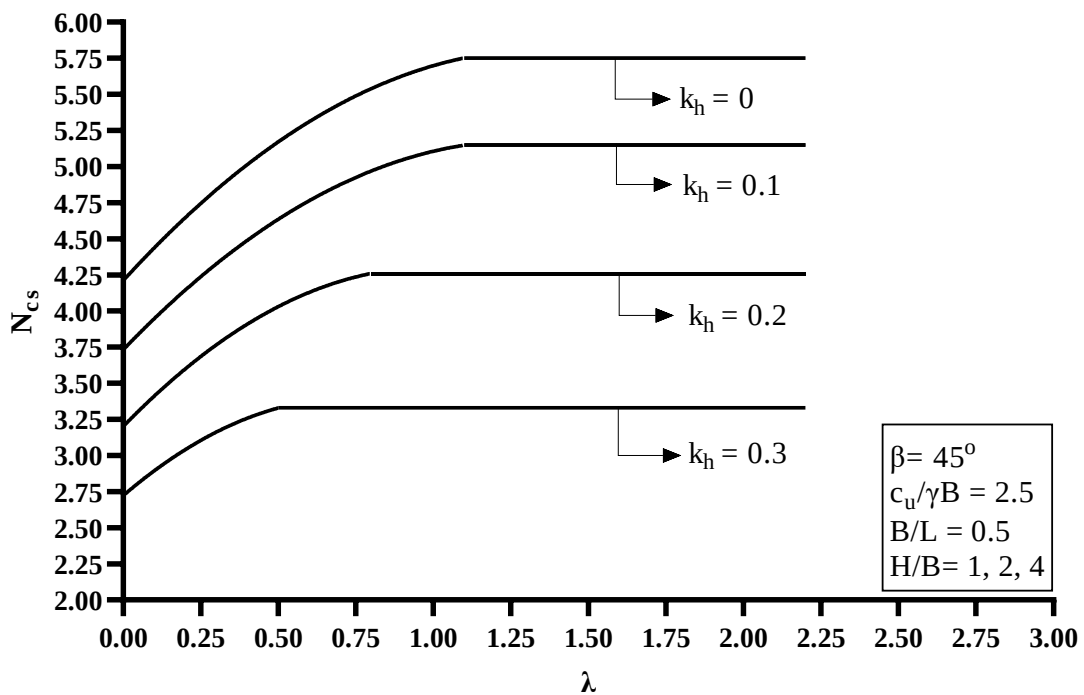


Figure 94: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 2.5$, $H/B = 1, 2, 4$, $B/L = 0.5$).

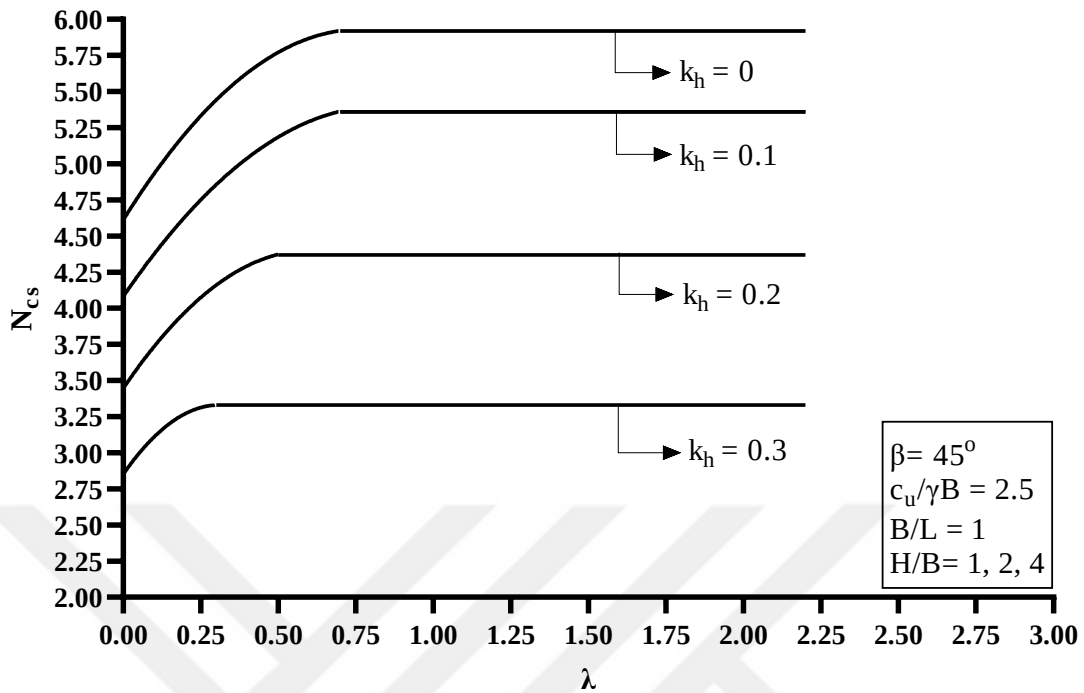


Figure 95: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 2.5$, $H/B = 1, 2, 4$, $B/L = 1$).

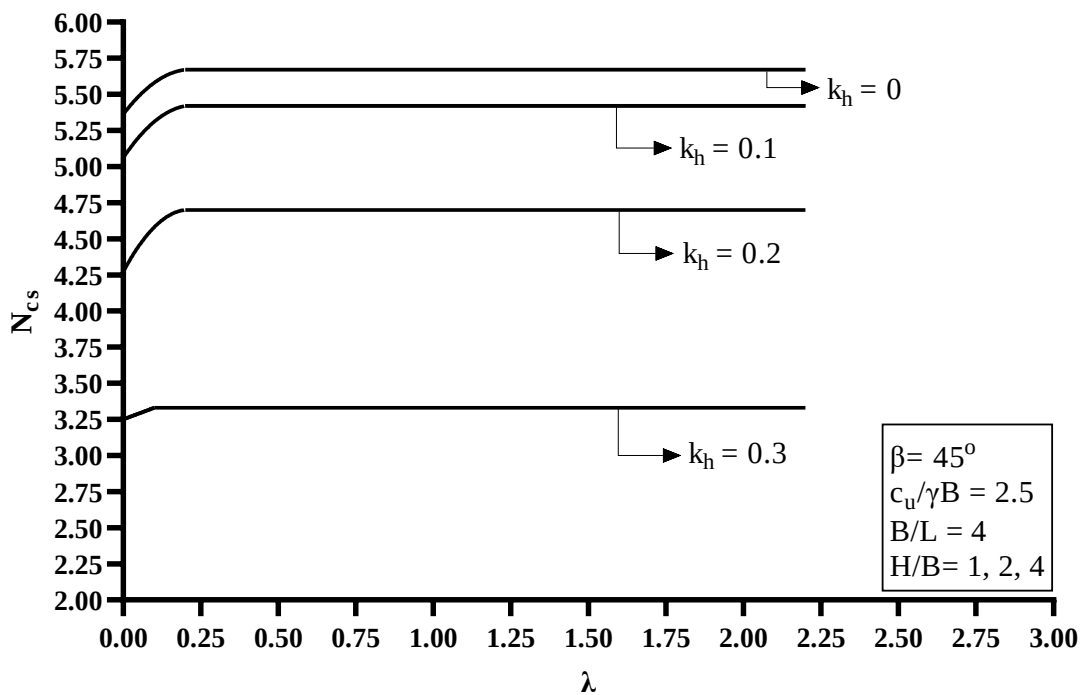


Figure 96: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 2.5$, $H/B = 1, 2, 4$, $B/L = 4$).

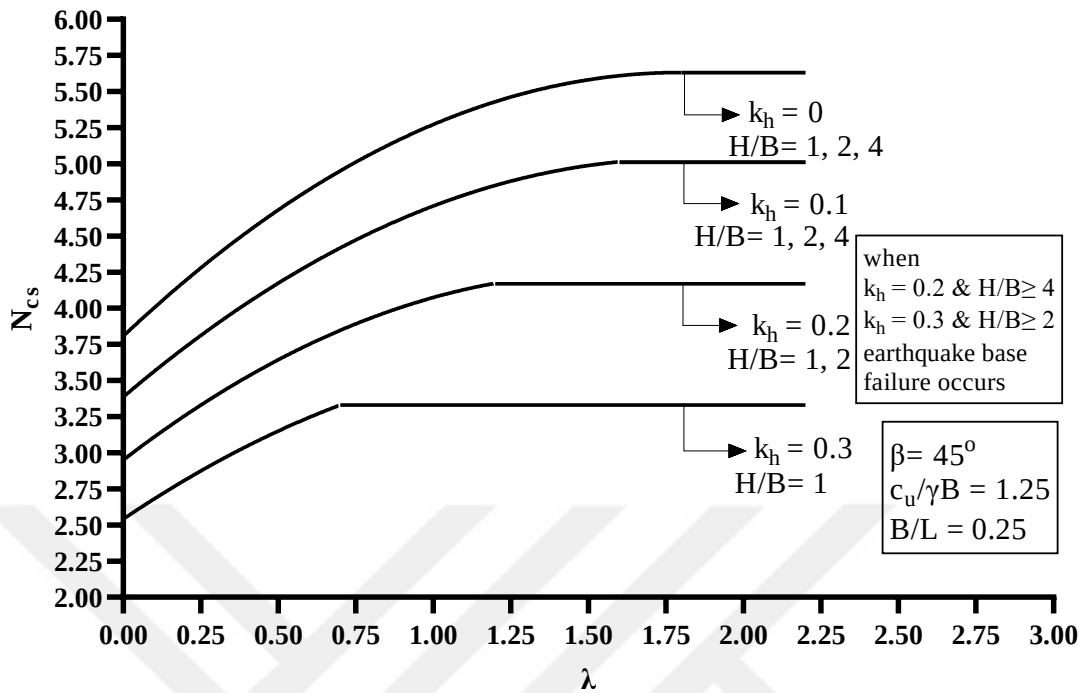


Figure 97: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 1.25$, $H/B = 1, 2, 4$, $B/L = 0.25$).

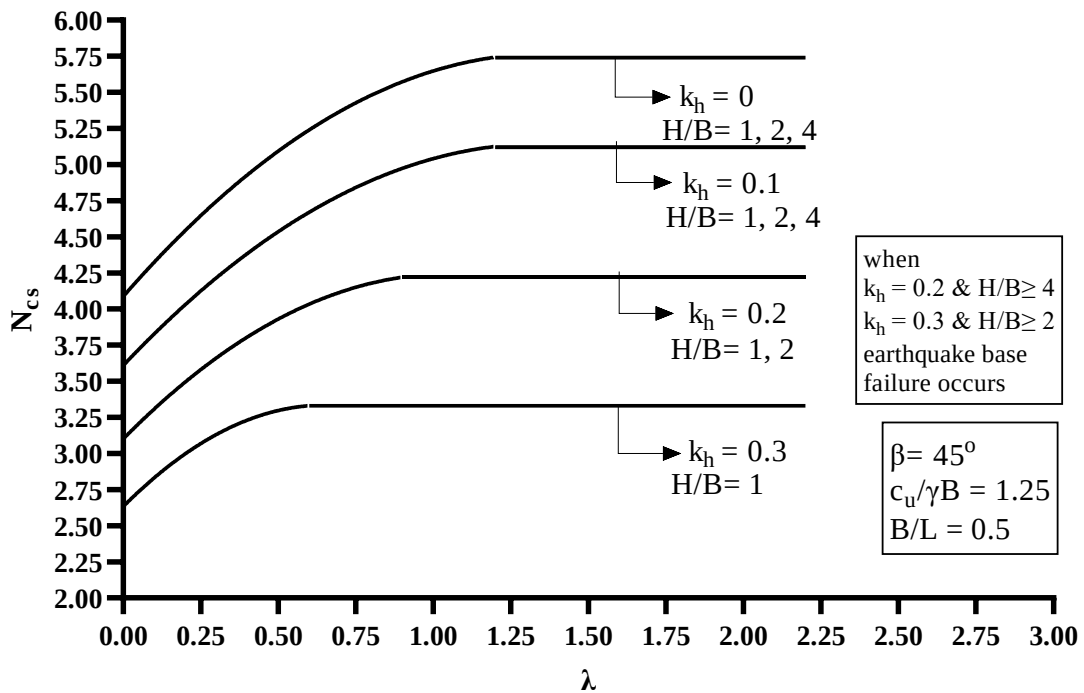


Figure 98: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 1.25$, $H/B = 1, 2, 4$, $B/L = 0.5$).

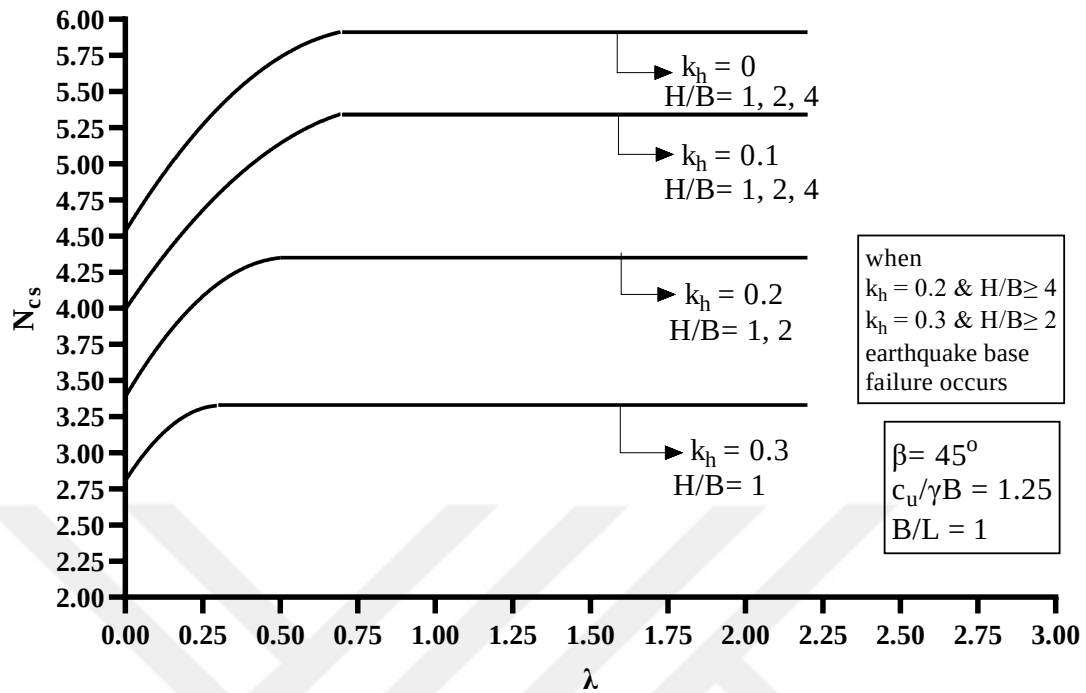


Figure 99: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 1.25$, $H/B = 1, 2, 4$, $B/L = 1$).

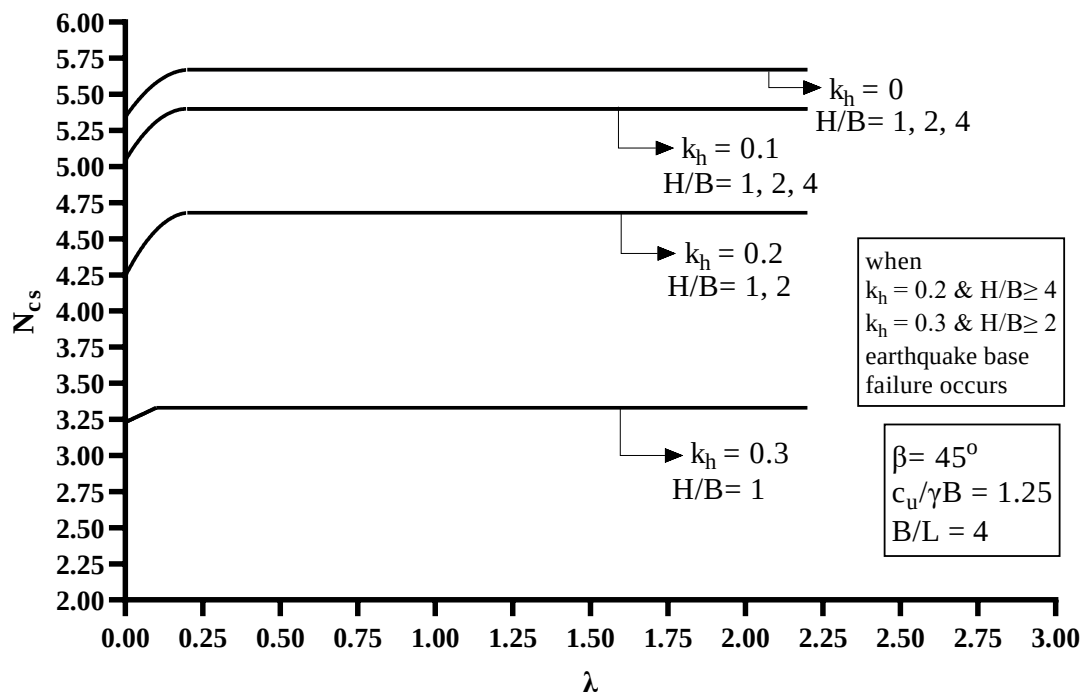


Figure 100: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 1.25$, $H/B = 1, 2, 4$, $B/L = 4$).

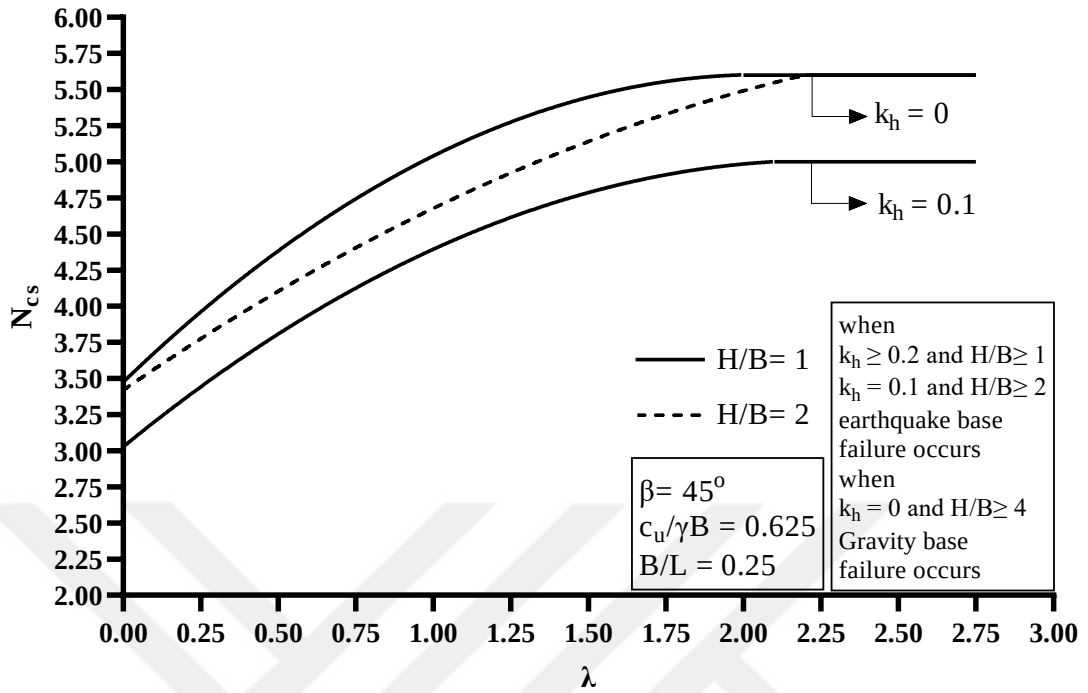


Figure 101: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 0.625$, $H/B = 1, 2$, $B/L = 0.25$).

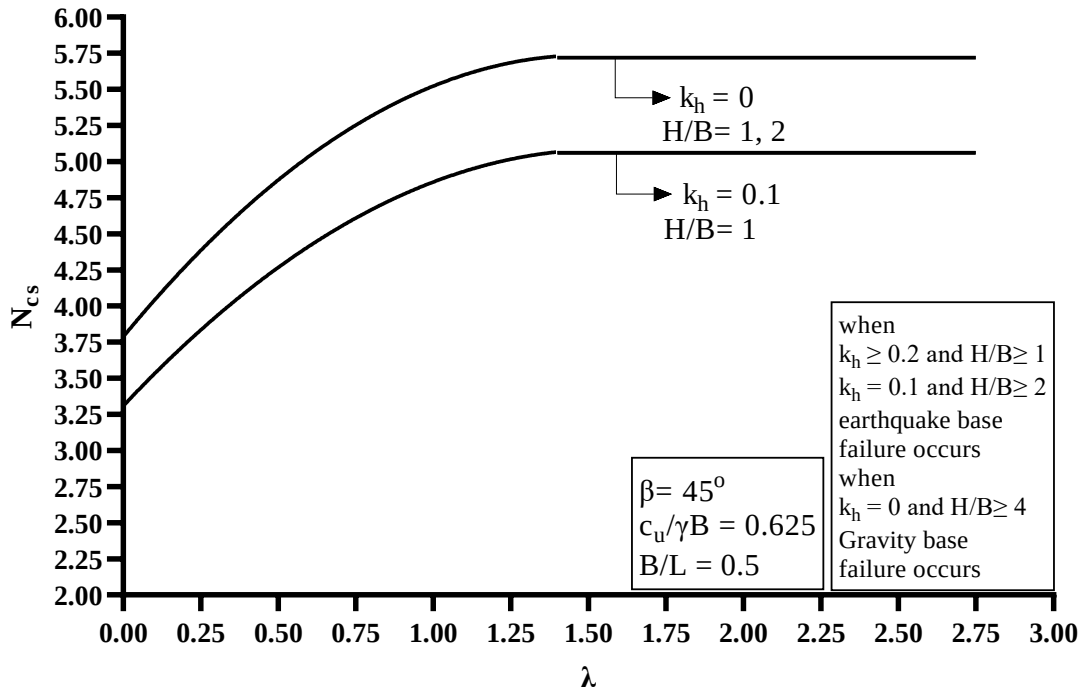


Figure 102: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 0.625$, $H/B = 1, 2$, $B/L = 0.5$).

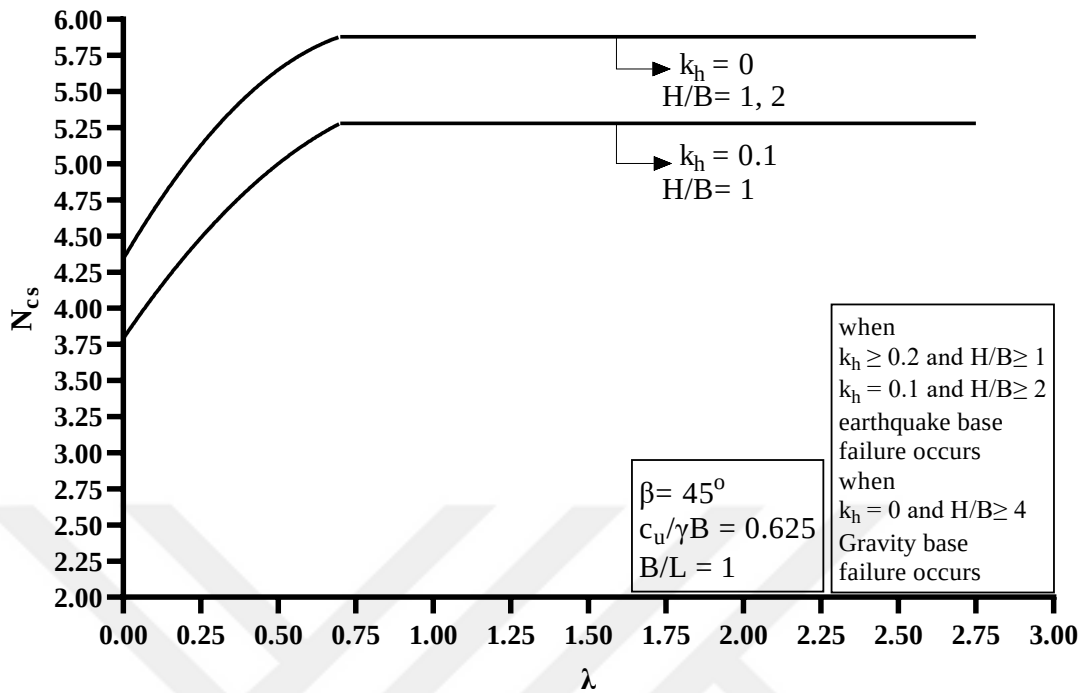


Figure 103: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 0.625$, $H/B = 1, 2$, $B/L = 1$).

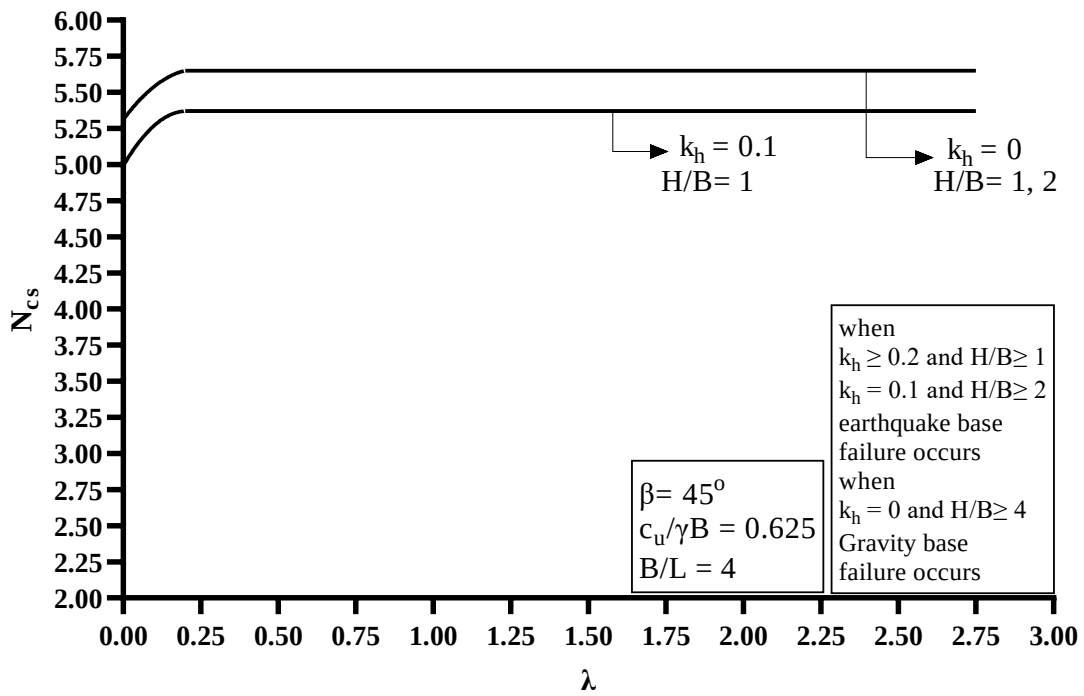


Figure 104: Design chart for N_{cs} of rectangular surficial footings near cohesive slopes ($\beta = 45^\circ$, $c_u/\gamma B = 0.625$, $H/B = 1, 2$, $B/L = 4$).

4.4 Regression Analysis

Regression analysis is a powerful statistical tool widely employed in various fields to explore relationships between variables and make predictions. It enables researchers to model the impact of multiple independent variables on a dependent variable, thus uncovering valuable insights into complex systems. In this thesis, regression analysis is to estimate the N_{cs} values, including the parameters and their variables utilized during the finite element analysis. There are several types of regression analysis, each tailored to suit the specific characteristics of the data and research objectives. Among them, multiple linear regression stands out as an appropriate choice for our investigation. This technique is ideally suited when the dependent variable is influenced by more than one independent variable, making it suitable for examining the intricate relationship between N_{cs} values and the influencing factors. Multiple linear regression is used to develop a mathematical model that estimates the N_{cs} values based on the interplay of several independent variables that used during the finite element analysis. By integrating these variables into the regression model, we can understand their combined influence on the N_{cs} and develop a predictive tool for future analyses. During the regression analysis, two crucial metrics is employed to evaluate the validity and significance of the regression model: R-squared (R^2) and p-values. R-squared measures the goodness of fit of the model, indicating the proportion of variability in the N_{cs} values that can be explained by the independent variables. A higher R-squared value signifies a stronger correlation between the variables and better predictive performance. On the other hand, p-values assess the statistical significance of each independent variable's contribution to the model. A lower p-value suggests that the variable has a significant impact on the N_{cs} values, whereas a higher p-value indicates that the variable may not

be statistically significant in predicting N_{cs} . To optimize the regression model and enhance its predictive accuracy, stepwise linear regression is employed. This automated variable selection method adds or removes independent variables based on specific criteria such as p-values or significance levels. Throughout this process, each variable's contribution to the model will be iteratively assessed, discarding those that fail to meet the significance threshold. As part of the stepwise regression process, the variable H/B was identified as insignificant, and therefore, it was removed from the final regression models. This approach ensures that the model includes only the most influential variables, thus mitigating the risk of overfitting and producing a more robust and reliable estimation of the N_{cs} values.

In this thesis, a comprehensive exploration of N_{cs} value estimation was undertaken through four distinct types of analysis. Each approach was meticulously crafted to shed light on the intricate relationships between variables, with varying degrees of complexity considered.

4.4.1 Regression Analysis without Interactions

The initial analysis focused on examining each variable independently, without incorporating any interactions between them. This straightforward approach allowed to understand the individual impact of each independent variable on the N_{cs} values, laying the foundation for further investigation. It can be observed that the parameter H/B is removed during the analysis due to its insignificance. The analysis resulted with five models, each model has different variables with their constant values and different R^2 values, while the fifth model has the highest R^2 value with 0.753. The fifth model is chosen to estimate N_{cs} and it's represented by Equation (34).

$$N_{cs} = 4.906 - 6.694k_h + 0.194 \frac{B}{L} + 0.356 \lambda - 0.314 \tan\beta + 0.041 \frac{c_u}{\gamma B} \quad (34)$$

Equation (34) was used to estimate some N_{cs} values to be compared with the actual values. As shown in Figure 105, the actual N_{cs} values are close to those calculated using Equation (34). However, there are still some points with high error, especially when λ is between 0.25 and 0.75.

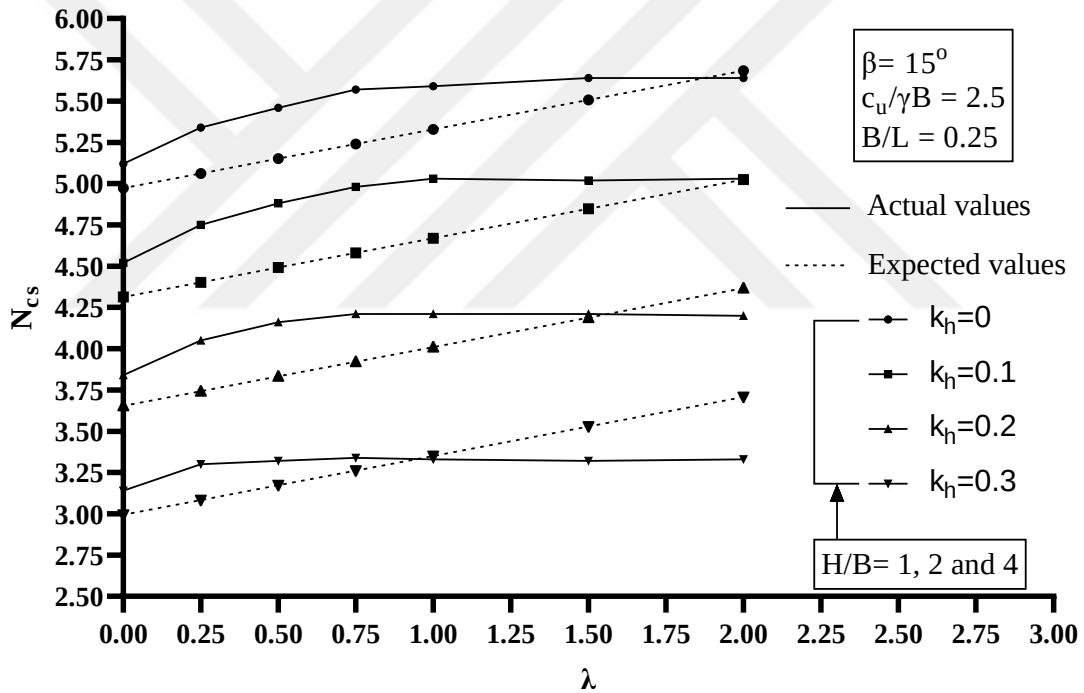


Figure 105: Comparison of the estimated and actual values of N_{cs} ($\beta = 15^\circ$, $c_u/\gamma B = 2.5$, $B/L = 0.25$) for regression analysis without interactions.

4.4.2 Regression Analysis with Interactions (Multiplied Variables)

Building upon the insights gained from the independent variable analysis, the second type of analysis delved into the realm of interactions between variables. By introducing

interaction terms obtained through multiplication, it's aimed to capture the combined effects of the independent variables on N_{cs} values. This more nuanced approach uncovered potential synergies or synergistic influences between variables. This analysis is resulted with seventeen models. The seventeenth model has the highest R^2 value of 0.811, but with 17 variables which is very complex. The eleventh model is chosen to estimate the N_{cs} as it has a close R^2 value of 0.807 to those of the seventeenth model with lower variable number of only 11. The eleventh model is represented by Equation (35).

$$\begin{aligned}
 N_{cs} = & 5.185 - 6.162k_h + 0.226 \frac{B}{L} + 0.351 \lambda \\
 & - 0.947 \tan\beta + 0.284 \lambda \tan\beta \\
 & - 0.098 \lambda \frac{B}{L} + 0.123 \frac{B}{L} \tan\beta \\
 & + 0.046 \frac{c_u}{\gamma B} \tan\beta - 0.968 \lambda k_h \\
 & - 0.442 k_h \frac{B}{L} + 0.999 k_h \tan\beta
 \end{aligned} \tag{35}$$

As shown in Figure 106, the model provides better estimation of N_{cs} values, for λ values between 0.25 and 1 when k_h is 0.3 and 0.2, while it estimates very far values for the rest k_h values.

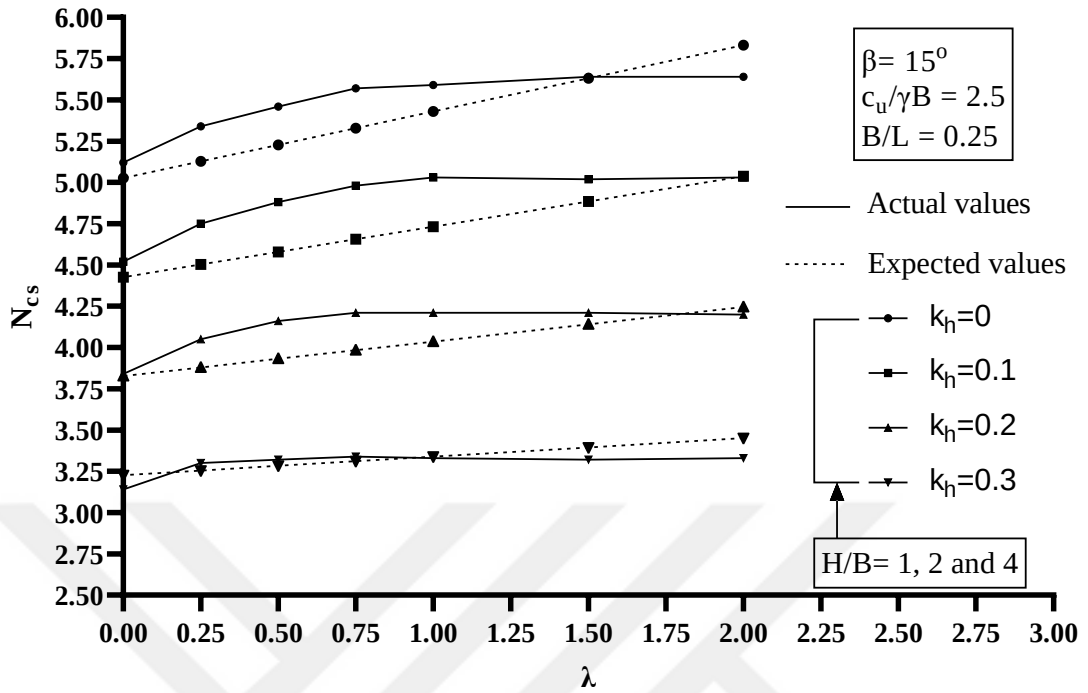


Figure 106: Comparison of the estimated and actual values of N_{cs} ($\beta = 15^\circ$, $c_u/\gamma B = 2.5$, $B/L = 0.25$) for regression analysis with interactions (Multiplied Variables).

4.4.3 Regression Analysis with Interactions (Squared Variables)

Recognizing the potential for nonlinear relationships, the third analysis included squared values of the independent variables. This expansion aimed to capture curvilinear or quadratic effects on the N_{cs} values, revealing nonlinear patterns that might have remained elusive in the previous analyses. This analysis is resulted with twenty-five models. The model number twenty-five has the highest R^2 value of 0.909, but with 25 variables which is very complex. The sixteenth model is chosen to estimate the N_{cs} as it has a close R^2 value of 0.905 to those of the model number twenty-five with lower variable number of only 16. The sixteenth model is represented by Equation (36).

$$\begin{aligned}
N_{cs} = & 4.781 - 0.350k_h + 1.727 \frac{B}{L} + 0.8 \lambda - 1.406 \left(\frac{B}{L}\right)^2 \\
& - 0.836 \tan\beta - 0.544 \lambda^2 + 0.265 \lambda \tan\beta \\
& - 0.370 k_h^2 - 0.189 \lambda \frac{B}{L} + 0.21 \frac{B}{L} \tan\beta \\
& + 0.105 \frac{c_u}{\gamma B} \tan\beta - 0.161 \lambda k_h \\
& - 0.124 \frac{B}{L} k_h + 0.143 k_h \tan\beta \\
& + 0.263 \tan^2\beta + 0.036 \frac{H}{B} \tan\beta
\end{aligned} \tag{36}$$

Figure 107 shows a comparison between the actual values and estimated values using Equation (36). The estimated values show high similarity with the actual values as this equation provides a nonlinear relationship which is more realistic to simulate the actual relationship. It can be concluded that the Squared Variables method provides more realistic relationship and more accurate values since it satisfies the non-linearity of the real relationship. However, the equation (36) which is generated by this model is quite complex.

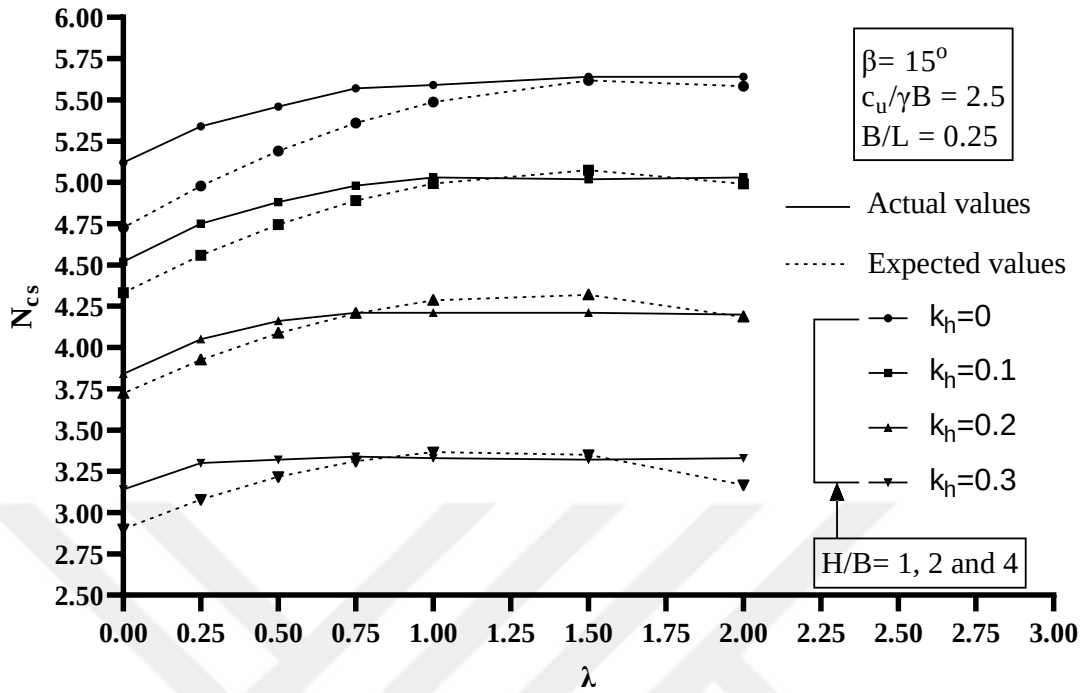


Figure 107: Comparison of the estimated and actual values of N_{cs} ($\beta = 15^\circ$, $c_u/\gamma B = 2.5$, $B/L = 0.25$) for regression analysis with interactions (Squared Variables).

4.5 Design Problems

This chapter will utilize the design charts produced in Chapter 4.3 to ascertain the N_{cs} value for various design problems. Additionally, Equation (36) will be employed to estimate the N_{cs} value. Subsequently, a comparison will be made between these calculated values and the actual values acquired through FEM analysis for the same problems.

Problem 1: For a footing with width $B = 4$ m and length $L = 8$ m located at 2 m distance to the crest of a slope with height $H = 8$ m and angle of slope of 30° . The soil of this slope is supposed to be cohesive homogenous with $c_u = 400$ kPa and $\gamma = 20$ kN/m³. The area has a seismic activity that can be represented with $k_h = 0.1$. Ignoring

the vertical seismic activity, find the ultimate seismic bearing capacity factor N_{cs} of this footing.

Solution: For this problem $H/B = 2$, $B/L = 0.5$, $\lambda = 0.5$ and $c_u/\gamma B = 5$. Using Figure 78 the N_{cs} value can be found as 4.91, while the value estimated by regression analysis using Equation (36) is 4.77.

The same problem was solved by FEM using Plaxis 3D and the resulted N_{cs} value is found as 4.94 with only %0.6 error compared to those calculated by using the design charts while it has %3.44 error compared to the value estimated using regression analysis.

Problem 2: For a footing with width $B = 3$ m and length $L = 3$ m located at 3 m distance to the crest of a slope with height $H = 12$ m and angle of slope of 20° . The soil of this slope is supposed to be cohesive homogenous with $c_u = 150$ kPa and $\gamma = 20$ kN/m³. The area has a seismic activity that can be represented by $k_h = 0.15$. Ignoring the vertical seismic activity, find the ultimate seismic bearing capacity factor N_{cs} of this footing.

Solution: For this problem $H/B = 4$, $B/L = 1$, $\lambda = 1$ and $c_u/\gamma B = 2.5$. Using Figure 78 the N_{cs} value can be found for $k_h = 0.1$ and 0.2 as, 5.38 and 4.41, respectively. By interpolation the N_{cs} for $k_h = 0.15$ is 4.895. The N_{cs} value estimated by regression analysis using Equation (36) is 5.15.

The same problem was solved by FEM using Plaxis 3D and the resulted N_{cs} value is found as 4.92 with only %0.5 error compared to those calculated by using the design charts while it has %4.67 error compared to the value estimated using regression analysis.

Problem 3: For a footing with width $B = 2$ m and length $L = 8$ m located at 1.5 m distance to the crest of a slope with height $H = 4$ m and angle of slope of 45° . The soil of this slope is supposed to be cohesive homogenous with $c_u = 25$ kPa and $\gamma = 20$ kN/m³. The area has no seismic activity risk. Find the ultimate seismic bearing capacity factor N_{cs} of this footing.

Solution: For this problem $H/B = 2$, $B/L = 0.25$, $\lambda = 0.75$ and $c_u/\gamma B = 0.625$. Using Figure 102 the N_{cs} value can be found as 4.402. The N_{cs} value estimated by regression analysis using Equation (36) is 4.68.

The same problem was solved by FEM using Plaxis 3D and the resulted N_{cs} value is found as 4.41 with only %0.18 error compared to those calculated by using the design charts while it has %6.12 error compared to the value estimated using regression analysis.

CHAPTER V

CONCLUSION

This thesis has explored the pivotal relationship between footing shape and the undrained seismic bearing capacity of foundations near cohesive slopes, employing comprehensive 3D Finite Element Method (FEM) analyses. The research has been enriched by the development of practical design charts that provide a simplified equation for estimating the undrained ultimate seismic bearing capacity.

The investigation into the influence of footing shape on the bearing capacity factor, N_{cs} , has yielded insightful results. In general, it is observed that as the shape of the footing transitions from a strip foundation to a square shape, N_{cs} values progressively increase until a critical point is reached. Beyond this peak, the N_{cs} values begin to decrease as the footing shape approaches that of a strip foundation once more.

Furthermore, the study found that increasing the horizontal seismic acceleration, k_h , consistently reduces the seismic bearing capacity factor. The impact of increasing the slope angle on the seismic bearing capacity factor is contingent on whether the failure is influenced by the slope. When the foundation is located at a sufficient distance from the slope, changes in the slope angle have no significant effect on the seismic bearing capacity factor. Generally, alterations in slope height do not significantly affect the seismic bearing capacity factor, except in cases where soil strength is exceptionally low, leading to a transition from bearing capacity failure to overall slope failure.

The study also noted that changes in the soil strength, c_u , do not directly affect the seismic bearing capacity factor, but exceedingly low soil strength values can lead to model failure. Several failure mechanisms were observed throughout this research, including bearing capacity failure, gravity-induced failure, seismic inertia-induced failure, and sliding failure.

Moreover, through rigorous regression analysis, a formula has been developed to estimate the N_{cs} value with a high R-squared value of 0.9, providing a valuable tool for practical application. This research contributes to understanding of the complex relationship between footing shape, slope conditions, and seismic bearing capacity, offering important insights for the field of geotechnical engineering and foundation design.

Appendix-I: Stability check of cohesive slopes under static conditions using Taylor's method.

Table I 1. H_{cr} value and stability check using Taylor's method.

H (m)	D	β (°)	m	γ (kN/m ³)	c_u (kPa)	H_{cr} (m)	Stability Check
2	7	15	0.182	20	200	54.9	Stable
2	7	30	0.182	20	200	54.9	Stable
2	7	45	0.182	20	200	54.9	Stable
2	7	60	0.1916	20	200	52.20	Stable
2	7	15	0.182	20	100	27.5	Stable
2	7	30	0.182	20	100	27.5	Stable
2	7	45	0.182	20	100	27.5	Stable
2	7	60	0.1916	20	100	26.10	Stable
2	7	15	0.182	20	50	13.7	Stable
2	7	30	0.182	20	50	13.7	Stable
2	7	45	0.182	20	50	13.7	Stable
2	7	60	0.1916	20	50	13.05	Stable
2	7	15	0.182	20	25	6.9	Stable
2	7	30	0.182	20	25	6.9	Stable
2	7	45	0.182	20	25	6.9	Stable
2	7	60	0.1916	20	25	6.53	Stable
4	4	15	0.176	20	200	56.8	Stable
4	4	30	0.18	20	200	55.6	Stable
4	4	45	0.18	20	200	55.6	Stable
4	4	60	0.1916	20	200	52.20	Stable
4	4	15	0.176	20	100	28.4	Stable
4	4	30	0.18	20	100	27.8	Stable
4	4	45	0.18	20	100	27.8	Stable
4	4	60	0.1916	20	100	26.10	Stable
4	4	15	0.176	20	50	14.2	Stable
4	4	30	0.18	20	50	13.9	Stable
4	4	45	0.18	20	50	13.9	Stable
4	4	60	0.1916	20	50	13.05	Stable
4	4	15	0.176	20	25	7.1	Stable
4	4	30	0.18	20	25	6.9	Stable
4	4	45	0.18	20	25	6.9	Stable
4	4	60	0.1916	20	25	6.53	Stable

8	2.5	15	0.154	20	200	64.9	Stable
8	2.5	30	0.175	20	200	57.1	Stable
8	2.5	45	0.18	20	200	55.6	Stable
8	2.5	60	0.1916	20	200	52.20	Stable
8	2.5	15	0.154	20	100	32.5	Stable
8	2.5	30	0.175	20	100	28.6	Stable
8	2.5	45	0.18	20	100	27.8	Stable
8	2.5	60	0.1916	20	100	26.10	Stable
8	2.5	15	0.154	20	50	16.2	Stable
8	2.5	30	0.175	20	50	14.3	Stable
8	2.5	45	0.18	20	50	13.9	Stable
8	2.5	60	0.1916	20	50	13.05	Stable
8	2.5	15	0.154	20	25	8.1	Unstable
8	2.5	30	0.175	20	25	7.1	Unstable
8	2.5	45	0.18	20	25	6.9	Unstable
8	2.5	60	0.1916	20	25	6.53	Unstable

Appendix II: Stability check of cohesive slopes under seismic conditions using Koppula's method.

Table II 1. H_{cr} value and stability check using Koppula's method.

H (m)	D	β (°)	k_h	N2	γ (kN/m ³)	c_u (kPa)	H_{cr} (m)	Stability Check
2	7	30	0.1	3.24	20	200	32.4	Stable
2	7	30	0.2	2.11	20	200	21.1	Stable
2	7	30	0.3	1.49	20	200	14.9	Stable
2	7	30	0.1	3.24	20	100	16.2	Stable
2	7	30	0.2	2.11	20	100	10.6	Stable
2	7	30	0.3	1.49	20	100	7.5	Stable
2	7	30	0.1	3.24	20	50	8.1	Stable
2	7	30	0.2	2.11	20	50	5.3	Stable
2	7	30	0.3	1.49	20	50	3.7	Stable
2	7	30	0.1	3.24	20	25	4.1	Stable
2	7	30	0.2	2.11	20	25	2.6	Stable
2	7	30	0.3	1.49	20	25	1.9	Unstable
2	7	45	0.1	2.73	20	200	27.3	Stable
2	7	45	0.2	1.68	20	200	16.8	Stable
2	7	45	0.3	1.15	20	200	11.5	Stable
2	7	45	0.1	2.73	20	100	13.7	Stable
2	7	45	0.2	1.68	20	100	8.4	Stable
2	7	45	0.3	1.15	20	100	5.8	Stable
2	7	45	0.1	2.73	20	50	6.8	Stable
2	7	45	0.2	1.68	20	50	4.2	Stable
2	7	45	0.3	1.15	20	50	2.9	Stable
2	7	45	0.1	2.73	20	25	3.4	Stable
2	7	45	0.2	1.68	20	25	2.1	Stable
2	7	45	0.3	1.15	20	25	1.4	Unstable
2	7	60	0.1	4.583	20	200	45.8	Stable
2	7	60	0.2	3.975	20	200	39.8	Stable
2	7	60	0.3	3.278	20	200	32.8	Stable
2	7	60	0.1	4.583	20	100	22.9	Stable
2	7	60	0.2	3.975	20	100	19.9	Stable
2	7	60	0.3	3.278	20	100	16.4	Stable
2	7	60	0.1	4.583	20	50	11.5	Stable
2	7	60	0.2	3.975	20	50	9.9	Stable

2	7	60	0.3	3.278	20	50	8.2	Stable
2	7	60	0.1	4.583	20	25	5.7	Stable
2	7	60	0.2	3.975	20	25	5.0	Stable
2	7	60	0.3	3.278	20	25	4.1	Stable
4	4	30	0.1	3.24	20	200	32.4	Stable
4	4	30	0.2	2.11	20	200	21.1	Stable
4	4	30	0.3	1.49	20	200	14.9	Stable
4	4	30	0.1	3.24	20	100	16.2	Stable
4	4	30	0.2	2.11	20	100	10.6	Stable
4	4	30	0.3	1.49	20	100	7.5	Stable
4	4	30	0.1	3.24	20	50	8.1	Stable
4	4	30	0.2	2.11	20	50	5.3	Stable
4	4	30	0.3	1.49	20	50	3.7	Unstable
4	4	30	0.1	3.24	20	25	4.0	Unstable
4	4	30	0.2	2.11	20	25	2.6	Unstable
4	4	30	0.3	1.49	20	25	1.9	Unstable
4	4	45	0.1	2.73	20	200	27.3	Stable
4	4	45	0.2	1.68	20	200	16.8	Stable
4	4	45	0.3	1.15	20	200	11.5	Stable
4	4	45	0.1	2.73	20	100	13.7	Stable
4	4	45	0.2	1.68	20	100	8.4	Stable
4	4	45	0.3	1.15	20	100	5.8	Stable
4	4	45	0.1	2.73	20	50	6.8	Stable
4	4	45	0.2	1.68	20	50	4.2	Stable
4	4	45	0.3	1.15	20	50	2.9	Unstable
4	4	45	0.1	2.73	20	25	3.4	Unstable
4	4	45	0.2	1.68	20	25	2.1	Unstable
4	4	45	0.3	1.15	20	25	1.4	Unstable
4	4	60	0.1	4.583	20	200	45.8	Stable
4	4	60	0.2	3.975	20	200	39.8	Stable
4	4	60	0.3	3.278	20	200	32.8	Stable
4	4	60	0.1	4.583	20	100	22.9	Stable
4	4	60	0.2	3.975	20	100	19.9	Stable
4	4	60	0.3	3.278	20	100	16.4	Stable
4	4	60	0.1	4.583	20	50	11.5	Stable
4	4	60	0.2	3.975	20	50	9.9	Stable
4	4	60	0.3	3.278	20	50	8.2	Stable
4	4	60	0.1	4.583	20	25	5.7	Stable
4	4	60	0.2	3.975	20	25	5.0	Stable
4	4	60	0.3	3.278	20	25	4.1	Stable
8	2.5	30	0.1	4.12	20	200	41.2	Stable

8	2.5	30	0.2	3.16	20	200	31.6	Stable
8	2.5	30	0.3	2.59	20	200	25.9	Stable
8	2.5	30	0.1	4.12	20	100	20.6	Stable
8	2.5	30	0.2	3.16	20	100	15.8	Stable
8	2.5	30	0.3	2.59	20	100	13.0	Stable
8	2.5	30	0.1	4.12	20	50	10.3	Stable
8	2.5	30	0.2	3.16	20	50	7.9	Unstable
8	2.5	30	0.3	2.59	20	50	6.5	Unstable
8	2.5	30	0.1	4.12	20	25	5.2	Unstable
8	2.5	30	0.2	3.16	20	25	4.0	Unstable
8	2.5	30	0.3	2.59	20	25	3.2	Unstable
8	2.5	45	0.1	3.58	20	200	35.8	Stable
8	2.5	45	0.2	2.51	20	200	25.1	Stable
8	2.5	45	0.3	1.97	20	200	19.7	Stable
8	2.5	45	0.1	3.58	20	100	17.9	Stable
8	2.5	45	0.2	2.51	20	100	12.6	Stable
8	2.5	45	0.3	1.97	20	100	9.9	Stable
8	2.5	45	0.1	3.58	20	50	9.0	Stable
8	2.5	45	0.2	2.51	20	50	6.3	Unstable
8	2.5	45	0.3	1.97	20	50	4.9	Unstable
8	2.5	45	0.1	3.58	20	25	4.5	Unstable
8	2.5	45	0.2	2.51	20	25	3.1	Unstable
8	2.5	45	0.3	1.97	20	25	2.5	Unstable
8	2.5	60	0.1	4.583	20	200	45.8	Stable
8	2.5	60	0.2	3.975	20	200	39.8	Stable
8	2.5	60	0.3	3.278	20	200	32.8	Stable
8	2.5	60	0.1	4.583	20	100	22.9	Stable
8	2.5	60	0.2	3.975	20	100	19.9	Stable
8	2.5	60	0.3	3.278	20	100	16.4	Stable
8	2.5	60	0.1	4.583	20	50	11.5	Stable
8	2.5	60	0.2	3.975	20	50	9.9	Stable
8	2.5	60	0.3	3.278	20	50	8.2	Stable
8	2.5	60	0.1	4.583	20	25	5.7	Unstable
8	2.5	60	0.2	3.975	20	25	5.0	Unstable
8	2.5	60	0.3	3.278	20	25	4.1	Unstable

Appendix III: FEM Results of Level Ground Case under Static and Seismic Conditions

The result of this part is available on the following link: [here](#).



Appendix IV: FEM Results of Slope Case under Static and Seismic Conditions

The result of this part is available on the following link: [here](#).



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