OPTIMUM DESIGN OF REINFORCED CONCRETE RAFT FOUNDATIONS USING FINITE ELEMENT ANALYSIS

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DECLARATION

I declare that the work in thisthesis entitled "Optimum Design of Reinforced Concrete Raft Foundations Using Finite Element Analysis" has been performed by me in the Department of Civil Engineering. The information derived from the literature has been dulyacknowledged in the text and a list of references provided. No part of this thesis was previously presented for another degree or diploma at this or any otherInstitution.

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(MSc./ENG/706/2010-2011) Signature Date

CERTIFICATION

This thesis entitled "OPTIMUM DESIGN OF REINFORCED CONCRETE RAFT FOUNDATIONS USING FINITE ELEMENT ANALYSIS" by Mahmud Abdulkadir GARBA meets the regulationsgoverning the award of the degree of Masters (M.Sc) in Structures of the Ahmadu Bello University, and is approved for its' contribution to knowledge andliterary presentation.

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DEDICATION

This thesis is dedicated to my late parents AlhajiGarba Ja Abdulkadir and Hajiya Mairo Garba Ja Abdulkadir.

ACKNOWLEDGEMENT

Thanks are due to almighty Allah for sparing my life and for giving me strength and courage to carry out this research work.

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ABSTRACT

This work presents the finite element analysis (FEA) of the requirements of compression reinforcements in raft foundations using ABAQUS. The model helps to confirm and provide a valuable supplement to the theoretical design. For validation, a reinforced concrete raft foundationis modeled whichis conventionally designed according to Eurocode 2 (EN 1992-1-1:2004). The result indicates that there is differential settlement within the raft foundation based on the settlement and stress patterns obtained from the finite element model (FEM). This is followed by the addition of compression reinforcement, from 0.1% to 0.9% of the cross sectional area of the raft slab, until uniform settlement is obtained. The results suggest that a suitable percentage of the concrete cross sectional area of raft slab foundations should be used as compression reinforcement, when designing conventionally using Eurocode 2, in order to prevent differential settlements. The required area of compression reinforcement is 0.9% of the cross sectional area of the concrete section.

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NOMENCLATURE

Symbol	Definition	Unit
f_c	Concrete Compressive Strength (British Code)	N/mm^2
f_{ck}	Concrete Compressive Strength (Eurocode)	*MPa*
f_{yk}	Steel (Reinforcement) Strength (Eurocode)	*MPa*
b	Breadth of Section	*mm*
b_w	Breadth of the Rib	*mm*
d	Depth of Section	*mm*
C_{nom}	Nominal Concrete Cover	*mm*
U	Perimeter of Column	*mm*
Z	Lever Arm	*mm*
A_s	Area of Steel	mm^2
A_{sreq}	Area of Steel Required	mm^2
A_{sprov}	Area of Steel Provided	mm^2
γ_s	Partial Safety Factor for Steel	-
γ_c	Partial Safety Factor for Concrete	-
E_s	Modulus of Elasticity of Steel	*GPa*
υ	Poisson's Ratio	-
M_{max}	Maximum Moment	*KNm*
K_{factor}	Effective Length Factor	-
K_{bal}	Balanced Length Factor	-
V_{ED}	Applied Shear Force	*KN*
V_{max}	Maximum Shear Force	*KN*
$V_{Rd,c}$	Design Shear Resistance Force	*KN*
$C_{Rd,c}$	Factor for the Aggregate Interlock	*KN*
ρ	Compression Reinforcement Ratio	-

Abbreviations

FEA	Finite Element Analysis
ASCE	American Society of Civil Engineers
EN	Eurocodes
3D	Three-dimensional
C3D8R	Brick element with reduced integration (finite element)
U	Displacement
UR	Rotational displacement
YASYMM	Anti-Symmetry about a plane y = constant

CHAPTER ONEINTRODUCTION

1.1 Preamble

The raft foundation was invented in the 19th century (Paul, 2010). Its development was necessitated by engineering requirements to build tall buildings (Paul, 2010). Initially, raft foundations were used for commercial and industrial developments (Paul, 2010). However, once the advantages of the concept were realised, the raft foundation became popular within residential developments (Paul, 2010).

A raft foundation is usually used when building in low soil bearing conditions to spread the load from a structure over a large area, normally the entire area of the structure (UWE, 2012). They are used when column loads or other structural loads are close together and individual pad foundations would interact (UWE, 2012). Raft foundations may be used for buildings on compressible ground such as very soft clay, alluvial deposits and compressible fill material where strip, pad or pile foundations would not provide a stable foundation without excessive excavation (Stephen and Christopher, 2010). The reinforced concrete raft is designed to transmit the load of the building and distribute the load over the whole area under the raft, reducing the load per unit area placed on the ground (Stephen and Christopher, 2010). Distributing the loads this way causes little, if any, appreciable settlement (Stephen and Christopher, 2010).

Structurally, raft foundations resting directly on soil act as a flat slab or a flat plate, upside down, i.e., loaded upward by the bearing pressure and downward by the concentrated column reactions (Mahdi, 2008). The raft foundation develops the maximum available bearing area under the building (Mahdi, 2008). Raft foundations are designed as inverted beam and slab system (Singh and Singh, 2006). The weight of the raft is not considered in the structural design (Singh and Singh, 2006). If all the loads transferred to the raft foundation are equal, raft may be a simple flat slab type, without any beam (Singh and Singh, 2006). In case loads are not equal, slab and beam system is usually adopted (Singh and Singh, 2006). Differential and total settlements usually govern the design (GEO, 2006).

Finite element analysis or elastic continuum method is preferred for the design of raft foundations (French, 1999; Poulos, 2000).Subgrade reaction models are often not appropriate (Eurocode 7, 2004). More precise methods, such as finite element computations, should be used when ground-structure interaction has a dominant effect (Eurocode 7, 2004).

After laying a mat or raft foundation on the soil, soft soil for example, there is tendency of cracks developing in areas between columns (lower part) and in areas near and under columns (upper part) (Babak, 2011). Then there is need usually to reinforce upper part of foundations near columns and lower part between columns (Babak, 2011). Compression reinforcement is usually not applied in foundations. However, there is need to apply a minimum amount of reinforcement in the upper part of the foundation due to practical points of view. Then the additional minimum compression reinforcement may heighten the center of compression and increase the resisting moment provided by the section.

In the case of an under-design, there is a very high risk of potential failure, which if occurs, amounts to greater financial costs due to refurbishment, redesign and reconstruction. While in the case of an over design, the initial financial costs of design and construction will be higher with less financial risks of failure occurring.

1.2 Justification for The Study

The design of a raft foundation is prone to significant uncertainties. Many of such uncertainties are related to the estimation of suitable soil properties. The sources of uncertainties for soil properties are classified into three main components: inherent soil variability, measurement error and transformation model uncertainty (Filippas et al., 1988). Other uncertainties are associated with the site investigation and settlement technique. Also, the variation of elastic modulus of soil and presence of rock media plays a significant role and affects the moments and deformations of raft foundation (Venkatesh et al., 2009). The effect of the spatial variation of soil properties which induces foundation stresses and/or

displacements that cannot be predicted when assuming soil homogeneity (Niandou et al., 2006), the variation of elastic modulus of soil, the presence of rock media and the design uncertainties may give rise to differential settlement within the raft foundation and subsequently its structural failure.

Eurocode 7 2004 specifies that more precise methods, such as finite element computations, should be used when ground-structure interaction has a dominant effect. This implies that the conventional method of design has little precision and should be complemented with a more advanced design method. This work presents the FEA modeling of the requirements of compression reinforcements in raft foundations using ABAQUS. The model helps to confirm and provide a valuable supplement to the conventional design.

1.3 Aim and Objectives

The aim of this research is to use finite element analysis in the optimum design of reinforced concrete raft foundations. The detailed objectives are to:

a. Design a simple reinforced concrete raft foundation structure using the conventional method of design. The design will then be subjected to deformation using finite element analysis in order to obtain the stress pattern and settlement.
b. Identify the need to provide additional compression reinforcement to the design at different percentages of the reinforcement ratio based on the cross sectional area of the raft slab and hence determine its effectiveness in providing resistance against differential settlement.
c. Appreciate the need to use finite element analysis in the design of reinforced concrete raft foundations.

1.4 Methodology

The structural design of the raft foundation will be carried out using the conventional method of design (i.e. hand calculation) and finite element analysis. The conventional design will be carried out according to Eurocode 2 (EN 1992-1-

1:2004), which is to specify the depth of foundation, area and amount of reinforcement, and all the necessary checks needed in the design calculations.

The finite element analysis (FEA) will be carried out with the aid of a computer program. The program that will be used is SIMULIA ABAQUS 6.10. ABAQUS is a finite element analysis software that is used in a wide range of industries like automotive, aerospace etc., and also is extensively used in academic and research institutions due to its capability to address non-linear problems (Manjunath, 2009). The ABAQUS program can be used to model reinforced concrete structures analyze and generate test results using a state of the art 3D modeling and finite element technology.

The finite element analysis (FEA) will be used to test the designed reinforced concrete raft foundation. Other models will also be designed and tested which have compression reinforcement at various percentages of the reinforcement ratio based on the cross sectional area. This is to determine the effect of the compression reinforcement in providing resistance against differential settlement.

The raft foundation consists of a regular arrangement of eight column loads with four corner and four internal loads. All the corner columns carry a load of 458.33 *KN* each and the internal columns carry 666.66 *KN* each. Each column is 0.5 *m* by 0.5 *m*. Bearing capacity of the soil will be taken as 100 KN/m^2. The characteristic strengths of the concrete and steel to be used in the design are 45 *MPa* and 500 *MPa* respectively. The Poisson ratio and density of the concrete will be taken as 0.2 and 2400 kg/m^3 respectively while the Poisson ratio and density of the steel will be taken as 0.3 and 7850 kg/m^3 respectively.

During the modeling in ABAQUS, the analysis parts for the soil, slab, and reinforcement will be created and assigned material and section properties. The embedded element option will be used to embed the reinforcements in the slab. The elastic foundation option will be used to model the soil surface to make it act as springs to ground which includes the stiffness effects of a support (such as the soil under a building) without modeling the details of the support. The parts will

then be assembled together, the loads and boundary conditions will be imposed and the job executed to obtain the results.

1.5 Scope and Limitation

This research work is limited to the design, modeling and analysis of the flat reinforced concrete raft foundation without any experimental study.

CHAPTER TWOLITERATURE REVIEW

2.1 Site Investigation

All successful designs require greater geotechnical input including well planned site investigations, field and laboratory testing, together with consideration of the method of construction (GEO, 2006). A broad understanding of the ground conditions, site constraints, geological profile, site history and the properties of the various strata are necessary for the success of a foundation project.

Sites with a history of industrial developments involving substances which may contaminate the ground (e.g. dye factories, oil terminals) will require detailed chemical testing to evaluate the type, extent and degree of possible contamination (GEO, 2006 ; Ijimdiya, 2010a,b). An understanding of the geology of the site is a fundamental requirement in planning and interpreting the subsequent ground investigation (GEO, 2006). A useful summary of the nature and occurrence of rocks and soils should be obtained (GEO, 2006). Information on the groundwater regime is necessary for the design and selection of foundation type and method of construction (GEO, 2006).

It is always a recommended practice to retrieve good quality soil samples and continuous rock cores from boreholes for both geological logging and laboratory testing (GEO, 2006). For a rational design, it is necessary to have data on the strength and compressibility of the soil and rock at the appropriate stress levels within the zone of influence of the proposed foundations (GEO, 2006). The variation of elastic modulus of soil and presence of rock media plays a significant role and affects the moments and deformations of raft foundation (Venkatesh et al., 2009).

An appropriate geological model of a site is an essential requirement for safe foundation design (GEO, 2006). There are inherent uncertainties in any geological models given that only a relatively small proportion of the ground can be investigated, sampled and tested (GEO, 2006). It is therefore important that all

available information is considered in characterising the ground profile and compiling a representative geological model for the site (GEO, 2006).

2.1.1 Bearing Capacity of Foundations

In 1921, Prandtl published the result of his study in the penetration of hard bodies, such as metal punches, into a softer material. Terzaghi (1943) developed an analytical bearing capacity equation, based on superposition. In the past decades, the concept of bearing pressure in foundation design was introduced to investigate the excessive settlements occurring in buildings (Terzaghi and Peck 1967).

More recently, conventional finite element analyses (Griffiths, 1982; Burd and Frydman, 1997) have been used to predict the upper- and lower-bounds of bearing capacity of soils. These techniques have reduced the subjectivity and empiricism associated with the bearing capacity factors (Jason, 2006).

Corrections to the bearing capacity equations are also required for water table location (Small, 2001) and the friction angle of the soil obtained using the triaxial test (Meyerhof, 1963). Because of the different bearing capacity factors and correction factors, Bowles (1997) suggested a use for the more common solutions. However, he also indicated that more than one solution should be predicted to allow verification. Bowles (1997) discussed several procedures that yield estimates of the bearing capacity of a soil directly from in situ test results.

2.1.2 Total and Differential Settlements

Bowles (1997) considered settlement estimates of a foundation as a best guess of the footing deformation after a load has been applied. Holtz (1991) observed that the design of a shallow foundation is typically governed by a limiting settlement criterion while Bowles (1997) noted that most structural distress is caused by excessive settlements and not the shear failures associated with bearing capacity. Settlement occurs in three stages: immediate or distortion; consolidation; and secondary compression settlement (Holtz, 1991).

Small (2001) suggested that it is generally acceptable to assume elastic behavior, as the working loads are typically lower than those governing the bearing capacity of the foundation. This is because settlement is typically estimated after the foundation has been designed for bearing capacity (Holtz, 1991). However, Small (2001) warned that the adopted elastic modulus must be appropriate for the stress range in the soil.

Bowles (1997) suggested that the term elastic modulus is not strictly correct as soil is not an elastic medium, even though, the elastic modulus is the most common term used for this parameter. Additional methods have been used to estimate the immediate settlement under the corner of a footing (Harr, 1966; Perloff, 1975; Mayne and Poulos, 1999).

Bowles (1997) suggested that differential settlements are the major cause of structural distress and therefore should be controlled by the designer. However, Day (1999) commented that the total settlement of a foundation can have serious effects on the use of the structure being supported. Therefore, it is recommended that both total and differential settlement be considered alike. Numerical methods like Finite Element Analysis (FEA) are excellent means for estimating the predicted settlement of a raft but there are several simplified methods that do not require such numerical procedures. In these methods, it is important that the actual stiffness of the raft is considered (Small, 2001). He also warned that analyses representing the raft as a Winkler foundation do not represent the true behavior of the soil and the analyses using elastic continuum is not site specific.

2.1.3 Soil Horizontal Variability

Soil properties are known to vary from one location to another and this may have a significant effect on the overall design of a raft foundation. Even when soils are considered reasonably homogenous, soil properties exhibit considerable variability (Vanmarcke, 1977a). This variability is due to the complex and varied physical phenomena experienced during their formation (Jaksa, 1995). Variability between soil properties is called *spatial variability* and has recently been modeled

as a random variable (Spry et al., 1988). Soil variability has been categorised by Jason (2006) into the following:

i. Property randomness,
ii. Statistical parameters of soil properties,
iii. Modeling spatial variability.

2.1.4 Other Uncertainties Involved in Site Investigation

Jason (2006) outlined other sources of uncertainty inherent in the design process and these are:

1. Statistical uncertainty,
2. Measurement error, and
3. Transformation model uncertainty.

The statistical uncertainties associated with a geotechnical model are a result of limited sampling that may not provide an accurate representation of the underlying conditions (Jason, 2006).

Measurement errors arise from the inability of geotechnical tests to accurately estimate the soil properties being tested (Jason, 2006). Sources of measurement error can be separated into two categories: systematic and random (Filippas et al., 1988).

The results of common geotechnical insitu tests do not typically provide applicable soil properties that are useful for design relationships (Phoon and Kulhawy, 1999). Rather, the raw test results are processed using a transformation model into a suitable design parameter and such models are obtained empirically through back substitution or calibration (Jason, 2006). Phoon and Kulhawy (1999) further stated that uncertainty still exists if the transformation is based on a theoretical relationship because of idealizations and simplifications in the theory. It is therefore necessary to estimate uncertainties due to transformation model error.

2.2 Raft Foundations

Raft or mat foundation is a combined footing that covers the entire area beneath a structure and supports all walls and columns. This raft or mat normally rests directly on soil or rock, but can also be supported on piles as well. A raft is used when loads are large and pad foundations give excessive settlements. Total and differential settlements usually govern the design. A detailed structural design is necessary which provides slab thickness and reinforcement to resist bending and shear.

2.2.1 Need for Raft Foundations

Gupta (2007) outlined that raft foundation is generally suggested in the following situations:

a. Whenever building loads are so heavy or the allowable pressure on soil so small that individual footings would cover more than floor area.
b. Whenever soil contains compressible lenses or the soil is sufficiently erratic and it is difficult to define and assess the extent of each of the weak pockets or cavities and thus estimate the overall and differential settlement.
c. When structures and equipment to be supported are very sensitive to differential settlement.
d. Where structures naturally lend themselves for the use of raft foundation such as silos, chimneys, water towers, etc.
e. Floating foundation cases wherein soil is having very poor bearing capacity and the weight of the super-structure is proposed to be balanced by the weight of the soil removed.
f. Buildings where basements are to be provided or pits located below ground water table.
g. Buildings where individual foundation, if provided, will be subjected to large widely varying bending moments which may result in differential rotation and differential settlement of individual footings causing distress in the building.

In case of soils having low bearing pressure, Gupta (2007) also outlined three advantages of using a raft foundation:

a. Ultimate bearing capacity increases with increasing width of the foundation bringing deeper soil layers into the effective zone.
b. Settlement decreases with increased depth.
c. Raft foundation equalises the differential settlement and bridges over the cavities.

2.2.2 Types of Raft Foundations

Gupta (2007) classified raft foundation into various types on the following basis:

1. Based on the method of their support, raft can be:

   a. Raft supported on soil,
   b. Raft supported on piles, and
   c. Buoyancy raft.

2. On the basis of structural system adopted for the structure of the raft, these can be classified as:

   a. Plain slab rafts, which are flat concrete slabs, having a uniform thickness throughout. This can be with pedestals or without pedestals.
   b. Beam and slab raft which can be designed with down stand beam or up stand beam systems.
   c. Cellular raft or framed raft with foundation slab, walls, columns and one of the floor slabs acting together to give a very rigid structure.

Raft of uniform depth is most popular due to its simplicity of design and construction (Gupta, 2007). This type is most suitable where the column loads are moderate and the column spacing fairly small and uniform (Gupta, 2007). Pedestals are utilised to distribute the load on a bigger area in case of heavy column loads (Gupta, 2007).

2.2.3 Design of Raft Foundations

Methods for the design of rafts can be separated into three main groups: 'static' or approximate, theoretical, and numerical methods (Hemsley, 2000). Another way of classifying them is according to the model used to represent the soil: methods based on Winkler's hypothesis and on a solid continuum (Hemsley, 2000).

According to Mahdi (2008), the design of raft foundations may be carried out by one or two methods:

i. The conventional rigid method and;
ii. The finite element method utilizing computer programs.

The conventional method is easy to apply and the computations can be carried out using hand calculations (Mahdi, 2008). However, the application of the conventional method is limited to rafts with relatively regular arrangement of columns (Mahdi, 2008).

In contrast, the finite element method can be used for the analysis of raft regardless of the column arrangements, loading conditions and existence of cores and shear walls (Mahdi, 2008). Commercially available computer programs can be used (Mahdi, 2008). The user should, however, have sufficient background and experience (Mahdi, 2008).

2.2.4 Concrete Under Compression

The concrete stress-strain relation exhibits nearly linear elastic response up to about 30% of the compressive strength (Kwak et al., 1990). This is followed by gradual softening up to the concrete compressive strength, when the material stiffness drops to zero (Kwak et al., 1990). Beyond the compressive strength the concrete stress-strain relation exhibits strain softening until failure takes place by crushing (Kwak et al., 1990). For stress above 0.3 f_c', micro cracks form at the mortar-coarse aggregate interfaces and propagate through the mortar upon further loading (ASCE, 1981). Owning to these micro cracks, concrete begins to soften until it reaches the peak stress at a strain of 0.002 to 0.003 (Schnobrich and Hu,

1985). Beyond the peak, with increasing compressive strain, damage to concrete continues to accumulate and concrete enters the descending portion of its stress-strain curve, a region marked by the appearance of macroscopic cracks (Schnobrich and Hu, 1985).

Compressive failure is usually characterized by ductility and the compressive type of failure is defined as "crushing" where many small distributed cracks appear and the two principal stresses cannot be kept constant at the peak stress condition. Crushing failure will take place under high compression-low tension stress state (Schnobrich and Hu, 1985).

When concrete is subjected to compressive stresses, experimental results (Sinha et al., 1964) have indicated that the nonlinear deformations of concrete are basically inelastic, because upon unloading only a portion of the strains can be recovered from the total strains. Therefore, the stress-strain behavior may be separated into the recoverable part which can be treated within elasticity theory and the irrecoverable parts which can be treated by plasticity theory (Schnobrich and Hu, 1985). Plasticity based models have been extensively used in recent years to describe the behavior of concrete. Initially plasticity models described concrete as an elastic-perfectly plastic material (Mikkola and Schnobrich, 1970; Hand et al., 1972; Abdel-Rahman, 1982). Later models incorporated a hardening behaviour (Chen and Chen, 1975; Buyukozturk, 1977; Chen and Ting, 1980).

Under high compression, it is known that concrete undergoes flow somewhat like a ductile material on the yield surface before reaching its crushing surface (analogous to the yield surface but in terms of strain) (Schnobrich and Hu, 1985). This limited plastic flow ability of concrete before crushing can be represented by the introduction of an elastic-perfectly plastic model (Schnobrich and Hu, 1985).

2.3 Finite Element Analysis

Finite Element Analysis (FEA) is best described as a numerical procedure to analyse structures or continua (Cook et al., 1989). FEA can be traced back to 1906, when lattice analogy was introduced in stress analysis by Wieghardt (1906),

Riedel (1927), Hrennikoff (1941) and Ergatoudis et al. (1968). The use of the finite element method to analyse geotechnical and reinforced concrete structures started with the advent of digital computing and advances made in terms of analytical and numerical techniques. With the availability of affordable computing power, its use has increased exponentially and its status has changed from luxury to necessity. It is a powerful tool in structural analysis of simple to complicated geometries (Venkatesh et al., 2009). Venkatesh et al. (2009) outlined the basic steps involved in the finite element method as mentioned below:

i. Discretization of the continuum.
ii. Calculation of the element stiffness matrices.
iii. Assembling the element stiffness matrices.
iv. Calculation of the element load vectors.
v. Assembling the element load vectors.
vi. Imposition of boundary conditions.
vii. Imposition of external forces.
viii. Calculation of the displacement vectors.
ix. Calculation of the strains and stress field.

The first solution which employed the finite element method for the analysis of foundation structures on an elastic half-space was obtained by Cheung et al., (1968). The finite element method is an approximate technique, and as such, results computed using the finite element method must be critically evaluated before being relied upon in a design application. Thus, the number of elements used in a model can greatly affect the accuracy of the solution (Deaton, 2005). In general, as the number of elements, or the fineness of the mesh, is increased, the accuracy of the model increases as well (Deaton, 2005). The type of element applied in the analysis also can significantly affect the quality of the results because various finite elements are derived from different assumptions (Deaton, 2005).

2.4 Overview of The Abaqus Program

ABAQUS is a finite element analysis software. It is used in a wide range of industries like automotive, aerospace etc., and also is extensively used in academic and research institutions due to its capability to address non-linear problems (Manjunath, 2009). In the race to deliver new and innovative products to market faster, manufacturers face many challenges, including globalization, cost reductions, and shorter development cycles (Simulia, 2012). To gain a competitive advantage in the marketplace, manufacturers have started to leverage the robust capabilities of realistic simulation to lessen dependency on physical testing, reduce part weight, or evaluate the use of alternative materials during the product design phase to ensure optimum product performance (Simulia, 2012).

ABAQUS is a powerful and comprehensive tool which provides the user with the following:

i. A powerful modeling environment,
ii. An extensive library of material models,
iii. A comprehensive meshing environment,
iv. Comprehensive contact modeling capabilities,
v. An advanced analysis which include linear, nonlinear and robust multi-physics capabilities,
vi. A high performance computing,
vii. Best-in-class visualization capabilities, and
viii. Analysis using specialized techniques.

CHAPTER THREERESEARCH METHODOLOGY

3.1 Introduction

The methodology in this chapter involves the design of a simple reinforced concrete raft foundation using the conventional method of design. Other models are designed and tested using FEA which have additional compression reinforcement at various percentages of the reinforcement ratio based on the cross sectional area. This is to determine the effect of the compression reinforcement in providing resistance against differential settlement. The design is carried out according to Eurocode 2 (EN 1992-1-1:2004), which specified the depth of foundation, area and amount of reinforcement, and all the necessary checks used in the design calculations.

The finite element analysis (FEA) is carried out with the aid of a computer program. The program that is used is SIMULIA ABAQUS 6.10. ABAQUS is a finite element analysis software that is used in a wide range of industries like automotive, aerospace etc., and is also extensively used in academic and research institutions due to its capability to address non-linear problems (Manjunath, 2009). The ABAQUS program can be used to model reinforced concrete structures,analyse and generate test results using a state of the art 3D modeling and finite element technology. The type of analysis carried out in this research is non-linear involving reinforced concrete.

During the modeling in ABAQUS, the analysis parts for the soil, slab, and reinforcement are created and assigned material and section properties. The embedded element option is used to detail the reinforcements in the slab. The elastic foundation option is used to model the soil surface to make it act as springs to ground which includes the stiffness effects of a support (such as the soil under a building) without modeling the details of the support. The parts are then assembled together, the loads and boundary conditions imposed and the job executed to obtain the results.

This research work is limited to the design, modeling and analysis of the flat reinforced concrete raft foundation without any experimental study.

3.2 Design of Raft Foundation

The raft foundation consists of a regular arrangement of eight column loads with four corner and four internal loads. All the corner columns carry a load of 458.33 *KN* each and the internal columns carry 666.66 *KN* each. Each column is 0.5 *m* by 0.5 *m*. Bearing capacity of the soil will be taken as 100 KN/m^2. The characteristic strengths of the concrete and steel to be used in the design are 45 *MPa* and 500 *MPa* respectively. The Poisson ratio and density of the concrete will be taken as 0.2 and 2400 *kg/m^3* respectively while the Poisson ratio and density of the steel will be taken as 0.3 and 7850 *kg/m^3* respectively. The full details of the design are shown in chapter four.

3.3 Finite Element Analysis

The size and dimension of the soil layer and raft foundation model adopted for the finite element analysis is as shown in Fig. 3.1.

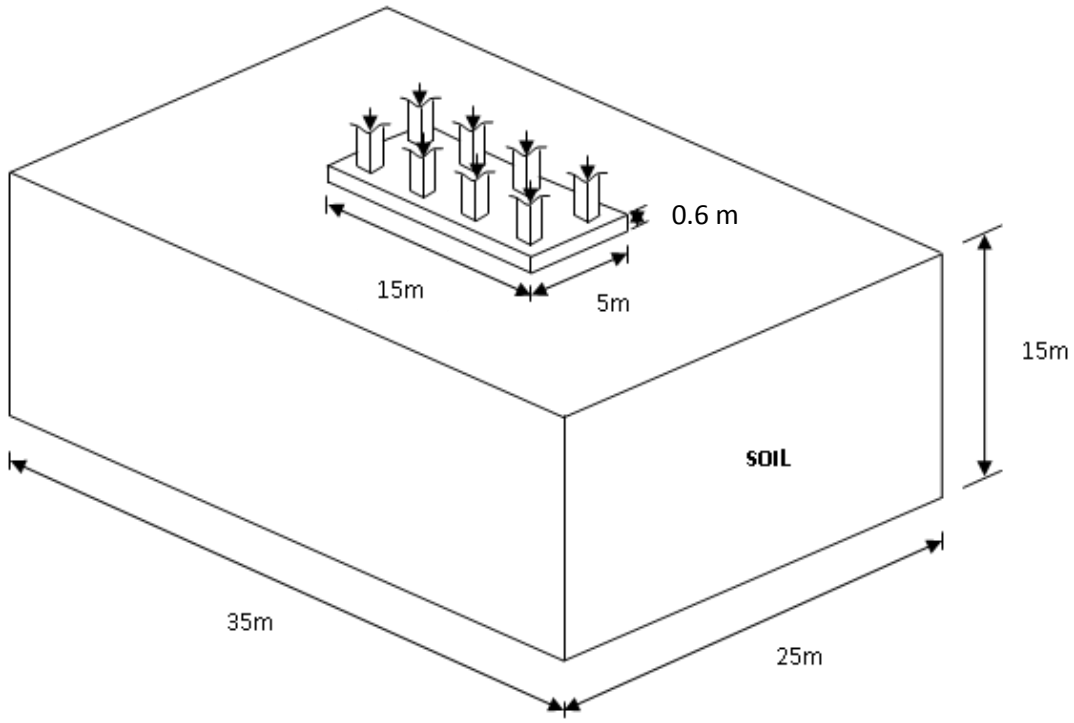


Fig. 3.1 The Raft Foundation and Soil layer Configuration Adopted for The Finite Element Analysis.

The 3D deformable solid parts involved in the analysis consist of the soil layer, concrete slab and steel reinforcements. The dimensions of the steel reinforcements used are obtained from the foundation design as seen in Fig.4.9 while the dimension of the soil layer and the slab can be seen in Fig. 3.1 above.

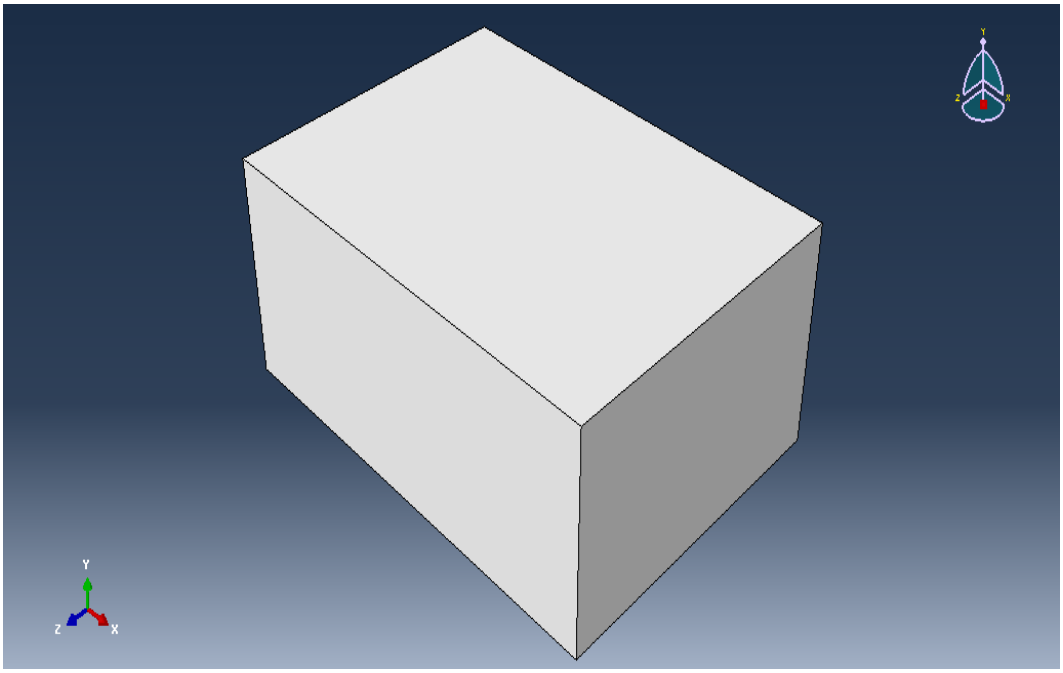


Fig. 3.2 The Soil Layer Part in ABAQUS

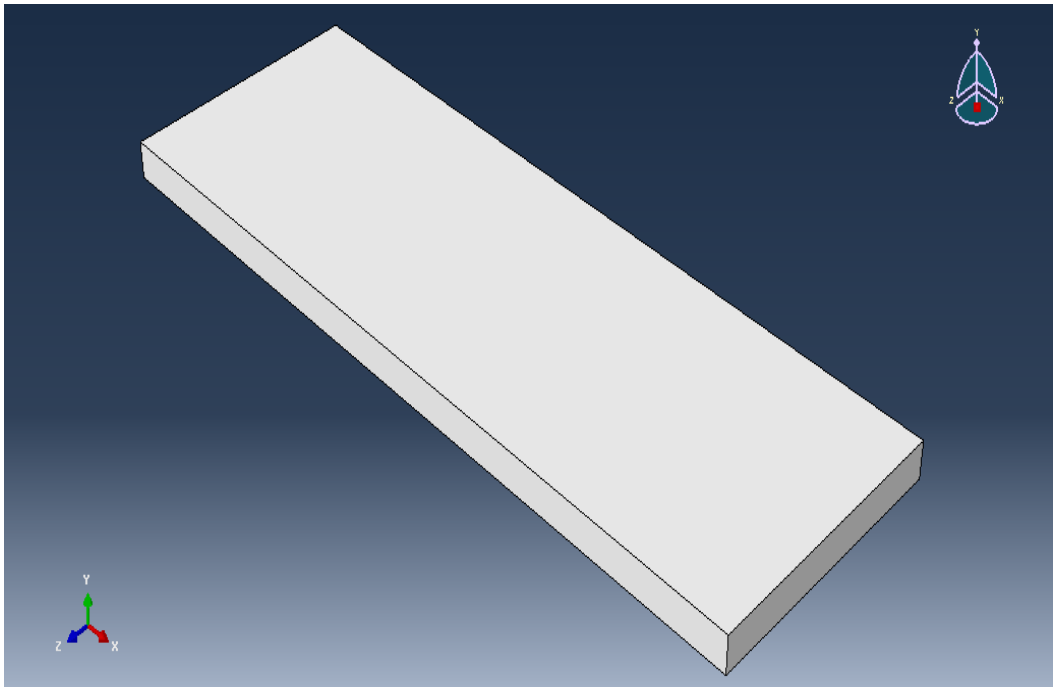


Fig. 3.3 The Concrete Slab Part in ABAQUS

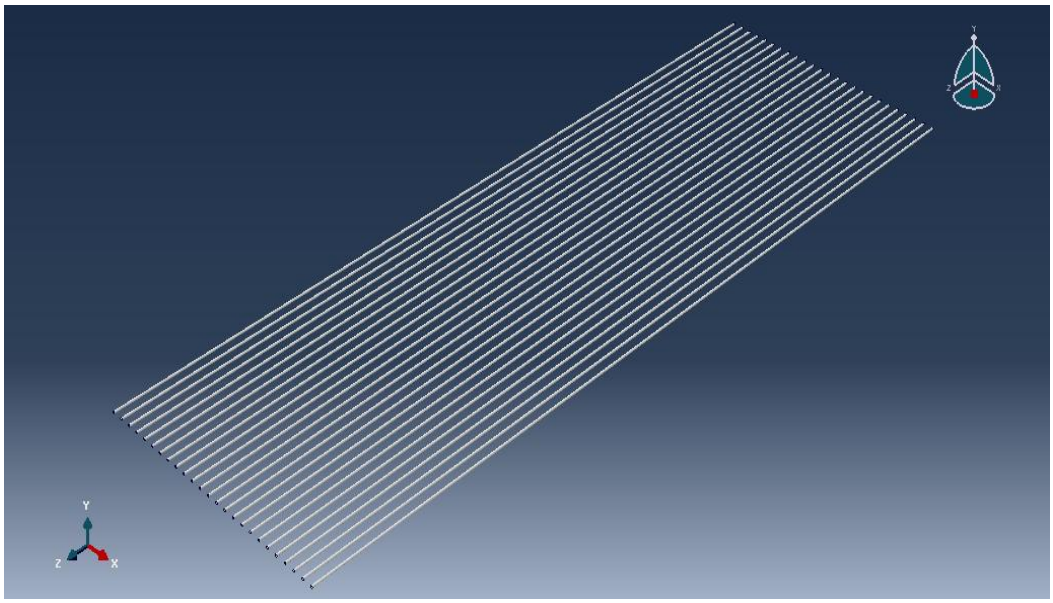


Fig. 3.4 The Tensile Reinforcement Running Along The Length

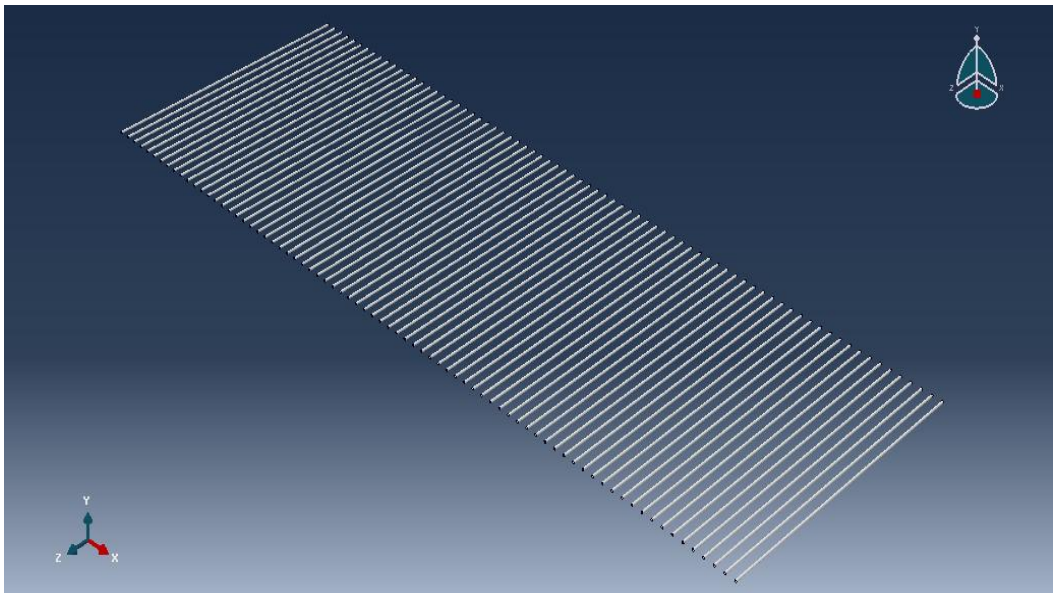


Fig. 3.5 The Tensile Reinforcement Running Along The Width

Elastic soil, concrete and steel are used to define the properties of the parts in the model. Soil, concrete and steel sections are then created using these properties and the sections are assigned to the individual parts accordingly. An elastic material is used for the soil with an isotropic hardening rule. Elastic foundations allow the modeling of the stiffness effects of a distributed support without actually modeling the details of the support (Simulia, 2012).

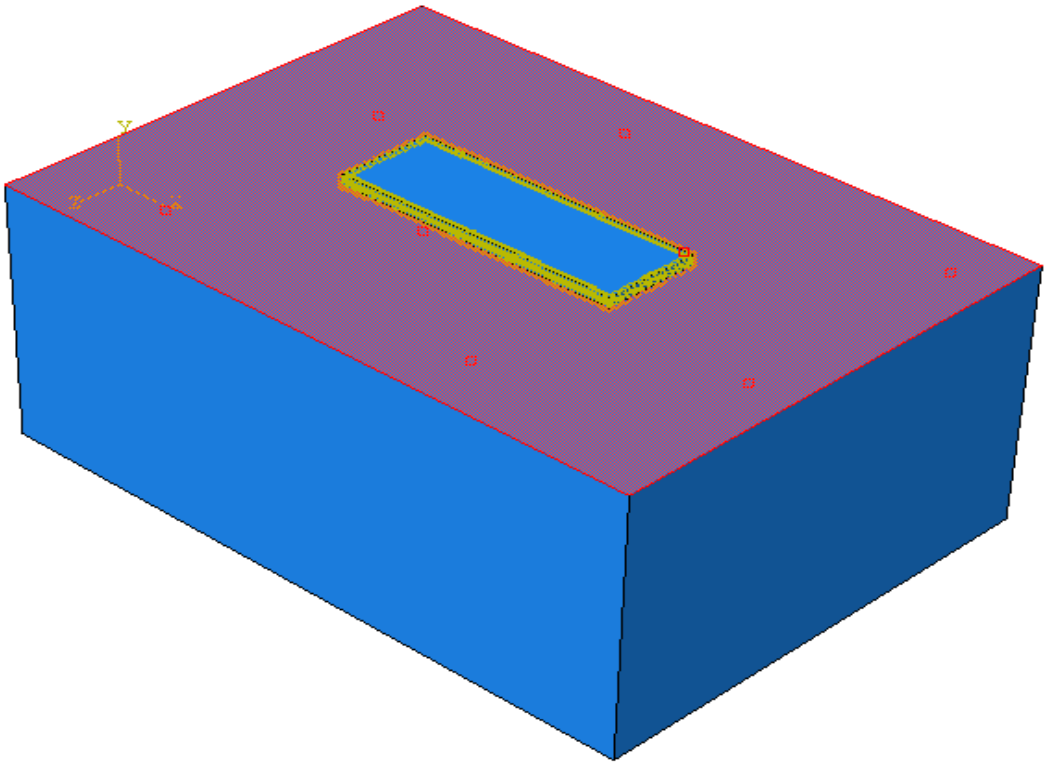


Fig. 3.6 The Interaction Between The Concrete Slab and The Elastic Soil Surface

The soil material is assumed to have a density of $1900 kg/m^3$ and is assigned a Poisson's ratio of 0.3. Foundation stiffness per area of $100 \times 10^3 N/m^2$ is applied to the top surface of the soil layer. A compressive strength of $45 MPa$ is assigned to the concrete part. A plastic strain of 0.0035 and a density of $2400 kg/m^3$ are also assumed for the concrete. The young's modulus and the Poisson's ratio are taken to be $36 GPa$ and 0.2 respectively. An elastic, perfectly plastic material is used for the reinforcing bars. The reinforcing bars are 3D solid elements embedded in the concrete. They exhibit an elastic-plastic behavior and the transfer of loads to the concrete through the reinforcements is achieved by introducing tension stiffening to the concrete model. The embedded element option is used to model the interaction/bond between the concrete and the reinforcing bars. The reinforcing bars are the embedded elements while the concrete slab is the host element. A solid homogeneous steel section is assigned to the reinforcing bars with isotropic hardening. The steel is assigned a tensile strength and strain of $500 MPa$ and 0.003 respectively. The density of steel is taken as $7850 kg/m^3$. A Young's modulus and

Poisson's ratio of 200*GPa* and 0.3 are assigned to the reinforcements respectively.

Linear hexahedral elements of type C3D8R are used in defining the mesh for the entire assembly. This fine mesh helps in improving the accuracy of the results obtained after the analysis.

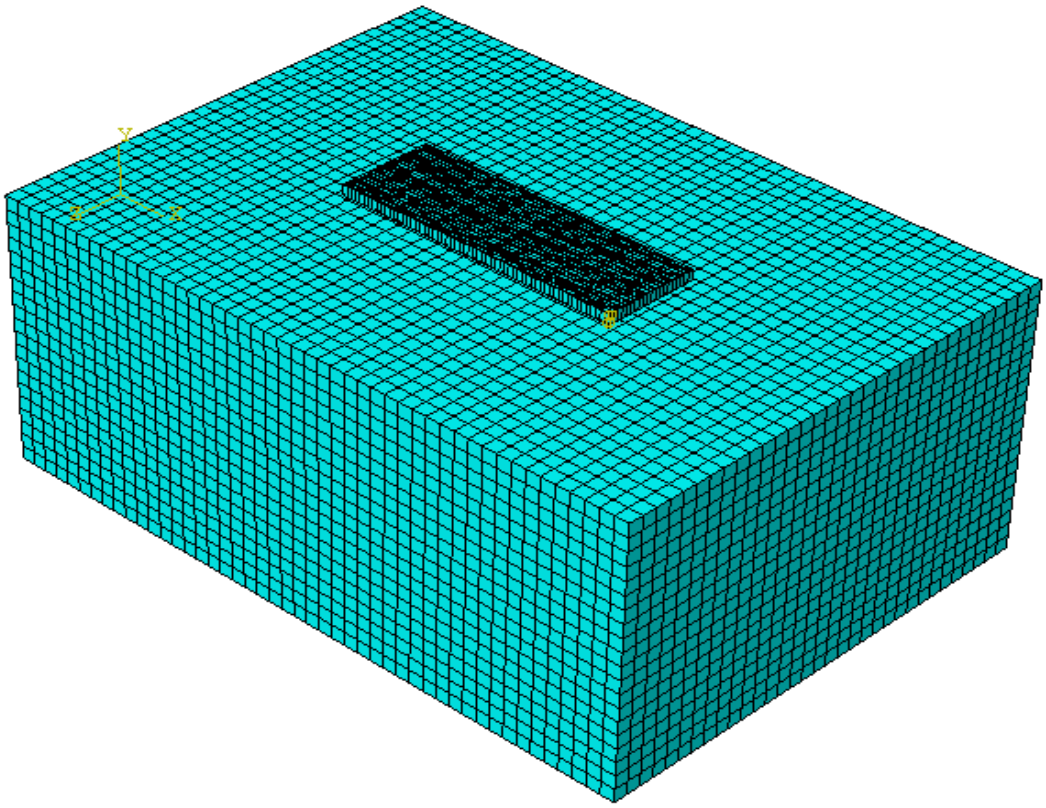


Fig. 3.7 The Discretization of The Structure into Elements and Nodes.

Concentrated loads and boundary conditions are applied to the assembly on the surfaces of the parts. The loads are applied on the surface of the nodes of the concrete slab. Four edge loads and four internal loads are applied with magnitudes of $458.33\times10^{3}N$ and $666.66\times10^{3}N$ respectively. The loads are applied along the y-axis and are given a negative value for downward action. An encastre boundary condition (U1 = U2 = U3 = UR1 = UR2 = UR3 = 0) is applied to the sides and bottom of the soil block which restricted it from moving or rotating in all directions. The top surface of the soil layer is not restricted and is allowed to

deform in all directions. The edges of the slab are assigned a boundary condition in the form of YASYMM (U1 = U3 = UR2 = 0) which only allows movement along the y-axis and rotation along both x-axis and z-axis. The aim of this is to allow the raft foundation to deform in the direction of the loads.

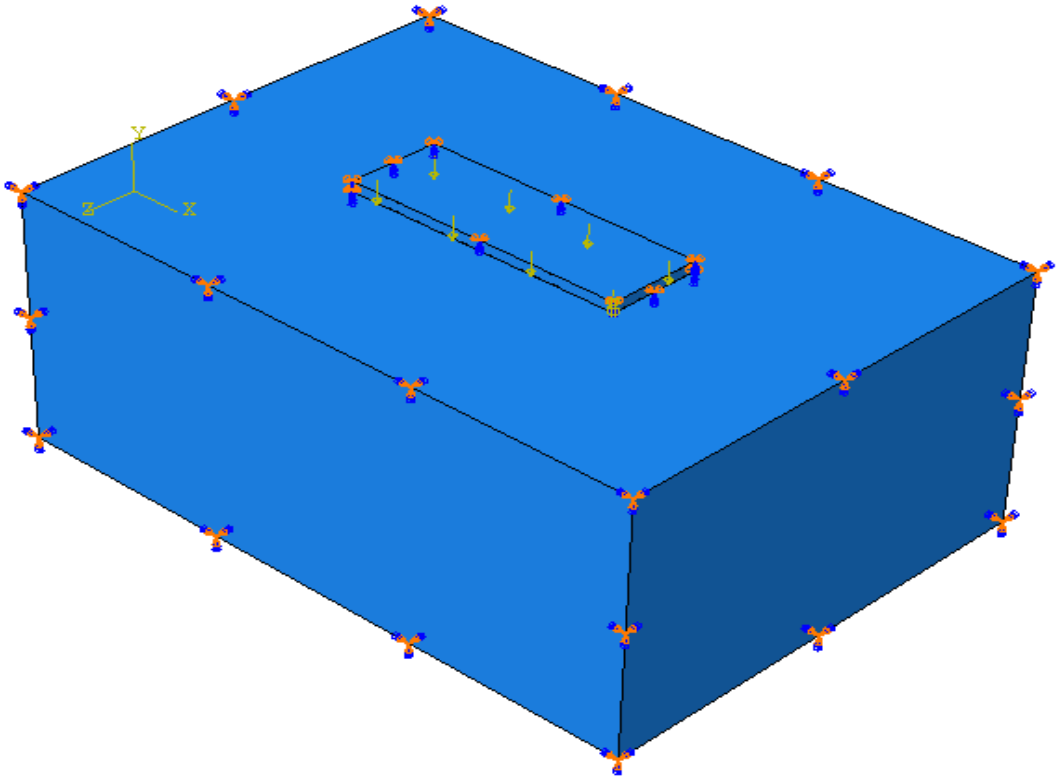


Fig. 3.8 The Applied Loads and Boundary Conditions

Tl ted "*slab load step*". Interactions are created in the initial step while loads and boundary conditions are created and applied in the slab load step. The slab load step has a maximum number of 100 increments. The initial increment size is 1, the minimum 1E-005 and the maximum 1.

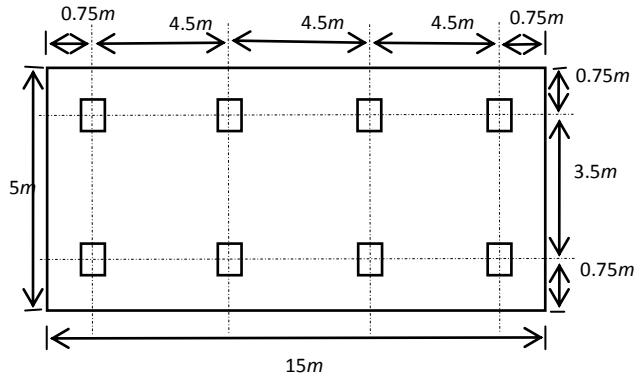
A set of field output and history output requests are created in the slab load step. A full analysis job is created, submitted and run with results obtained. The analysis is repeated for models with additional amount of compression reinforcement; the additional amount ranging from 0.1% to 0.9% of the cross sectional area of the raft slab.

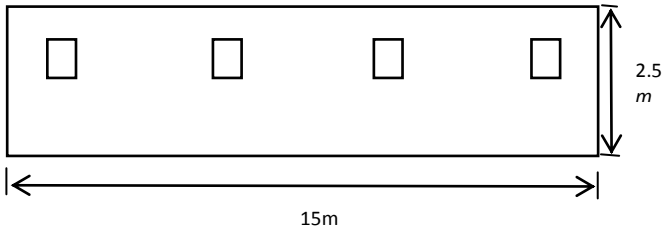
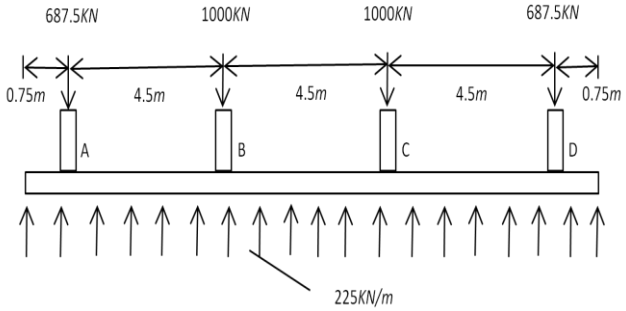
CHAPTER FOUR

RESULTS

4.1 Design of Raft Foundation According to Eurocode 2

S/N	CALCULATION	OUTPUT
	The raft consists of 8 column loads with 4 corner and 4 internal loads of an office building. All the corner columns carry a load of 458.33 *KN* each and internal columns carry 666.66 *KN* each. Bearing capacity of the soil is 100 KN/m^2. Each column is 0.5 *m* by 0.5 *m*	
	4.1.1 Design of a Simple Raft Foundation	
Clause 2.3.1 EN 1992-1-1:2004	*Design loads:* Each internal column load = 1.5 × 666.66 = 1000 *KN* Each corner column load = 1.5 × 458.33 = 687.5 *KN* Dimension of columns = 0.5 *m* × 0.5 *m*	
Clause 6.8 EN 1997-1:2004	*Soil definition:* *Allowable bearing pressure = 100 KN/m^2 *The bearing capacity is assumed to be distributed linearly. *Number of types of soil forming the sub-soil = Two or more types *Soil density = Firm	
Table 3.1 EN 1992-1-1:2004	*Raft slab definition:* Concrete strength, f_{ck} = 45 *MPa* Ultimate strain of concrete = 0.0035	To avoid punching shear
Clause 3.1.3 EN 1992-1-1:2004	Poisson's ratio of concrete, $\nu = 0.2$	
Clause 3.2.2 EN 1992-1-1:2004	Slab mesh reinforcement strength, f_{yk} = 500 *MPa*	
Table 2.1N EN 1992-1-1:2004	Partial safety factor for concrete, $\gamma_c = 1.5$ Partial safety factor for steel reinforcement, γ_s = 1.15	

S/N	CALCULATION	OUTPUT
Clause 4.4.1 EN 1992-1-1:2004	Concrete cover top and bottom, C_{nom} = 40 *mm* Assume effective depth, d = 520 *mm* Overall depth, h = 600 *mm*	
Clause 3.2.7 EN 1992-1-1:2004	Density of concrete = 2400 Kg/m^3 Density of steel reinforcement = 7850 Kg/m^3 Modulus of elasticity of steel, Es = 200 *GPa* *Basic Loading* 1. Load transferred by columns $(4 \times 687.5) + (4 \times 1000) = 6750$ *KN* 2. Self weight of the foundation at 10% $(0.1 \times 6750) = 675$ *KN* Total load = 6750 + 675 = 7425 *KN* Area of foundation required = Total load/Bearing capacity = 7425/100 = 74.25 m^2 Hence, provide 5 *m* by 15 *m* (75 m^2) foundation as shown in Fig. 4.1  *Fig. 4.1 Plan of The Raft Foundation* Net upward pressure = $\frac{(1000 \times 4) + (4 \times 687.5)}{(5 \times 15)}$ = 90 KN/m^2 = 90,000 N/m^2 The raft is designed as a continuous footing in two directions.	**Total load = 7425 *KN*** **Area of foundation = 74.25 m^2** **Net upward pressure 90,000 N/m^2**

S/N	CALCULATION	OUTPUT
	4.2.1.1 Design along the longer span (Bottom)  *Fig. 4.2 Half the Width of The Raft Foundation* Upward pressure per metre length = Net upward pressure × Width = 90,000 × 2.5 = 225,000 *N/m* = 225 *KN/m* *Shear force and bending moment (S.F. and B.M.):*  *Fig. 4.3 Loads and Reaction for The Longer Span* *Shear Force (S.F.)* *S.F. at cantilever end, = 225 × 0.75 = 168.75 *KN* *S.F. at point A, = 168.75 – 687.5 = -518.75 *KN* *S.F. between points A and B, = -518.75 + (4.5 ×225) = 493.75 *KN* *S.F. at point B, = 493.75 -1000 = -506.25 *KN* *S.F. between points B and C, = -506.25 + (4.5 × 225) = 506.25 *KN* *S.F. at point C, = 506.25 – 1000 = -493.75 *KN* *S.F. between points C and D, = -493.75 + (4 × 225) = 518.75 *KN* S.F. at point D, = 518.75 – 687.5 = -168.75 *KN*	**Upward pressure = 225 *KN/m***

S/N	CALCULATION	OUTPUT

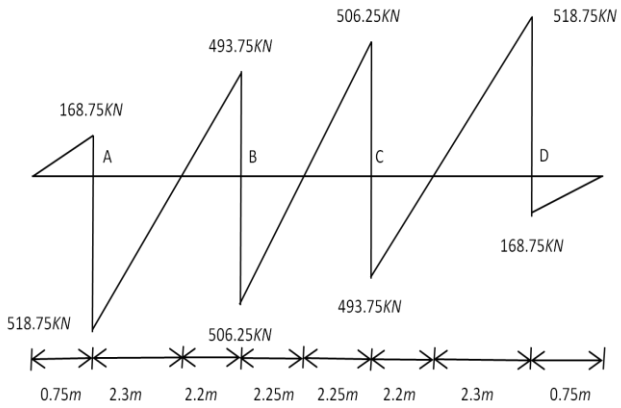


Fig. 4.4 Shear Force Diagram (S.F.D.)

Bending Moment (B.M.)

B.M. at both cantilever ends, $= \frac{225 \times 0.75^2}{2}$

$= 63.28\ KNm$

B.M. between points A & B and points C & D

$= (687.5 \times 2.3) - \frac{(225 \times 3.05^2)}{2} = 534.72\ KNm$

B.M. at points B and C

$= \frac{(225 \times 5.25^2)}{2} - (687.5 \times 4.5) = 7.03\ KNm$

B.M. between points B & C at 7.5 m

$= (687.5 \times 6.75) - \frac{(225 \times 7.5^2)}{2} + (1000 \times 2.25)$

$= 562.50\ KNm$

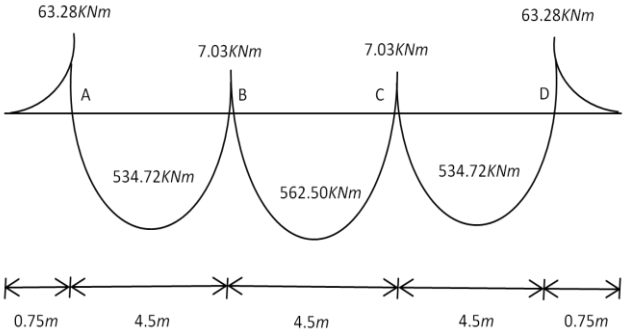
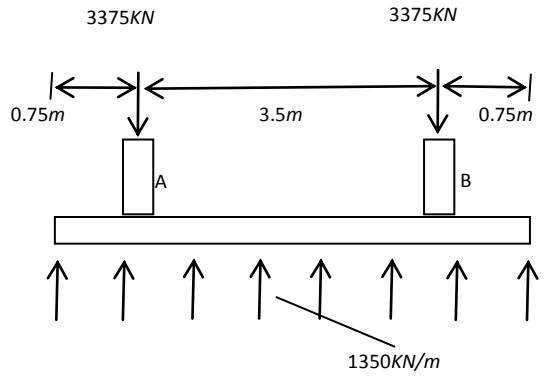


Fig. 4.5 Bending Moment Diagram

S/N	CALCULATION	OUTPUT
	Tension reinforcement required in cantilever: Maximum cantilever moment, M_{max} = 63.28 *KNm*	**M_{max} = 63.28 *KNm***
Clause 6.1 EN 1992-1-1:2004	$K_{factor} = \frac{M_{max}}{f_{ck}bd^2} \leq K_{bal} = 0.167$ $= \frac{63.28 \times 10^6}{45 \times 2500 \times 520^2} = 0.00208 = 2.08 \times 10^{-3}$	**K = 0.00208**
	$K_{factor} = 0.00208 < K_{bal} = 0.167$ Therefore, no compression reinforcement is required.	
Clause 6.1 EN 1992-1-1:2004	Lever arm, $Z = d(0.5 + \sqrt{\left(0.25 - \frac{K}{1.134}\right)}$ = 519.04 *mm*	**Z = 519.04 *mm***
Clause 6.1 EN 1992-1-1:2004	Area of steel required, $As = \frac{M_{max}}{\left(\frac{1.0}{\gamma s}\right) f_{yk} Z}$ or $= \frac{M_{max}}{0.87 f_{yk} Z}$	
	$= \frac{63.28 \times 10^6}{0.87 \times 500 \times 519.04} = 280.27\ mm^2$	**As = 280.27 *mm²***
	For full length of 5 m, As = 280.27 × 2 = 560.54 *mm²*	
	Provide 17H8 @ 300 *mm* B (854.52 *mm²*) Bottom (cantilever)	**17H8 @ 300 *mm* B**
	Tension reinforcement required for internal slab Maximum cantilever moment, M_{max} = 562.5 *KNm*	
Clause 6.1 EN 1992-1-1:2004	$K_{factor} = \frac{M_{max}}{f_{ck}bd^2} \leq K_{bal} = 0.167$ $= \frac{562.50 \times 10^6}{45 \times 2500 \times 520^2} = 0.0185 = 18.5 \times 10^{-3}$	**K = 0.0185**
	$K_{factor} = 0.0185 < K_{bal} = 0.167$ Therefore, no compression reinforcement is required.	

S/N	CALCULATION	OUTPUT
	Lever arm, $Z = d(0.5 + \sqrt{\left(0.25 - \frac{K}{1.134}\right)}$ = 511.37 *mm*	
Clause 6.1 EN 1992-1-1:2004	Area of steel required, As = $\frac{M_{max}}{\left(\frac{1.0}{\gamma s}\right) f_{yk} Z}$ or $= \frac{M_{max}}{0.87 f_{yk} Z}$ $= \frac{562.50 \times 10^6}{0.87 \times 500 \times 511.37}$ = 2528.704 mm^2	
	For full length of 5 m, As = 2528.704 × 2 = 5057.41 mm^2	
	Provide 17H20 @ 300 *mm* B (5340.71 mm^2) Bottom (spans)	**17H20 @ 300 mm B**
	Design checks: *Check for shear*	
Clause 6.2.2 EN 1992-1-1:2004	$V_{ED} = V_{max}$ V_{ED} = 518.75 *KN* b_w = 2500 *mm* d = 520 *mm* f_{ck} = 45 *MPa*	**V_{ED} = 518.75 *KN***
Clause 6.2.2 EN 1992-1-1:2004	$\rho_1 = \frac{A_{st}}{b_w d} \leq 0.02$ $\rho_1 = \frac{2528.70}{2500 \times 520}$ = 0.00195 < 0.02	**ρ_1 = 0.00195**
	$C_{Rd,c} = \frac{0.18}{\gamma_c}$ = 0.12	**$C_{Rd,c}$ = 0.12**
	$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ K = 1.62 < 2.0	**K = 1.62**
	$V_{Rd,c} = [C_{Rd,c} K \sqrt[3]{(100 \rho_1 f_{ck})}] b_w d$ $= [0.12 \times 1.62 \times \sqrt[3]{100 \times 0.00195 \times 45} \times 2500 \times 520$ = 521.26 *KN*	**$V_{Rd,c}$= 521.26 KN**

S/N	CALCULATION	OUTPUT
	$V_{Rd,c} = 521.26\,KN > V_{ED} = 518.75\,KN$ Therefore, the shear capacity of the slab is adequate. *Check for deflection*	
Clause 7.4.2 EN 1992-1-1:2004	$\rho = \frac{100A_{req}}{bd}$ $\rho = \frac{100 \times 2528.71}{2500 \times 520}$ = 0.1945 %	
Table 7.4N EN 1992-1-1:2004	From table 7.4N, K = 1.5 From figure, Basic span-effective depth ratio = 36 Basic span-effective depth ratio = 36 × 1.5 = 54	
Clause 7.4.2 EN 1992-1-1:2004	$Modification\ ratio = 54 \times \frac{A_{s,prov}}{A_{s,req}}$ $Modification\ ratio = 54 \times \frac{3925}{2528.71}$ = 83.82	**MR = 83.82**
	$Ratio_{allow}$ = 83.82	**R_{allow} = 83.82**
	$Ratio_{actual} = \frac{Span}{d} = \frac{1500}{520} = 28.85$	**R_{actual} = 28.85**
	$Ratio_{actual} < Ratio_{allow}$ Therefore, slab span to depth ratio is adequate.	
Clause 6.4.3 EN 1992-1-1:2004	*Punching shear check* V_{ED} = 518.75 *KN* $V_{Rd,c} = 0.5ud \times \left[0.6\left(\frac{1-f_{ck}}{250}\right)\right] \times 1.5f_{ck}$	**V_{ED} = 518.75 KN**

S/N	CALCULATION	OUTPUT
	Where u = perimeter of column = 4 × 500 = 2000 *mm* d = 520 *mm* f_{ck} = 45 *MPa* $V_{Rd,c}$ = 7675.2 *KN* $V_{Rd,c}$ = 7675.2 *KN*> V_{ED} = 518.75 *KN* Therefore, the shear capacity of the slab is adequate. *4.2.1.2 Design along the shorter span (Bottom)* Upward pressure per metre length = Net upward pressure × Width = 90,000 × 15 = 1,350,000 *N/m* = 1350 *KN/m*  *Fig. 4.6 Loads and Reaction for The Shorter Span* *Shear force and bending moment (S.F. and B.M.):* *Shear Force (S.F.)* S.F. at cantilever end, = 1350 × 0.75 = 1012.5 *KN* S.F. at point A, = 1012.5 – 3375 = -2362.5 *KN* S.F. between points A and B, = -2362.5 + (3.5 × 1350) = 2362.5 *KN* S.F. at point B, = 2362.5 – 3375 = 1012.5 *KN*	**$V_{Rd,c}$= 7675.2 *KN*** **Upward pressure = 1350 *KN/m*** **Upward pressure = 1350 *KN/m***

S/N	CALCULATION	OUTPUT

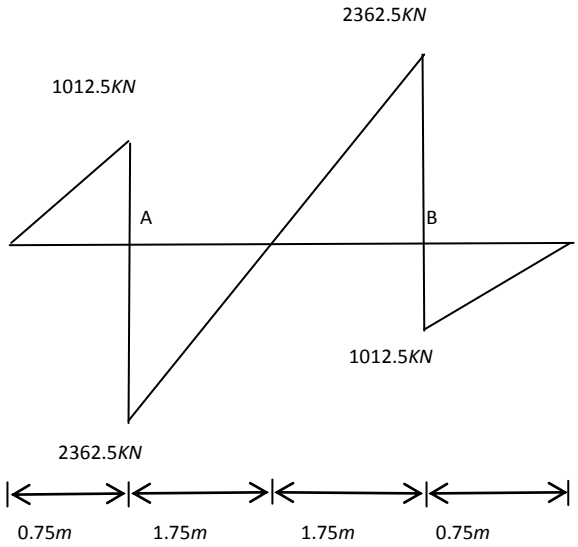


Fig. 4.7 Shear Force Diagram (S.F.D.)

Bending Moment (B.M.)

$$B.M.\,at\ both\ cantilever\ ends, = \frac{1350\ \times 0.75^2}{2}$$

$$= 379.69\ KNm$$

$$B.M.\,at\ both\ cantilever\ ends,$$

$$= 3375 \times 1.75 - \left(\frac{1350\ \times 3.05^2}{2}\right)$$

$$= 1687.50\ KNm$$

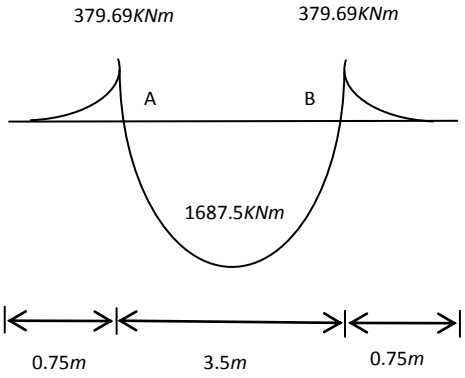


Fig. 4.8 Bending Moment Diagram

S/N	CALCULATION	OUTPUT

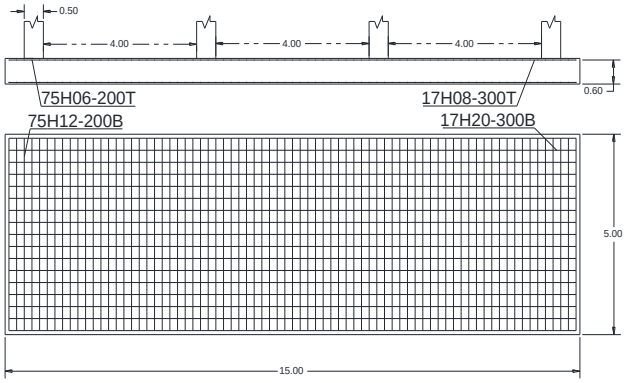
	Tension reinforcement required in cantilever Maximum cantilever moment, M_{max} = 379.69 *KNm*	**M_{max} = 379.69 *KNm***
Clause 6.1 EN 1992-1-1:2004	$K_{factor} = \frac{M_{max}}{f_{ck} b d^2} \leq K_{bal} = 0.167$	
	$= \frac{379.69 \times 10^6}{45 \times 15000 \times 520^2} = 0.00208 = 2.08 \times 10^{-3}$	**K = 0.00208**
	$K_{factor} = 0.00208 < K_{bal} = 0.167$ Therefore, no compression reinforcement is required.	
Clause 6.1 EN 1992-1-1:2004	Lever arm, $Z = d(0.5 + \sqrt{\left(0.25 - \frac{K}{1.134}\right)}$ = 519.04 *mm*	**Z = 519.04 *mm***
Clause 6.1 EN 1992-1-1:2004	Area of steel required, $As = \frac{M_{max}}{\left(\frac{1.0}{\gamma s}\right) f_{yk} Z}$ or $= \frac{M_{max}}{0.87 f_{yk} Z}$	
	$= \frac{379.69 \times 10^6}{0.87 \times 500 \times 519.04} = 1681.66\ mm^2$	**As = 1681.66 *mm²***
	Provide 75H6 @ 200 *mm* B (2120.58 mm^2) Bottom (cantilever)	**75H6 @ 200 *mm* B**
	Tension reinforcement required for internal slab Maximum cantilever moment, M_{max} = 1687.5 *KNm*	**M_{max} = 1687.5 *KNm***
Clause 6.1 EN 1992-1-1:2004	$K_{factor} = \frac{M_{max}}{f_{ck} b d^2} \leq K_{bal} = 0.167$	
	$= \frac{1687.5 \times 10^6}{45 \times 15000 \times 520^2} = 0.00925 = 9.25 \times 10^{-3}$	**K = 0.00925**
	$K_{factor} = 0.00925 < K_{bal} = 0.167$ Therefore, no compression reinforcement is required.	

S/N	CALCULATION	OUTPUT

Clause 6.1 EN 1992-1-1:2004	Lever arm, $Z = d(0.5 + \sqrt{\left(0.25 - \frac{K}{1.134}\right)}$ = 515.72 *mm*	**Z = 515.72 *mm***
Clause 6.1 EN 1992-1-1:2004	Area of steel required, As = $\frac{M_{max}}{\left(\frac{1.0}{\gamma s}\right) f_{yk} Z}$ or $= \frac{M_{max}}{0.87 f_{yk} Z}$ $= \frac{1687.50 \times 10^6}{0.87 \times 500 \times 515.72} = 7522.13\ mm^2$	**As = 7522.13 *mm*^{2}**
	Provide 75H12 @ 200 *mm* B (8482.3 *mm*^{2}) Bottom (spans)	**75H12 @ 200 *mm* B**
Clause 6.2.2 EN 1992-1-1:2004	*Design checks:* *Check for shear* $V_{ED} = V_{max}$ V_{ED} = 2362.5 *KN* b_w = 15000 *mm* d = 520 *mm* f_{ck} = 45 *MPa*	**V_{ED} = 2362.5 *KN***
Clause 6.2.2 EN 1992-1-1:2004	$\rho_1 = \frac{A_{st}}{b_w d} \leq 0.02$ $\rho_1 = \frac{7522.13}{15000 \times 520} = 0.000964 < 0.02$	**ρ_1 = 0.000964**
	$C_{Rd,c} = \frac{0.18}{\gamma_c} = 0.12$	**$C_{Rd,c}$ = 0.12**
	$K = 1 + \sqrt{\frac{200}{d}} \leq 2.0$ K = 1.62 < 2.0	**K = 1.62**
	$V_{Rd,c} = [C_{Rd,c} K \sqrt[3]{(100 \rho_1 f_{ck})}] b_w d$ $= [0.12 \times 1.62 \times \sqrt[3]{100 \times 0.000964 \times 45} \times 15000 \times 520$ = 2472.98 *KN*	**$V_{Rd,c}$= 2472.98 *KN***
	$V_{Rd,c} = 2472.98\ KN > V_{ED} = 518.75\ KN$ Therefore, the shear capacity of the slab is adequate.	

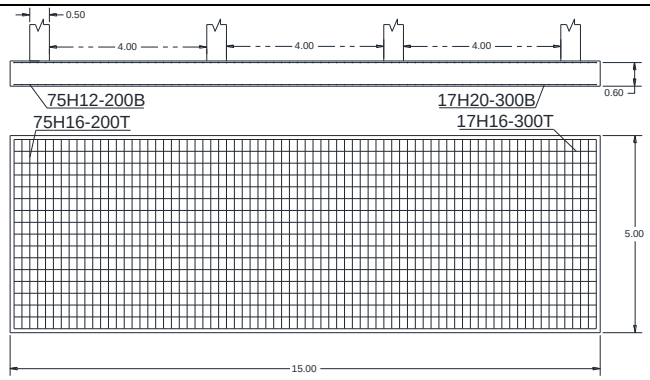
S/N	CALCULATION	OUTPUT
Clause 7.4.2 EN 1992-1-1:2004	*Check for deflection* $$\rho = \frac{100A_{req}}{bd}$$ $$\rho = \frac{100 \times 7522.13}{15000 \times 520}$$ = 0.0964 %	$\boldsymbol{\rho = 0.0964}$ %
Table 7.4N EN 1992-1-1:2004	From table 7.4N, K = 1.5 From figure, Basic span-effective depth ratio = 36 Basic span-effective depth ratio = 36 × 1.5 = 54	
Clause 7.4.2 EN 1992-1-1:2004	$Modification\ ratio = 54 \times \frac{A_{s,prov}}{A_{s,req}}$ $Modification\ ratio = 54 \times \frac{10050}{7522.13}$ = 72.15	**MR = 72.15**
	$Ratio_{allow}$ = 72.15	**R_{allow} = 72.15**
	$Ratio_{actual} = \frac{Span}{d} = \frac{5000}{520} = 9.62$	**R_{actual} = 9.62**
	$Ratio_{actual} < Ratio_{allow}$ Therefore, slab span to depth ratio is adequate.	
Clause 6.4.3 EN 1992-1-1:2004	*Punching shear check* V_{ED} = 2362.5 *KN* $V_{Rd,c} = 0.5ud \times \left[0.6\left(\frac{1-f_{ck}}{250}\right)\right] \times 1.5f_{ck}$ Where u = perimeter of column = 4 × 500 = 2000 *mm* d = 520 *mm* f_{ck} = 45 *MPa*	**V_{ED} = 2362.5 KN**
	$V_{Rd,c}$ = 7675.2 *KN*	**$V_{Rd,c}$= 7675.2 *KN***

S/N	CALCULATION	OUTPUT
	$V_{Rd,c}$ = 7675.2 *KN*> V_{ED} = 2362.5 *KN* Therefore, the shear capacity of the slab is adequate.	
Clause 6.1 EN 1992-1-1:2004	*4.2.1.3 Design along the longer span (Top)* Maximum cantilever moment, M_{max} = 63.28 *KNm*	**M_{max} = 63.28 *KNm***
	$K_{factor} = \frac{M_{max}}{f_{ck} b d^2} \leq K_{bal} = 0.167$ $= \frac{63.28 \times 10^6}{45 \times 2500 \times 520^2} = 0.00208 = 2.08 \times 10^{-3}$	**K = 0.00208**
	K_{factor} =0.00208 <K_{bal} = 0.167 Therefore, no compression reinforcement is required.	
Clause 6.1 EN 1992-1-1:2004	Lever arm, $Z = d(0.5 + \sqrt{\left(0.25 - \frac{K}{1.134}\right)}$ = 519.04 *mm*	**Z = 519.04 *mm***
Clause 6.1 EN 1992-1-1:2004	Area of steel required, As = $\frac{M_{max}}{\left(\frac{1.0}{\gamma_s}\right) f_{yk} Z}$ or $= \frac{M_{max}}{0.87 f_{yk} Z}$	
	$= \frac{63.28 \times 10^6}{0.87 \times 500 \times 519.04}$ = 280.27 mm^2	**As = 280.27 mm^2**
	For full length of 5 m, As = 280.27 × 2 = 560.54 mm^2	
	Provide 17H8 @ 300 *mm* T (854.52 mm^2) Top (column supports)	**17H8 @ 300 *mm* T**
Clause 6.1 EN 1992-1-1:2004	*3.2.1.4 Design along the shorter span (Top)* Maximum cantilever moment, M_{max} = 379.69 *KNm* $K_{factor} = \frac{M_{max}}{f_{ck} b d^2} \leq K_{bal} = 0.167$	
	$= \frac{379.69 \times 10^6}{45 \times 15000 \times 520^2} = 0.00208 = 2.08 \times 10^{-3}$	**K = 0.00208**

S/N	CALCULATION	OUTPUT
Clause 6.1 EN 1992-1-1:2004	K_{factor} =0.00208 <K_{bal} = 0.167 Lever arm, $Z = d(0.5 + \sqrt{\left(0.25 - \frac{K}{1.134}\right)}$ = 519.04 *mm* Area of steel required, As = $\frac{M_{max}}{\left(\frac{1.0}{\gamma s}\right) f_{yk} Z}$ or $= \frac{M_{max}}{0.87 f_{yk} Z}$ = $\frac{379.69 \times 10^6}{0.87 \times 500 \times 519.04}$ = 1681.66 mm^2 Provide 75H6 @ 200 *mm* T (2120.58 mm^2) Top (column supports)  BOTTOM PLAN *Fig. 4.9 Reinforcement Mesh for Simple Raft Foundation* *4.2.2 Design of Simple Raft Foundation With Additional 0.1% Compression Reinforcement* This model is exactly the same as the first model but with 0.1% compression reinforcement. The compression reinforcement was designed and added to the first model using compression reinforcement ratio based on the cross sectional area.	**Z = 519.04 *mm*** **As = 1681.66 mm^2** **75H6 @ 200 *mm* T**

S/N	CALCULATION	OUTPUT
	Compression reinforcement required along the longer span $Compression\ reinforcement\ ratio\ (\%), \rho = \frac{100A_s}{bd}$ Where, As = area of reinforcement b = width of the section d = depth of the section Therefore, $A_s = \frac{\rho bd}{100}$ $= \frac{0.1 \times 15000 \times 520}{100}$ = 7800 mm^2 Total area = 7800 + 1681.66 = 9481.66 mm^2 Provide 75H16 @ 200 *mm* T (15079.64 mm^2) Top *Compression reinforcement required along the shorter span* $A_s = \frac{\rho bd}{100}$ $= \frac{0.1 \times 5000 \times 520}{100}$ = 2600 mm^2 Total area = 2600 + 560.54 = 3160.54 mm^2 Provide 17H16 @ 300 *mm* T (3418.05 mm^2) Top	**As = 9481.66 mm^2** **75H16 @ 200 mm** **As = 3160.54 mm^2** **17H16 @ 300 mm T**

S/N	CALCULATION	OUTPUT



TOP PLAN

Fig. 4.10 Reinforcement Mesh of Raft Foundation with 0.1% Compression Reinforcement

4.2.3 Design of Simple Raft Foundation With Additional 0.2% Compression Reinforcement

This model is exactly the same as the first model but with 0.2% compression reinforcement. The compression reinforcement was designed and added to the first model using compression reinforcement ratio based on the cross sectional area.

Compression reinforcement required along the longer span

$$Compression\ reinforcement\ ratio\ (\%), \rho = \frac{100A_s}{bd}$$

Where, As = area of reinforcement
b = width of the section
d = depth of the section

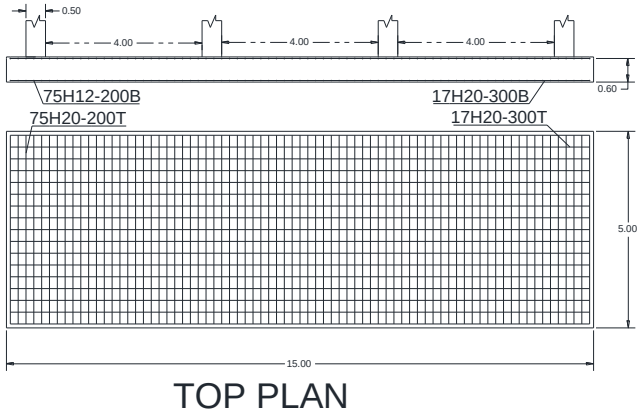
Therefore, $A_s = \frac{\rho bd}{100}$

$$= \frac{0.2 \times 15000 \times 520}{100}$$

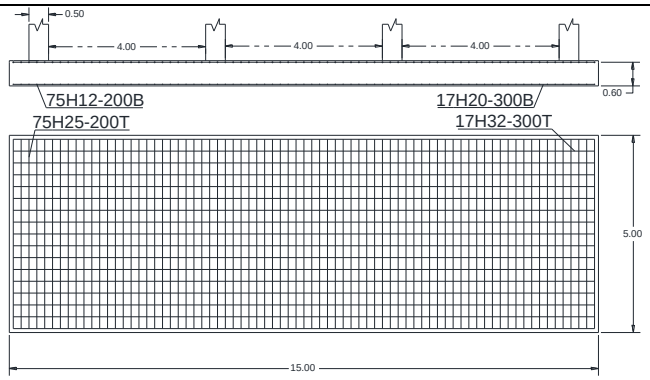
$$= 15600\ mm^2$$

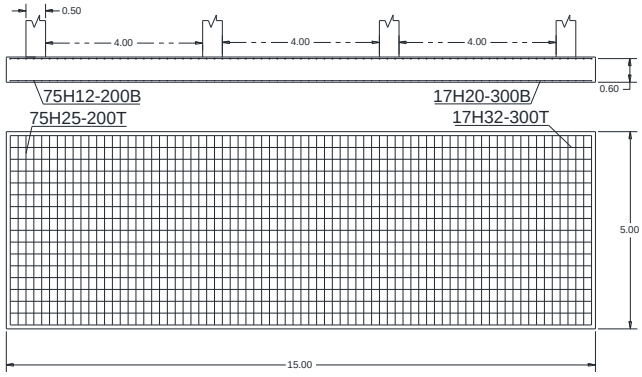
Total area = 15600 + 1681.66 = 17281.66 mm^2

As = 17281.66 mm^2

S/N	CALCULATION	OUTPUT
	Provide 75H20 @ 200 *mm* T (23561.94 mm^2) Top *Compression reinforcement required along the shorter span* $$A_s = \frac{\rho bd}{100}$$ $$= \frac{0.2 \times 5000 \times 520}{100}$$ = 5200 mm^2 Total area = 5200 + 560.54 = 5760.54 mm^2 Provide 17H20 @ 300 *mm* T (8344.86 mm^2) Top  TOP PLAN *Fig. 4.11 Reinforcement Mesh of Raft Foundation with 0.2% Compression Reinforcement* *4.2.4 Design of Simple Raft Foundation With Additional 0.3% Compression Reinforcement* This model is exactly the same as the first model but with 0.3% compression reinforcement. The compression reinforcement was designed and added to the first model using compression reinforcement ratio based on the cross sectional area.	**75H20 @ 200 mm T** **As = 5760.54 mm^2** **17H20 @ 300 mm T**

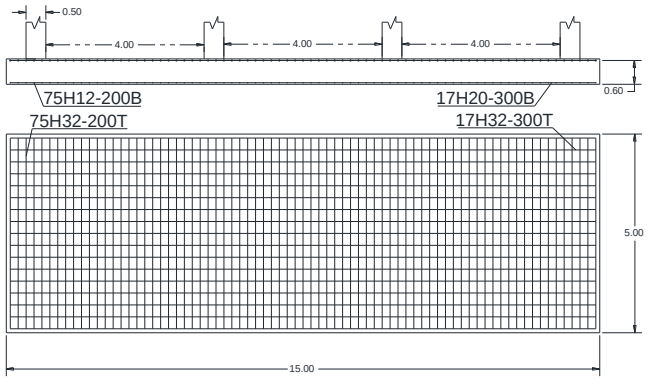
S/N	CALCULATION	OUTPUT
	Compression reinforcement required along the longer span $Compression\ reinforcement\ ratio\ (\%), \rho = \frac{100A_s}{bd}$ Where, As = area of reinforcement b = width of the section d = depth of the section Therefore, $A_s = \frac{\rho bd}{100}$ $= \frac{0.3 \times 15000 \times 520}{100}$ = 23400 *mm²* Total area = 23400 + 1681.66 = 25081.66 *mm²* Provide 75H25 @ 200 *mm* T (36815.54 *mm²*) Top *Compression reinforcement required along the shorter span* $A_s = \frac{\rho bd}{100}$ $= \frac{0.3 \times 5000 \times 520}{100}$ = 7800 *mm²* Total area = 7800 + 560.54 = 8360.54 *mm²* Provide 17H32 @ 300 *mm* T (13672.21 *mm²*) Top	**As = 25081.66 *mm²*** **75H25 @ 200 mm T** **As = 8360.54 *mm²*** **17H32 @ 300 mm T**

S/N	CALCULATION	OUTPUT
	 TOP PLAN *Fig. 4.12 Reinforcement Mesh of Raft Foundation with 0.3% Compression Reinforcement* *4.2.5 Design of Simple Raft Foundation With Additional 0.4% Compression Reinforcement* This model is exactly the same as the first model but with 0.4% compression reinforcement. The compression reinforcement was designed and added to the first model using compression reinforcement ratio based on the cross sectional area. *Compression reinforcement required along the longer span* $Compression\ reinforcement\ ratio\ (\%), \rho = \frac{100A_s}{bd}$ Where, As = area of reinforcement b = width of the section d = depth of the section Therefore, $A_s = \frac{\rho bd}{100}$ $= \frac{0.4 \times 15000 \times 520}{100}$ = 31200 mm^2 Total area = 31200 + 1681.66 = 32881.66 mm^2	**As = 32881.66 mm^2**

S/N	CALCULATION	OUTPUT
	Provide 75H25 @ 200 *mm* T (36815.54 mm^2) Top *Compression reinforcement required along the shorter span* $$A_s = \frac{\rho bd}{100}$$ $$= \frac{0.4 \times 5000 \times 520}{100}$$ $$= 10400\ mm^2$$ Total area = 10400 + 560.54 = 10960.54 mm^2 Provide 17H32 @ 300 *mm* T (13672.21 mm^2) Top  TOP PLAN *Fig. 4.13 Reinforcement Mesh of Raft Foundation with 0.4% Compression Reinforcement* *4.2.6 Design of Simple Raft Foundation With Additional 0.5% Compression Reinforcement* This model is exactly the same as the first model but with 0.5% compression reinforcement. The compression reinforcement was designed and added to the first model using compression reinforcement ratio based on the cross sectional area.	**75H25 @ 200 mm T** **As = 10960.54 mm^2** **17H32 @ 300 mm T**

S/N	CALCULATION	OUTPUT
	Compression reinforcement required along the longer span $Compression\ reinforcement\ ratio\ (\%), \rho = \frac{100A_s}{bd}$ Where, As = area of reinforcement b = width of the section d = depth of the section Therefore, $A_s = \frac{\rho bd}{100}$ $= \frac{0.5 \times 15000 \times 520}{100}$ = 39000 mm^2 Total area = 39000 + 1681.66 = 40681.66 mm^2 Provide 75H32 @ 200 *mm* T (60318.58 mm^2) Top *Compression reinforcement required along the shorter span* $A_s = \frac{\rho bd}{100}$ $= \frac{0.5 \times 5000 \times 520}{100}$ = 13000 mm^2 Total area = 13000 + 560.54 = 13560.54 mm^2 Provide 17H32 @ 300 *mm* T (13672.21 mm^2) Top	**As = 40681.66 mm^2** **75H32 @ 200 mm T** **As = 13560.54 mm^2** **17H32 @ 300 mm T**

S/N	CALCULATION	OUTPUT



TOP PLAN

Fig. 4.14 Reinforcement Mesh of Raft Foundation with 0.5% Compression Reinforcement

4.2.7 Design of Simple Raft Foundation With Additional 0.6% Compression Reinforcement

This model is exactly the same as the first model but with 0.6% compression reinforcement. The compression reinforcement was designed and added to the first model using compression reinforcement ratio based on the cross sectional area.

Compression reinforcement required along the longer span

$$Compression\ reinforcement\ ratio\ (\%), \rho = \frac{100A_s}{bd}$$

Where, As = area of reinforcement
b = width of the section
d = depth of the section

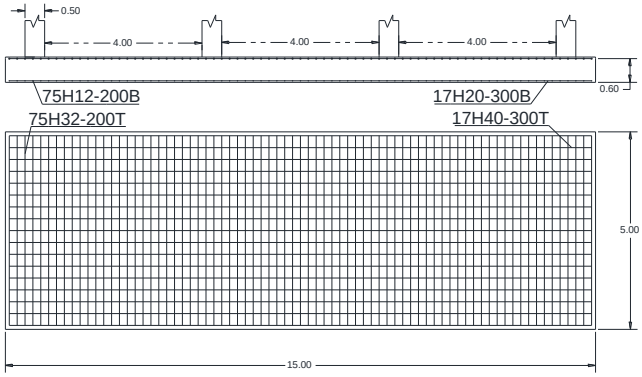
Therefore, $A_s = \frac{\rho bd}{100}$

$$= \frac{0.6 \times 15000 \times 520}{100}$$

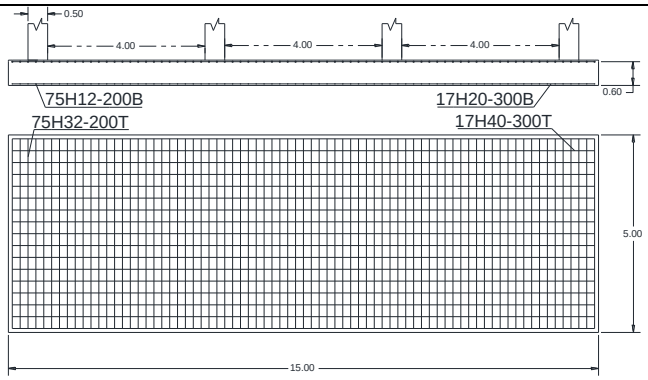
$$= 46800\ mm^2$$

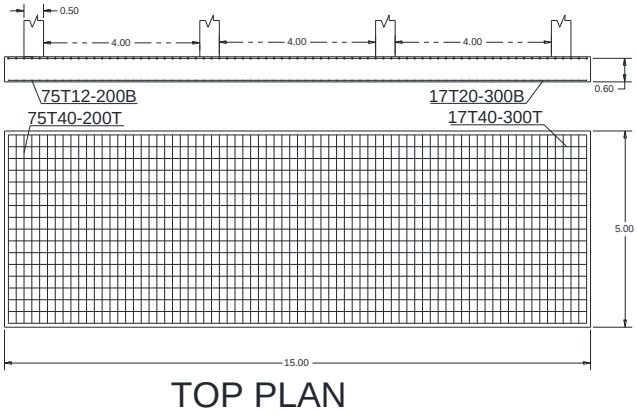
Total area = 46800 + 1681.66 = 48481.66 mm^2

As = 48481.66 mm^2

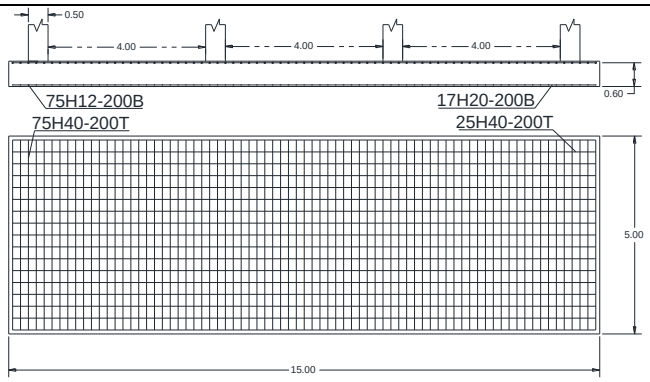
S/N	CALCULATION	OUTPUT
	Provide 75H32 @ 200 *mm* T (60318.58 mm^2) Top *Compression reinforcement required along the shorter span* $$A_s = \frac{\rho bd}{100}$$ $$= \frac{0.6 \times 5000 \times 520}{100}$$ $$= 15600\ mm^2$$ Total area = 15600 + 560.54 = 16160.54 mm^2 Provide 17H40 @ 300 *mm* T (21362.83 mm^2) Top  TOP PLAN *Fig. 4.15 Reinforcement Mesh of Raft Foundation with 0.6% Compression Reinforcement* *4.2.8 Design of Simple Raft Foundation With Additional 0.7% Compression Reinforcement* This model is exactly the same as the first model but with 0.7% compression reinforcement. The compression reinforcement was designed and added to the first model using compression reinforcement ratio based on the cross sectional area.	**75H32 @ 200 mm T** **As = 16160.54 mm^2** **17H40 @ 300 mm T**

S/N	CALCULATION	OUTPUT
	Compression reinforcement required along the longer span $Compression\ reinforcement\ ratio\ (\%), \rho = \frac{100A_s}{bd}$ Where, As = area of reinforcement b = width of the section d = depth of the section Therefore, $A_s = \frac{\rho bd}{100}$ $= \frac{0.7 \times 15000 \times 520}{100}$ = 54600 mm^2 Total area = 54600 + 1681.66 = 56281.66 mm^2 Provide 75H32 @ 200 *mm* T (60318.58 mm^2) Top *Compression reinforcement required along the shorter span* $A_s = \frac{\rho bd}{100}$ $= \frac{0.7 \times 5000 \times 520}{100}$ = 18200 mm^2 Total area = 18200 + 560.54 = 18760.54 mm^2 Provide 17H40 @ 300 *mm* T (21362.83 mm^2) Top	**As = 56281.66 mm^2** **75H32 @ 200 mm T** **As = 18760.54 mm^2** **17H40 @ 300 mm T**

S/N	CALCULATION	OUTPUT
	 TOP PLAN *Fig. 4.16 Reinforcement Mesh of Raft Foundation with 0.7% Compression Reinforcement* *4.2.9 Design of Simple Raft Foundation With Additional 0.8% Compression Reinforcement* This model is exactly the same as the first model but with 0.8% compression reinforcement. The compression reinforcement was designed and added to the first model using compression reinforcement ratio based on the cross sectional area. *Compression reinforcement required along the longer span* $Compression\ reinforcement\ ratio\ (\%), \rho = \frac{100A_s}{bd}$ Where, As = area of reinforcement b = width of the section d = depth of the section Therefore, $A_s = \frac{\rho bd}{100}$ $= \frac{0.8 \times 15000 \times 520}{100}$ = 62400 mm^2 Total area = 62400 + 1681.66 = 64081.66 mm^2	**As = 64081.66 mm^2**

S/N	CALCULATION	OUTPUT
	Provide 75H40 @ 200 *mm* T (94247.78 mm^2) Top *Compression reinforcement required along the shorter span* $$A_s = \frac{\rho bd}{100}$$ $$= \frac{0.8 \times 5000 \times 520}{100}$$ = 20800 mm^2 Total area = 20800 + 560.54 = 21360.54 mm^2 Provide 17H40 @ 300 *mm* T (21362.83 mm^2) Top  *Fig. 4.17 Reinforcement Mesh of Raft Foundation with 0.8% Compression Reinforcement* *4.2.10 Design of Simple Raft Foundation With Additional 0.9% Compression Reinforcement* This model is exactly the same as the first model but with 0.9% compression reinforcement. The compression reinforcement was designed and added to the first model using compression reinforcement ratio based on the cross sectional area.	**75H40 @ 200 mm T** **As = 21360.54 mm^2** **17H40 @ 300 mm T**

S/N	CALCULATION	OUTPUT
	Compression reinforcement required along the longer span $Compression\ reinforcement\ ratio\ (\%), \rho = \frac{100A_s}{bd}$ Where, As = area of reinforcement b = width of the section d = depth of the section Therefore, $A_s = \frac{\rho bd}{100}$ $= \frac{0.9 \times 15000 \times 520}{100}$ = 70200 mm^2 Total area = 70200 + 1681.66 = 71881.66 mm^2 Provide 75H40 @ 200 *mm* T (94247.78 mm^2) Top *Compression reinforcement required along the shorter span* $A_s = \frac{\rho bd}{100}$ $= \frac{0.9 \times 5000 \times 520}{100}$ = 23400 mm^2 Total area = 23400 + 560.54 = 23960.54 mm^2 Provide 25H40 @ 200 *mm* T (31415.93 mm^2) Top	**As = 71881.66 mm^2** **75H40 @ 200 mm T** **As = 23960.54 mm^2** **25H40 @ 200 mm T**

S/N	CALCULATION	OUTPUT
	 TOP PLAN *Fig. 4.18 Reinforcement Mesh of Raft Foundation with 0.9% Compression Reinforcement*	

4.2 Stress Patterns in the Raft Foundation

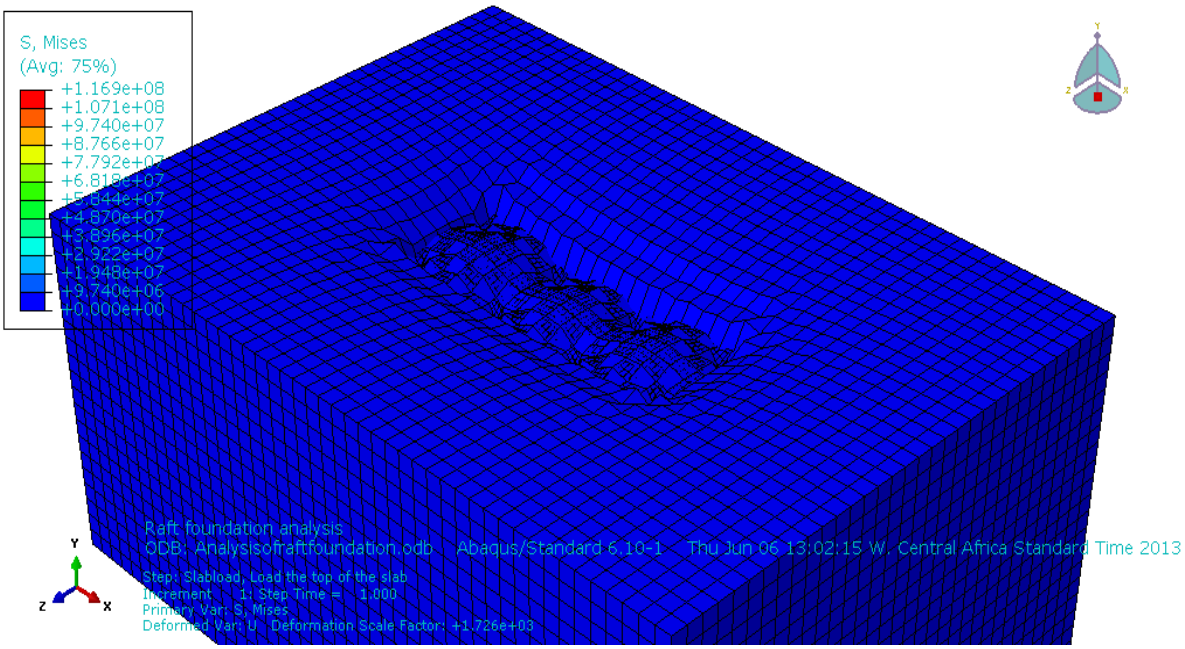


Fig. 4.19Result of Stress Analysis (0%)

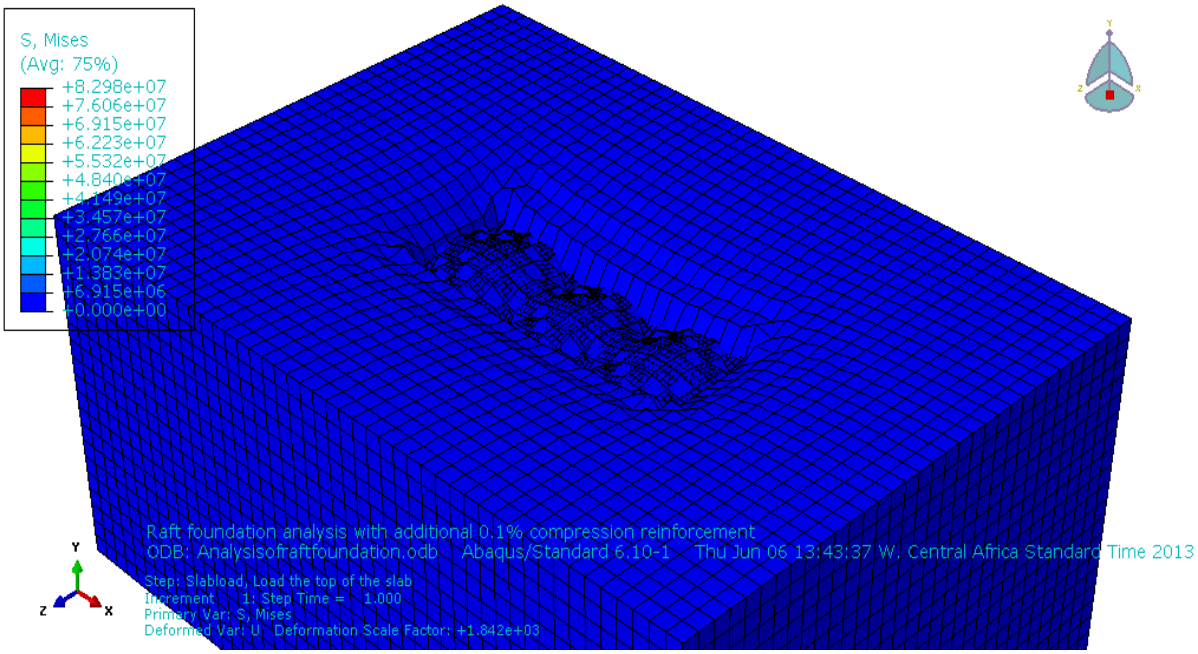


Fig. 4.20Result of Stress Analysis (0.1%)

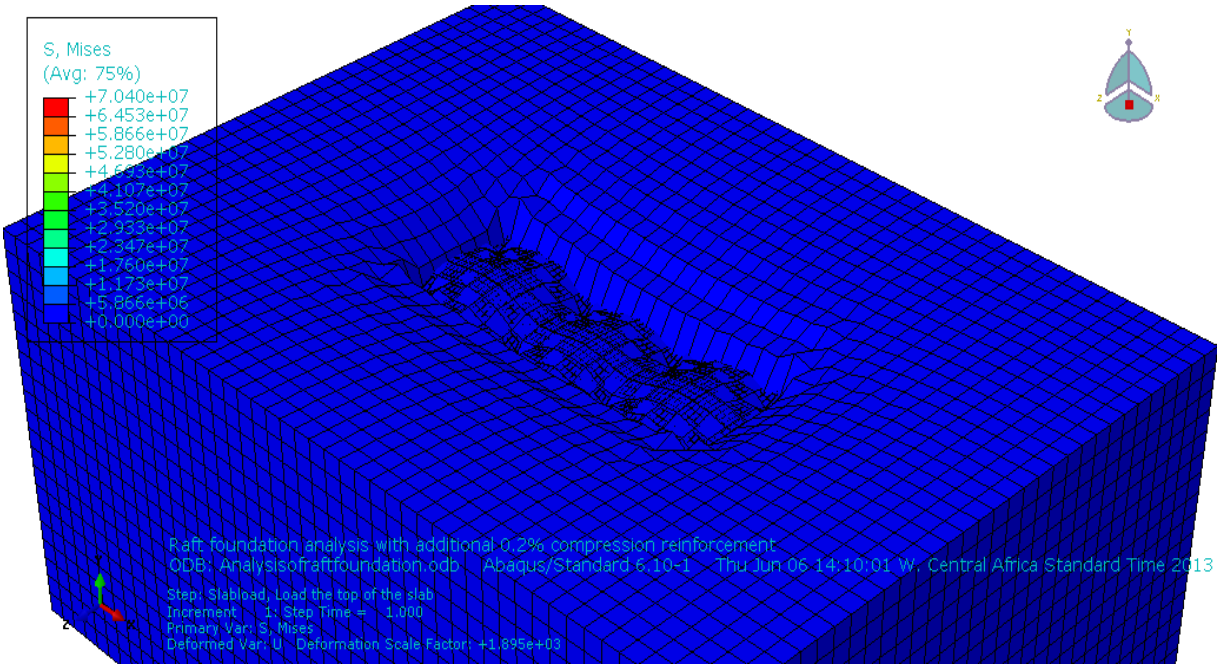


Fig. 4.21Result of Stress Analysis (0.2%)

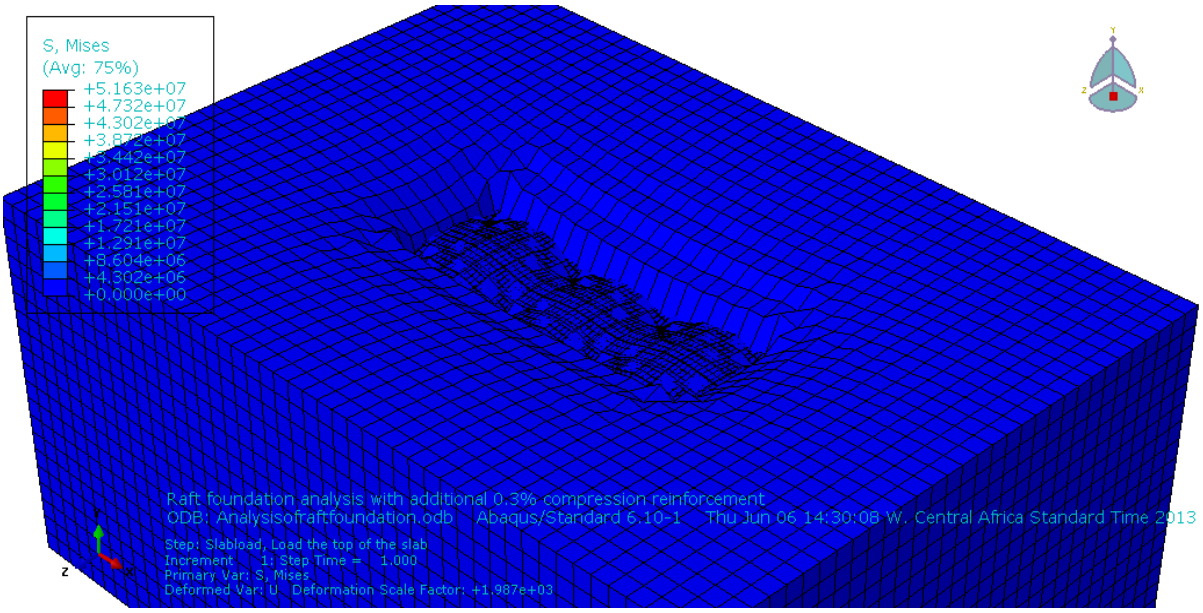


Fig. 4.22Result of Stress Analysis (0.3%)

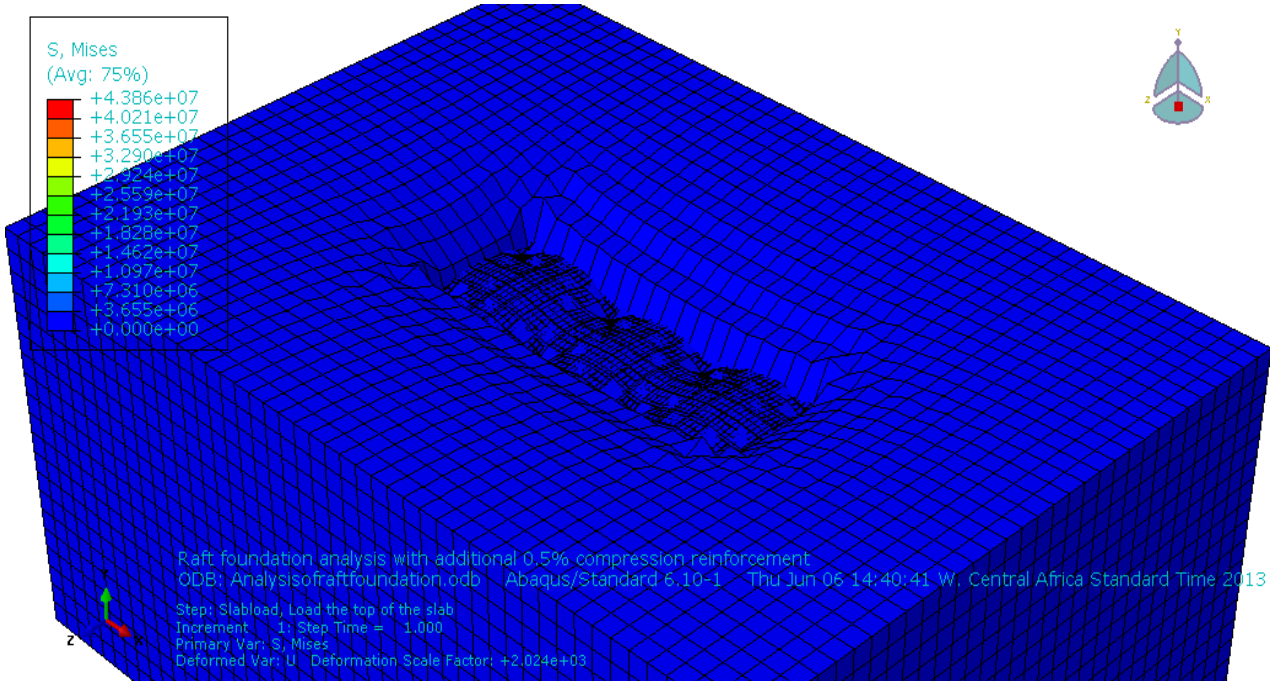


Fig. 4.23Result of Stress Analysis (0.5%)

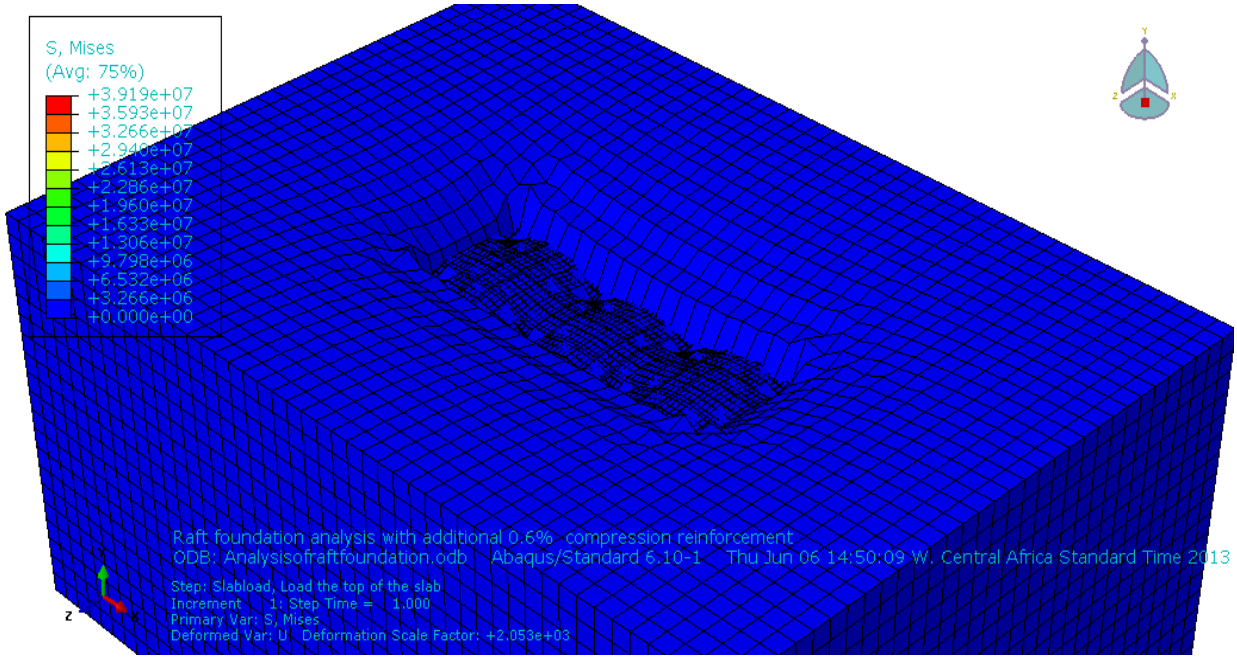


Fig. 4.24Result of Stress Analysis (0.6%)

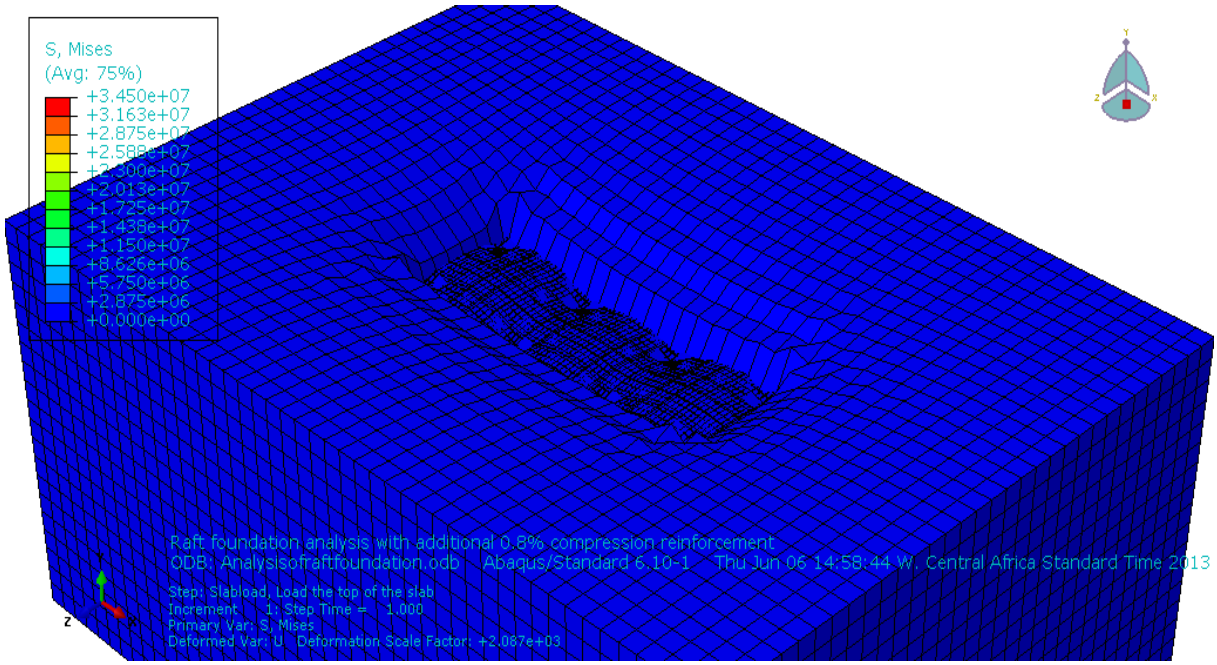


Fig. 4.25Result of Stress Analysis (0.8%)

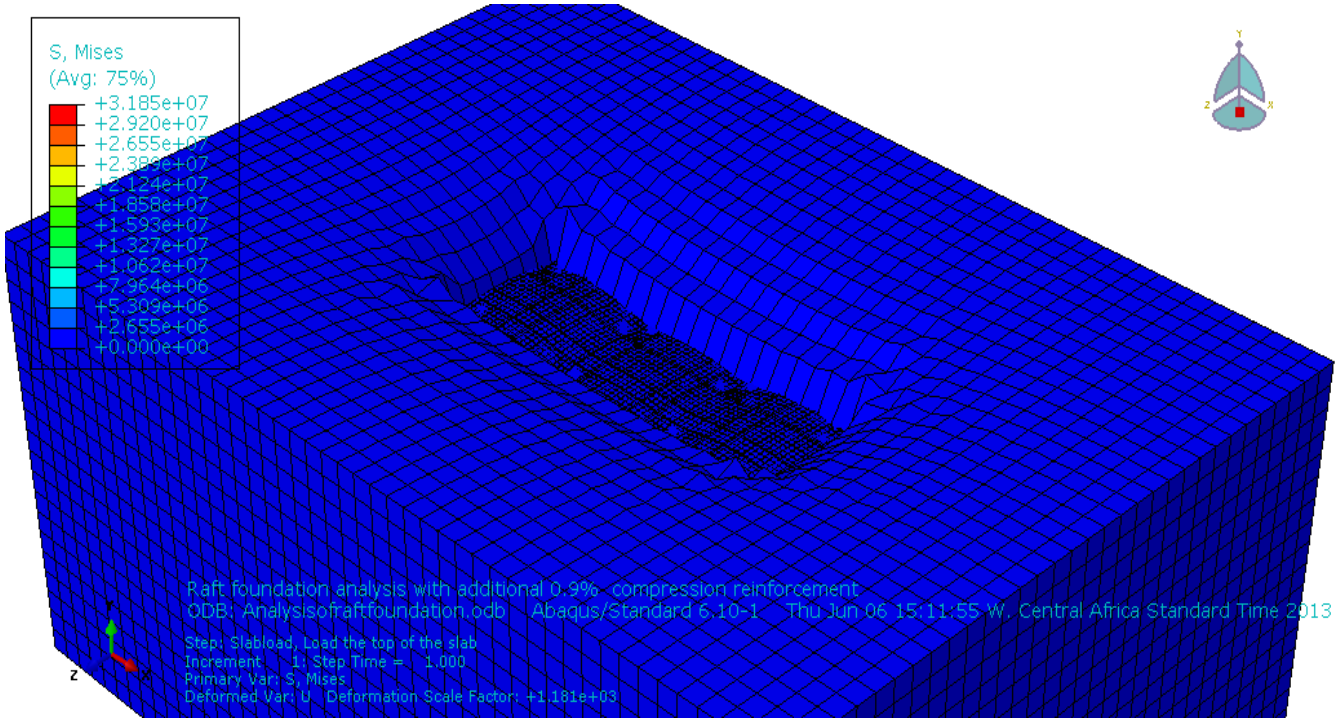


Fig. 4.26Result of Stress Analysis (0.9%)

4.3 Settlement of the Raft Foundation

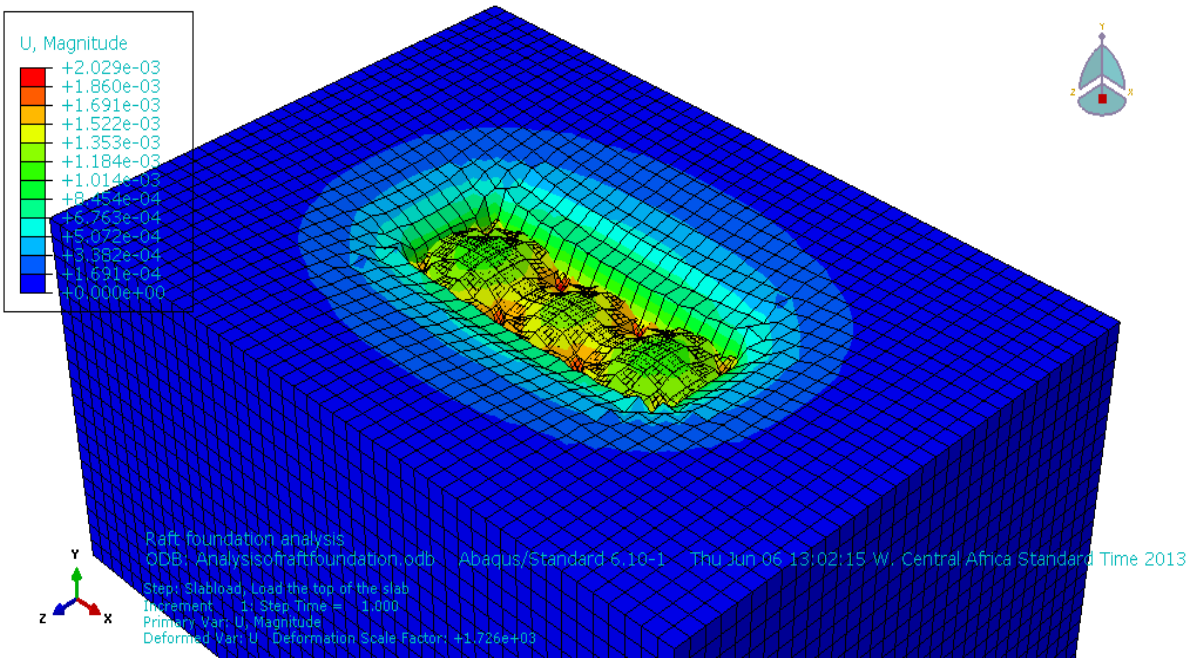


Fig. 4.27Result of Settlement Analysis (0%)

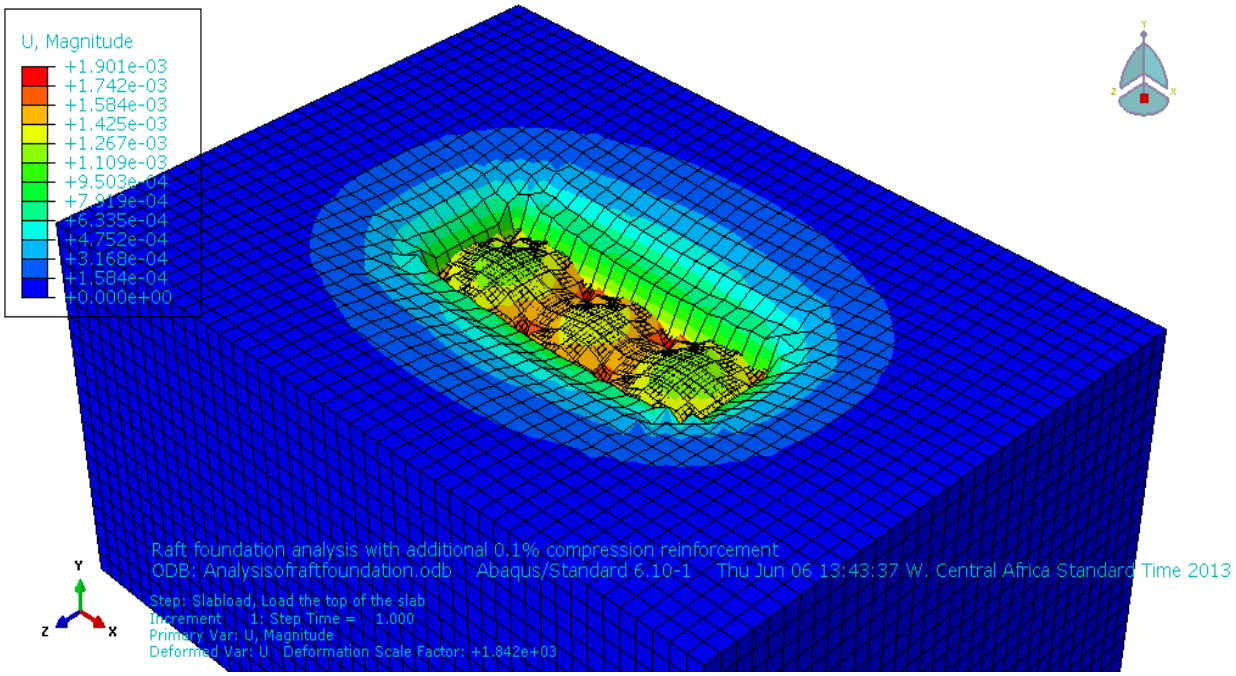


Fig. 4.28Result of Settlement Analysis (0.1%)

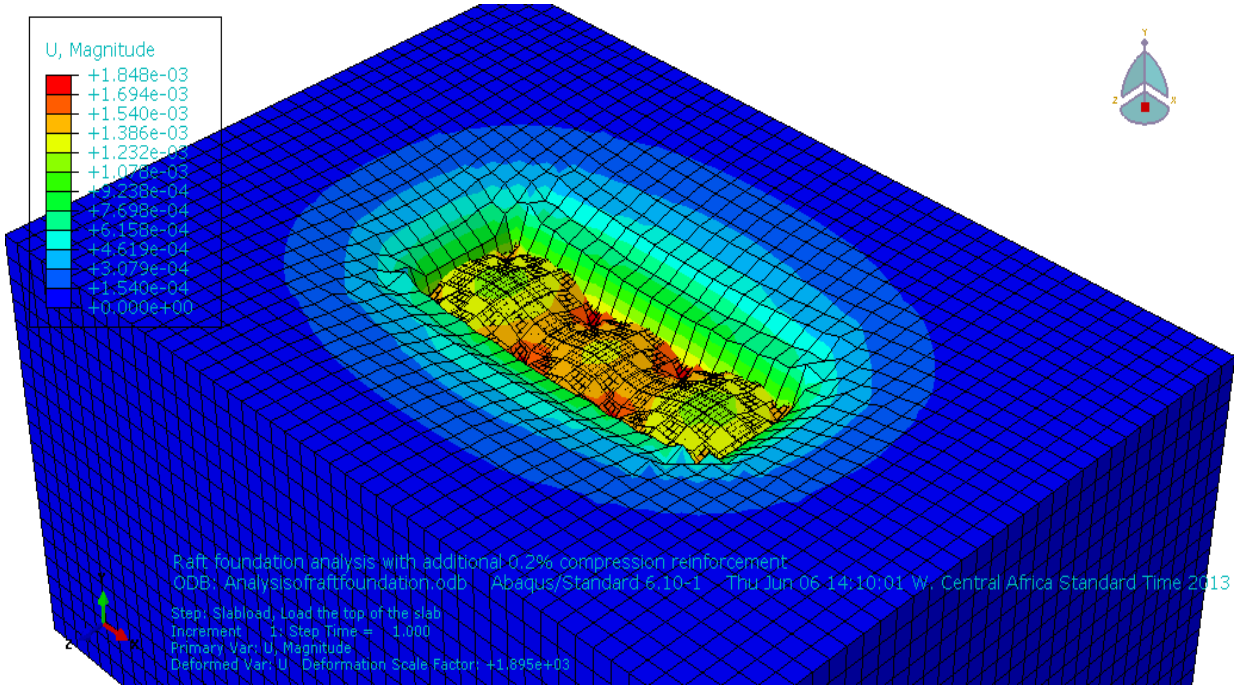


Fig. 4.29Result of Settlement Analysis (0.2%)

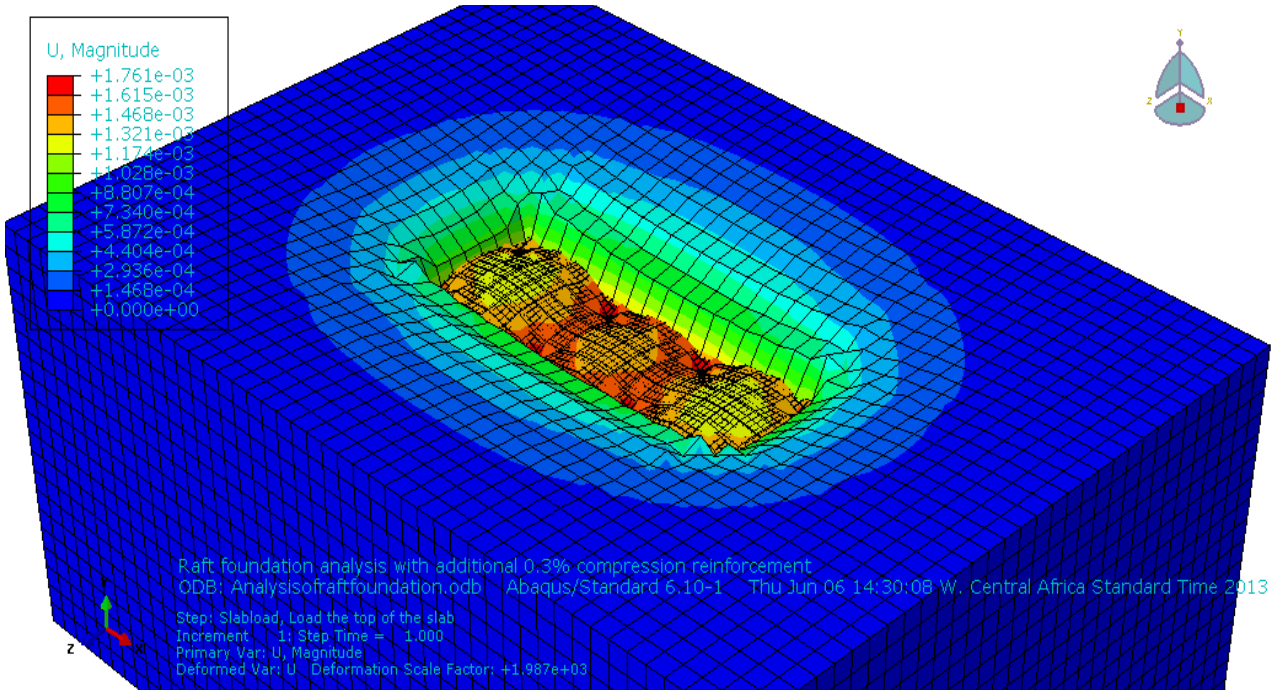


Fig. 4.30Result of Settlement Analysis (0.3%)

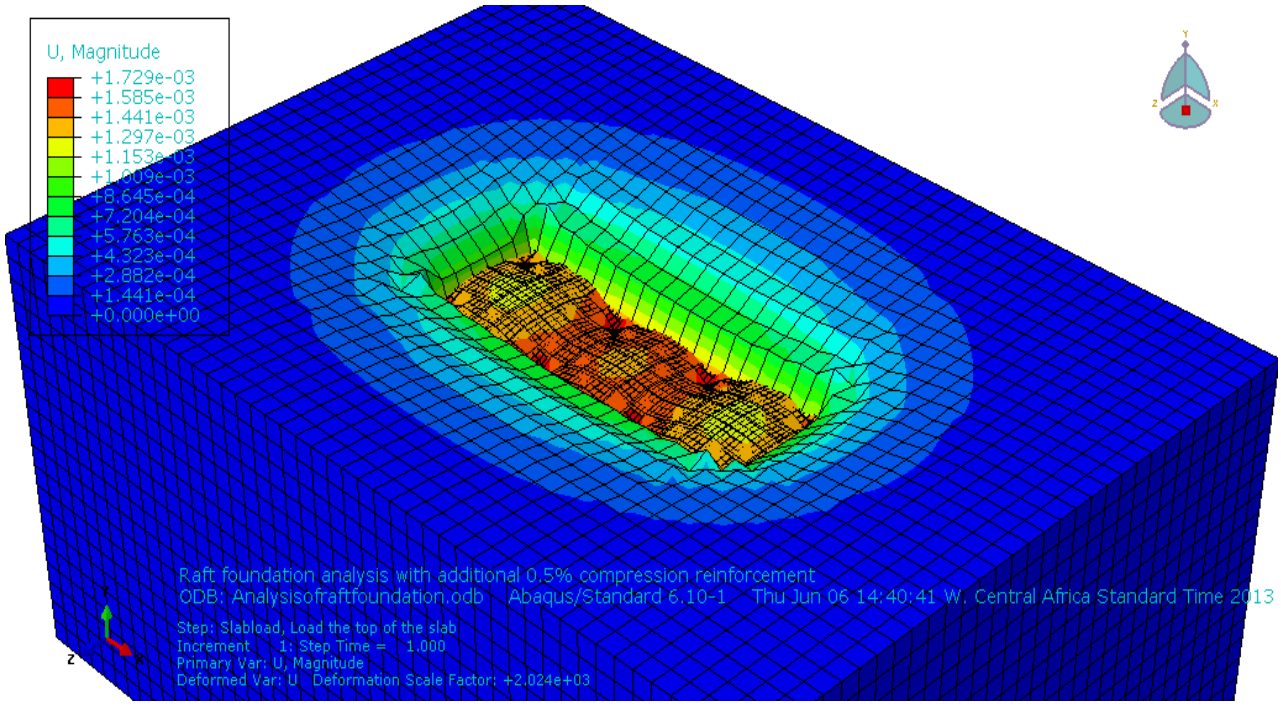


Fig. 4.31Result of Settlement Analysis (0.5%)

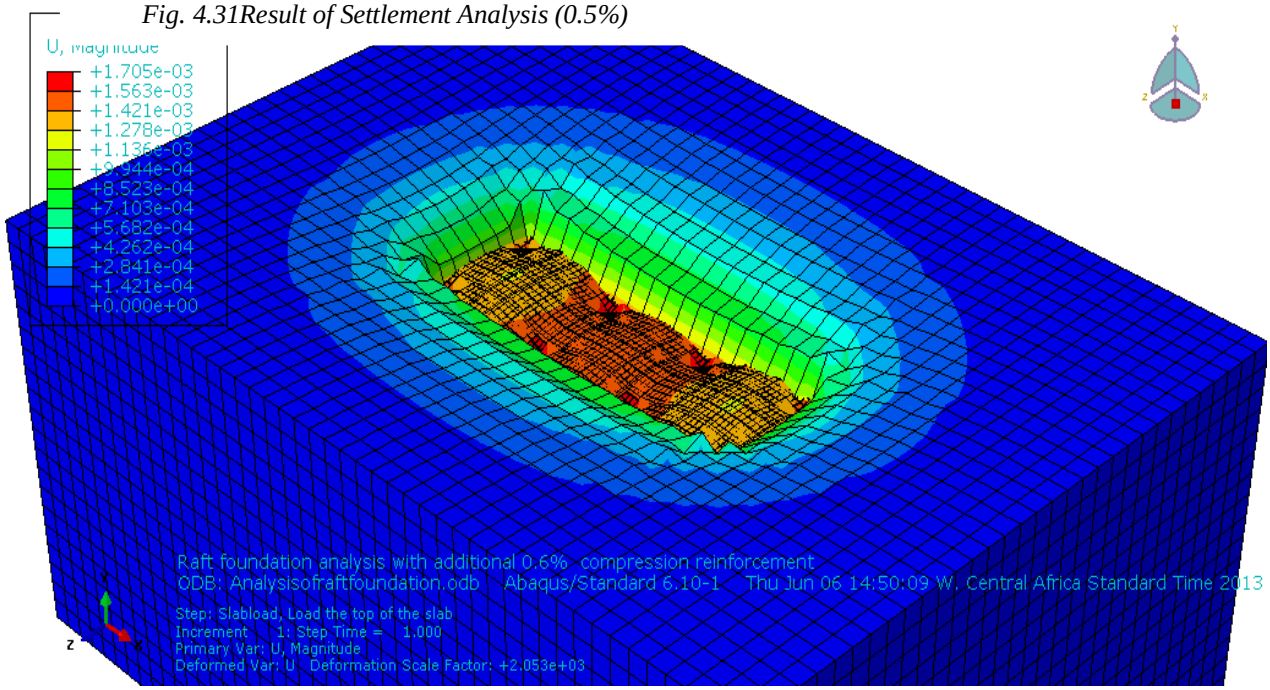


Fig. 4.32Result of Settlement Analysis (0.6%)

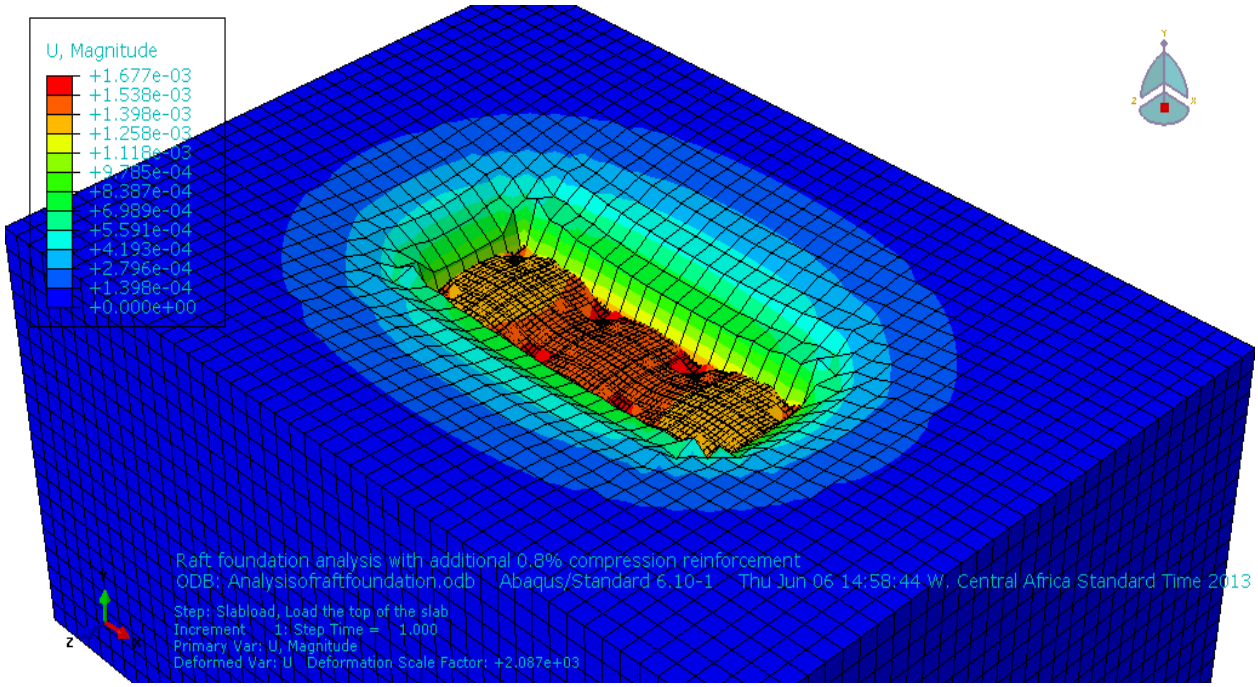


Fig. 4.33Result of Settlement Analysis (0.8%)

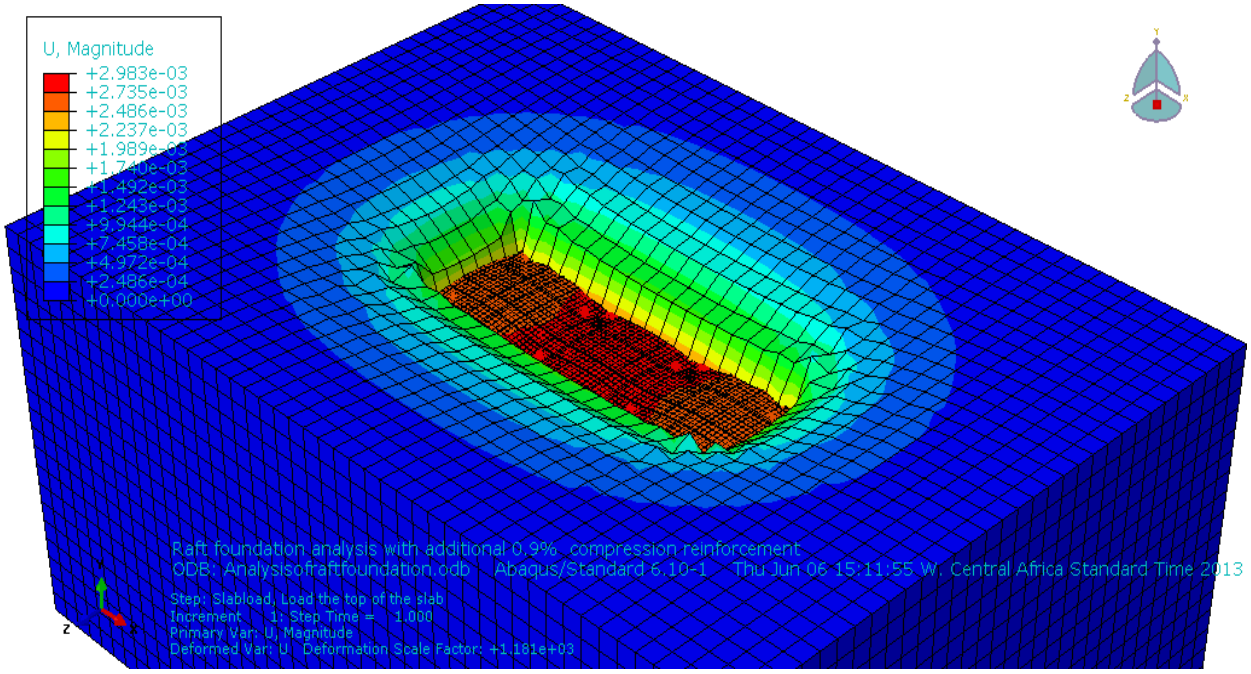


Fig. 4.34Result of Settlement Analysis (0.9%)

CHAPTER FIVE

DISCUSSION

5.1 Stress Patterns in the Raft Foundation

The Von Mises stress pattern obtained after the analysis for the raft foundation models can be seen in the deformed diagram of the models shown in Fig. 4.19 - 4.26. The Von Mises stress refers to the theory called Maxwell-Huber-Hencky-Von Mises criterion for ductile failure. The analysis shows that there are no Von Mises stresses within all the raft foundation models. The contour blue indicates zero Von Mises stresses and hence it is an indication that the foundation is stable after the deformation caused by the applied loads.

5.2 Settlement of the Raft Foundation

Bowles (1997) considered settlement estimates of a foundation as a best guess of the footing deformation after a load has been applied. He noted that most structural distress is caused by excessive settlements and not the shear failures associated with bearing capacity. Eurocode 2 (2008) specifies that where differential settlements are taken into account a partial safety factor for settlement effects should be applied. Also Eurocode 7 (2004) specifies that differential movements of foundations leading to deformation in the supported structure shall be limited to ensure that they do not lead to a limit state in the supported structure.

The immediate settlement of the raft foundation models takes place in the direction in which the load is applied. The spatial displacement of the models can be seen in Fig. 4.27 - 4.34. It can be seen that the maximum immediate settlement in the first raft foundation model occurred at the points where the loads are applied while the minimum immediate settlement occurred at the center where there is an upward heave. The result indicates that the tensile reinforcement provided is insufficient to provide adequate resistance against deformation and differential settlement.The addition of compression reinforcement increases the

resistance until uniform settlement is obtained at a value 0.9%, a percentage of the cross sectional area of the raft slab.

CHAPTER SIX SUMMARY, CONCLUSIONS AND RECOMMENDATION

6.1 Summary

In the course of this research, the following were carried out:

1. A simple raft foundation is designed using the conventional method of design according to Eurocode 2.
2. The design is subjected to finite element analysis in order to obtain the stress and settlement patterns.
3. The settlement pattern indicates there is differential settlement within the raft foundation and hence additional compression reinforcement is provided to this initial design from 0.1% to 0.9% of the cross sectional area of the raft slab.
4. The raft foundation design with the additional reinforcements is then subjected to finite element analysis until uniform settlement is obtained at 0.9%.

6.2Conclusions

This work has successfully achieved its objectives through the literature review and the studies conducted. On the basis of the study carried out, the following conclusions may be drawn:

i. The variation of the amount of reinforcement in compression and tension in a raft foundation plays a significant role and affects the moments and deformations of the foundation.
ii. Compression reinforcement is effective in providing resistance against differential settlement in a reinforced concrete raft foundation.
iii. Settlement estimate of a foundation is the best guess of the footing deformation after a load has been applied as proved from the nodal representations of elements in the finite element analysis display.

iv. The overall settlement of the raft foundation was reduced by about 20% due to the increase in the stiffness of the foundation as proved from the displacement of nodes and elements in the finite element analysis results.
v. The finite element analysis is a good method for the design and analysis of raft foundations.

6.3Recommendation

This work suggests that a suitable percentage of the concrete cross sectional area of raft slab foundations should be used as compression reinforcement in order to prevent differential settlements. The percentage should be derived using finite element analysis (FEA) by testing the conventional design of the raft foundations. This value is obtained as 0.9% of the concrete cross section when using Eurocode 2 and may be generally applied for the design of reinforced concrete raft foundations.

REFERENCES:

Abdel-Rahman H. H., (1982). "Computational Models for the Nonlinear Analysis of Reinforced Concrete Flexural Slab System"**.***PhD Thesis*, University College of Swansea.

ASCE Task Committee on Concrete and Masonary Structure (1981). "State of the Art Report on Finite Element Analysis of Reinforced Concrete"**.***ASCE*.

Babak E. H., (2011). "Raft Foundations". Available: http://www.allexperts.com. Accessed 4/9/2012.

Bowles J. E., (1997)." Foundation Analysis and Design"**.***McGraw-Hill*, Singapore.

Burd H. J. and Frydman S., (1997)."Bearing Capacity of Plane-Strain footings on Layered Soils".*Canadian Geotechnical Journal*, 34(2), pp. 241-253.

Buyukozturk O., (1977). "Nonlinear Analysis of Reinforced Concrete Structure"**.***Computers and Structures*, Vol. 7, pp. 149-156.

Chen A. C. T. and Chen W. F. (1975)."Constitutive Relations for Concrete"**.***Journal of the Engineering Mechanics Division*, ASCE, Vol. 101, No. EM4, Proc. Paper 11529, pp. 465-481.

Chen W. F. and Ting E. C., (1980). "Constitutive Models for Concrete Structures"**.***Journal of the Engineering Mechanics Division*, ASCE, vol. 106, No.EM1, Proc. Paper 15177, pp.1-19.

Cheung, Y. K., King, I. P., and Zienkiewicz, O. C., (1968). "Slab Bridges with Arbitrary Shape and Support Conditions: A General Method of Analysis Based on Finite Elements". *Institution of Civil Engineers - Proceedings*, vol. 40, pp. 9–36.

Cook R. D., Malkus D. S. and Plesha M. E. (1989). "Concepts and Applications of Finite Element Analysis".*John Wiley and Sons Inc*, Toronto, Canada.

Day R., (1999). "Forensic Geotechnical and Foundation Engineering".*McGraw-Hill Companies Inc*, USA.

Deaton J. B., (2005). "A Finite Element Approach to Reinforced Concrete Slab Design".*School of Civil and Environmental Engineering*, Georgia Institute of Technology.

EN 1992-1-1, (2008). "Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings". European Standard was approved by CEN on16 April 2004, ICS 91.010.30; 91.080.40.

EN 1997-1, (2004). "Eurocode 7: Geotechnical design - Part 1: General rules". European Standard was approved by CEN on 23 April 2004, ICS 91.120.20.

Ergatoudis J., Irons B. M. and Zienkiewicz O. C. (1968)."Three Dimensional Analysis of Arch Dams and Their Foundations".Research C/R/74/67, University of Wales, Swansea, Wales.

Filippas O. B., Kulhawy F. H. and Grigoriu M. D. (1988). "Reliability-Based Foundation Design for Transmission Line Structures: Uncertainties in Soil Property Measurement". *Research Report 1493-3 EL-5507*, Vol. 3, Cornell University/Electric Power Research Institute, Palo Alto, California, USA.

French S. E., (1999). "Design of Shallow Foundations". American Society for Civil Engineers, Press, pp. 374.

GEO, Civil Engineering and Development Department (2006)."Foundation Design and Construction". Hong Kong, Special Administrative Region: GEO.

Griffiths D. V., (1982)."Computation of Bearing Capacity on Layered Soils".*4th International Conference on Numerical Methods in Geomechanics*, Edmonton, Canada, pp. 163-170.

Gupta S. C., (2007). "Raft Foundation (Design and Analysis with a Practical Approach)". : New Age International Publishers.

Harr M. E., (1966). "Foundations of Theoretical Soil Mechanics".*McGraw Hill*, New York, USA.

Hemsley J. A. (2000). "Design Applications of Raft Foundations".Review.

Holtz R. D., (1991). "Stress Distribution and Settlement of Shallow Foundations".*Foundation Engineering Handbook*, H. Y. Fang, ed., Van Nostrand Reinhold, New York, USA, pp. 166-223.

Hrennikoff A. R., (1941). "Solution of the Problems in Elasticity by the Framework Method".Journal of Applied Mechanics, 8(4), pp. A169-A175.

Ijimdiya T. S., (2010a). "Geotechnical Characteristics of Oil Contaminated Soils". *International Journal of Engineering (IJE)*, vol. 4, No. 4, pp.539-547.

Ijimdiya T. S., (2010b). "The Effect of Compact Effort on the Compaction Characteristics of Oil Contaminated Lateritic Soils".*International Journal of Engineering (IJE)*, vol. 4, No. 4, pp.594-554.

Jaksa M. B., (1995). "The Influence of Spatial Variability on The Geotechnical Design Properties of a Stiff Overconsolidated Clay". PhD, The University of Adelaide, Adelaide.

Jason Scott Goldsworthy, (2006). "Quantifying the Risk of Geotechnical Site Investigations".

Kwak H-G and Filip C. F., (1990). "Finite Element Analysis of Reinforced Concrete Structures Under Monotonic Loads". Report No. UCB/SEMM-90/14, Structural Engineering Mechanics and Materials, Department of Civil Engineering, University of California, Berkeley.

Mahdi H. A., (2008). "Design of Raft Foundations".For Students of Foundation's Project – Faculty of Engineering, Sixth October University.

Manjunath M., (2009)."Finite Element Project - ABAQUS Software (Tutorial)".

Mayne P. W. and Poulos H. G., (1999)."Approximate Displacement Influence Factors for Elastic Shallow Foundations".Ournal of Geotechnical and Geoenvironmental Engineering. 125(6), pp. 453-460.

Meyerhof G. G., (1963)."Some Recent Research on the Bearing Capacity of Foundations".*Canadian Geotechnical Journal*, 1(1), pp. 16-26.

Mikkola M. J. and Schnobrich W. C. (1970)."Material Behavior Characteristics for Reinforced Concrete Shell Stressed Beyond the Elastic Range".Civil Engineering Studies, SRS No. 367, University of Illinois, Urbana, Illinois.

Niandou, Halidou/Breysse, Denys, Computers and Geotechnics, (2006)."Reliability Analysis of a Piled Raft Accounting for Soil Horizontal Variability".34 (2), p71-80.

Paul B., (2010). "Raft Foundation". Available: http://www.spancon.co.za/. Last accessed 23/9/2012.

Perloff W. H., (1975). "Pressure Distribution and Settlement".*Foundation Engineering Handbook*, H. Winterkorn and H. Fang, eds., Van Nostrand Reinhold, New York, USA, pp. 148-196.

Phoon K. K., Kulhawy F. H. (1999). "Evaluation of Geotechnical Property Variability".*Canadian Geotechnical Journal*, 36(4), pp. 625-639.

Poulos H. G., (2000). "Foundation Settlement Analysis – Practice Versus Research". The Eighth Spencer J. Buchanan Lecture, Texas, pp.34.

Prandtl L., (1921). "Uber die Eindringungsfestigkeit (Harte) PlasticherBaustoffe und die Festigkeit von Schneiden".Zeitschrift fur AngewandteMathematik und Mechanik, Basel, Switzerland, vol. 1, No. 1, pp.15-20.

Riedel W. (1927)."BeitragezurLosung des ebenen Problems eineselastichenKorpersmittels der AiryschenSpannungsfunktion".Zeitschrift fur AngewandteMathematik und Mechanik, 7(3), pp. 169-188.

Schnobrich W. C. (1977). "Behavior of Reinforced Concrete Structures Predicted by the Finite Element Method". Computers and Structures, Vol. 7, pp. 365-376.

Schnobrich W. C. and Hu Hsuan-Teh, (1985)."Use of Finite Element Techniques to Predict Nonlinear Behaviour of Reinforced Concrete Structures".Pergamon Press Ltd, *Proc. Int. Conf. Finite Element in Computational Mechanics*, Department of Civil Engineering, University of Illinois at Urbana – Chanpaign, USA.

Simulia D. S., (2012). "Abaqus Unified FEA". 3DS DassaultSystemes. Available: http://www.3ds.com/. Last accessed 4/10/2012.

Singh G. and Singh J., (2006)."Theory and Design of RCC Structures (including limit state design)".5th ed., A. K. Jain for Standard Publishers, NaiSarak, Delhi.

Sinha B. P., Gerstle K. H. and Tulin L. G. (1964). "Stress-Strain Relations for Concrete Under Cyclic Loading". *Journal of the American Concrete Institute*, Vol. 61, No. 2, pp. 195-211.

Small J. C. (2001). "Shallow Foundations".Site Investigation in Construction, Thomas Telford Services Ltd., London, England.

Spry M. J., Kulhawy F. H. and Grigorou M. D. (1988). "Reliability-Based Foundation Design for Transmission Line Structures: Geotechnical Site Charectirization Strategy". Research Report 1493-1 EL-5507, Vol. 1, Cornell University/Electric Power research Institute, Palo Alto, California, USA.

Stephen E. and Christopher G. A., (2010)."Barry's Introduction to Construction of Buildings". 2nd Edition, Published by John Wiley and Sons, pp.61-64.

Terzaghi K., (1943). "Theoretical Soil Mechanics".John Wiley and Sons Inc., New York, USA.

Terzaghi K. and Peck R B, (1967). “Soil Mechanics in Engineering Practice”.John Wiley, New York, USA.

UWE, (2012). “Foundations” .Faculty of Environment and Technology, University of the West England. Available: http://www.environment.uwe.ac.uk/. Last accessed 19/9/2012.

Vanmarcke E. H., (1977a). “Probabilistic Modeling of Soil Profiles”.*Journal of the Geotechnical Engineering Division*, 103(GT11), pp.1227-1246.

Venkatesh K., Samadhiya N. K. and Pandey A. D., (2009).“Response of Raft Foundation on Varying Stratum”.1 (6), *International Journal of Recent Trends in Engineering, Department of Civil Engineering and Earthquake Engineering*, IIT Roorkee, India.

Wieghardt K. (1906). “UbereinenGrenzuberganggerElastizitatslehreind seine Andwedndung auf sieStatikhochgradigstatischunbestimmterFachwerke”. Verhandlungen des Vereins z. Beforderung des Gewerbefleisses.Abhandlungen, 85, pp. 139-176.