ISTANBUL KULTUR UNIVERSITY

INSTITUTE OF SCIENCE

“The Effect Of The Rigidity On The Analysis Of Rafts”

MSc THESIS

Mehmet Onur TANAYDIN

509070001

Programme : Geotechnical Engineering

Science Programme : Civil Engineering

Thesis Supervisor : Prof. Dr. Akın ÖNALP

September 2008

PREFACE

Firstly , I would like to thank Prof. Dr. Akın Önalp for his help during the prepartion of my project and his invaluable contributions to my development in my career.

I also would like to thank Wijtze Pieter Kikstra who always supported me during the preparation process of my project.

Furthermore I am indepted to.Murat Türk and.Edip Seçkin for their suggestions and guidance.

Finally I would like to acknowledge my family for the support they have given at every stage of my education.

CONTENTS

LIST OF SYMBOLS

A_{mat}	Area of mat
B	Width of foundation
L	Length of foundation
Δ	Settlement of foundation
δ	Differential settlement
ω	Angle of base tilt
K_r	Rigidity factor mat
k	Coefficient of subgrade reaction
c	Unit soil cohesion
ø	Angle of internal friction of soil
σ	3D Element streeses
E_s	Elastic soil modulus
E_c	Elastic concrete modulus
ν	Poisson's ratio for soil or concrete
ψ	Angle of shear zone failure
M_{xx}	
M_{yy}	Bending momonets on mat
M_{xy}	
Q_{xy}	Shear forces on mat
Qyz	
Ko	Coefficient of eart pressure at rest
γ_{soil}	Unit soil weight
γ_{con}	Density concrete grout
OCR	Overconsolidation ratio

NL	Normally consolidated clay
OC	Over consolidated clay

LIST OF TABLES

LIST OF FIGURES

PART 1

INTRODUCTION

The use of the finite element method for geotechnical engineering began in 1966, when Clough and Woodward used it to determine stresses and movements in embankments, and Reyes and Deere described its application to analysis of underground openings in rock. Many research studies and practical applications have taken place in the intervening 40 years. During this period, considerable advances have been made in theory and practice, and the cost of computers has diminished to a small fraction of the cost 40 years ago. (US Army Corps of Engineers,1995)

The finite element method has been applied to a wide variety of geotechnical engineering problems where stresses,movements, pore pressures, and groundwater flow were of interest.

It is clear that that the finite element method can be used to calculate stresses, movements, and groundwater flow in virtually any condition that arises in geotechnical engineering practice.

Accordingly, the FEM accounts for complex geometries, a variety of loading conditions, nonlinear material behavior, nonhomogeneous material distribution, and soil-structure interaction effects that are not accounted for in the simpler procedures in foundation engineering.

In this study a three dimensional finite element method instead of traditional subgrade-coefficient has been performed. Superstructure-foundation-soil interaction which is not usually taken into account in current civil engineering practice is considered in designing mat foundations

The following are the features of this FEM study;

- A four storey reinforced concrete structure with columns and shear wall,

- Earthquake conditions,
- Heavy loads and moments, (4600 kN-m earthquake moments a shear wall)
- A mat with diffrent thicknesses,
- Normally- and overconsolidated clay behaviour,
- Undrained conditions,
- Non linear analysis.

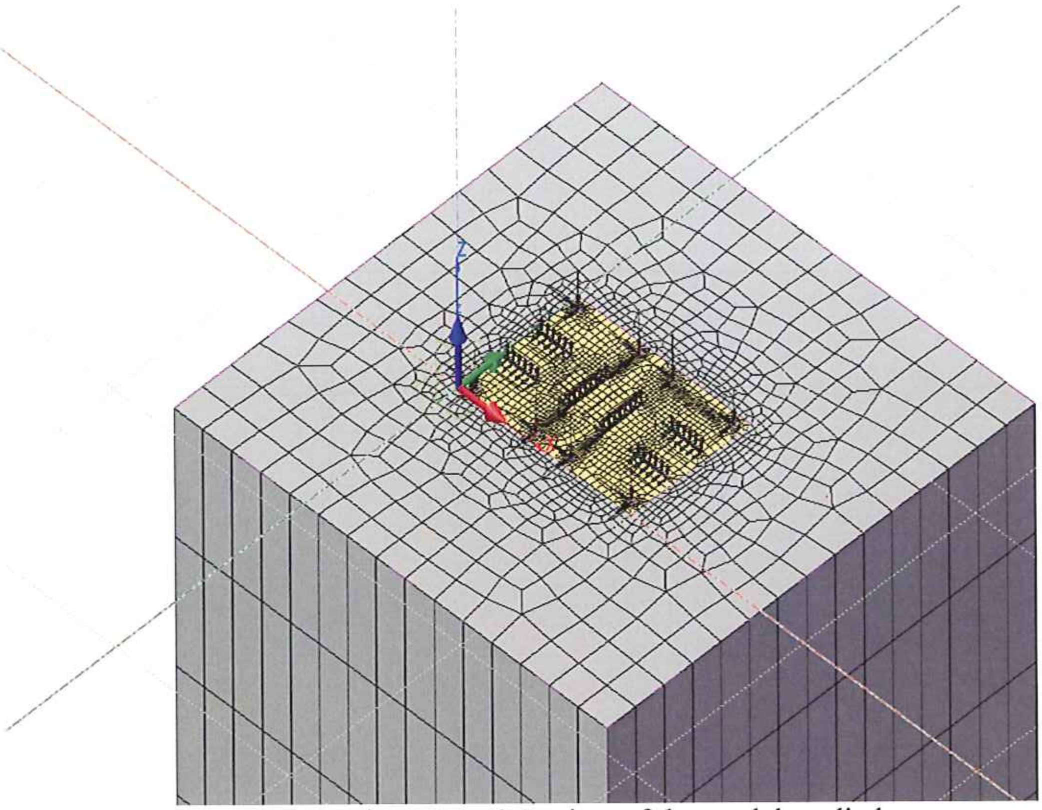


Figure1.1 3-D view of the model studied

The model of the building is exhibited in Figure 1.1. The compression of the soil, bending moments and shear forces in the raft,3D stresses on foundation elements and 3D stresses set up in weak and strong subsoil representing normally consolidated and overconsolidated conditions are considered.

PART 2

MAT FOUNDATIONS

2.1 DEFINITION

Mat(raft) foundation or *radier* as it is called in the Turkish engineering community by its French name, is a combined shallow foundation type whose reinforced flat slab interacts with the the soil.

A large percentage of structures in Turkey are nowadays founded on mats with the purpose of securing a close interaction with the subsoil. It is believed that the performance of the building in seismic conditions will be considerably improved.

Mat foundations are typically used when the building loads are excessive and/or the soil is so weak that individual footings would cover more than half the building area.

A mat is a flat concrete slab, heavily reinforced with steel, which carries the downward loads, moments and shear forces of the individual columns or walls. The resulting mat load per unit area that is transmitted to the underlying soil is small in magnitude and is distributed over the entire area.

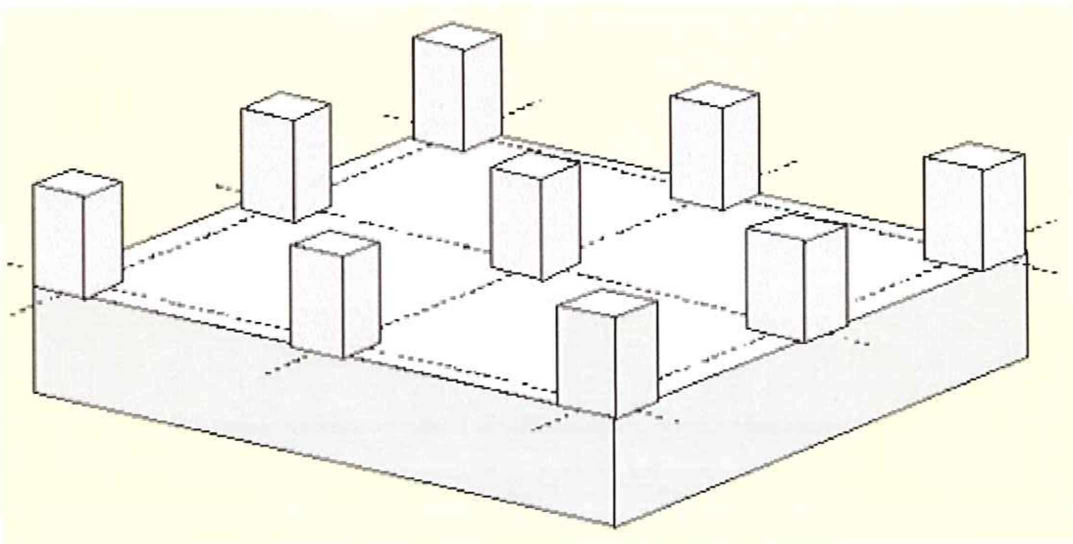


Figure 1.2 Mat Foundation

2.2 COMMON TYPES OF MATS

2.2.1 Flate Plate

The flat plate mat is preferred by reason of construction edge and furnishing a full flat area on basement.Especially the accessories are disposed in least resistance. Although this is thicker than other types of mat, it is the most economical type of all. A flate plate type of mat was used for four storey reinforced concrete structure in my study.

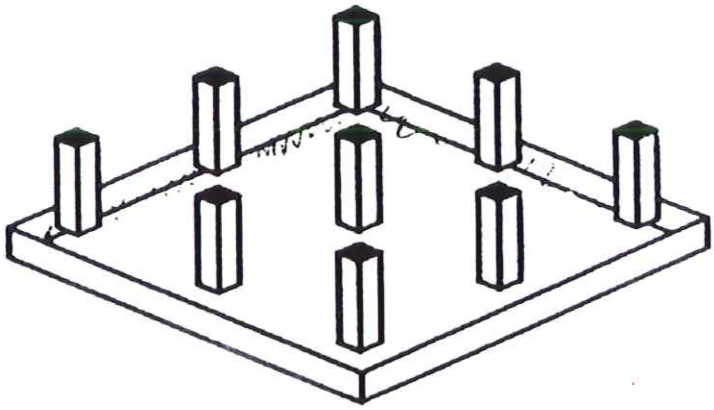


Figure 2.2 Flate Plate

2.2.2 Flate Plate Thickned Under Columns

The shear forces are caused here because the column loads increase in excess.Only it can be selected the way of thickening areas under column because that effect causes which the thicknesses didn't come up for preventing to punch on flate plate.

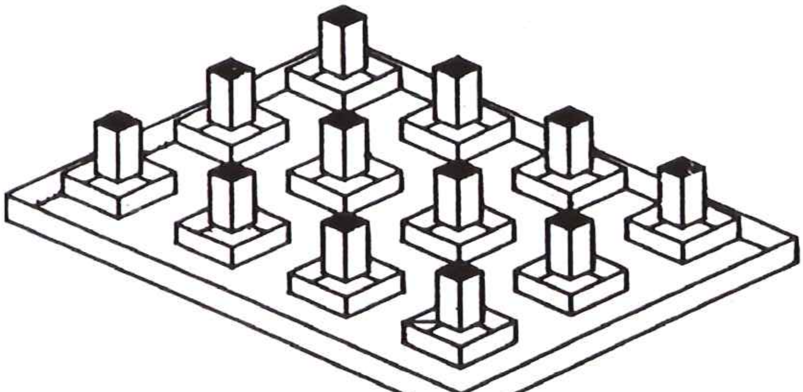


Figure 2.3 Flate Plate Thickned Under Columns

2.2.3 One Way or Two Way Beam or Slab

More economic solution could be procured by flat and cross beam plate. Symmetric beam installation between plates provides reducing thickness of plate.

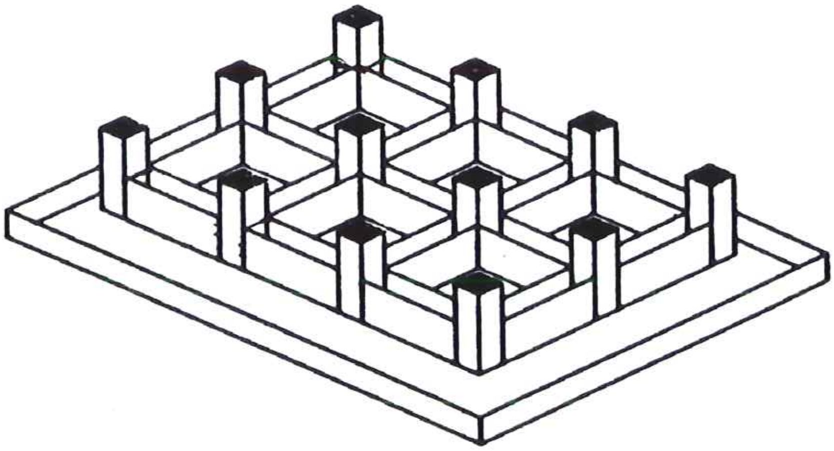


Figure 2.4 One Way or Two Way Beam or Slab

2.2.4 Basement Wall as A Rigid Frame

Above mentioned mat foundation becomes not counterbalance transferred weight by increasing highness and heaviness of building, and ground recruitment or without considering deep foundation alternatives way out of foundation " flotation " might be emerged. The purpose of flotation term is to add weight of excavating enough lower level of ground to carrying capacity.

After foundation excavation cell form of light reinforced concrete whether could be removed or made on surface then dig into soft ground. This kind of foundation can not be used as a basement because of its cells.

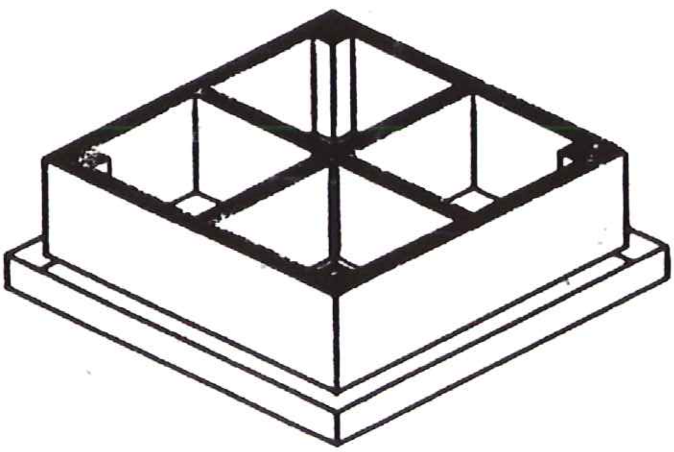


Figure 2.5 Basement Wall

2.3 DESIGN PARAMETERS

1- Determine the bearing capacity of the foundation under the design loading
2- Determine the settlement and the differential settlemement of the raft

3-Bending moments and shears for the structural design of the reinforced concrete slab.

2.4 DESIGN METHODS FOR MATS

Several sections and annexes of EN 1997-2 give additional information regarding semi-empirical calculations models for bearing resistance and settlement evaluation using soil parameters.

The Ultimate Limit States (ULS) design check can be performed using analytical calculations models for bearing or sliding resistance, or using semi-emprical calculation model where the bearing resistance is assessed directly as a derived value from in situ test results.

Serviceability limit State(SLS) design check must be performed by using the settlement calculations either by analytical methods or semi-empirical models.

2.4.1 Direct Method

This approach involves two seperate checks,

- Firstly using a calculation model as close as possible to the ULS failure mechanism
- Secondly using a settlement calculations to satisfy the SLS

2.4.2 Indirect Method

Simplified SLS method use comparable experience and the results of field or laboratory measurements or obdervations, chosen in relation to SLS loads, to satisfy the requirement of all relevant limit states.

The use of the simplified method for SLS checking is subject to the following conditions:

- Well-established and documented successful experience must exist.
- There is no explicit settlement limit specified for checking SLSs and ULSs in the supported structure due to foundation movements.
- Exceptional loading conditions do not prevail, such as for highly inclined or eccentric loads, highly variable or cyclic loads or climatic loads (such as snow and wind). Where such exceptional loading conditions prevail, extreme care is required, and the authors do not recommend the use of semi-empirical methods for lightweight structures.
- The method is not applicable for soft clays and for highly organic soils, for which settlement calculations are always required.

The indirect method consists of a very simple procedure that corresponds to traditional practice, with no calculation of settlements being required. In many design situations, the strength parameters of the ground are known with much greater confidence than the ground deformation parameters, and consequently the simplified method may be more appropriate than settlement calculations.

2.4.3 Prescriptive Method

A sample method for deriving the presumed bearing resistance for spread foundations on rock is given in Annex G.

Annex G.(method for deriving the presumed bearing resistance for spread foundations on rock)(EN 1997-1)

2.5 REASONS FOR PREFERRING MAT FOUNDATION IN DWELLINGS

- Stellement are controlled, thus faulty or issufficient soil investigations do not present serious problems
- It is easier to excavate the whole "footprint" of the house
- It has been found that mats behave more favourably in earthquakes
- It is easier to waterproof and insulate the building at the basement

- Mass concrete is cheaper than previously

2.6 FACTORS AFFECTING THE SELECTION OF MAT

- Soil types and ground water table conditions.
- Structural requirements
- Construction requirements.
- Site condition and environmental factor.
- Economy.

2.7 ADVANTAGES AND DISADVANTAGES OF MAT FOUNDATION

Safety : With have excellent load and moment carrying capability of mat foundations it is the safest type of shollow foundations

Environment : It can be extremely difficult to pour a mat foundation in cold weather due to the excessive amount of concretes used.

Durability : Mat foundations are very durable and can withstand most environmental and loading conditions.

Loading : Mat foundations are very strong and have excellent load carrying capability.

Soil Bearing : Mat foundations have good soil bearing capabilities.

Cost : Type and the engineering properties of the soil is important to make a choice of foundation type. At soft soils it may be more feasible to make mat foundation , at the other hand at hard soils making a mat foundation will may have more costs.

Expandability : Expandability can be easily accomplished

Design Life : The design life of a mat is excellent

Framework: It is easy to build this type of foundations than the others.

PART 3

SUPERSTRUCTURE – MAT FOUNDATION – SOIL INTERACTION

3.1 INTRODUCTION

The aim of structure foundation interaction researches are the complex equations between the forces and deformation among the foundation soil and superstructure and understanding of this equations statical and dynamical.

Structure foundation is a system which transforms the formation of loads into the form which the soil can carry this load safely. With this qualification foundations effects from soil and structure.So the design of foundations is problem of structure soil interaction.

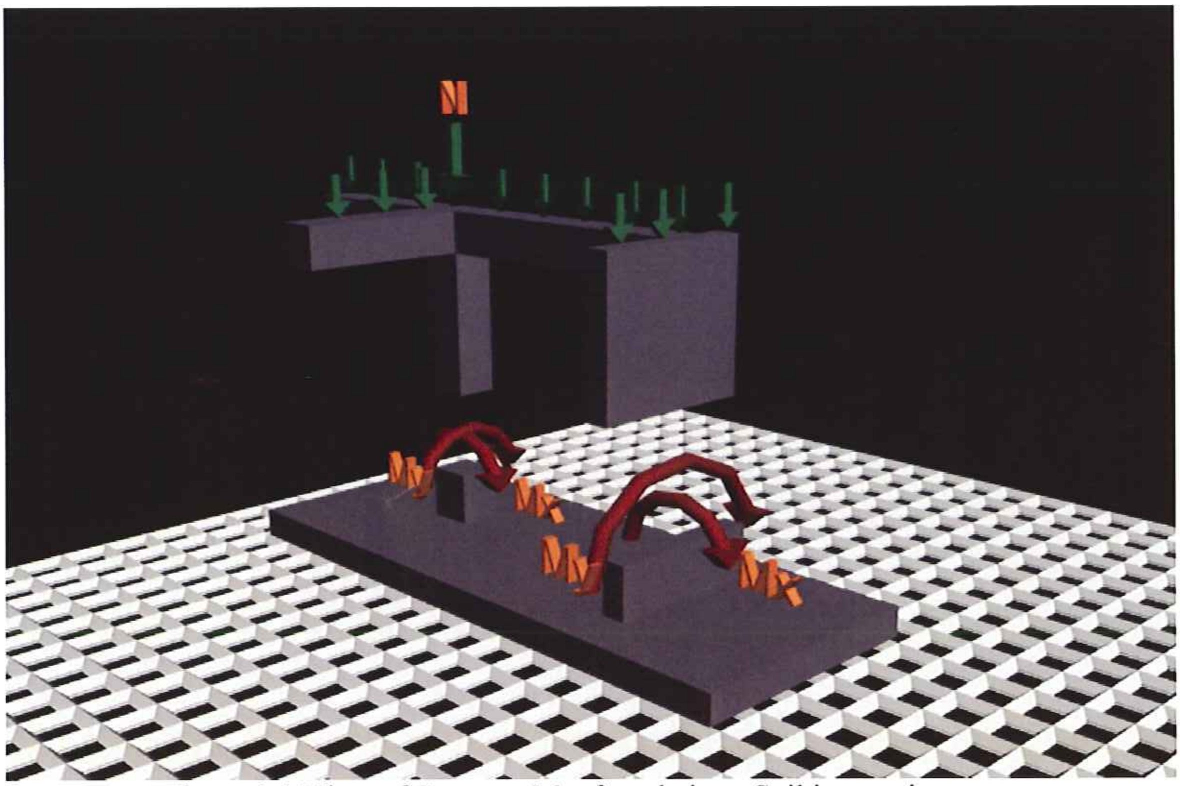


Figure 3.1 View of Stucture-Mat foundation –Soil interaction

The rigity of superstructure and its foundation , compressibility of soils are the most important factors which effect the systems whole behaviour and soil pressures under the foundations. Especially under vertical loading, structure creates non linear streeses on soil.

Settlement values of foundation and superstructure constrained deformations are connected each other. Beside soil pressures it must be known different settlements values to researched foundation and structure staticaly.because only diffrent settlemnts causes extra constrains on the structure.

At the time of eartquake the structure and its foundation, not only causus the compesses the soil which can be deformed also they exposed to deformations each.

So designing of mat foundations considering Superstructure-Mat foundation – Soil interaction system, depending Serviceability Limit State (SLS) by using the settlement calculations come forward for foundations engineers.

3.2 EFFECTS AND TYPES OF SETTLEMENTS

Significant aspects of settlement from static and dynamic loads are total and differential settlement. Total settlement is the magnitude of downward movement. Differential settlement is the difference in vertical movement between various locations of the structure and distorts the structure. Limitations to total and differential settlement depend on the function and type of structure.

Type of Structure	Settlement (cm)
Plain brick walls Length/Height ≥ 2.5	7.5
Length/Height ≤ 1.5	10
Framed structure	10
Reinforced brick walls and brick walls with reinforced concrete	15
Solid reinforced concrete foundations supporting smokestacks, silos, towers, etc	30

Table 3.1 Maximum Allowable Average Settlement of Some Structures

(US Army Corps of Engineers,1992)

It is recommended that total settlements be computed or estimated at a sufficient number of points or typical footings to establish the likely overall settlement pattern for the structure. From this pattern, it is possible to determine the overall settlement profile and the greatest differential settlement between adjacent foundation units, columns etc. Determination of the settlement that will cause significant architectural or structural damage is an extremely complex indeterminant analytical problem. Factors affecting such an analysis include the variability in the soil properties, uncertainty of the structural materials and rigidity, construction sequence, time rate and uniformity of settlement, contact pressures, stiffness and rigidity of the footing connections, and the nature of the actual loads transmitted to the foundation units. Consequently, the analysis of tolerable settlements and development of criteria for tolerable settlements have been established almost entirely empirically on the basis of observations of settlement and damage in actual buildings.

If an attempt is made to model analytically the structure and calculate the effect of differential settlements, one obtains ridiculously low allowable differential settlements because of the large bending moments that will be calculated in the frame. Some yielding in the structure actually occurs, but how much is unknown; therefore, empirical evidence from the performance of actual buildings is used to establish tolerable settlement criteria. It is important to be able to predict reasonably well the actual settlements of a structure so that proper preparation may be made for tolerating those settlements either in the foundation, structure, or perhaps in some sort of soil improvement or alteration of the structure's geometry, configuration, or in some rare cases, even its stiffness. Types of settlement include total, tilting, and distortion, or differential movement (Hsai-Yang Fang, 1990).

3.2.1. Definitions

Figure 3.2 indicates the definitions of settlement terminology

The deflection ratio Δ/L is a measure of the maximum differential movement Δ in the span length L.

Angular distortion $\beta = \delta/l$ is a measure of Differential movement δ between two adjacent points separated by the distance l.

Figure 3.2 (a) settlement without tilt (ω) and (b) settlement with tilt. The differential settlement, δ_{AB} results in an angular distortion of β_{AB} between points A and B. Distortion can be expressed in terms of either δ/l or a distortion angle β. Both are used.

Both architectural and structural damage may be observed. Architectural damage includes (1) cracking of plaster and masonry brick walls, which may be unsightly, (2) damage that may be unpleasant (in the case of broken sewer lines and connections to loading docks), and (3) damage that may even be dangerous (as in the case of gas lines breaking, or tilting damage to adjacent buildings). Architectural damage can occur from causes other than foundation settlement. Examples include changes in temperature, moisture (swelling clays; shrinkage due to trees), vibrations due to traffic, wind, blasting, etc.

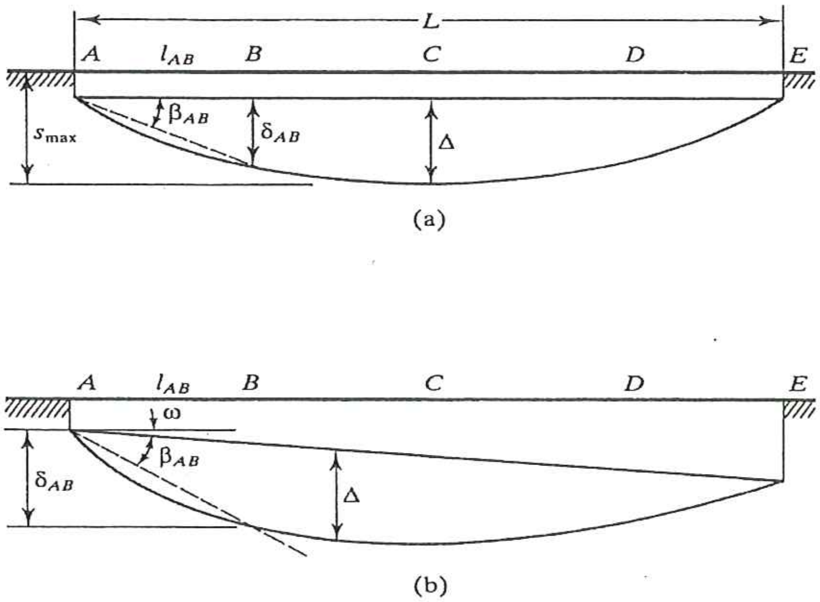


Figure 3.2 Definitions of settlement (Wahls, 1981)

Structural damage is any damage that reduces the ultimate ability of the structure to carry and resist the load imposed on it. Structural damage in any situation is unacceptable, and the foundation design must be such that structural damage is avoided at all costs. In foundation design, there is no point in trying to estimate the severity of structural damage. It must be simply avoided.

The propensity for structural damage is influenced greatly by the rate at which the settlements occur and the rigidity and type of framing system. Settlements that occur very slowly, over periods of decades or more, may be tolerable by masonry or reinforced concrete frame structures. These same settlements, if they occurred within a period of a few months or few years, would result in severe structural damage (Hsai-Yang Fang, 1990).

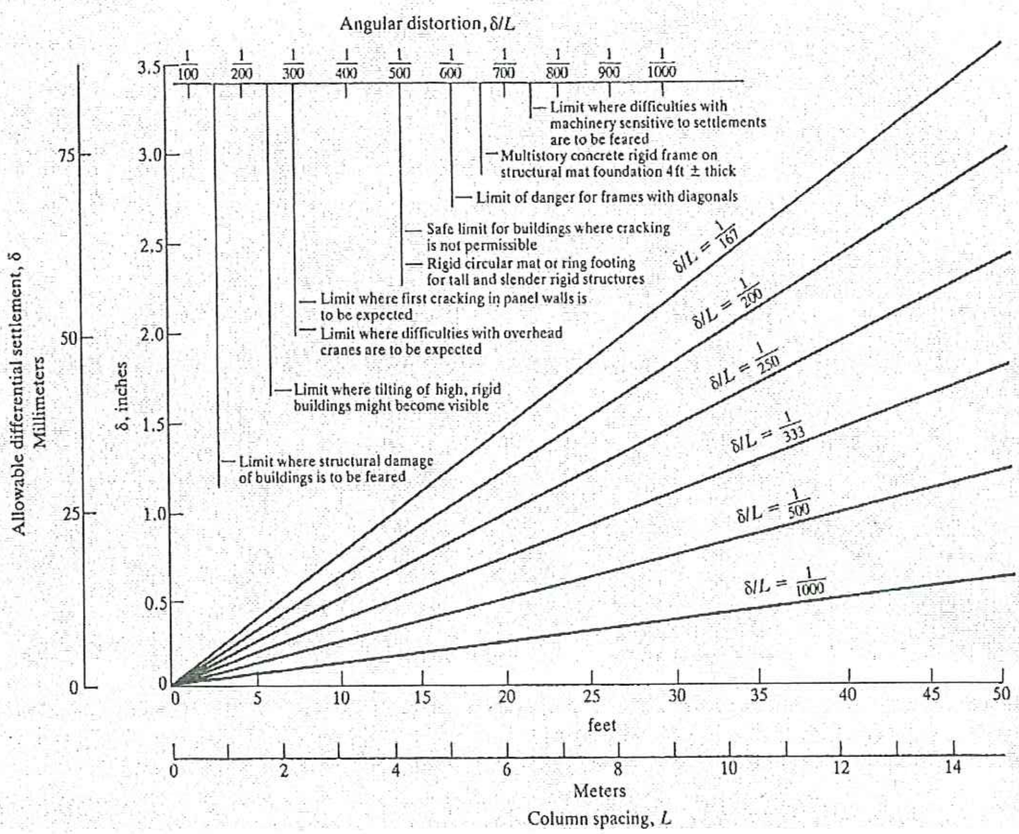
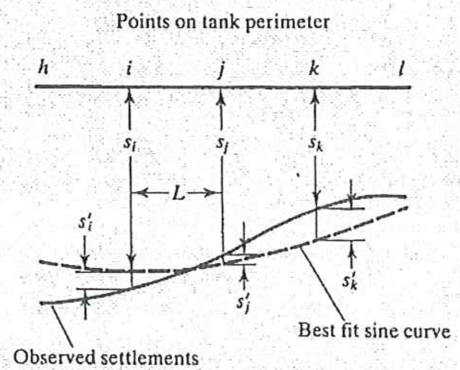


Figure 3.3 Tolerable settlements for buildings(U.S. Navy,1982)

3.2.2 Differential Settlement

Differential settlement, which causes distortion and damages the structures, is a function of the uniformity of the soil, stiffness of the structure, stiffness of the soil, and distribution of loads within the structure. Limitations to differential settlement depend on the application. Embankments, dams, one or two story facilities, and multistory structures with flexible framing systems are sufficiently flexible such that their stiffness often need not be considered in settlement analysis. Pavements may be assumed to be completely flexible. Differential settlement may lead to tilting that can interfere with adjacent structures and disrupt the performance of machinery and people. Differential settlement can cause cracking in the structure, distorted and jammed doors and windows, uneven floors and stairways, and other damages to houses and buildings (US Army Corps of Engineers,1992).

STRUCTURE		TOLERABLE DISTORTION $\frac{\Delta_{max}}{L}$ or β
A. Unreinforced load-bearing walls	Sagging for $L/H < 3$	$\frac{\Delta_{max}}{L}$ = 1/3500 to 1/2500
(*L* and *H* are respectively length and height of the wall from top of footing)	for $L/H > 5$	$\frac{\Delta_{max}}{L}$ = 1/2000 to 1/1250
	Hogging for $L/H = 1$	$\frac{\Delta_{max}}{L}$ = 1/5000
	for $L/H = 5$	$\frac{\Delta_{max}}{L}$ = 1/2500
B. Jointed rigid concrete pressure conduits (Maximum angle change at joint 2 to 4 times average slope of settlement profile. Longitudinal extension affects damage.)		1/65
C. Circular steel petroleum or fluid storage tanks.		$\beta < 1/300$ β' = 1/500 to 1/300



$$\beta = \frac{s_i - s_j}{L}$$

$$\beta' = \left(s'_j - \frac{s'_i + s'_k}{2}\right)\frac{1}{L}$$

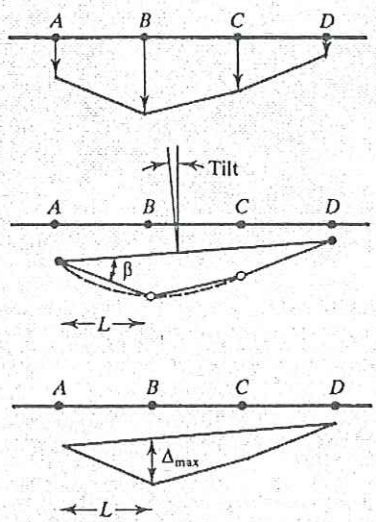


Table 3.2 Tolerable differential settlement for miscellaneous structures. (U.S. Navy,1982)

3.2.2.1 Differential Settlement of Mats (Das,1995)

Calculating the Differential movement of the mat by using rigidity factor K_r ;

Rigidity factor K_r is calculated as

$$K_r = \frac{E' I_b}{E_s B^3} \quad \text{..........................(3.1)}$$

where ;

$E^{/}$ = Modulus of elasticity of the foundation material

E_s = Modulus of elasticity of the soil

B = Width of foundation

I_b = Moment of inertia of the structure at right angles to B

The term E/ Ib can be defined as

$$E^{/}I_b = E^{/}\left(I_F + \sum I_{b^{/}} + \sum \frac{ah^3}{12}\right) \text{.....................} \quad (3.2)$$

where;

E/ Ib =Flexural rigidity of the foundation at right angles to B

Σ E/ Ib/ =Flexural rigitidy of the framed members

Σ($E'ah^3/12$) =Flexural rigitidity of the shear walls

A = Shear wall thickness

H = Height of the shear wall

Based on the value of K_r, the ratio of the diffrential settlement to the total settlement can be estimated in the following manner

1- If K_r>0,5 it can be treated as a rigid mat and δ=0
2- If K_r = 0,5 then δ=0,1
3- If K_r=0 then δ= 0,35 for square mats and δ=0,5 for strip footings (B/L=0)

Edge and Corner settlement of a flexible mat or footing will be approximately 1/2 and 1/4 of the center settlement, respectively. Differential movement of the mat or footing may be calculated from Figure 3.4

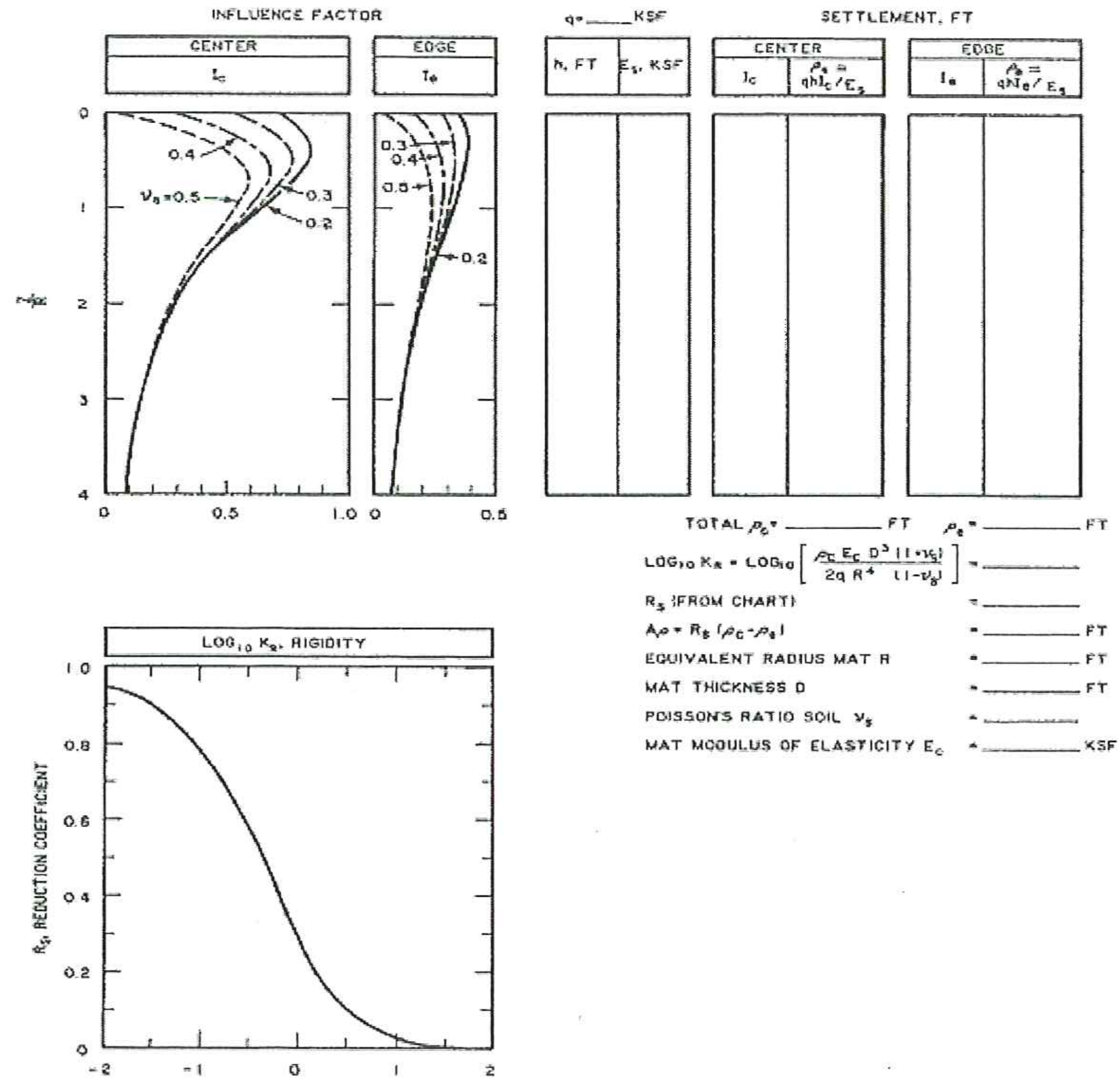


Figure 3.4 Computation of Elastic Settlement Beneath a Mat
(US Army Corps of Engineers,1992)

z = depth beneath mat,

R = equivalent radius,

3.3 Research On The Effects Of Structural Rigidity(stiffness of mat foundation) On Structure-Foundation-Soil interaction

It has been recognised for many years that the stiffness of a structure will affect the distribution of settlements along a strip or raft foundation, and that in turn the distribution of structural loads and moments will be affected by the foundation flexibility. Methods of incorporating the foundation-soil interaction into a settlement analysis have been described by several authors, including, Lee and Brown (1972), Lee (1975) and Poulos (1975). In general, it has been found that the stiffness of the structure generally leads to a reduction in the differential settlements, compared to the usual methods which take the structural loads as being constant and statically determinant. An excellent example of the improvement in differential settlement prediction which may result from incorporating the structural stiffness is presented by Lopes and Gusmao (1991). For a 15 storey apartment building in Brazil,supported by a system of strip footings, the settlement distribution is predicted more closely if the stiffness of the structure is included in the settlement analysis.

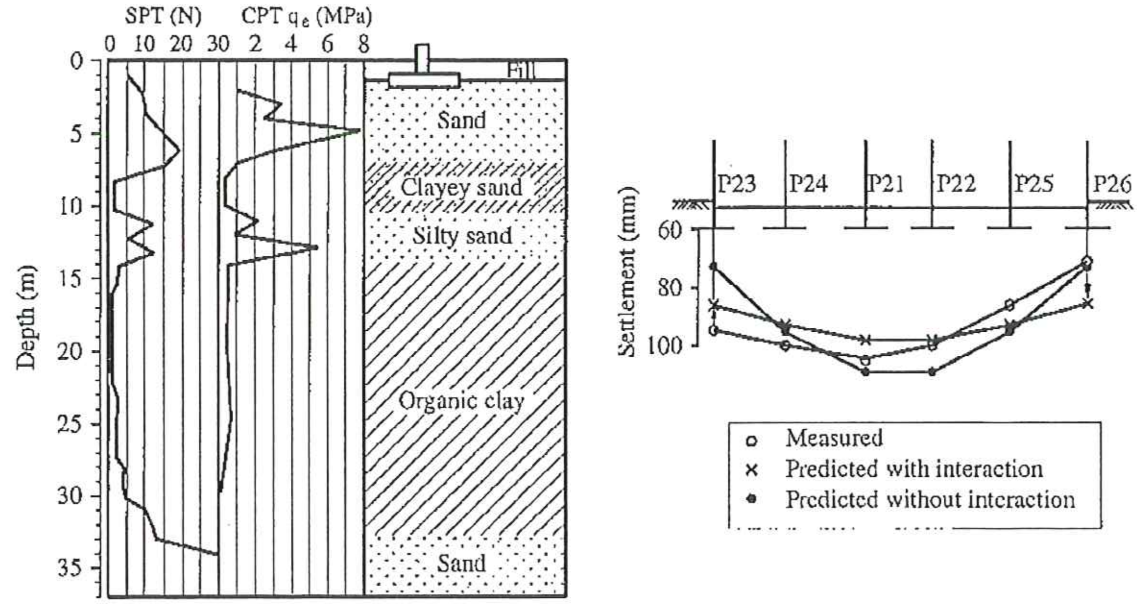


Figure 3.5 Effect of including structure-foundation interaction on predicted settlements

Lee (1975) has studied the effects of raft flexibility on the column loads in two-dimensional and three-dimensional structural frames, and has found that increasing raft flexibility leads to a more uniform distribution of structural loads than is the case for a rigid foundation (the usual case assumed by structural analysts).

Lee also found that the use of the Winkler soil model predicted the reverse trend, and attributed this incorrect trend to the different settlement profiles which emerge from the subgrade reaction theory. Lee made the following observation: "With the advent of large high speed computers, the justification for the Winkler model is removed, and it is clear that it is now only of historical importance and this is no real reason for its continued use". In the intervening 24 years, computer power has increased by orders of magnitude, yet there is still an unfortunate but widespread persistence with the Winkler concept because of its convenience and simplicity. The price of this simplicity is high, given the potential for unreliable and unrealistic results and the enduring problem of assessing an appropriate modulus of subgrade reaction (Poulos, 1999).

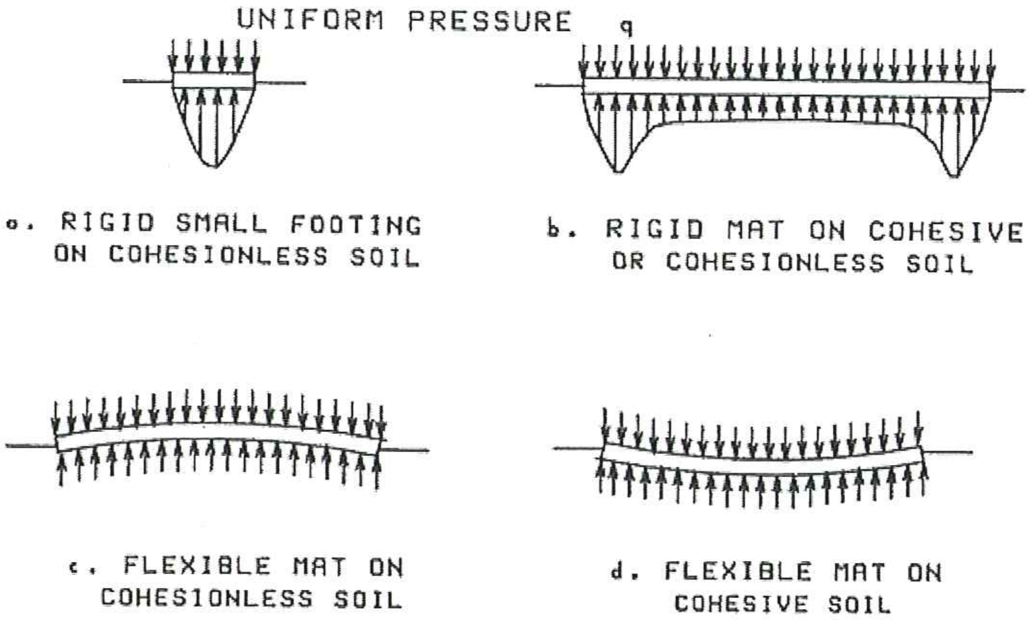


Figure 3.6 Contact pressures under rigid and nonrigid foundations

If the structure is completely rigid (Figure 3.6), uniform total settlement and thus no differential settlement would occur. On the other hand, complete flexibility implies uniform contact pressures between the foundation and the soil and differential settlements (Hsai-Yang Fang, 1990).

3.3.1 Behaviour of Stiffened Mat Foundation

Poulos(1996) at the 7th ANZ conference on Geomechanics(1996) explained the differential settlements and the maximum moments are analysed for different mat foundation stiffnesses in conjunction with the free-field soil movements.
A non linear approach is adopted to undertake the analysis of stresses at the soil -slab interface using the finite element technique,
Poulos increased the stiffnesses of the mat foundation by putting the beams into place under mat as shown in the Figure 3.7a-b

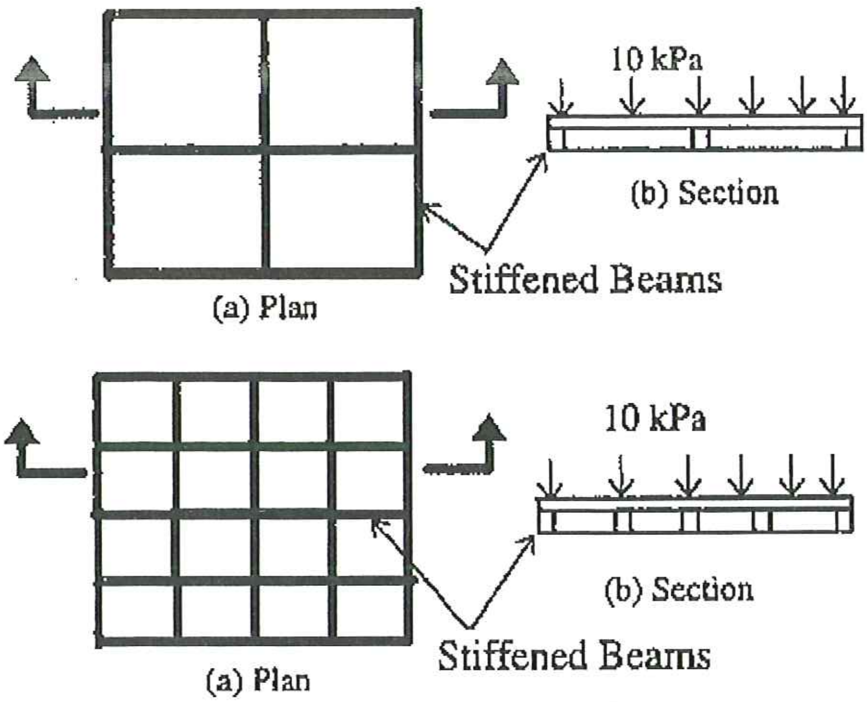


Figure 3.7 Mat foundation strength by beams

A 10m by l0m square raft and incorporating the effect of lift off and local soil yield at 300kPa underneath the slab. The Young's moduli of the raft and soil are adopted as 20GPa and 10 MPa respectively. The Poisson's ratio of the soil is considered as 0.3. The concrete cross -section is adopted as uncracked for the analysis and the shear stresses are assumed to be not significant in the analysis. Then Poulos reached the conclusions as;

- Increase in the mat stiffness leads to larger moments but smaller differential settlements.
- Differential settlement of a raft slab may be reduced by introducing :stiffened beams at
  strategic locations.
- Increase in beam depth causes reduction in differential settlement and increases in the
  maximum moment values.

3.4 CALCULATION METHODS FOR MAT FOUNDATIONS

3.4.1. Rigid Methods

In the conventional rigid method it is assumed that the mat is infinetely rigid and that the bearing pressure against the bottom of the mat follows a planar distribution where the centroid of the bearing pressure coincides with the line of action of the resuldant force of all loads acting on the mat.

The prodecure of design by this method is follows;

1- Calculate total column loads on the mat $\sum S$

2- Calculate the soil pressure under the mat by equation;

$$q = \frac{\sum S}{A} \pm \frac{M_x \bar{y}}{I_y} \pm \frac{M_y \bar{x}}{I_x} \quad \text{......................} \quad (3.3)$$

Where;

ΣS = Total loads on the mat

A = Total area of the mat

X ,Y = Coordinates of any given point on the mat with respect to the x and y axes passing through the centroid of the area of the mat

Mx = $\Sigma S \cdot e_x$ (Bending moment of mat for x direction) ex:coordinates of resultant force

My = $\Sigma S \cdot e_y$ (Bending moment of mat for y direction) ey: coordinates of resultant force

I_x, I_y = Moment of inertia of the area of the mat with respect to the x and y axes respectively.

3- Calculate the soil pressures under the columns using the equation 3.3 by putting the x, y values which points do you want to calculate soil pressure under the mat

4- Draw the mat into several strips x and y direction

5- By calculate moments and shears on strip obtain solution.

3.4.1. Nonrigid Methods

3.4.1.1 Coefficient of Subgrade Reaction

The earliest use of these "springs" to represent the interaction between soil and foundation was done by Winkler in 1867; the model is thus referred to as the Winkler method

In the Winkler foundation model, the foundation for beams or plates is assumed to act like a set of springs. Using Hooke's law, the foundation model can be solved. The foundation model then depends on Young's modulus for the filling material and the height of the springs(Das,1995).

The coefficient of subgrade reaction (modulus of foundation , subgrade modulus)is defined as the ratio between the pressure against the footing or mat and the settlement at a given point

$$k = \frac{q}{S} \qquad \text{.....................................} (3.4)$$

where;

k= Coefficient of subgrade reaction

q = Pressure against the footing or mat at a given point

S = Settlement of the same point of the mat or footing

In clayey soils,settlement under the load takes place over a long period of time and the coefficient should be determined on the basic of the final settlement.On purely granular soils settlement takes place shortly after load application.

Equation (3.4) is based on two assumptions

1- The value k is independent of the magnitude of the pressure
2- The value k has the same value for every point of the surface of footing or mat

However, despite its theoretical convenience, the Winkler soil model has a number of important limitations which are not always appreciated. These include the following:

a. A Winkler soil model only deflects if a pressure is applied to it. Thus unloaded areas in a Winkler soil model do not deflect, and hence there is no stress transmission or interaction within the soil
b. A Winkler soil responds to loading only in the direction of that loading. Thus, for example, vertical loading will produce only vertical displacements, and no horizontal displacements
c. A Winkler soil is usually characterised by the modulus of subgrade reaction, which has units of force-length . The modulus of subgrade reaction is not a fundamental soil parameter, but is dependent on the dimensions of the foundation.
d. A Winkler soil model cannot incorporate properly the effects of soil layering since it does not allow stress transmission. The assessment of the modulus of subgrade reaction for a layered soil profile therefore involves considerable uncertainty which is sometimes resolved by resorting to elastic theory to obtain an equivalent value (Poulos,1999).

3.4.1.2 Approximate Flexible method

In rigid method the mat is assumed to be infinitely rigid. Also the soil pressure is distributed in a straight line and the centroid of the soil pressure is coincidental with the line of the action of the resultant column loads . in the approximate flexible method of design the soil is assumed to be equivalent to an infinite number of elastic springs. The elastic constant of these assumed springs is referred to as the coefficient of subgrade reaction,k

For a surface load acting on a rigid pavement the maxsimum tensile stress occurs at the base of the slab. To estimate the magnitude of the max horizantal tensile stress developed at the base of the rigid pavement, elastic solutions involving slabs on Winkler foundations are useful (Das,1995).

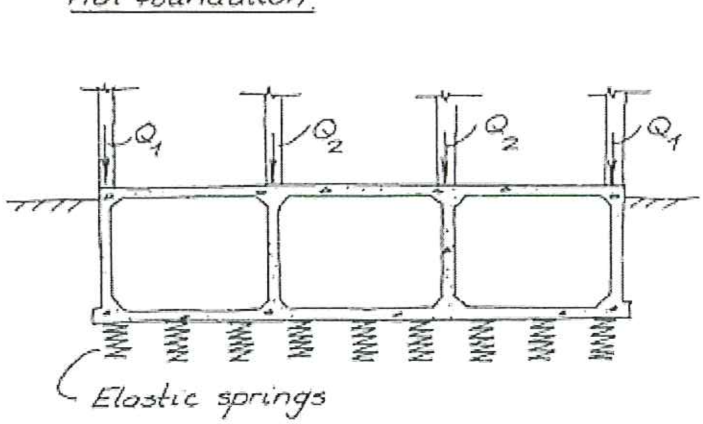


Figure 3.8 Elastic Springs

3.4.1.2 Finite Difference Method

This method assumes that the soil behaves like an infinite number of individual coil springs each of which is not affected by others. The elastic constant of the springs are assumed to be able to resist tension or compression.This assumption was first used by E. Winkler(1867) and this type of foundation is referred to as a Winkler foundation.It is compared to foundation supported on a dense liquid whose unit weight is equal to the coefficient of subgrade reaction

The differential equation for deflection of such a mat foundation is;

$$\nabla^4 w = \frac{q - kw}{D}$$

where $\nabla^4 w = \frac{\delta^4 w}{\delta . x^4} + 2\frac{\delta^4 w}{\delta .^2 x \delta^2 . y} + \frac{\delta^4 w}{\delta^4 y}$

q=Subgrade reaction per unit area of mat

k=Coefficient of subgrade reaction

w=Deflection

D=Rigitity of the mat $= \frac{Et^3}{12(1-\mu^2)}$

E=Modulus of elasticity

T=Thickness of mat

μ=Poisson's ratio

3.4.1.3 Finite Element Method

The finite element method (FEM) is a modern numerical technique which can on the details of finite element modeling, be used to solve problems in geotechnical engineering.

For mat foundation analysis, FEM method divides the mat and soil into 3D finite elements and can go on to non-linear analysis which take care on elastic and plastic material properties of soil.

In part 4 this method describes the datils of this approach .

PART 4

FINITE ELEMENT ANALYSIS WITH TNO DIANA

Big percent of standart finite element pacages can solve only a limited range of conventional engineering problems. However;
DIANA obtain advanced modelling and analysis functionality by offering the solution for all types of analysis where Non-conventional engineering problems require such as complex non linear material behaviour, complex models where the structure interacts with soil/fluid, stresses induced from extreme loading conditions
In this section a general view to computer program TNO DIANA(Displacement Analyzer), based on the Finite Element technique and researched Element types and the Nonlinear model type which are are using on the thesis study.

4.1 ELEMENT TYPES

4.1.1. Curved Shell Elements(using define the concrete mat foundation,columns and shear walls)

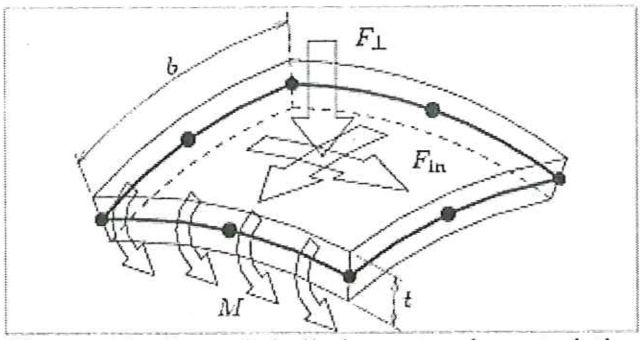


Figure 4.1: Curved shell elements, characteristics

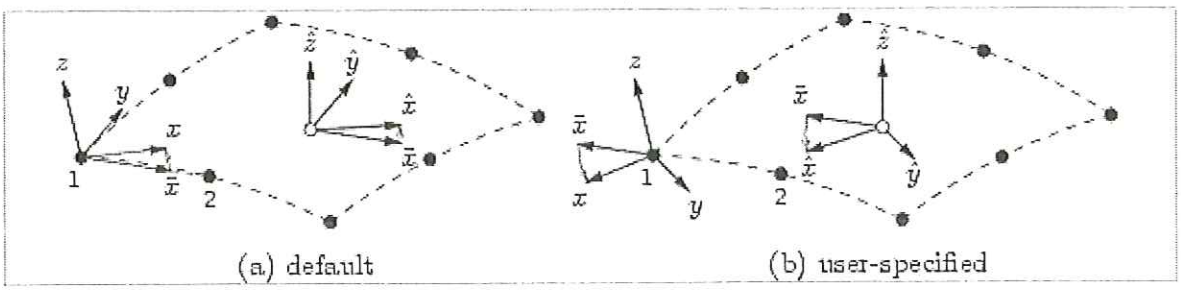


Figure 4.2: Axes

4.1.1.1 Displacements

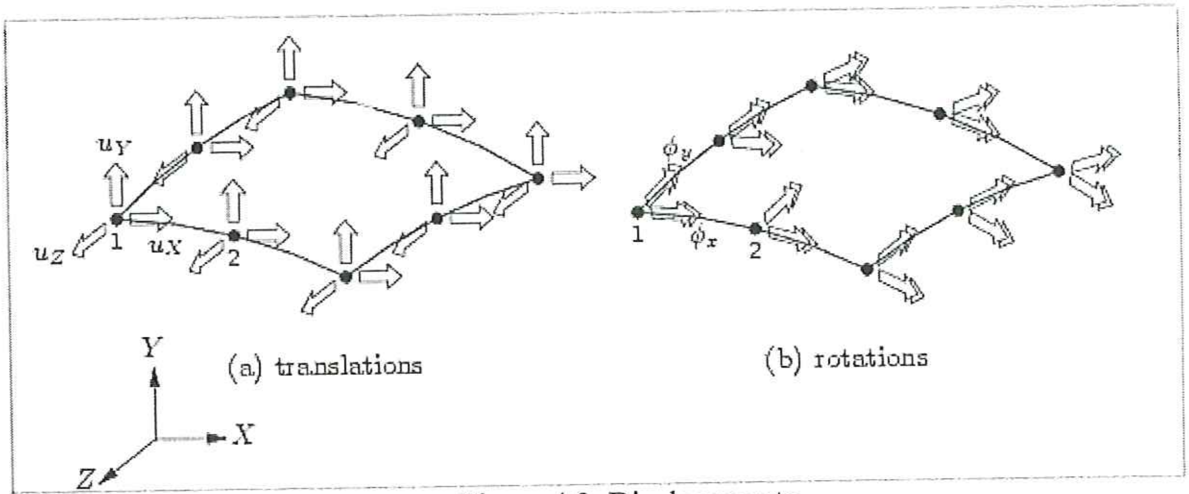


Figure 4.3: Displacements

The basic variables in the nodes of the curved shell elements are the translations u_X, u_Y and u_Z in the global XYZ directions [Fig4.12 a] and the rotations ϕ_x and ϕ_y respectively around the local $+x$ and $+y$ axes in the tangent plane [Fig 4.3b].

4.1.1.2. Stresses

Diana can calculate and output two types of stresses for curved shell elements: Cauchy stresses and generalized moments and forces.

4.1.1.3 Cauchy Stresses

$$\sigma = \left\{ \begin{array}{l} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{zz} = 0 \\ \sigma_{xy} = \sigma_{yx} \\ \sigma_{yz} = \sigma_{zy} \\ \sigma_{zx} = \sigma_{xz} \end{array} \right\} \quad \text{............(4.1)}$$

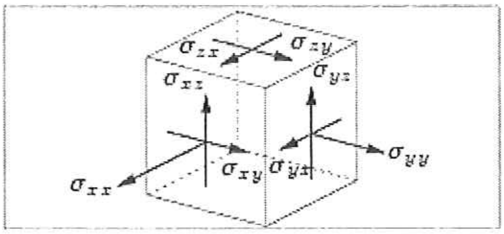


Figure 4.4: Cauchy stresses

Figure 4.4 shows these stresses on a unit cube in their positive direction. Note that tension stress is positive.

4.1.1.4 Generalized Moments and Forces

Diana can derive the bending moments **m** and forces **f** of Equation 4.2

$$\mathbf{m} = \left\{ \begin{array}{l} m_{xx} \\ m_{yy} \\ m_{xy} = m_{y.} \end{array} \right\} \quad \mathbf{f} = \left\{ \begin{array}{l} n_{xx} \\ n_{yy} \\ n_{xy} = n_{yx} \\ q_{xz} \\ q_{yz} \end{array} \right\} \quad \ldots(4.2)$$

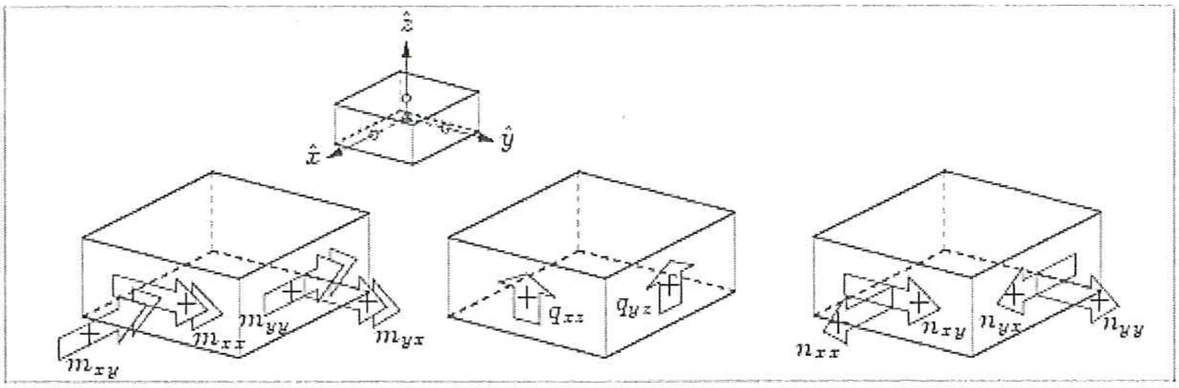


Figure 4.5: Generalized moments and forces

The sign convention is that a positive moment yields positive stresses in the upper plane and that a positive shear force yields positive shear stresses.

4.1.2. Solid Elements(define the soil)

Solid elements are general purpose elements. However, because of their tendency to produce large systems of equations, these elements are usually applied only when other elements are unsuitable or would produce inaccurate analysis results. Solid elements are characterized by the following properties [Fig.4.6]:

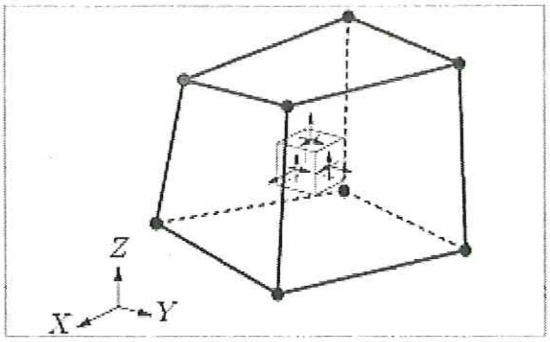


Figure 4.6: Solid elements, characteristics

The stress situation is three-dimensional, the loading may be arbitrary, the dimension in three axial directions X, Y and Z are of the same order of magnitude.

4.1.2.1 Axes

For solid elements Diana needs no special user input data to set up the element axes. By default, the element x , y and z axes are set up parallel to the global X , Y and Z axes respectively [Fig.4.7a].

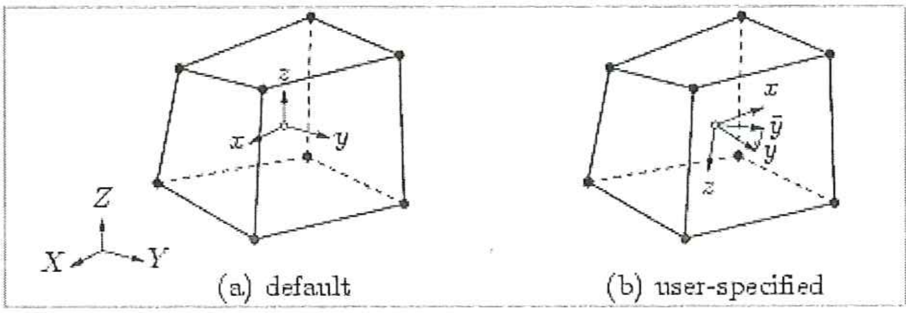


Figure 4.7: Axes

4.1.2.2 Displacements

The basic variables in the nodes of solid elements are the translations u_x , u_y and u_z in the local element directions [Fig.4.8]

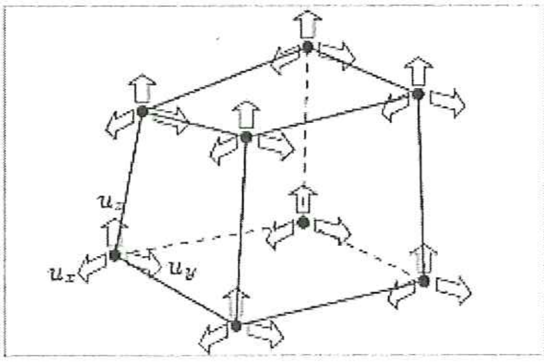


Figure 4.8: Displacements

$$\mathbf{u}_e = \begin{Bmatrix} u_x \\ u_y \\ u_z \end{Bmatrix} \text{......} \quad (4.3)$$

4.1.2.3. Stresses

Diana can calculate and output Cauchy stresses for all types of solid elements. For some element types, Diana can determine and output generalized moments and forces by integrating the Cauchy stresses in a user-specified thickness direction.

4.1.2.4. Cauchy Stresses

$$\sigma = \begin{Bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{zz} \\ \sigma_{xy} = \sigma_{yx} \\ \sigma_{yz} = \sigma_{zy} \\ \sigma_{zx} = \sigma_{xz} \end{Bmatrix} \text{..........} (4.4)$$

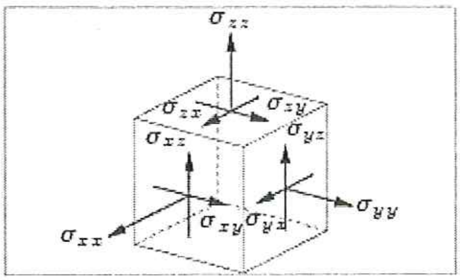


Figure 4.9: Cauchy stresses

Figure 4.9 shows these stresses on a unit cube in their positive direction. Note that tension stress is positive.

4.2 MOHR-COULOMB

The yield condition of Mohr-Coulomb [Fig.4.10a] is an extension of the Tresca yield condition to a pressure dependent behavior. The formulation of the yield function can be expressed in the principal stress space ($\sigma_1 \geq \sigma_2 \geq \sigma_3$) as

$$f(\sigma,\kappa)=\frac{1}{2}(\sigma_1-\sigma_3)+\frac{1}{2}(\sigma_1+\sigma_3)\sin\phi(\kappa)-\bar{c}(\kappa)\cos\phi_0$$

with $\bar{c}$ (κ) the cohesion as a function of the internal state variable κ, and ϕ the angle of internal friction which is also a function of the internal state variable.

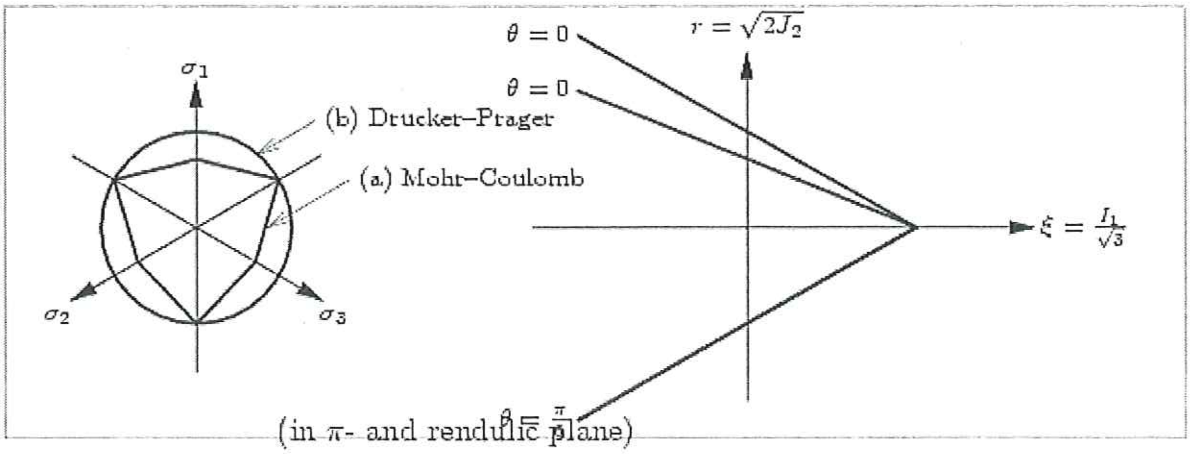


Figure 4.10: Mohr-Coulomb and Drucker-Prager yield condition

The initial angle of internal friction is given by ϕ_0. The flow rule is given by a general non-associated flow rule $g \neq f$, but with the plastic potential given by

$$g(\sigma,\kappa)=\frac{1}{2}(\sigma_1-\sigma_3)+\frac{1}{2}(\sigma_1+\sigma_3)\sin\phi(\kappa) \quad \text{........ (4.5)}$$

which results for the plastic strain rate vector

$$\dot{\varepsilon}^p = \dot{\lambda} \begin{Bmatrix} \frac{1}{2}(1+\sin\psi) \\ 0 \\ -\frac{1}{2}(1-\sin\psi) \end{Bmatrix} \text{..................(4.6)}$$

4.2.1 Hardening

The relation between the internal state variable κ and the plastic process is given by the hardening hypothesis. For the Mohr-Coulomb yield condition we consider only the strain hardening hypothesis.

4.2.2 Strain hardening.

In the case of strain hardening the relation is given in the principal space by

$$\kappa = \sqrt{\tfrac{2}{3}\left(\dot{\varepsilon}_1^p \dot{\varepsilon}_1^p + \dot{\varepsilon}_2^p \dot{\varepsilon}_2^p + \dot{\varepsilon}_3^p \dot{\varepsilon}_3^p\right)} \qquad \text{............(4.7)}$$

which can be elaborated to

$$\kappa = \dot{\lambda} \sqrt{\tfrac{1}{3}\left(1 + \sin^2 \psi\right)} \qquad \text{..................(4.8)}$$

<u>Relation c- κ</u>

The translation of uniaxial experimental data to the equivalent cohesion-internal state variable, the $\bar{c}$ - κ relation, depends on the hardening hypothesis. In the following example it will give the derivation for a cohesion hardening material with constant friction and dilatation angle, i.e., ϕ $(\kappa) = \phi_0$ and ψ $(\kappa) = \psi_0$, and a strain hardening hypothesis.

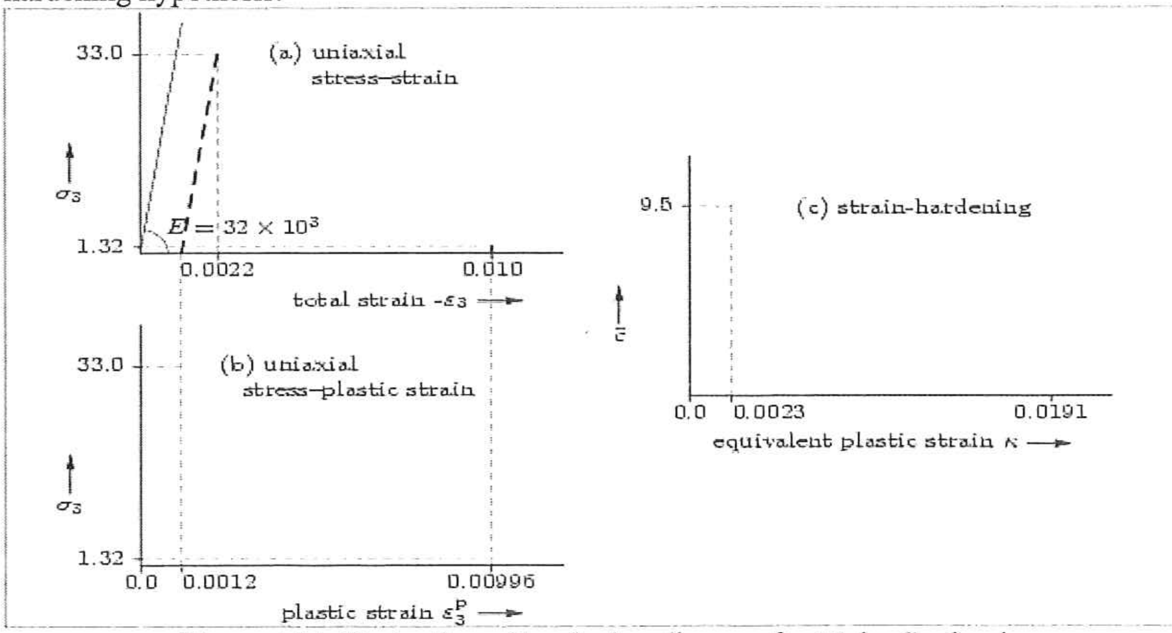


Figure 4.11: Derivation of hardening diagram for Mohr-Coulomb

Consider the uniaxial stress-strain diagram σ_3 - ε_3 of Figure 4.8a. The plastic strain ε_3^p is assumed to be given by ε_3 - ε_3^e. Figure 4.8b shows the uniaxial stress-plastic strain diagram. For uniaxial stressing, $(\sigma_1, \sigma_2, \sigma_3) = (0, 0, \sigma_3)$, plastic flow occurs at a vertex of the yield surface. Symmetry conditions dictate that the two possible yield directions contribute equally to the plastic strain rate vector

$$\dot{\varepsilon}^p = \begin{Bmatrix} \dot{\varepsilon}_1^p \\ \dot{\varepsilon}_2^p \\ \dot{\varepsilon}_3^p \end{Bmatrix} = \dot{\lambda} \begin{Bmatrix} \frac{1}{4}(1+\sin\psi_0) \\ \frac{1}{4}(1+\sin\psi_0) \\ -\frac{1}{2}(1-\sin\psi_0) \end{Bmatrix} \quad \text{.....................(4.9)}$$

With the relation derived previously, we find for the relation between the uniaxial plastic strain and the internal state variable for a strain hardening hypothesis

$$\dot{\kappa} = -\frac{\sqrt{1+\sin^2\psi_0} - \frac{2}{3}\sin\psi_0}{1-\sin\psi_0}\dot{\varepsilon}_3^p \quad \text{............................(4.10)}$$

The relation between the uniaxial stress $\sigma_3 = -f_c$ and the equivalent cohesion $\bar{c}$ is given by

$$\bar{c} = f_c \frac{1-\sin\phi_0}{2\cos\phi_0} \quad \text{.............(4.11)}$$

Figure 4.8illustrates the procedure for $\phi_0 = \psi_0 = 30°$.

4.3. VON MISES

The yield condition of Von Mises is a smooth approximation of the Tresca yield condition: a circular cylinder in the principal stress space [Fig.4.12b].

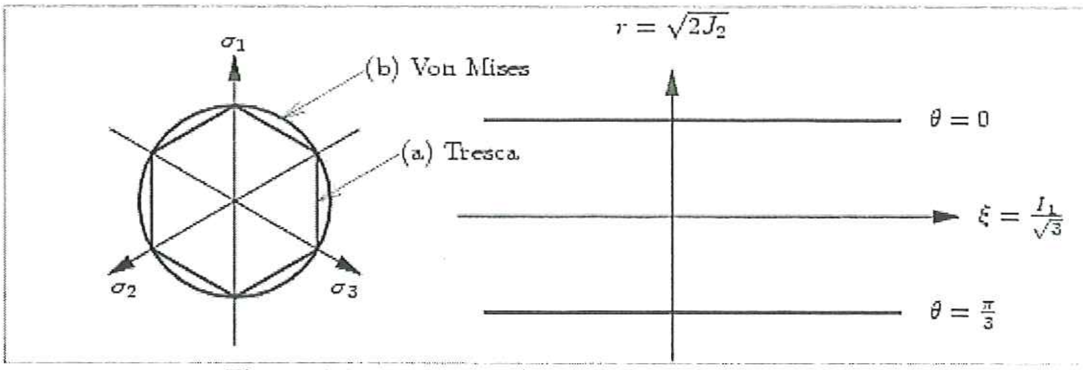


Figure 4.12: Tresca and Von Mises yield condition

The yield function of Von Mises is given by the square root formulation

$$f(\sigma,\kappa) = \sqrt{3J_2} - \sigma(\kappa) = \sqrt{\frac{1}{2}\sigma^T P\sigma} - \sigma(\kappa) \quad \text{...............(4.12)}$$

4.4. SOIL BEHAVIOR

The initial state of stress in the soil can be characterized by the volumetric weight of the soil γ, the depth z and the lateral pressure ratio K_0 . This ratio is defined as the quotient of the horizontal (principle) effective stress σ_1^l and the vertical effective stress σ_3^l:

$$K_0 = \frac{\sigma_1^l}{\sigma_3^l} = \frac{\sigma_1^l}{\gamma_z} \quad \text{... (4.13) and} \quad \sigma_h^l = K_0 \sigma_v^l \quad \text{......(4.14)}$$

During the evaluation of the initial state for the nonlinear analysis, DIANA will compose the initial nonlinear stress components from the calculated elastic vertical stress of the specified load set and multiplication factor

The linear load set must contain a dead weight load from which Diana derives the (vertical) direction of gravity.

4.3.1. Undrained Behavior

In a geotechnical analysis, the permeability of a saturated soil is small in comparison with the loading rate, the behavior is incompressible in a short term. This *undrained* *behavior* can be modeled by defining an excess pore fluid pressure p_e in the material as

$$p_e = - K_{f,\,num}\ \varepsilon_v \quad \ldots\ldots\ldots\ldots(4.15)$$

with ε_v the volumetric strain and K_f the undrained compression modulus which is equal to the drained compression modulus times a penalty factor: $K_f = fac \times K$ [$fac = 500$]

This method does not create separate degrees of freedom for the pore pressure, though it can be used in combination with mixture elements.Note that the drained compression modulus is derived from the specified constant Young's modulus and the Poisson's ratio. Therefore, in case of nonlinear elasticity, one should use this option with care. Also note that the theoretical formula

$$p_e = -\frac{K_f}{n}\varepsilon_v \quad \ldots\ldots\ldots\ldots\ldots(4.16)$$

with n the porosity, is *not* applied because DIANA uses the fluid compression modulus only as a numerical artifice which value should be just sufficiently larger than the drained compression modulus.

PART 5

NONLINEAR ANALYSIS OF THE MODEL

5.1 DESCRIPTION OF THE MODEL

A four storey reinforced concrete structure with 8 columns and 6 shearwalls rests on a mat foundation which dimensions 14^m by 20^m.

The statical analysis of superstructure was performed by Sta4.cad computer software. The columns , shearwall loads, moments and dimensions were taken from Sta4 cat, was given on the Table 5-1

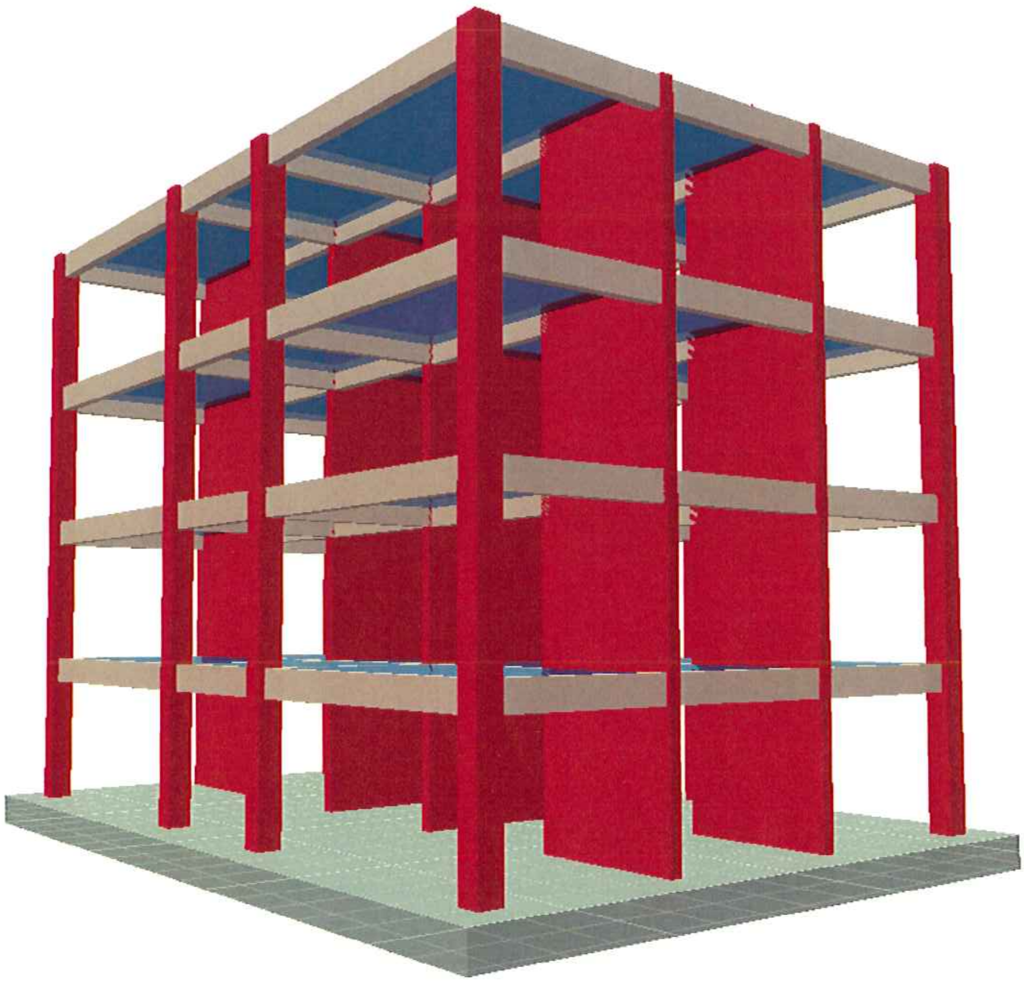
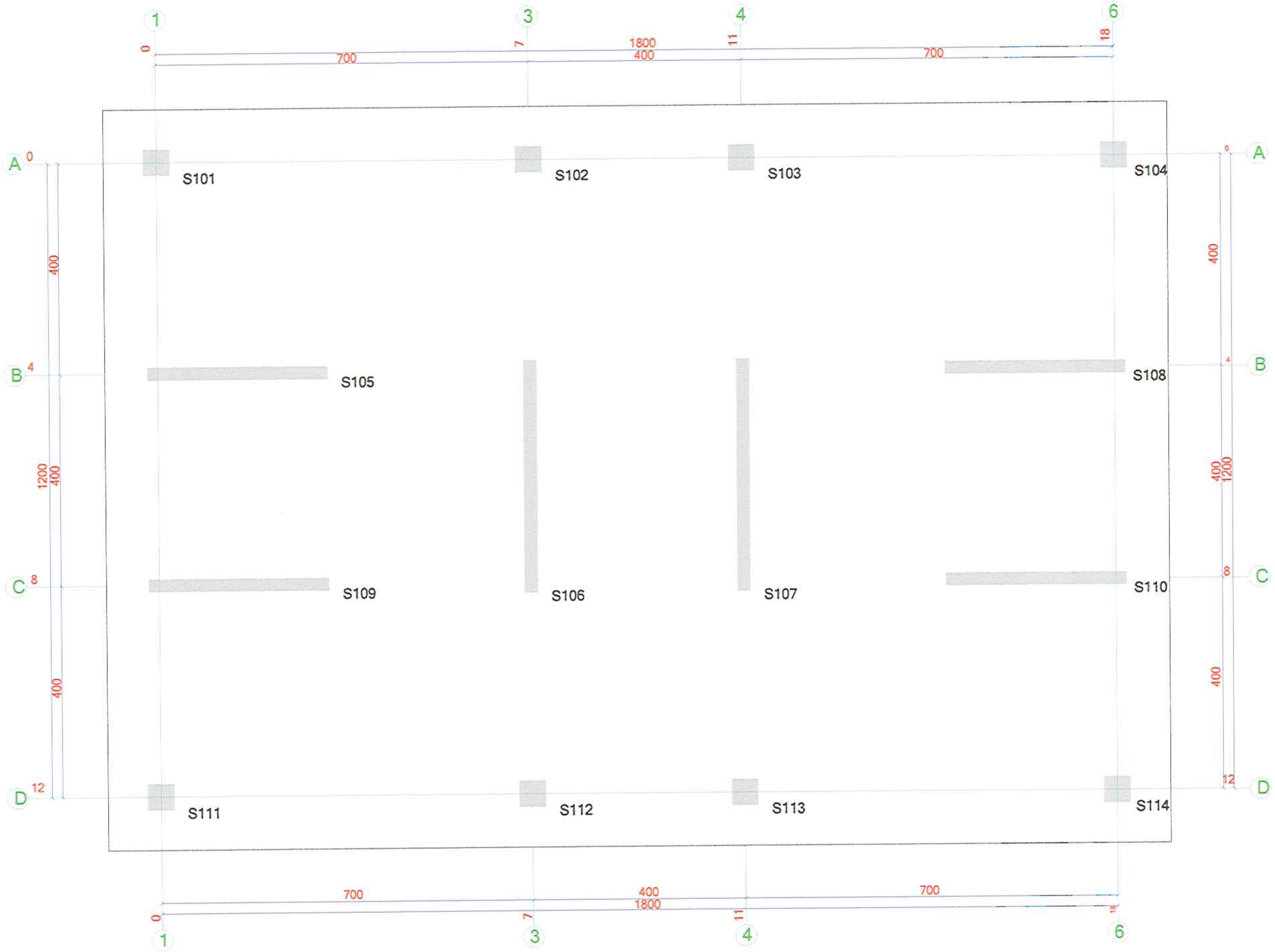


Figure 5.1 3D view of superstructure

Table 5.1 Column and Shear Wall Loads- Moments and Dimensions

Columns and Shearwalls	Loads kN	Earthquake M_x Moment kN-m	Earhtquake M_y Moment kN-m	Column Dimensions (m)	
				X	Y
S101	456	31,9	50,1	0,5	0,5
S102	632	38,2	40,3	0,5	0,5
S103	632	38,2	40,3	0,5	0,5
S104	456	31,9	50,1	0,5	0,5
S105	1062	2393	49,3	3,40	0,25
S106	1714	43	4626,5	0,25	4,40
S107	1714	43	4626,5	0,25	4,40
S108	1062	2393	49,3	3,40	0,25
S109	1062	2393	49,3	3,40	0,25
S110	1062	2393	50,1	3,40	0,25
S111	456	31,9	4,6	0,5	0,5
S112	632	38,2	40,3	0,5	0,5
S113	632	38,2	40,3	0,5	0,5
S114	456	31,9	50,1	0,5	0,5
	ΣLoads = 12026,6 kN				


1
3
4
6
700
1800
400
700
A
B
C
D
400
1200
400
400
S101
S102
S103
S104
S105
S106
S107
S108
S109
S110
S111
S112
S113
S114

5.2 DESCRIPTION OF THE CASES STUDIED

Dimensions of mat : 14^m by 20^m

Area of mat: 280 m^2

ΣLoads = 12026,6 kN

Thickness of mat:	$0,6^m$	$0,9^m$	1.2^m
Mass of mat:	4032 kN	6048 kN	8064 kN

Table 5.2 Self Weights of Mat (Unit weight of concrete=24 kN/m^3)

Table 5.3 NON-LINEAR STATIC ANALYSIS 1A- 1B-1C Properties

Concrete		**Soil (OC CLAY)**	
Elastic Modulus of Concrete	$3.2 * 10^4$ MPa	Elastic Modulus of Soil	5 MPa
Mass Density	0.24 kN/m3/g	Mass Density	0.16 kN/m3/g
Poisson's ratio	0.30	Poisson's ratio	0.25
Analysis 1A thickness of mat is $0,6^m$		Model Type	Mohr Coulomb
Analysis 1B thickness of mat is $0,9^m$		Cohesion (c)	60 kPa
		Friction Angle(ø)	25 Deg
Analysis 1C thickness of mat is $1,2^m$		K_0	0.577

Table 5.4 NON-LINEAR STATIC ANALYSIS 2A-2B-2C Properties

Concrete		**Soil (NL CLAY)**	
Elastic Modulus of Concrete	$3.2 * 10^4$ MPa	Elastic Modulus of Soil	2 MPa
Mass Density	0.24 kN/m3/g	Mass Density	0.16 kN/m3/g
Poisson's ratio	0.30	Poisson's ratio	0.25
Analysis 2A thickness of mat is $0,6^m$		Model Type	Mohr Coulomb
Analysis 2B thickness of mat is $0,9^m$		Cohesion (c)	30 kPa
Analysis 2C thickness of mat is $1,2^m$		Friction Angle (ø)	12,5 Deg
		K_0	0,784

5.3. MODELLING THE GEOMETRY

5.3.1 Modelling Soil and Mat foundation

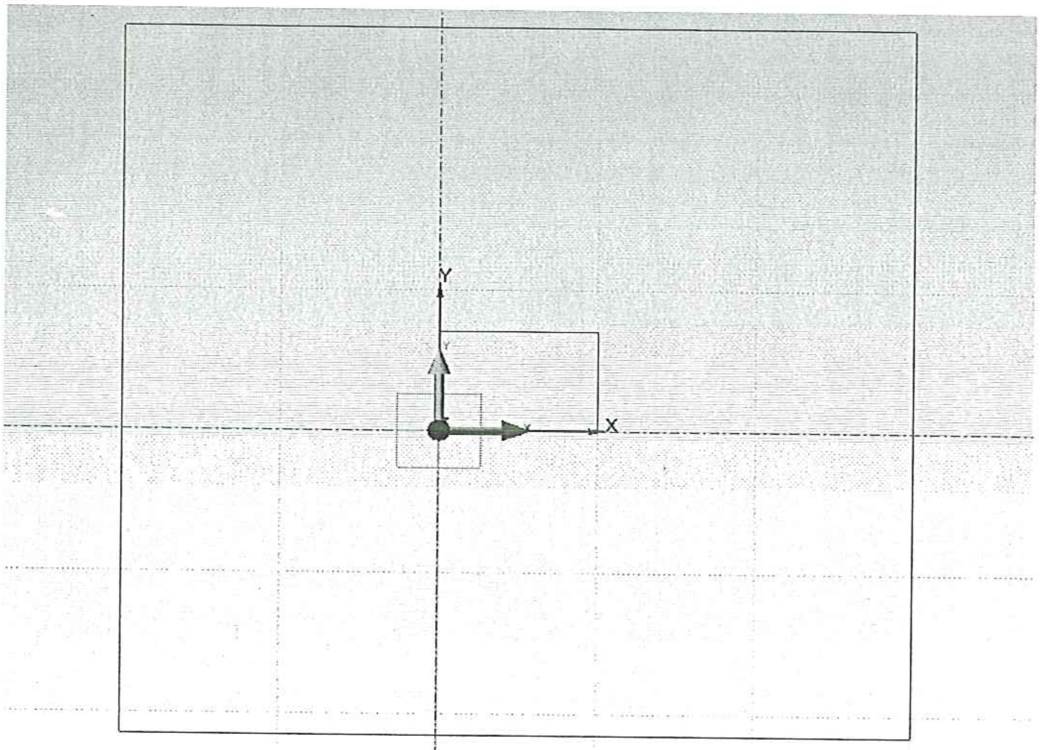


Figure 5.2 Modelling Soil and Mat foundation

5.3.2 Modelling columns and shearwalls

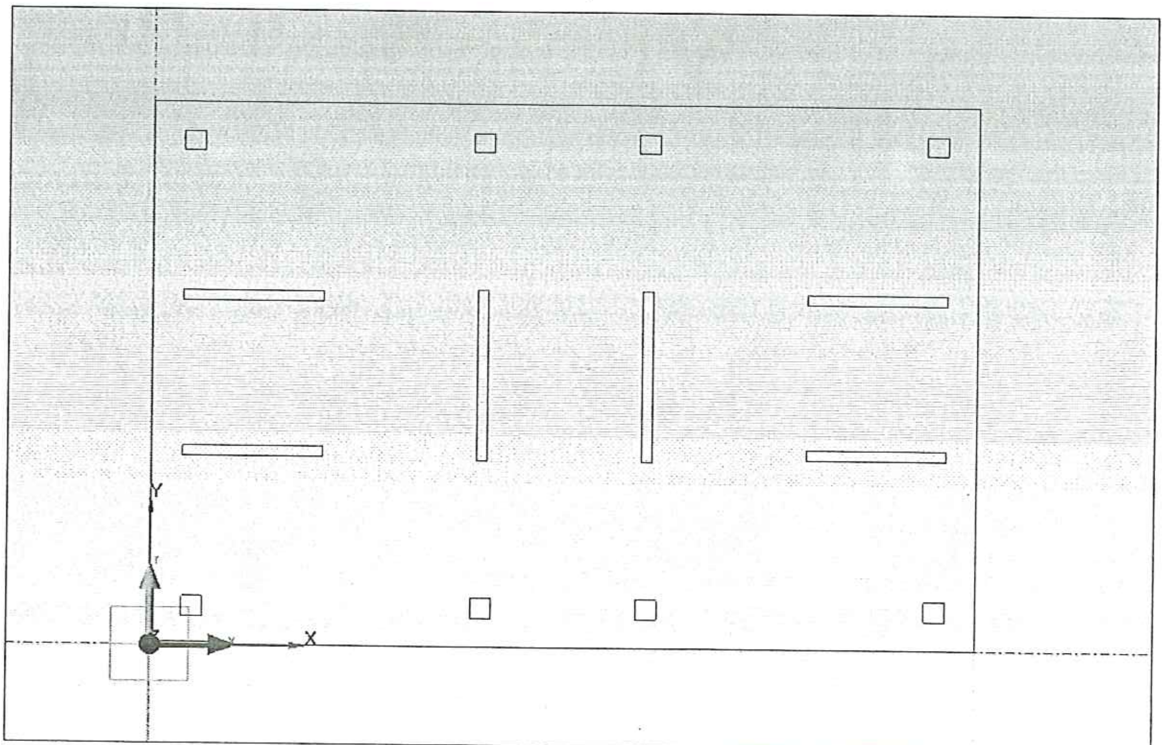


Figure 5.3 Modelling columns and shearwalls

5.3.3 Mesh Generation

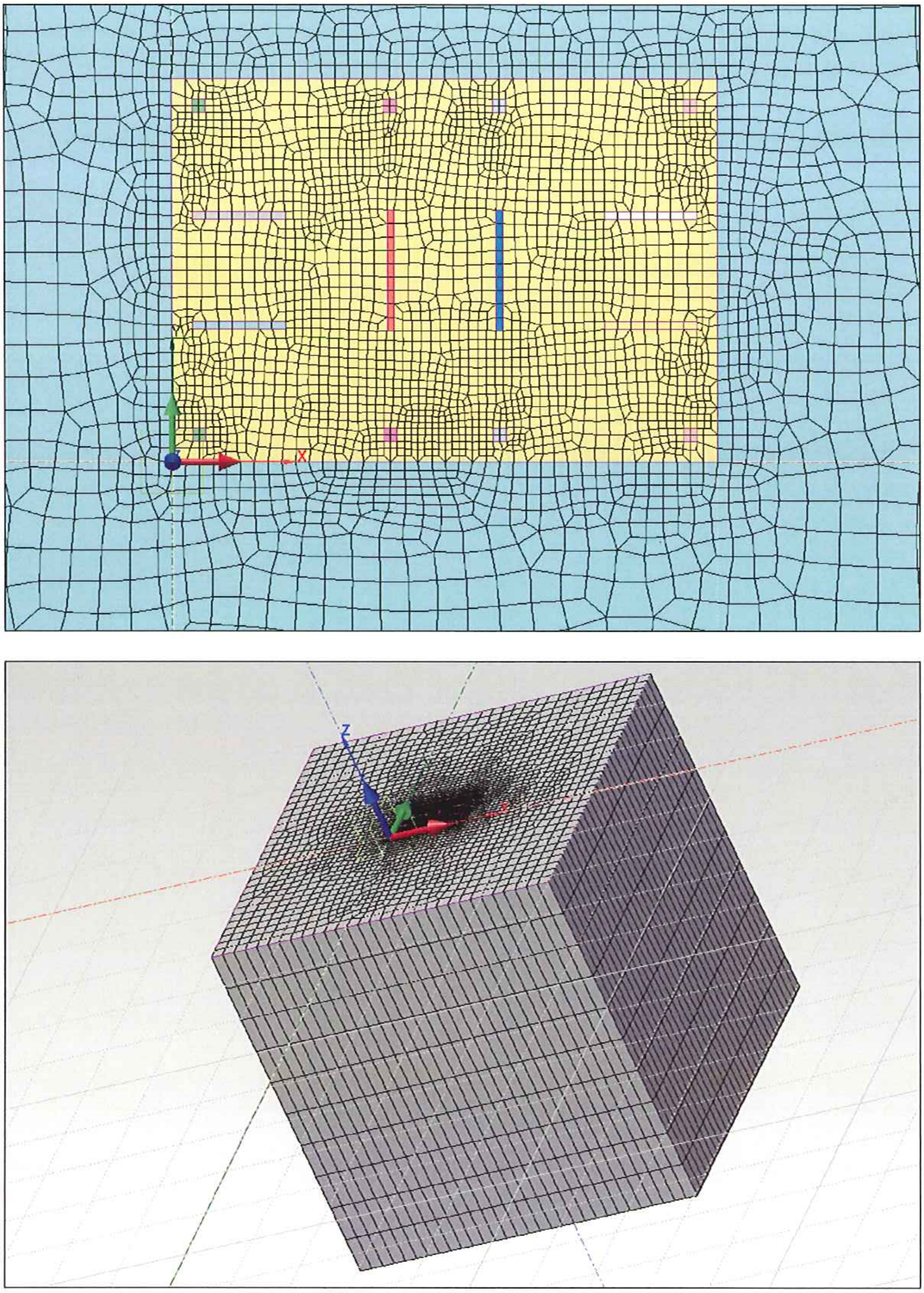


Figure 5.4 Mesh Generation of model

5.3.4 Boundary conditions

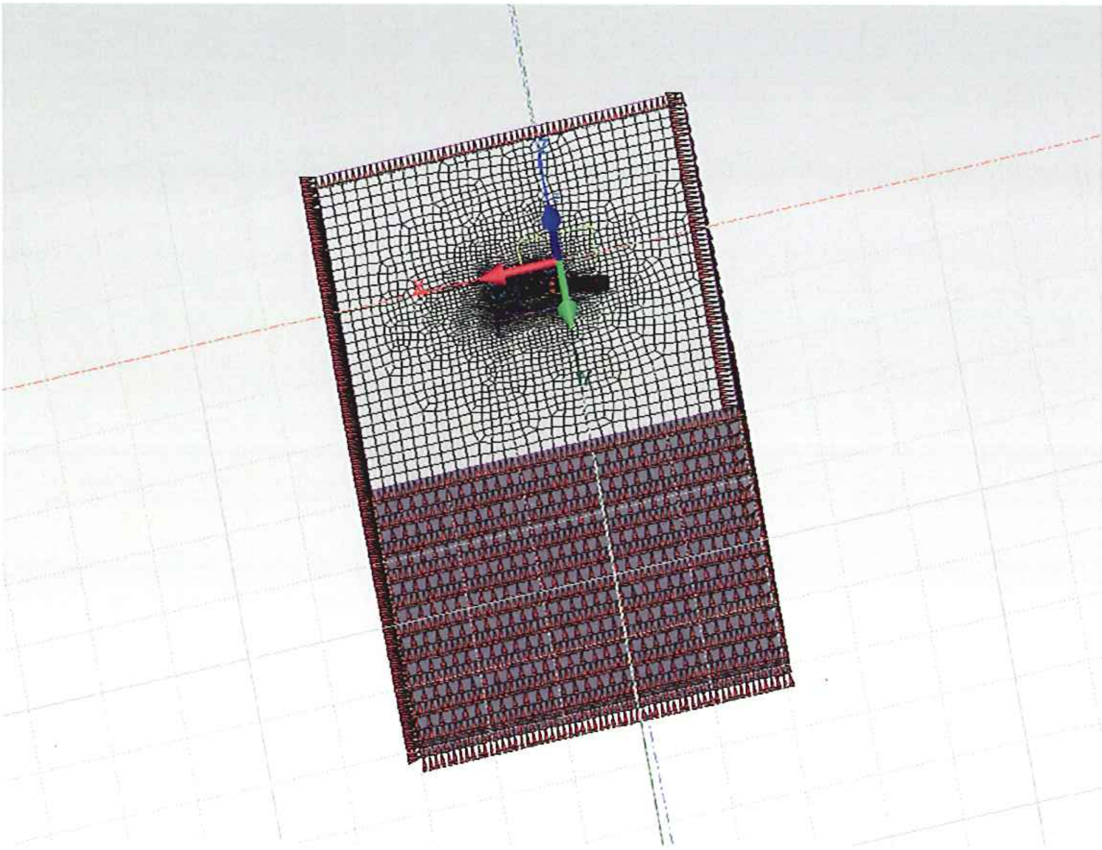


Figure 5.5 Boundary conditions

5.3.5 RESULTS

Next step Analysis 1A results are shown as contour plots.Other results of analysis 1B-1C-2A-B-C will be shown at results tables 5.14 and 5.15

5.4. Analysis 1A (thickness of the mat is $0{,}6^{m}$ / OC clay)

5.4.1. Settlements of the raft

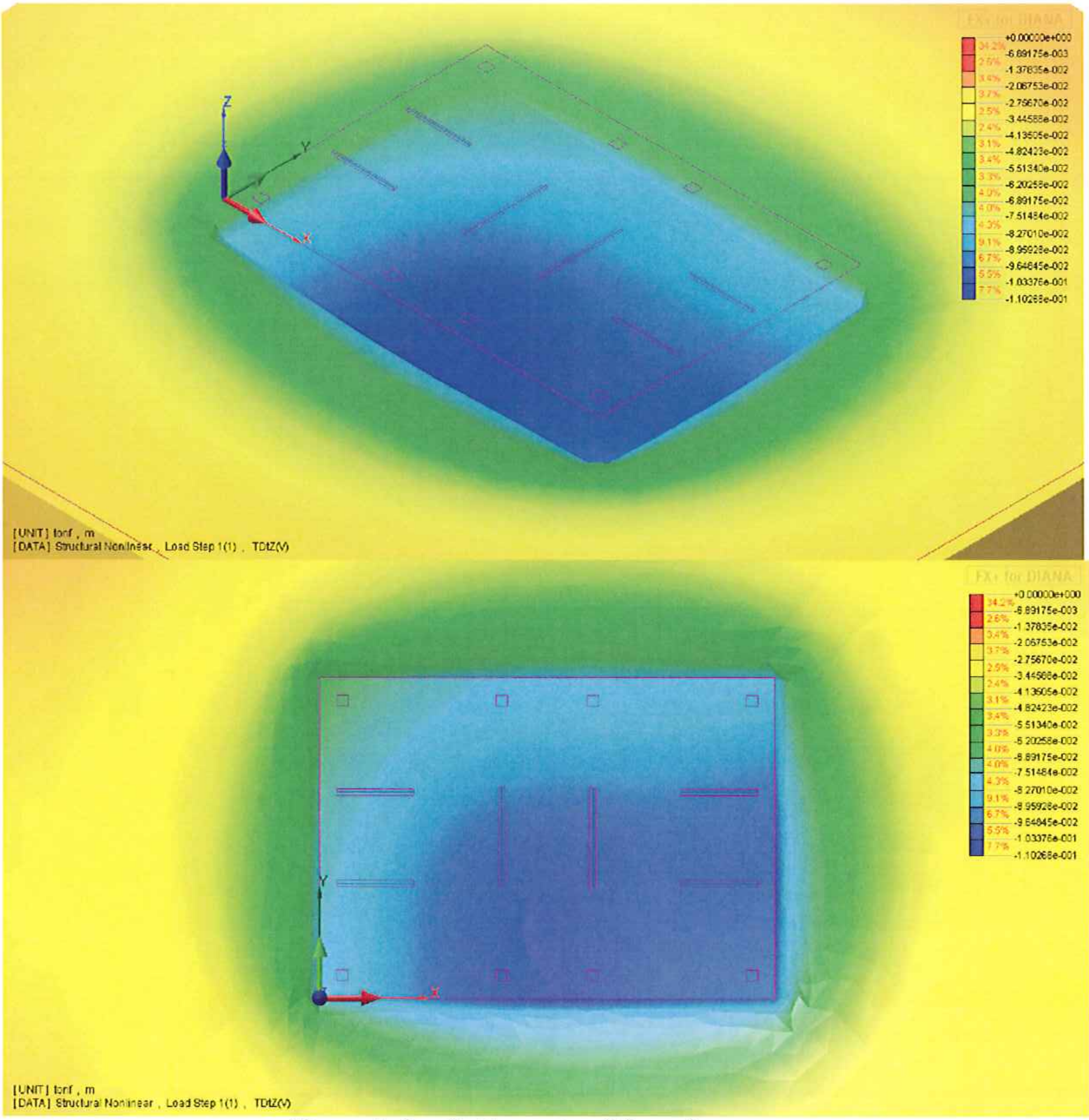


Figure 5.6a Settlements of the raft

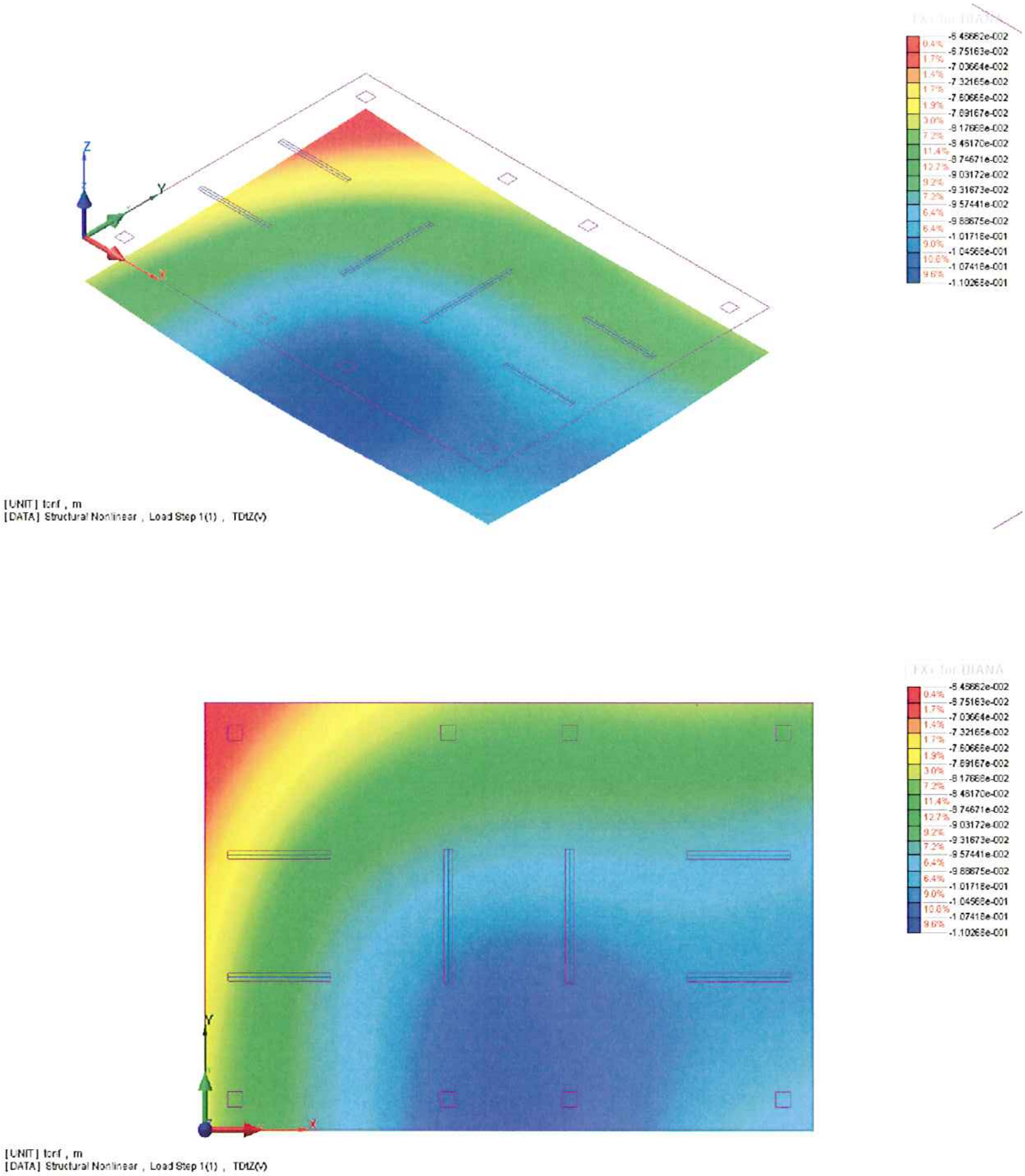
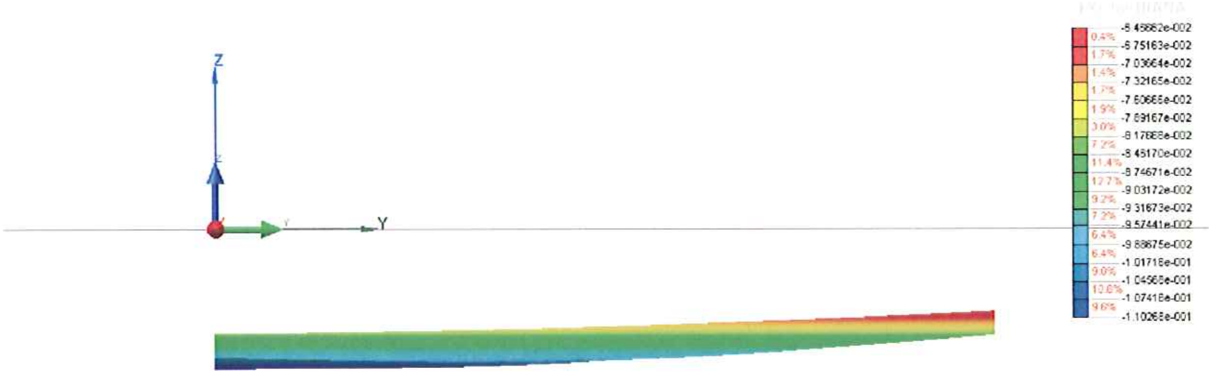


Figure 5.6b Settlements of the raft



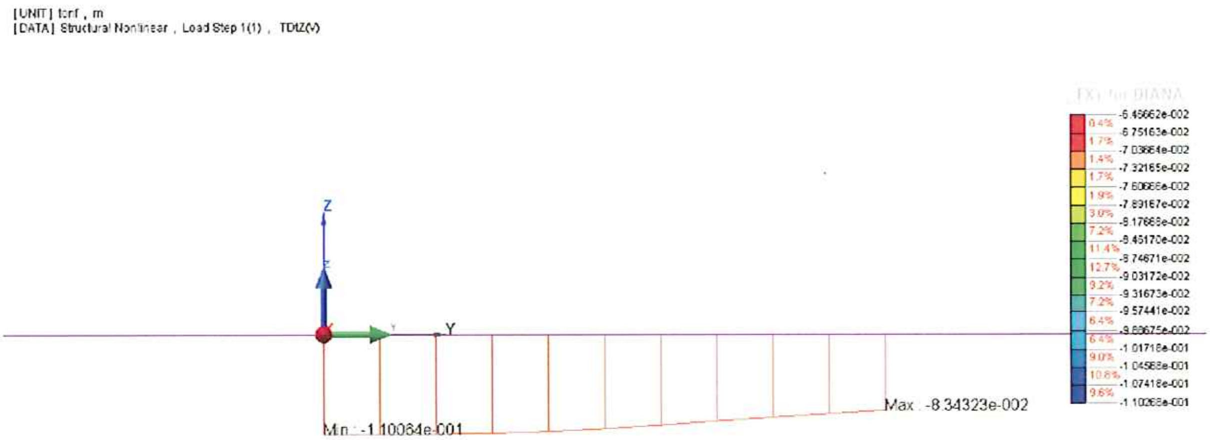


Figure 5.6c Settlements of the raft

Table 5.5 Vertical Displacements along the Centerline

X (m)(L=20^{m})	Y(m)(B=14^{m})	Value**(cm)**
9,66	14.00	-8.34
9,66	12,60	-8.72
9,66	11,20	-9.11
9,66	9,80	-9.51
9,66	8,40	-9.94
9,66	7.00	-10.34
9,66	5,60	-10.65
9,66	4,20	-10.84
9,66	2,80	-10.92
9,66	1,40	-10.96
9,66	0.00	-11.01

5.4.2. Shear forces on the mat

5.4.2.1 Shear forces at yz direction

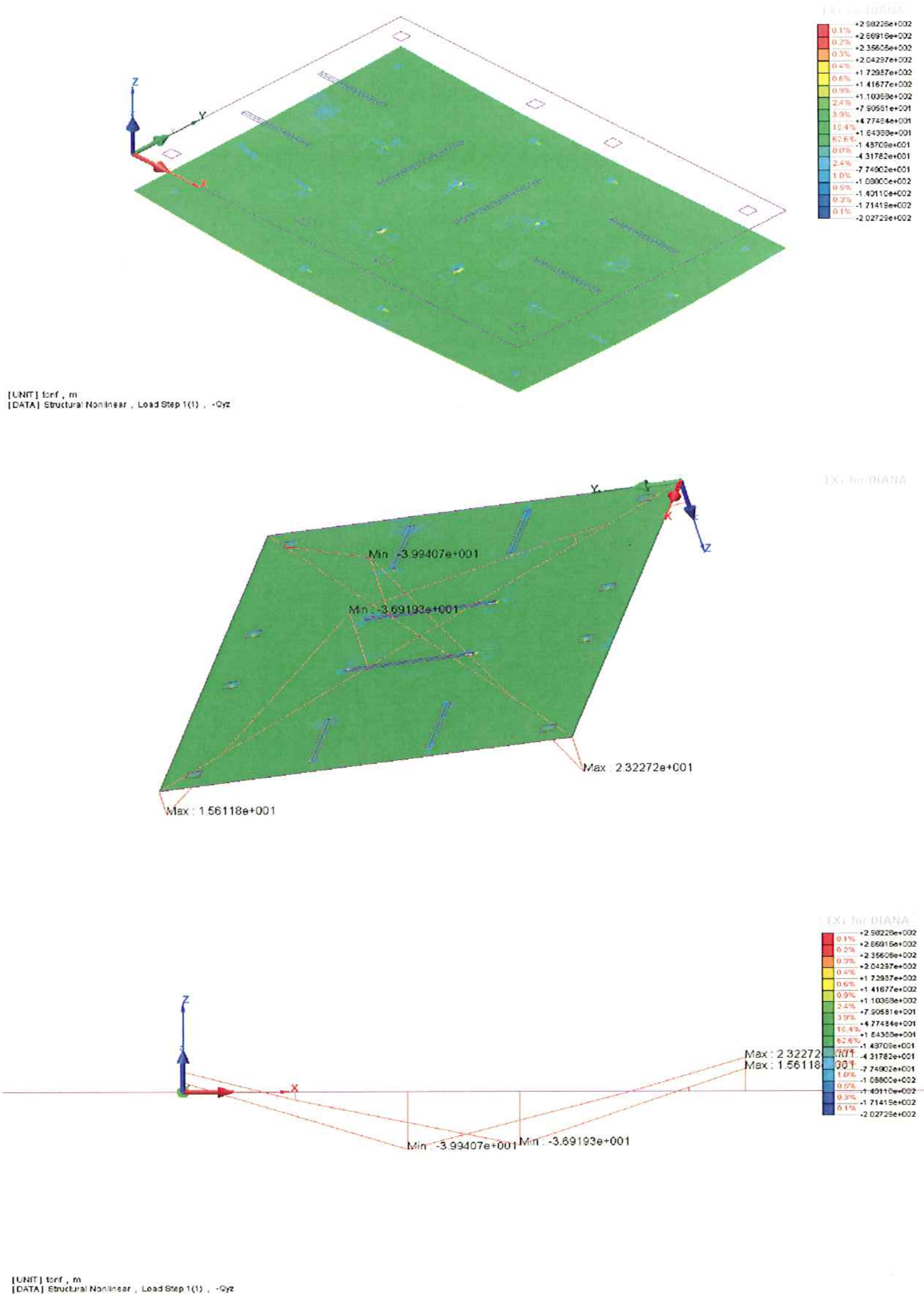


Figure 5.7 Shear forces at yz direction

5.4.2.1 Shear forces at zx direction

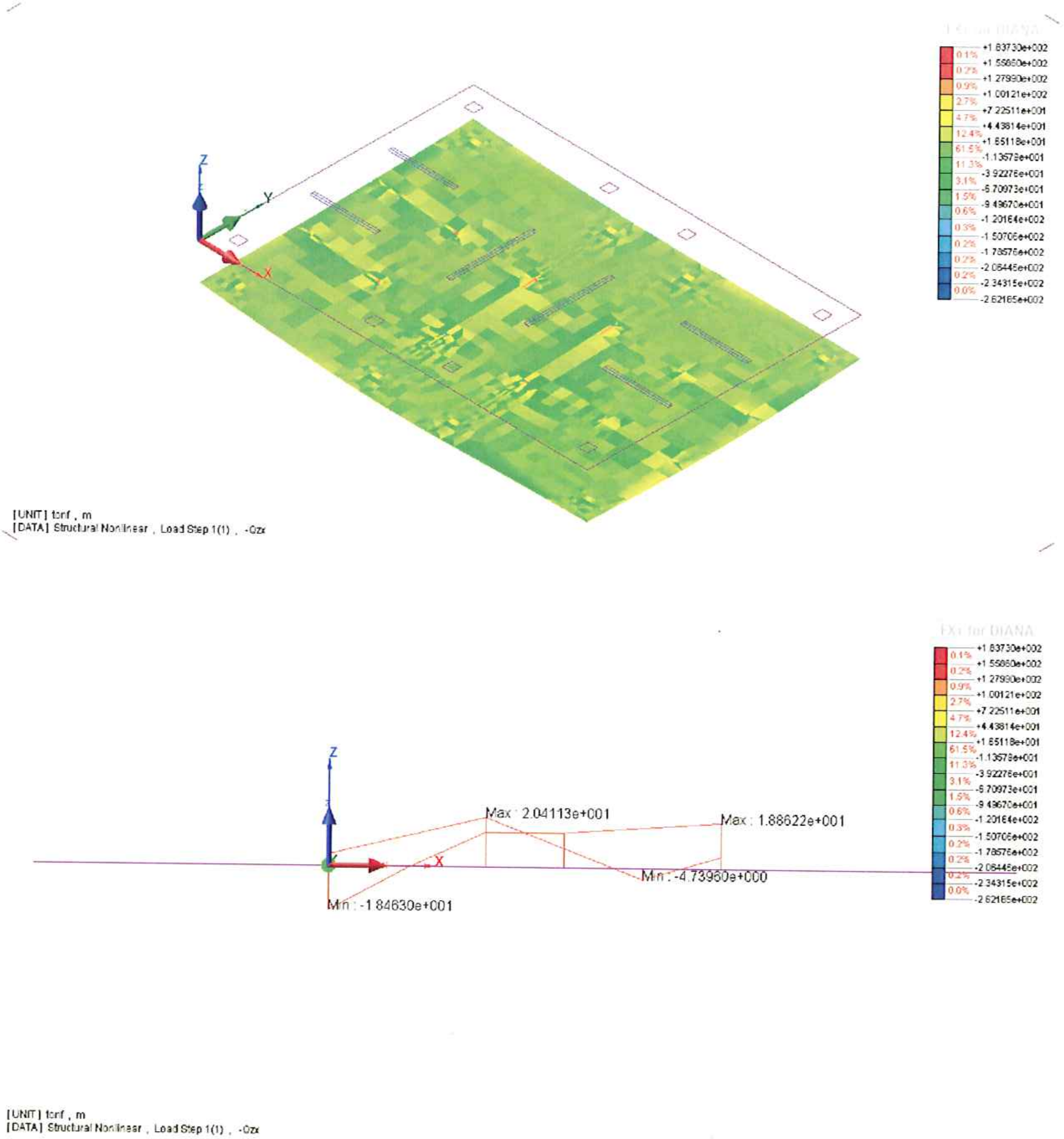


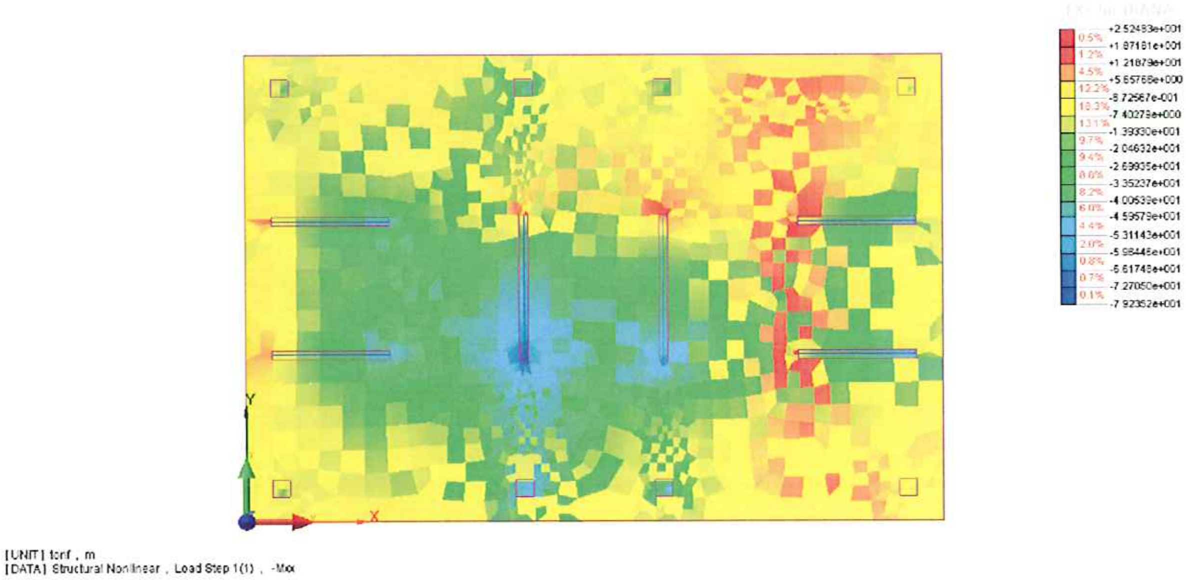
Figure 5.8 Shear forces at zx direction

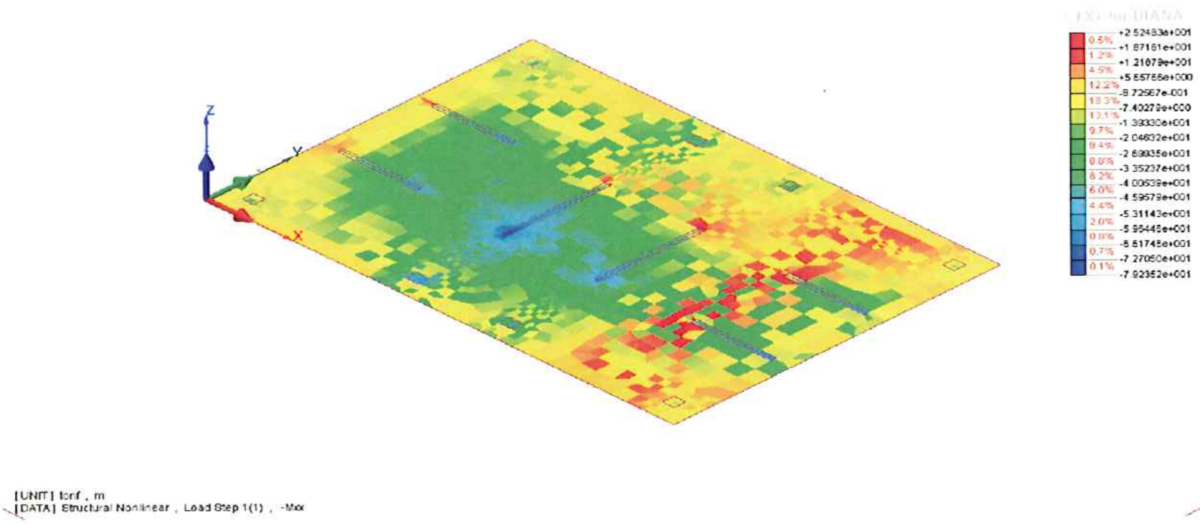
Table 5.6 Shear forces at zx and yz directions

Qzx (kN)	-184,6
	204,1
Qyz (kN)	-399,4
	232,2

5.4.3. Bending moments on the mat

5.4.3.1 Mxx moment





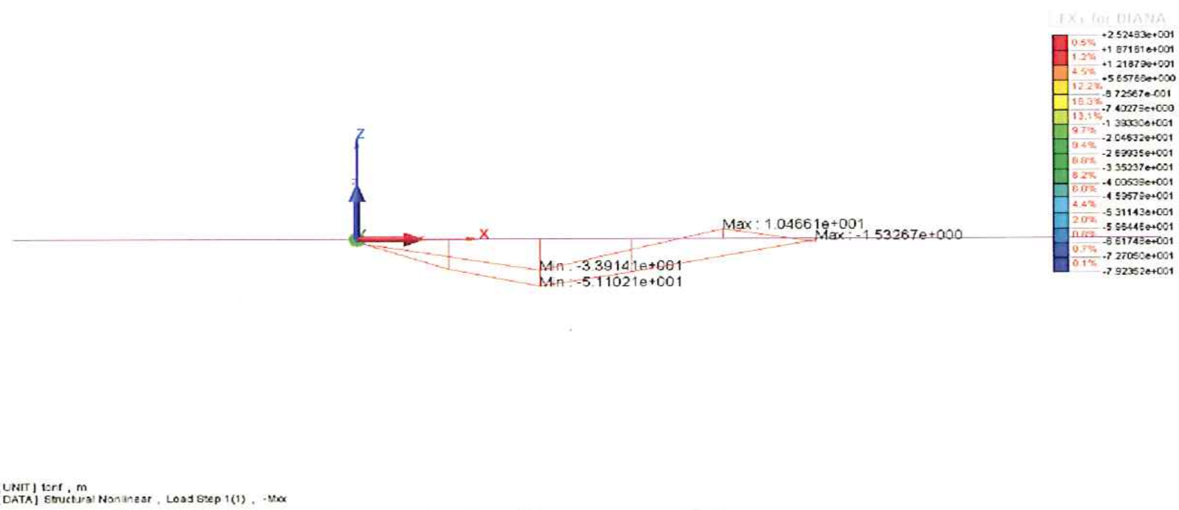


Figure 5.9 Bending moment Mxx

5.4.3.2 Myy moment

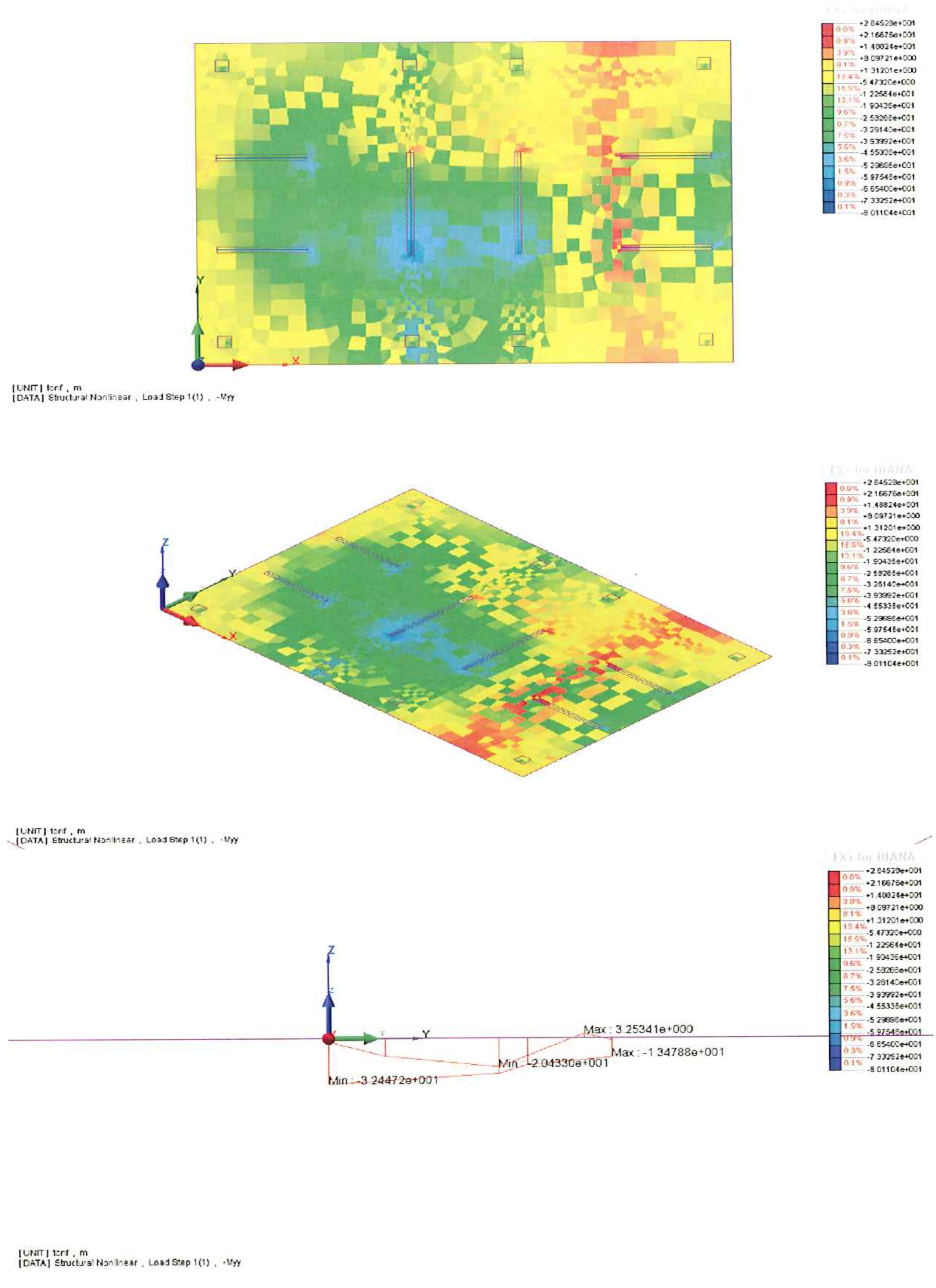


Figure 5.10 Bending moment Myy

5.4.3.3 Mxy moment

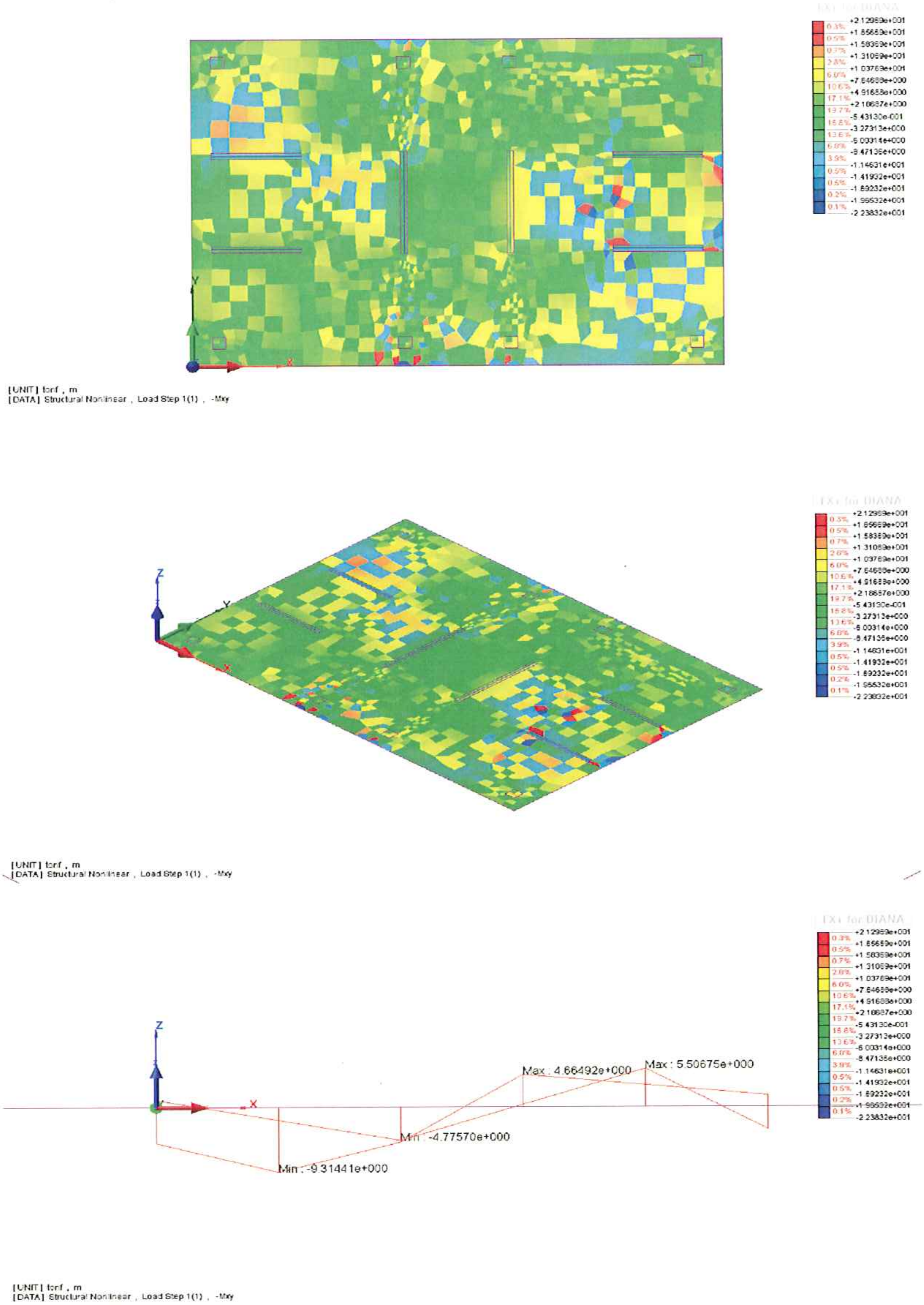


Figure 5.10 Bending moment Mxy

5.4.4. Three dimensional element stresses of mat

5.4.4.1 σ_{yz} stresses

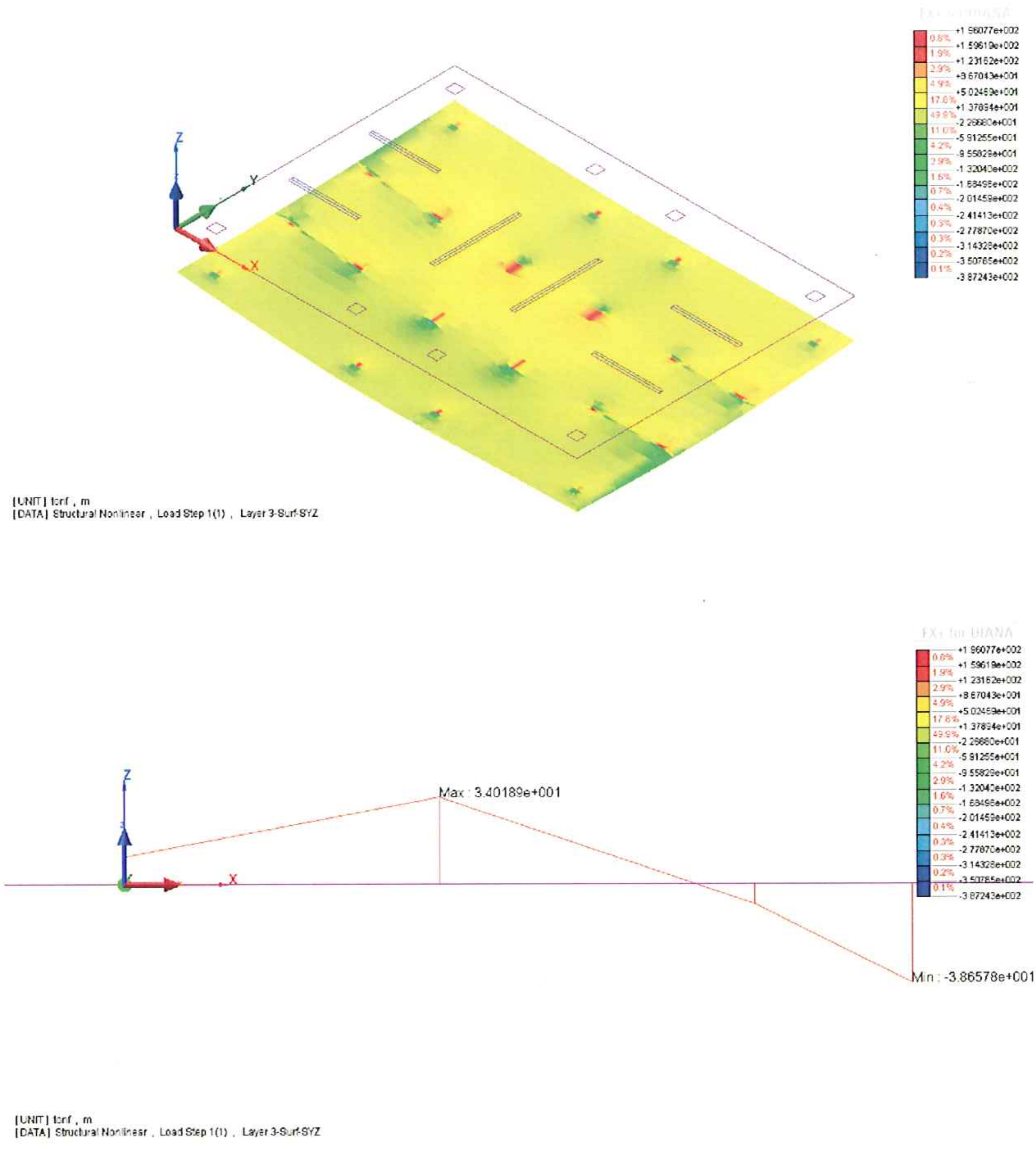


Figure 5.11 σ_{yz} stresses of mat

5.4.4.2 σ_{zx} stresses

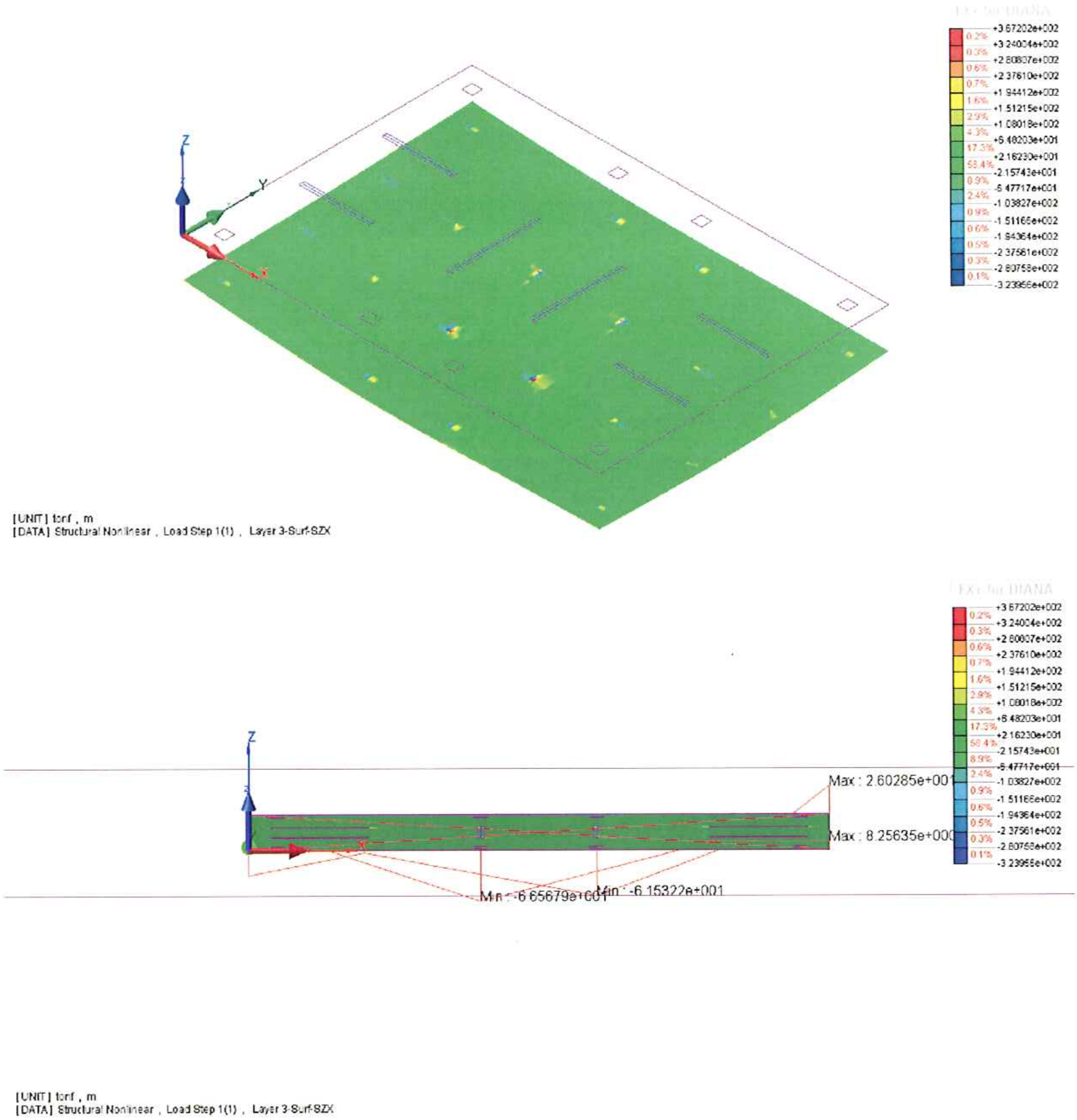


Figure 5.12 σ_{zx} stresses of mat

Table 5.7 Mat stresses at zx and yz directions

Mat σyz(kPa)	-380
	340
Mat σxz(kPa)	-665,6
	260

5.4.5. Three-dimensional soil stresses

5.4.5.1 σ_{xx} stresses

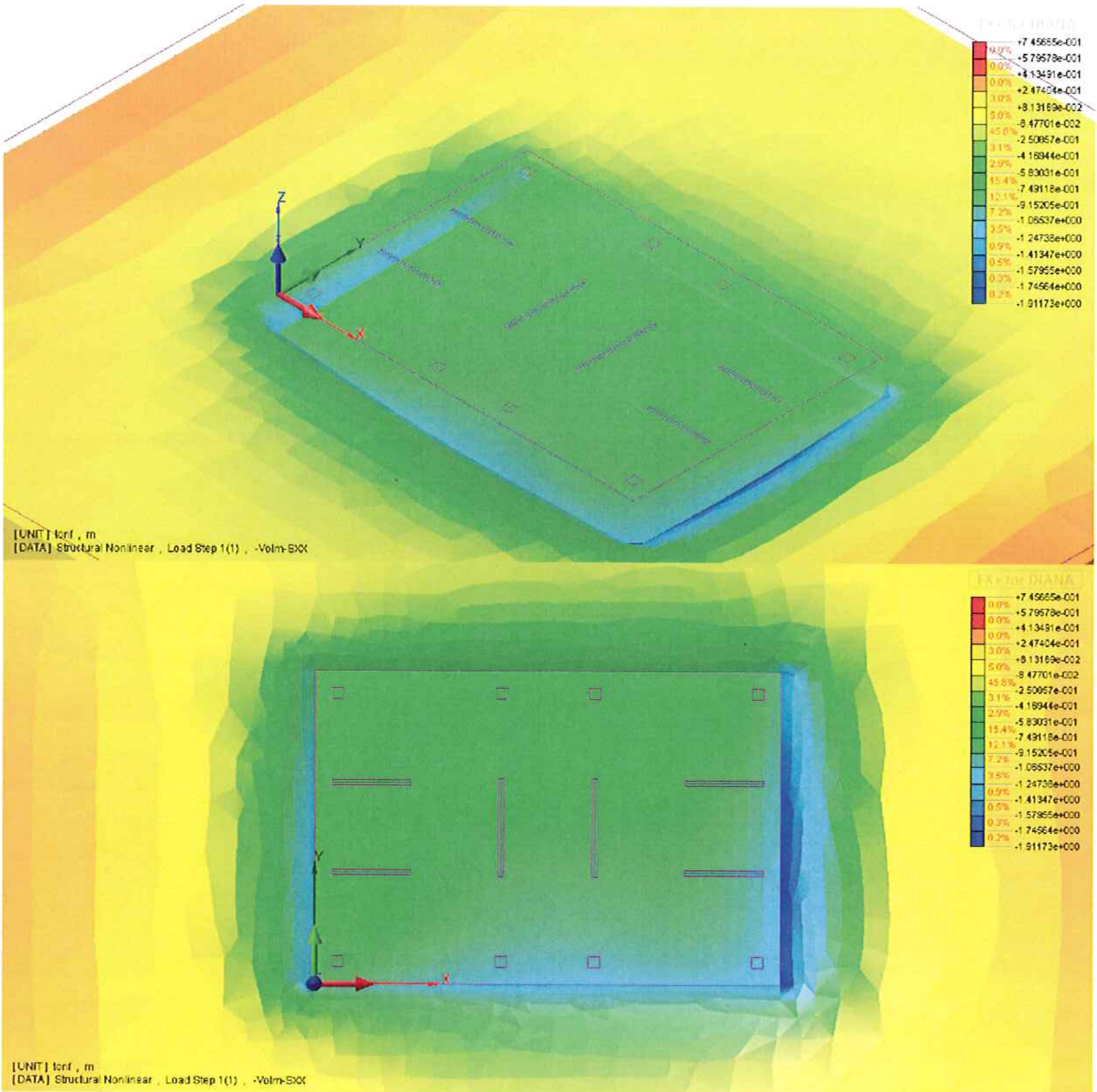


Figure 5.13 σ_{XX} stresses of soil

Table 5.8 Soil stresses at xx direction

X (m)	Y (m)	Value(kPa)
-3,03	-1.78	2.966
-0,36	0.01	14.089
3,64	2,71	8.172
11,65	8,11	7.570
15.65	10,8	7.059
19.66	13.50	9.178
22.33	15.30	3.773
23.66	16.20	1.726

5.4.5.2 σ_{yy} stresses

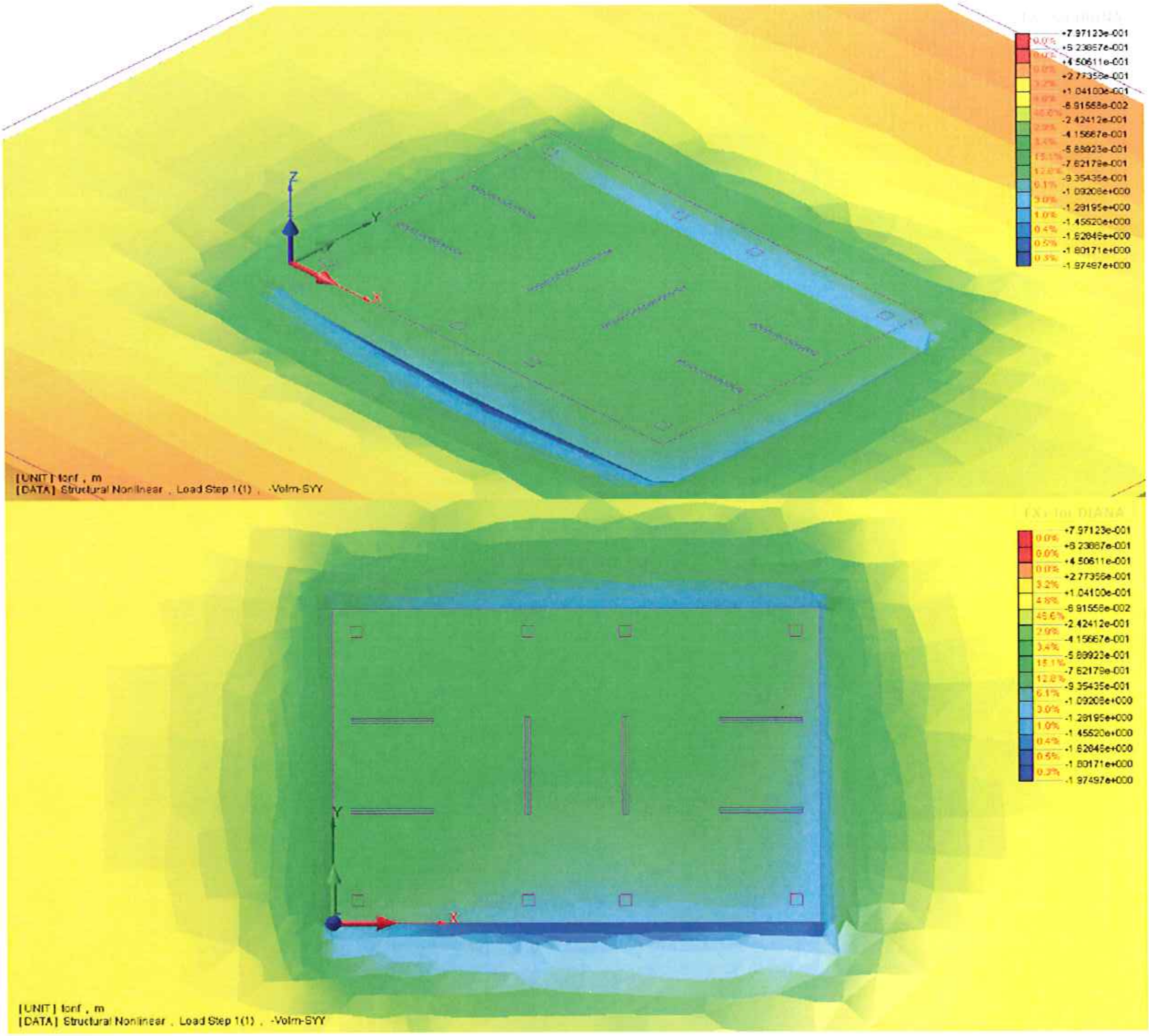


Figure 5.14 σ_{yy} stresses of soil

Table 5.9 Soil stresses at yy direction

X (m)	Y (m)	Value(kPa)
-3.56	16.30	1.320
6,81	8,83	7.161
19.77	-0.51	18.282
21,06	-1.44	8.713
22.36	-2.37	4.065

5.4.5.3 σ_{zz} stresses

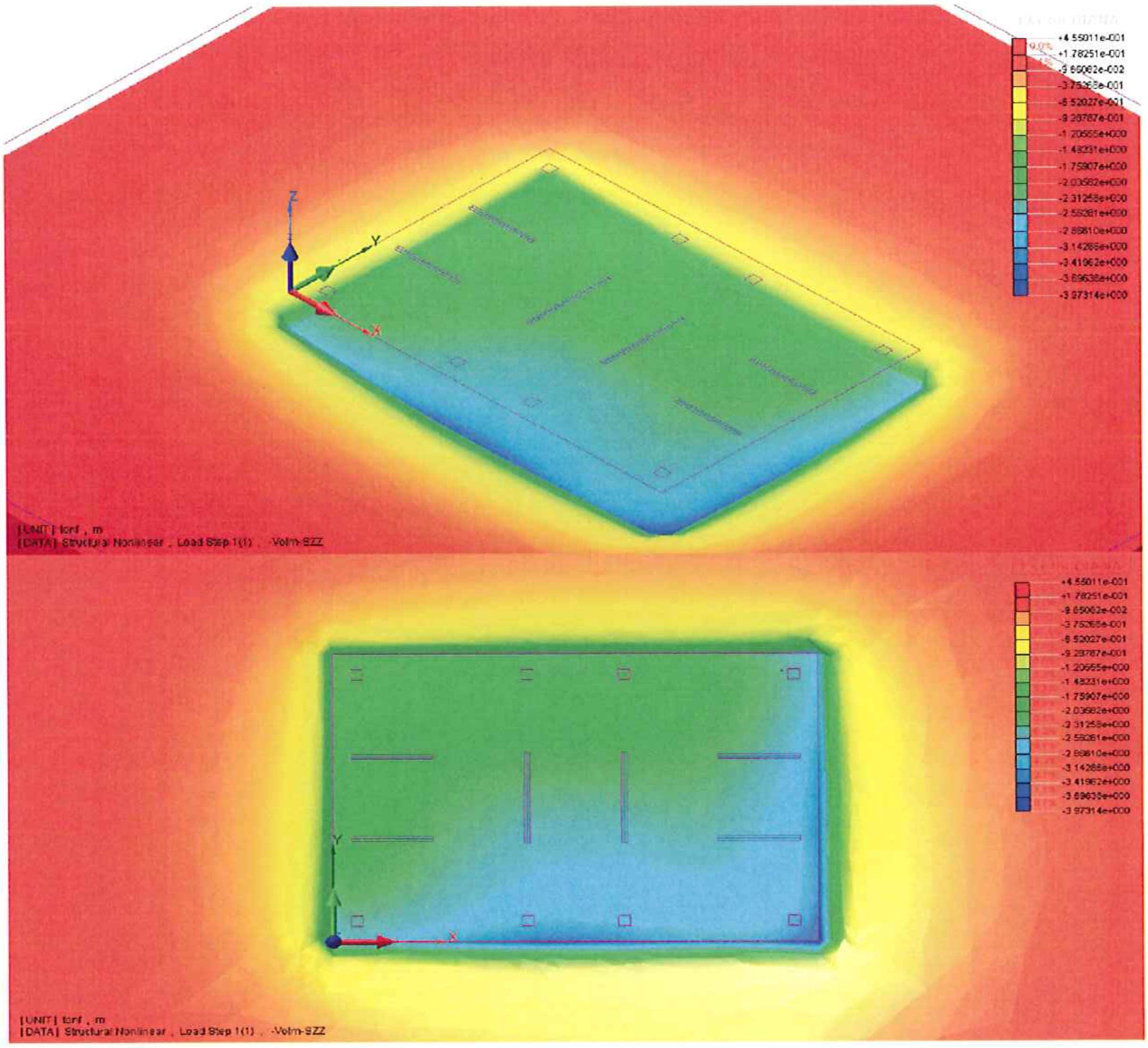


Figure 5.15 σ_{zz} stresses of soil

Table 5.10 Soil stresses at zz direction

X (m)	Y (m)	Value(kPa)
9,88	18,14	4.121
9,92	14.77	12.585
9,94	12,53	20.367
9,97	10,29	20.682
10,06	2,43	28.813
10,11	-2.06	10.647
10,12	-3.18	7.487
10,14	-4.30	5.519

5.4.5.4 σ_{yz} stresses

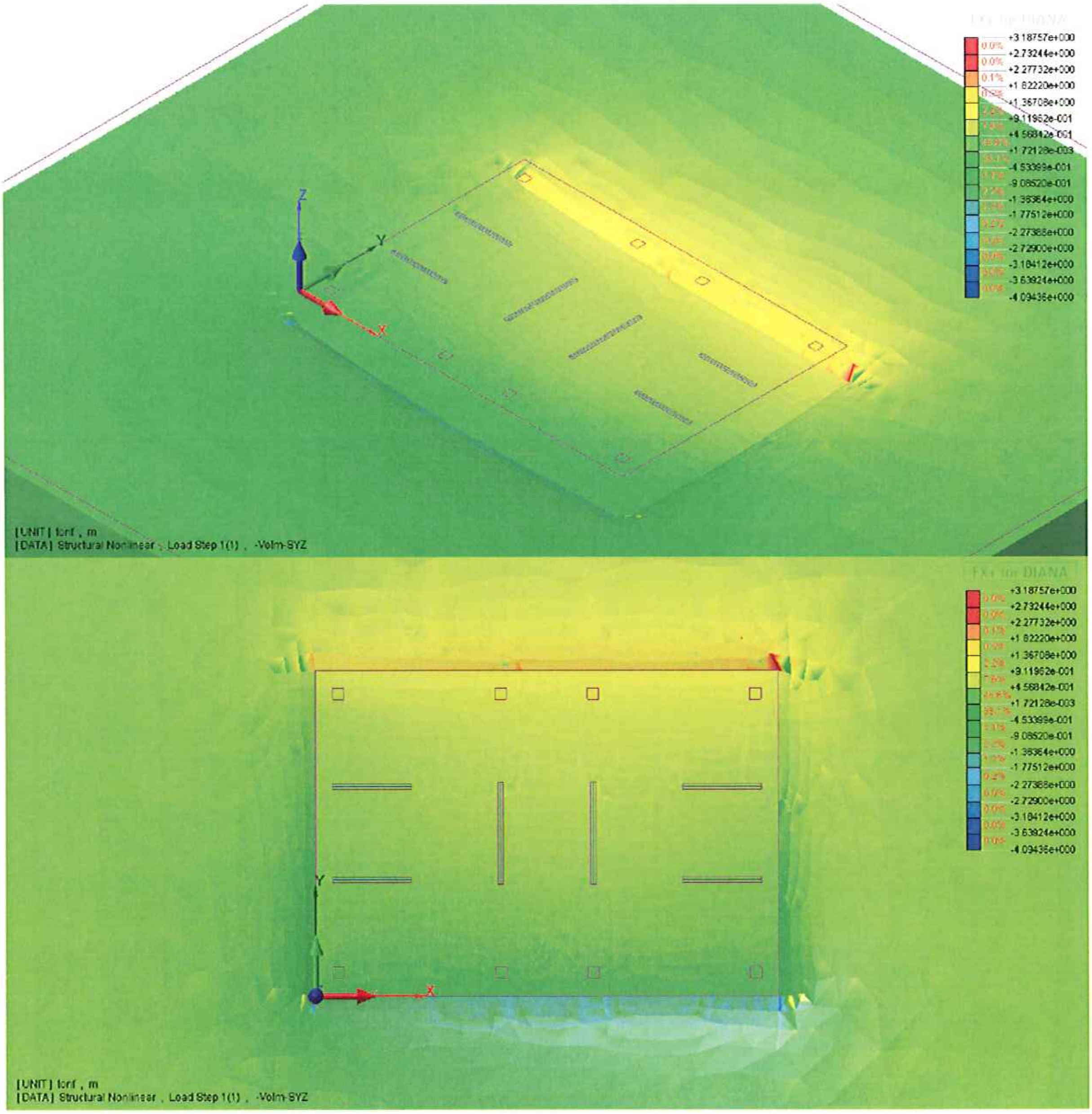


Figure 5.15 σ_{yz} stresses of soil

Table 5.11 Soil stresses at zz direction

X (m)	Y (m)	Value(kPa)
9,88	18,14	5.310
9,92	14.77	0.102
9,94	12,53	4.895
9,97	10,29	3.523
10,06	2,43	1.381
10,11	-2.06	11.427
10,12	-3.18	8.582
10,14	-4.30	6.647

5.4.5.5 σ_{zx} stresses

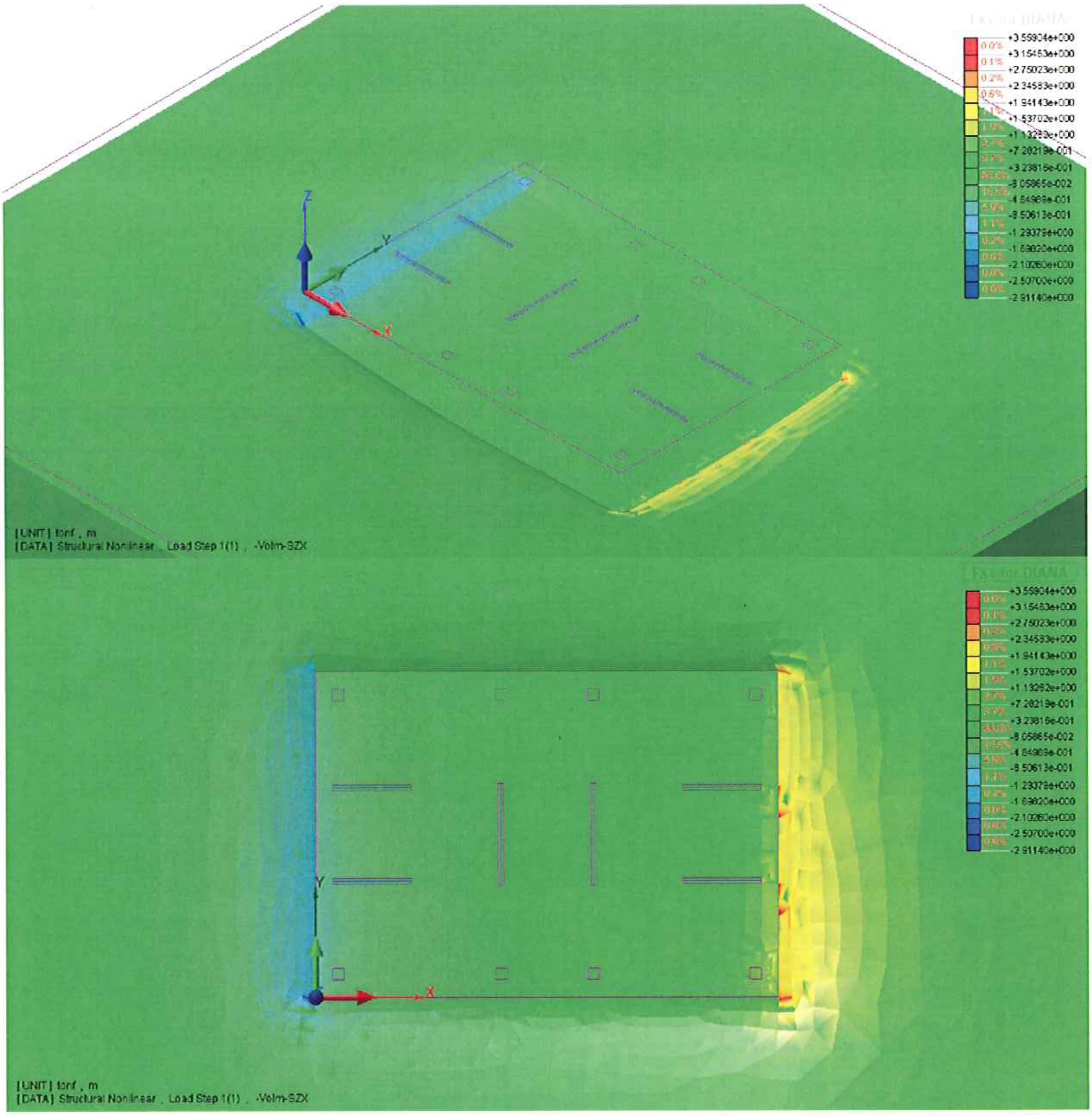


Figure 5.15 σ_{yz} stresses of soil

Table 5.12 Soil stresses at zz direction

X (m)	Y (m)	Value(kPa)
-3,03	-1.78	3.900
-0,36	0.01	25.535
3,64	2,71	4.210
11,65	8,11	0.090
15.65	10,8	0.561
19.66	13.50	3.457
22.33	15.30	3.645
23.66	16.20	2.324

5.4.5.6 σ_{equ} (Von Mises)

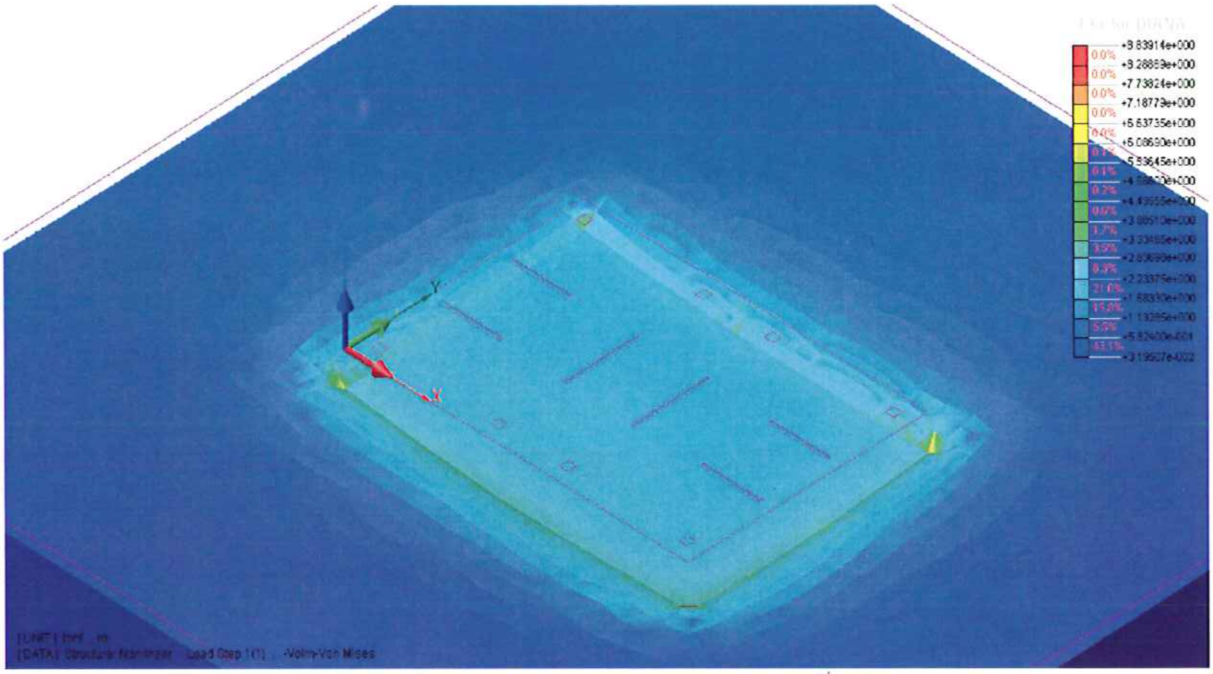


Figure 5.16 σ_{equ} stresses of soil

Table 5.13 σ_{equ} soil stresses

X (m)	Y (m)	Value(kPa)
-3,03	-1.78	7.851
-0,36	0.01	54.054
3,64	2,71	17.890
11,65	8,11	16.049
15.65	10,8	15.640
19.66	13.50	21.978
22.33	15.30	9.496
23.66	16.20	6.098

X (m)	Y (m)	Value(kPa)
-3.56	16.30	5.763
6,81	8,83	15.717
19.77	-0.51	38.994
21,06	-1.44	16.868
22.36	-2.37	8.669

5.5 ANALYSIS RESULTS

5.5.1 Analysis of the 1A-B-C Results

Analysis results of Analysis 1A,1B and 1C are given in Table 5.14

Table 5.14 Analysis 1A-B-C Results

OC CLAY	Calculated Max. Value		
Mat Thickness	0.6 cm	0.9 cm	1.2 cm
Settlement (cm)	-11	-9,6	-8,53
Mxx (kN/m)	-511	-619,8	-662,2
	104,6	73,8	58,7
Myy (kN-m)	-324,4	-449,3	-493,9
	32,5	23,4	17,4
Mxy (kN-m)	-93,1	-53,1	-55
	55	79	90,3
Qzx (kN)	-184,6	-113,6	-170,2
	204,1	220,5	221,9
Qyz (kN)	-399,4	-409,6	-401,7
	232,2	256,9	258,2
Mat σyz(kPa)	-380	-284,8	-214,7
	340	238,9	189,6
Mat σxz(kPa)	-665,6	-445,5	-331,8
	260	180,3	135,2
Soil σxx(kPa)	14	14,9	15,2
Soil σyy(kPa)	18	19,4	19,8
Soil σzz(kPa)	28	30,6	31,7
Soil σyz(kPa)	11,4	22	22,9
Soil σzx(kPa)	25,5	30,3	32
Soil σequ(kPa)	54	64,9	68,8

5.5.2 Analysis 2A-B-C Results

Analysis results of Analysis 2A,2B and 2C are given at table 5.15

Table 5.15 Analysis 2A-B-C Result

NL CLAY	Calculated Max. Value		
Mat Thickness	**0.6 cm**	**0.9 cm**	**1.2 cm**
Settlement (cm)	-26,81	-27,9	-28,22
Mxx (kN-m)	-598,7	-664,3	-684,6
	80,9	58,3	50,49
Myy (kN-m)	-474,1	-502,2	-523,8
	26,5	18,2	14,6
Mxy (kN-m)	-106,4	-54,66	-55,7
	72,9	87,8	94,5
Qzx (kN)	-218,3	-199,5	-111,4
	214,2	195,5	228,4
Qyz (kN)	-401,6	-402,6	-402,7
	274,4	273,5	265,7
Mat σ_{yz}(kPa)	-4568	-303,1	-220,7
	3506	247,4	190,4
Mat σ_{xz}(kPa)	-669,4	-447,4	-335,6
	91,9	191,8	139,4
Soil σ_{xx}(kPa)	14,8	15,3	15,5
Soil σ_{yy}(kPa)	19,2	19,8	20
Soil σ_{zz}(kPa)	30	31,7	32
Soil σ_{yz}(kPa)	21,6	23	23
Soil σ_{zx}(kPa)	29	32	32
Soil σ_{equ}(kPa)	62,9	69	70,8

5.5.3 Analysis1A-B-C settlements results

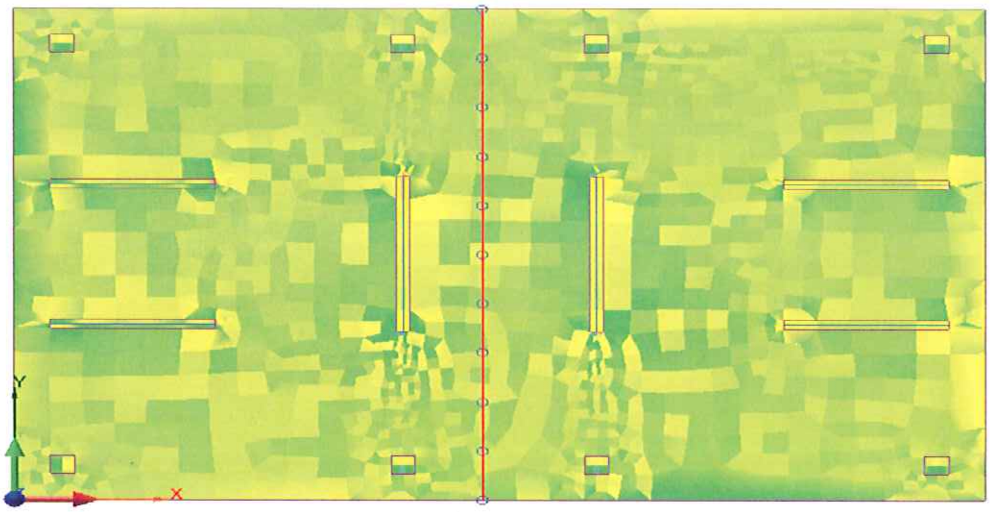


Figure 5.17a Settlement direction of raft

Table 5.16 Analysis 1A-B-C Result

OC CLAY		Settlement values (cm)		
X (m)	Y (m)	**0.6 m**	**0.9 m**	**1.2 m**
9,66	14.00	-8.34	-7.25	-6.41
9,66	12,60	-8.72	-7.60	-6.73
9,66	11,20	-9.11	-7.95	-7.06
9,66	9,80	-9.51	-8.32	-7.40
9,66	8,40	-9.94	-8.72	-7.77
9,66	7.00	-10.34	-9.09	-8.12
9,66	5,60	-10.65	-9.37	-8.37
9,66	4,20	-10.84	-9.53	-8.51
9,66	2,80	-10.92	-9.58	8.54
9,66	1,40	-10.96	-9.59	-8.53
9,66	0.00	-11.01	-9.61	-8.52

5.5.4 Analysis2A-B-C settlements results

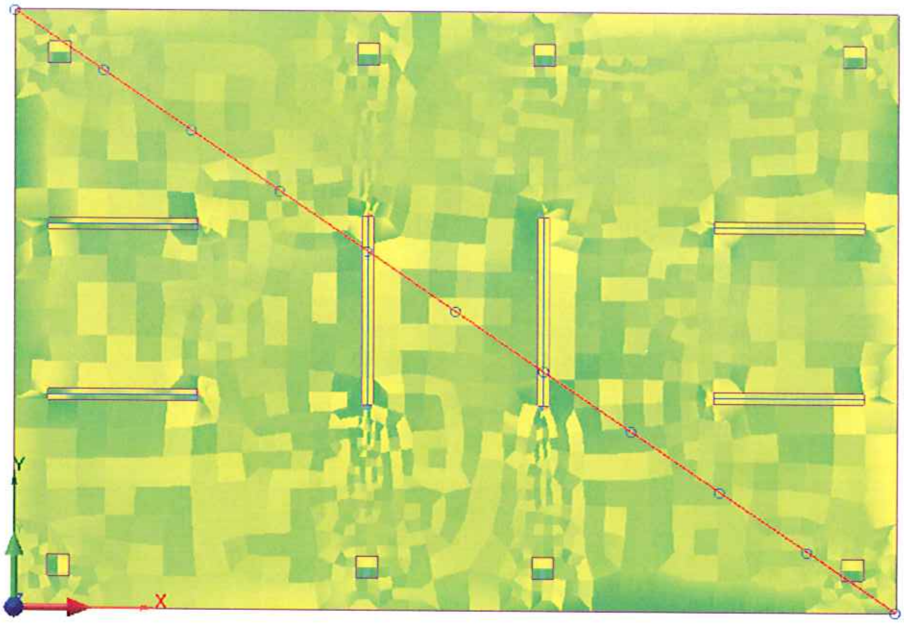


Figure 5.17b Settlement direction of raft

Table 5.17 Analysis 2A-B-C Result

NL CLAY		Settlement values (cm)		
X (m)	Y (m)	**0.6 m**	**0.9 m**	**1.2 m**
0.00	14.00	-16.97	-17.58	-17.77
2.00	12.60	-18.67	-18.86	-18.92
4.00	11.20	-20.35	-20.13	-20.08
6.00	9.80	-21.92	-21.37	-21.21
8.00	8.40	-23.34	-22.55	-22.32
10.00	7.00	-24.55	-23.66	-23.40
12.00	5.60	-25.48	-24.68	-24.44
14.00	4.20	-26.03	-25.56	-25.42
16.00	2.80	-26.39	-26.38	-26.37
18.00	1.40	-26.65	-27.16	-27.31
20.00	0.00	-26.82	-27.90	-28.22

PART 6

CONCLUSIONS AND RECOMMENDATIONS FOR FURTHER STUDY

The purpose of this study was designated as the influence of the superstructure and the foundations on the stresses and displacements in the soil, with special reference to the influence of the shear walls, under static and seismic conditions. This aim was partly achieved due to the daunting volume of work that appeared subsequently.

The results of the computations of raft settlements due to varying raft stiffnesses are presented in this thesis.

Rafts of 0.6, 0.9 and 1.2m thickness were placed on two soil types : overconsolidated and normally consolidated clay.

The following table summarises the results on overconsolidated clay

OC CLAY	Mat Thickness (t)		
	0.6 m	0.9 m	1.2 m
Settlement Δ (cm)	-11	-9,6	-8,52
Differential settlement δ (cm)	2.67	2.36	2.11
M_{xx} (kN-m)	-511	-619,8	-662,2
	104,6	73,8	58,7
Soil σ (kPa)	54	64,9	68,8
Mat σ_{xz} (kPa)	-665,6	-445,5	-331,8
	260	180,3	135,2

Table 6.1 Analysis 1A-B-C Result

The results that can be derived from the analyses are

- Increasing slab thickness reduces total settlements by up to 20%;
- The moments in the slab increase at the top and decrease at bottom as t increases;

- The stresses in the foundation slab however hane decreased;
- The contact stresses appear to have unchanged
- The differential settlements checked diagonally seemed to have remained unchanged

When the normally consolidated soil was subjected to the same static and dynamic loads the results in the following table were obtained

NL CLAY	Mat Thickness (t)		
	0.6 cm	**0.9 cm**	**1.2 cm**
Settlement Δ(cm)	-26,81	-27,90	-28,22
Differential settlement δ (cm)	9.85	10.32	10.45
M_{xx} (kNm)	-598,7	-664,3	-684,6
	80,9	58,3	50,49
Soil σ (kPa)	62,9	69	70,8
Mat σ_{xz}(kPa)	-669,4	-447,4	-335,6
	91,9	191,8	139,4

Table 6.2 Analysis 2A-B-C Result

The following conclusions can be drawn from those results:

- There is a slight increase in the total settlements as slab thickness increases
- The contact stresses remain unchanged despite increases in the mass of the raft,due to increases in t
- The moments surprisingly increase as t increases. This may be due to predominant effect of the concrete
- Conversely, the stresses in the mat diminish markedly as thickness increased.

TNO DIANA has been found to be a very detailed and complicated software during its first use in Turkey. The volume of work to be performed proved to be overwhelming under this heading. It is recommended that the future work with this software be handled separartely for the superstructure and a varying soil profile. The incorporation of shear walls is another interesting study to be undertaken

REFERENCES

1- U.S. Army Corps of Engineers, Engineers and Design, Settlement Analysis, 1990, Washington

2- U.S. Army Corps of Engineers, Engineers and Design, Geotechnical Analysis by Finite Element Method, Washington

3- Das Braja M, Principles of Foundation Enginering, 1995, Pws Publishing company

4- H.G.Poulos,Behaviour of Stiffened Raft Foundation, 7th ANZ conference on Geomechanics Conference,1996, Australia

5- H.G.Poulos, Common Procedures for Foundation Settlement Analysis,8th ANZ conference on Geomechanics,1999, Australia

6- Hsai-Yang Fang, Foundation Engineering Handbook,1990, New York

7- U.S. Navy "Foundations and Earth Structures",1982, Alexandria

8- Teng,W.C.Foundation Design,1962,Prentice –Hall,Inc Englewood Cliffs, New Jersey

9- A.Önalp- S.Sert, Geoteknik Bilgisi 3 Bina Temelleri, 2006, İstanbul

10- DIANA Software User's Manual, 2008, TNO DIANA BV, Delft, The Netherlands.