STUDY ON THE PERFORMANCE OF PAD FOOTINGS ON SOFT SOILS IMPROVED WITH CONCRETE CYLINDERS AND CONTROLLED MODULUS COLUMNS (CMC)

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Abstract

In construction projects on soft soils, various methods for structural enhancement and ground improvement, such as micro-piles, concrete cylinders with filling materials, larger footings, and Controlled Modulus Columns (CMCs), are employed. However, there remains a limited understanding of the effectiveness of using concrete cylinders with filling materials. Despite increasing adoption in Europe, the performance of CMCS in building applications continues to be less studied.

This research investigated the structural and geotechnical performance of concrete cylinders with quarry dust filling and the CMC technique through three-dimensional (3D) finite element (FE) modelling using the MIDAS GTS-NX software. Soil layers are simulated, incorporating both Modified Cam Clay (MCC) and Mohr-Coulomb constitutive soil models, while all the structural elements were modelled using the elastic material model to examine both immediate and consolidation settlements. Validation of the models was carried out through a theoretical comparison, using field test data of Texas A&M University and Green Cove Springs, and through lab model tests conducted at the University of Moratuwa.

A comprehensive parametric analysis was carried out to assess the impacts of embedment depth of the footing, diameter, height, spacing, and the LTP thickness of the CMC on foundation settlement behaviour. Additionally, the impacts of concrete cylinders with the quarry dust fill were studied for various heights and diameters of the cylinders. The results showed that both methods significantly reduce both immediate and consolidation settlements, with some optimal design parameters.

This study provides a comprehensive understanding of the performance of these two techniques as an economic improvement for domestic construction on soft soils. Furthermore, the numerical models used in this research could serve as a foundation for further studies on various ground improvement techniques and will provide insights into the feasibility of these methods, supporting an economically and structurally sound foundation on soft soils.

Keywords: Footing, Consolidation Settlements, Bearing Capacity, Controlled Modulus Columns (CMCs), Concrete Cylinders, Finite Element Analysis, Soft Clay, Modified Cam Clay, Ground Improvement, Midas GTS NX

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Declaration

I declare that this is my own work and this thesis/dissertation does not incorporate without acknowledgement any material previously submitted for a degree or diploma in any other University or Institute of higher learning, and to the best of my knowledge and belief, it does not contain any material previously published or written by another person except where the acknowledgement is made in the text. I retain the right to use this content in whole or in part in future works (such as articles or books).

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The above candidate has carried out research for the Masters thesis under my supervision. I confirm that the declaration made above by the student is true and correct.

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CHAPTER 1: INTRODUCTION

1.1. Background

One of the most essential parts of a structure is its foundation. It is the part that spreads the weight of the structure over a large area of soil, ensuring that settlement remains within acceptable limits and does not exceed the soil's ultimate bearing capacity. The solid ground beneath the foundation is called the foundation bed.

Soil comes in different types, each with its own bearing capacity. When selecting a foundation, engineers consider the soil profile, as well as the size and weight of the structure. Foundations are classified into two types: shallow and deep. Depending on how deep the soil is, foundations are categorised as either shallow or deep. Pad footing is a type of shallow foundation discussed in this research.

Individual pad footings (Figure 1-1) or combined footings (Figure 1-2) are commonly used in domestic buildings. Cost-effective ground improvement techniques have become increasingly important in the construction industry, as most constructions now occur on weaker soils due to urbanisation. Innovative alternatives for foundation solutions are in higher demand because traditional methods (preloading, wick drains, deep piles) lack economic sustainability or are more time-consuming.

Figure 1-1: Individual Pad Footing

Figure 1-2: Combined Footings

The concept of Controlled Modulus Columns (CMC) was originally considered as an alternative to stone columns for ground improvement. According to K Coghlan et al. (2016), the first CMC project was undertaken in 1996 for the Amein Football Stadium project in Northern France and is now used across Europe, North America, and Asia. CMC is founded on a load-sharing system between the soil and the column. (Figure 1-

3). This method is used to bridge the gap between costly pile solutions and traditional soil improvement methods. The concrete caissons filled with quarry dust (Figure 1-4) represent another rigid inclusion solution. These caissons can be filled with either plain concrete made from waste materials or crushed concrete. They act like larger column inclusions or piles to strengthen the soil while redistributing the load to deeper layers. However, these methods have not been as extensively studied as other solutions, especially regarding long-term performance.

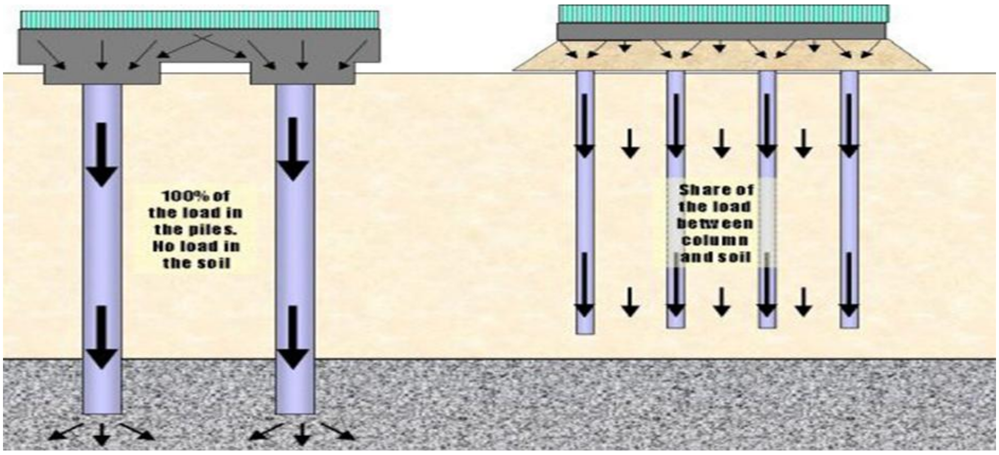


Figure 1-3: Load-sharing mechanism of CMC

(https://www.slideshare.net/slideshow/a-presentation-of-the-main-ground-improvement-techniques/250299018)

Figure 1-4: Cylinders filled with quarry dust

This study will assess the structural adequacy of these methods using the MIDAS Finite Element (FE) analysis software: Midas GTS NX. As a result, it will pinpoint key design parameters and critical factors for crucial structural enhancements and ground improvements, particularly when building structures on weaker soils.

1.2. Problem statement

Foundations on weaker clay soils need larger footings, or ground improvement methods like micro-piles, concrete cylinders, or Controlled Modulus Columns (CMCs). However, there is limited knowledge about how effective these methods are in residential building construction. This research aims to assess the geotechnical and structural performance of CMC and quarry dust-filled concrete cylinders as improvement techniques through numerical modelling using MIDAS FE analysis software: MIDAS GTS-NX.

1.3. Aim and objectives

This research aims to evaluate and enhance design practices for constructing domestic buildings on weaker compressible soils by analysing column footings with CMCs and quarry dust-filled cylinders as strength improvement techniques.

The research will pursue the following objectives:

1. Identify current design practices employed in construction on weaker soils.
2. Develop comprehensive Finite Element (FE) models utilising MIDAS software for the analysis of column footings.
3. Conduct a parametric study on column footings with selected ground enhancement techniques like CMCs and quarry dust-filled cylinders.
4. Compare the structural performance and feasibility of these techniques.

1.4. Significance of the research

This research will offer insights into the structural adequacy and feasibility of various ground improvement and enhancement techniques, addressing the research gap, particularly regarding consolidation settlements on weaker soils. Additionally, it will inform design decisions for necessary structural enhancements in domestic building construction on weak soils and contribute to safer, more economical construction practices through improved understanding of strength enhancement methods.

1.5. Outline of the thesis

This thesis provides an analysis and results from a FE modelling study on the structural performance of pad footings with different ground improvements.

This thesis is structured as follows.

- Chapter 1: Overview of the research, covering the background, problem statement, goals, objectives, and importance of the study.
- Chapter 2: Comprehensive literature review
- Chapter 3: Research methodology detailing the FE modelling approaches, their validation, and analysis.
- Chapter 4: Results of FE analyses carried out throughout the study
- Chapter 5: Conclusion of the research, including a summary and recommendations.

CHAPTER 2: LITERATURE REVIEW

2.1. Introduction

This chapter offers a thorough review of existing ground improvement methods, concentrating on Controlled Modulus Columns (CMC) and concrete caissons filled with quarry dust as techniques, as well as studies using Finite Element (FE) modelling on weaker soils. The review includes findings from several research papers, highlighting key research gaps. Key insights emphasise the importance and practicality of FE analysis in modelling soil-structure interactions and assessing the suitability of these improvement methods. It also identifies significant gaps regarding the long-term performance of cylinders and CMCs, while underlining the need for improvements in design procedures.

2.2. Recommended foundation types based on soil type

Soil types are crucial factors to consider when selecting the foundation type for a specific construction project. Table 2-1 provides recommended foundation types based on soil type (Marwa Hasan, 2022).

Table 2-1: Foundation design and soil type

Soil Type	**Recommended Foundation Type**
Rocky soil	Strap foundations
Gravel and sand	For high bearing capacity, footing or strip foundations are recommended. For low bearing capacity, combined footing or wide strip foundations are suggested.
Hard Clay	Raft foundation
Soft clay	Footing and strap foundations are Unsuitable. Pile foundations are recommended.
Peat	After excavating a layer of peat from the ground, if stable soil is found, use strap foundations. If no stable soil is found, use raft foundations.

Soft clay presents the most significant challenges when constructing buildings, making pad footings and strap footings unsuitable; therefore, pile foundations are the recommended solution. However, pile foundations in soft soils commonly lead to more costly and time-consuming processes. This creates more demand for alternatives that are more economical and less time-consuming.

2.3. Micro piles

Micro-piles are smaller-diameter grouted steel piles that are typically used to increase bearing capacity, mainly in the compression zones of soil. They are installed into the ground in groups at various angles (Figures 2-1 and 2-2) to reinforce the soil mass. Micro-piles function as distributed piles but still need pile caps and concrete reinforcement. Furthermore, micro-piles can be designed to resist tension and minor bending. Numerical simulations (Murthy et al., 2002) have demonstrated that the bearing capacity of a foundation can be greatly enhanced by installing the micro-pile close to the foundation. "Babu et al., 2004" carried out a study to evaluate the effectiveness of micro-piles in resisting additional structural loads. They also performed a comprehensive FE analysis, which confirmed that the proposed treatment matched field data and validated the FE models.

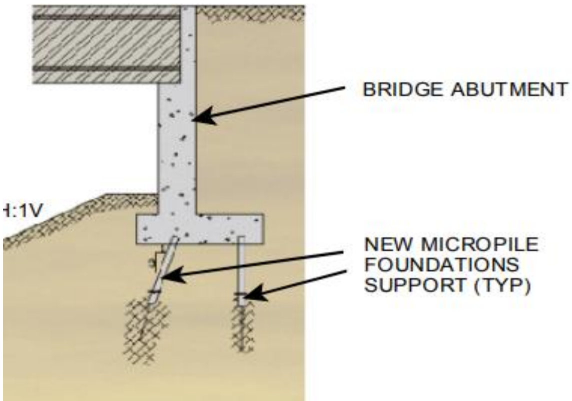


Figure 2-1: Foundation support

https://m.blog.naver.com/pilestory/220973297709

Figure 2-2: Micro pile group

https://www.machinesl.com/micropile-foundation/

Analysis was carried out using PLAXIS FE software, with all nodes at the bottom of the mesh fixed both horizontally and vertically, and all nodes on either side restricted from horizontal movement. These are the assigned boundary conditions. Mohr-Coulomb material was employed to simulate the foundation soil, while elastic material was used for the micro-pile.

The material properties and dimensions used in the case study are as follows.

- 2-story building on loose sandy soil
- STP values in the range of 6 to 8
- $c' = 0$ and $\varphi = 25°$
- Bulk density (γ_b) 17 kN/m^3
- 600 kN Load
- The allowable bearing capacity is 120 kN/m².
- 2.5×2.0 m^2 Column footings

Converting 3D problems with regularly spaced piles into 2D problems requires averaging the effects in 3D over the distance between the piles. According to Donovan et al., (1984) linear scaling of material qualities is a simple and effective way to evenly distribute the discrete effects of elements throughout the space between them. Figure 2-3, shown below, illustrates the FE model used in the analysis, along with the corresponding boundary conditions. Table 2-2 shows the material properties used in the study.

Table 2-2: Material properties used

Material	Model	Cohesion c' (kPa)	ϕ' (°)	E (MPa)	υ	γ_b (kN/m^3)
Loose Sand	Mohr-Coulomb	1	25	13	0.3	18
Dense Sand	Mohr-Coulomb	1	35	30	0.3	20
Micro pile	elastic	-	-	2.1×10^5	0.25	78.5

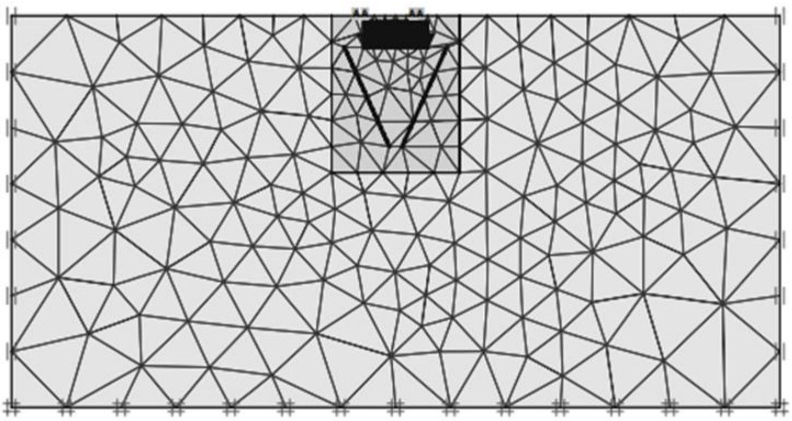


Figure 2-3: FE model

Figure 2-4 displays the results from numerical simulations. Curve 1 illustrates the load-displacement behaviour of the soil at the site. The load-displacement curve when micro-piles are used is represented by curve 2. The footing with micro-piles results in less settlement for a given soil bearing capacity, while greatly improving the overall bearing capacity.

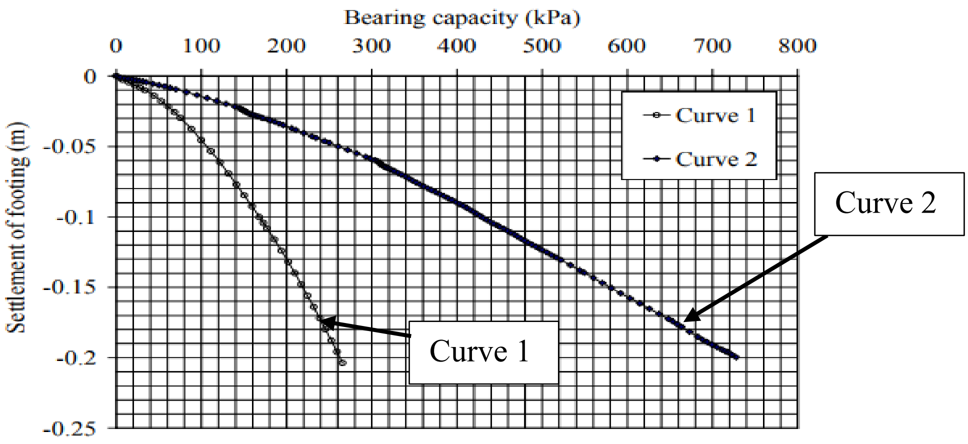


Figure 2-4: Load versus Settlement Curve

Mollaali et al., (2014) also performed a numerical modelling and analysis (using PLAXIS) to observe the improvement of soil layers after the use of micro-piles. Properties of the soil layers before and after improvement are presented in Table 2-3 and Table 2-4, respectively.

Table 2-3: Soil Properties

Layer	E (kg/cm^2)	ϕ°	ψ°	C (kg/cm^2)	γ (gr/cm^3)
Weak	100	28	-	0	1.8
Medium	200	28	-	0	1.8
Clayey Gravel	375	33	3	0.15	2

Table 2-4 : Improved Soil Properties

Layer	E (kg/cm^2)	ϕ°	ψ°	C (kg/cm^2)	γ (gr/cm^3)
Improved Weak Layer	1290	28	-	0.1	1.8
Improved Medium Layer	1390	28	-	0.1	1.8
Improved Clayey Gravel	1564	33	3	0.256	2

Analysis of the numerical data was carried out for a site with three distinct soil layers, each with an elastic modulus of 100 kg/cm², 200 kg/cm², and 375 kg/cm². The variations of settlement versus pressure for different soil fills are shown in Figures 2-5, 2-6, and 2-7.

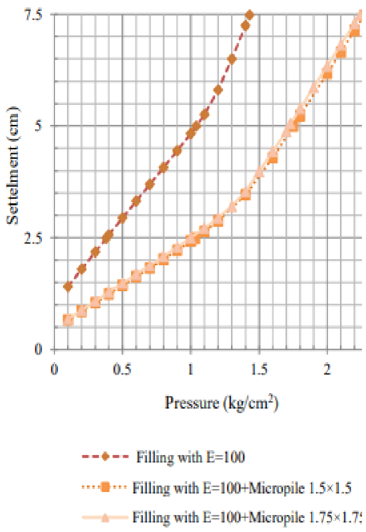


Figure 2-5: 100 kg/cm^2

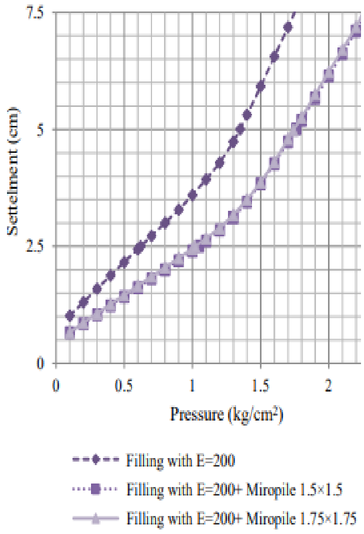


Figure 2-6: 200 kg/cm^2

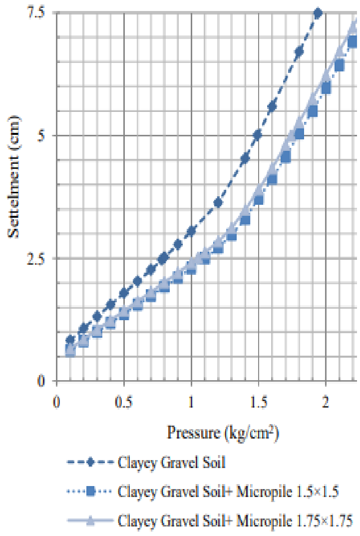


Figure 2-7: 375 kg/cm^2

Figure 2-8 shows the improvement of soil bearing capacity for different subgrade models after installing micro piles. The findings of this study indicate that applying micro-piles to enhance soil bearing capacity is much more effective for weaker soil layers than for stiff soil layers.

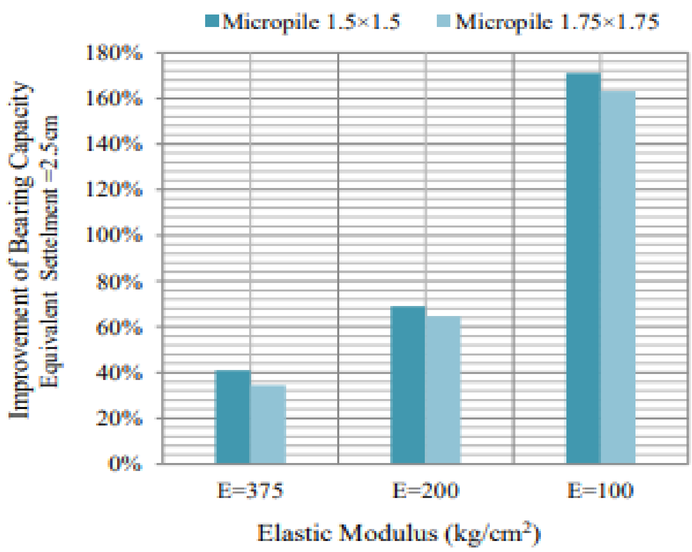


Figure 2-8: Impact on bearing capacity

2.4. Stone (granular) columns

Stone columns are a traditional method for ground improvement. Construction involves creating holes using vibro-float equipment, which consists of 2-3 m long, 0.2-0.5 m wide tubes fitted with specialised jets to excavate the soil. During this process, gravel or crushed stone (6-40 mm) is filled and compacted into boreholes to form stiff columns. These columns support most of the footing's load, and the lateral confinement they provide reduces settlement in the soil. They are widely used for embankments, tanks, and foundations on soft ground. Stone columns are particularly effective for stabilising soft clay by allowing vertical drainage and soil reinforcement. (*Ground-Improvement-Techniques-Soil-Stabilization*, n.d.)

"(Grizi et al., 2022)" carried out a numerical analysis on the field trial test conducted at the Bothkennar Site (Serridge, n.d.) using Plaxis 3D software**.** They explored key design factors crucial to the settlement performance of narrow footings with small groups of stone columns. These factors included column spacing, diameter, and length, as well as footing shape, column strength, and the effect of a crust layer. The conclusions from the study are as follows.

- Settlements show negligible values at 2-3 times the footing length. (Figure 2-9)
- Column length will improve both settlement and bearing capacity performances up to the critical length, and beyond that, it will only improve settlement performance.
- Increasing the column diameter and strength will result in a reduction in settlements.
- The presence of a robust crust layer exerts a negligible effect on settlement performance under low load conditions.

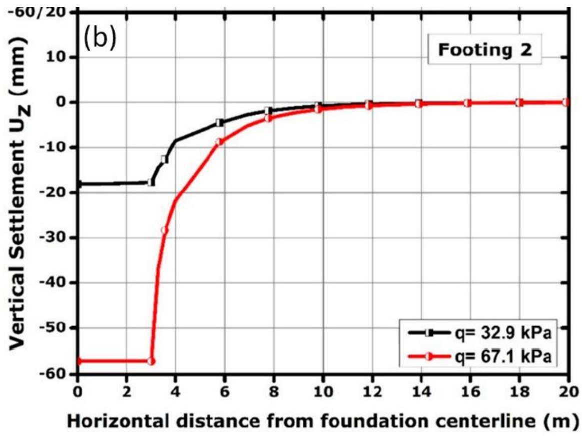


Figure 2-9: Settlement with the distance from the footing centre

2.5. Controlled modulus columns (CMC)

Controlled Modulus Columns (CMCs) are made with plain concrete without steel rebars, which are mostly constructed with the use of a large diameter auger on site. While the auger is removing, concrete grout with high slump is injected to form a stiff column. (Nguyen et al., n.d.) CMCs are installed in groups with a diameter of 0.2 m - 0.5 m in the clay soils. Unlike deep piling solutions, CMCs share the footing load with the surrounding soil, forming a system to bear the load where about 50-80% of the load is taken by CMCs and the rest is from the surrounding soil. This load-sharing system is created by a layer of compacted aggregate or stiff soil, called the "load transfer platform" (LTP), which is placed on top of the columns (ASIRI National Project, 2013). There is a rule in the industry that LTP thickness should satisfy equation 1 given below.

$$H_M > 1.5(s - a) \tag{1}$$

Here, "H_M" is the thickness of the LTP layer, "s" is the centre-to-centre spacing between two CMCs, and "a" is the diameter of the CMC cap as displayed in Figure 2-10.

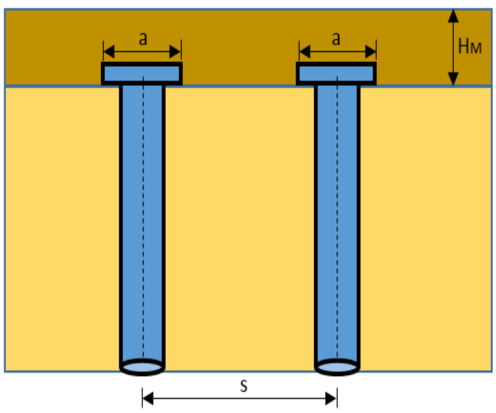


Figure 2-10: CMC with LTP

CMCs do not need a rigid connection to the footing (pile cap), unlike traditional piles. This helps in using very thin footings, leading to a significant reduction in the usage of concrete and steel. Additionally, the installation process of the CMCs is a soil displacement process, which creates minimal spoil, unlike bored piles (Frederic Masse, 2017). These advantages and the convenience in construction make CMC an economical alternative to deep piles for soft soils. Since the columns are made with controlled-strength concrete, it prevents weak soil from being overstressed (Nguyen et al., n.d.). CMCs work efficiently under uniformly distributed loads and shallow foundations, which makes them unsuitable for use in high-rise buildings and situations with overturning moments. Additionally, a lack of a proper LTP layer will lead to long-term settlements and differential settlements. Research shows the need for a better understanding of consolidation settlement, lateral deformation, and optimal LTP stiffness in CMC design.

According to Nguyen et al. (n.d.), The results of 3D FLAC3D simulations showed that driving in a new column near the old one can cause lateral soil heave on its neighbours. Additionally, they found that the sequence of installation also matters. These studies indicate that CMCs must be installed carefully.

2.6. Partial soil replacement

Traditionally, improving soil quality has meant replacing poor soil with better material. However, the high costs and environmental constraints of mining and reclamation mean that restoring an entire weak soil zone is not feasible or practical. Therefore, considering these factors, a more reasonable strategy is to selectively replace soil in targeted areas. Such a selective replenishment concept, featuring a granular trench beneath the footing, was developed by Madhav A N & Vitkar, (1978)

Fattah et al. (2010) applied a FE method to simulate the behaviour of reinforced granular trench soil and soft cohesive soil using the SIGMA/W computer programme. Nonlinear elastic soil models were used to represent the behaviour of both cohesive and granular soils, while linear elastic models were used to simulate the reinforcing material. Multiple parameters were systematically varied during the simulations, including the depth, width, and shape of the granular trench, the friction angle of the trench soil, the modulus of elasticity of the reinforcement material, as well as the quantity and positioning of the reinforcing layers.

In both lined and unlined scenarios, the performance of sloped granular trenches was analysed. The findings showed that adding a granular trench beneath the foundation can improve bearing capacity and decrease settlement. Furthermore, using polymers as reinforcement materials has a significant impact on both bearing capacity and settlement outcomes. The depth ratio is a key factor in determining the settlement ratio for both reinforced and unreinforced granular trenches; as the depth ratio rises, the settlement ratio decreases.

The depth ratio was found to have a practical optimal value of 2. Creating a trench with a width of (X) that exceeds the foundation's width of (B) also decreases settlement; the width ratio (X/B) of 0.65 achieves the best performance (Fattah et al., 2010).

Ornek et al., (2012) provided numerical calculations to analyse the scale effects of circular footings supported by compacted, partially replaced layers on natural clay deposits. Different footing sizes were used to examine the phenomenon of scale effects. Computational analyses were carried out using a two-dimensional, axisymmetric FE software.

Before starting the analysis, they carried out field experiments using seven different footing diameters, ranging from 0.3 to 0.9 m, and three distinct thicknesses of partial replacements to validate the constitutive model. They found that the Mohr-Coulomb model accurately describes the behaviour of the partial replacement system and circular footings on natural clay soil. The Mohr-Coulomb model's parameters were established using results from standard laboratory and field experiments.

Additional numerical analysis was carried out by increasing the footing diameter to 2.5 m and investigating partial replacement thicknesses of up to twice the footing diameter, once the test results and numerical analysis findings showed good agreement. The parametric study results revealed that stabilisation had a substantial impact on the bearing capacity of the circular footing. For a given height-to-diameter ratio (H/D), the ultimate bearing capacity increased nonlinearly as the footing diameter increased.

A study was conducted by Fattah et al. (2015) to increase the bearing capacity of a footing lying on soft clay by replacing a portion of the clay with a small area of granular material. For models of soft clays with undrained shear strength (cu) values between 7 and 17 kPa, eight model experiments were carried out, split into two series. The first series had four models with a square soil replacement pattern, and the second series had four models with a trench soil replacement pattern.

According to the results, replacing soil in soft clay beds can substantially boost the foundation's bearing capacity. Specifically, the trench soil replacement enhanced the bearing capacity by a factor of 4.9 to 8.9, while the square soil replacement increased it by 3.9 to 8.1 times. Additionally, implementing soil replacement contributed to an increase in undrained shear strength of approximately 5.5% to 15%.

These findings suggest that placing a load on a rigid foundation over a wider area than the footing can enhance both the lateral and vertical strength of the surrounding soft soil. The larger bearing surface, paired with the extra support from the soil replacement beneath the footing, results in less bulging and a higher ultimate load capacity.

The following are conclusions from their experimental work,

- Replacing soft soil with granular soil can boost the bearing capacity of footings. The greatest improvement happens when the soil is partially replaced using a trench pattern, which involves extending the replacement by half the width (B/2) on all sides, and to a depth of 1.5 times the width (1.5B).
- Increasing the width of the replacement instead of the depth enhances the soil replacement method's effectiveness in boosting bearing capacity.
- By reducing the water content, the stone used as a substitute material helps to boost the shear strength of soft soil.
- Implementing soil replacement results in an increase of 5.5 to 15% in the undrained shear strength. Trench soil replacement has an improvement ratio that is roughly 10 to 20% higher than square soil replacement. (Shown in Figure 2-11).

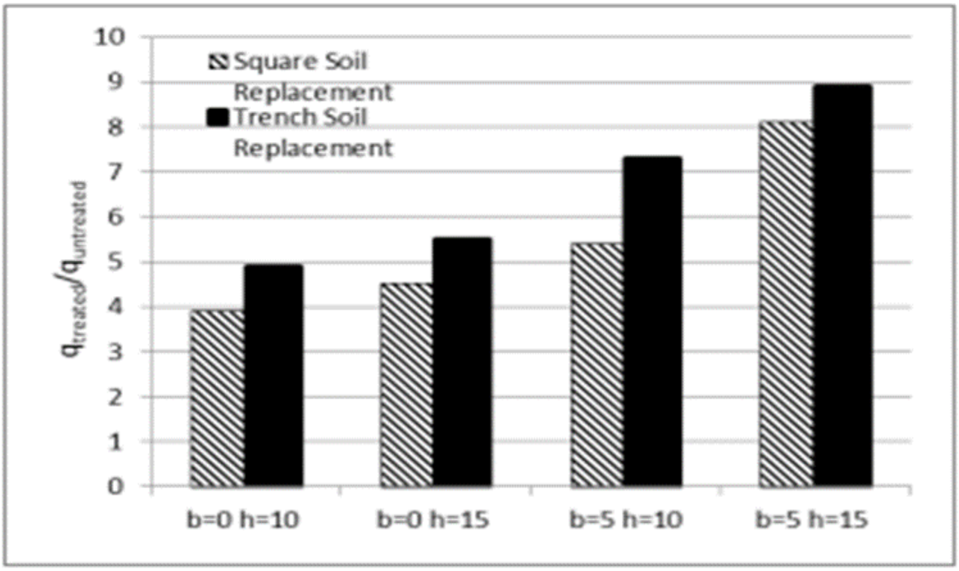


Figure 2-11 : Bearing capacity improvement ratio.

Mohammed Al-Waily (2019) examined the impact of using recycled crushed concrete as a partial replacement to increase bearing capacity and minimise soft clay settlement. The experimental procedures were carried out utilising a four-cubic steel soil tank and encompassed strip and square footings of four distinct dimensions: 250 × 250 × 250 mm, 300 × 300 × 300 mm, 400 × 400 × 400 mm, and 500 × 500 × 500 mm. Four sizes were utilised for each strip and square footing, while varying the replacement depths for the square footings at 0.2, 0.4, and 0.6 times their width, and for the strip footings at 0.33, 0.67, and 1 times their width.

The study assessed the impact of these replacements on settlement control and bearing capacity. The results showed that treated soil exhibited significantly less settlement and greater bearing capacity compared to untreated soil for both strip and square footings, especially as the depth of the partial replacement layer increased. Specifically, employing a partial replacement layer for strip footings with a depth of 0.4 times the width led to an average 72% reduction in settlement and an average 96% increase in bearing capacity. Moreover, the results showed a significant rise in bearing capacity for all replacement depths under square footings, with minimal settlement.

The following are conclusions from the study,

- The relationship between the settlement ratio (Sr) and the size of a strip footing remains unclear.

- As the level of partial replacement increases, the settlement ratio (Sr) becomes more apparent.
- For both types of footings, the bearing capacity ratio significantly increases as the depth of the partial replacement layer grows.
- Using crushed concrete beneath the square footing usually works better than beneath the strip footing.

2.7. Quarry dust-filled cylinders or columns

A variation of rigid inclusions involves the use of concrete-filled cylinders or quarry-dust columns. In this method, larger diameter drilled shafts or shallow caissons (typically ranging from 0.5 to 1.0 meters in diameter) are left in place and filled with either plain concrete or waste materials, such as crushed rock dust. These cylinders operate similarly to large piles or column inclusions, reinforcing the surrounding soil. While they have not been studied as comprehensively as stone columns or Controlled Modulus Columns, recent research indicates that they show better performance in soft soil conditions.

Srijan (2025) carried out a test on small quarry dust columns in soft clay soil, incorporating CBR loading tests. Conventional (uncased) quarry dust columns with diameters of 2.54 cm and 3.8 cm boosted load capacities by around 9% and 106%, respectively, compared to untreated soil. In contrast, encased columns, which were capped with a geotextile, achieved increases of 58% and 196%. Additionally, floating columns, which do not have end-bearing support, also demonstrated improvements in load capacity. Research has shown that both standard and encased quarry dust columns can greatly improve load-bearing capacity and minimise settlement in soft clay soils.

Laboratory tests on clay that have been enhanced with quarry-dust columns demonstrate significant improvements. One study (Vinod B et al., 2021) on "loose red earth" found that incorporating even a few short quarry-dust columns greatly increased the load-carrying capacity while reducing settlement. The settlement curves indicated that columns, especially when encased, stiffened the composite ground. For example, the data revealed that for a given settlement of 30 mm, groups of four columns supported 46% to 71% more load than untreated soil.

A useful metric for evaluating performance is the load improvement ratio (LIR), which compares the reinforced load to the unreinforced load at a fixed settlement. This ratio increases with the length and number of columns: in one dataset, a single 15 cm column had an LIR of approximately 1.43, while five 15 cm columns had an LIR of approximately 1.00. In simpler terms, having more or longer columns results in greater stiffness and load capacity.

Encasement of these columns further enhances their stiffness. The same studies indicate that cylinders wrapped or encased in stiff geosynthetics or foil experience less settlement than those left unwrapped, and higher encasement stiffness leads to lower settlements. Essentially, encasement restricts the lateral expansion of the column, thereby increasing its effective modulus.

2.8. Effect of a bounding wall on the bearing capacity of a footing

Fattah et al., (2014) conducted a study to test the behaviour of model footings concealed by walls at varying distances and depths from the footing resting on sandy soil. Several parameters must be considered when evaluating bearing capacity, including the specific gravity of the sand, the distance from the edge of the footing to the wall (denoted as d), the width of the footing (B), and the depth of the wall (h). Test results show that the presence of a wall considerably improves the bearing capacity, with the extent of enhancement varying depending on the wall's depth and its proximity to the foundation edge. This enhancement is primarily due to increased soil confinement beneath the footing.

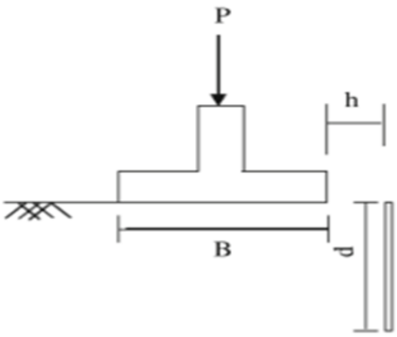
For rectangular footings supported by walls in loose sand, the greatest increase in bearing capacity is achieved when h/B = 0.5 and d/B = 2, resulting in a nearly 34% increase. In medium sand, using the same ratios (h/B = 0.5 and d/B = 2) leads to an approximate 33% increase in bearing capacity. In contrast, for dense sand with a ratio of h/B = 0 and d/B = 2, the largest increase in bearing capacity reaches about 52%.

Additionally, the presence of a wall reduces vertical settlement, with reductions varying between 3% and 24% depending on the wall's depth and its distance from the footing. While the impact of the wall on bearing capacity in dense sand is greatest when it directly touches the footing edge (h/B = 0), the most significant effects in loose and medium sand occur when the distance (d) is set to 0.5.

Table 2-5 provides a summary of the predicted and actual bearing capacities.

Table 2-5 : Summary of the observed bearing capacities

Test no.	Distance of the wall (h/B)	Depth of the wall (d/B)	Predicted bearing capacity from Terzaghi's equation (kN/m^2)			Observed bearing capacity (kN/m^2)		
			$*\phi = 34°$	$*\phi = 37°$	$*\phi = 41°$	$*\phi = 34°$	$*\phi = 37°$	$*\phi = 41°$
1	–	–	16.5	30.1	66.8	32.5	35.7	116.3
2	0	0.5				34.3	40.0	137.3
3	0	1				38.0	40.7	162.0
4	0	1.5				40.8	46.0	170.0
5	0	2				41.8	46.0	177.0
6	0.5	0.5				33.5	39.8	133.2
7	0.5	1				40.2	41.0	150.0
8	0.5	1.5				42.0	46.0	166.0
9	0.5	2				43.4	47.5	172.0
10	1	0.5				33.5	37.4	120.2
11	1	1				35.5	41.3	130.0
12	1	1.5				36.0	42.0	141.8
13	1	2				37.0	42.7	146.0
14	1.5	2				35.5	41.4	138.2
15	2	2				33.3	40.1	133.0



2.9. Summary

In summary, the literature indicates that both rigid inclusions and granular reinforcements can enhance the settlement performance of footings on soft clay. However, several practical design questions remain unresolved. The identified gaps regarding consolidation effects, parametric optimisation, and the direct evaluation of cylinder-type solutions highlight the need for this study. This research aims to address these issues by modelling quarry dust-filled cylinders and Controlled Modulus Columns (CMCs) using Finite Element Modelling (FEM) in MIDAS GTS NX, while also varying key design ratios. The goal is to determine whether quarry dust-filled cylinders and CMCs can serve as efficient ground-improvement alternatives to piles or micro-piles, and to establish design guidelines for their use in consolidation scenarios.

Key findings of the literature review are shown below.

- Pad footings are not recommended for buildings on soft clays
- Micro-piles are a costly solution to reduce settlements in clay soils, and they also require pile caps
- Stone columns improve clay soil significantly, but are less effective in very soft clays
- Partial soil replacements (trenches or layers) can reduce settlements and increase bearing capacity significantly
- CMCs have a load-sharing mechanism and are very effective in reducing settlement on clay; however, there are limited studies on the consolidation behaviour
- Quarry dust-filled cylinders show potential in reducing settlements, but there are limited studies on the consolidation behaviour
- Research gaps identified:
    - Lack of FEM-based consolidation studies for quarry-dust cylinders and CMCs
    - Few comparative studies between CMCs and cylinders
    - Absence of ratio-based parametric design guidelines

CHAPTER 3: METHODOLOGY

3.1. Introduction

This chapter describes the methodology used in stress analysis and consolidation analysis, including the development of models, the types of soil models used, the material properties utilised, and the validation process. Both stress and consolidation analyses were carried out using MIDAS GTS-NX Finite Element (FE) analysis software. A 1.2 m × 1.2 m footing with a thickness of 0.4 m is employed in this study, while keeping 0.8 m of embedment depth, except in Sections 3.8 and 3.9, where varying embedment depths are studied.

3.2. Three-dimensional modelling and stress analysis

For the three-dimensional models, model domain sizes were taken as described in the research paper published by Ahmadi et al. (2016). Both the model's width and length have been set to 8B, with the model height set to 4B to ensure accurate results from FE analysis, where B represents the width of the square footing.

B = 1.2 m

8B = 9.6 m,

4B = 4.8 m

Figure 3-1 illustrates the stress distribution of the model when the full load is applied, confirming that beyond the above-mentioned region, the influence of the load is negligible.

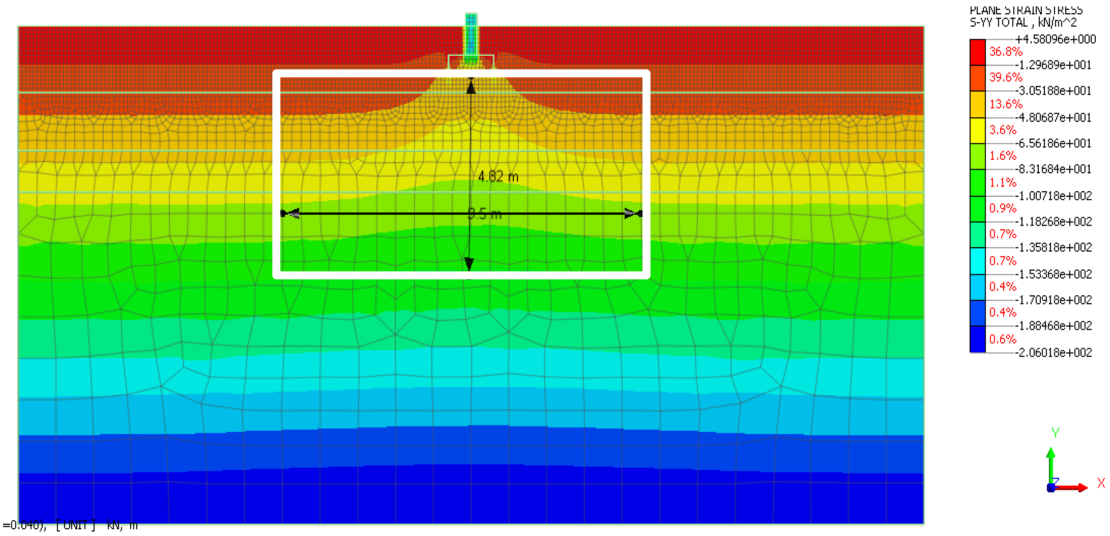


Figure 3-1: Influence Zone

3.3. Material properties used in 3d stress analysis

All soil types were modelled using the Mohr-Coulomb soil model due to its suitability and convenience in stress analysis, while the structural elements were modelled using the Elastic material model. Soil parameter values, shown below in Table 3-1, were extracted from the research paper "Behaviours of Pipe-Soil Interaction on Unstable Slopes by Finite Element Simulation", which was published by (Susila & Agrensa, 2018)

Table 3-1: Soil parameters used in the stress analysis

Type	γ (kN/m³)	E (kN/m²)	C (kN/m²)	ϕ°	Poisson's Ratio
Soft Clay	14	4800	24	0	0.3
Loose Sand	16	15000	7	21	0.3
Dense Sand	19	55000	1	38	0.3
Quarry Dust	18	10000	4	30	0.3
Footing	25	3.6×10^7	-	-	0.3
Cylinder	23	2.3×10^7	-	-	0.25
CMC	22	1×10^7	-	-	0.25

The modulus of elasticity and Unit weight of reinforced concrete footing were calculated according to the equations 1 & 2 given below.

$$E = E_c \times \frac{V_c}{V_{tot}} + E_s \times \frac{V_s}{V_{tot}} \quad \text{Eq 1}$$

$$\gamma = \gamma_c \times \frac{V_c}{V_{tot}} + \gamma_s \times \frac{V_s}{V_{tot}} \quad \text{Eq 2}$$

$E = 3.6\times10^7$ kN/m²

$\gamma = 25$ kN/m³

3.4. Comparison between full-scale model and quarter-sized model – stress analysis

The initial model (Figure 3-2) was modelled with a width and length of 12 m each and a height of 6 m, while another model (Figure 3-3) was developed as a quarter of the initial model by utilising symmetrical boundary conditions, as shown in Figure 3-4 below. Both models consist of three layers, each with a depth of 2 m, where the first layer is soft clay, the second layer is loose sand, and the third layer is dense sand. This was done to reduce the computational complexity, thus reducing computational time.

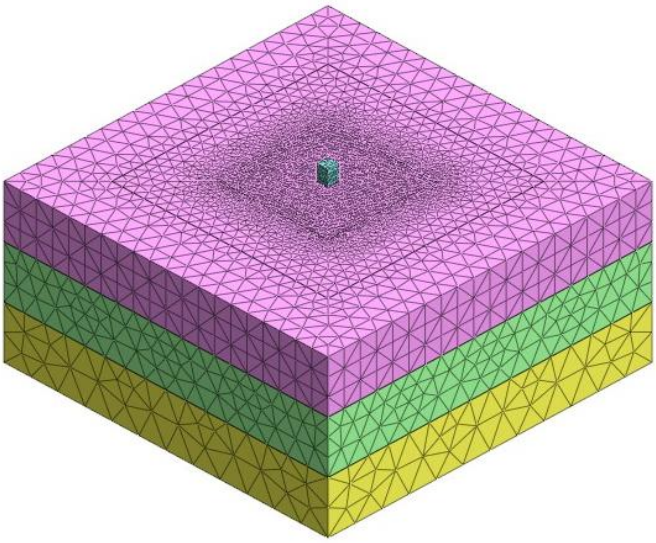


Figure 3-2 : Geometry of the full 3D model

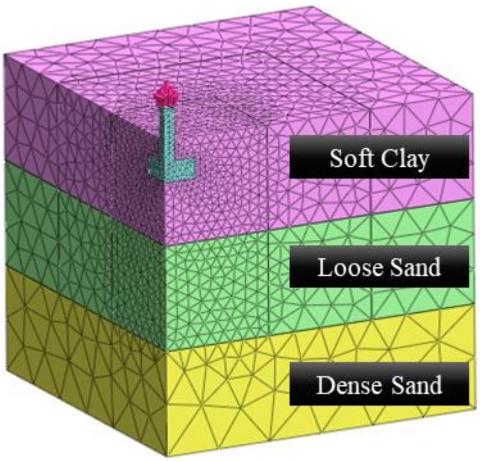


Figure 3-3: Quarter-sized model

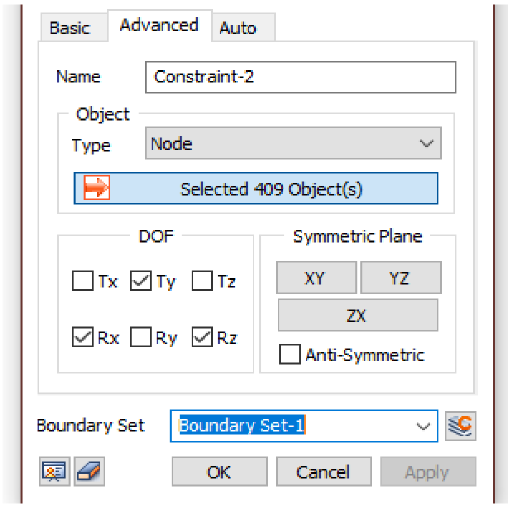


Figure 3-4: Symmetric boundary conditions used in the ZX plane.

In addition to the above models, another stress analysis was conducted with the implementation of a hollow concrete cylinder to the above models, as shown in Figure 3-5 and Figure 3-6. The concrete cylinder has an inner diameter of 1.8 m, a thickness of 0.1 m, and a height of 2 m. For both analyses, it was assumed that the water table is below the influence zone.

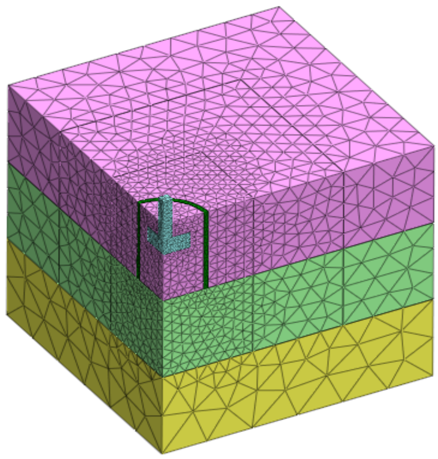


Figure 3-5: Quarter-sized model with Cylinder

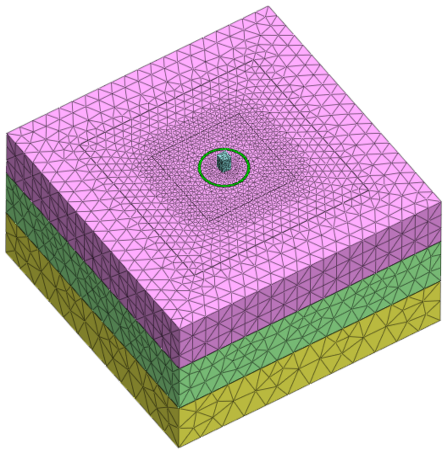


Figure 3-6: Full-sized model with Cylinder

3.5. Mesh sensitivity analysis

To further reduce computational time while maintaining accuracy, a mesh sensitivity analysis was performed to identify the optimal mesh size. The first two layers were divided into three divisions, as shown in Figure 3-7, and used three different mesh size sets as shown in Table 3-2 below.

Table 3-2: Mesh Sizes

Set Number	Layer Number	Size (m)		
		Part 1	Part 2	Part 3
1	1	0.1	0.25	0.5
	2	0.2	0.4	0.65
	3	1	-	-
2	1	0.2	0.4	0.6
	2	0.25	0.5	0.75
	3	1	-	-
3	1	0.3	0.5	0.7
	2	0.4	0.7	0.8
	3	1	-	-

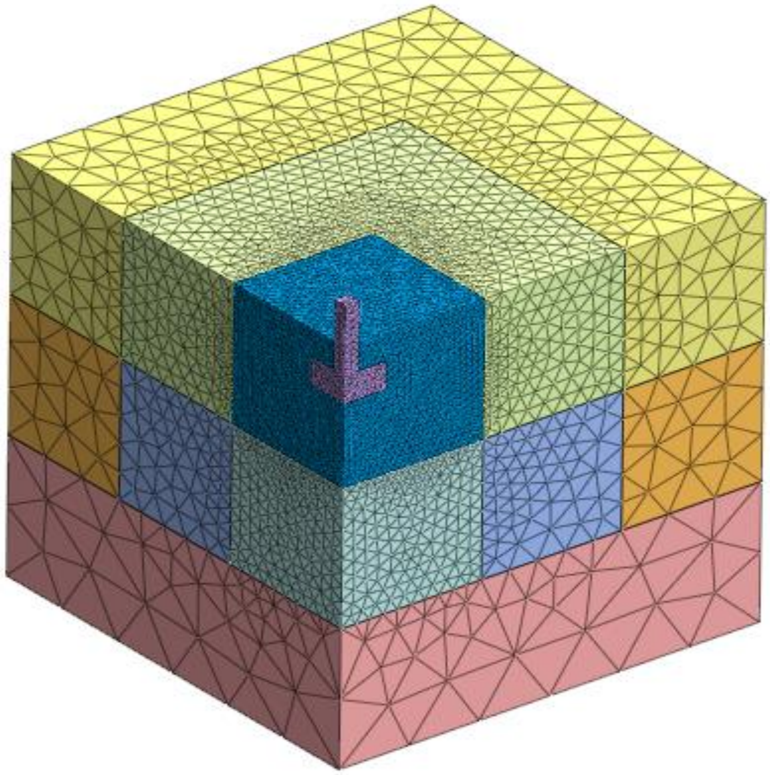


Figure 3-7: Different mesh sizes used in the FE model

Above mentioned stress analyses (including those in chapter 3.4) were conducted through a construction stage analysis, which includes three construction stages as described in Table 3-3 below.

Table 3-3: Construction Stages

First Stage	All the soil profiles and boundary conditions were activated in this stage.
Second Stage	Footing / Concrete cylinders were activated in this stage.
Third Stage	Loading stage

During the loading stage, a 300 kN load was applied to the footing in 50 load increments, where every increment was 6 kN, while in quarter-sized models, a 75 kN load was utilised with the same number of increments, which is equivalent to 300 kN in the full-scale model.

3.6. Three-dimensional model validation using stress analysis

The validation process of the soil models was conducted in three ways for stress analysis and from a lab model test for consolidation analysis, as detailed below.

Stress analysis:

- From comparing with theoretical Settlements
- Compared with in situ data from Texas A&M University
- Compared with in situ data from Green Cove Springs, Florida

Consolidation analysis:

- Comparing FEM with lab model test results

3.6.1. Model validation through theoretical settlement analysis

Predicted settlement values obtained through stress analysis of the quarter-sized model were compared with linear elastic settlement values calculated from the equation (Janbu et al., 1966) given below.

$$S = qB\left(\frac{1-\mu^2}{E_s}\right)\mu_0\mu_1$$

q - Applied pressure

μ - Poisson's ratio

Es - Elastic Modulus

μ_1 and μ_0 rely on the position of the footing beneath the ground surface (D) and the thickness of the stratum (H).

B = 1.2 m, μ = 0.3, Es = 4800 kN/m^2, μ_1= 0.45, μ_0 = 0.8

3.6.2. Model validation through Texas A&M university site experiment data

Jean-Louis Briaud & Robert Gibbenst (1997) conducted five large-scale footing load tests at the National Geotechnical Experimentation Site at Texas A&M University. Footing number 1 (3 m × 3 m) was modelled (Figure 3-8) and loaded with a load of 4 MN. Table 3-4 below shows the soil properties used in the model. The water table was maintained at a depth of 4.9 m below the ground surface. Modulus of elasticity values were calculated from the SPT and CTP test data obtained from the field test.

Table 3-4: Soil Properties

Type	γ (kN/m³)	E (kN/m²)	C (kN/m²)	φ°	Poisson's Ratio
Layer 1	15	82000	1	35	0.3
Layer 2	16	110000	1	30	0.3
Layer 3	15	125000	5	20	0.3
Layer 4	17	150000	4	20	0.3
Footing	23	3.6×10^{7}	-	-	0.3

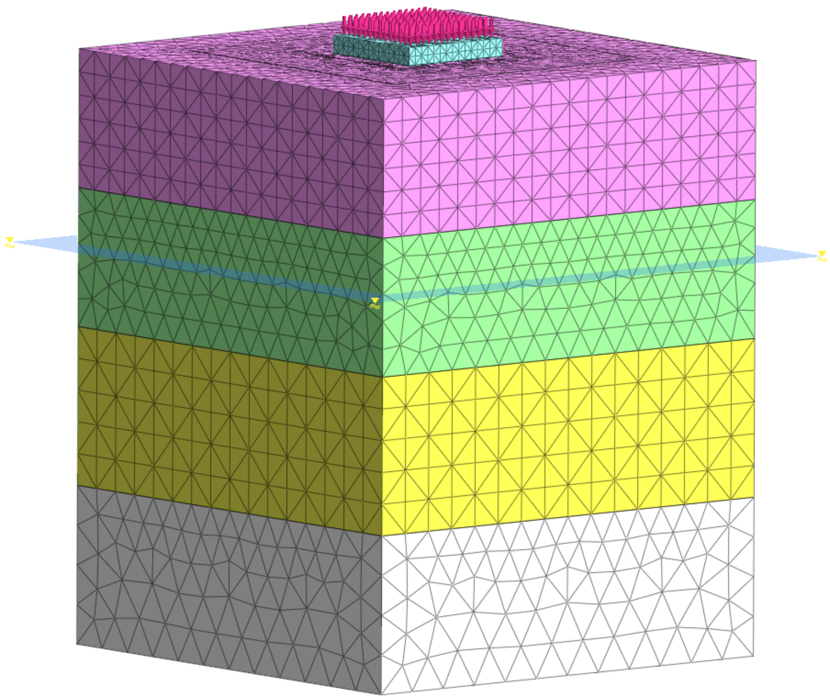


Figure 3-8: FE Model of Footing 1

3.6.3. Model validation through green cove springs, Florida experiment data

A circular 0.6 m-thick reinforced concrete footing with a diameter of 1.8 m, which was 0.67 m embedded in the ground, was loaded with a load of 222 kPa in Green Cove Springs, Florida (Anderson et al., n.d.). The circular footing was modelled (Figure 3-9) as a square footing with a width of 1.6 m in the FE analysis. Soil properties used in the FE model are displayed in Table 3-5 below. The water table was kept at 1.7 m below the ground surface.

Table 3-5: Soil Properties

Type	γ (kN/m^3)	E (kN/m^2)	C (kN/m^2)	ϕ^O	Poisson's Ratio
Layer 1	18.2	37000	1	47.5	0.3
Layer 2	17.3	18700	1	42	0.3
Layer 3	15.7	15460	1	38	0.3
Layer 4	15.7	4000	380	0	0.3
Footing	24	3.6×10^7	-	-	0.3

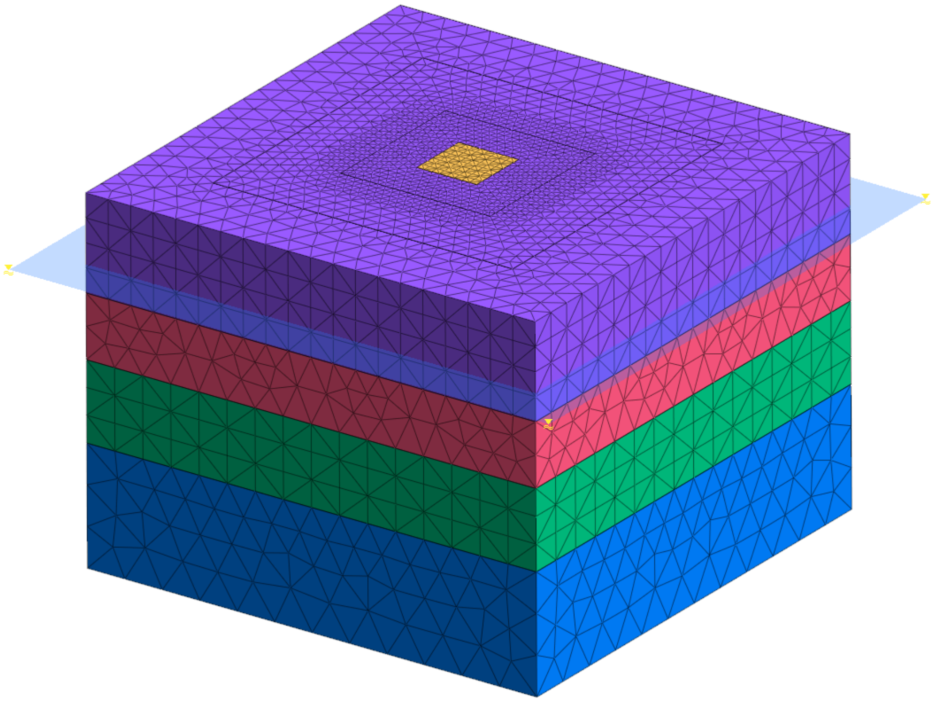


Figure 3-9: Model of equivalent square footing

3.7. Three-dimensional model validation using consolidation analysis

Validation of the soil model for consolidation analysis was conducted by comparing the consolidation settlements obtained from a lab model test done by K.I. Keshani at the University of Moratuwa. The lab model is shown in Figure 3-10, which was conducted in a 0.45 m × 0.45 m container. The first layer consists of 6 cm of deep sand, the second layer is made of 30 cm deep peat soil, and the third layer consists of 10 cm of deep sand.

Two consolidation tests were done to obtain soil parameters of peat soil, which are shown in Table 3-6 below.

Table 3-6: Soil properties from the consolidation tests

	N1	N2
Cc	0.444267	0.416899
Cs	0.049495	0.045677
Pc	40 kPa	38 kPa

Gs – 2.1

M.C – 0.65

e_0 – 1.365

K – 1.174e-6 m/s

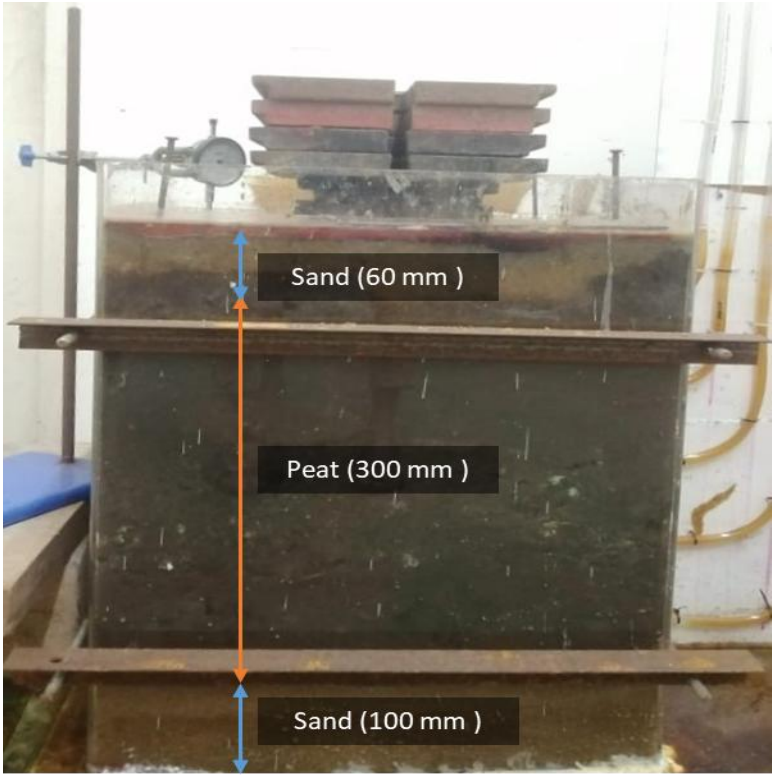


Figure 3-10: Lab Model Test

Soil parameters used in the FE model (Figure 3-11) are displayed in Table 3-7. Loading was done in two stages: in the first stage, a 5 kg load was applied for 1 hour, and in the second stage, a total of 10 kg load was applied for another two hours.

Table 3-7: Material Properties

Type	Peat	Sand	Footing
Material Model	MCC	Mohr-Coulomb	Elastic
γ (kN/m^3)	10	16	25
E (kN/m^2)	4800	15000	3.6×107
C (kN/m^2)	-	7	-
ϕ^O	-	21	-
Poisson's Ratio	0.38	0.3	0.3
e_0	1.365	0.8	-
λ	0.187	-	-
k	0.021	-	-

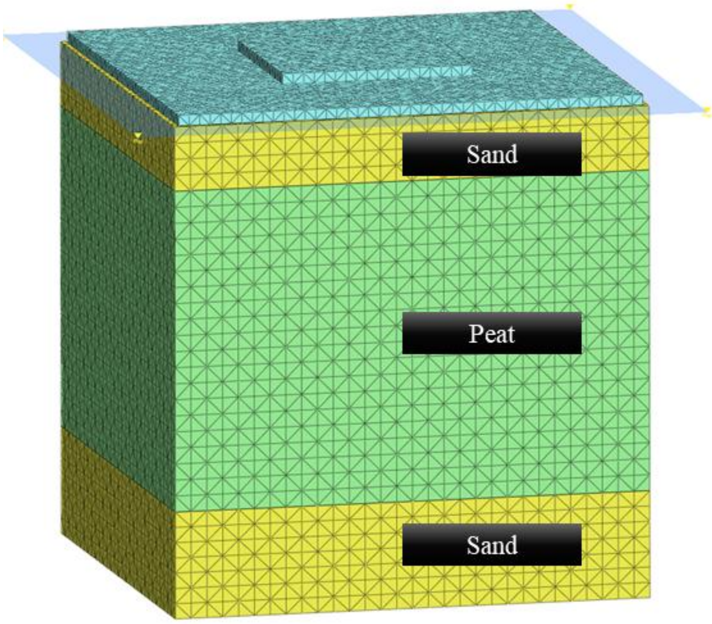


Figure 3-11: FE model of Lab model test

The thesis employed a two-stage validation approach rather than a comprehensive parametric calibration.

Direct calibration parameters obtained:

- Compression index: Cc = 0.444 (Test N1), 0.417 (Test N2)
- Swelling index: Cs = 0.049 (Test N1), 0.046 (Test N2)
- Pre-consolidation pressure: Pc = 40 kPa (Test N1), 38 kPa (Test N2)
- Initial void ratio: e_0 = 1.365
- Permeability: k = 1.174×10^{-6} m/s

MCC Parameter Derivation:
Standard relationships used to convert consolidation parameters to MCC:

- $\lambda = C_c/(2.303(1+e_0)) = 0.187$
- $\kappa = C_s/(2.303(1+e_0)) = 0.022$
- $M = 6\sin\varphi'/(3-\sin\varphi')$ (critical state line slope)

Table 3-8 : MCC Model Parameters with Sources

Parameter	Peat (Validated)	Soft Clay (Used)	Source
λ	0.187	0.22	(Surarak et al., 2012)
κ	0.022	0.03	(Surarak et al., 2012)
e_0	1.365	3.0	(Surarak et al., 2012)

3.8. Impact of embedment depth of the footing - stress analysis

To examine the effect of the footing embedment depth on the settlement using stress analysis, two different models: one with a 2 m deep soft clay soil layer (Figure 3-12) and one with a 4 m deep clay soil layer (Figure 3-13), were utilised. For both models, embedment height was varied from 0.1 m to 1.5 m from the ground surface. For both models, a 300 kN load was applied in 50 increments in the loading stage, while keeping the water table below the influence zone. The material properties used in this stress analysis are the same as displayed in Table 3-1.

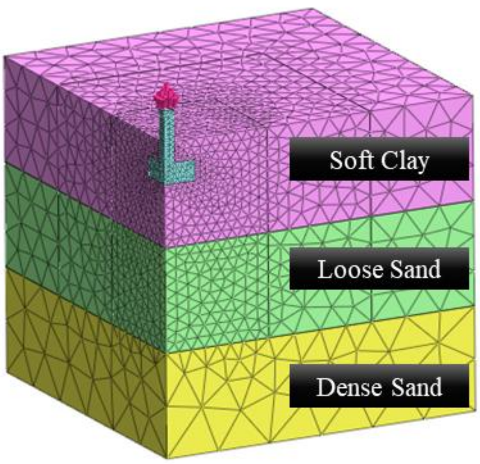


Figure 3-12: Model with 2m clay layer

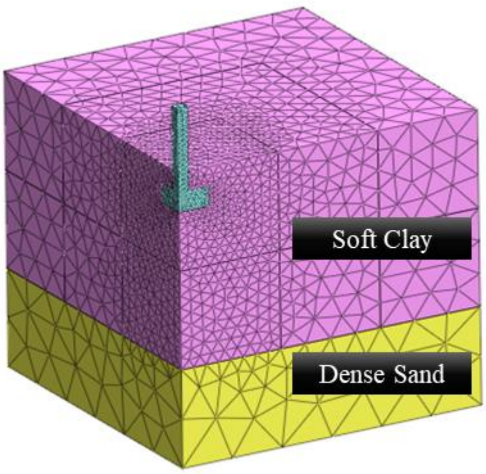


Figure 3-13: Model with 4m clay layer

3.9. Impact of embedment depth of the footing - consolidation analysis

Analysing the impact of footing embedment depth on settlement using consolidation analysis required a model with a 4 m deep clay soil layer (Figure 3-14). The embedment height was adjusted from 0.1 m to 1.5 m above the ground surface. This analysis was conducted through a construction stage setup, as outlined in Table 3-8 below, with the water table set 1 m below the ground level. The material properties used in the consolidation analysis are presented in Table 3-9. Analyses were conducted for pre-consolidation stresses of 100 kPa and 250 kPa, while the permeability coefficient was maintained at 1e-9 m/s for clay.

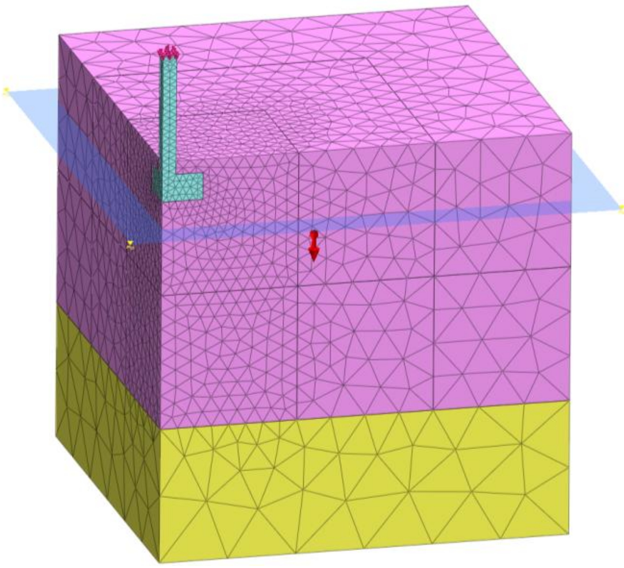


Figure 3-14: Quarter-sized model

Table 3-9: Construction Stage Setup

First Stage	All the soil profiles and boundary conditions were activated in this stage (1 day)
Second Stage	Loading Stage (150 days, 300kN of load in 10 increments)
Third Stage	After Construction (600 days)

Table 3-10: Material Properties

Type	Clay	Dense Sand	Footing
Material Model	MCC	Mohr-Coulomb	Elastic
γ (kN/m^3)	14.5	19	25
E (kN/m^2)	5000	55000	3.6×107
C (kN/m^2)	-	1	-
ϕ^O	-	38	-
Poisson's Ratio	0.2	0.3	0.3
e_0	3	0.4	-
λ	0.22	-	-
k	0.03	-	-

3.10. Impact of controlled modulus columns (CMC) – stress analysis

To determine the effect of Controlled Modulus Columns (CMC) on settlement using stress analysis, a model (Figure 3-15) consisting of three layers was employed. The first layer consists of 1.5 m of loose sand, the second layer is filled with 2.5 m of soft clay, and the third layer consists of 2 m of dense sand. Nine CMCs, each with a 0.2 m diameter and 2.7 m in length, were employed in a 3×3 grid with a 1 m spacing between them, as illustrated in Figures 3-16 and 3-17. The top level of the CMC was maintained at 1.5 m below the ground level, thus creating a 0.7 m thick soil layer between the footing bed and the top of the CMC. This layer, known as the Load Transfer Platform (LTP), is employed to distribute the load between the CMCs and the soil in between, ensuring an effective load-sharing mechanism.

A stress analysis was conducted using the same construction stage setup and loading steps, as discussed in Section 3.5, and compared the settlement values predicted from FE model with and without CMCs. The Plane Interface tool was used to ensure the soil-structure interaction of CMCs, as shown in Figure 3-18.

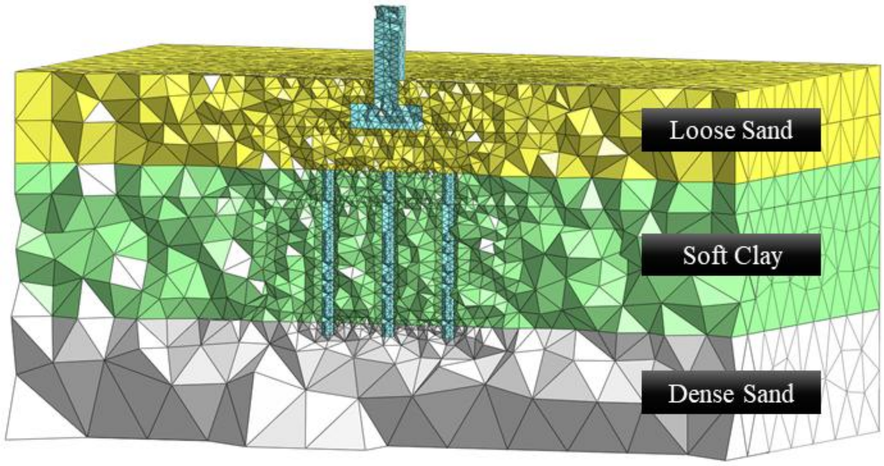


Figure 3-15: Cross-section of the model with CMC

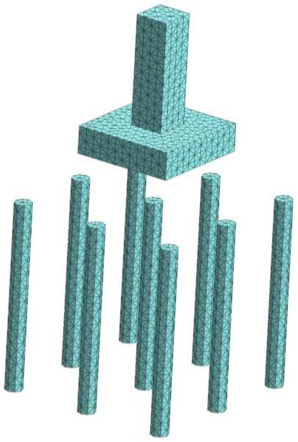


Figure 3-16: CMC system

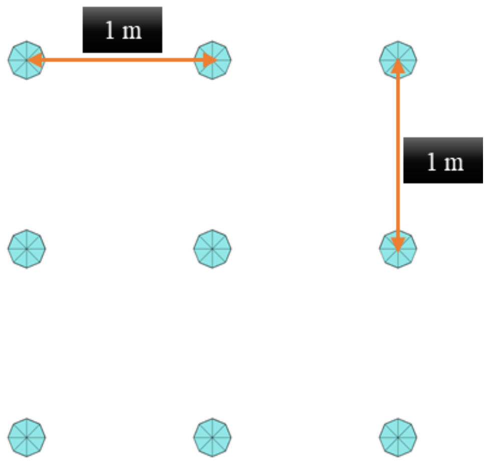


Figure 3-17: CMC spacing

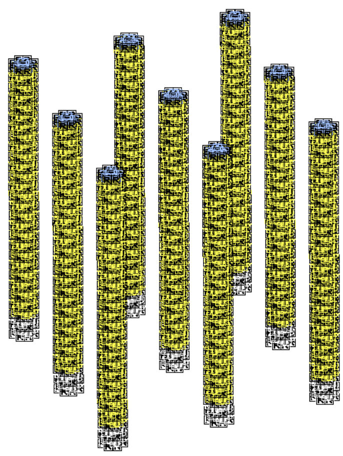


Figure 3-18: CMC interface

3.11. Impact of controlled modulus columns (CMC) – consolidation analysis

The same model that was shown in section 3.10. was used with a changing material model and properties of soft clay, and utilising properties for LTP as displayed in Table 3-10 and Figure 3-19. For this analysis, the water table was held at 1 m below the ground level, while the permeability coefficient was set at 1×10^{-9} m/s for soft clay. The CMC configuration utilised in Section 3.10 was used while using a pre-consolidation stress of 250 kPa for clay soil. This analysis was conducted using the same construction stage setup and loading steps, as discussed in Section 3.9, and compared the settlement values predicted from FE model, with and without CMCs.

Table 3-11: Material Properties

Type	Clay	CMC	LTP	Dense Sand	Footing
Material Model	MCC	Elastic	Mohr-Coulomb	Mohr-Coulomb	Elastic
γ (kN/m^3)	14.5	22	18	19	25
E (kN/m^2)	5000	1×10^7	40000	55000	3.6×10^7
C (kN/m^2)	-	-	5	1	-
ϕ^O	-	-	35	38	-
Poisson's Ratio	0.2	0.25	0.3	0.3	0.3
e_0	3	-	1	0.4	-
λ	0.22	-	-	-	-
k	0.03	-	-	-	-

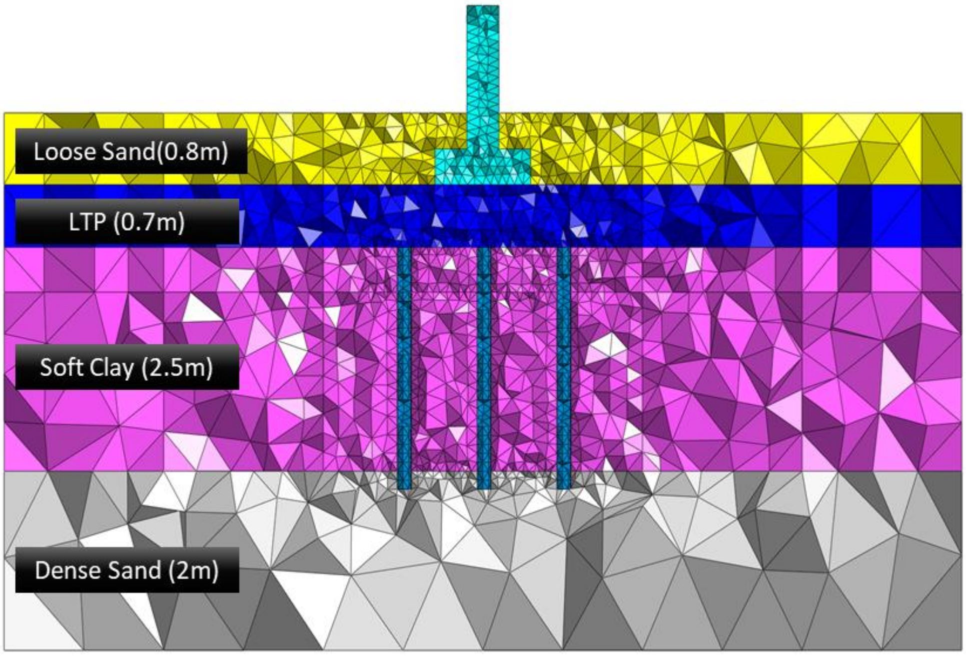


Figure 3-19: Model with LTP and CMCs

Clay parameters were chosen based on the soil properties outlined in previous research (Surarak et al., 2012). The Table 3-12 indicates the void ratio range for soft clay, and for the model, a ratio of 3 was selected.

Table 3-12 : properties of Bangkok clays

Properties	Weathered clay	Soft clay	Stiff clay
Natural water content (%)	133 ± 5	122–130	20–24
Natural voids ratio	3.86 ± 0.15	3.11–3.64	1.10–1.30
Grain size distribution			
Sand (%)	7.5	4.0	23
Silt (%)	23.5	31.7	43
Clay (%)	69	64.3	34
Specific gravity	2.73	2.75	2.74
Liquid limit (%)	123 ± 2	118 ± 1	46 ± 2
Plastic limit (%)	41 ± 2	43 ± 0.5	19 ± 2
Dry unit weight (kN/m^3)	15.8 ± 0.3	16.5	15.5–16.5
Consistency	Soft	Soft	Stiff
Colour	Dark grey	Greenish grey	Greenish grey
Degree of saturation (%)	95 ± 2	98 ± 2	94–100

This study examines a very soft organic clay layer with high water content (e_0 = 3.0). It corresponds to Site Class E or F soils based on building codes, which necessitate significant ground improvement. Table 3-13 illustrates the consistency with other parameters employed in the study.

Table 3-13 : Consistency with Other Parameters

Parameter	**Value Used**	**Typical for e_0=3.0**	**Typical for e_0=1.0**
Unit weight (γ)	14.5 kN/m³	12-14 kN/m³ ✓	16-18 kN/m³
Undrained shear strength	cu=24 kPa (stress analysis)	10-30 kPa ✓	30-60 kPa
Elastic modulus	E=5000 kPa	2000-8000 kPa ✓	5000-15000 kPa
λ	0.22	0.15-0.30 ✓	0.10-0.20

3.12. Parametric study on CMC technique – consolidation analysis

This section discusses the parametric study conducted on CMCs through consolidation analysis. The parametric study investigates several factors: the effect of CMC height on settlements, the impact of CMC diameter on settlements, the impact of CMC spacing on settlements, and the effect of LTP layer thickness on settlements. The study was done using a pre-consolidation stress of 250 kPa for the clay soil. The whole study was conducted using material properties, and the construction stage setup discussed in Section 3.11, and the water table was kept at 1 m below ground level.

3.12.1. The effect of CMC height on settlements

The height of the CMCs was varied from 2.5 m to 3.0 m, while the top remained at 1.5 m below ground level. Cross sections of models with CMC heights of 2.5 m and 3.0 m are shown in Figures 3-20 and 3-21, respectively. For this study, the CMC diameter is kept at 0.2 m, the CMC spacing is set to 1 m, and the LTP layer thickness is maintained at 0.7 m.

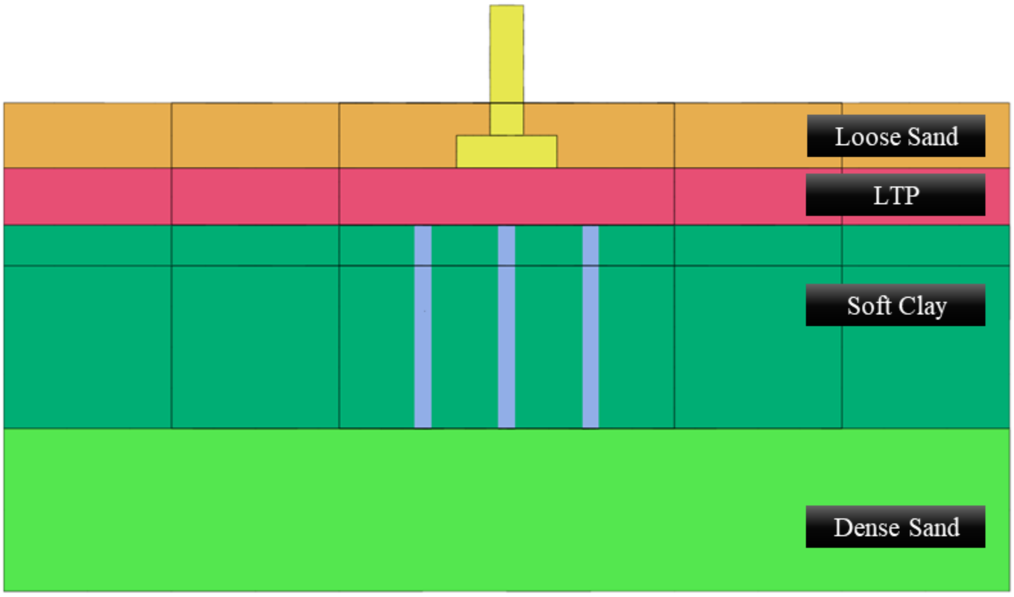


Figure 3-20: Cross-section of the model with CMC heights of 2.5 m

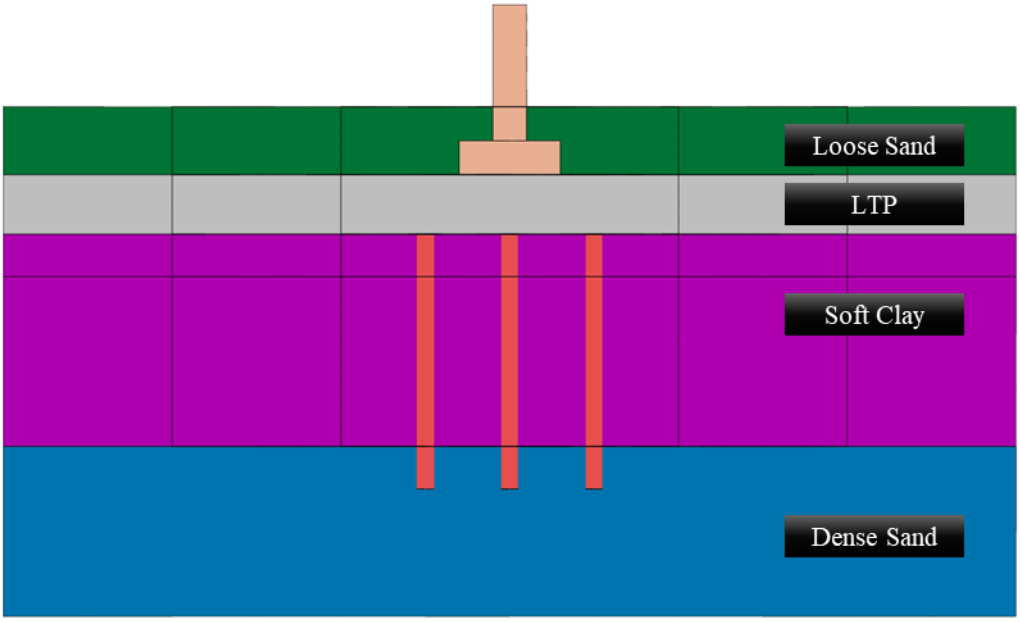


Figure 3-21: Cross-section of the model with CMC heights of 3 m

3.12.2. The impact of CMC diameter on settlements

CMC was set to a height of 2.5 m, spaced 1 m apart, with diameters ranging from 0.1 m to 0.4 m. The CMC top was kept at 1.5 m below ground level. Figures 3-22 and 3-23 show cross-sections of models with CMC diameters of 0.1 m and 0.4 m, respectively. For this study, the LTP layer thickness is maintained at 0.7 m.

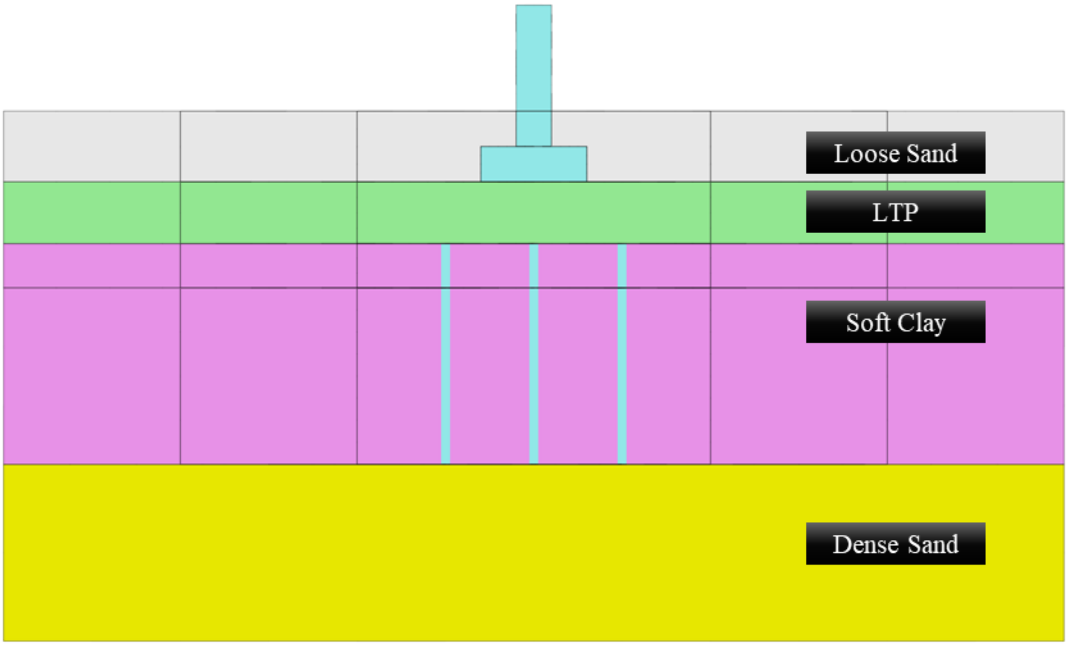


Figure 3-22: Cross-section of the model with CMC diameters of 0.1 m

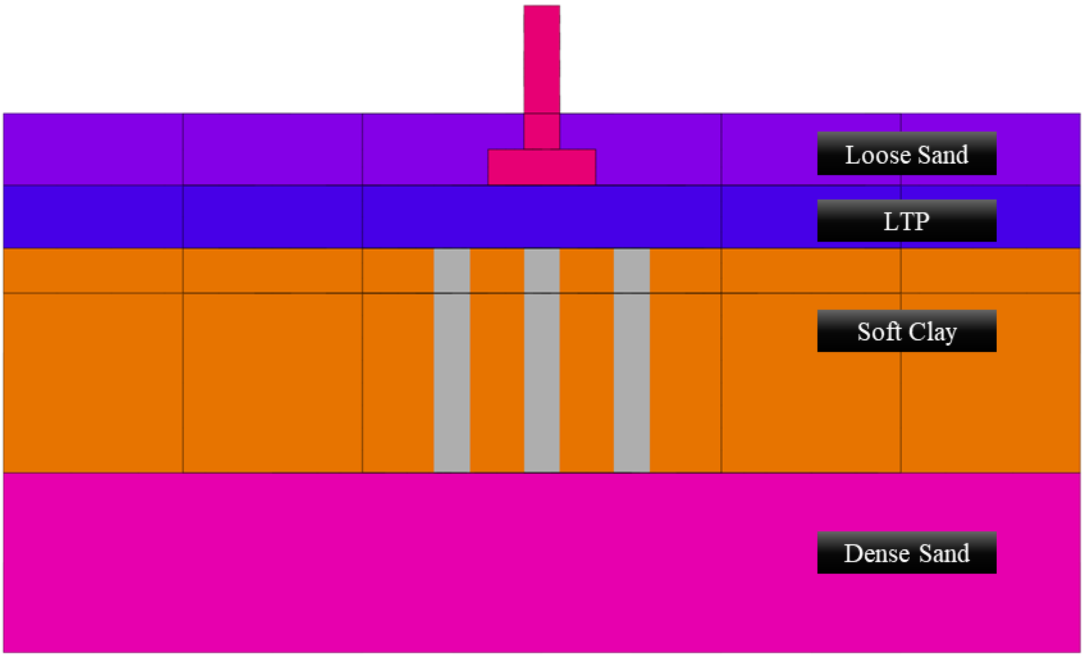


Figure 3-23: Cross-section of the model with CMC diameters of 0.4 m

3.12.3. The impact of CMC spacing on settlements

CMC height was set at 2.7 m, with a diameter of 0.2 m, and spacing varied from 0.6 m to 1.2 m. The CMC top stayed 1.5 m below ground level. Figures 3-24 and 3-25 show cross sections of models with CMC spacing of 0.6 m and 1.2 m, respectively. In this study, LTP layer thickness is kept at 0.7 m.

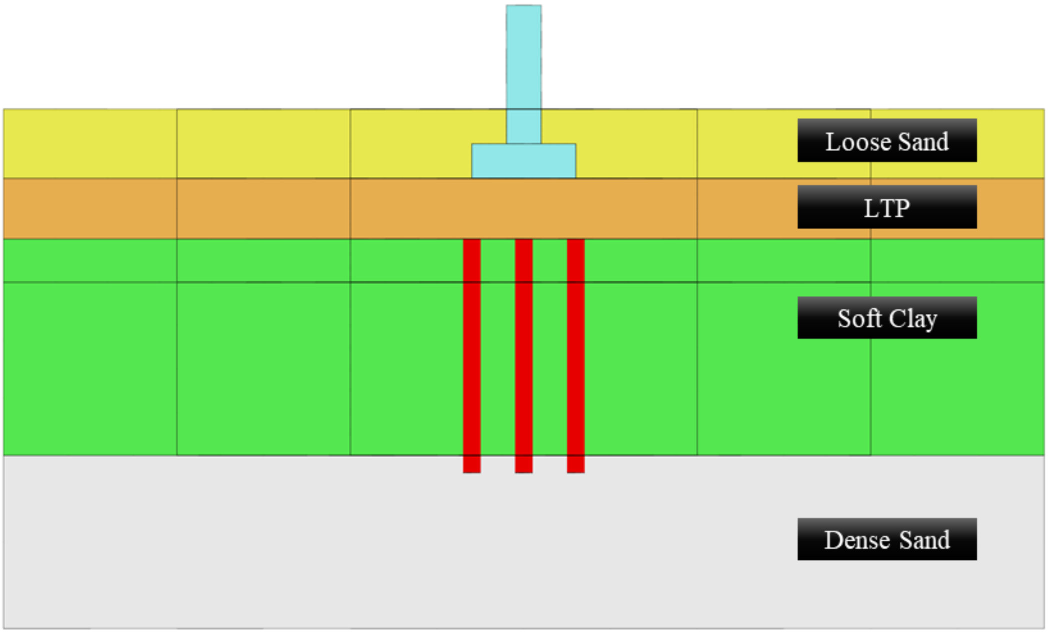


Figure 3-24: Cross-section of the model with CMC spacing of 0.6 m

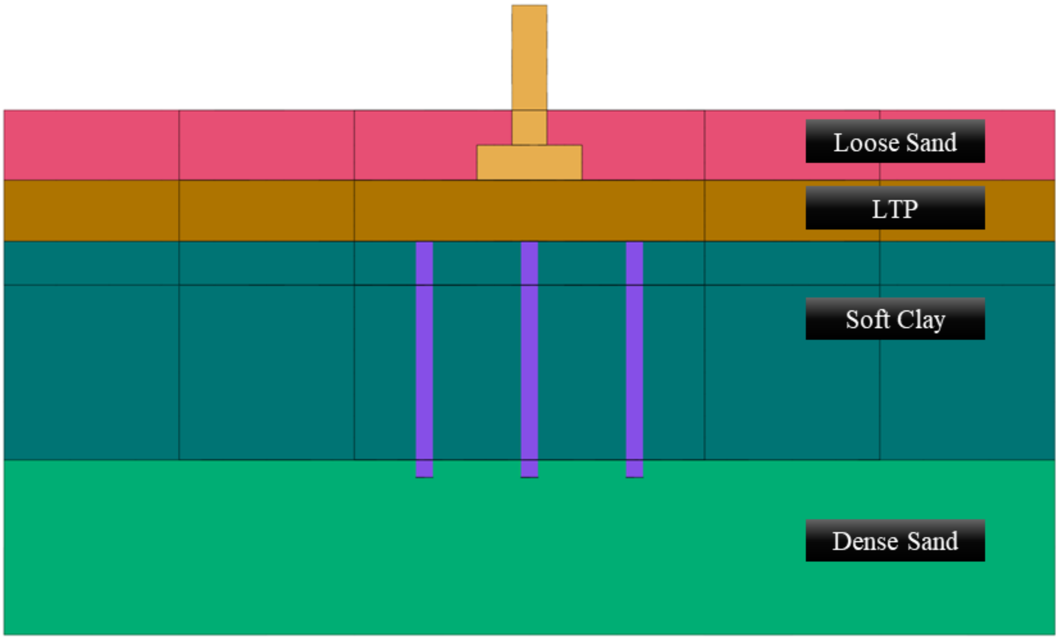


Figure 3-25: Cross-section of the model with CMC spacing of 1.2 m

3.12.4. The effect of LTP thickness on settlements

The Diameter of the CMC was set at 0.2 m, with a CMC spacing of 1 m, while the LTP thickness ranged from 0.3 m to 0.9 m. The level of CMC top also varied with the LTP layer thickness from 1.1 m to 1.7 m. Cross sections of models with LTP thicknesses of 0.3 m and 0.9 m are shown in Figures 3-26 and 3-27, respectively.

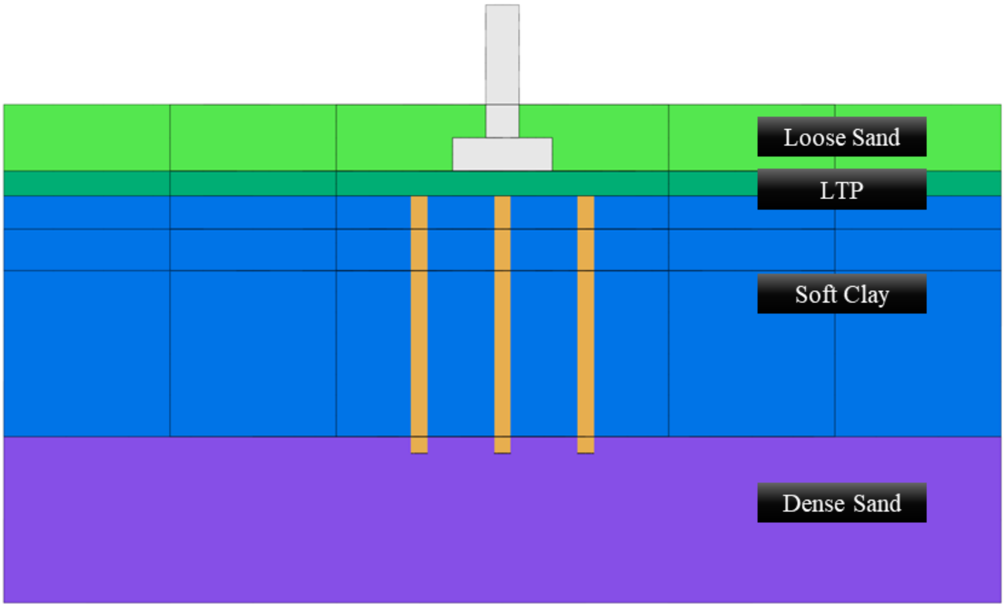


Figure 3-26: Cross-section of the model with LTP thicknesses of 0.3 m

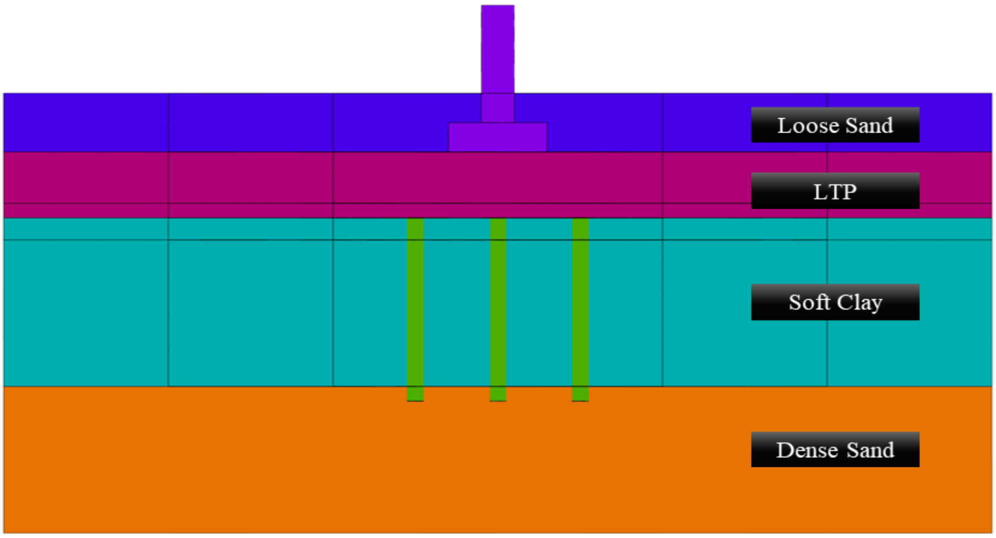


Figure 3-27: Cross-section of the model with LTP thicknesses of 0.9 m

3.13. Impact of using concrete cylinders – stress analysis

To study the effect of concrete cylinders on reducing settlements, a stress analysis was carried out with the use of quarter-sized models with concrete cylinders, as mentioned in Section 3.4. Material properties described in Table 3-1, construction stage setup, and the loading steps mentioned in Section 3.5 were used in this analysis. The heights of these cylinders varied from 0.8m to 2m.

The analysis included several models, and models with cylinder heights of 2 m and 1.4 m are shown in Figures 3-28 and 3-29, respectively.

3.14. Impact of using quarry dust as a soil substitute inside concrete cylinders – stress analysis

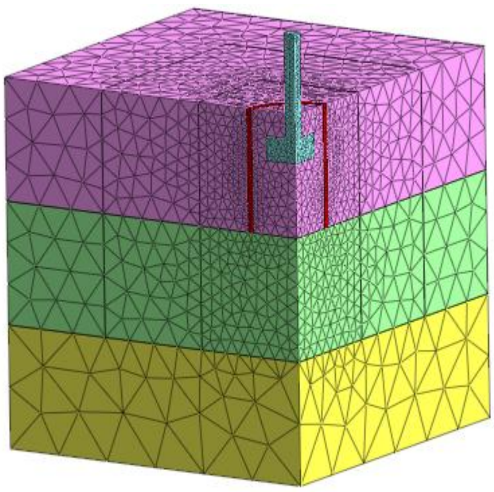


Figure 3-28: 2m deep Cylinder

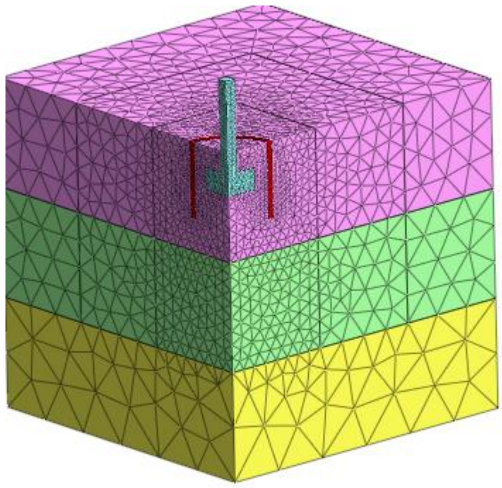


Figure 3-29: 1.4m deep Cylinder

To examine the influence of using quarry dust as a soil substitute inside concrete cylinders, models described in Section 3.13 were used by replacing soil inside the cylinders with quarry dust. The analysis included several models, with both cylinder heights and the height of quarry dust fill varying simultaneously from 0.8 m to 2 m. The model in which both cylinder height and the height of the quarry dust fill are kept at 1.4 m is presented in Figure 3-30.

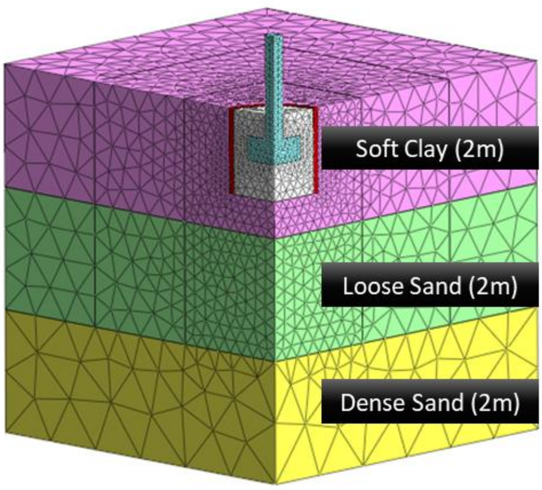


Figure 3-30 : Model with a 1.4 m deep Cylinder with quarry dust fill

3.15. Parametric study on use of concrete cylinders with quarry dust fill – consolidation analysis

This section discusses the parametric study conducted on the use of concrete cylinders with quarry dust fill (Figure 3-31) through a consolidation analysis. The parametric study investigates several factors: the effect of cylinder height on settlements, the impact of cylinder diameter on settlements, and the effect of pre-consolidation stress of clay on settlements. The study was done using pre-consolidation stresses of 250 kPa and 100 kPa for the clay soil. The study was conducted using the material properties displayed in Table 3-14. The construction stage setup discussed in Section 3.9 was used in this analysis, and the water table was maintained at 1 m below the ground.

Table 3-14: Material Properties

Type	Clay	Cylinder	Quarry dust	Dense Sand	Footing
Material Model	MCC	Elastic	Mohr-Coulomb	Mohr-Coulomb	Elastic
γ (kN/m3)	14.5	23	18	19	25
E (kN/m2)	5000	2.3×10^{7}	10000	55000	3.6×10^{7}
C (kN/m2)	-	-	4	1	-
ϕ^{O}	-	-	30	38	-
Poisson's Ratio	0.2	0.25	0.3	0.3	0.3
e_0	3	-	1	0.4	-
λ	0.22	-	-	-	-
k	0.03	-	-	-	-

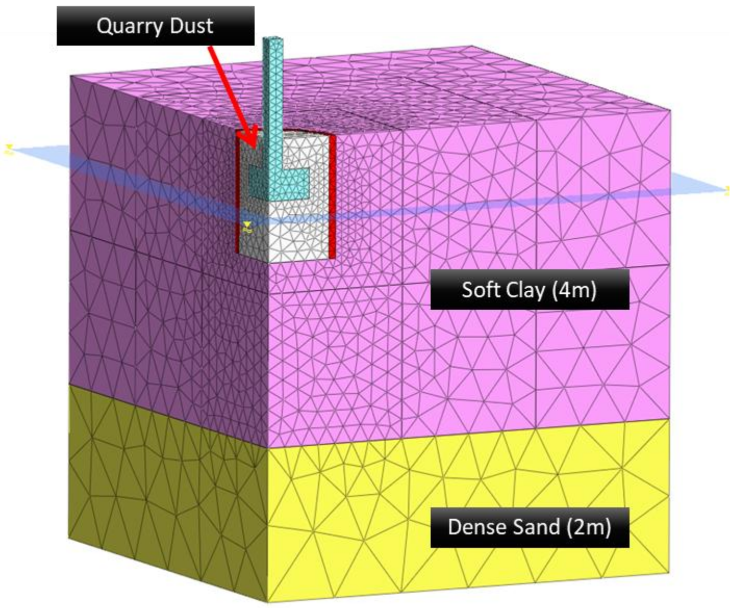


Figure 3-31: Model with Quarry dust-filled Cylinder

3.15.1. The impact of cylinder height on settlements

The analysis included several models, with the height of both the cylinder and the quarry dust fill varied simultaneously from 0.8 m to 2 m, while the top remained at ground level, and assessed for pre-consolidation stress values of 100 kPa and 250 kPa. Models featuring cylinder and quarry dust fill heights of 0.8 m and 2.0 m are illustrated in Figures 3-32 and 3-33, respectively.

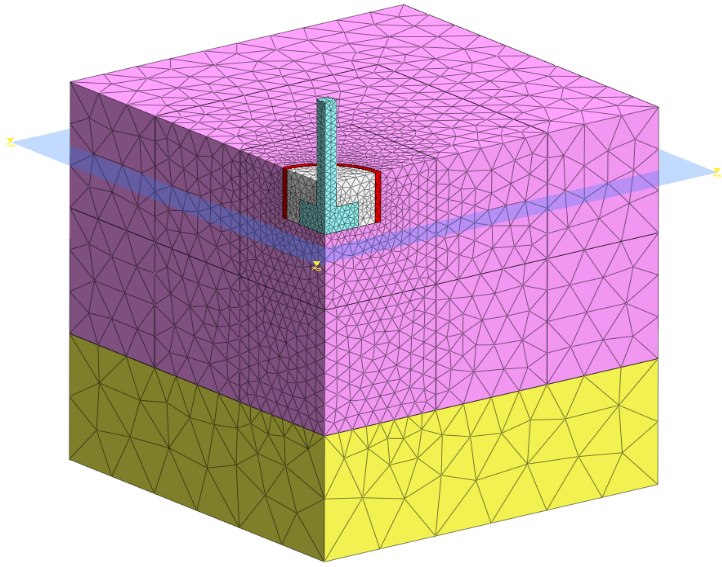


Figure 3-32: Model with the cylinder and quarry dust fill height of 0.8 m

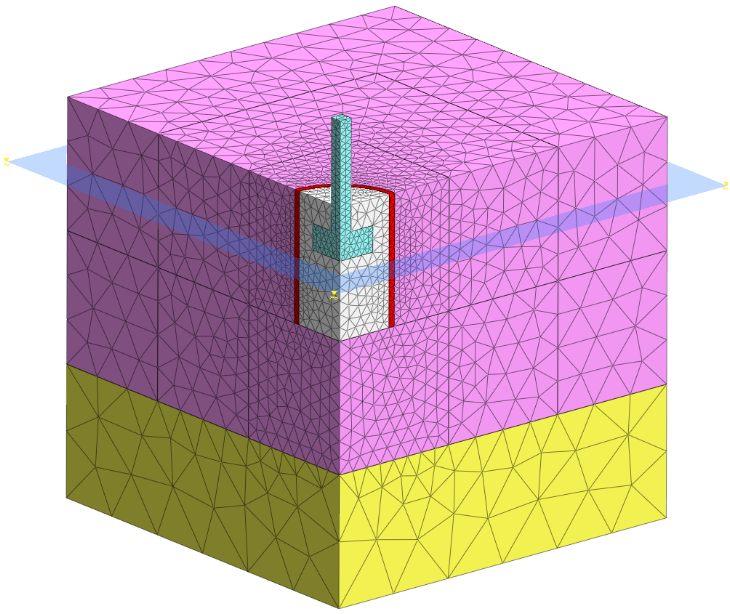


Figure 3-33: Model with the cylinder and quarry dust fill height of 2 m

3.15.2. The impact of cylinder diameter on settlements

The analysis included several models, with the diameter of the cylinder ranging from 1.8 m to 2.4 m, while the top remained level with the ground, and the height was maintained at 1.6 m. This study assessed pre-consolidation stress values of 100 kPa and 250 kPa. Models with cylinder diameters of 1.8 m and 2.4 m are shown in Figures 3-34 and 3-35, respectively.

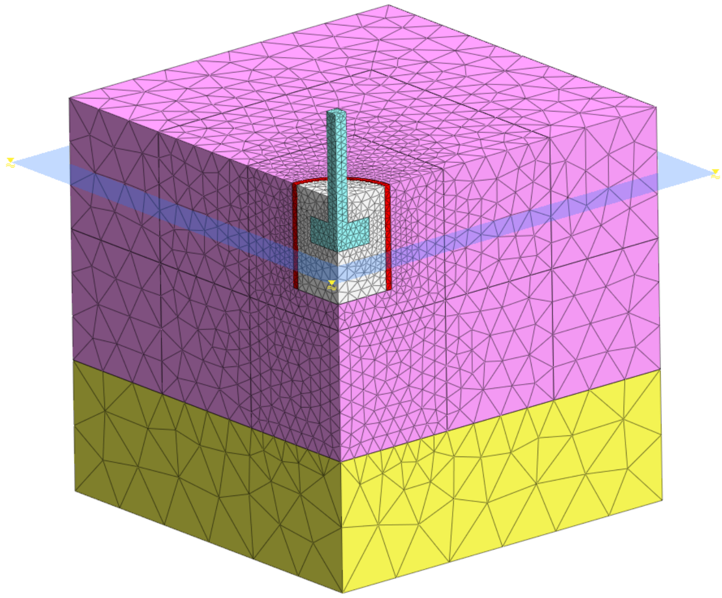


Figure 3-34: Model with the cylinder diameter of 1.8 m

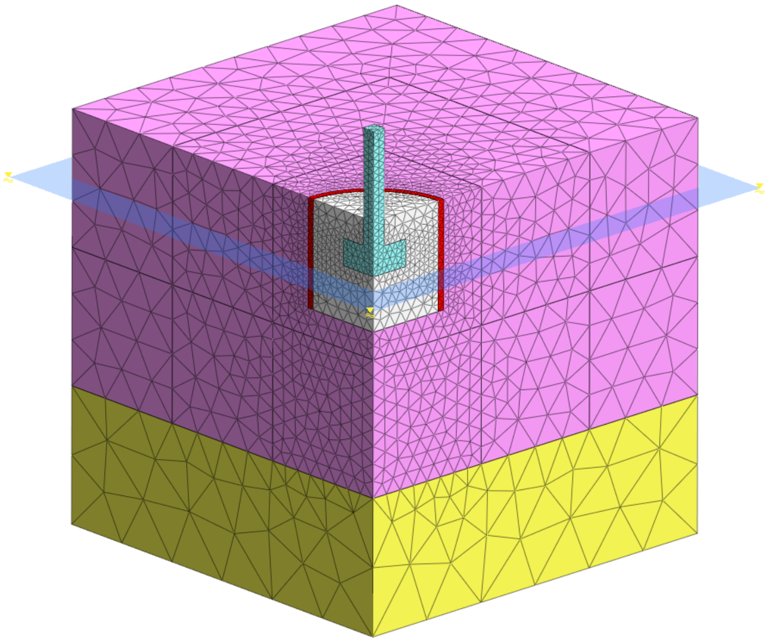


Figure 3-35: Model with the cylinder diameter of 2.4 m

3.16. Assumptions and limitations

Geometric Assumptions

A quarter model was employed with symmetrical boundaries, assuming the footing, loading, and soil layers are perfectly symmetric. This method is appropriate for uniform loading but does not capture asymmetric or irregular conditions. The soil was modelled as homogeneous layers ranging from 2 to 4 m thick with consistent properties, although natural soils usually exhibit gradual transitions and variability, which can affect settlement.

Material Model Assumptions

For stress analysis, the Mohr–Coulomb model was employed, assuming linear elastic, perfectly plastic, and isotropic behaviour. It does not account for strain-softening or stress-path dependency, resulting in slightly conservative outcomes. For consolidation, the Modified Cam–Clay model was used, assuming fully saturated, isotropic soil within the critical state framework. The model does not consider structured or anisotropic clay behaviour or principal stress rotation.

Hydraulic and Consolidation Assumptions

Permeability ($k = 1\times10^{-9}$ m/s) was treated as constant and isotropic ($k_h = k_v$). In practice, permeability decreases during consolidation, potentially extending the consolidation period.

Interface Assumptions

While perfect bonding was assumed at soil–CMC interfaces, in reality, partial slippage might occur, reducing the load transfer efficiency.

Ground Improvement Assumptions

Effects of CMC installation, like remolding, excess pore pressure, and smear zones, were not considered, which could lead to an overestimation of improvement efficiency. Columns were assumed to be perfectly vertical and uniform, and the load transfer platform was modelled as a homogeneous granular layer with consistent stiffness.

Numerical and Validation Assumptions

Mesh Set 2 was employed following convergence testing, with variations under 5%. Implicit time integration provided stability, although a detailed time-step validation was not carried out. Material parameters were sourced from literature and validated against theoretical solutions, primarily for homogeneous soil conditions. The sensitivity of OCR and the impact of variations in λ, κ, and M were not examined. Pore water pressure was included in the coupled formulation but not discussed in depth, since the primary focus was on settlement behaviour.

Summary

The FE analysis uses idealised soil behaviour, assumes constant permeability and homogeneous layers, and neglects installation effects. While such simplifications are typical in initial geotechnical models, they introduce some uncertainty. Future work should include site-specific calibration, consider installation effects, and implement field monitoring to verify settlement predictions and improve model precision.

CHAPTER 4: RESULTS

4.1. Introduction

This chapter discusses both the immediate and consolidation settlement behaviours of the footing, along with various aspects of concrete cylinders filled with quarry dust and the CMC technique, based on results predicted from FE analyses. Additionally, it describes the stress distribution and concentration on CMCs and concrete cylinders when applying the techniques mentioned above.

4.2. Comparison between full-scale model and quarter-sized model

The immediate vertical settlement versus the applied load graph of both the full-sized and quarter-sized models is shown in Figure 4-1, while the graph for the models incorporating concrete cylinders without quarry dust fill is displayed in Figure 4-2.

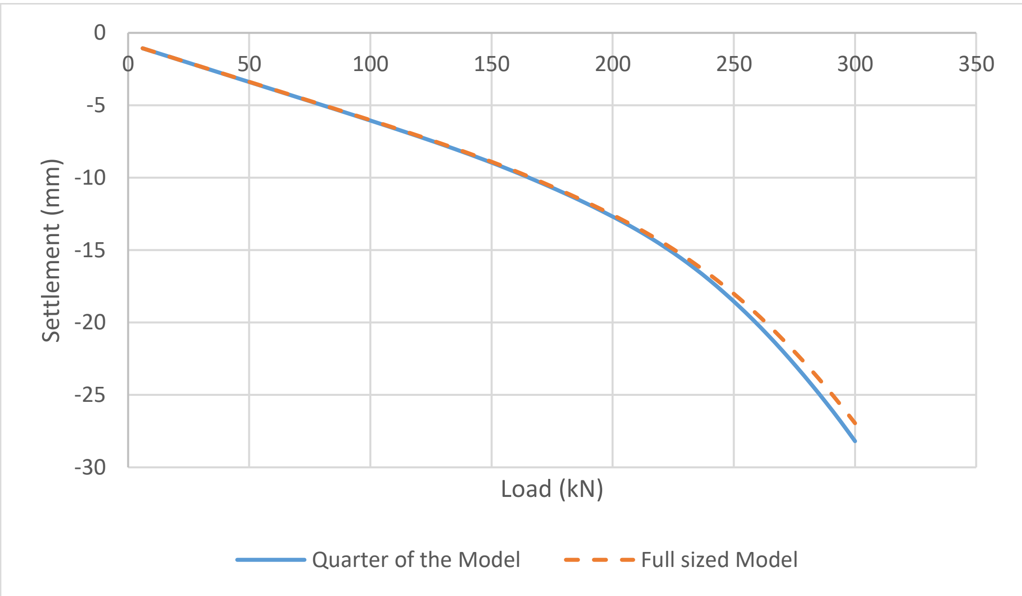


Figure 4-1 : Load vs settlement curves for both full and quarter models without concrete cylinders

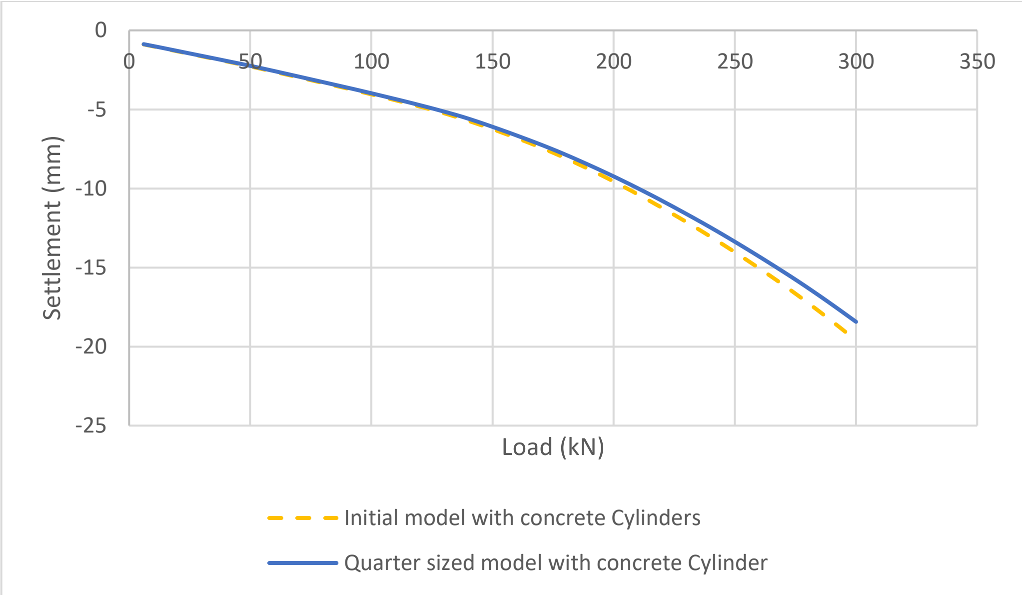


Figure 4-2 : Vertical settlement versus load curves for both full and quarter models with the implementation of concrete cylinders

The above graphs demonstrate that the results obtained from both models are nearly identical. Therefore, to reduce computational complexity and time, the quarter-sized model was selected for most of the further analysis.

4.3. Mesh sensitivity analysis

The immediate vertical settlement versus the applied load graph for the model with mesh sizes as described in Table 4-1 is shown in Figure 4-3.

Table 4-1 : Mesh Sizes

Set Number	Layer Number	Size (m)		
		Part 1	Part 2	Part 3
1	1	0.1	0.25	0.5
	2	0.2	0.4	0.65
	3	1	-	-
2	1	0.2	0.4	0.6
	2	0.25	0.5	0.75
	3	1	-	-
3	1	0.3	0.5	0.7
	2	0.4	0.7	0.8
	3	1	-	-

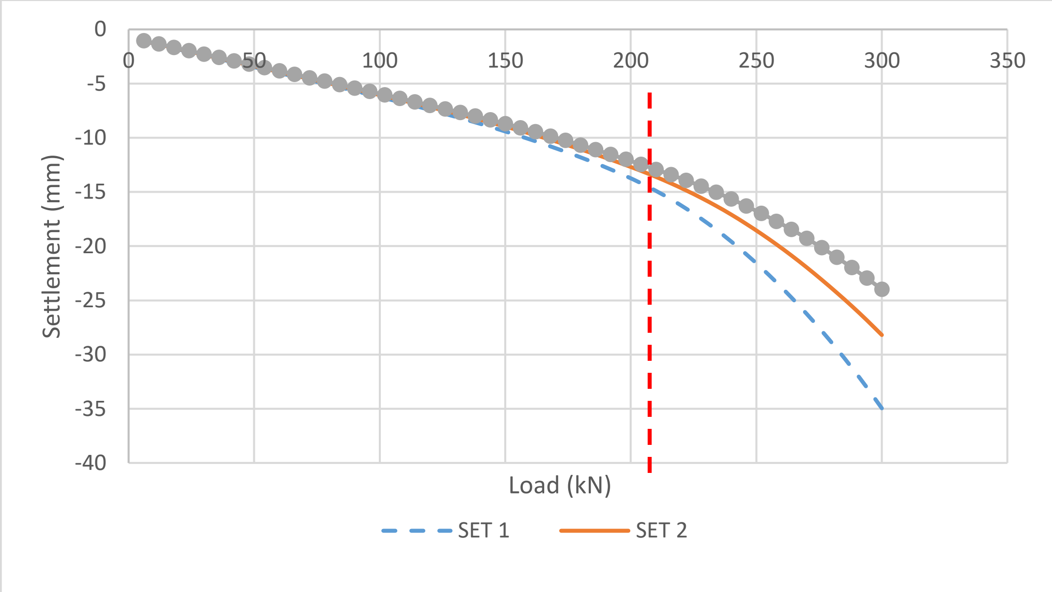


Figure 4-3 : Load vs Settlement curves for different mesh sizes

The graph indicates that within the linear range of the curves, mesh sensitivity for different mesh sizes is minimal. Since this research does not consider data beyond the bearing capacity (208 kN), which is indicated by the dotted line in the graph, mesh size set 2 was chosen for further analysis. This decision was based on the understanding that mesh size set 2 provides an adequate level of accuracy without significantly increasing computational time.

4.4. Three-dimensional model validation using stress analysis

The applied load versus vertical settlement curves, derived from theoretical calculations (using Janbu's equation) and predicted values from the FE model, are shown in Figure 4-4. It is clear that the settlements predicted by the model, before reaching the bearing capacity limit indicated by the dotted line in the graph, closely match the theoretical values, thereby confirming the model's validity. The reason for the variation is that, after reaching the bearing capacity, the soil will experience plastic deformation, whereas the equation only calculates elastic deformations.

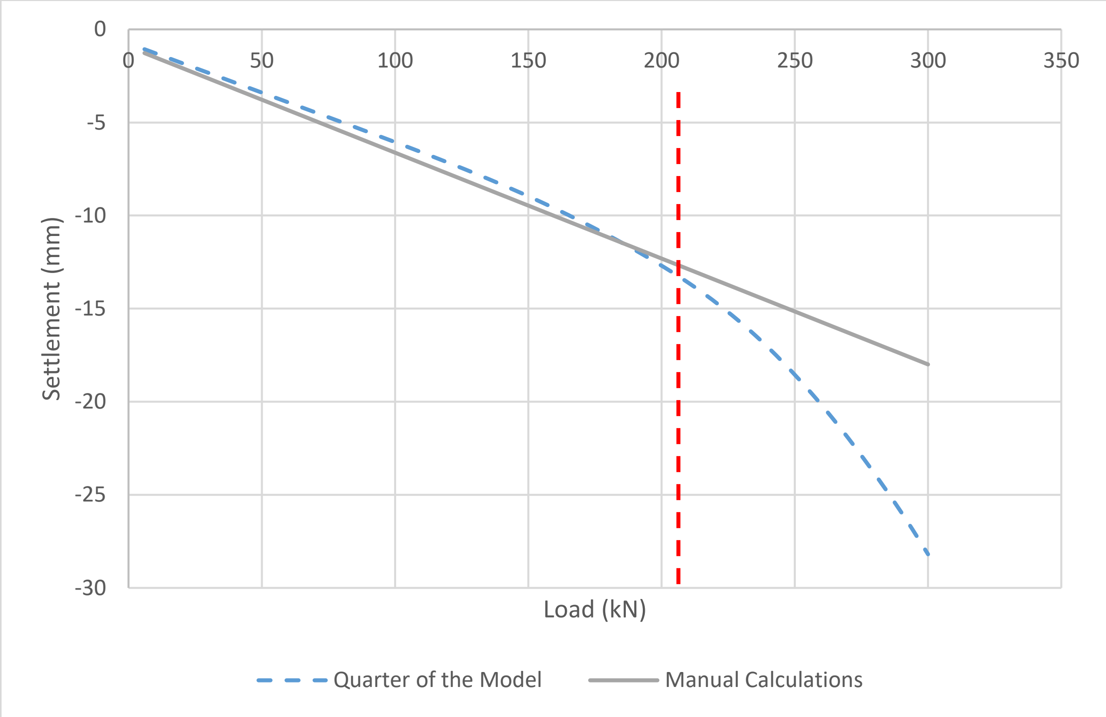


Figure 4-4: The applied load versus vertical settlement curves, derived from theoretical calculations and predicted settlement by the FE model

Figure 4-5 shows the applied load versus vertical settlement curves for Footing 1, comparing the predicted values from the FE model with the field test data, as outlined in Section 3.6.2. Clearly, the model's predictions of settlement closely match the field test values, confirming the model's validity.

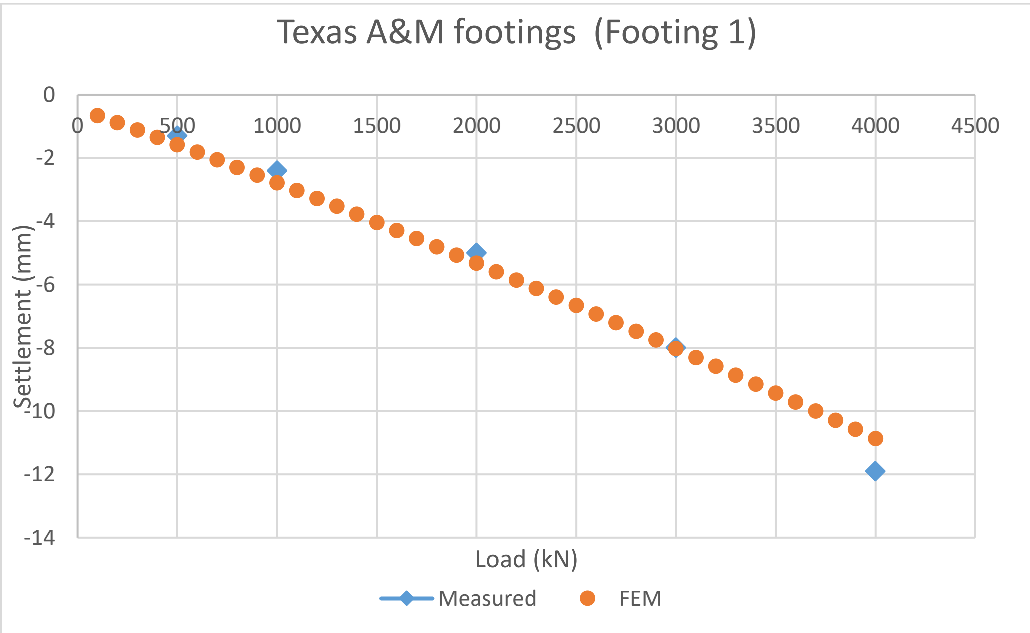


Figure 4-5: FE model and field test data of Footing 1

The curves depicting the relationship between applied load and vertical settlement are shown in Figure 4-6. These curves include values predicted by the FE model and field test data from the loading test conducted at Green Cove Springs, Florida, as described in section 3.6.3. It is clear that the model-predicted settlements closely match the field-test values obtained from the Dilatometer Test (DMT), thereby affirming the model's validity. The FEM's modest underprediction of settlements aligns with expectations for numerical models calibrated using idealized soil parameters and boundary conditions. Despite this small discrepancy, the strong correlation between the FEM and DMT results suggests that the model accurately reflects the key soil–structure interaction behaviour and remains a valid and dependable tool for predictive analysis.

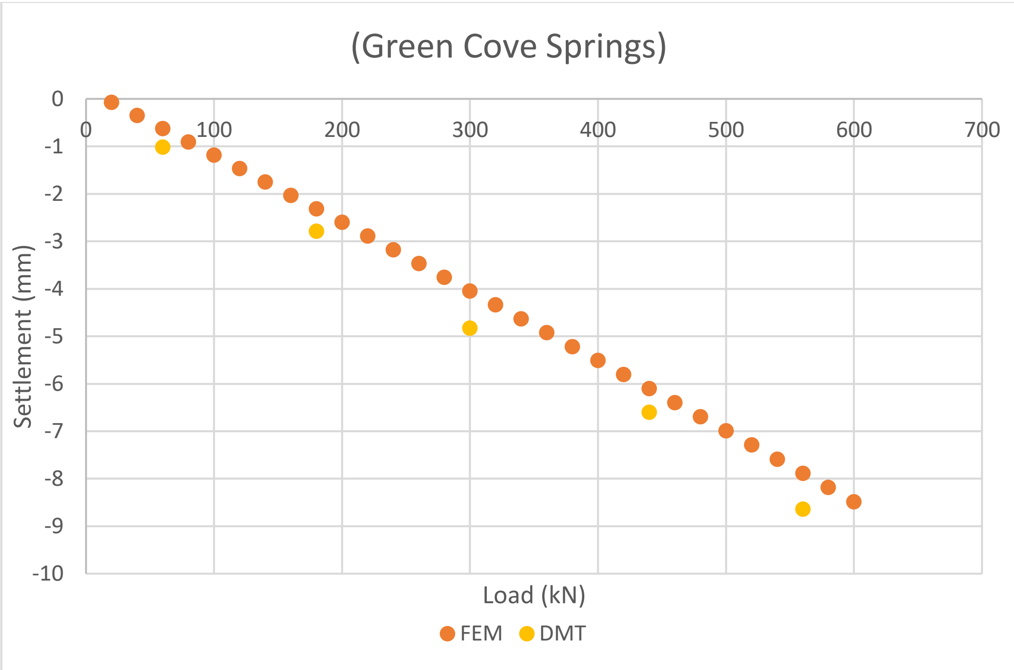


Figure 4-6: Applied load and vertical settlement

4.5. Three-dimensional model validation using consolidation analysis

The curves illustrating the relationship between Time and vertical settlement are shown in Figure 4-7. These curves include values predicted by the FE model as well as laboratory model test data from the test carried out at the University of Moratuwa, as described in section 3.7. The graphs indicate that within the embedment depth range of 0.1 m to 0.5 m, settlements decrease markedly due to increased confinement and elevated effective stress in the surrounding soil, which enhances stiffness and diminishes compressibility. Beyond 0.5 m, the reduction in settlement proceeds gradually, as additional depth offers limited improvements in stress distribution and confinement. From a soil mechanics perspective, this signifies a point of diminishing returns where the influence zone of the foundation is already adequately confined, and further embedment results in only marginal gains in stiffness and settlement mitigation.

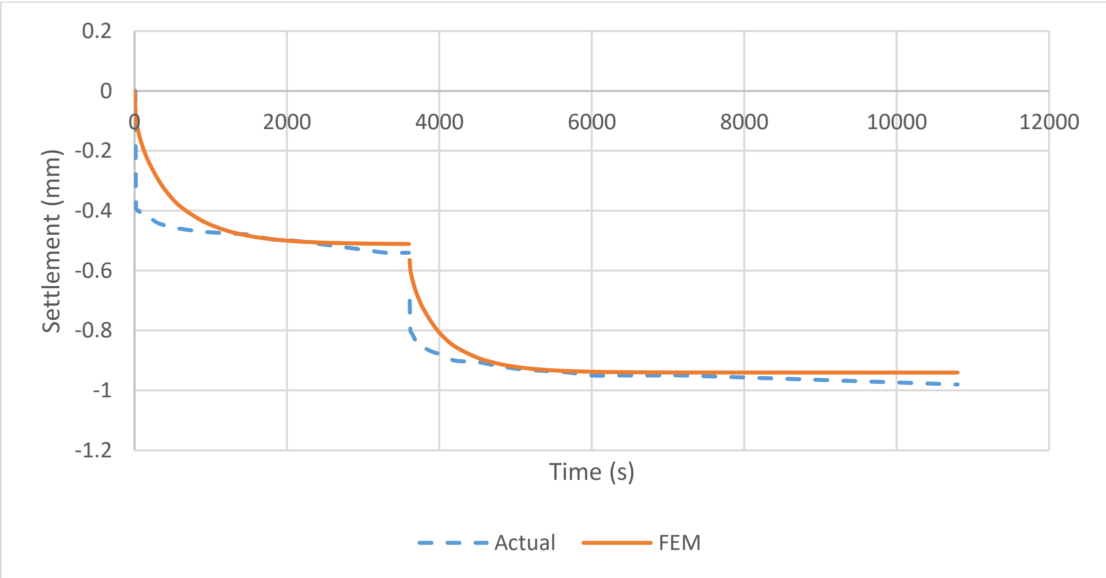


Figure 4-7: Time vs vertical settlement for Lab model test

4.6. Impact of embedment depth of the footing - stress analysis

The relationship between settlement and embedment depth was analysed for both the 2 m deep clay model and the 4 m deep clay model. The results of these analyses are presented in Figures 4-8 to 4-11, which illustrate the relationship between settlement and embedment depth across several models.

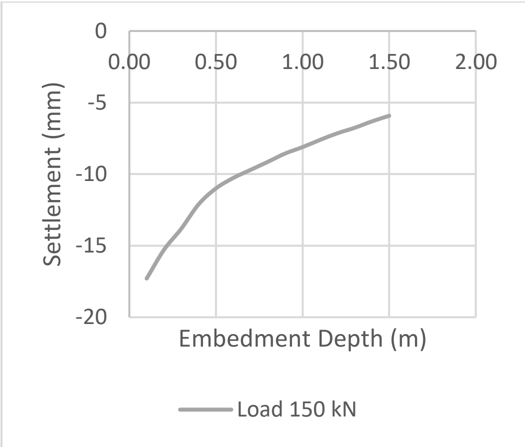


Figure 4-8 : Settlement vs Embedment depth for 150kN load (2m Clay)

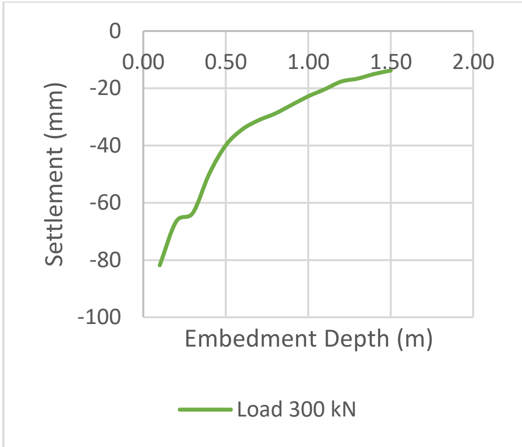


Figure 4-9 : Settlement vs Embedment depth for 300kN load (2m Clay)

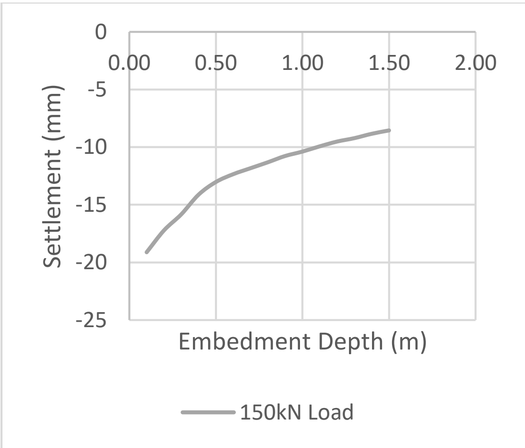


Figure 4-10 : Settlement vs Embedment depth for 150kN load (4m Clay)

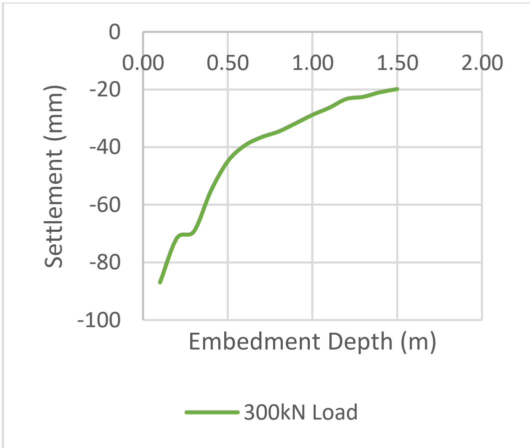


Figure 4-11 : Settlement vs Embedment depth for 300kN load (4m Clay)

The curves shown above illustrate that within the embedment depth range of 0.1 m to 0.5 m, there is a steep reduction in settlements. However, for depths greater than 0.5 m, the decline in settlements becomes increasingly gradual. Figure 4-12 shows the comparison between the embedment depth and settlement curves for a 2 m-thick and a 4 m-thick clay layer, each subjected to a 300 kN load.

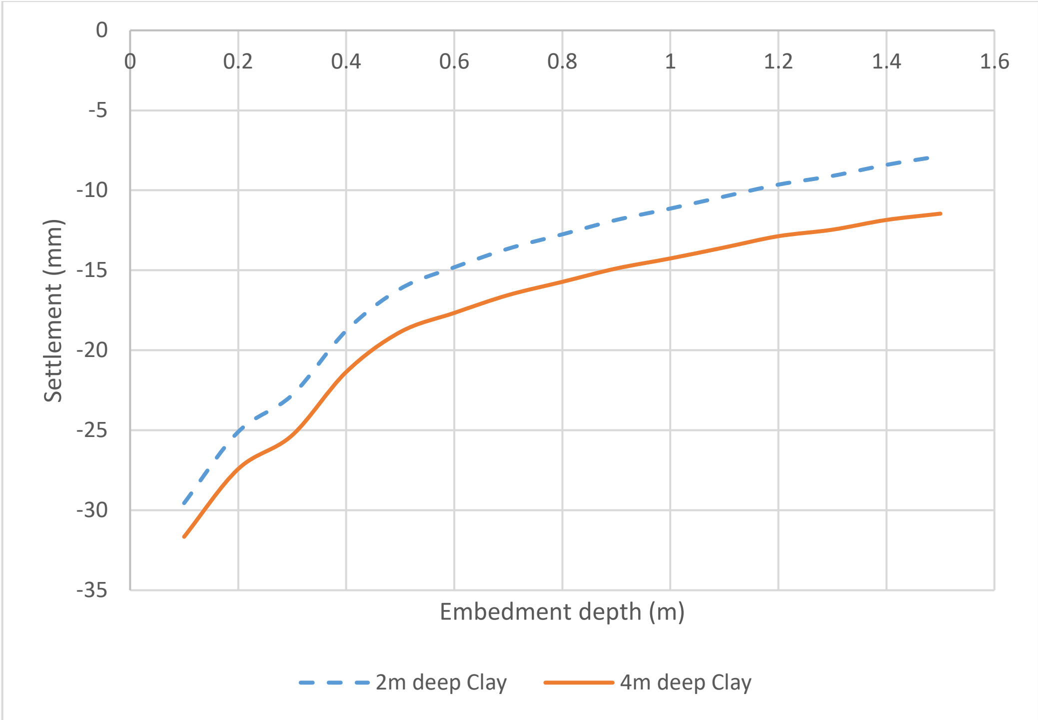


Figure 4-12: Embedment depth versus settlement curves

Load versus settlement curves for various embedment depths of the footing, obtained from the model with a 2m thick clay layer, are illustrated in Figure 4-13.

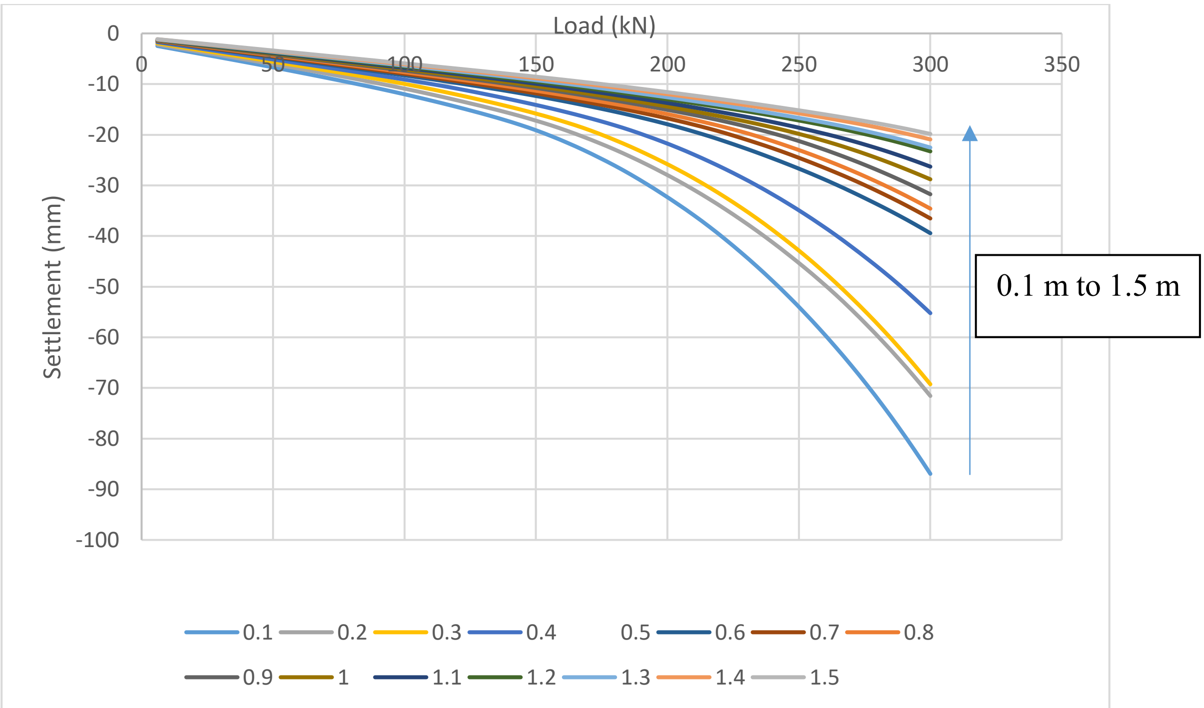


Figure 4-13: Load versus settlement curves for various embedment depths

The graph shown above provides important insights into how embedment depth relates to the linearity of the load versus settlement curve, which ultimately influences bearing capacity. It is clear that as embedment depth increases, the load versus settlement curves tend to become more linear, indicating a rise in bearing capacity.

Based on this observation, it is clear that for domestic buildings with a typical footing stress range of 150-175 kN/m², choosing the right embedment depth is crucial. The graphs above indicate that an embedment depth of between 0.6 m and 1 m is optimal for a 1.2 m wide square footing.

4.7. Impact of embedment depth of the footing - consolidation analysis

The relationship between consolidation settlement and embedment depth was analysed for the 4 m deep clay model. This study was conducted for pre-consolidation stresses of 250 kPa and 100 kPa, as explained in Section 3.9, and settlement versus time curves are illustrated in Figures 4-14 and 4-15, respectively.

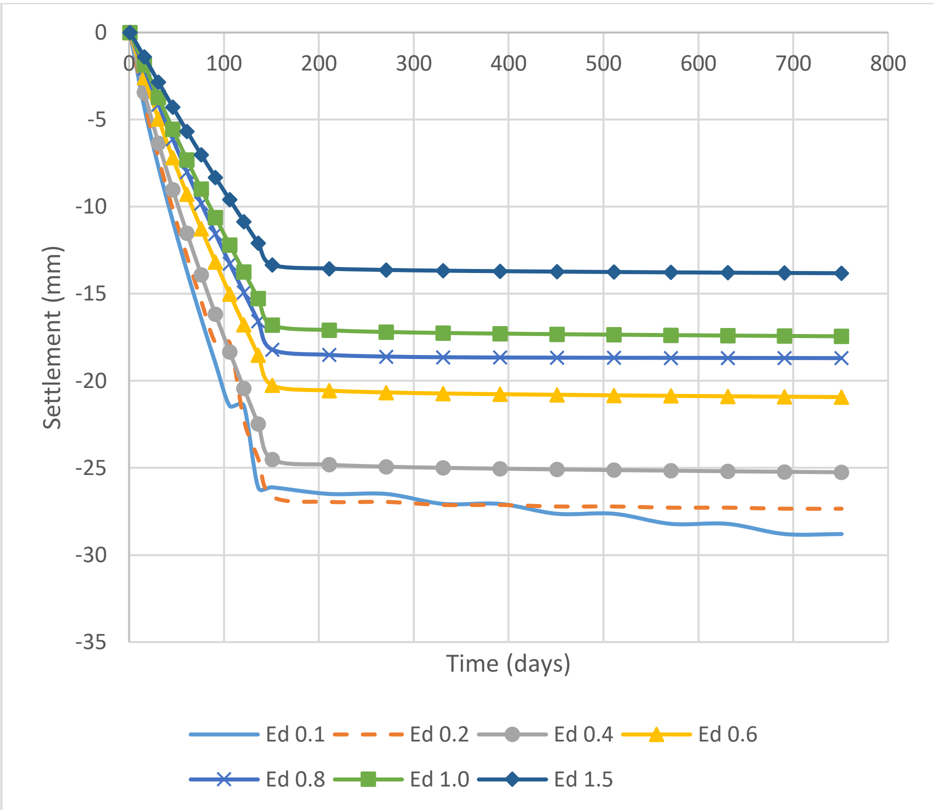


Figure 4-14: Time vs Settlement (Pc – 250 kPa)

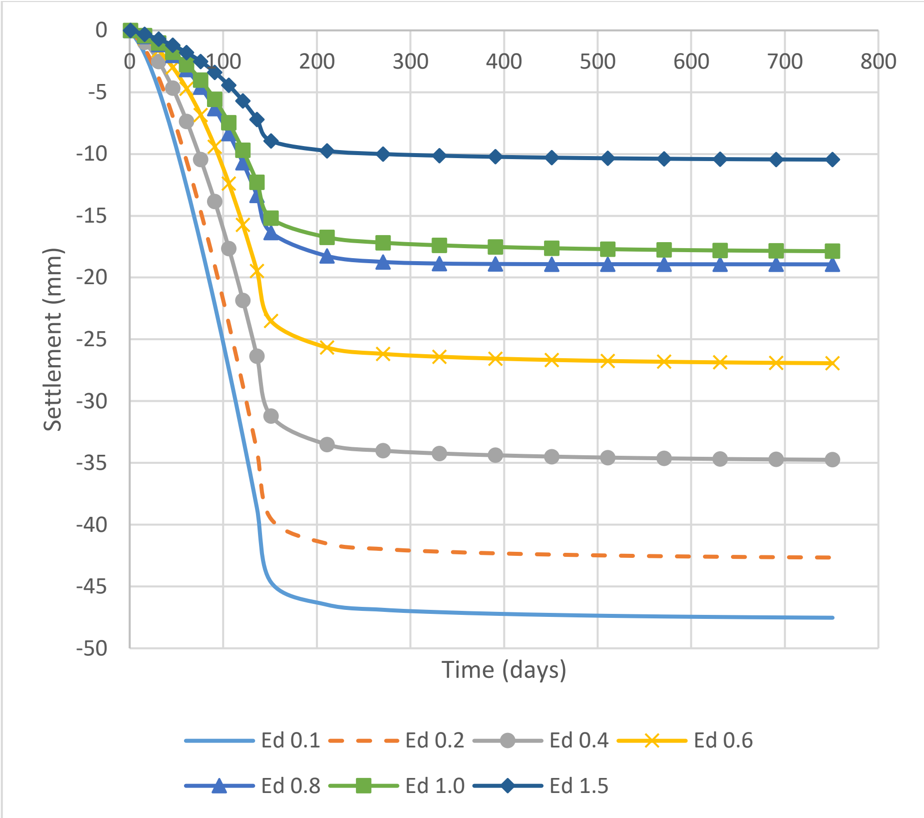


Figure 4-15: Time vs Settlement (Pc – 100 kPa)

It is evident that embedment depth significantly influences settlement reduction in the pre-consolidation stress of 100 kPa. Since the applied load is 208 kPa (300 kN), we can conclude that the effect of embedment depth is much more crucial for normally consolidated clay than for over-consolidated clay.

Embedment depth versus settlement at the end of the loading stage (150Th day) curves for pre-consolidation stresses of 100 kPa and 250 kPa are illustrated in Figure 4-16, while curves at 750th day are shown in Figure 4-17. It is observed that settlements decrease as the embedment depth increases, and beyond the 0.6m to 0.8m range, this effect diminishes, indicating that the optimal embedment depth for a 1.2 m × 1.2 m footing lies within this range. Beyond an embedment depth of 0.6 m to 0.8 m, the majority of the compressible upper soil is already bypassed. Therefore, additional depth offers only a marginal decrease in settlement, as the footing predominantly interacts with the more rigid soil strata.

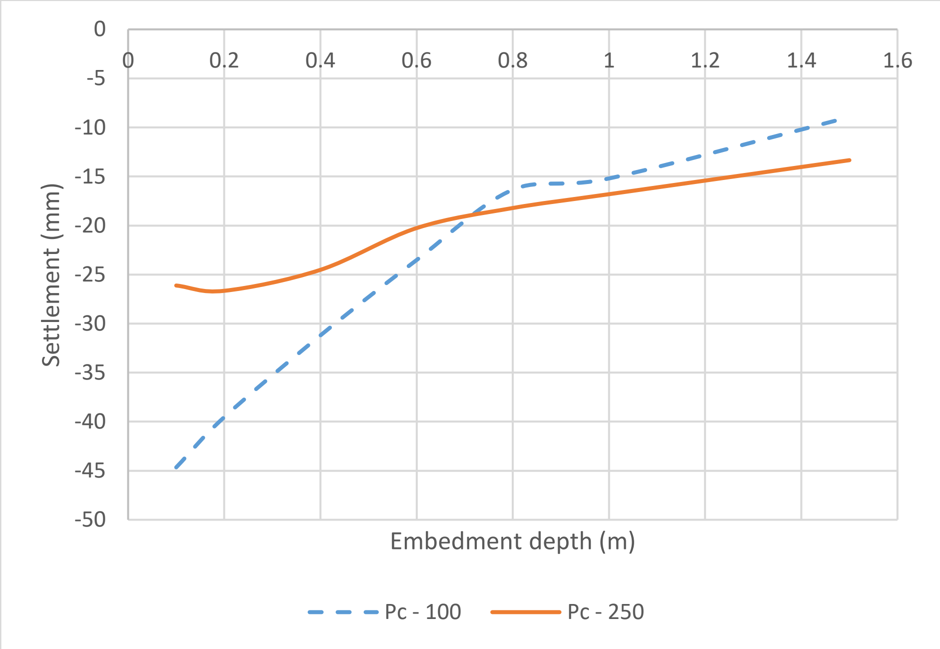


Figure 4-16: Embedment depth versus settlement at the end of the loading stage

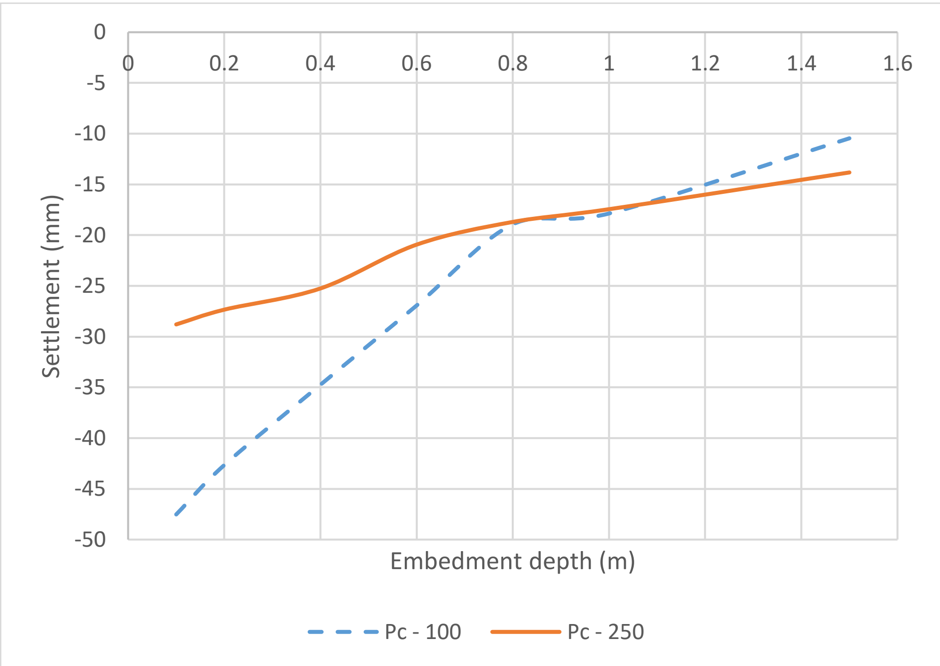


Figure 4-17: Embedment depth versus settlement at the end of Day 750

Another study was conducted using the model with a pre-consolidation stress of 250 kPa to investigate the effect of the initial water table level on settlements. The water level versus settlement graph is shown in Figure 4-18, while the time versus settlement curves for different water levels are presented in Figure 4-19. These curves indicate that when the water level decreases (measured from the ground level), settlements also decrease. However, the rate of decrease diminished, and after 2 m below ground level, settlements showed no further impact. Beyond approximately 2 meters, the soil is less influenced by the applied load due to the dissipation of vertical stress with depth, and the alterations in pore pressure become negligible. Below this depth, the soil behaves like an undisturbed medium, with minimal variations in load-induced stress, resulting in negligible settlement changes attributable to pore pressure effects beyond this threshold.

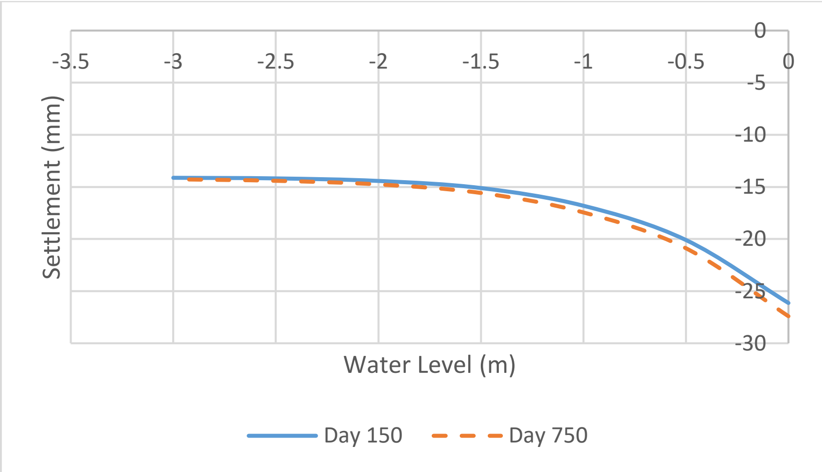


Figure 4-18: The water level versus settlement

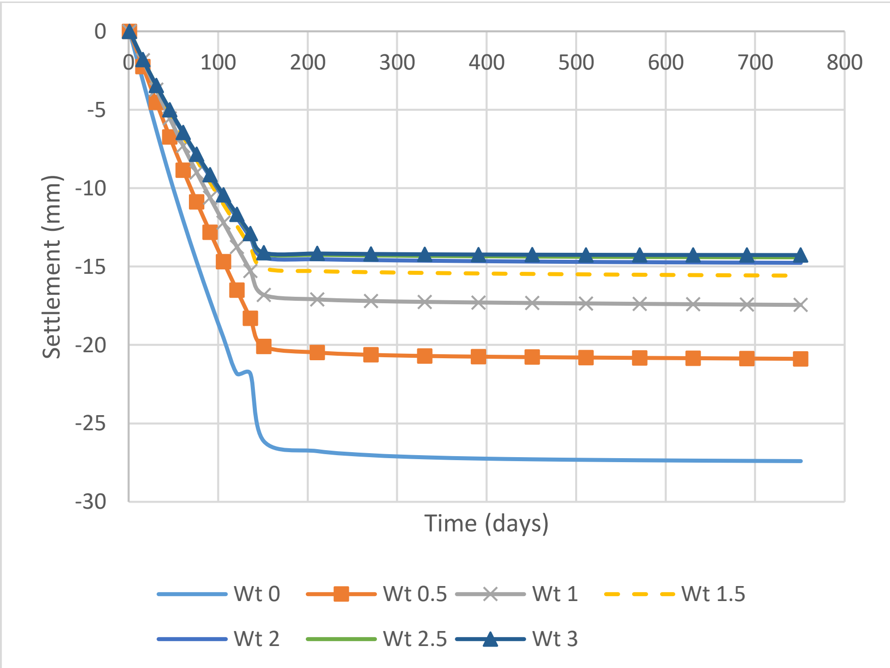


Figure 4-19: Time versus settlement curves for different water levels

4.8. Impact of controlled modulus columns (CMC) – stress analysis

Applied load versus settlement curves, with and without CMCs from stress analysis conducted as mentioned in Section 3.10, are shown in Figure 4-20. The graph demonstrates the improvement in bearing capacity achieved by using CMCs and the resulting reduction of settlements.

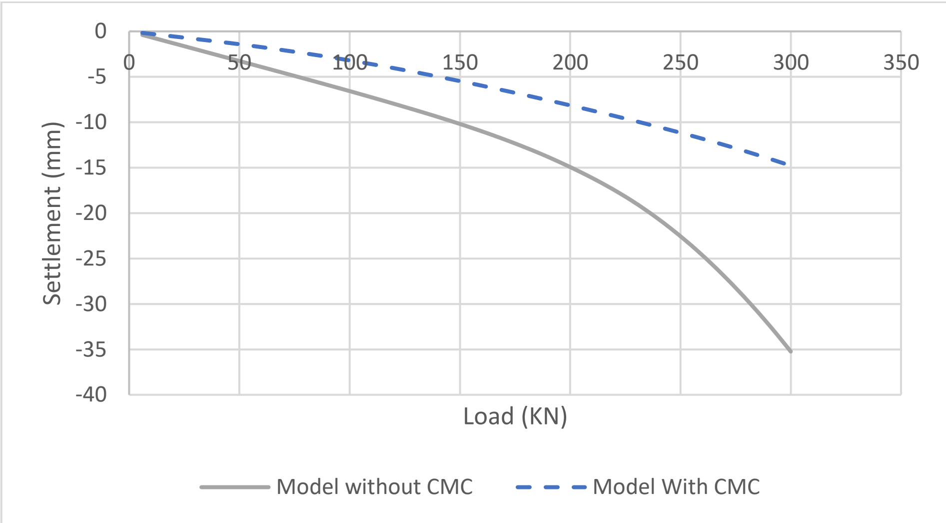


Figure 4-20: Load versus settlement curves, with and without CMCs

Figures 4-21 and 4-22 illustrate the vertical stress distribution of the models without and with the use of CMCs, respectively. These graphics illustrate how CMCs will distribute stress to deeper layers while maintaining the load-sharing mechanism.

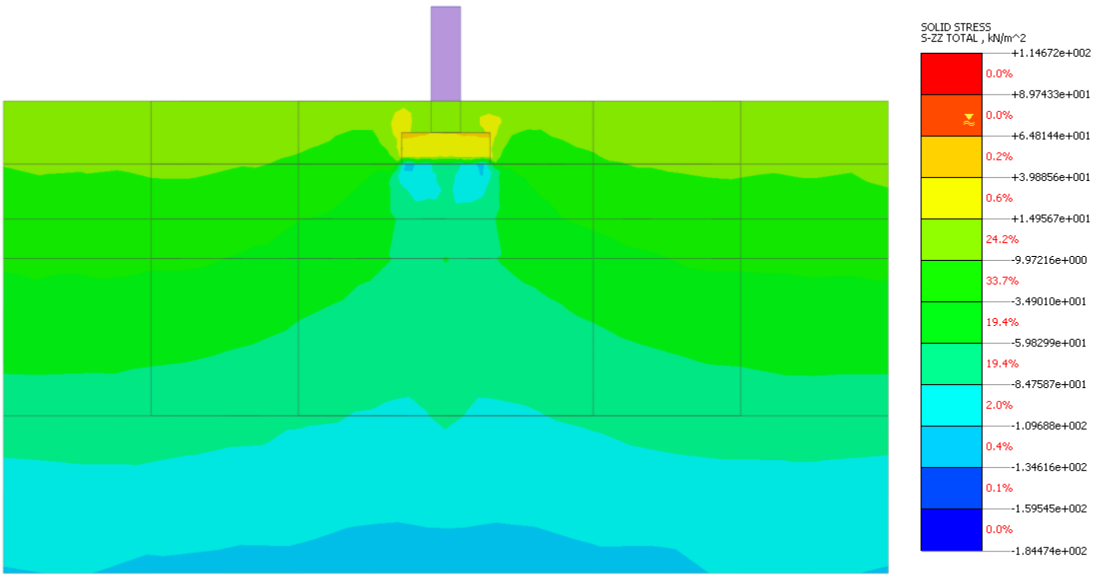


Figure 4-21: Distribution of vertical stress in the model without CMC

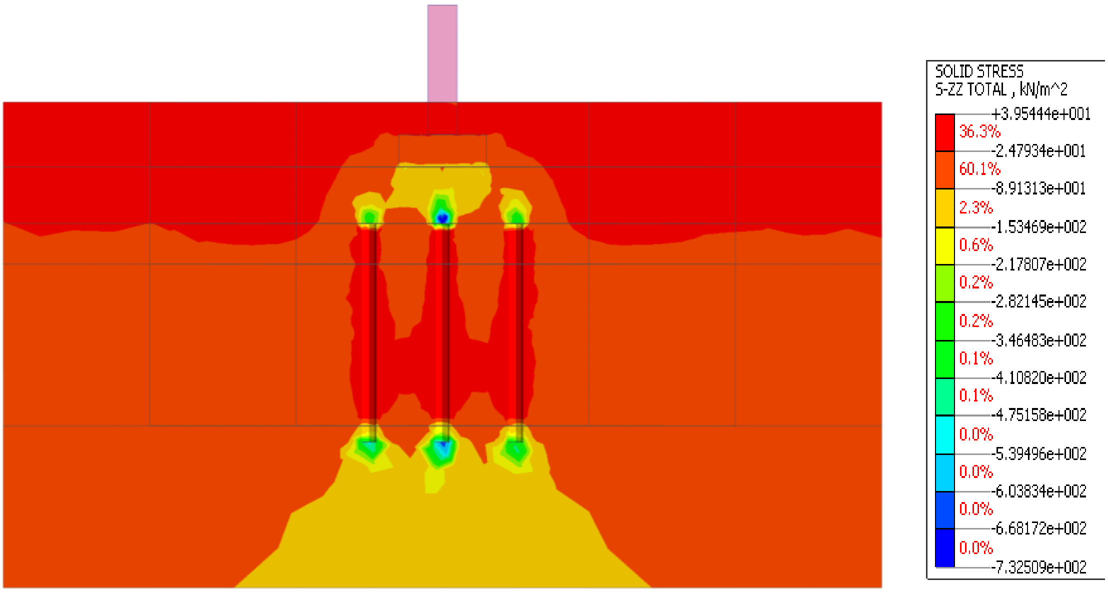


Figure 4-22: Distribution of vertical stress in the model with CMC

4.9. Impact of controlled modulus columns (CMC) – consolidation analysis

Time versus settlement curves, with and without CMCs from consolidation analysis conducted as mentioned in Section 3.11, are shown in Figure 4-23. The graph demonstrates the improvement in bearing capacity achieved by using CMCs and the resulting reduction of long-term settlements by 65%.

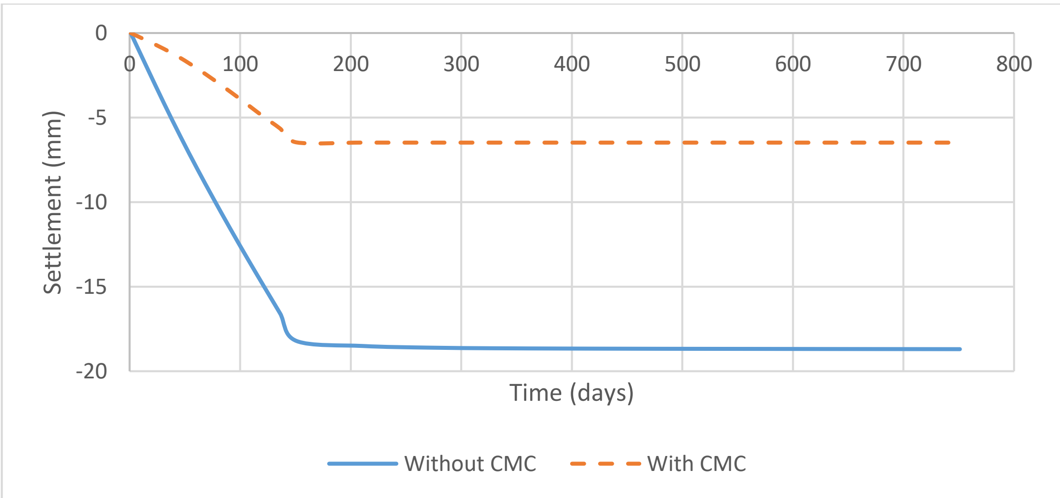


Figure 4-23: Time versus settlement curves, with and without CMCs from consolidation analysis

Figures 4-24 and 4-25 show the vertical stress distribution of the models without and with CMCs, respectively, based on the consolidation analysis. These graphics show

how CMCs distribute stress to deeper layers, minimising the impact on the soft soil layer, while maintaining the load-sharing mechanism.

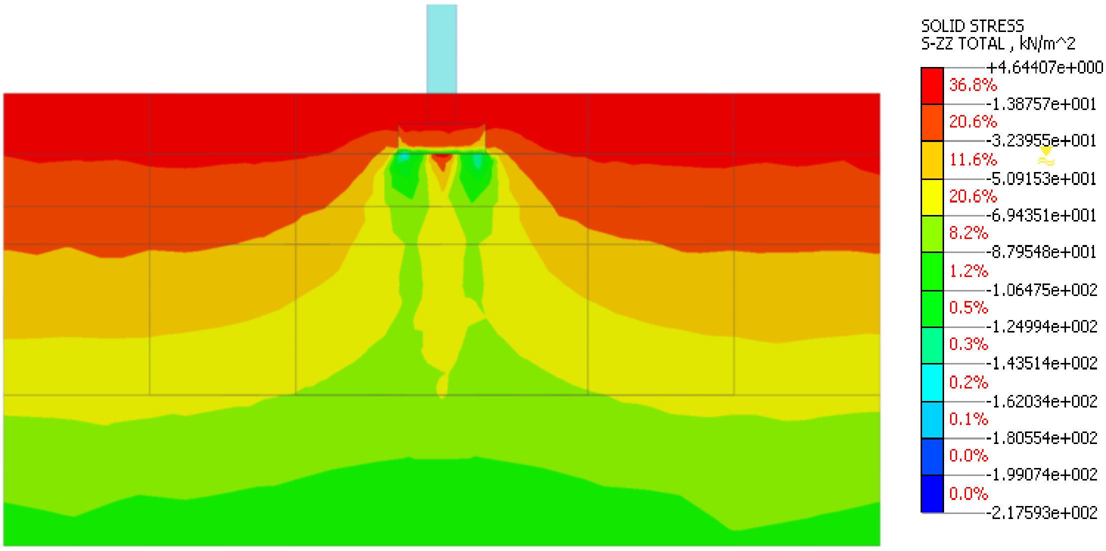


Figure 4-24: Distribution of vertical stress in the model without CMC (Consolidation Analysis)

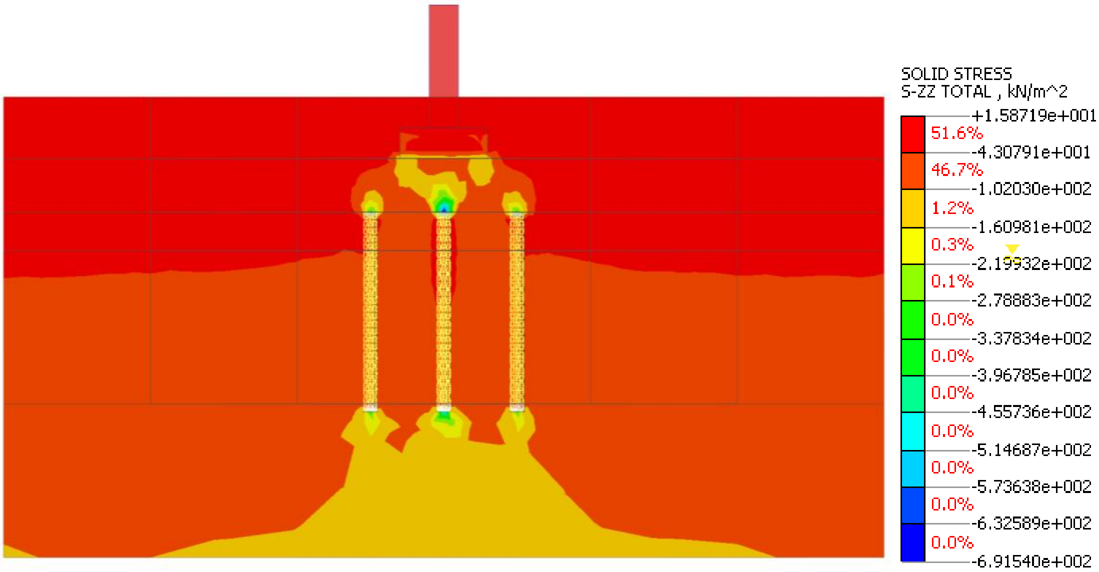


Figure 4-25: Distribution of vertical stress in the model with CMC (Consolidation Analysis)

4.10. Parametric study on CMC technique – consolidation analysis

The relationship between CMC height and settlement is shown in Figure 4-26, which displays the results from the analysis described in Section 3.12.1. Even though the graph indicates a reduction in settlement with increasing CMC height, a 0.5 m increment results in only a 3-4% decrease in settlement. This suggests that increasing the height of CMCs that are already near the hard soil layer has a limited effect.

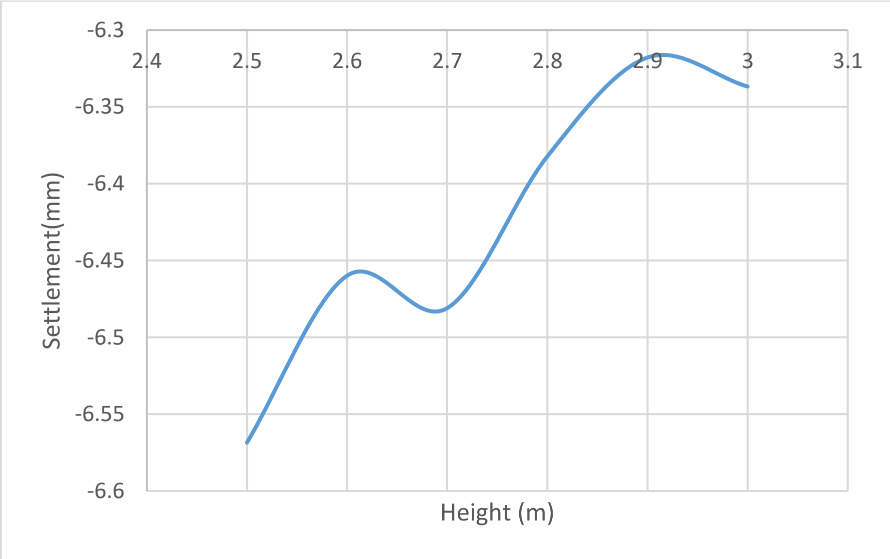


Figure 4-26: The relationship between CMC height and settlement

The relationship between CMC diameter and settlement is illustrated in Figure 4-27, which presents the results from the analysis described in Section 3.12.2. The graph shows a decrease in settlement as the CMC diameter increases, with a 0.3 m increase leading to a 37% reduction in settlement. This indicates that increasing the diameter of CMCs has a greater impact than increasing their height.

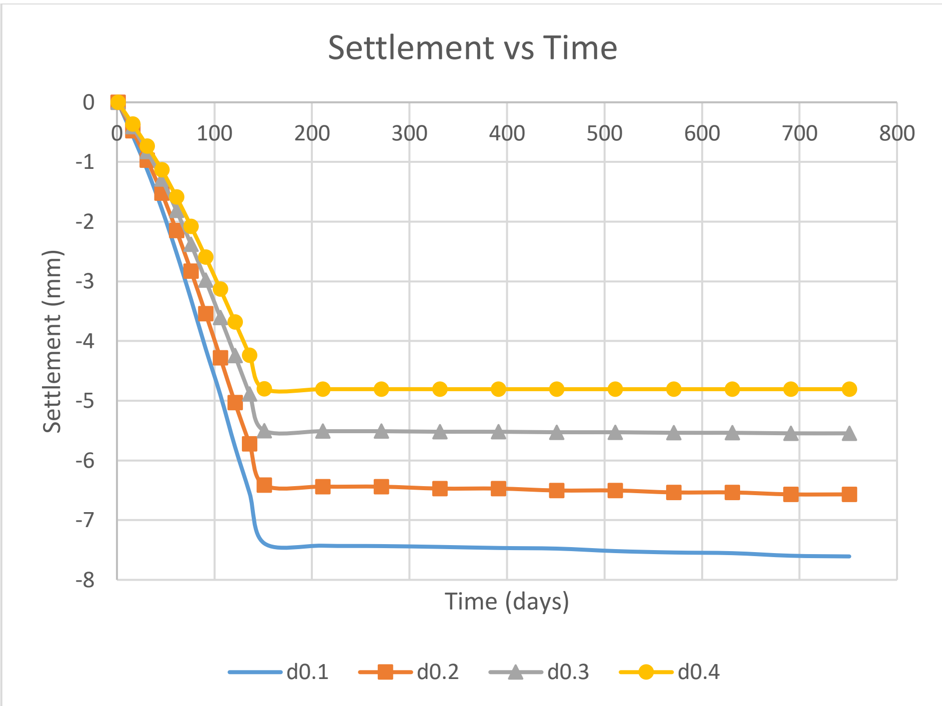


Figure 4-27: The relationship between CMC diameter and settlement

Figure 4-28 shows the relationship between CMC spacing and settlement, presenting the results from the analysis described in Section 3.12.3. The graph indicates that settlement decreases as CMC spacing decreases, with a 0.6 m reduction leading to a 28% decrease in settlement. The rate of increase in settlement slows after 1 m.

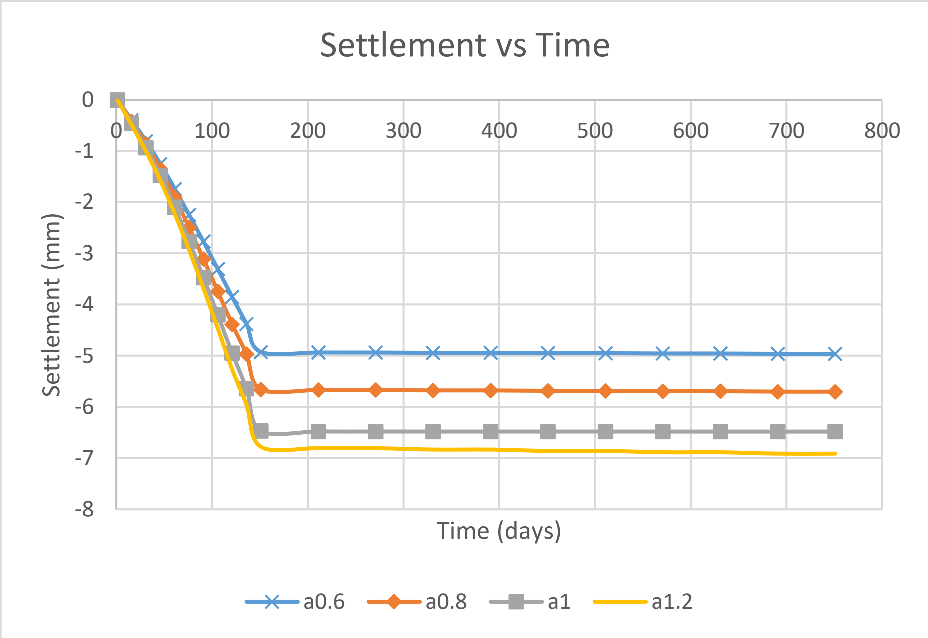


Figure 4-28: The relationship between CMC spacing and settlement

The relationship between LTP thickness and settlement is shown in Figure 4-29, which displays the results from the analysis described in Section 3.12.4. The graph indicates that settlement decreases as LTP thickness increases; specifically, increasing the thickness from 0.3 m to 0.9 m results in a 35% reduction in settlement, although the rate of reduction lessens.

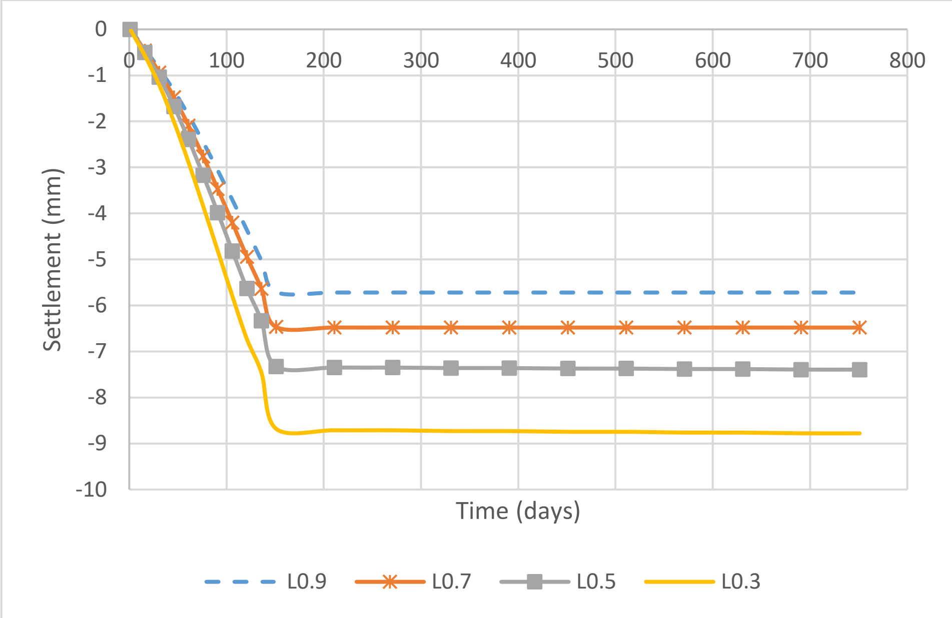


Figure 4-29: The relationship between LTP thickness and settlement

4.11. Impact of using concrete cylinders – stress analysis

The Load versus settlement curves for a 0.8 m high cylinder and without a cylinder are compared and shown in Figure 4-30. The graph in Figure 4-31 shows the change in settlement with the height of the concrete cylinder. A clear observation from these curves is that increasing the depth of the cylinder significantly reduces settlement up to a certain point. It is clear from the graph that beyond a height of around 1.2m, there is no noticeable effect on settlement. This suggests that above this height, the settlement remains relatively constant, and further increasing the cylinder's height may not lead to a substantial decrease in settlement.

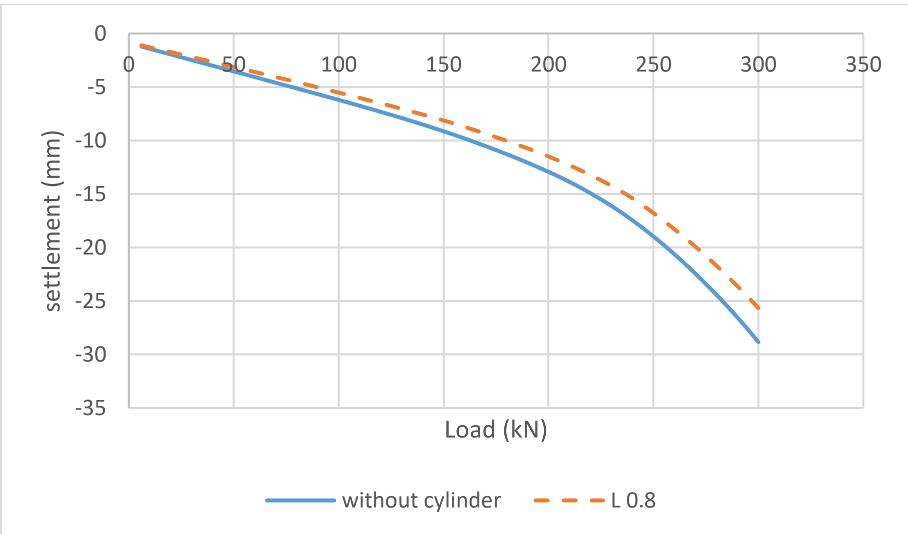


Figure 4-30 : Load vs settlement for with and without cylinder

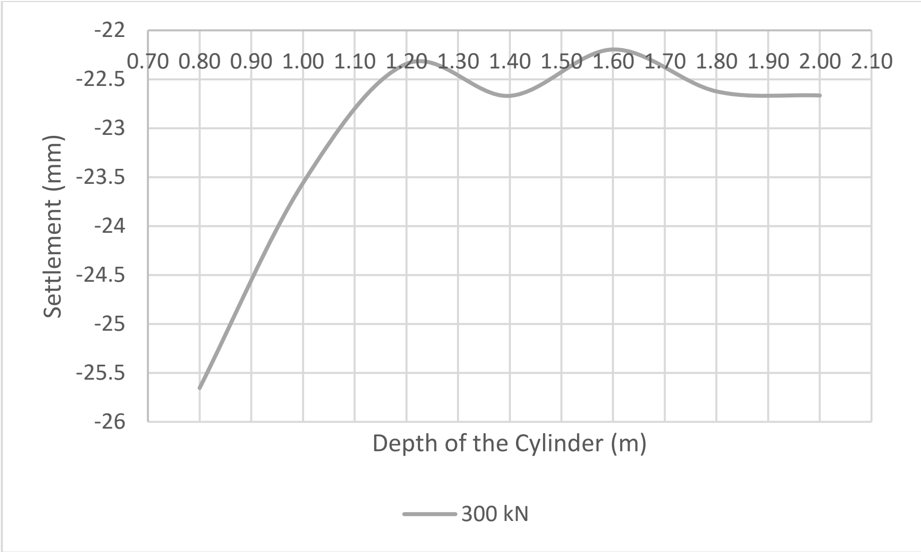


Figure 4-31 : Variation of settlement with cylinder depth for a load of 300kN

Figures 4-32 and 4-33 illustrate the stress distribution for models with and without cylinders. These visual representations offer valuable insights into how stress distribution varies. A key observation from the figures is that adding cylinders causes a redistribution of stress to greater depths.

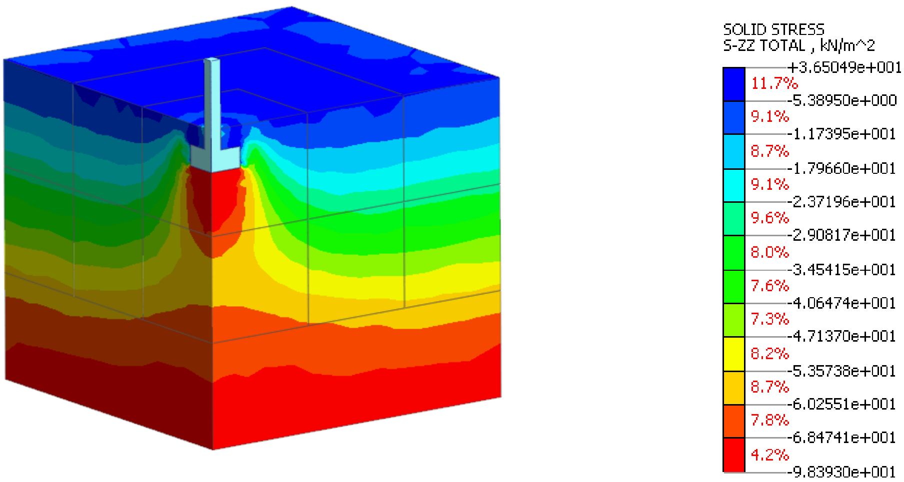


Figure 4-32: Stress distribution without cylinder

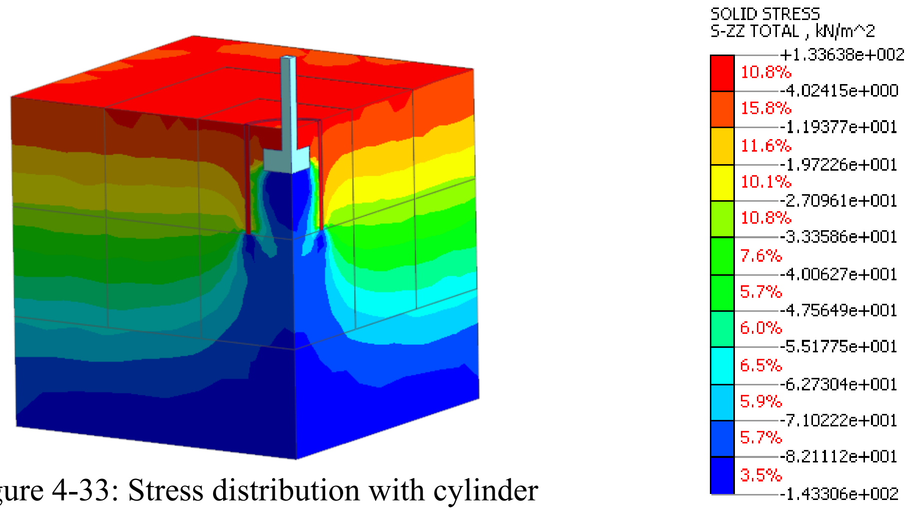


Figure 4-33: Stress distribution with cylinder

4.12. Impact of using quarry dust as a soil substitute inside concrete cylinders – stress analysis

The Load versus Settlement curves for systems without and with quarry dust replacement of 1.2 m, as explained in Section 3.14, are presented in Figures 4-34. It can be observed that the replacement of quarry dust significantly reduces settlement and increases bearing capacity. The graph shown in Figure 4-35 illustrates the variation of settlement with the cylinder and replacement height. The reduction in settlement increases with the height of the quarry dust layer.

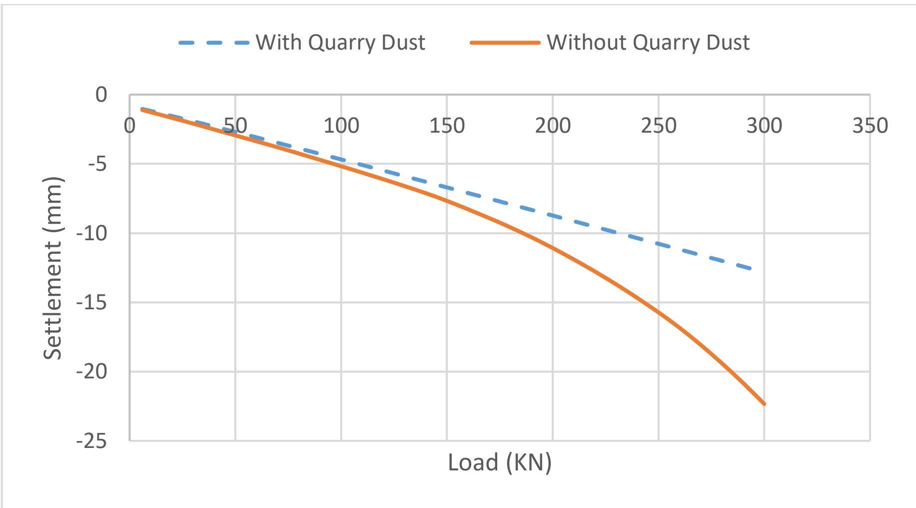


Figure 4-34 : Load vs Settlement with and without quarry dust replacement 1.2m

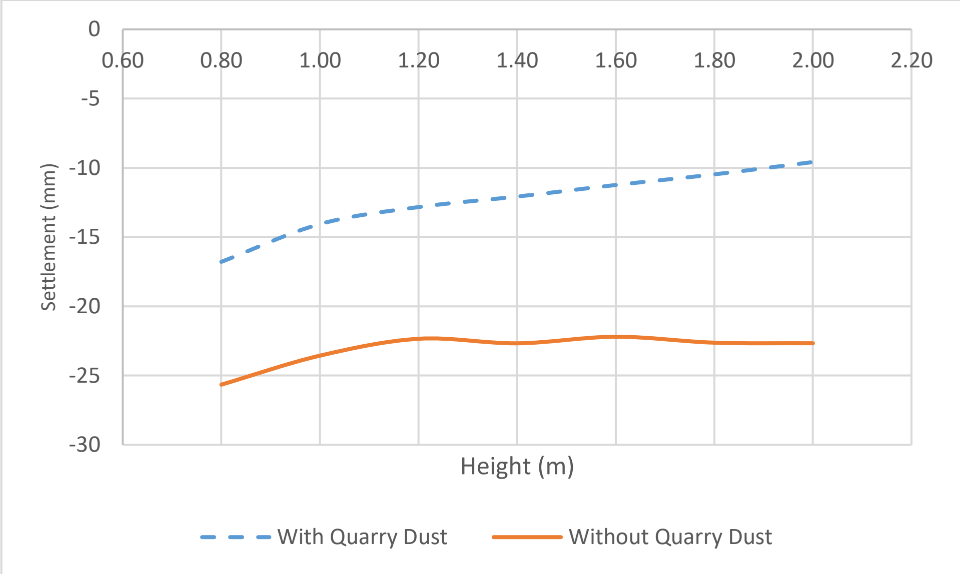


Figure 4-35 : Settlement vs Cylinder height

4.13. Parametric study on the use of concrete cylinders with quarry dust fill – consolidation analysis

Time versus settlement curves, shown in Figures 4-36 and 4-37, illustrate the relationship between settlement and the heights of quarry-dust-filled cylinders, as determined from the results of the analysis described in Section 3.15.1, for pre-consolidation stresses of 100 kPa and 250 kPa, respectively. Figure 4-38 displays the settlement-height curves on the 750th day.

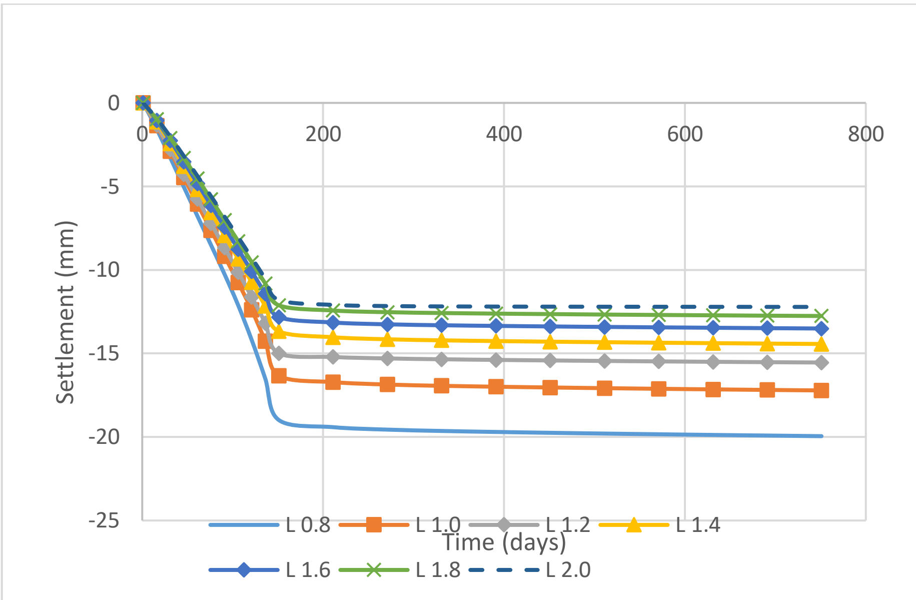


Figure 4-36: Time versus settlement curves (Pc -100 kPa)

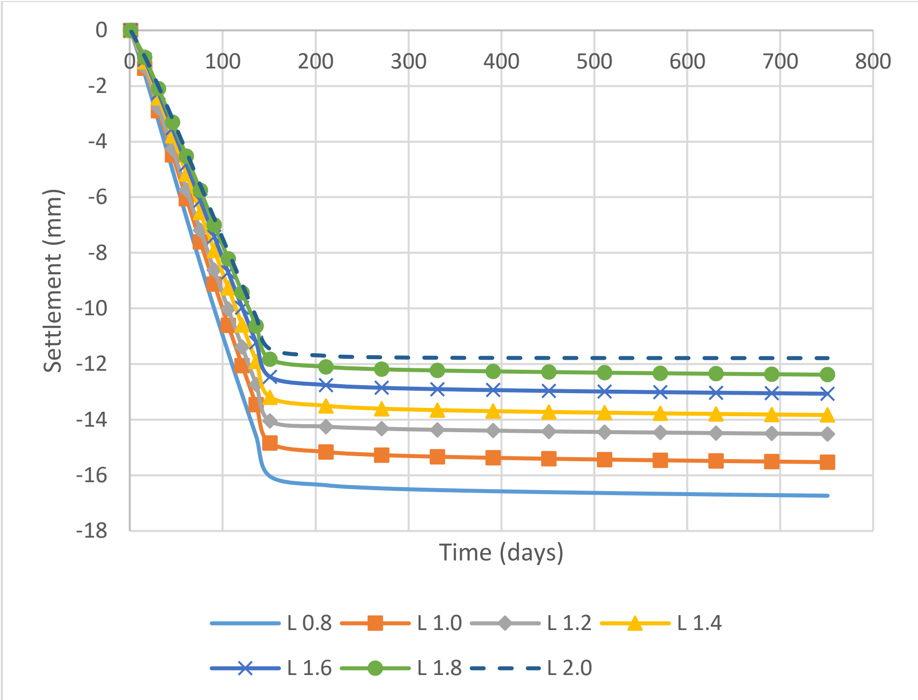


Figure 4-37: Time versus settlement curves (Pc-250 kPa)

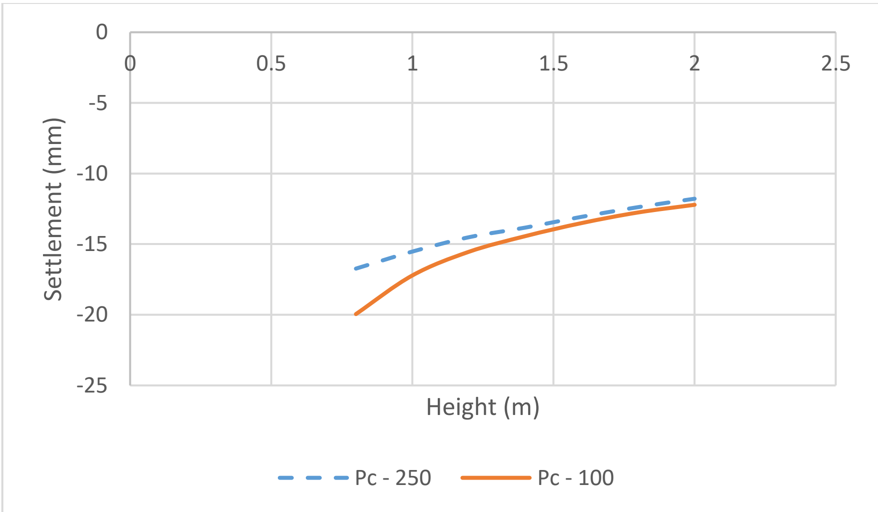


Figure 4-38: Cylinder height versus settlement

The cylinder height has a greater effect on the normally consolidated clay (Pc - 100 kPa), resulting in a 39% reduction in settlement, compared to a 30% reduction on over-consolidated clay (Pc – 250 kPa).

Cylinder diameter versus settlement curves, shown in Figure 4-39, illustrate the relationship between settlement and cylinder diameter, as determined from the analysis described in Section 3.15.2 for pre-consolidation stresses of 100 kPa and 250 kPa, respectively. The cylinder diameter has a greater effect on the pre-consolidation stress of 250 kPa than on that of 100 kPa.

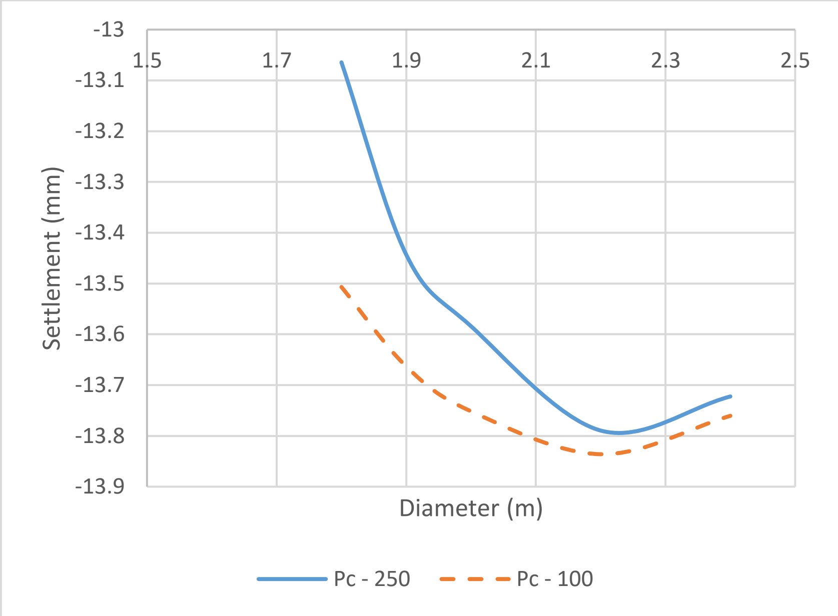


Figure 4-39: Cylinder diameter versus settlement

4.14. Stress distribution and concentration

Figure 4-40 illustrates the configuration of the CMCs and the vertical stress distribution. Table 4-1 displays the maximum vertical and shear stresses for each CMC. For CMCs, these stresses remain below the strengths of M-15 concrete. Notably, the middle CMC carries about twice the load compared to the corner CMCs.

Table 4-2: Stresses of CMCs

CMC	Vertical Stress (kPa)	Maximum Shear (kPa)
A	7412	353
B	1154	564
C	743	353
D	1147	565
E	1487	760
F	1143	564
G	744	356
H	1140	559
I	737	350

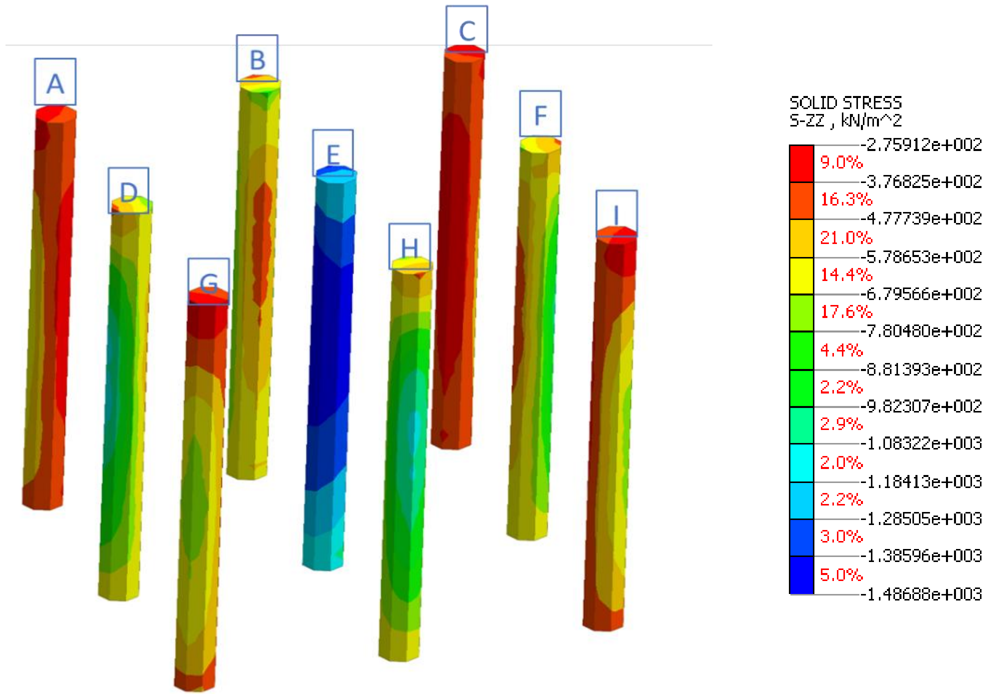


Figure 4-40: Configuration of the CMCs and the vertical stress distribution

Figures 4-41 and 4-42 show the stress redistribution resulting from the use of CMCs and cylinders, respectively. The stress distribution indicates that both enhancement techniques enable deeper load transfer.

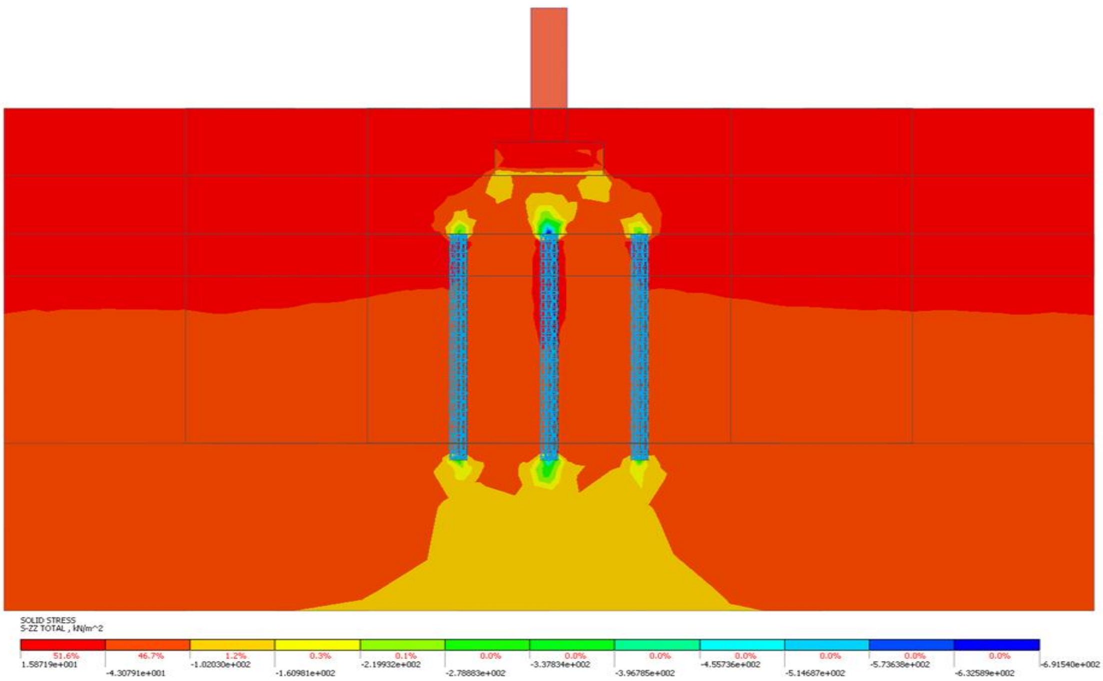


Figure 4-41: Stress redistribution resulting from the use of CMCs

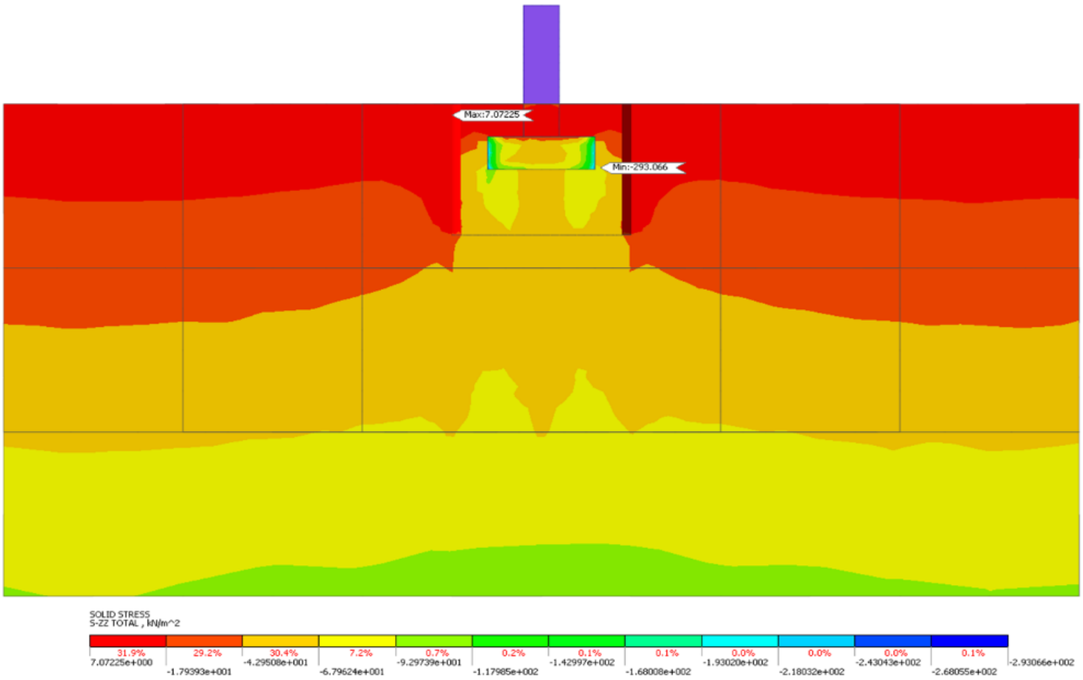


Figure 4-42: Stress redistribution resulting from the use of cylinders

Figures 4-43, 4-44, and 4-45 show vertical stress, maximum shear stress, and horizontal stress on concrete cylinders, respectively. The cylinder is subjected to both compressive and tensile stresses, with the maximum stress occurring near the four corners of the footing. In the case of cylinders, horizontal stress is the dominant factor, in contrast to vertical stress, which is more significant in CMCs.

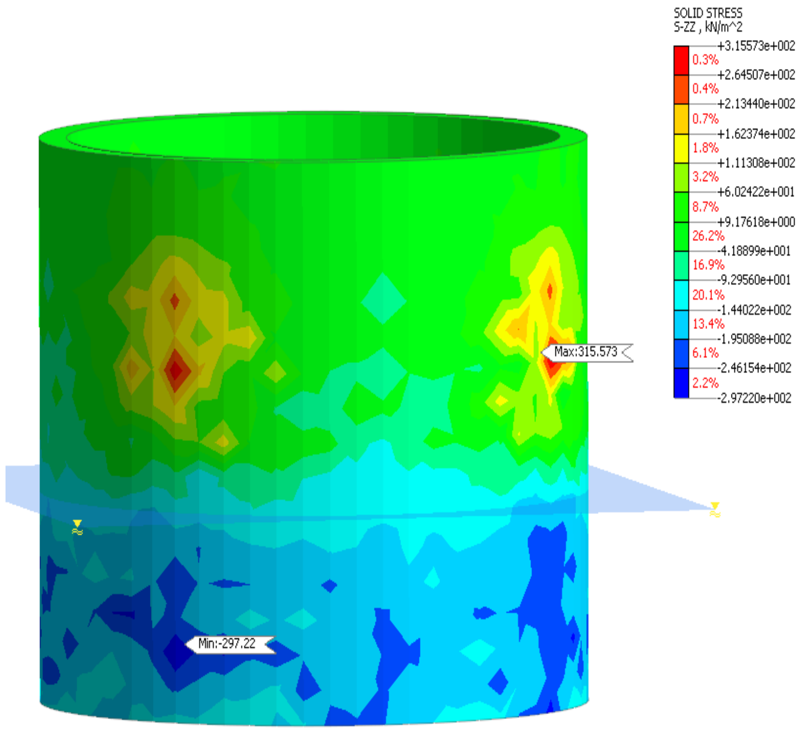


Figure 4-43: Vertical stress on concrete cylinders

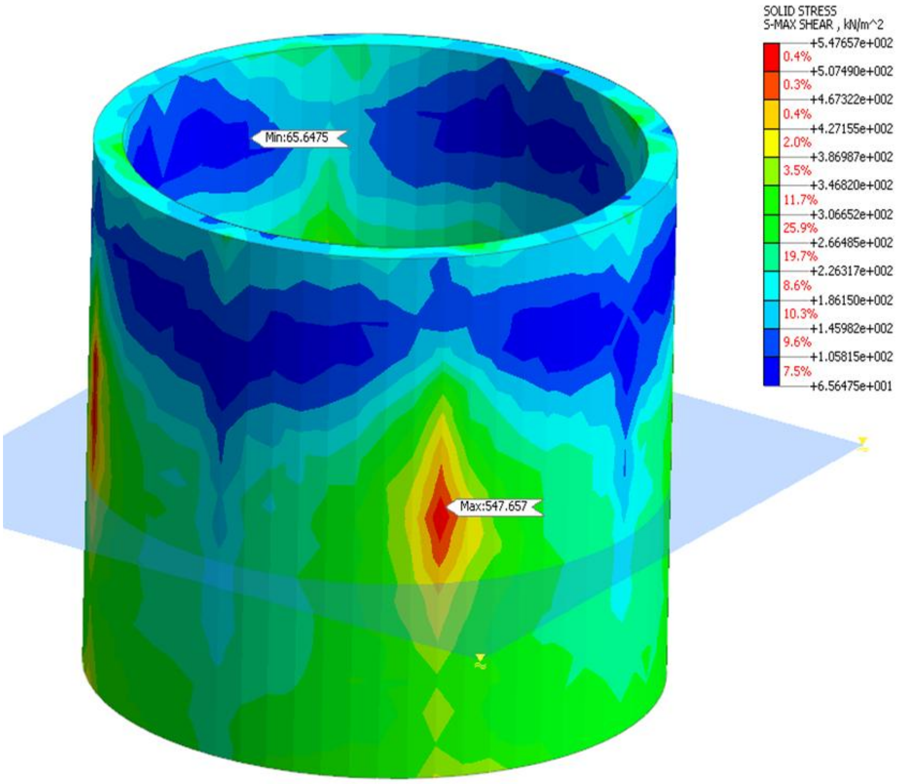


Figure 4-44: Maximum shear stress on concrete cylinders

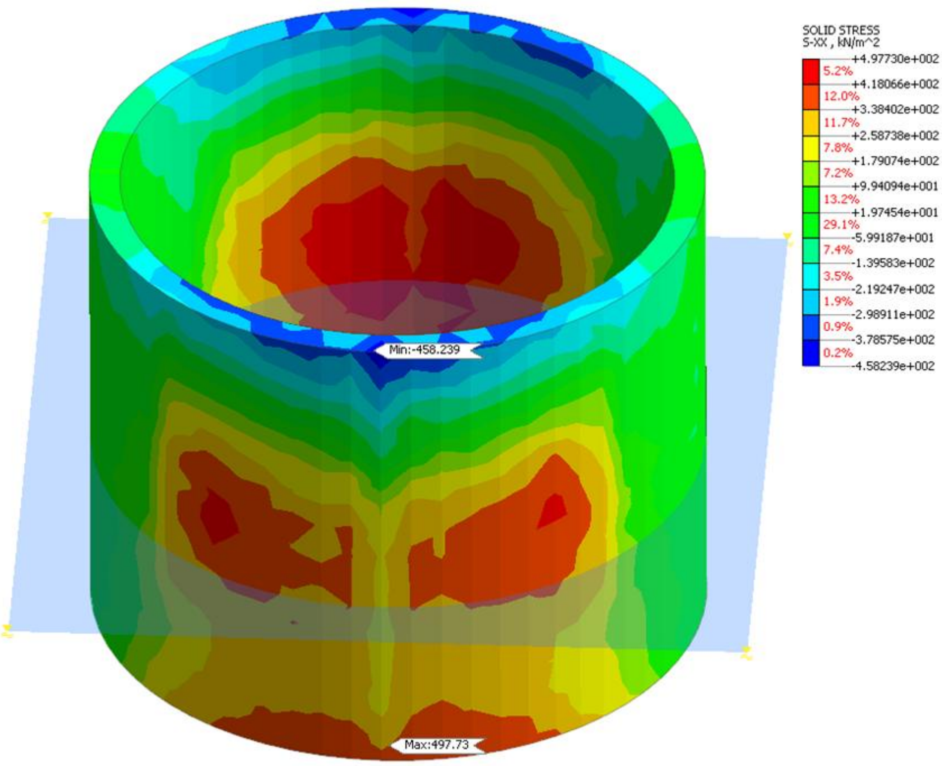


Figure 4-45: Horizontal stress on concrete cylinders

4.15. Explanation of results through soil mechanics

4.15.1. Soil mechanics interpretation of embedment effects

Mechanism 1: Stress Distribution with Depth

Using Boussinesq theory, the vertical stress at depth z beneath a loaded area is given by: $\Delta\sigma z = q \times$ Influence factor (I).

For footing at the ground surface: $I \approx 1.0$ at the footing level. For embedded footing (Df = 1.0m): The soft clay begins at z = 1.0m below the load, with $I \approx 0.6$-0.7.

- Stress on compressible clay decreases by 30-40%.
- Settlement is proportional to $\Delta\sigma z$, so it decreases in direct proportion.

Mechanism 2: Lateral Confinement

- Embedment exerts passive pressure from the surrounding soil, calculated as $K p \gamma' D f$.
- For Df = 1.0m, the passive pressure is approximately $0.3 \times 14 \times 1.0 \approx 4.2$ kPa, assuming $Kp \approx 3$.
- This pressure helps confine the soil, decreasing lateral bulging and vertical settlement.

Mechanism 3: Effective Stress Increase

- Overburden stress $\sigma'v0 = \gamma'z$ rises with embedment depth.
- For Df = 1.0m: $\sigma'v0 \approx 14$ kPa.
- Increased initial effective stress results in a stiffer soil response (higher E).
- According to MCC: $E \propto (1+e)/\kappa \times p' \rightarrow$ as p' increases, E also increases.

Why Effect Diminishes Beyond 0.8m:

Once the embedment ratio Df/B exceeds 0.6-0.7 (around 0.7-0.8m for B=1.2m):

- The stress bulb is situated beneath the most compressible layer.
- Additional confinement provides minimal advantages.
- Increasing embedment leads to higher excavation costs without proportional benefits.

4.15.2. Consolidation behaviour

During the loading period, consolidation took place as a total load of 300 kN was applied in 10 steps, with 30 kN added every 15 days over 150 days. Each load increase caused excess pore water pressure in the clay, roughly equal to the increase in total stress ($\Delta u \approx \Delta\sigma$) under undrained conditions. For the final surface load of 208 kPa (which corresponds to 300 kN over 1.44 m²), the initial excess pore pressure is estimated at about 200 kPa. The dissipation of this pore pressure follows Terzaghi's one-dimensional consolidation theory, expressed as,

$$U(t) = 1 - \sum\left(\frac{2}{M^2} \times e^{-M^2 T_v}\right) \qquad \text{Eq 4.1}$$

where $T_v = \frac{c_v t}{H^2}$.

The coefficient of consolidation (c_v) is determined using

$$c_v = \frac{k(1+e_0)}{\gamma_w \times m_v} \qquad \text{Eq 4.2}$$

and with $k = 1 \times 10^{-9}$m/s, $e_0 = 3$, and $m_v = 0.0002$m²/kN, the calculated c_v is approximately 2×10^{-6} m²/s, or 0.17 m²/day. For a clay layer thickness of 4 m with single drainage (drainage path $d = 4$m), the time required for 90% consolidation is

$$t_{90} = T_{90} \times \frac{d^2}{c_v} = 0.848 \times \frac{16}{0.17} \approx 80 \text{ days.} \qquad \text{Eq 4.3}$$

Most primary consolidation happens within roughly 80 days. Since the load was applied gradually in steps, excess pore pressures partially dissipated between these steps, enabling the soil to adapt gradually. Consequently, primary and secondary consolidation occurred simultaneously during loading. The incremental stress increase facilitated early creep or secondary compression alongside pore pressure dissipation. This behaviour is common in incrementally loaded clays, where structural rearrangement and time-dependent deformation start before loading is completed.

Around 150 days, there's a clear shift in the settlement rate, indicating the end of the loading phase, where no further stress is applied. At this stage, the deformation transitions from being load-driven to being time-dependent (secondary compression). According to consolidation parameters, about 90–95% of primary consolidation has already occurred. The settlement curve starts to level off, showing a reduction in deformation speed as secondary compression takes over.

For the normally consolidated clay (Pc = 100 kPa), the applied stress of 208 kPa far exceeds the pre-consolidation pressure, causing the soil to deform along the virgin compression line. With a high compression index (Cc = 2.03), the soil undergoes rapid primary consolidation, reaching about 95% settlement in roughly 150 days.

For the over-consolidated clay (Pc = 250 kPa), the applied stress of 208 kPa is below the pre-consolidation pressure, so the soil initially follows the recompression line. The ratio of compression indices (Cs/Cc ≈ 0.03/0.22 ≈ 0.136) signifies lower compressibility, resulting in faster consolidation and smaller total settlement.

4.15.3 Elastic vs. Consolidation settlement ratio

Based on the stress analysis summarised in Table 3-1, the drained modulus of the clay is 4,800 kPa, while the undrained modulus is roughly three times higher at approximately 14,400 kPa (assuming $\nu = 0.5$). Figure 4-1 illustrates that the immediate elastic settlement under a 300 kN load is roughly 10 mm. In contrast, the consolidation analysis discussed in Section 4.7 reveals significantly greater long-term settlements, about 80 mm for normally consolidated clay (Pc = 100 kPa) and 40 mm for over-consolidated clay (Pc = 250 kPa) after 750 days.

The ratio of consolidation to elastic settlement (Sc/Se) is approximately 8:1 for normally consolidated soil and 4:1 for over-consolidated soil. This indicates that most settlements in soft clay result from consolidation rather than immediate elastic deformation.

The high Sc/Se ratios mainly result from the clay's significant compressibility (Cc = 2.03, $e_0 = 3.0$), which causes a steep relationship between void ratio and stress. Stress history influences this as well: normally consolidated clay follows the virgin compression line, experiencing maximum compression, whereas over-consolidated clay initially follows the flatter recompression line, leading to less settlement. Additionally, drainage conditions change from undrained ($\nu \approx 0.5$, stiffer response) during loading to drained ($\nu \approx 0.2$–0.3, softer response) during consolidation. Secondary compression adds 15–30% to settlement in soft organic clays.

CHAPTER 5: CONCLUSION

5.1. Summary

This study provides a comprehensive numerical investigation of a 1.2 m wide square footing constructed on soft clay, incorporating Controlled Modulus Columns (CMC) and quarry dust-filled concrete cylinders as ground improvement techniques. Finite Element (FE) modelling and analysis were carried out using MIDAS GTS-NX to assess immediate settlements through stress analysis and long-term settlements via consolidation analysis, under a 300 kN vertical load. Soil layers were modelled using the Mohr-Coulomb constitutive model for stress analysis. In contrast, the clay layer was modelled with Modified Cam Clay (MCC) for consolidation analysis. For both analyses, structural elements (footing, CMC, cylinder) were modelled from the elastic material model. To minimise computational complexity, optimal mesh sizes were determined through a mesh sensitivity analysis, and a quarter-scale model was developed by applying symmetrical boundary conditions. The validation process involved comparing model predictions with (i) large-scale field load tests at Texas A&M, Green Cove Springs, and with theoretical calculations for immediate settlement, and (ii) laboratory model test results from the University of Moratuwa for consolidation behaviour.

A comprehensive parametric study was carried out to investigate the impact of the following critical design parameters:

1. Footing Embedment Depth: To assess the impact on bearing capacity and settlement reduction, the embedment depth was varied from 0.1 m to 1.5 m, measured from ground level, for both normally consolidated and over-consolidated clay soils.
2. Water Table Level: Conducted by varying the water level from ground level to 3 m below ground level through consolidation analysis.
3. CMC Geometry: Carried out for column height (2.5 m- 3 m), diameter (0.1 m-0.4 m), spacing (0.6 m-1.2 m), and Load Transfer Platform thickness (0.3 m-0.9 m) to study the effect on settlement reduction.
4. Cylinder Geometry: Conducted analysis by changing cylinder heights from 0.8 m to 2 m and adjusting the cylinder diameter from 1.8 m to 2.4 m, both with and without replacing the soil inside the cylinder with quarry dust.

5. Pre-Consolidation Stress of Clay: Analysed the pre-consolidation stresses of 100 kPa and 250 kPa to simulate normally consolidated and over-consolidated situations.

5.2. Key findings and conclusions

5.2.1. Key findings

1. Embedment Depth:
    - Changing embedment depth from 0.1 m to 1.5 m results in a 64% reduction in immediate settlements, while the reduction from 0.1 m to 0.6 m is 44%. After the 0.6 m to 1 m range, the rate of reduction diminishes.
    - Changing embedment depth from 0.1 m to 1.5 m results in a 78% reduction in consolidation settlements for a pre-consolidation stress of 100 kPa and 52% for 250 kPa. Embedment depths below 0.8 m show a lower rate of settlement reduction.
2. Water Table Level:
    - When the initial water level varied from ground level to 3 m below ground, there was a 48% reduction in consolidation settlements, while below 1.5 m, the reduction is negligible.
3. CMC Geometry:
    - Using 2.7 m high, 0.2 m diameter CMCs arranged in a 3×3 configuration with 1 m spacing results in a 58% reduction in immediate settlement and a 65% reduction in consolidation settlement.
    - The increase in CMC height from 2.5 m to 3 m results in only a 3% reduction in consolidation settlement. Meanwhile, as the CMC diameter increases from 0.1 m to 0.4 m, the consolidation settlement decreases by 37%.
    - Changing CMC spacing from 1.2 m to 0.6 m results in a 28% reduction in consolidation settlements, while the increase in settlement after 1 m up to 1.2 m is less.
    - Increasing Load Transfer Platform (LTP) thickness from 0.3 m to 0.9 m leads to a 35% reduction in consolidation settlements, although the effect diminishes after 0.5-0.7 m.

4. Cylinder Geometry:
    - Increasing cylinder height from 0.8 m to 2 m without quarry dust fill results in a 12% reduction in immediate settlement, while the model with quarry dust fill shows a 43% reduction. After reaching 1.2 m in height, the rate of reduction becomes slower.
    - For the pre-consolidation stress of 100 kPa, changing cylinder heights displays a 39% reduction in consolidation settlement, while showing a 29% reduction for 250 kPa.
    - Increasing the diameter from 1.8 m to 2.4 m results in only a 2% increase for 100 kPa models and a 5% rise in consolidation settlements for 250 kPa.
5. Stress Concentration and Distribution:
    - Stress diagrams show that the middle CMC in the 3×3 arrangement experiences approximately 1500 kPa of vertical stress and 760 kPa of shear stress, while CMCS in the corners endure about half of those stresses. Additionally, these stresses do not exceed the strength limits of grade 15 concrete.
    - For cylinders, the horizontal stresses (about 550 kPa) are more critical than the vertical stresses (300 kPa), and the maximum stresses are concentrated near the footing corners.
    - Both techniques show deeper stress redistribution, while CMC demonstrates the load-sharing mechanism between enclosed soil and CMCs.

5.2.2. Conclusions

- A D_f/B (Embedment depth/Footing width) ratio between 0.5 and 0.8 could reduce over 50% of both immediate and consolidation settlements for both normally consolidated and over-consolidated clay soils. Ratios above 0.8 indicate lower reduction, suggesting that the range of 0.5 to 0.8 is most optimal for D_f/B.
- The diameter of the CMC has a greater impact on settlement reduction than its height, and increasing the diameter will decrease vertical stress on CMCs. When the d/s (diameter/spacing) ratio varies from 0.1 to 0.4, settlement decreases by

37%, and when the ratio shifts from 0.17 to 0.33, the reduction is 28%. A ratio between 0.2 and 0.3 can be considered optimal based on these reduction rates.

- Changing s/B (spacing/footing width) from 1 to 0.5 results in a 28% reduction in settlement, while also decreasing the vertical stress on the middle CMC, leading to a more uniform distribution across CMCs. Thus, 0.5 can be considered the optimal value from this study.
- When LTP thickness/footing width (t/B) varies from 0.25 to 0.75, it results in a 35% reduction in settlements, while after 0.5, there is a diminishing rate of reduction in settlements. A 0.4 – 0.6 ratio can be considered optimal.
- The diameter of the cylinder has a much smaller effect on settlement reduction than its height. When the ratio of cylinder height to footing width (H/B) increases from 0.6 to 1.6, consolidation settlements decrease by over 25% for both Normally Consolidated and Over-Consolidated clays. Additionally, once the ratio reaches 1, the rate of reduction slows significantly, indicating that the optimal value is 1.
- When the initial water level varied from ground level to 3 m below ground, there was a 48% reduction in consolidation settlements. However, 90% of that reduction can be achieved when the water level is at 1.5 m, indicating that the water table near the embedment depth is more crucial.
- Normally consolidated clay (100 kPa) exhibits 30-50% higher consolidation compared to over-consolidated clay, indicating that design should be based on in-situ stress history.
- Horizontal stresses are more significant when using cylinders as an improvement method. However, for CMCs, vertical stress and stress concentration are more crucial in the design.

5.3. Recommendations

- This research was conducted solely for the 1.2 m × 1.2 m square footing. To obtain more detailed information, both testing and analysis should be carried out for different sizes.
- For CMCs, more configurations, material properties, and LTP layer properties should be tested.
- Conduct field tests to verify numerical ratios and scale effects.

REFERENCES

Ahmadi, M. H., Mortazavi, A., Davarpanah, M., & Zarei, H. (2016). *Civil Engineering Journal A Numerical Investigation of Segmental Lining Joints Interactions in Tunnels-Qomrud Water Conveyance Tunnel* (Vol. 2, Issue 7). www.CivileJournal.org

Anderson, J. B., Asce, M., Townsend, ; F C, & Rahelison, L. (n.d.). *Load Testing and Settlement Prediction of Shallow Foundation*. https://doi.org/10.1061/ASCE1090-02412007133:121494

ASIRI National Project (2013). (2013). Recommendations for the design, construction and control of rigid inclusion ground improvements. In *ASIRI National Project*.

Babu, G. L. S., Murthy, B. S., Murthy, D. S. N., & Nataraj, M. S. (2004). *Bearing Capacity Improvement Using Micropiles: A Case Study*. 692–699. https://doi.org/10.1061/40713(2004)14

Donovan, K. , Pariseau, W. G., & Cepak, M. (1984). *"Finite element approach to cable bolting in steeply digging VCR slopes". Geomechanics application in underground hardrock mining*.

Fattah, M. Y., Al-Baghdadi, W., Omar, M., & Shanableh, A. (2010). Analysis of strip footings resting on reinforced granular trench by the finite element method. *International Journal of Geotechnical Engineering*, *4*(4), 471–482. https://doi.org/10.3328/IJGE.2010.04.04.471-482

Fattah, M. Y., Al-Neami, M. A., & Al-Suhaily, A. (2015). *Improvement of Bearing Capacity of Footings on Soft Clay by Partial Soil Replacement Technique The 2nd International Conference of Buildings, Construction and Environmental Engineering (BCEE2-2015)*. https://www.researchgate.net/publication/299432899

Fattah, M. Y., Shlash, K. T., & Mohammed, H. A. (2014). Bearing Capacity of Rectangular Footing on Sandy Soil Bounded by a Wall. *Arabian Journal for Science and Engineering*, *39*(11), 7621–7633. https://doi.org/10.1007/s13369-014-1353-7

Frederic Masse. (2017, May 9). *COMPARISON OF CMC RIGID INCLUSIONS AND DEEP FOUNDATIONS.*

Grizi, A., Al-Ani, W., & Wanatowski, D. (2022). Numerical Analysis of the Settlement Behavior of Soft Soil Improved with Stone Columns. *Applied Sciences (Switzerland)*, *12*(11). https://doi.org/10.3390/app12115293

ground-improvement-techniques-soil-stabilization. (n.d.).

Janbu, N., Bjerrum, L., & Kjærnsli, B. (1966). Veiledning ved løsning av fundamenteringsoppgaver. *Norwegian Geotechnical Institute Publication Nr. 16*.

Jean-Louis Briaud, & Robert Gibbenst. (1997). *Large-Scale Load Tests and Data Base of Spread Footings on Sand*.

K Coghlan, C Plomteux, & Jerome Racinais. (2016). Execution and engineering principles of Control Modulus Column (CMC). In *Proceedings of the First Southern African Geotechnical Conference* (pp. 131–178). CRC Press. https://doi.org/10.1201/b21335-6

Madhav A N, M. R., & Vitkar, D. P. P. (1978). Strip footing on weak clay stabilized with a granular trench or pile. In *Can. Geotech. J* (Vol. 15). www.nrcresearchpress.com

Marwa Hasan. (2022, January 3). *The Best Foundations for 6 Common Soil Types*. https://www.angi.com/articles/how-does-soil-type-affect-your-home-s-foundation.htm

Mohammed Al-Waily, M. J. (2019). Effect of Partial Replacement with Crushed Concrete on Settlement and Bearing Capacity of Soft clay. *IOP Conference Series: Materials Science and Engineering*, *584*(1). https://doi.org/10.1088/1757-899X/584/1/012059

Mollaali, M., Alitalesh, M., Yazdani, M., & Shafie, M. B. (2014). Soil improvement using micropiles. *Numerical Methods in Geotechnical Engineering - Proceedings of the 8th European Conference on Numerical Methods in Geotechnical Engineering, NUMGE 2014*, *1*, 535–540. https://doi.org/10.1201/b17017-96

Murthy, B.R.S, Babu G.L.S., & Srinivas, A. (2002). *Analysis of bearing capacity improvement using micropiles. Proceedings of the ICE-Ground Improvement*.

Nguyen, H. H., Khabbaz, H., Fatahi, B., & Hsi, J. (n.d.). *INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING Effects of installing controlled modulus columns on previously installed columns Effets de l'installation de colonnes à module contrôlé sur les colonnes précédemment installées*. https://www.issmge.org/publications/online-library

Ornek, M., Laman, M., Demir, A., & Yildiz, A. (2012). Prediction of bearing capacity of circular footings on soft clay stabilized with granular soil. *Soils and Foundations*, *52*(1), 69–80. https://doi.org/10.1016/j.sandf.2012.01.002

Serridge, C. J. (n.d.). *An evaluation of partial depth dry bottom-feed vibro stone columns to support shallow footings in deep soft clay deposits*.

Srijan, S. (2025). Enhancing ground stability with quarry dust columns: A comparative study of ordinary and encased configurations. *Journal of King Saud University - Engineering Sciences*, *37*(1). https://doi.org/10.1007/s44444-025-00001-1

Surarak, C., Likitlersuang, S., Wanatowski, D., Balasubramaniam, A., Oh, E., & Guan, H. (2012). Stiffness and strength parameters for hardening soil model of soft and stiff Bangkok clays. *Soils and Foundations*, *52*(4), 682–697. https://doi.org/10.1016/j.sandf.2012.07.009

Susila, E., & Agrensa, F. (2018). Behaviors of Pipe-Soil Interaction on Unstable Slopes by Finite Element Simulation. *Jurnal Teknik Sipil*, *25*(2), 87. https://doi.org/10.5614/jts.2018.25.2.1

Vinod B, R., Geetha Kumari, T., Sunagar, P., & Nayak, S. G. (2021). Improvement of Loose Red soft soils using Aluminium foil encased stone columns. In *Volatiles & Essent. Oils* (Vol. 8, Issue 4).