Soil Settlement Analysis from Field Monitoring Data and its Simulation at Kanchpur Bridge

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A THESIS SUBMITTED

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PROJECT REPORT APPROVAL

The title of the thesis is “Soil Settlement Analysis from Field Monitoring Data and its Simulation at Kanchpur Bridge” and the explanatory title would be Soil settlement analysis in correspondence to field monitoring values and simulation on PLAXIS 2D- Hardening Soil Model and Soft Soil Model, and ground improvement using Geogrids by Saima Sadia, Mayesha Farzana Islam and Rafia Nusrat Khan Broti Student No. 160051035, 160051053 and 160051061 respectively, have been found as satisfactory and accepted as partial fulfilment of the requirement for the Degree Bachelor of Science in Civil engineering.

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DECLARATION OF CANDIDATE

We hereby declare that the undergraduate research work reported in this thesis has been performed by us under the supervision of Professor Dr. Hossain Md. Shahin and this work has not been submitted elsewhere for any purpose (except for publication).

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DEDICATION

We dedicate our thesis work to our family and some special friends who always kept up with us and continuously helped us whenever the support was needed. A special feeling of gratitude to our loving parents and our respected Prof. Dr. Hossain MD. Shahin.
It is a small token of appreciation towards all those who supported us throughout our endeavour and encouraged us to continue our work until the end. We also dedicate our thesis work in the benefit of human kind.

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"In the name of Allah, Most Gracious, Most Merciful."

All the praises to Almighty Allah Subhanahu wa'tala for blessing us with the opportunity and always being kind.

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ABSTRACT

Keywords: *Embankment, Settlement, Soft Soil, Soil Parameters, Finite Element Method, PLAXIS-2D, Numerical Analysis, Hardening Soil Model, Soft Soil Model, Geogrid.*

The study and measurement of the settlement is a concern for any Geotech engineer. For the serviceability of subgrade projects, the prediction of embankment settlement is a critically important issue. We have to calculate settlement especially differential settlement to ensure the design grade of embankment. This study includes the project Embankment Settlement of Kanchpur bridge. The embankment should be placed considering and risk associated with surcharge loads.

In this thesis, finite element analysis using PLAXIS-2D has been used for the analysis of finite elements. Hardening Soil Model and Soft Soil Model have been used in this simulation as a constitutive model. Various field tests have been done by ProSoil from where soil parameters have been determined. Using these parameters, settlement of embankment in soft soil has been estimated. Geogrid has been used for the improvement of the soil along with the determination of the settlement through conventional method.

Table of Contents

List of Tables

List of Figures

CHAPTER 1: Introduction

1.1 General

Settlements usually refer to the displacement of the soil in the vertical direction caused by shifts in stress. Because of volumetric transition, the consolidation settlement is triggered. Consolidation settlements are caused by a reduction in voids due to the gradual squeezing out of water, because soil particles are essentially incompressible. Depending on the soil's permeability and water drainage routes, the settlement process may be completed almost immediately or may continue for a considerable period of time (even decades). Cohesionless soils, in particular, have greater permeability than cohesive soils that have tiny voids that impede the passage of water.

Soft soils, some of which are found in significant cities, are common throughout the world. Because of low shear strength consistency, civil engineering constructions in soft soil, primarily soft clay deposits, are limited by their propensity to perform excessive settlement. Due to the wide void ratio and inherent compressibility of clays, consolidation and displacements can be visible under construction loads, which can be time consuming and frustrating for the structural engineer. The low shear strength and high compressibility of these soft clays have challenged the geotechnical design engineer's wit in solving problems related to the state of stability during construction and to the residual settlement during service, including differential settlement.

The subject of clay consolidation was partly chosen because it was the first major contribution by Terzaghi to what we now call geotechnical engineering, as well as because of the years of interest in the topic. This manuscript examines functional aspects of the forecasting of time-dependent settlement of soft clay embankments. The issues discussed include site exploration; solid field data modelling; problems associated with taking, treating, and analysing soil samples; efficient laboratory data reduction, storage, and display; and analytical methods to deal with such routine problems as consolidation coefficients based on efficient stress, large strains, nonlinear stress-strain curves, multidirectional flow.

Within the context of the conventional principles of soil mechanics, highly practical study of the consolidation of soft clays is possible. For the prediction of the magnitude and rate of settlement and pore water pressure dissipation, a practical method is defined. It provides a thorough interpretation and assessment of the consolidation properties of soft clays, including compressibility parameters, pre-consolidation pressure and permeability coefficient.

For numerical analysis of this thesis, Finite Element Method, Computer Program FEM-t_{ij} 2D is being used along with the comparison of the analysed data. Divergence should be made between drained and undrained strength of cohesive materials. Drained condition refers to the condition where drainage is allowed whereas undrained condition refers to the condition where drainage is restricted.

After collecting the settlement monitoring data, we used the soil characteristics and preloading values to carry out conventional analysis and software based analysis to see whether the results match or vary in three different methods. It is found that the values of the settlement almost match. We compared the results of settlement data in FEM analysis, conventional method and settlement monitoring data from the field and found the values obtained from the software analysis are more accurate and dependable in accordance to the real field data.

1.2 Objectives of the Study

Our main objectives are-

1. To compare the existing conventional theory with simulation of Plaxis 2D in two models- Soft Soil Model and Hardening Soil Model.
2. To estimate settlement of Soft Soil Model through PLAXIS 2D simulation.
3. Determining the suitable model for clay and silty clay soil between soft soil model and hardening soil model.
4. To compare the ground improvement by measuring settlement for “with geogrid” and “without geogrid” using PLAXIS 2D.
5. Determining the factor of safety improvement for “with geogrid” and “without geogrid” using soft soil model in PLAXIS 2D.

1.3 Scopes of the Study

1. To determine suitability of hardening soil model and soft soil model of PLAXIS 2D
2. PLAXIS 2D stands as a good means of simulation to design for clay type embankment.
3. Field analysis can be improved after using PLAXIS 2D simulation by matching some appropriate parameters.
4. We determined the suitability of geogrid as a ground improvement tool.
5. The subgrade structures like embankment's design and construction can be highly affected in economic aspects if we can predict the accurate value of settlement.
6. We can also consider which technology should be adopted for this region like PVD, geo-textile, geo-fibre and other modern technologies.

1.4 Background

Embankment is one of the significant parts of geotechnical engineering as it requires a proper understanding of the capacity of the underlying soils to bear the imposed loads, the necessary materials available and the guarantee of stability after construction. This usually requires cautious site investigation, sincere monitoring, sampling, testing, modelling, evaluation of potential construction materials, and stability analyses. Numerical modelling is much needed in order to understand the details of stress-strain-deformation behavior at each points of base ground.

CHAPTER 2: Literature Review

2.1 General

In this section, literature review has been done for identifying the studies which have been done so far.

The analysis on different papers related to embankment settlement and use of geosynthetic are conducted below:

A. S. Balasubramaniam et. al. (2010) explained about the difficulties in forecasting highway embankment settlements and reclamation works in deposits of the coastal, deltaic and estuarine type. Even after a century of innovations and contributions, the focus is on practical aspects and the difficulties faced in confidently estimating settlements.

Paulo J.Venda Oliveira et. at. (2011) stated that using a coupled soil-water mixture, the behaviour of an embankment constructed on normally consolidated soft soil strengthened with deep mixing columns is studied. In terms of settlements, increments in vertical effective stresses and excess pore pressures, the numerical predictions are evaluated.

Zhen et. al. (2019) measures the settlement of SDM column-supported embankment over soft soil, this paper established a theoretical solution. The total settlement of the SDM column-reinforced soft soil consisted of three components based on the unit cell principle, i.e., soil compression within the length of the stiffened core pile, soil compression from the core pile base to the base of the SDM column, and soil compression below the base of the SDM column. In the study, the upward and downward penetrations of the stiffened core pile were considered.

Werner W Muller & Fokke Saaathoff (2015) explained all the geotextiles materials including how overall they will benefit in terms of coastal and hydraulic area and discussed some case studies, and reasons.

M. siavoshnia et. al. (2010) conducted analysis on embankment settlement on soft clay using Geotextile Reinforcement. It is determined that the effect of number of geotextile layers, slope inclination, geotextile modulus and geotextile effective length on the behaviour of reinforced silty and sand embankment on soft clay.

Dov et. al. (2014) conducted an analysis includes limit analysis which has been discussed in this paper and sum up that which of the parameters are to be considered in which situation, that has been presented with different cases and overview of field condition consideration.

Hamed et. al. (2012) conducted their research on the role of using Geosynthetics, history of Geotextiles, Comparisons between Biodegradable and Non-Biodegradable Geotextile. It also includes the factors which attributes towards the selection of geotextile.

Anand et. al. (2019) conducted the analysis of field monitoring data and outlines initial design and construction details, along with a focus on early performance details, of the restoration work carried out on the embankment system.

2.2 Summary

By doing literature view and studying different research papers we came to know that the settlement analysis from field monitoring data in comparison with PLAXIS 2D simulation has been conducted in many countries. This software based works are rarely found in Bangladesh. In the analysis, hardening model and soft soil model for FEM has been used in PLAXIS 2D software. In addition, parallel determination of “with” and “without” geogrid along with safety factor analysis, created a statement of importance of geogrid in soil ground improvement.

CHAPTER 3: Methodology

3.1 General

Since the study has a wide insight into a variety of aspects, different methods have been adopted to properly achieve the goal of this study. A direct approach has been set out to fulfill the scope of the study by implementing these methods. In this chapter, the methods adopted and implemented are discussed thoroughly.

3.2 Study Area

The main work of our thesis includes the prediction of the settlement of embankment in soft soil. The process for the subsoil investigation work to be carried out by the ProSoil Foundation Contractor for the 'Kanchpur, Meghna, Gumti 2nd Bridges Construction and Reconstruction of Existing Bridges (Package No. PW-01)' at Kanchpur Bridge will be summarized in this method statement.

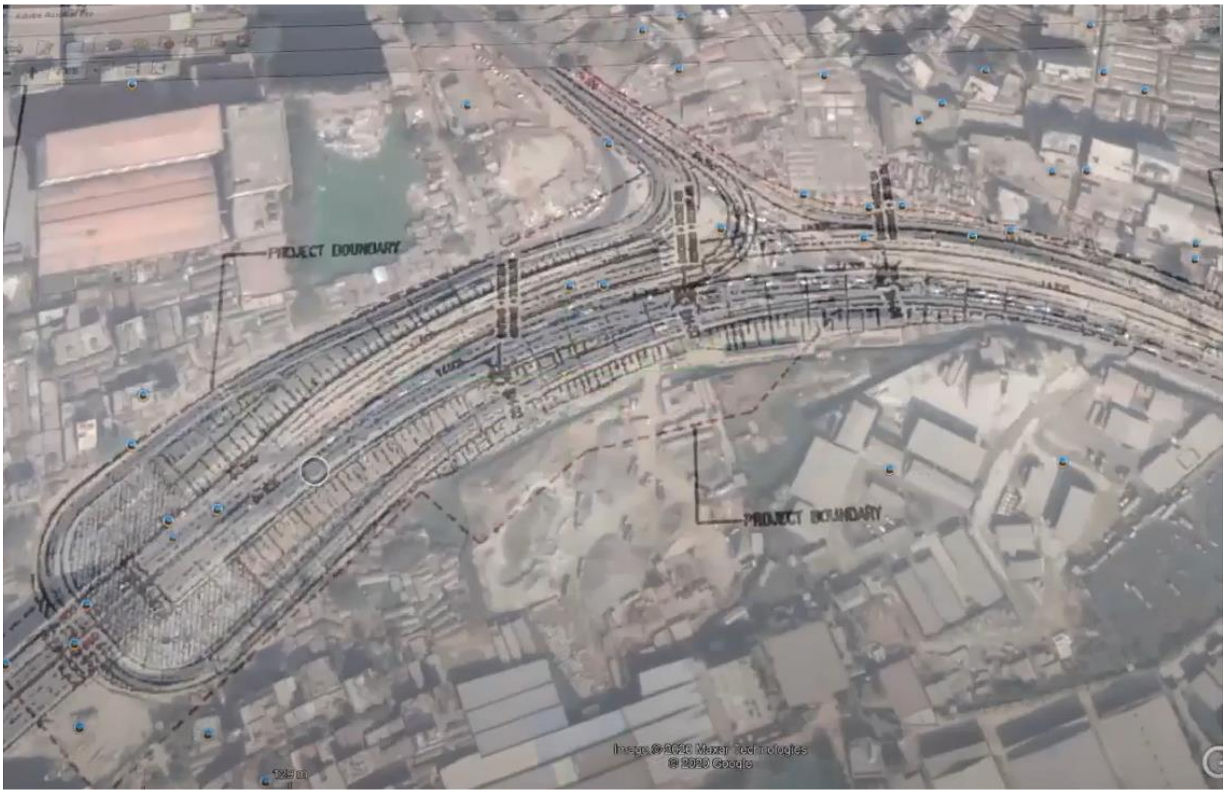


Figure 3.2 a : Clear map of Kanchpur, Meghna, Gumti 2nd Bridges Construction and Reconstruction of Existing overlay on the map

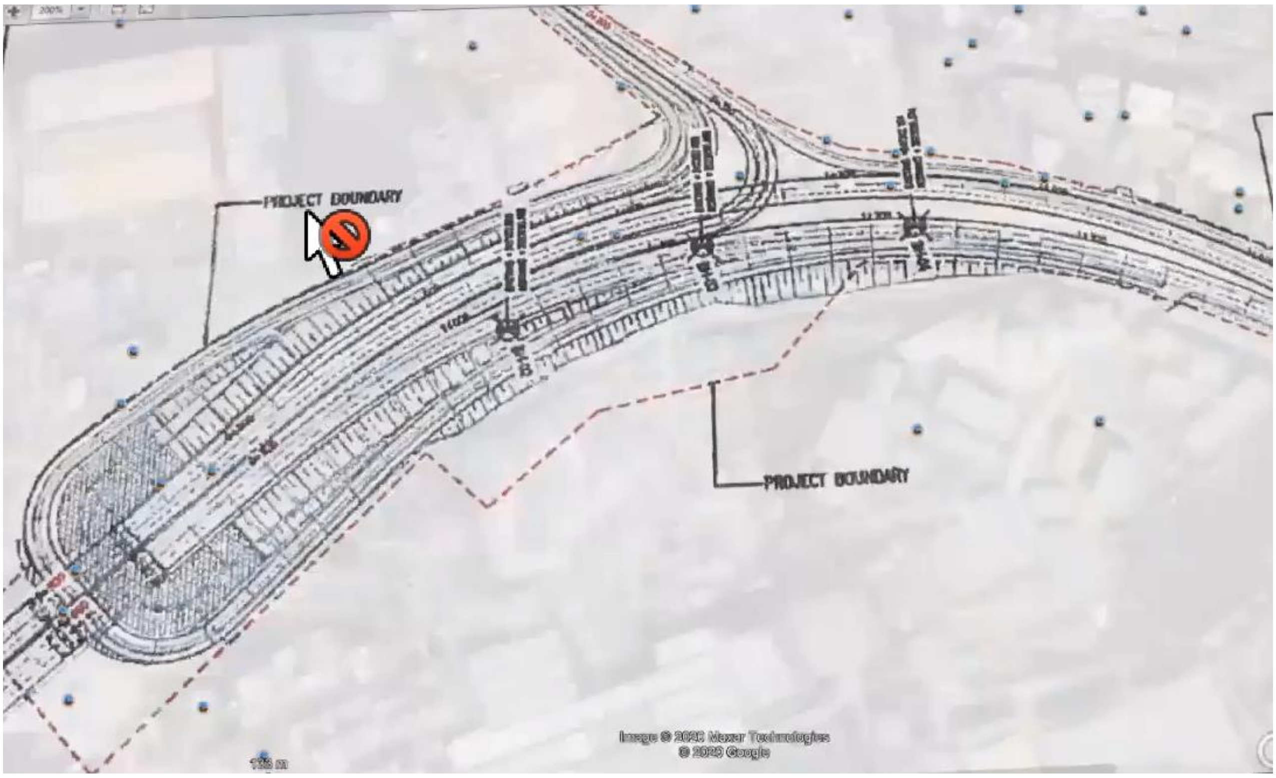


Figure 3.2 b: Kanchpur, Meghna, Gumti 2nd Bridges Construction and Reconstruction of Existing Bridges (Package No. PW-01) clear overlay map with study area

3.3 Field Investigation

The investigation program consisted of soil boring and sampling at desired intervals for subsequent observation and laboratory testing in order to economically and safely assess pile foundation capacity.

Field exploration program was conducted during the period from 13 May 2016 to 28 May 2016. The program was carried out by personnel from Prosoil Foundation Consultant who were responsible for measuring Standard Penetration Test (SPT) value and obtaining disturbed and undisturbed samples of the subsurface soils.

3.4 Methods of Field Investigation and Material Collection

The location with a diameter of 89 mm was advanced by the clayey/sandy layer boring unit. Jetting water that is pumped into the hollow drilling rods is advanced by a wash boring. Cuttings were removed by circulating water from the opening. By pulling and slackening the rope, the drilling rods are pushed up and down and are rotated back and forth by means of a driller at the same time. The water is pumped from a small swamp and the soil-laden water is discharged from the location into the same reservoir, where the coarse materials are settled and from which it is possible to secure the so-called 'wet samples.'

In soft or cohesionless soils, covers of the size of NX 89 mm are needed, but are often omitted in rigid, cohesive soils with only small representative samples as desired. Changes in soil character are determined partly by the feeling of the driller or the penetration resistance and partly by the inspection of the spoils in the water as they emerge from the casing. But only when representative samples are taken from the bottom of the locations can definitive identification of the soil be made.

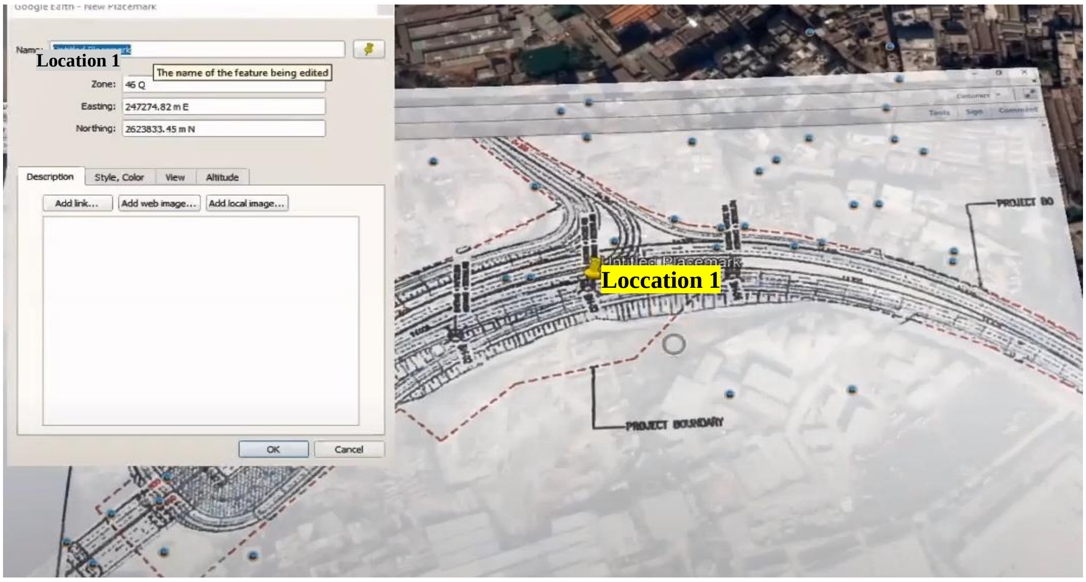


Figure 3.4. a: Kanchpur, Meghna, Gumti 2nd Bridges Construction and Reconstruction of Existing Bridges – Location 1

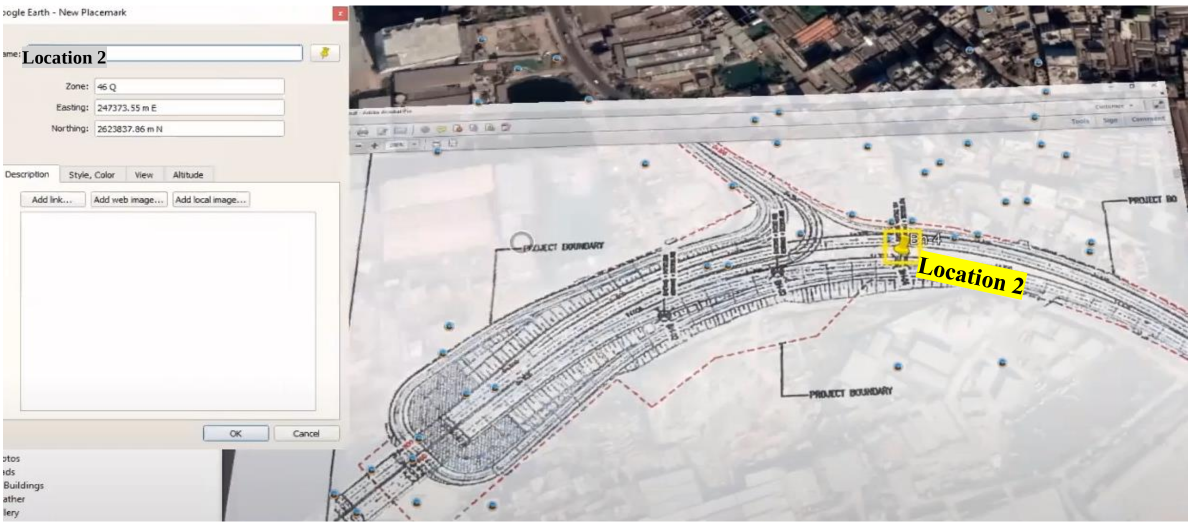


Figure 3.4. b: Kanchpur, Meghna, Gumti 2nd Bridges Construction and Reconstruction of Existing Bridges – Location 2

3.4.1 Sieve Analysis Test

Analyses of sieves is done by sieving (ASTM D 422). For oven-dry products, the particles retained in a 0.075 mm sieve were screened. The mass of soil retained on each sieve is measured and represented in sieve analysis as a percentage of the total mass of the sample. On a logarithmic scale, the particle size is plotted such that the curves of the distribution plot represent two soils having the same degree of uniformity. The study of hydrometers is based on the soil sedimentation theory. In water, the grains. The particles settle at various velocities when a soil specimen is dispersed in water, depending on their form, size, and weight.

3.4.2 Direct shear Test

The test is performed on three or four specimens from a relatively undisturbed soil sample. A specimen is placed in a shear box which has two stacked rings to hold the sample; the contact between the two rings is at approximately the mid-height of the sample. A confining stress is applied vertically to the specimen, and the upper ring is pulled laterally until the sample fails, or through a specified strain. The load applied and the strain induced is recorded at frequent intervals to determine a stress–strain curve for each confining stress.

3.4.3 Consolidation Test

Consolidation (ASTM D2435) is a mechanism by which volume decreases in soils. According to Karl von Terzaghi, "consolidation is any process which involves a decrease in water content of saturated soil without replacement of water by air." Generally speaking, it is the process in which volume reduction takes place by expelling water under long-term static loads. It occurs when stress is applied to a soil that causes the particles of the soil to pack more tightly together, thus reducing the volume of its bulk. Water will be squeezed out of the soil when this happens in a soil that is saturated with water. The magnitude of consolidation can be predicted by many different methods.

3.5 Numerical Analysis

Numerical analysis involves using approximation techniques to answer mathematical problems, taking into consideration the extent of possible errors. Although this analysis is an approximation, it is possible to produce results as accurately as desired.

In geotechnical engineering, numerical analysis is commonly used for the following:

- The simulation process is fast and simple to perform.
- The analysis is more reliable and realistic.
- Practically understanding and determining structural behaviour.
- The best analytical approach is to look at each structural behavioural step of the construction process.
- Resolve non-linear equation roots, solve large equation system.
- In this form of analysis, soil-structure interaction is adequately accounted for.
- In this study, interaction between soil and water can be modelled accurately.

3.5.1 Finite Element Method (FEM)

The finite element method (FEM) is a statistical technique used to achieve approximate solutions to boundary value issues in engineering, also referred to as finite element analysis (FEA). Simply put, a boundary value problem is a mathematical problem in which, within a specified domain of independent variables, one or more dependent variables must satisfy a differential equation everywhere and fulfil unique conditions on the boundary of the domain. The area is the domain of interest and represents a physical structure most frequently. The field variables are the interest dependent variables that the differential equation governs. Border conditions are the values defined by the field variables (or related variables, such as derivatives) on the field boundaries.

3.5.1.1 Procedure

The general techniques and terminology of finite element analysis will be introduced with reference to Figure 1.1. The figure depicts a volume of some material or materials having known physical properties. The volume represents the domain of a boundary value problem to be solved. For simplicity, at this point, we assume a two-dimensional case with a single field variable ϕ(x, y) to be determined at every point P(x, y) such that a known governing equation (or equations) is satisfied exactly at every such point.

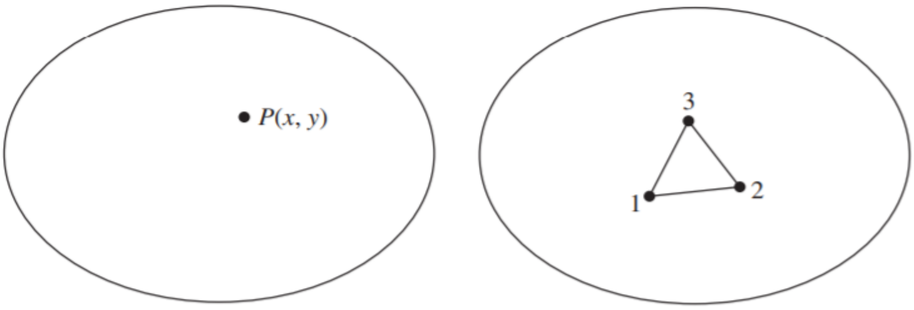


Figure 3.5. 1: A general two-dimensional domain of field variable and a three-node finite element defined in the domain

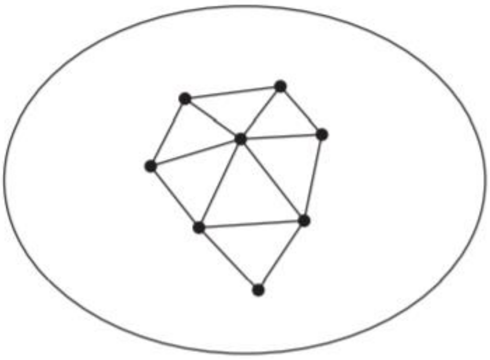


Figure 3.5.2 : Additional elements showing a partial finite element mesh of the domain

Note that this suggests a correct numerical arrangement and solution is obtained; that's, the solution may be a closed-form logarithmic expression of the free factors. In common sense issues, the space may be geometrically complex as is, regularly, the administering condition and the probability of getting a correct closed-form arrangement is exceptionally low. Hence, surmised arrangements based on numerical procedures and advanced computation are most frequently obtained in engineering investigations of complex issues. Limited component investigation could be an effective method for getting such surmised arrangements with great exactness.

Figure 1.1b shows a small triangular feature that encloses a finite-sized subdomain of the region of interest. It makes this a finite element that this element is not a differential element of size dx x dy. As we treat this example as a two-dimensional problem, it is assumed that in the differential equation, the thickness in the z direction is constant and z dependence is not indicated. To imply that these points are nodes, the vertices of a triangular element are numbered. A node is a particular point in the finite element where the value of the field variable must be determined directly. Exterior nodes are located on the finite element boundaries and can be used to connect an element to adjacent finite elements. Interior nodes are nodes that do not lie on element boundaries and cannot be attached to any other element. There are only exterior nodes in the triangular portion of Figure 1.1.b. If the values of the field variable are computed only at nodes, how are values obtained at other points within a finite element? The answer contains the crux of the finite element method: The values of the field variable computed at the nodes are used to approximate the values at nodal points (that is, in the element interior) by interpolation of the nodal values. For the three-node triangle example, the nodes are all exterior and, at any other point within the element, the field variable is described by the approximate relation

$$\phi(x, y) = N_1(x , y)\phi_1 + N_2(x , y)\phi_2 + N_3(x , y)\phi_3$$

where ϕ_1, ϕ_2, and ϕ_3 are the values of the field variable at the nodes, and N_1, N_2, and N_3 are the interpolation functions, also known as shape functions or blending functions. In the finite element approach, the nodal values of the field variable are treated as unknown constants that are to be determined. The interpolation functions are most often polynomial forms of the independent variables, derived to satisfy certain required conditions at the nodes.

3.5.1.2 Background of FEM

The mathematical origins of the method of finite elements go back at least half a century. The root of approximate techniques for solving differential equations using test solutions is much older. In order to approximate solutions of differential equations, Lord Rayleigh[9] and Ritz[12] used trial functions (in our case, interpolation functions). The same definition was used by Galerkin[5] for solutions. Compared to the current finite element method, the downside of the earlier methods is that the trial functions must extend to the whole field of the issue of concern. The finite element approach had its real beginning in the 1940s, when Courant[4] introduced the notion of piecewise-continuous functions in a subdomain. The term finite element was first used in the sense of plane stress analysis by Clough[3] in 1960 and has been in common use since that time.

3.6 Constitutive Soil Model

The physical properties of a given material are introduced or defined by the constitutive model. It binds the kinematic to kinetic motion definitions, thus closing the initial boundary value issue with the formulation. It is used as a model-based, practical simulation. It is the spectrum of fundamental material models that nowadays delimits the predictive value of large-scale simulations involving thousands of degrees of freedom. In short, it will focus on continuum-based material formulations that are commonly used for stress and deformation studies in engineering practice, science and education, as well as in commercial finite element software packages.

For modelling the stress-strain behaviour of soils, numerous constitutive models have been established over the past forty years. These models are to be used, some more robust than others, some based on experimental proof, with finite element and finite difference calculations of soil structures and soil interaction problems under axisymmetric, plane pressure, or general formulated on the basis of mechanical principles.

3.7 Relation between FEM and Constitutive Soil Model

It has become possible to analyse and forecast the behaviour of complex soil structures and soil/structure interaction issues through the advancement of numerical methods such as finite element and finite difference methods. These studies rely heavily on the representation of the relationships between stresses and strains of the different materials involved in the geotechnical structure. The relationships between stresses and strains in a given material are expressed in numerical computations by a so-called constitutive model, consisting of mathematical expressions that model the conduct of the soil in a single unit, as shown in Fig. 2.3 Because soils are the weakest materials most commonly involved in common geotechnical problems, deformations and the probability of structural failure are determined, and it is therefore important to accurately classify these materials across the whole range of stresses and strains to which they are exposed. Other building materials, such as concrete and steel, can remain rigid relative to the soil, and numerical computations can provide adequate precision under all loading conditions.

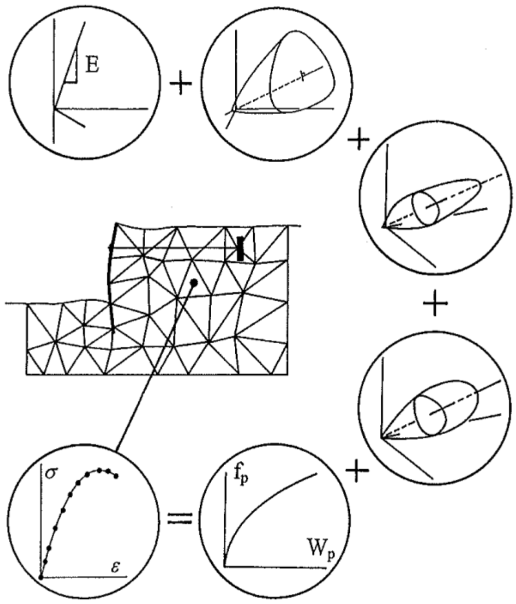


Figure 3.7. 1: Soil structure divided into a finite number of elements each of which is represented by a constitutive model based on elasticity and plasticity theories

3.8 Soil Models in PLAXIS 2D

There are various kinds of soil model in PLAXIS 2D. The name of the soil models are:

- Linear Elastic Model (LE): A linear elastic material is a mathematical model used to analyse the deformation of solid bodies.
- Mohr-Coulomb Model (MC): The plasticity model of Abaqus Mohr-Coulomb uses the classical Mohr-Coulomb yield function, which involves hardening and softening of isotropic cohesion. It also utilizes a smooth flow potential in the meridional stress plane that has a hyperbolic shape and in the deviatoric stress plane a piecewise elliptic shape. Hardening Soil Model (HS): The Hardening Soil model is an advanced model for the simulation of soil behaviour.
- Hardening Soil Model with small stress-strain stiffness (HS small)
- Soft Soil Model (SS): The Soft Soil model is a Cam-Clay type model especially meant for primary compression of near normally consolidated clay-type soils.
- Soft Soil Creep Model (SSC)
- Modified Cam-Clay Model (MCC)
- Sekivguchi-Ohta Model (Seki guchi-Ohta)
- Subloading t_{ij} Model

In this research work, our first work materials are modelled with Hardening Soil model and Soft soil model in PLAXIS 2D software.

3.8.1 Hardening Soil Model in PLAXIS

In order to model the plastic shear, the model requires friction hardening. To model the plastic volumetric strain in primary compression, strain in deviatoric loading and cap hardening. Two key hardening styles, namely shear hardening and compression hardening, can be distinguished. Shear hardening, due to primary deviatoric loading, is used to model irreversible strains. Compression hardening is used in oedometer loading and isotropic loading to model irreversible plastic strains due to primary compression. The present model includes all forms of hardening. The model's yield contour is shown below in three-dimensional space. The model is also accurate for problems requiring a reduction of mean effective stress and at the same time the mobilisation of shear strength because of the two forms of hardening. In excavation (retaining wall problems) and tunnel building projects, such situations occur.

The model shows a decrease in mean effective stress, as observed for soft soils, in undrained loading, although it can also display an increase in mean effective stress for harder soil types (dilative soils). This model can be used in various geotechnical applications to reliably predict displacement and failure for general kinds of soils. The model does not include anisotropic rigidity strength or time-dependent behaviour (creep). Its functionality for complex applications is minimal.

This model has some advantages over other soil model:

- ❑ By using three different input stiffnesses, soil stiffness is defined much more accurately: the triaxial stiffness E_{50}, the triaxial unloading stiffness E_{ur}, and the oedometer loading stiffness E_{oed}.
- ❑ One estimates a steady average stiffness for each sheet. Calculations are very rapid because of this constant stiffness and offer a good first impression of the problem.

3.8.2 Soft Soil Model in PLAXIS 2D

The Soft Soil model assumes a logarithmic relationship between the volumetric strain and the mean effective stress and is capable of modelling the compression behaviour of very soft soils. This relationship is formulated along the standard consolidation line during isotropic virgin compression. Soft soil model is capable to account for both elastic and plastic material behaviour. It is an advanced constitutive material model and the main features of the Soft Soil Model include: Failure behaviour, yield surface. Stiffness parameters can be obtained from oedometer-tests.

In this research work, our second work materials are modelled with Soft Soil model in PLAXIS 2D software. The main strengths of the Soft-Soil model include:

- ❑ Stress dependent stiffness (logarithmic compression behaviour).
- ❑ Distinction between primary loading and unloading-reloading.
- ❑ Memory for pre-consolidation stress.

CHAPTER 4: Model Consideration, Embankment Geometry and Soil Condition

4.1 Introduction

Based on the data that has been provided and the literature review, the following considerations have been taken:

- Finite Element Modelling of Settlement of soil
- Finding Factor of Safety

4.2 Selection of Construction Method

- PLAXIS 2D Software 2019 version
- Consider element with 15 nodes
- Consider plane strain condition for 2D Ground Model
- Materials are modelled with Hardening Soil model
- Materials are modelled with Soft Soil model
- Microsoft Excel for generating tables and graphs
- AutoCAD 2020 for drawing figures.

4.3 Embankment Section Geometry

Locations	Depth (m)	Slope	Length (m)
1	1.8	1:2	30
2	1.4	1:2	30

Table 4.3. I: Embankment section Geometry for models in PLAXIS 2D

4.4 Soil Parameters

Soil sample data are collected from Prosoil. This data is for Location 1 and 2 of Kachpur Bridge and the parameters are considered as the basic design input for the model. Soil parameters are extracted from the USCS soil classification, SPT values and different co-relations. From the soil report of Prosoil Foundation Consultant we get different important parameters for different co-relation.

Following tests are performed:

i. Particle size analysis-sieve
ii. Particle size analysis-Hydrometer
iii. Atterberg limits test
iv. Natural moisture content
v. Dry and apparent density
vi. Particle density
vii. Unconfined compressive strength
viii. Triaxial test (CU)
ix. Consolidation test

4.5 Soil Profile

4.5.1 Location Cases:

In this reaearch work we have considered two locations for the comparison of field monitoring data with PLAXIS 2D software simulation.

<table>
<tr><td rowspan="2">Project: The Construction of Kanchpur, Meghna, Gumti 2nd Bridges and Rehabilitation of Existing Bridges (Package No. PW-01)</td><td>Location-1</td></tr>
<tr><td>Location-2</td></tr>
</table>

Table 4.5.1: Location Cases

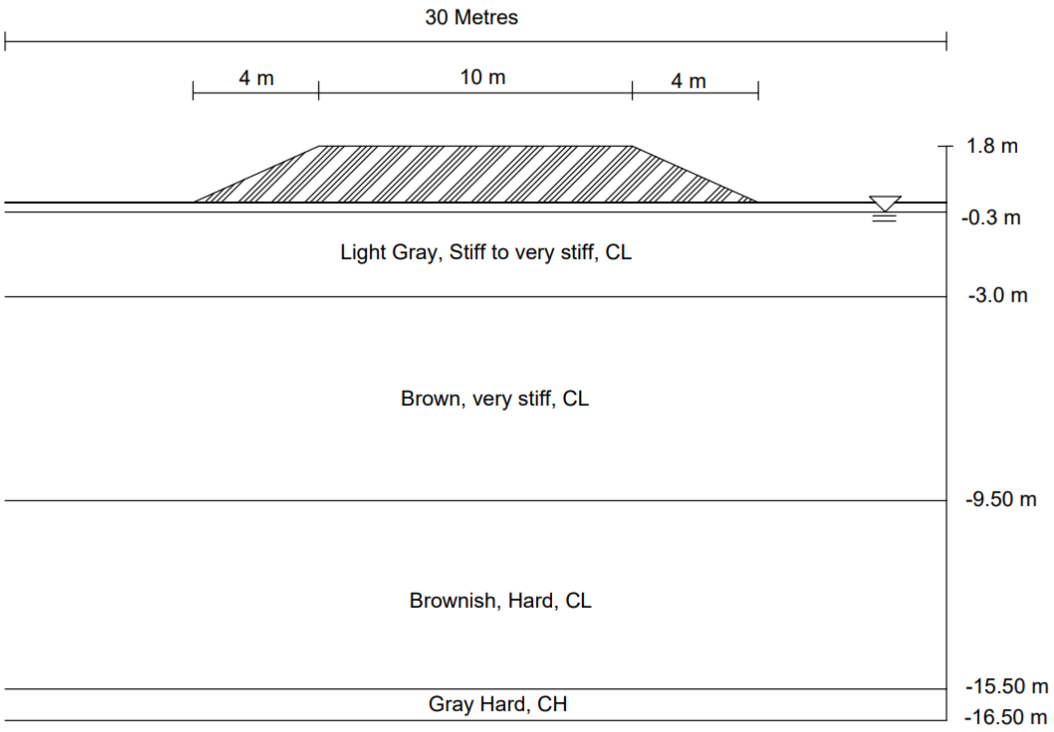


Figure 4.5.1a: Location 1

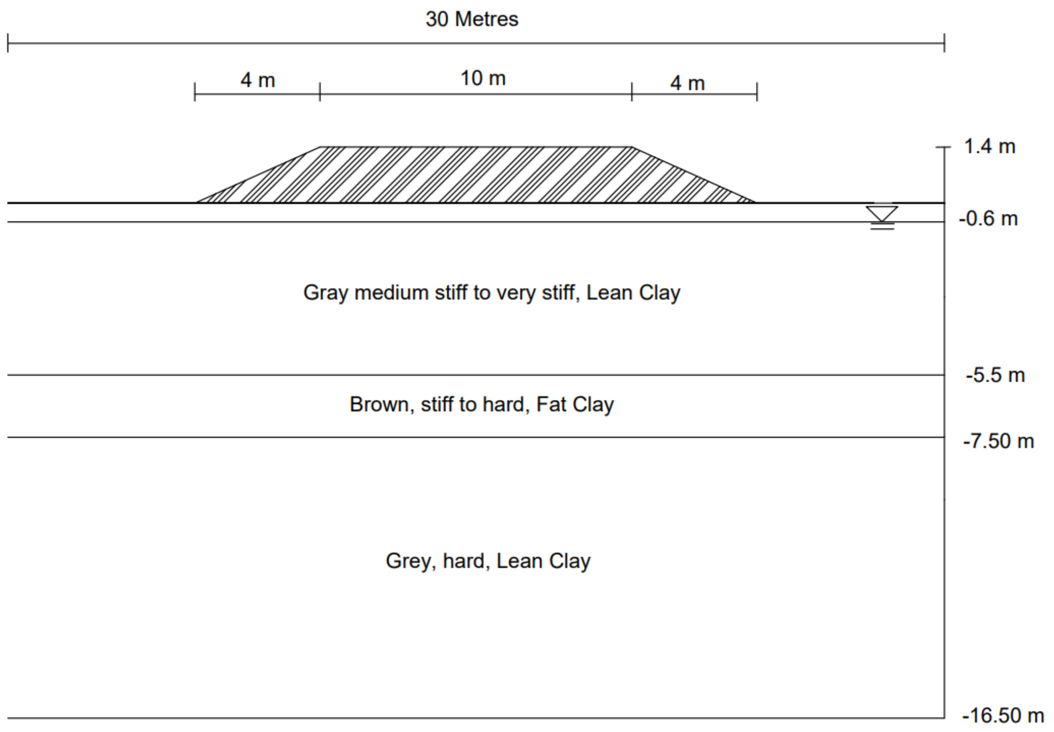


Figure 4.5.1b: Location 2

Soil layers | Water | Initial conditions | Preconsolidation

#	Layers Material		Location_ 1 Top	Bottom
1		Light Gray , stiff to ver	0.000	-3.000
2		Brown, very stiff, CL	-3.000	-9.500
3		Brownish, Hard, CL	-9.500	-15.50
4		Gray, Hard, CH	-15.50	-16.50

Figure 4.5.1.1: Bore-log of Location-1 from field data

Soil layers | Water | Initial conditions | Preconsolidation

#	Layers Material		Location_ 2 Top	Bottom
1		Medium stiff to very sti	0.000	-5.500
2		Stiff to hard, CH with s	-5.500	-7.500
3		Hard, CL with sand	-7.500	-16.50

Figure 4.5.1.2: Bore-log of Location-2 from field data

4.5.2 Basic Parameters

γ_{wt} = Wet (KN/m^3)

γ_d =Dry (KN/m3)

c = Cohesion

ϕ= Angle of internal friction

e_o= Void Ratio

C_c= Compression Index

C_r= Recompression Index

Location 1				B. W. T : 0.30 m below from E. G. L.	SUMMERY SHEET OF TEST RESULTS									
Layer	Depth(m)	Classification of soil(USCSC)	Description	Grain Size (%)			Avg. SPT	Avg. SPT		Consolidation Test			Direct Shaer Test	
				SAND	SILT	CLAY		γ_{wt}	γ_d	e_0	C_c	Cs	C	φ^o
1	0 to 3.5	CL	Light Gray , stiff to very stiff, Lean Clay	20.1	77.1	2.9	21	18.9	15.5	0.941	0.254	0.037	101	28
2	3.5 to 9.5	CL	Brown, very stiff, Lean Clay	21.7	74.9	3.5	36	19.61	16.5	0.773	0.081	0.005	59	27
3	9.5 to 15.5	CL	Brownish, Hard, Lean Clay	13.9	82.9	3.2	50	18	17	0.731	0.08	0.0234	190	25
4	15.5 to 16.5	CH	Gray, Hard, Fat Clay	18.6	78.6	2.8	31	18	17	0.77	0.16	0.065	230	25

Table 4.5. a: Parameters of soil from field data of Location-1

Location 2				B. W. T : 0.6 m below from E. G. L.	SUMMERY SHEET OF TEST RESULTS									
Layer	Depth(m)	Classification of soil(USCSC)	Description	Grain Size (%)			Avg. SPT	Avg. SPT		Consolidation Test			Direct Shaer Test	
				SAND	SILT	CLAY		γ_{wt}	γ_d	e_0	C_c	Cs	C	φ^o
1	0 to 5.5	CL	Gray medium stiff to very stiff Lean Clay	14.8	82.4	2.8	13	17.8	15.45	0.657	0.075	0.016	90	29
2	5.5 to 7.5	CH	Brown, stiff to hard Fat clay	19.3	72.8	7.9	46	17.5	15.05	0.821	0.06743	0.00843	170	28
3	7.5 to 16.5	CL	Gray, hard Lean Clay	11.5	77	11.5	50	17.6	15.03	0.7961	0.0921	0.01151	220	27

Table 4.5. b: Parameters of soil from field data of Location-2

4.5.2.a OCR Calculation

OCR is equal to maximum applied effective stress in past to the maximum applied effective stress in present.

OCR>1 (Over Consolidation Stage)

OCR=1 (Normal Consolidation Stage)

OCR<1 (Under Consolidation Stage)

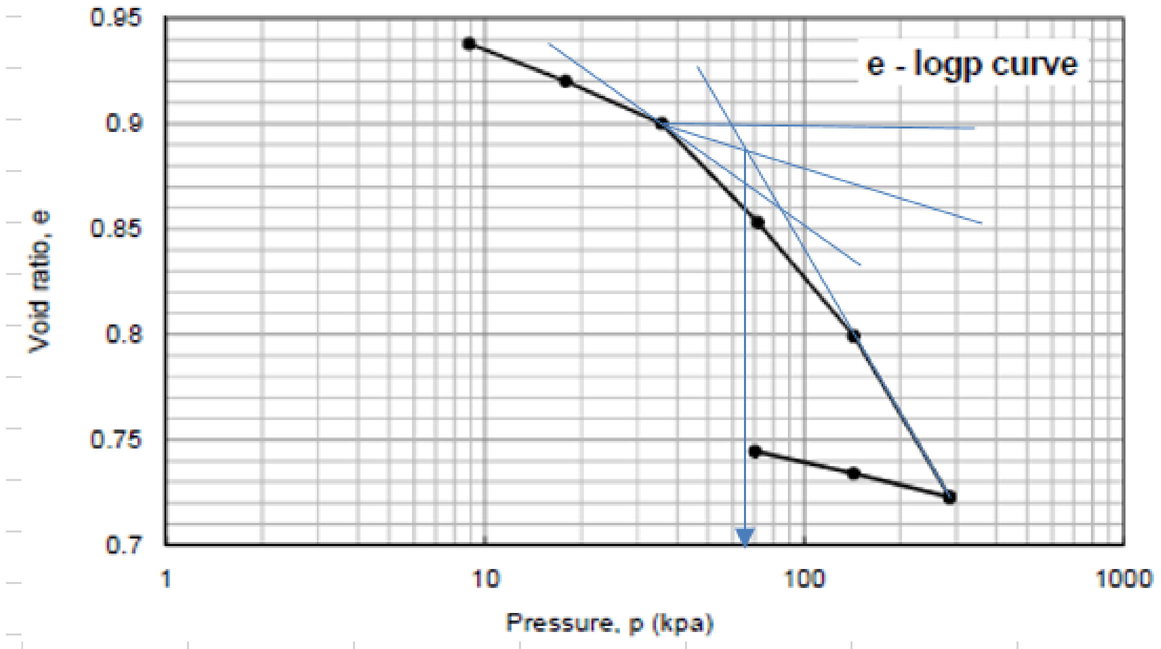
$OCR = \frac{\sigma'}{\sigma}$;Where, σ′= preconsolidation pressure

σ = vertical effective stress

(i) Location 01

Applied Beam Load (kg)	Applied Pressure (kpa)	Final Dial (cm)	Dial Change (cm)	Specimen Height (cm)	Void Height (cm)	Void Ratio
0	0	0.597	0	2.41	1.168	0.941
0.318	8.92	0.593	0.004	2.406	1.164	0.938
0.637	17.88	0.571	0.022	2.384	1.142	0.92
1.275	35.75	0.546	0.025	2.359	1.117	0.9
2.55	71.49	0.488	0.058	2.301	1.059	0.853
5.1	142.98	0.421	0.067	2.234	0.992	0.799
10.2	285.97	0.326	0.095	2.139	0.897	0.723
5.1	142.98	0.34	-0.014	2.153	0.911	0.734
2.5	70.09	0.353	-0.013	2.166	0.924	0.745

Table 4.5.2. 1: Test results of consolidation test report



Graph 4.5.2.1: e-logp curve

From graph:

σ'= 65.7

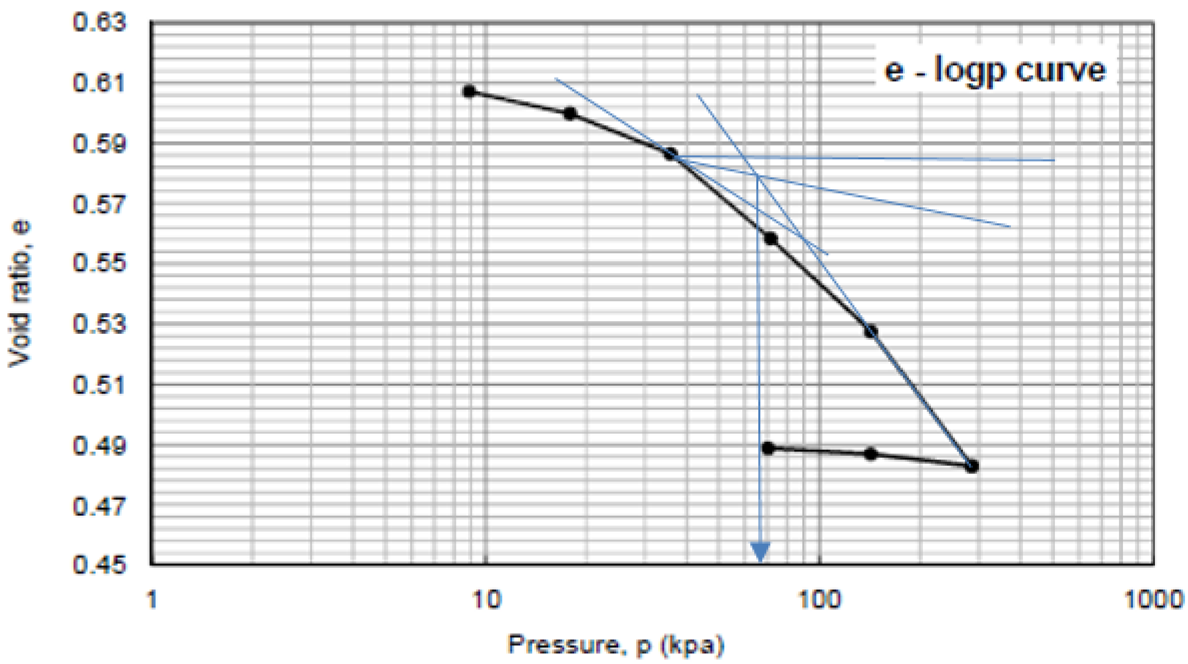
σ =29.64

OCR= $\frac{65.7}{29.64}$ = 2.22

(ii) Location 02

Applied Beam Load (kg)	Applied Pressure (kpa)	Final Dial (cm)	Dial Change (cm)	Specimen Height (cm)	Void Height (cm)	Void Ratio
0	0	0.462	0	2.41	0.914	0.611
0.318	8.92	0.456	0.006	2.404	0.908	0.607
0.637	17.88	0.445	0.011	2.393	0.897	0.6
1.275	35.75	0.425	0.02	2.373	0.877	0.588
2.55	71.49	0.383	0.042	2.331	0.835	0.558
5.1	142.98	0.337	0.046	2.285	0.789	0.528
10.2	285.97	0.27	0.067	2.218	0.722	0.483
5.1	142.98	0.276	-0.006	2.224	0.728	0.487
2.5	70.09	0.279	-0.003	2.227	0.731	0.489

Table 4.5.2. 2: Test results of consolidation test report



Graph 4.5.2.2: e-logp curve

From graph:

$\sigma' = 67$

$\sigma = 50.2$

OCR= $\frac{67}{50.2}$ = 1.4

4.6 Monitoring Record

Equipement used:

- **Settlement Plate**

In order to record the magnitude and rate of settlement under a load, settlement plates or points are usually mounted where significant settlement is expected. Therefore, after installing the vertical drains, they should be mounted immediately. This tool is, in the simplest form, a settlement plate consisting of a steel plate placed on the ground before the embankment is built. Surface settlement points, for example, along an embankment centerline, assess vertical displacement with depth. The settlement-monitoring platform is usually connected to a reference rod and a protective pipe. Settlement is also regularly calculated before the surcharge embankment is completed, and then the elevation of the top of the reference rod is determined at a reduced frequency. Benchmarks used for comparison data must be stable and distant from all other vertical motions that are possible.

Figure 4.6 I: Settlement plate in the field (from Prosoil)

4.6.1 Location 1

- Monitoring Record -Settlement Plate From 15 December '16 to 31 Jan '17
- Location: Ch: KB 1+020

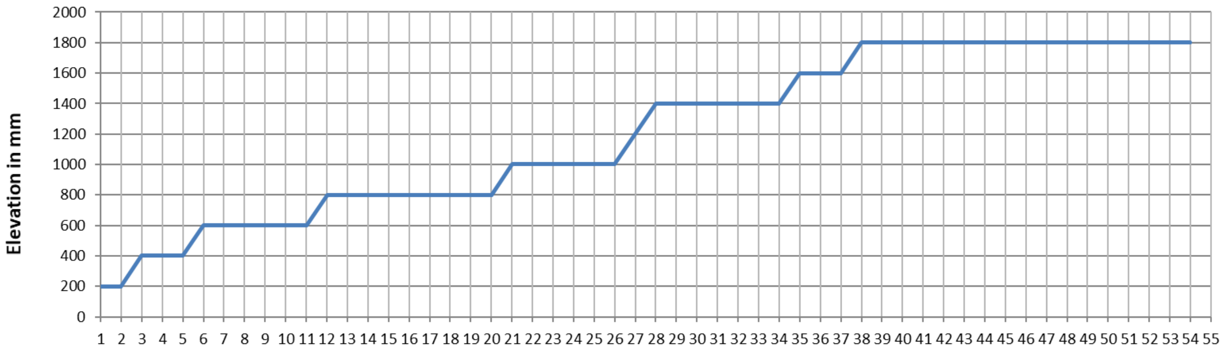


Figure 4.6.1. a: Elevation vs Day

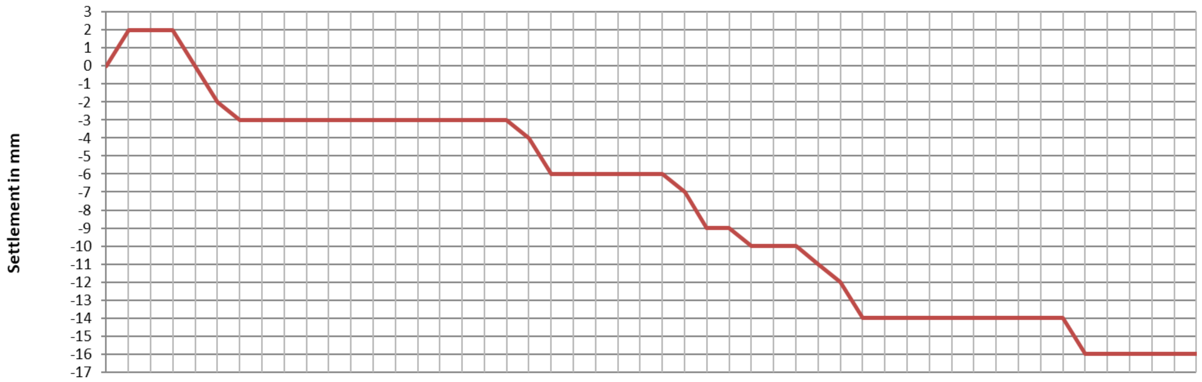


Figure 4.6.1. b: Settlement vs Day

4.6.2 Location 2

- Monitoring Record -Settlement Plate From 15 December '16 to 31 Jan '17
- Location: Ch: KB 1+020

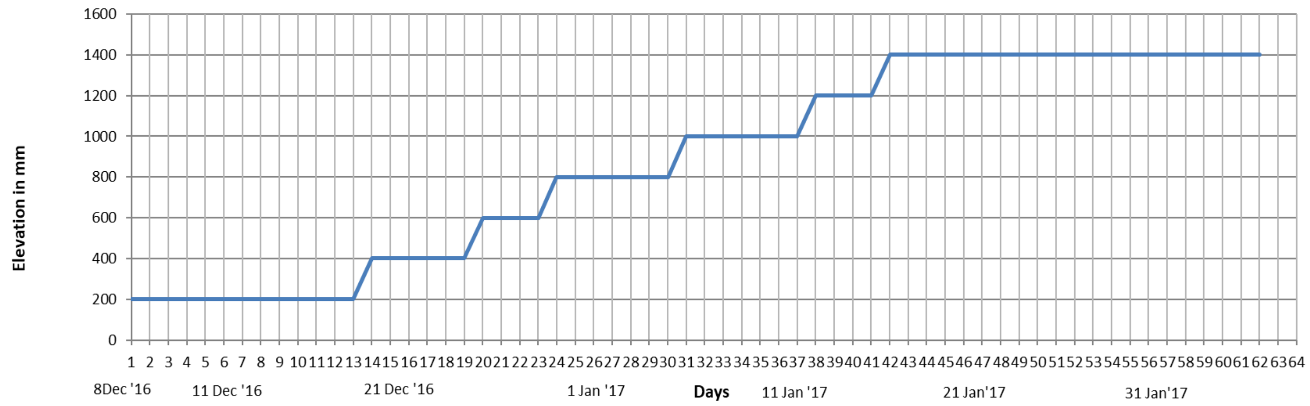


Figure 4.6.2. a: Elevation vs Day

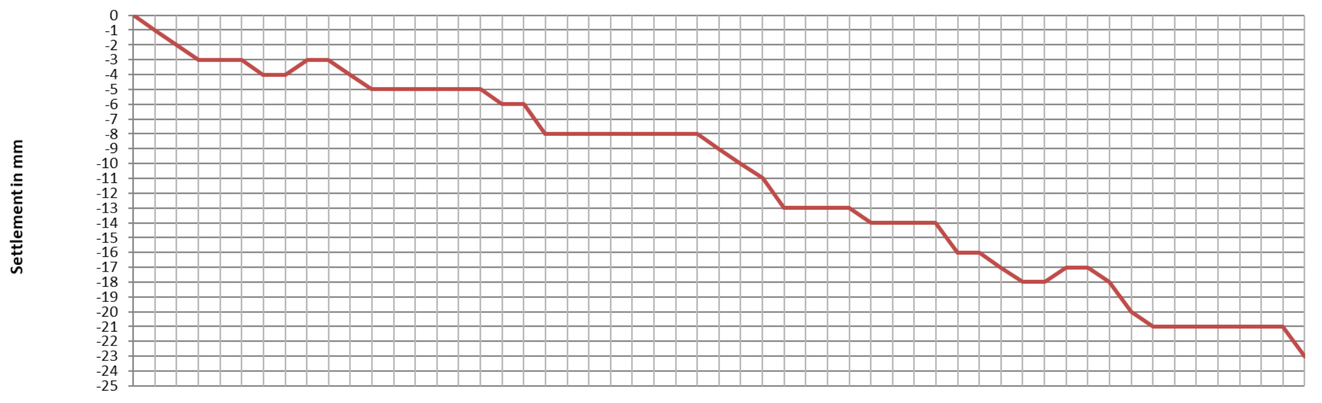


Figure 4.6.2. b: Settlement vs Day

4.7 Ground Improvement

Here, a surchage load of 20 KN/m is given on Location-1 and Location-2

CASE 1	**No geogrid**
CASE 2	**1 layer of geogrid**
CASE 3	**3 layers of geogrid**

- **Case-1:**

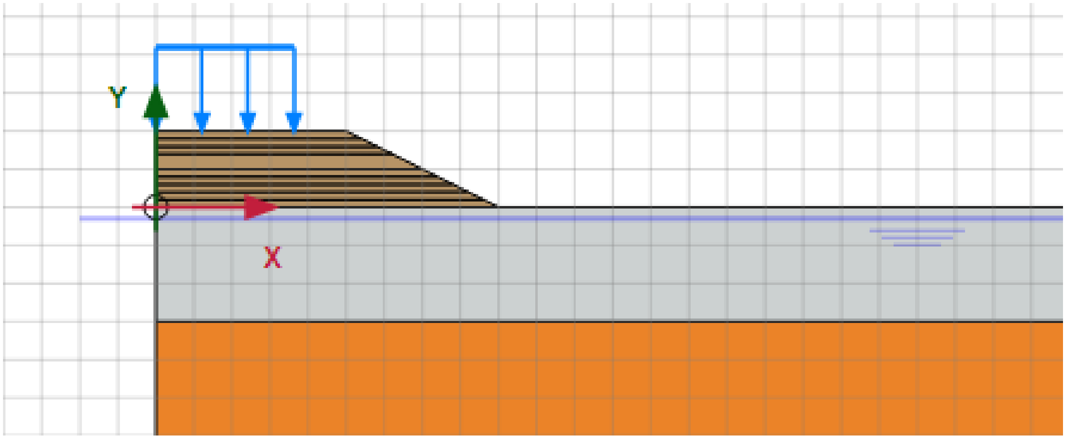


Figure 4.7. a: Case 1: no geogrid

- **Case-2:**

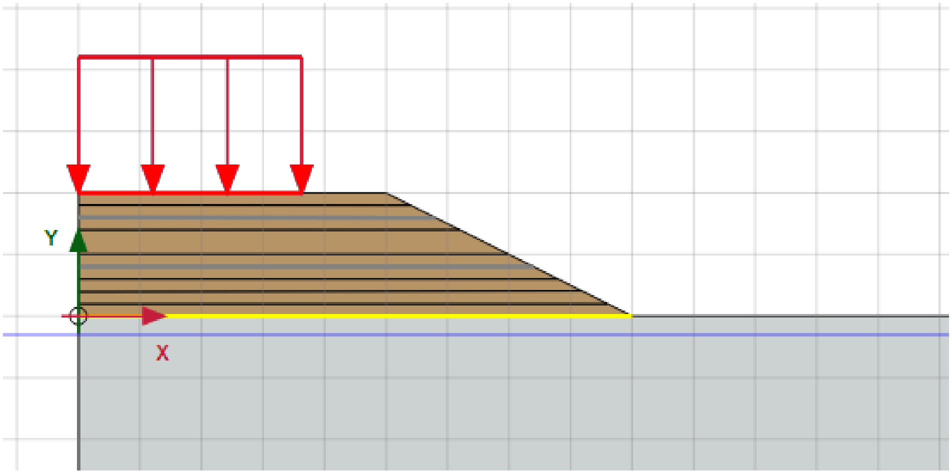


Figure 4.7. b: Case 2: 1 layer of geogrid

- **Case-3:**

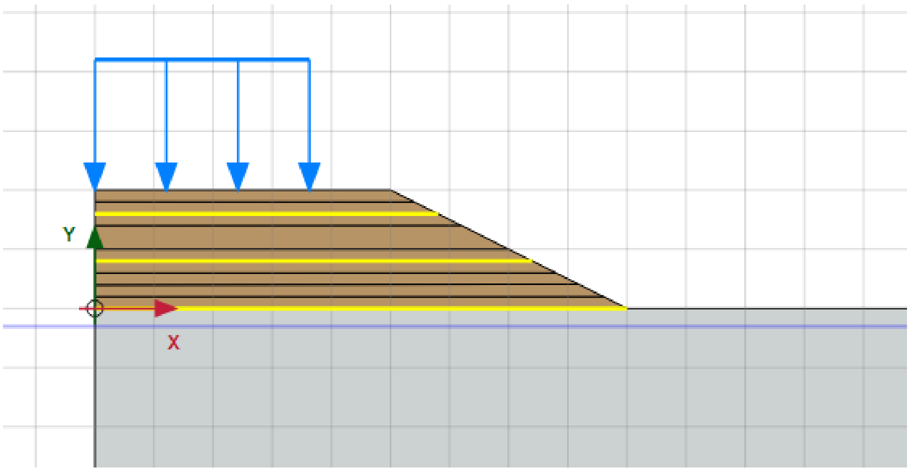


Figure 4.7. c: Case 3: 3 layers of geogri

CHAPTER 5: Results and Analysis

5.1 Introduction

Results from both the hardening and soft soil models are determined in comparison with field monitoring data:

The deformed mesh, settlement vs day graph for the settlement analysis; settlement vs step graph for different nodes for "with geogrid" and "without geogrid" are plotted for the embankment behavior, mentioned in this section.

In our research work, there are comparison of hardening soil model with soft soil model for the two locations, ground improvement determination by using geogrid and finally safety factor analysis of both the models.

5.2 Ground Condition

5.2.1 Hardening Soil Model

5.2.1.1 Location-1

Figure 5.1 shows total displacement of location 1. It is found that the total displacement is 39.88mm.

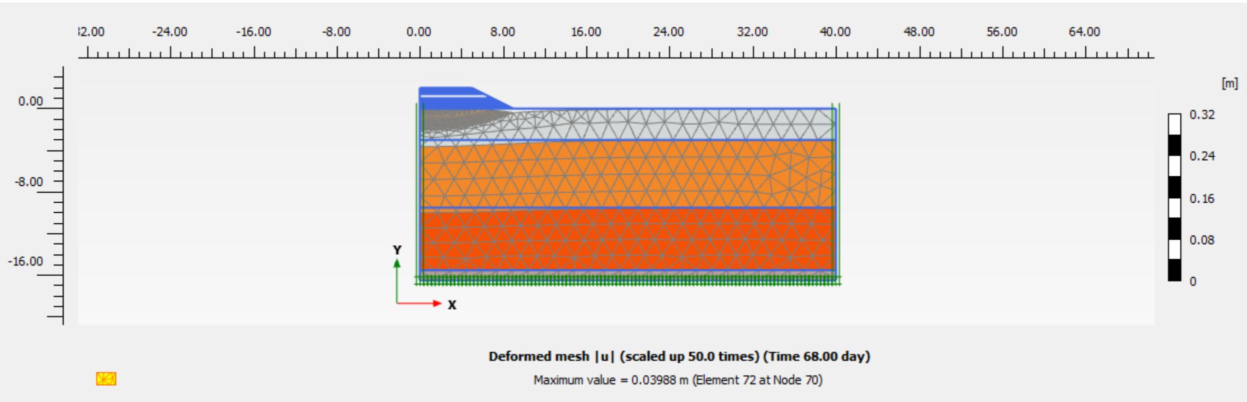
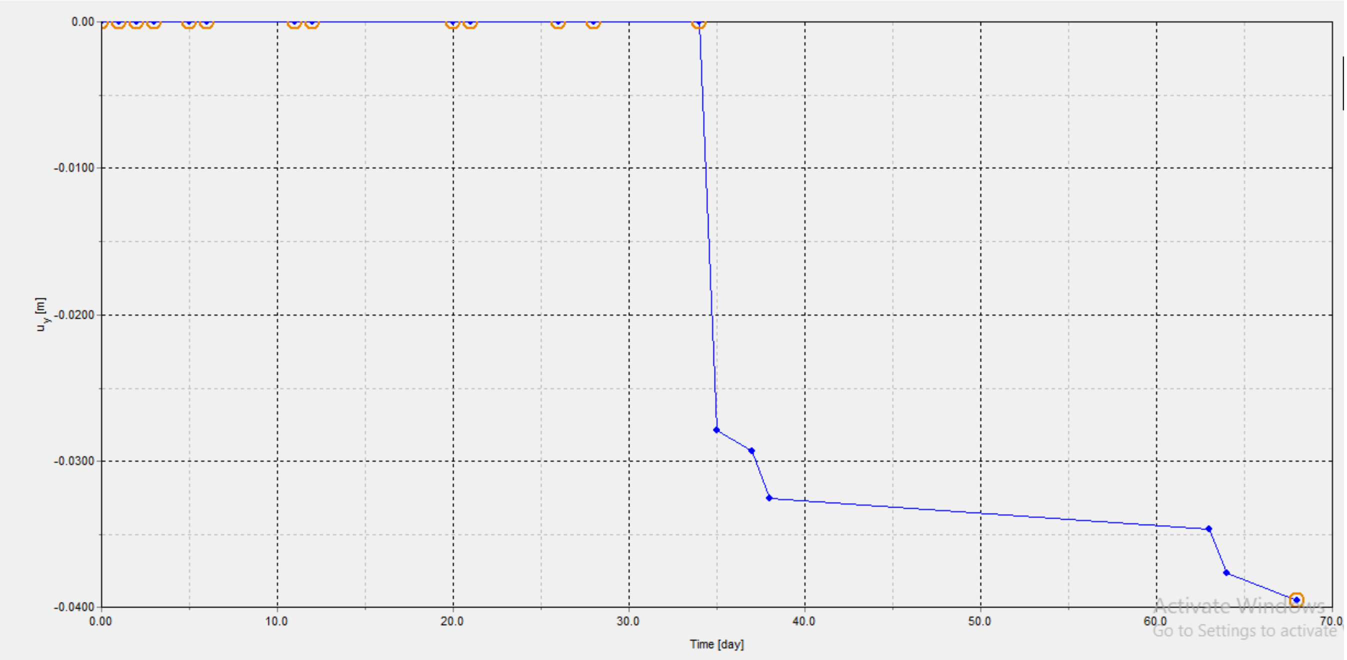


Figure 5.1. Deformed mesh of Location-1

PLAXIS 2D simulation:

Figure 5.1a shows settlement at the highest node of location 1. It is found that the total settlement is 40mm.



Graph 5.1a: Settlement vs Day using PLAXIS 2D

5.2.1.2 Location-2

Figuure 5.2 shows total displacement of location 2. It is found that the total displacement is 50.86mm.

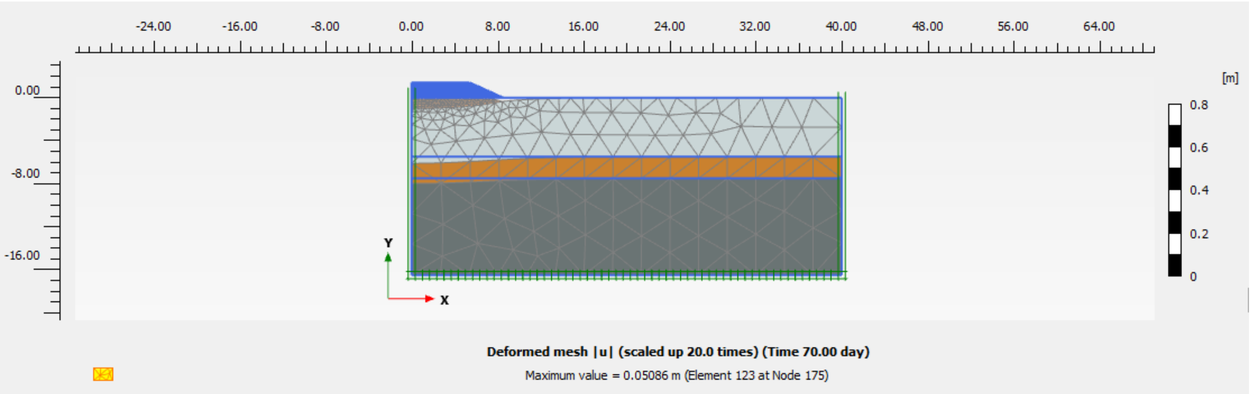
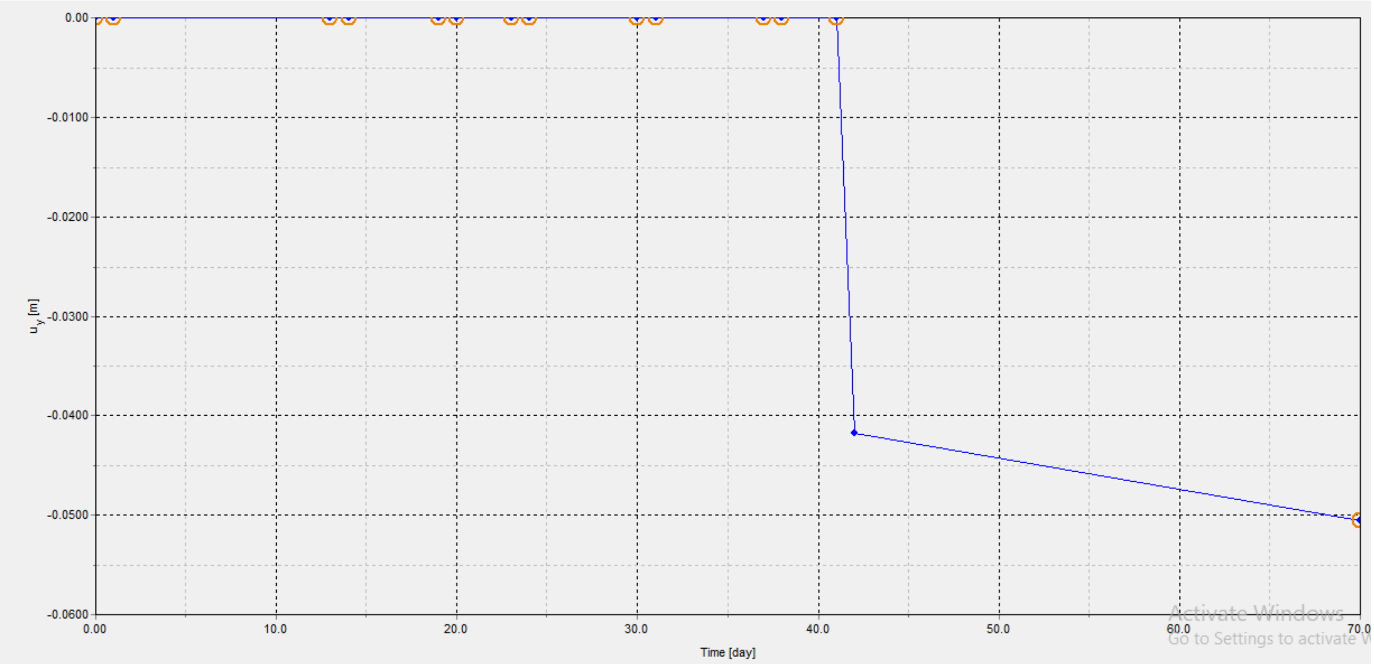


Figure 5.2: Deformed mesh of Location-2

PLAXIS 2D simulation:

Figure 5.2a shows settlement at the highest node of location 2. It is found that the total settlement is 51mm.



Graph 5.2: Settlement vs Day using PLAXIS 2D

5.2.2 Soft Soil Model

5.2.2.1 Location-1

Figure 5.3 shows total displacement of location 1. It is found that the total displacement is 16.67mm.

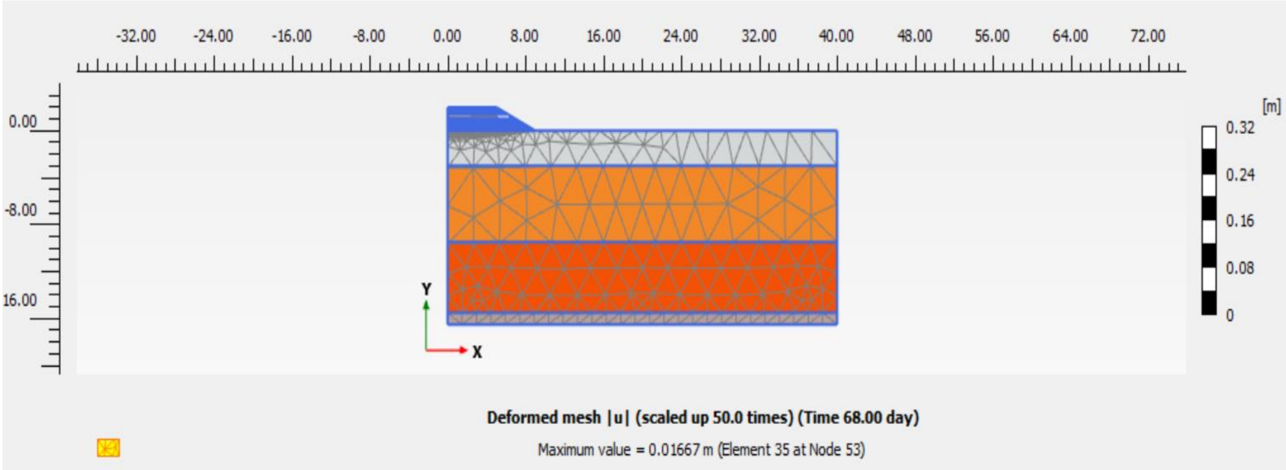
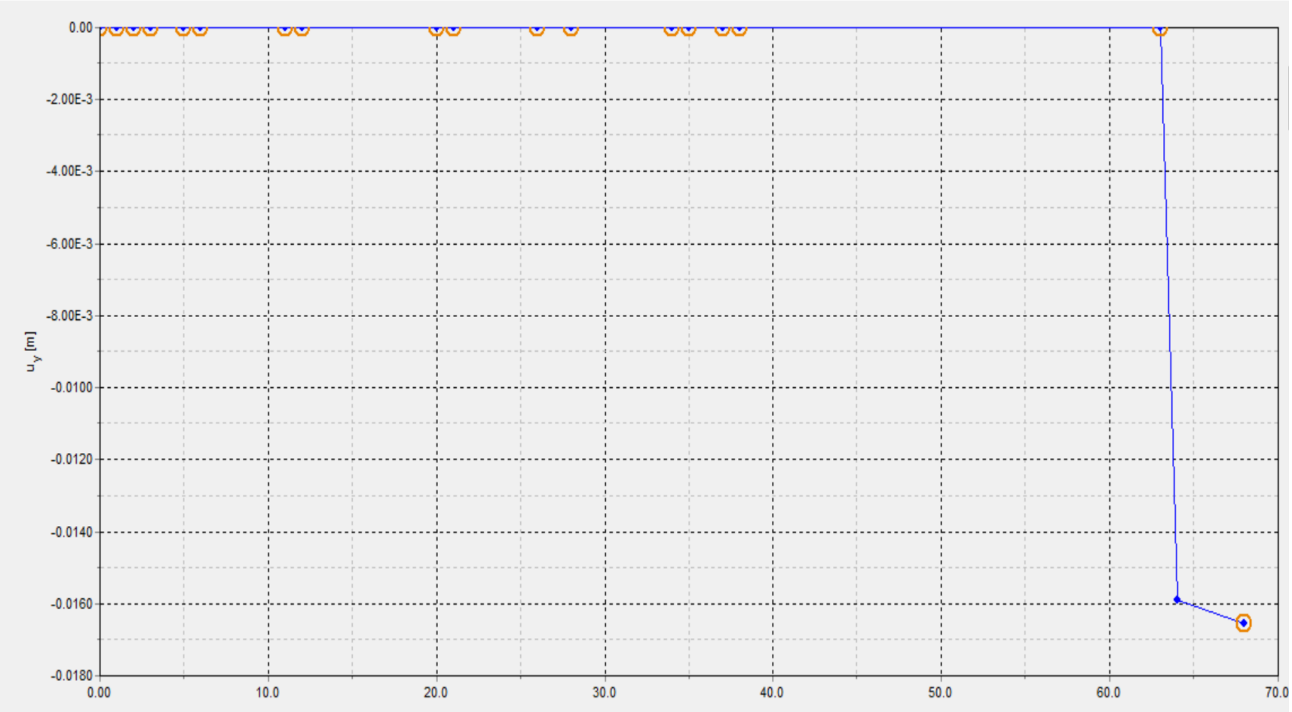


Figure 5.3: Deformed mesh of Location-1

PLAXIS 2D simulation:

Figure 5.3a shows settlement at the highest node of location 1. It is found that the total settlement is 17mm.



Graph 5.3a: Settlement vs Day using PLAXIS 2D

5.2.2.2 Location-2

Figure 5.4 shows total displacement of location 2. It is found that the total displacement is 25.65mm.

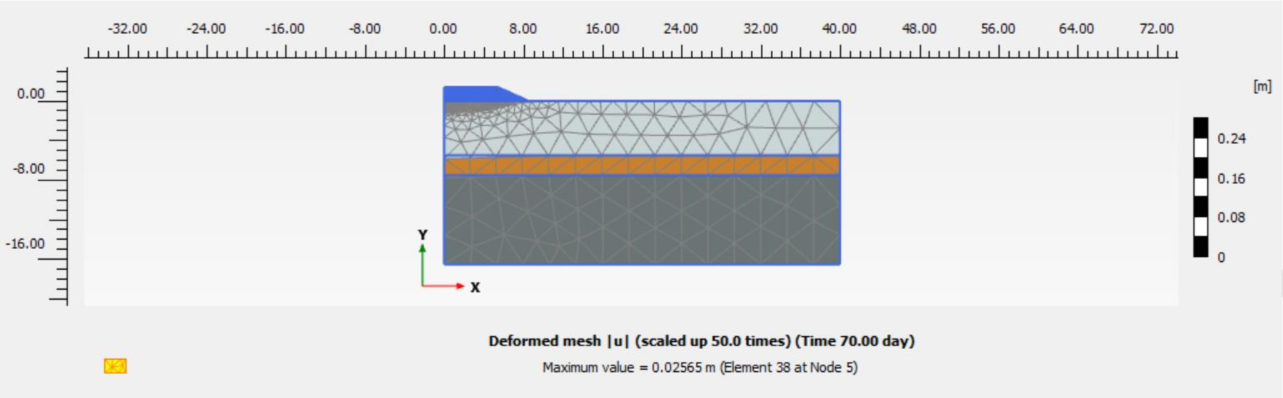
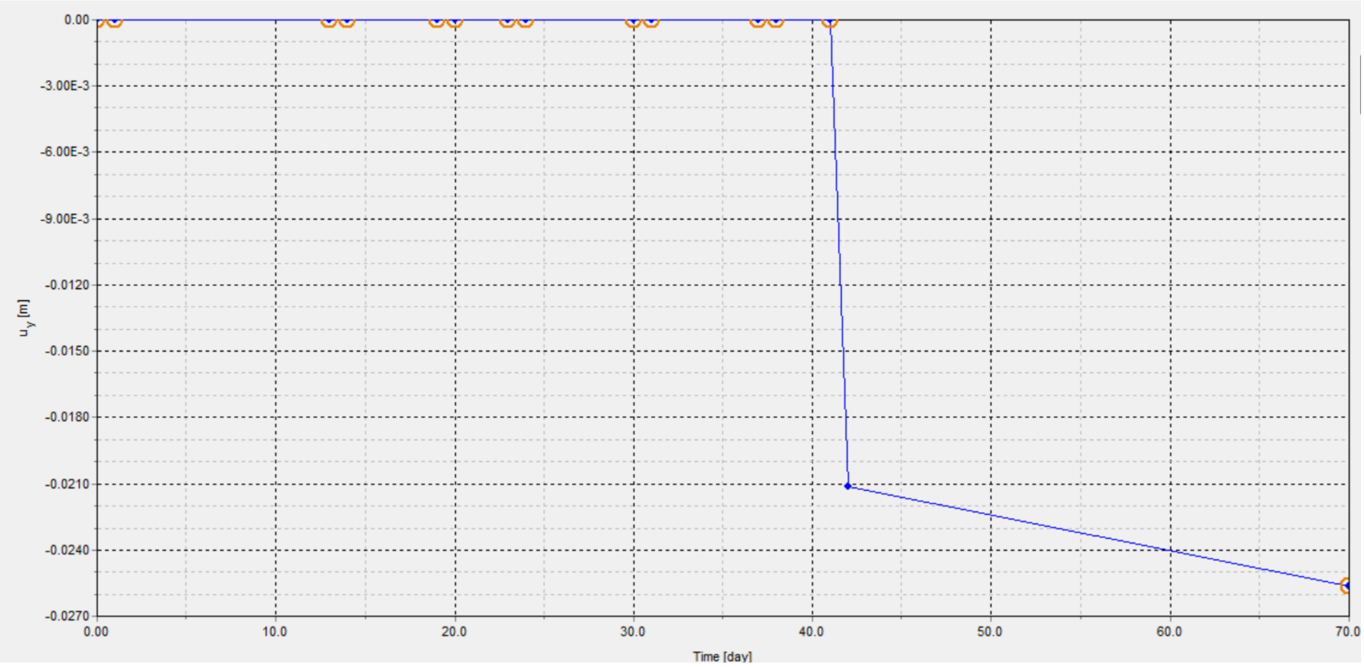


Figure 5.4: Deformed mesh of Location-2

PLAXIS 2D simulation:

Figure 5.4a shows settlement at the highest node of location 1. It is found that the total settlement is 26mm.



Graph 5.4a: Settlement vs Day using PLAXIS 2D

From the above analysis, the settlement value using PLAXIS 2D best matched with the field monitoring data while using soft soil model over hardening soil model. So, for further analysis to compare the settlement of embankment for "with geogrid" and "without geogrid", we again compared the hardening soil model and soft soil model and presented the result in the following subsection.

5.3. Ground Improvement

5.3.1 Hardening Soil Model

5.3.1.1 Location-1

Figure 5.5 shows total displacement of location 1. It is found that the total displacement is 71.19mm.

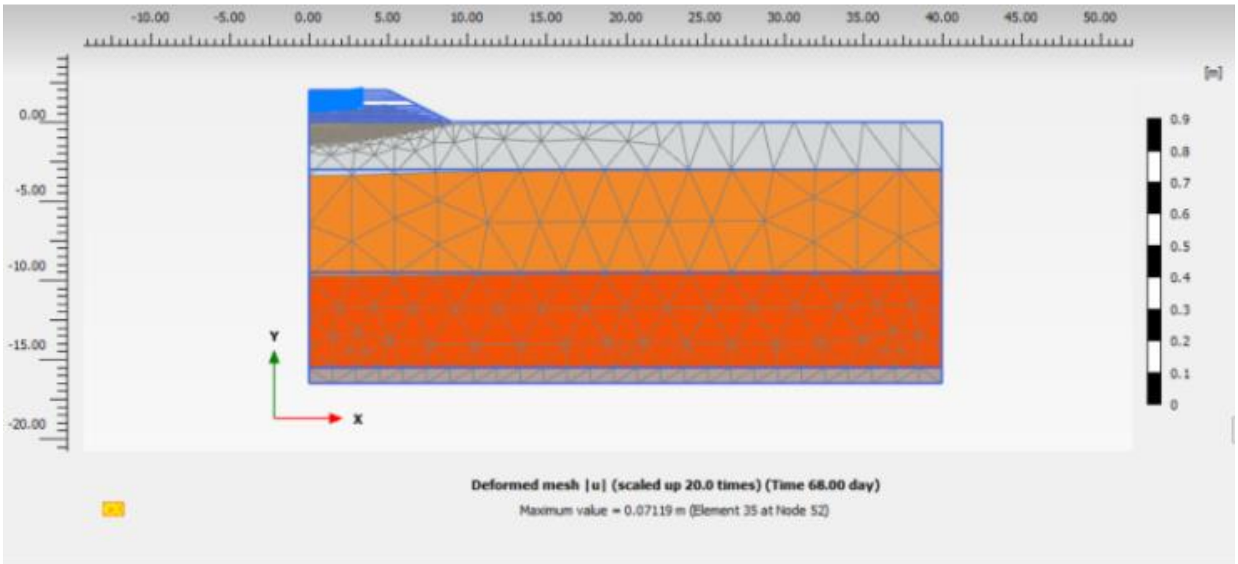
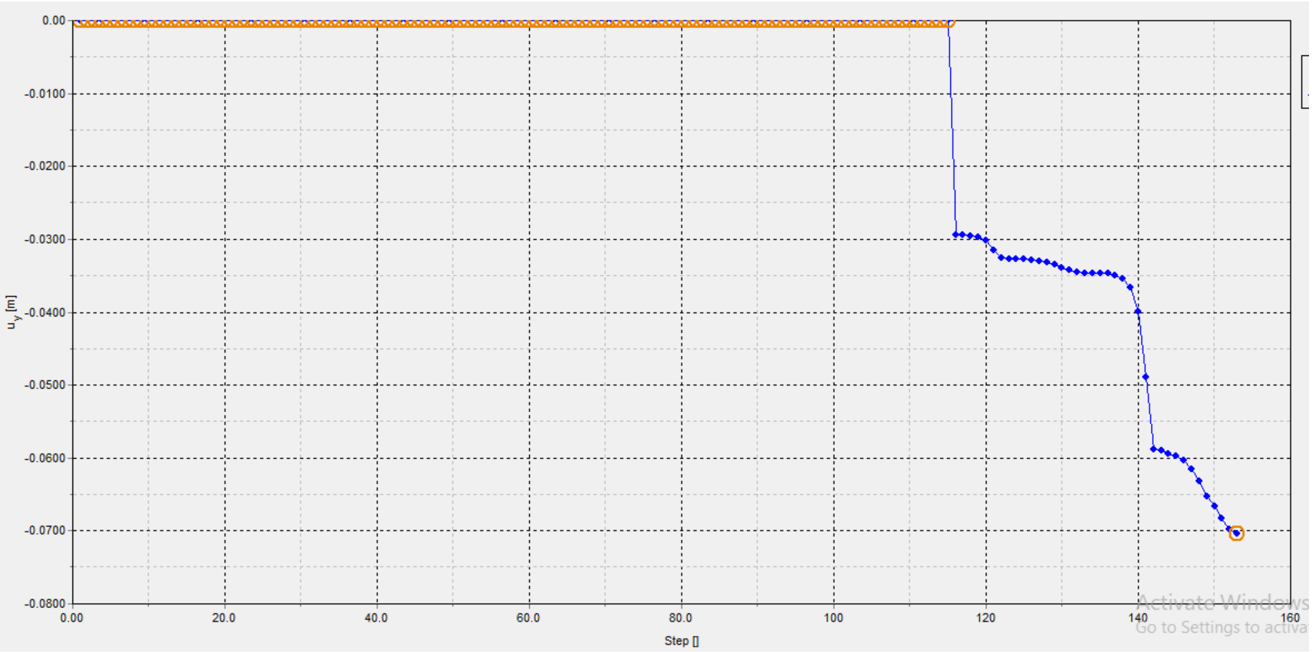


Figure 5.5:Deformed mesh of Location-1 after surcharge

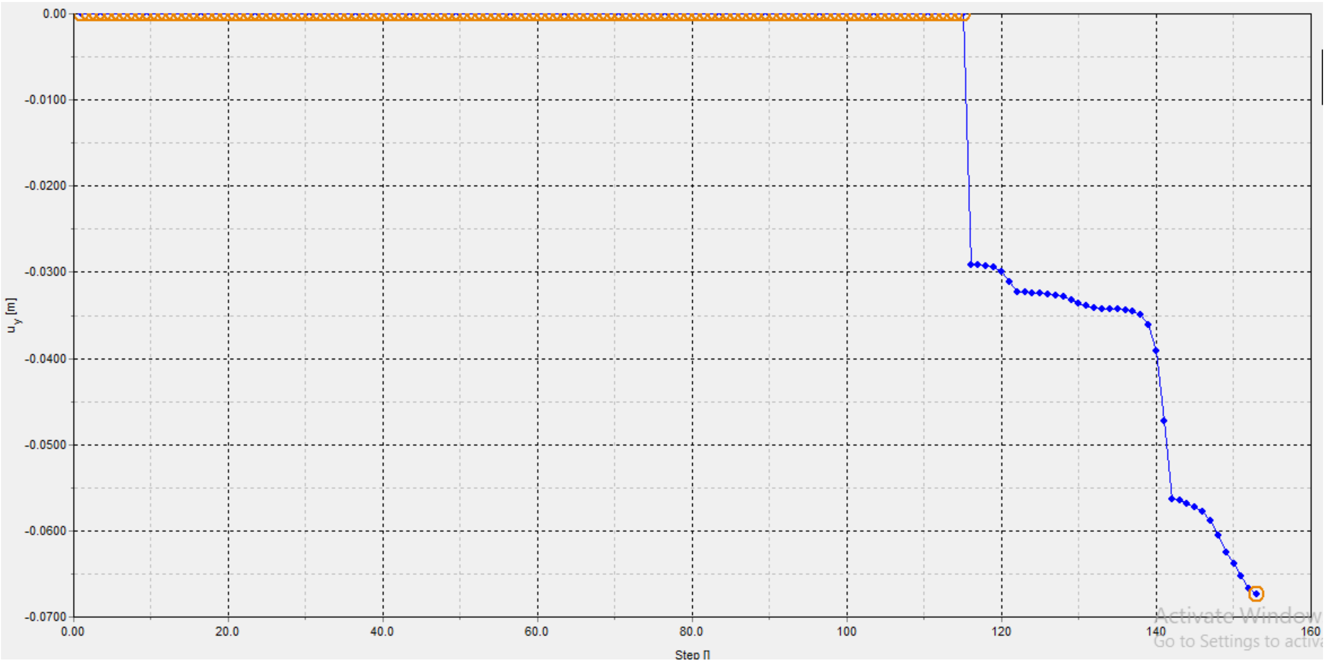
- **Case-1: No geogrid**

Figure 5.5.1a shows settlement at the node (1.04,1.8) of location 1. It is found that the total settlement is 70mm.



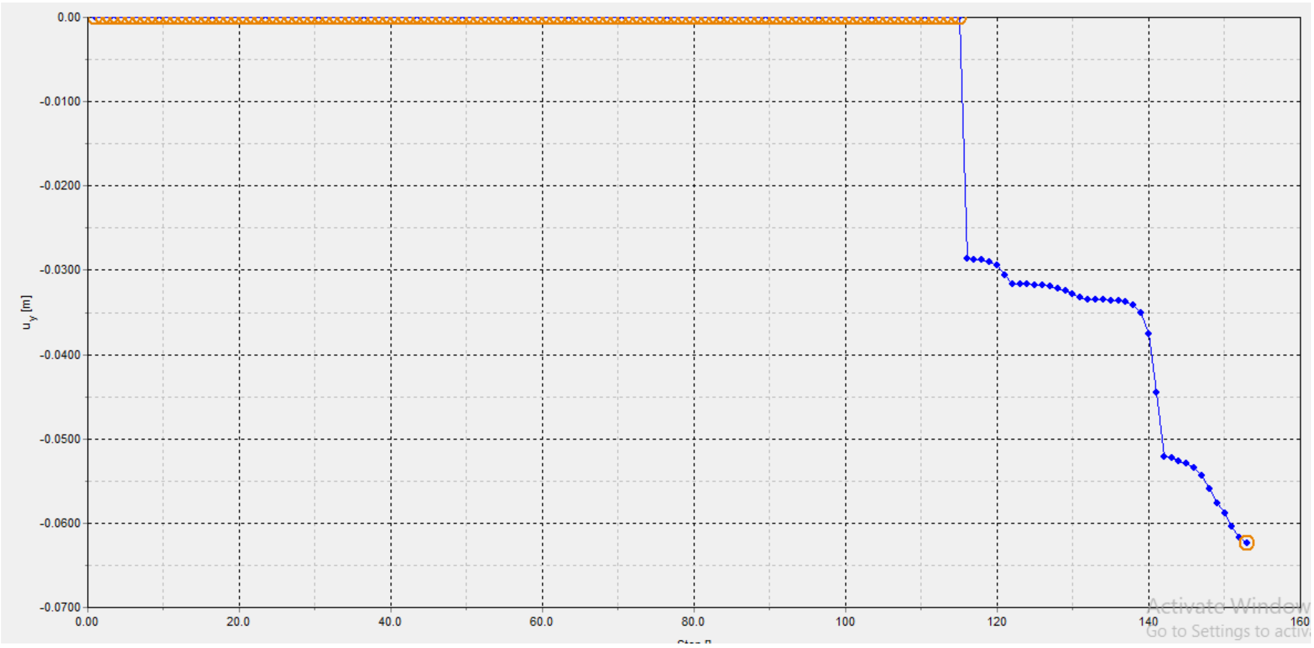
Graph 5.5.1a: Settlement vs Step at point (1.04, 1.8)

Figure 5.5.1b shows settlement at the node (2.0,1.8) of location 1. It is found that the total settlement is 68mm.



Graph 5.5.1b: Settlement vs Step at point (2.0, 1.8)

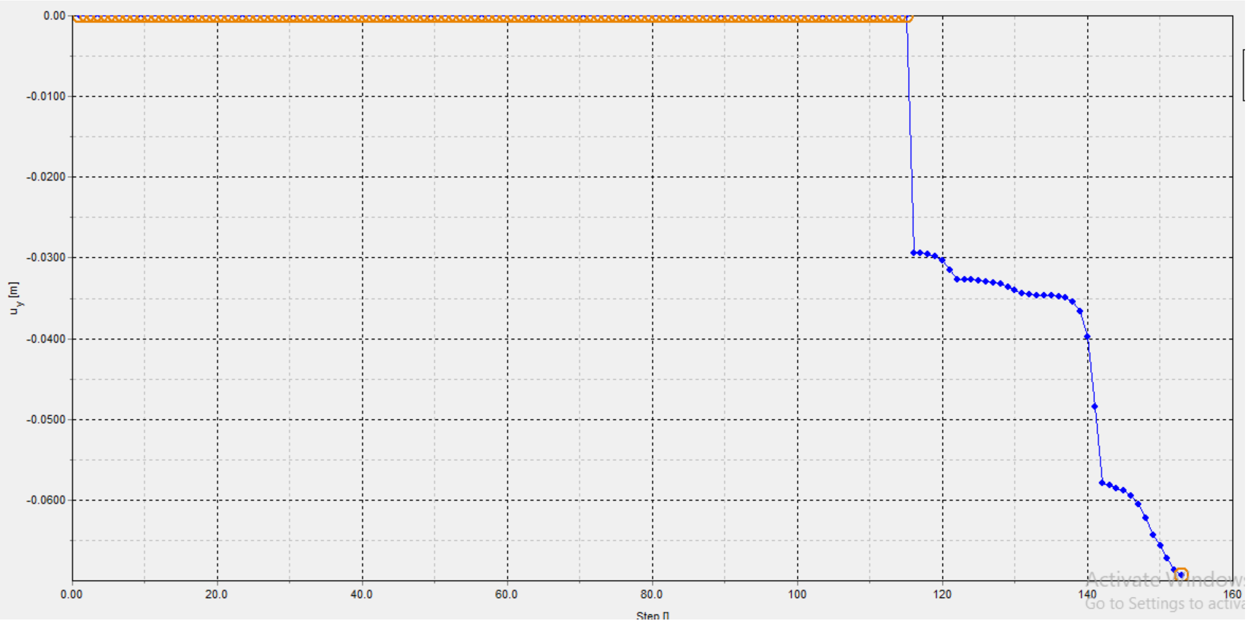
Figure 5.5.1c shows settlement at the node (2.98,1.8) of location 1. It is found that the total settlement is 63mm.



Graph 5.5.1c: Settlement vs Step at point (2.98, 1.8)

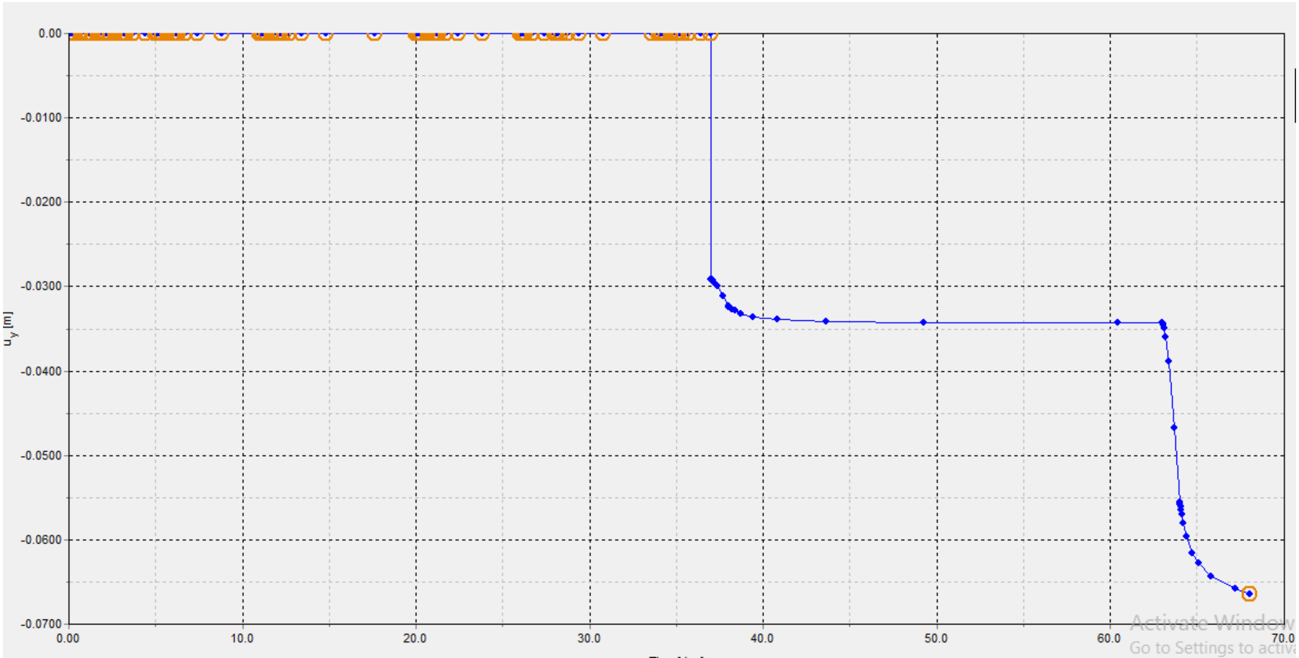
- **Case:2- 1 layer of geogrid**

Figure 5.5.2a shows settlement at the node (1.04,1.8) of location 1. It is found that the total settlement is 69mm.



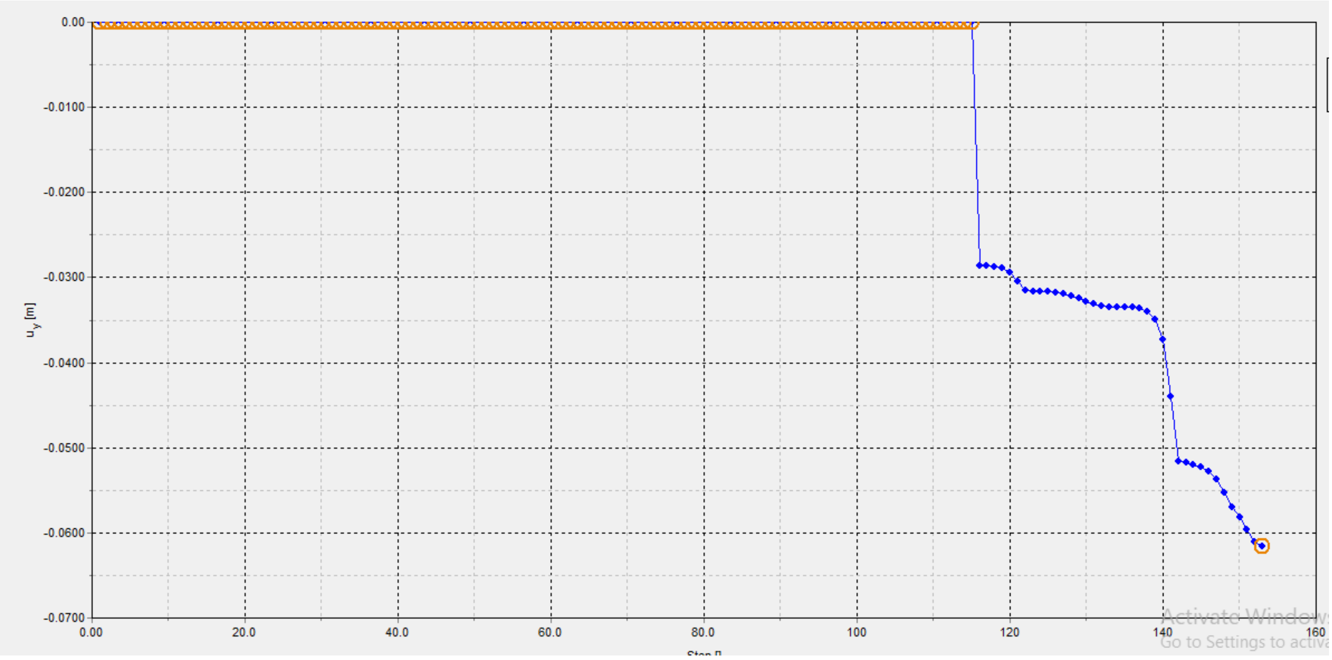
Graph 5.5.2a: Settlement vs Step at point (1.04, 1.8)

Figure 5.5.2b shows settlement at the node (2.0,1.8) of location 1. It is found that the total settlement is 66mm.



Graph 5.5.2b: Settlement vs Step at point (2.0, 1.8)

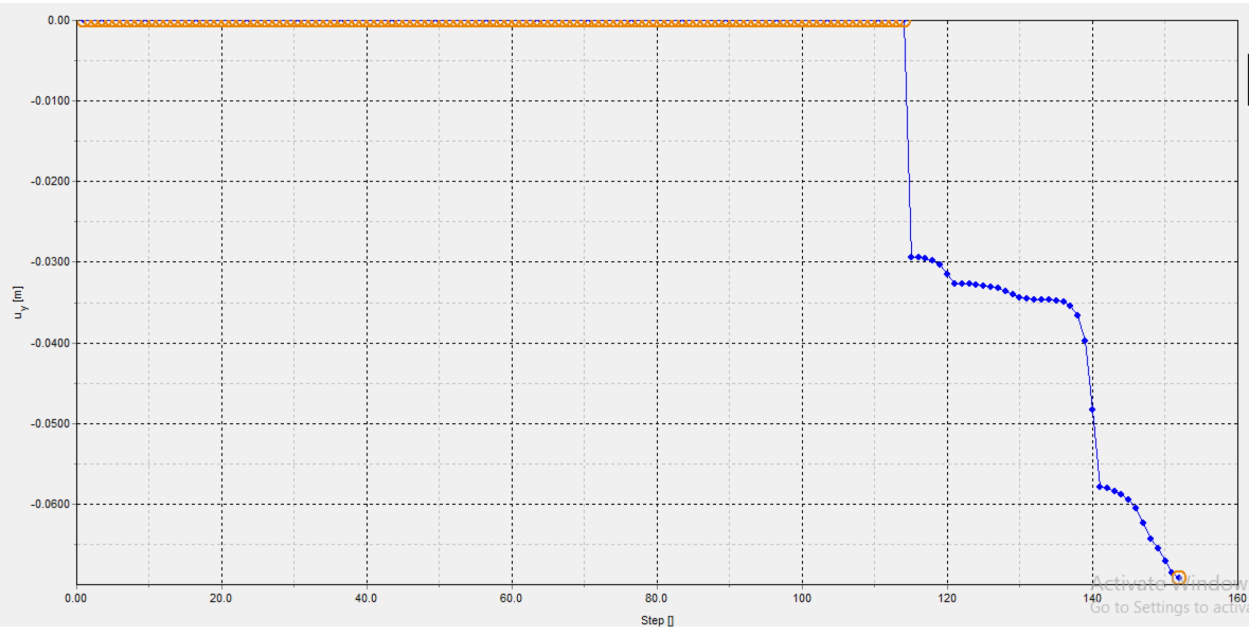
Figure 5.5.2c shows settlement at the node (2.98,1.8) of location 1. It is found that the total settlement is 62mm.



Graph 5.5.2c: Settlement vs Step at point (2.98, 1.8)

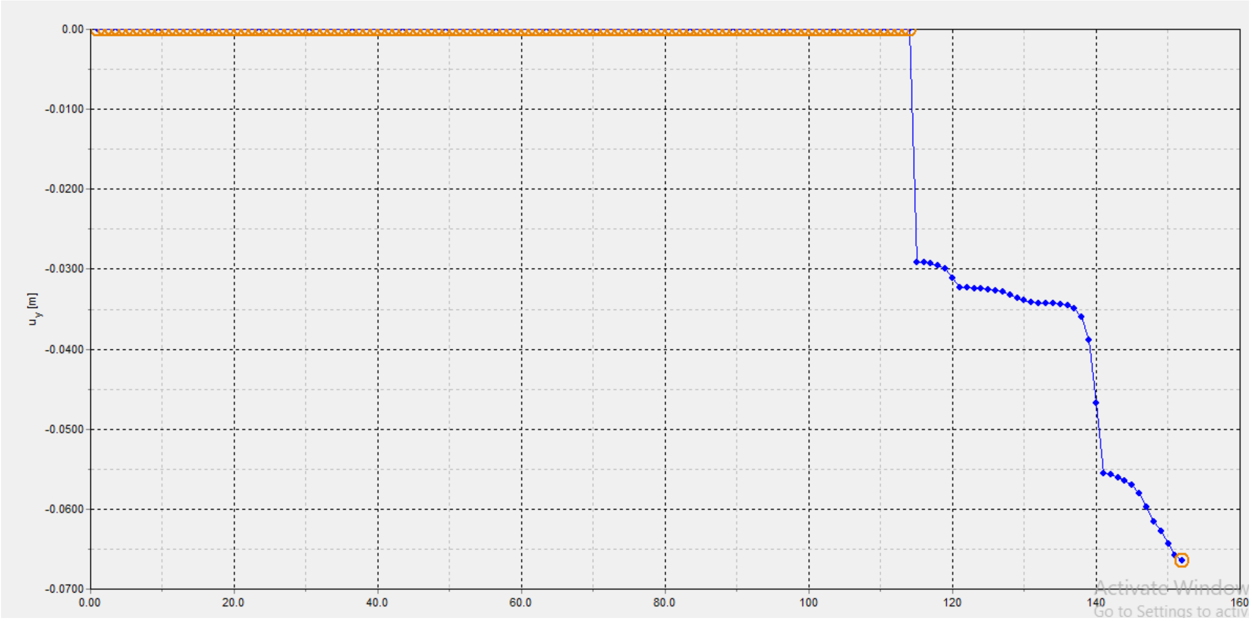
➢ **Case:3- 3 layers of geogrid**

Figure 5.5.3a shows settlement at the node (1.04,1.8) of location 1. It is found that the total settlement is 69mm.



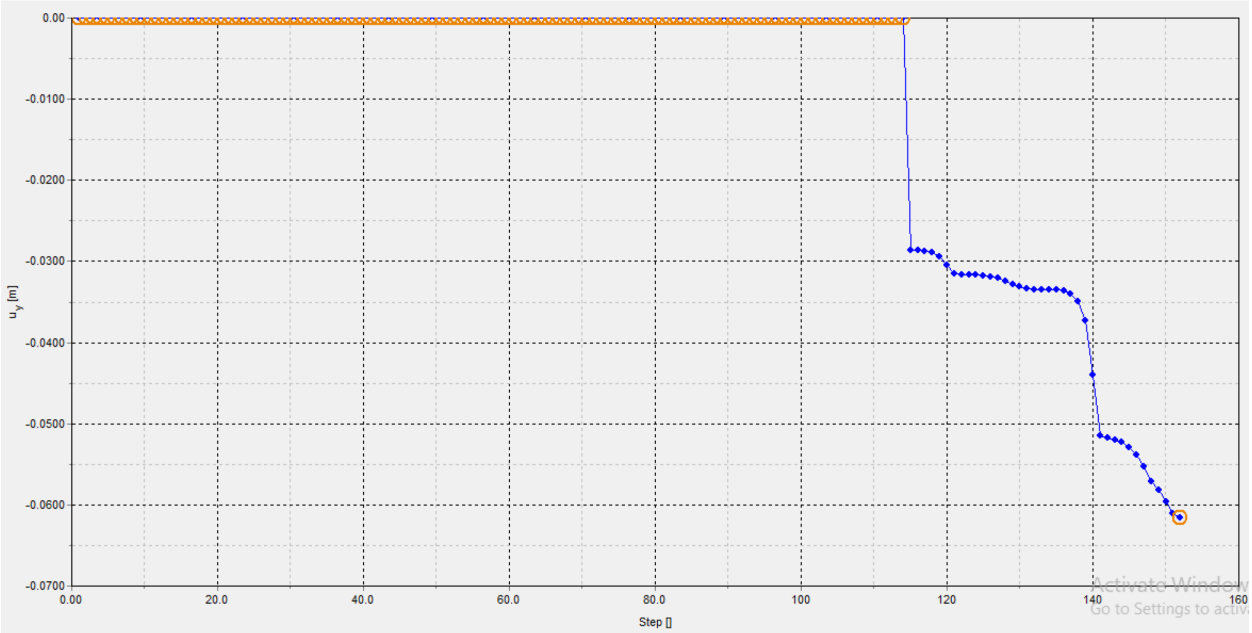
Graph 5.5.3a: Settlement vs Step at point (1.04, 1.8)

Figure 5.5.3b shows settlement at the node (2.0,1.8) of location 1. It is found that the total settlement is 66mm.



Graph 5.5.3b: Settlement vs Step at point (2.0, 1.8)

Figure 5.5.3c shows settlement at the node (2.98,1.8) of location 1. It is found that the total settlement is 62mm.



Graph 5.5.3c: Settlement vs Step at point (2.98, 1.8)

5.3.1.2 Location-2

Figure 5.6 shows total displacement of location 2. It is found that the total displacement is 93.71mm.

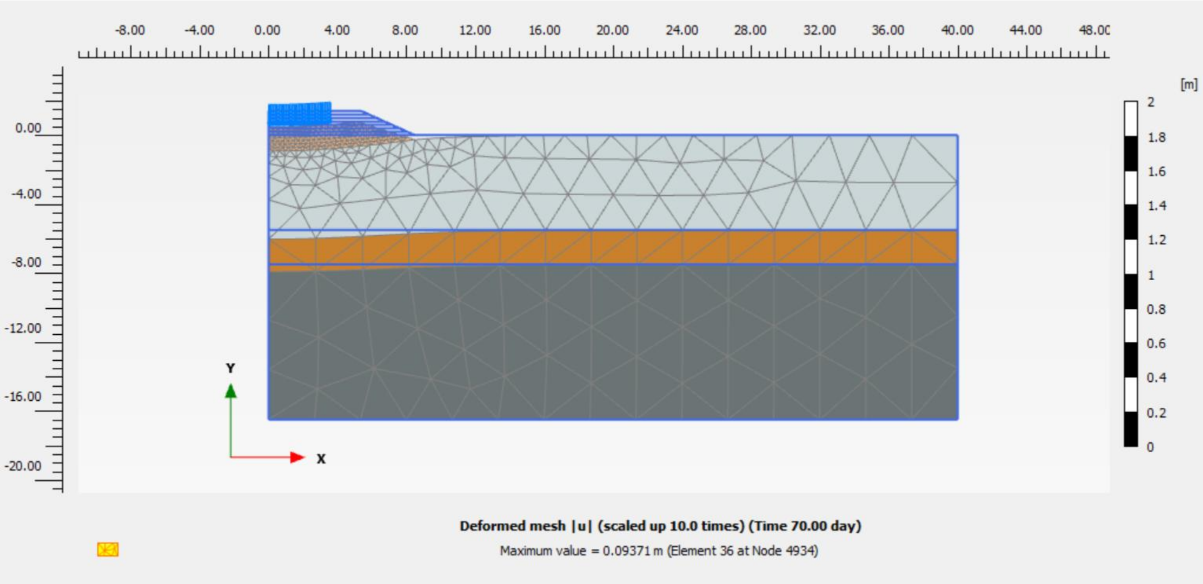
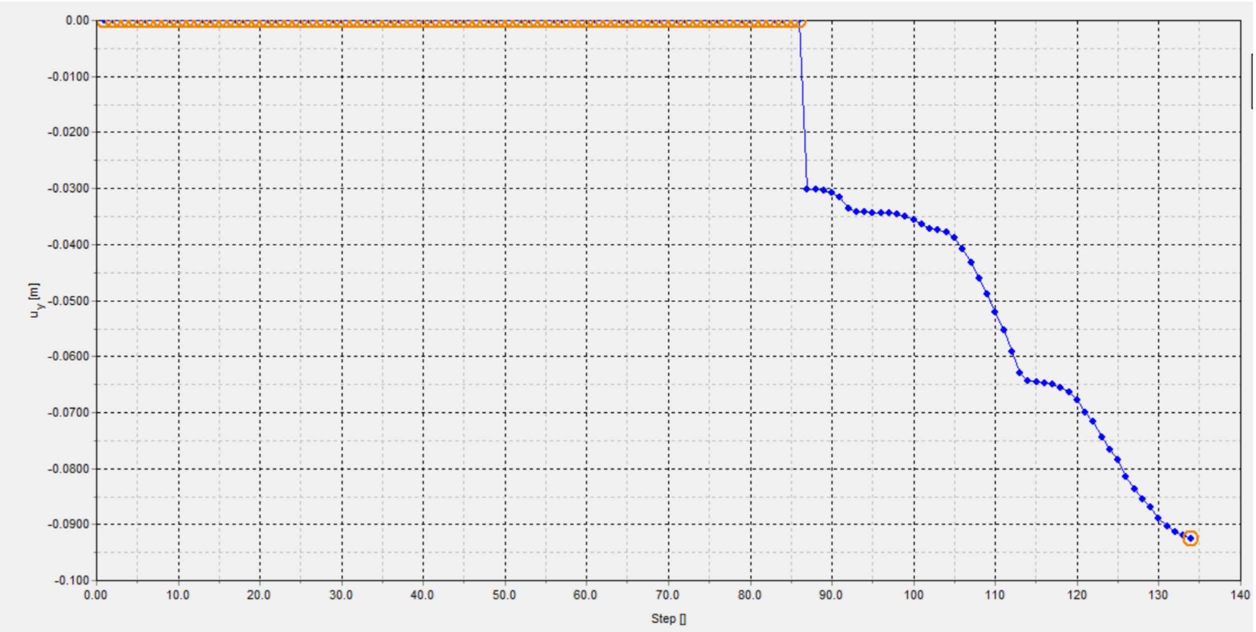


Figure 5.6: Deformed shape of Location-2 after surcharge

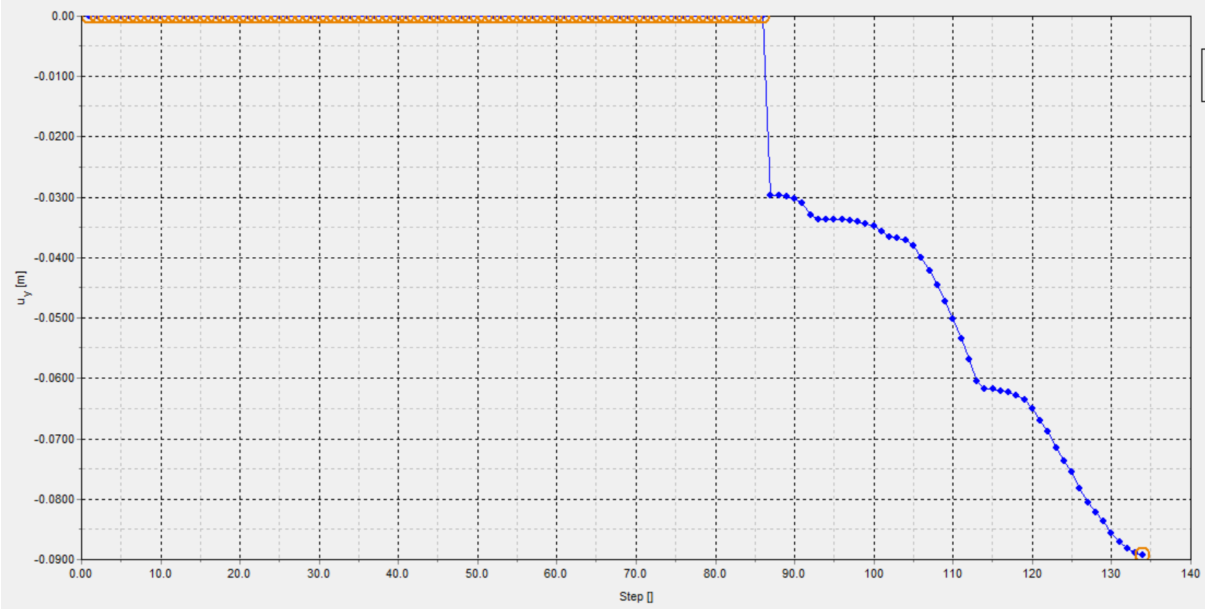
- **Case-1: No geogrid**

  Figure 5.6.1a shows settlement at the node (1.1,1.2) of location 2. It is found that the total settlement is 93.6mm.



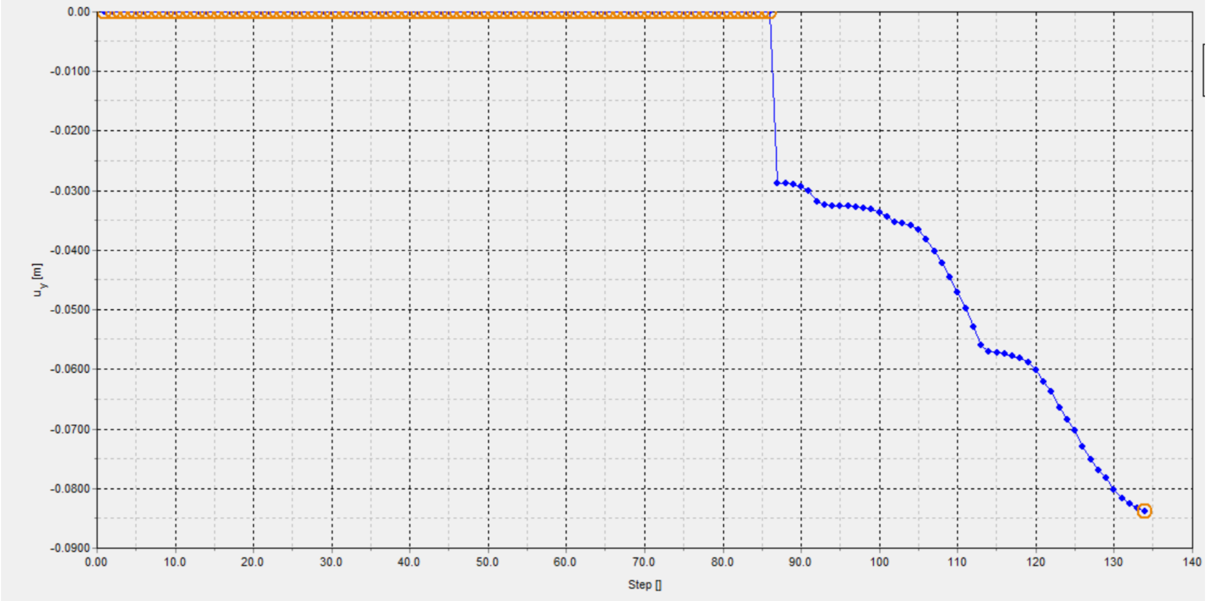
Graph 5.6.1a : Settlement vs Step at point (1.1,1.2)

Figure 5.6.1b shows settlement at the node (2.11,1.2) of location 2. It is found that the total settlement is 89mm.



Graph 5.6.1b: Settlement vs Step at point (2.11,1.2)

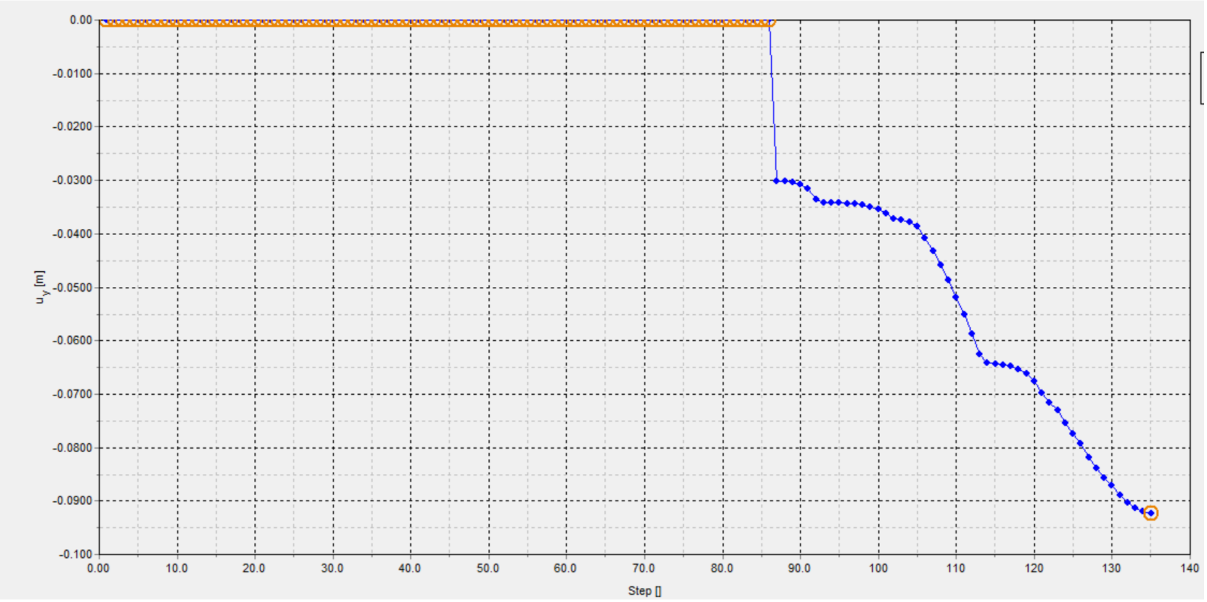
Figure 5.6.1c shows settlement at the node (3.09,1.2) of location 2. It is found that the total settlement is 84mm.



Graph 5.6.1c : Settlement vs Step at point (3.09,1.2)

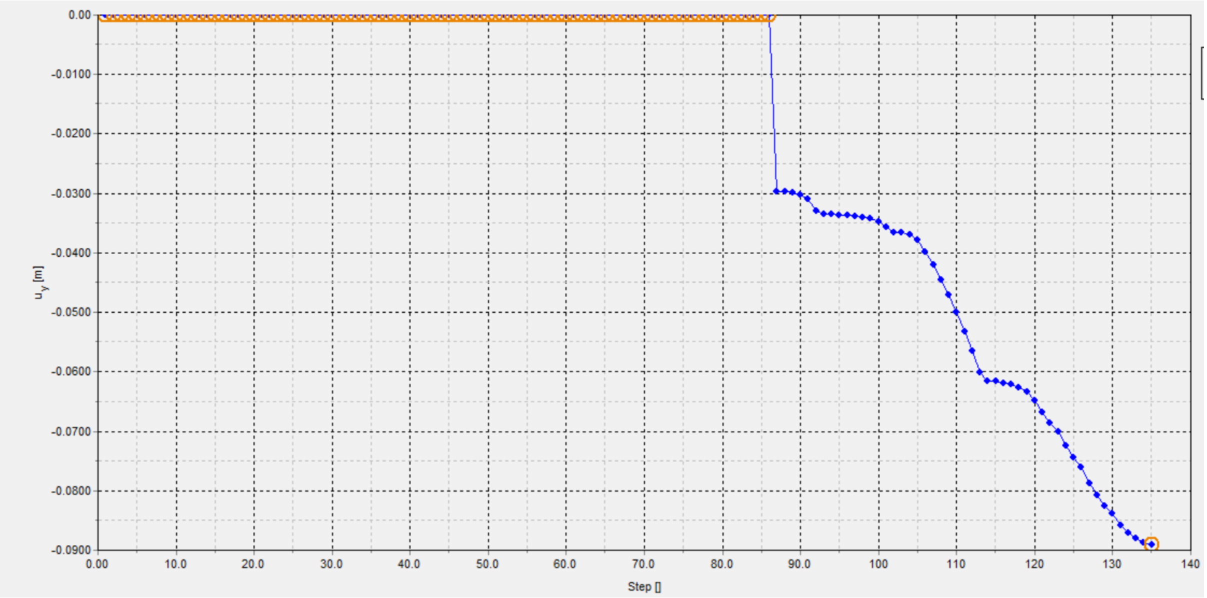
- **Case:2- 1 layer of geogrid**

Figure 5.6.2a shows settlement at the node (1.1,1.2) of location 2. It is found that the total settlement is 92.5mm.



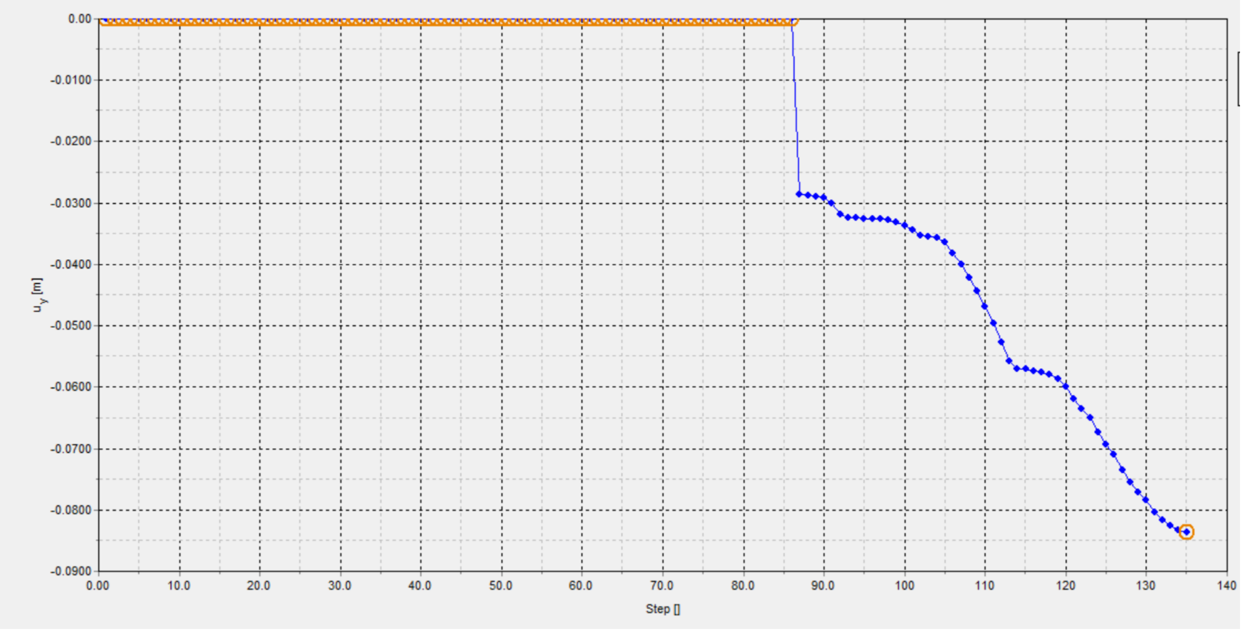
Graph 5.6.2a : Settlement vs Step at point (1.1,1.2)

Figure 5.6.2b shows settlement at the node (2.11,1.2) of location 2. It is found that the total settlement is 89mm.



Graph 5.6.2b: Settlement vs Step at point (2.11,1.2)

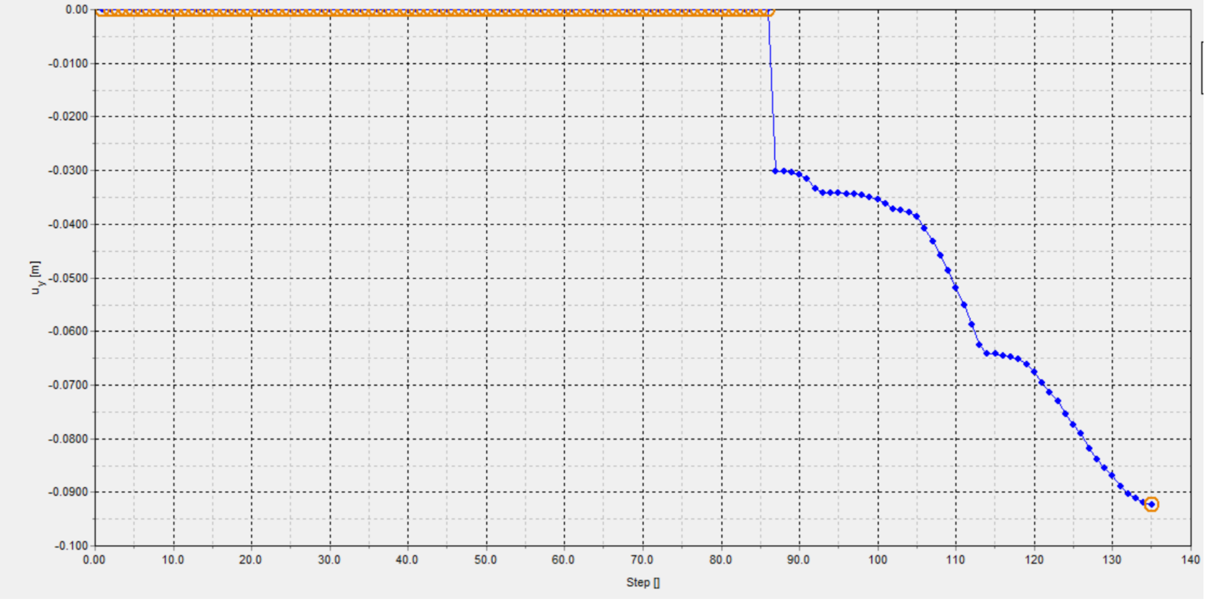
Figure 5.6.2c shows settlement at the node (3.09,1.2) of location 2. It is found that the total settlement is 84mm.



Graph 5.6.2c: Settlement vs Step at point (3.09,1.2)

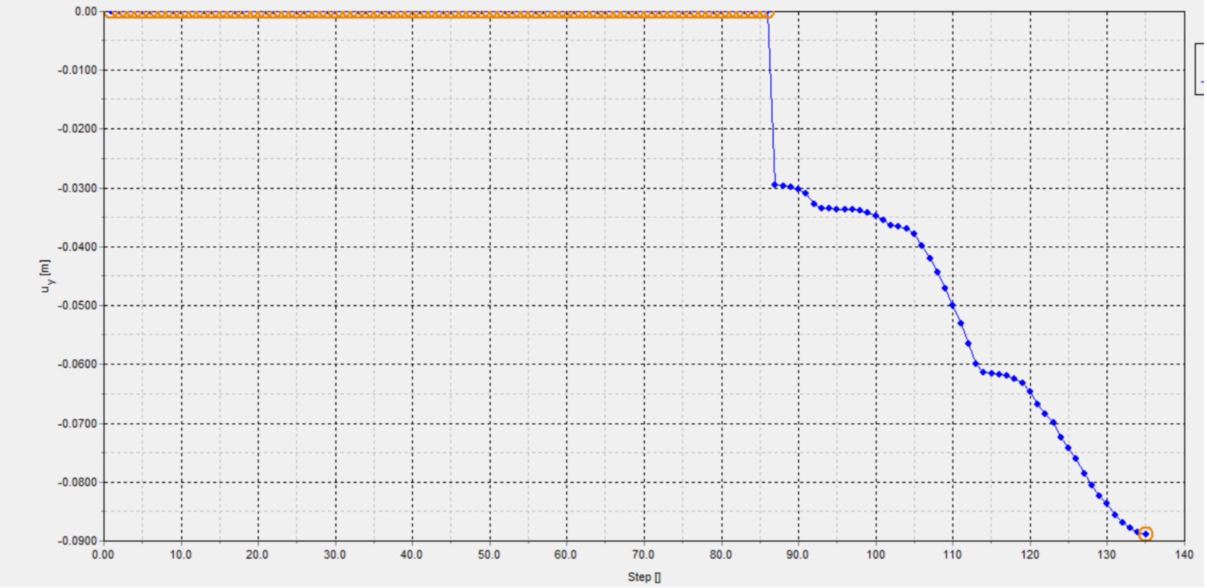
- **Case:3- 3 layers of geogrid**

Figure 5.6.3a shows settlement at the node (1.1,1.2) of location 2. It is found that the total settlement is 92.5mm.



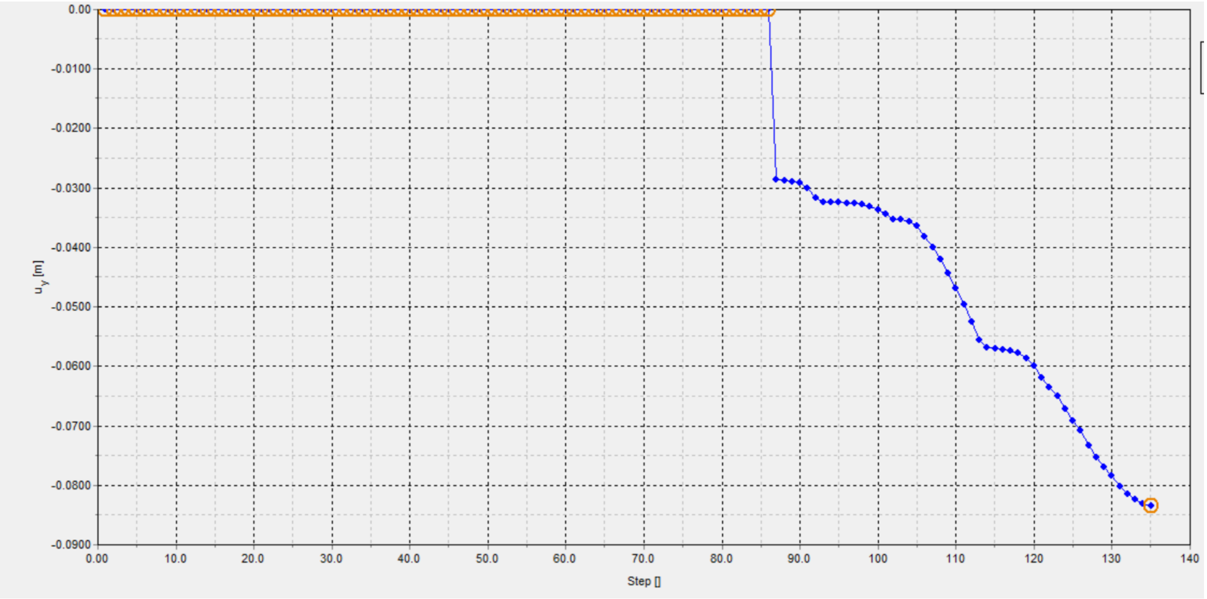
Graph 5.6.3a: Settlement vs Step at point (1.1,1.2)

Figure 5.6.3b shows settlement at the node (2.11,1.2) of location 2. It is found that the total settlement is 89mm.



Graph 5.6.3b: Settlement vs Step at point (2.11,1.2)

Figure 5.6.3c shows settlement at the node (3.09,1.2) of location 2. It is found that the total settlement is 84mm.



Graph 5.6.3c: Settlement vs Step at point (3.09,1.2)

5.3.2 Soft Soil Model

5.3.2.1 Location-1

Figure 5.7 shows total displacement of location 1. It is found that the total displacement is 32.45mm.

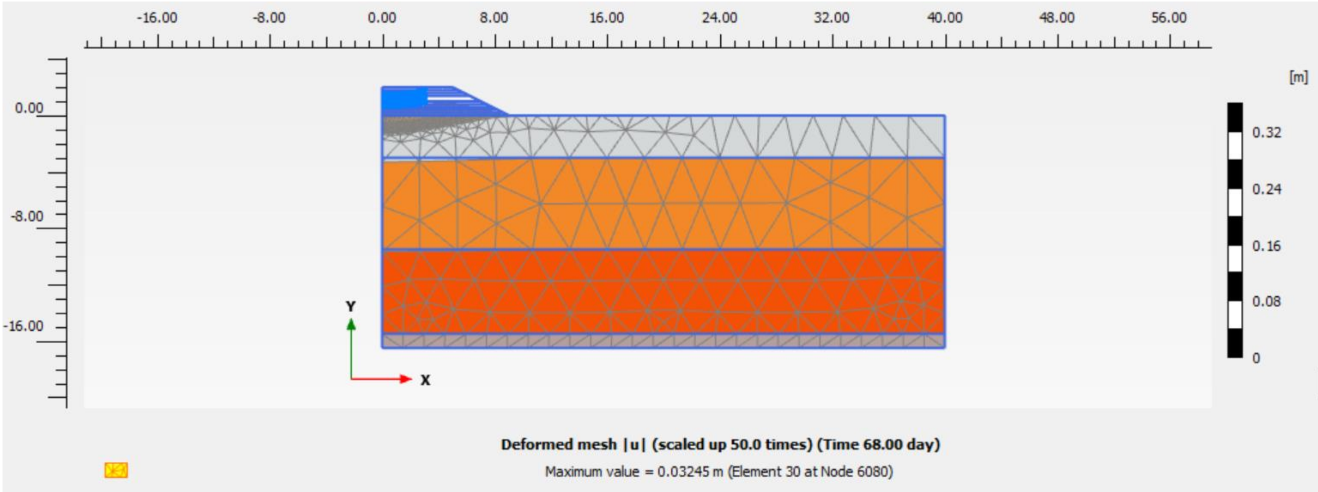
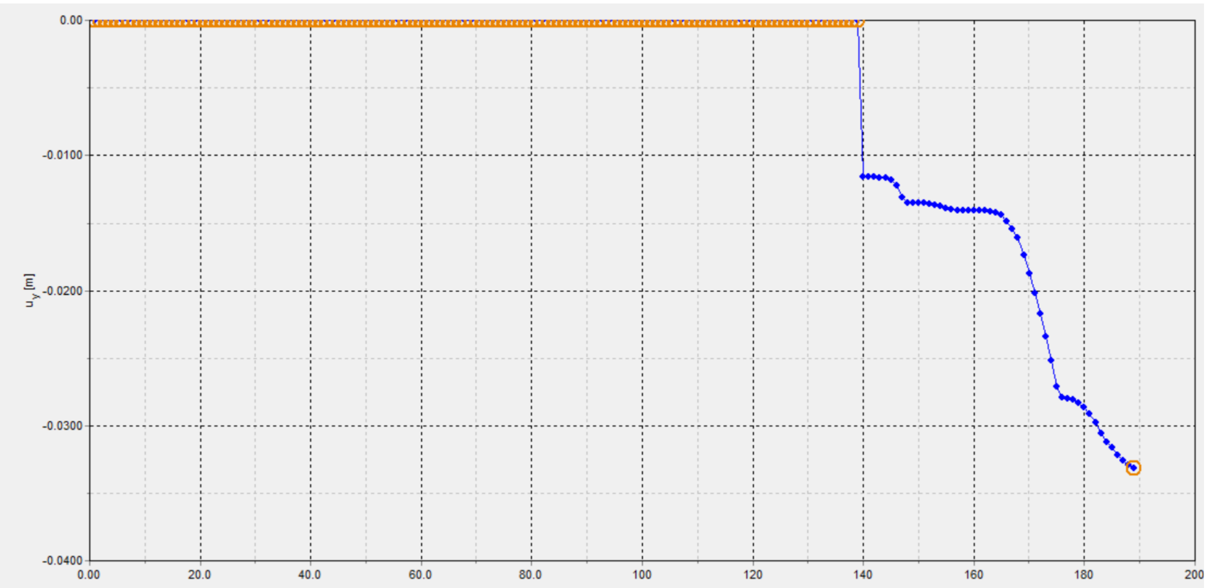


Figure 5.7: Deformed shape of Location-1 after surcharge

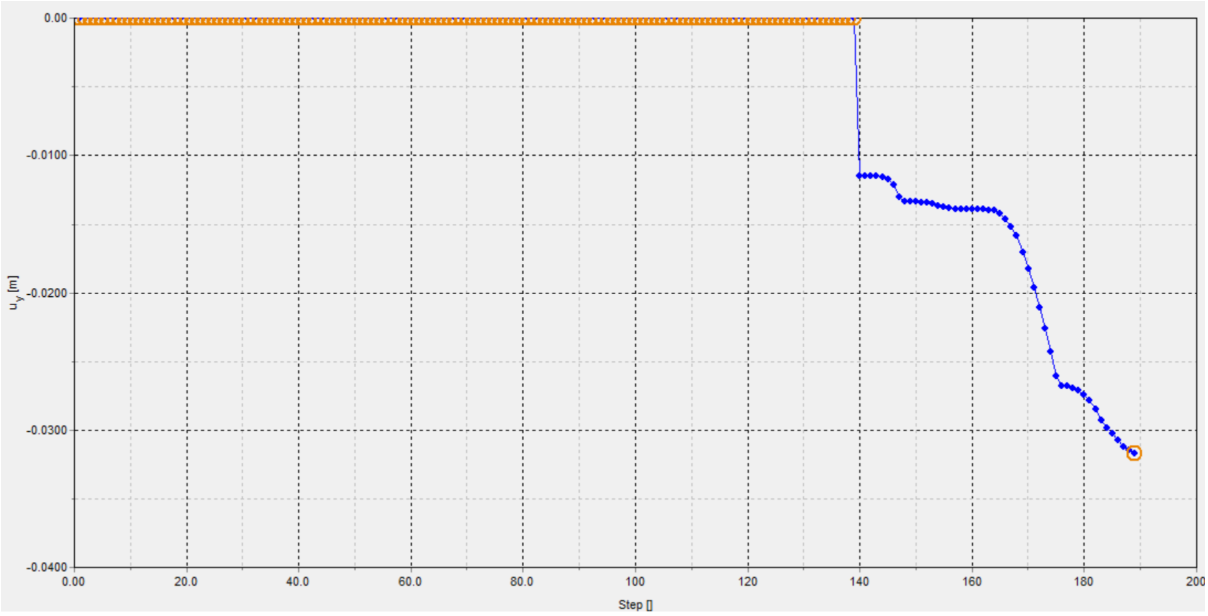
- **Case:1- No geogrid**

  Figure 5.7.1a shows settlement at the node (1.04, 1.8) of location 1. It is found that the total settlement is 33mm.



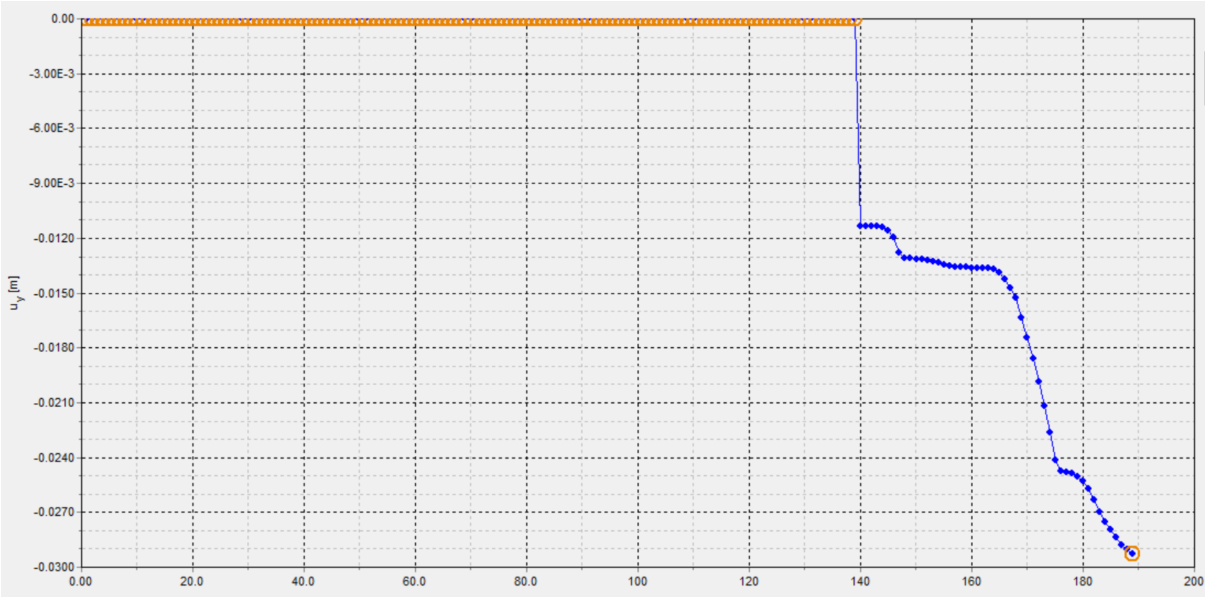
Graph 5.7.1a: Settlement vs Step at point (1.04, 1.8)

Figure 5.7.1b shows settlement at the node (2.0,1.8) of location 1. It is found that the total settlement is 32mm.



Graph 5.7.1b: Settlement vs Step at point (2.0, 1.8)

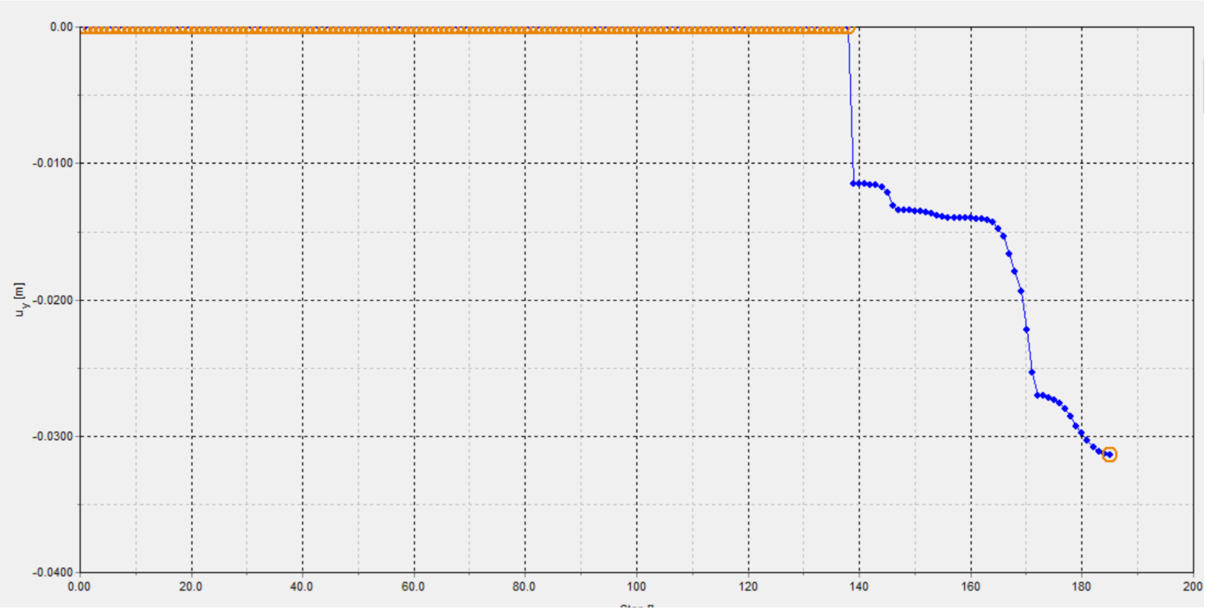
Figure 5.7.1c shows settlement at the node (2.98, 1.8) of location 1. It is found that the total settlement is 29mm.



Graph 5.7.1c: Settlement vs Step at point (2.98, 1.8)

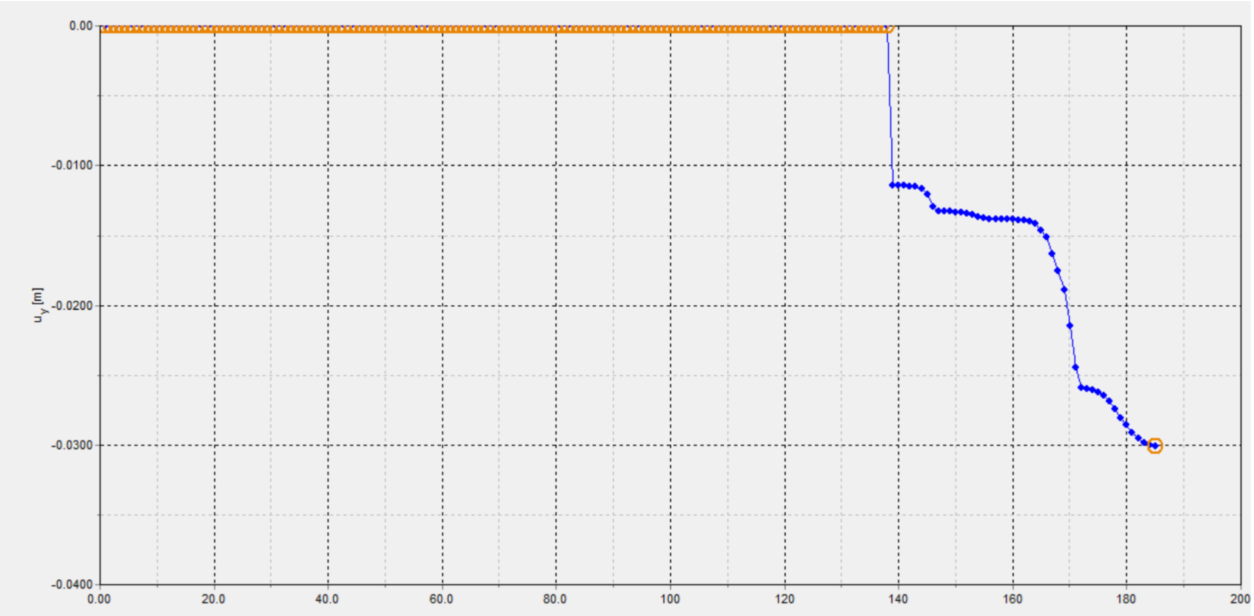
- **Case:2- 1 layer of geogrid**

  Figure 5.7.2a shows settlement at the node (1.04, 1.8) of location 1. It is found that the total settlement is 31mm.



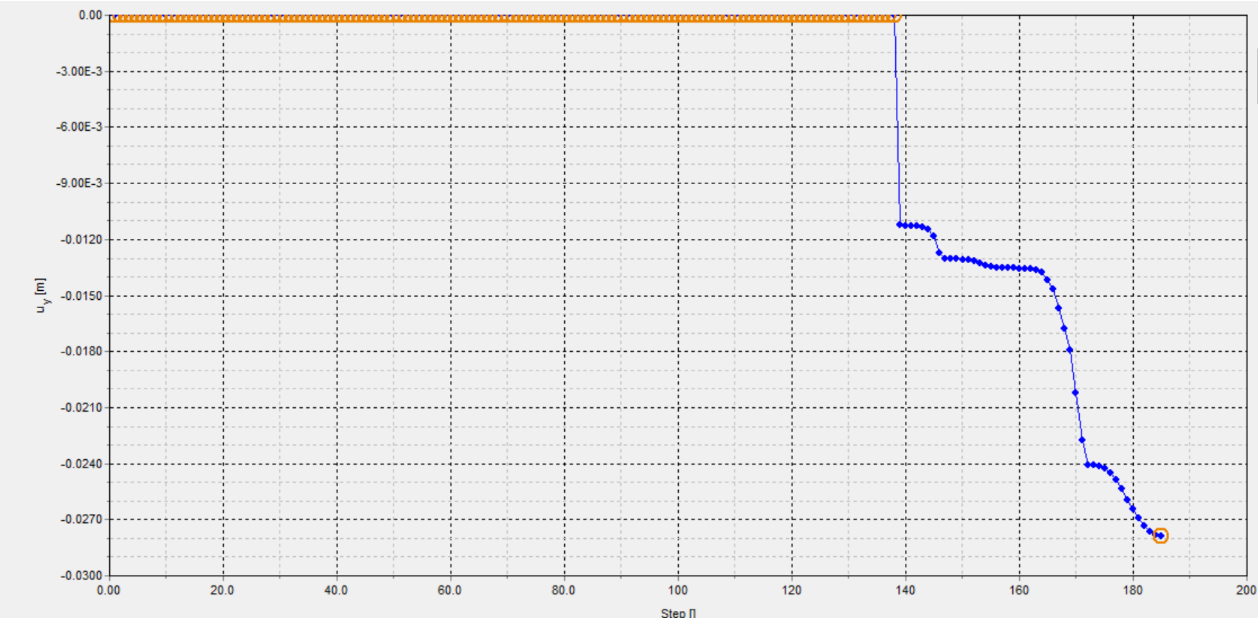
Graph 5.7.2a: Settlement vs Step at point (1.04, 1.8)

Figure 5.7.2b shows settlement at the node (2.0,1.8) of location 1. It is found that the total settlement is 30mm.



Graph 5.7.2b: Settlement vs Step at point (2.0, 1.8)

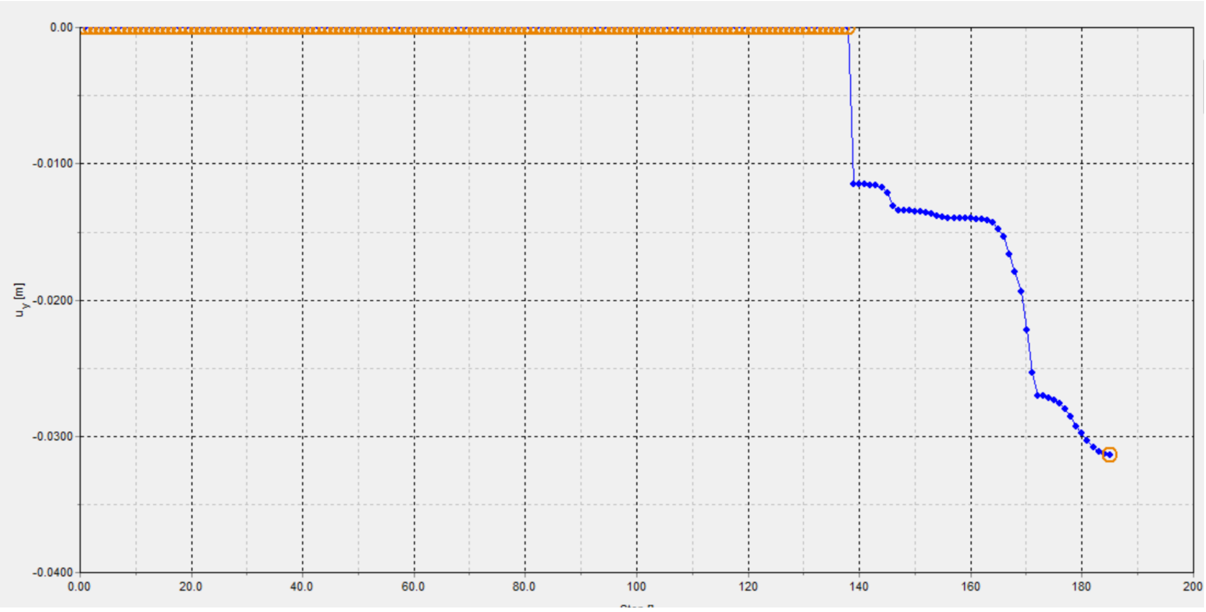
Figure 5.7.1c shows settlement at the node (2.98, 1.8) of location 1. It is found that the total settlement is 28mm.



Graph 5.7.1c: Settlement vs Step at point (2.98, 1.8)

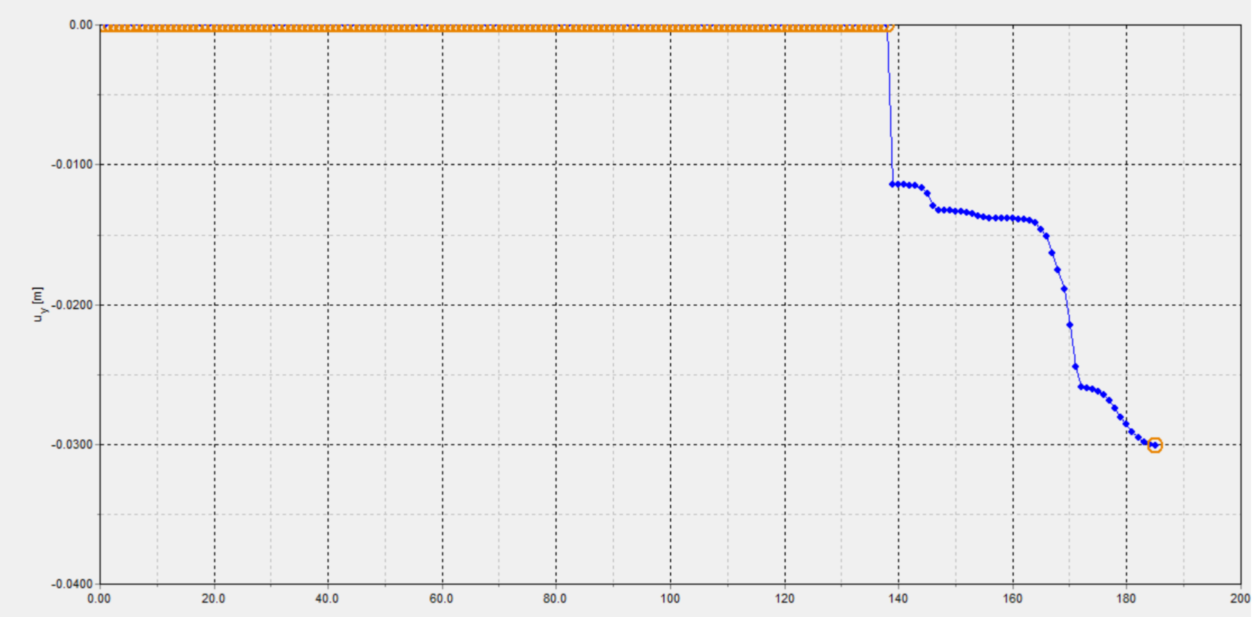
- **Case:3- 3 layers of geogrid**

Figure 5.7.3a shows settlement at the node (1.04, 1.8) of location 1. It is found that the total settlement is 31mm.



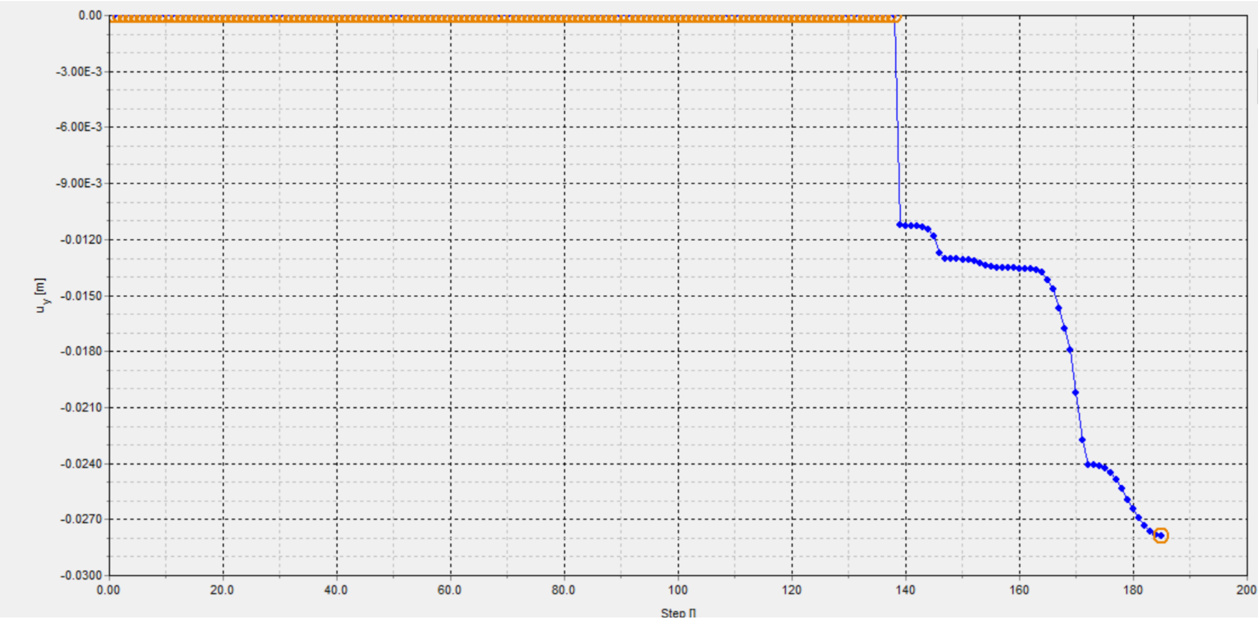
Graph 5.7.3a: Settlement vs Step at point (1.04, 1.8)

Figure 5.7.3b shows settlement at the node (2.0,1.8) of location 1. It is found that the total settlement is 30mm.



Graph 5.7.3b: Settlement vs Step at point (2.0, 1.8)

Figure 5.7.3c shows settlement at the node (2.98, 1.8) of location 1. It is found that the total settlement is 28mm.



Graph 5.7.3c: Settlement vs Step at point (2.98, 1.2)

5.3.2.2 Location-2

Figure 5.8 shows total displacement of location 2. It is found that the total displacement is 51.81mm.

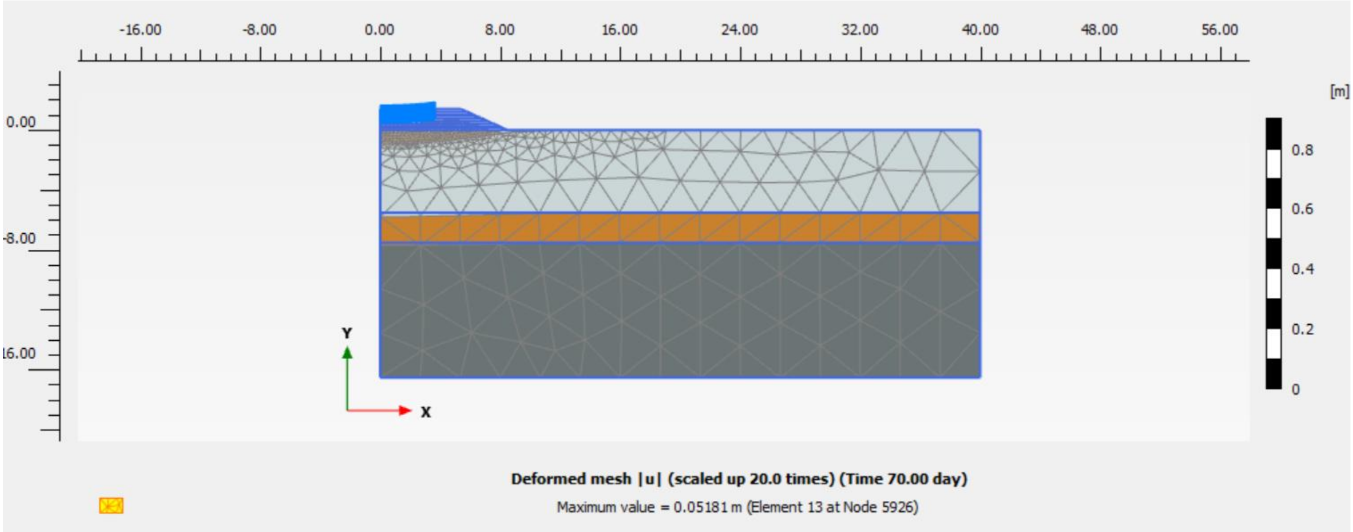
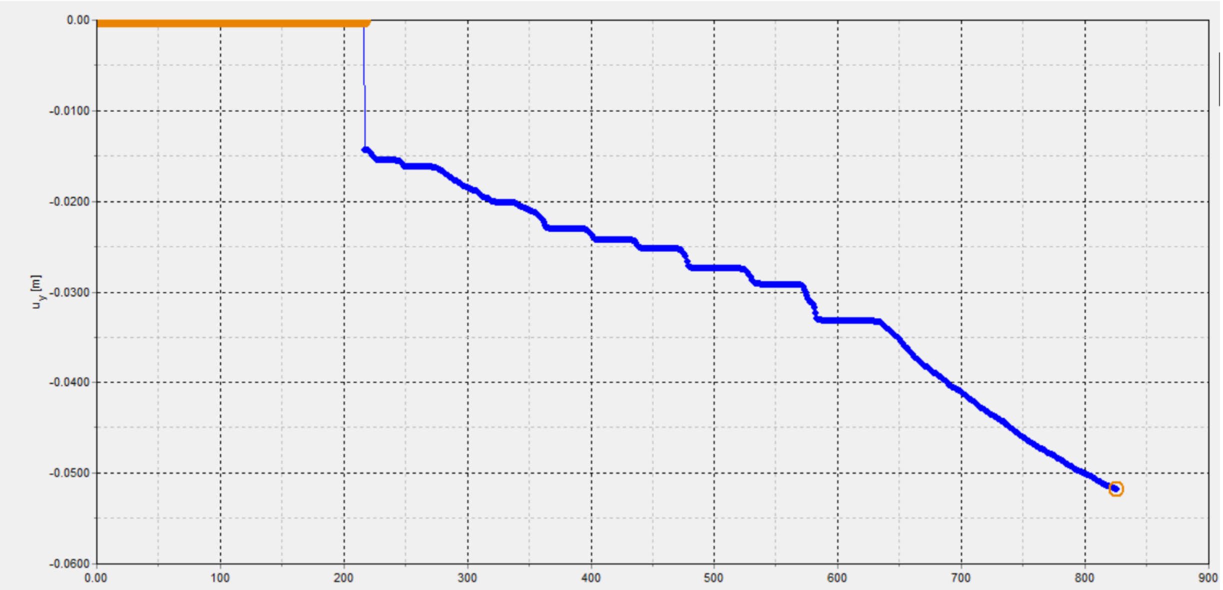


Figure 5.8: Deformed shape of Location-2 after surcharge

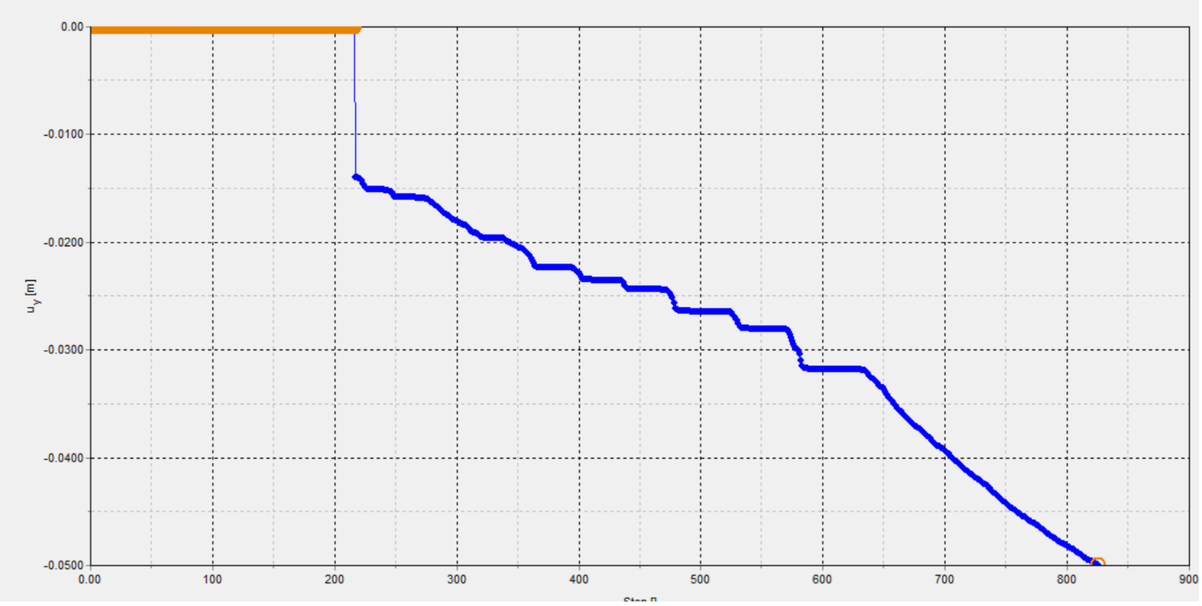
- **Case-1: No geogrid**

  Figure 5.8.1a shows settlement at the node (1.1,1.2) of location 2. It is found that the total settlement is 52mm.



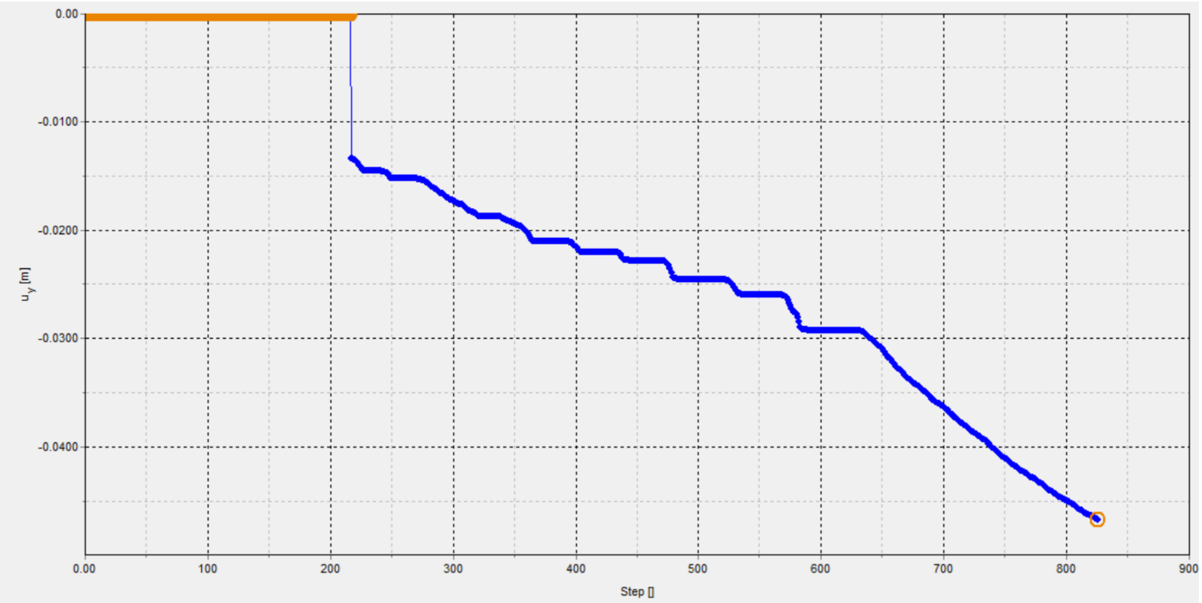
Graph 5.8.1a: Settlement vs Step at point (1.1, 1.2)

Figure 5.8.1b shows settlement at the node (2.11,1.2) of location 2. It is found that the total settlement is 50mm.



Graph 5.8.1b: Settlement vs Step at point (2.11, 1.2)

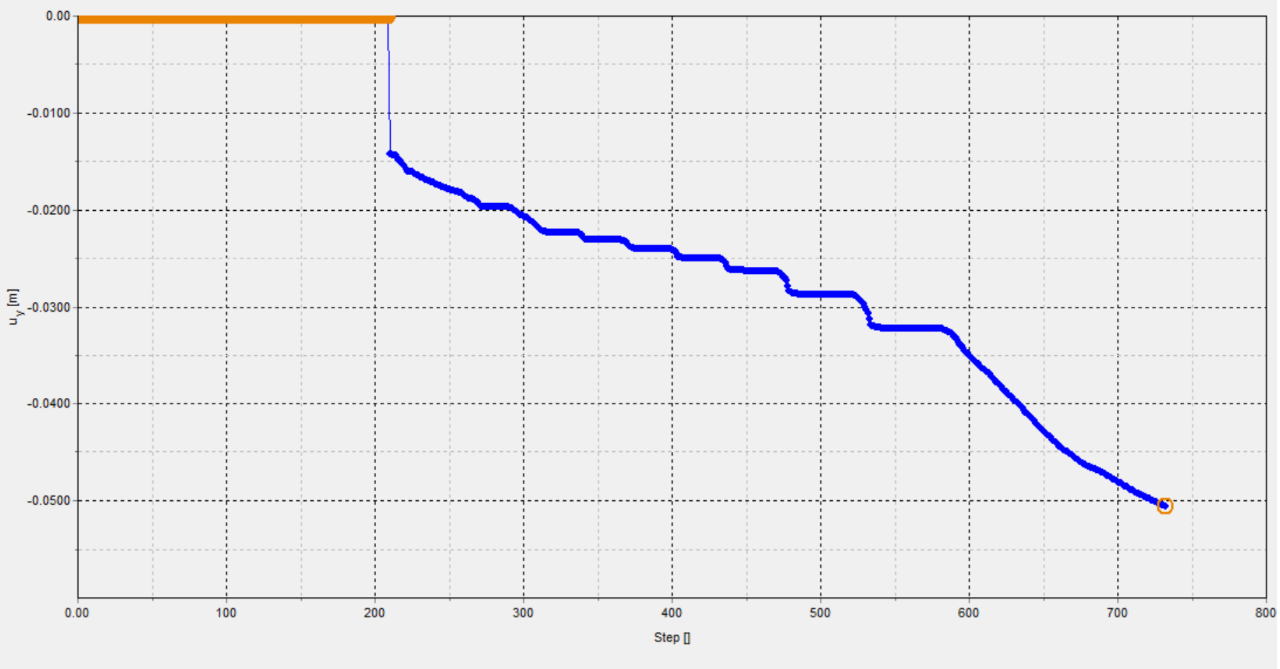
Figure 5.8.1c shows settlement at the node (3.09,1.2) of location 2. It is found that the total settlement is 47mm.



Graph 5.8.1c: Settlement vs Step at point (3.09, 1.2)

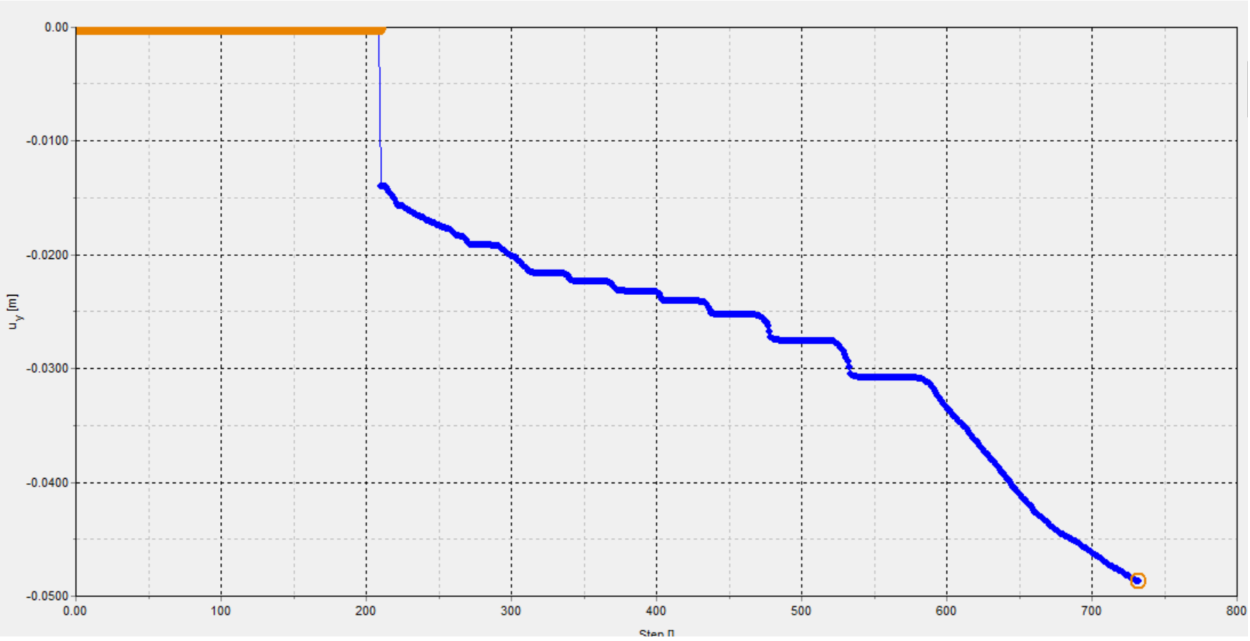
- **Case:2- 1 layer of geogrid**

Figure 5.8.2a shows settlement at the node (1.1,1.2) of location 2. It is found that the total settlement is 50mm.



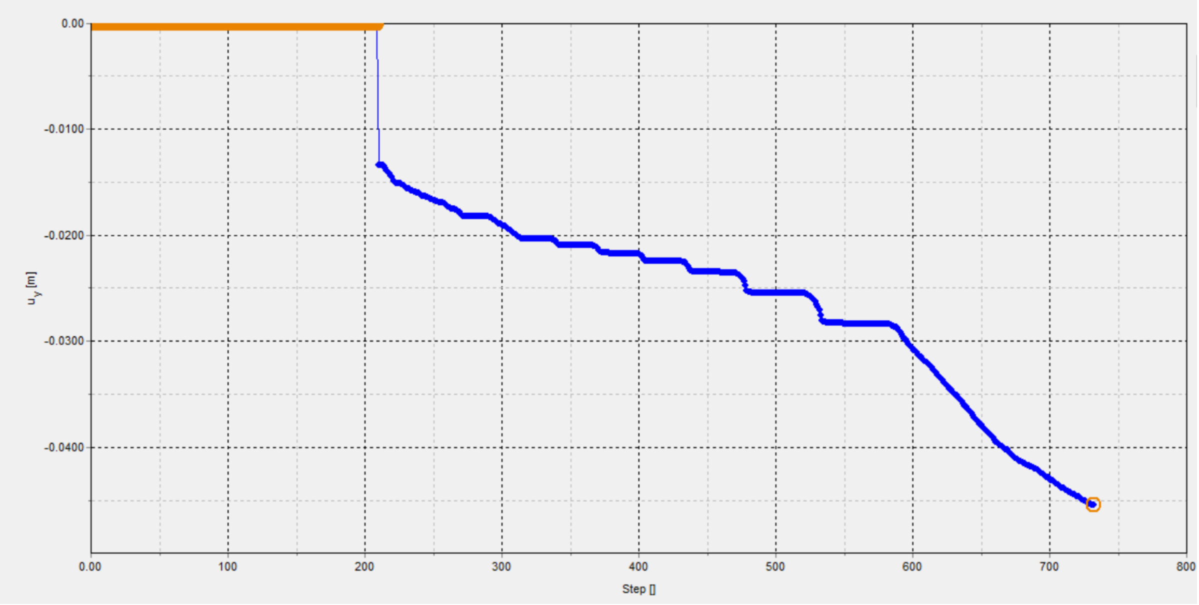
Graph 5.8.2a: Settlement vs Step at point (1.1, 1.2)

Figure 5.8.2b shows settlement at the node (2.11,1.2) of location 2. It is found that the total settlement is 49mm.



Graph 5.8.2b: Settlement vs Step at point (2.11, 1.2)

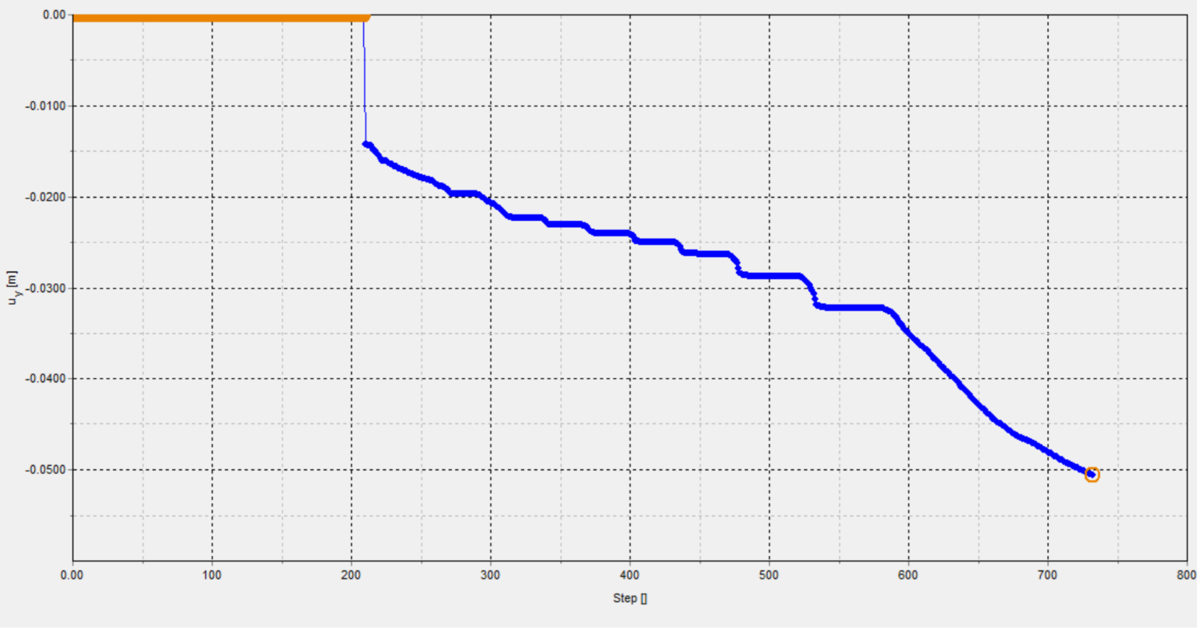
Figure 5.8.2c shows settlement at the node (3.09,1.2) of location 2. It is found that the total settlement is 45mm.



Graph 5.8.2c: Settlement vs Step at point (3.09, 1.2)

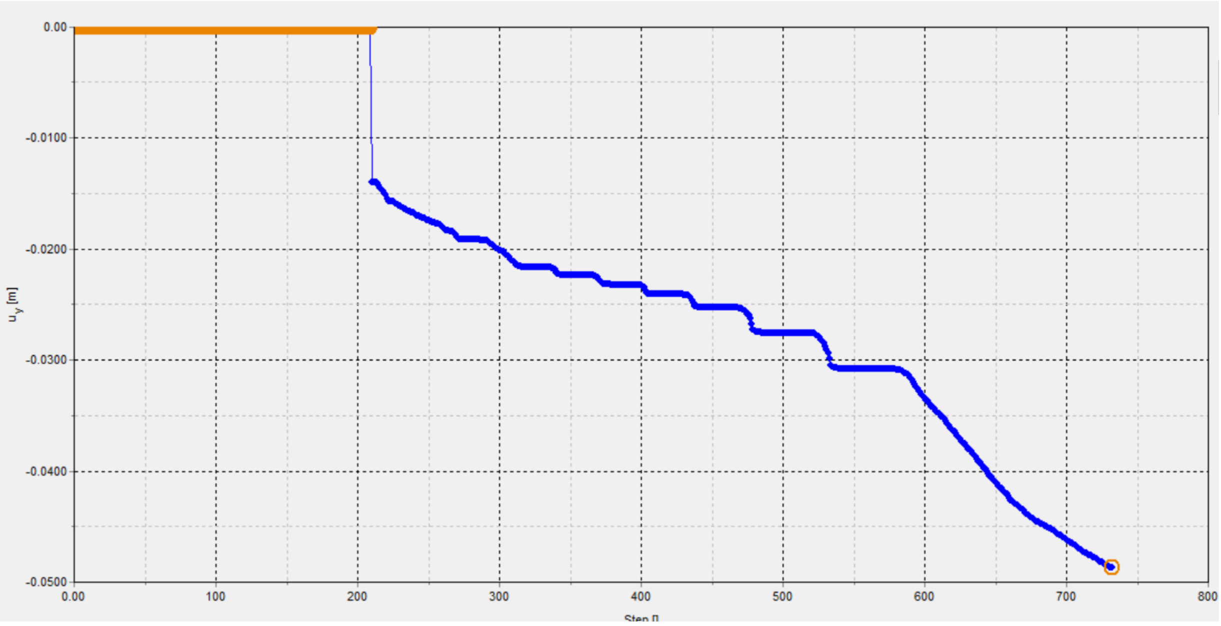
- **Case:3- 3 layers of geogrid**

  Figure 5.8.3a shows settlement at the node (1.1,1.2) of location 2. It is found that the total settlement is 50mm.



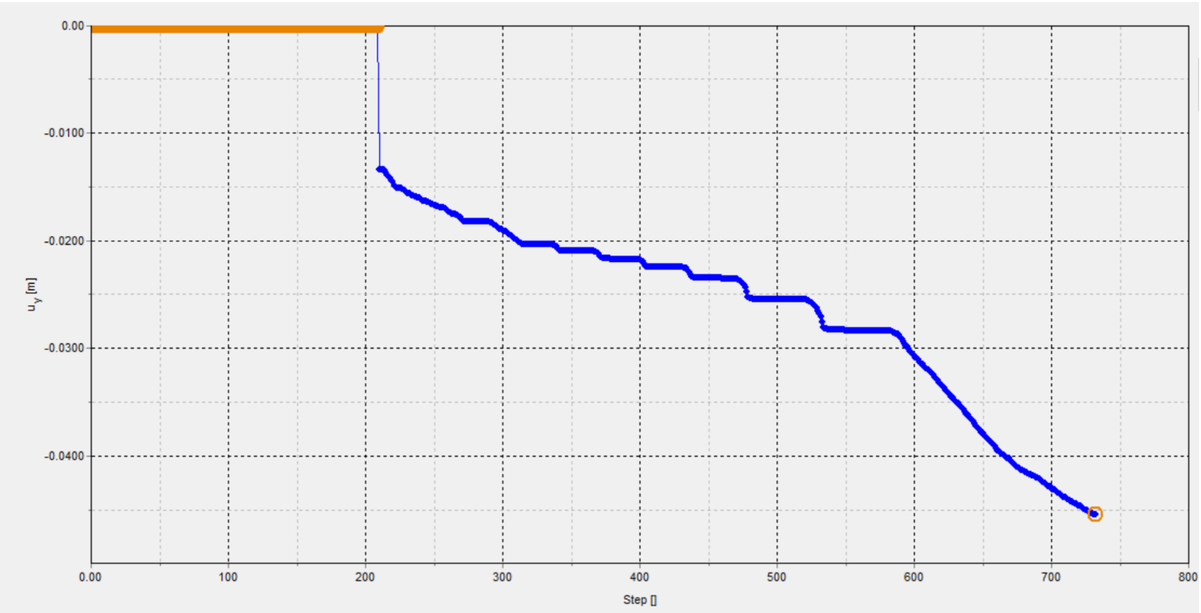
Graph 5.8.3a: Settlement vs Step at point (1.1, 1.2)

Figure 5.8.3b shows settlement at the node (2.11,1.2) of location 2. It is found that the total settlement is 49mm.



Graph 5.8.3b: Settlement vs Step at point (2.11, 1.2)

Figure 5.8.3c shows settlement at the node (3.09,1.2) of location 2. It is found that the total settlement is 45mm.



Graph 5.8.3c: Settlement vs Step at point (3.09, 1.2)

5.4 Factor of safety Analysis

- **Model used: Soft soil model**

5.4.1 Location-1

- **Case 1: Without Geogrid**

Figure 5.4.1a shows total displacement of location 1. It is found that the total displacement is 157.2m.

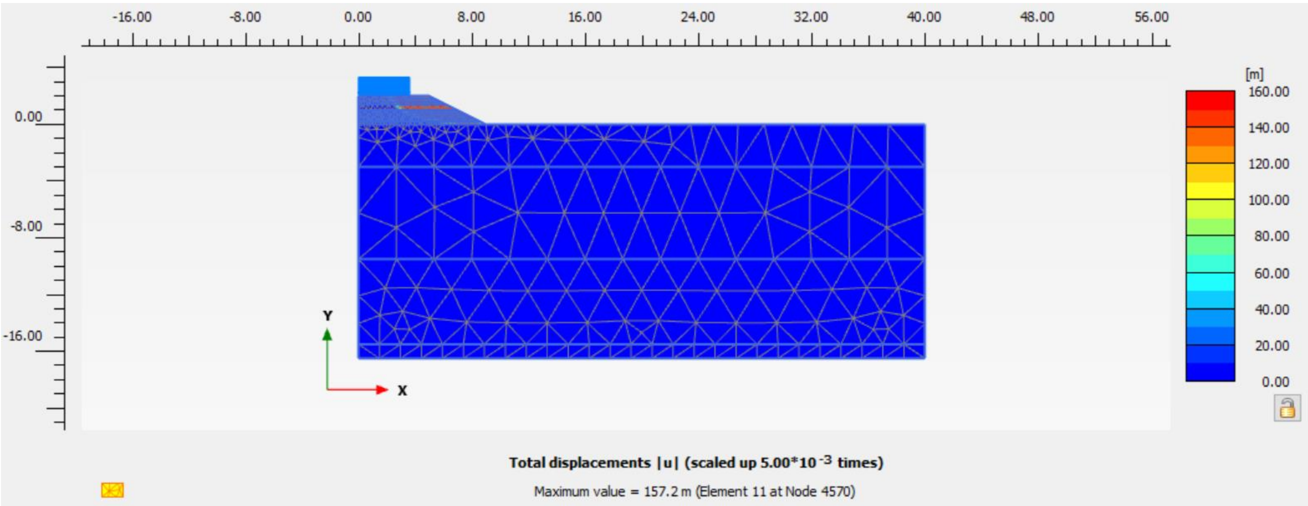
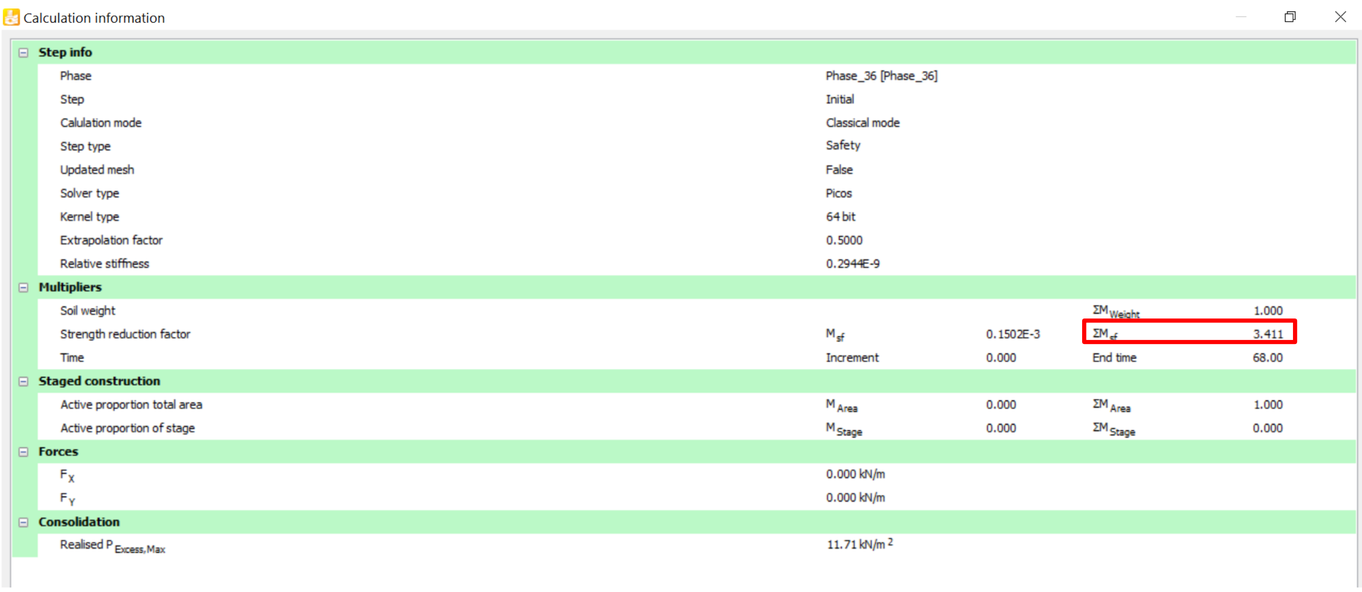


Figure 5.4.1a: Displacement without Geogrid

Figure 5.4.1b shows factor of safety of location 1. It is found that the factor of safety is 3.411 which is greater than 1. So, it is safe.


Calculation information

Step info				
Phase	Phase_36 [Phase_36]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	0.5000			
Relative stiffness	0.2944E-9			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	0.1502E-3	ΣM_{sf}	3.411
Time	Increment	0.000	End time	68.00
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	1.000
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	11.71 kN/m^2			

Figure 5.4.1a: Calculation information of Factor of safety

➢ Case 2: With Geogrid

Figure 5.4.1c shows total displacement of location 1. It is found that the total displacement is 2.557m.

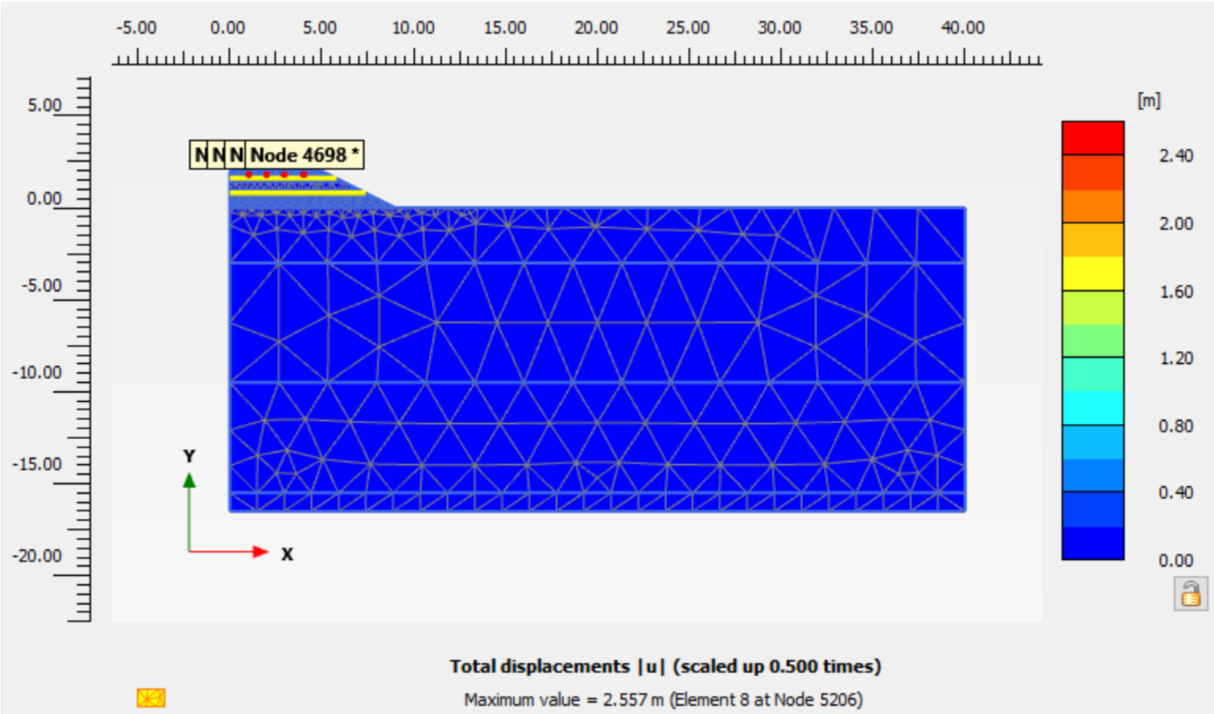


Figure 5.4.1b: Displacement with Geogrid

Figure 5.4.1d shows factor of safety of location 1. It is found that the factor of safety is 4.321 which is greater than 1. So, it is safe.

Calculation information

Step info				
Phase	Phase_36 [Phase_36]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	0.5000			
Relative stiffness	-5.103E-9			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	-0.5409E-3	ΣM_{sf}	4.321
Time	Increment	0.000	End time	68.00
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	1.000
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	20.34 kN/m^2			

Figure 5.4.1. c: Calculation information of Factor of safety

5.4.2 Location-2

- **Case 1: Without Geogrid**

  Figure 5.4.2a shows total displacement of location 2. It is found that the total displacement is 37.11m.

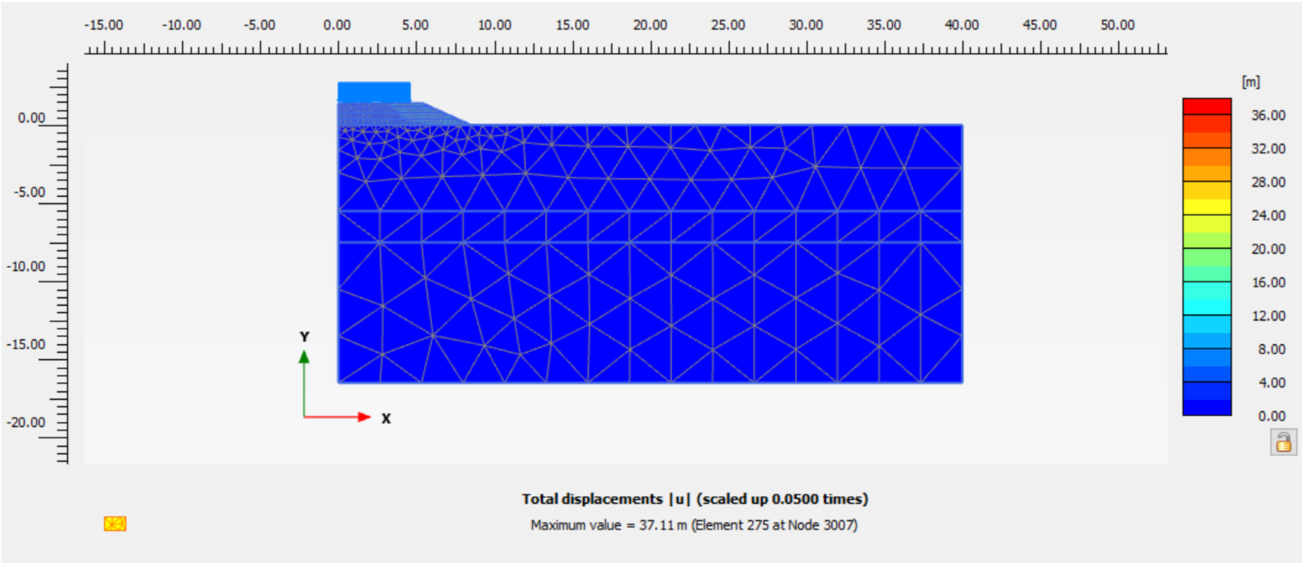


Figure 5.4.2a: Displacement without Geogrid

Figure 5.4.2b shows factor of safety of location 2. It is found that the factor of safety is 3.517 which is greater than 1. So, it is safe.

Step info				
Phase	Phase_28 [Phase_28]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	1.000			
Relative stiffness	1.712E-9			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	0.05143E-3	ΣM_{sf}	3.517
Time	Increment	0.000	End time	70.00
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	1.000
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	17.65 kN/m^2			

Figure 5.4.2b: Calculation information of Factor of safety

- **Case 2: With Geogrid**

Figure 5.4.2c shows total displacement of location 2. It is found that the total displacement is 0.0656m.

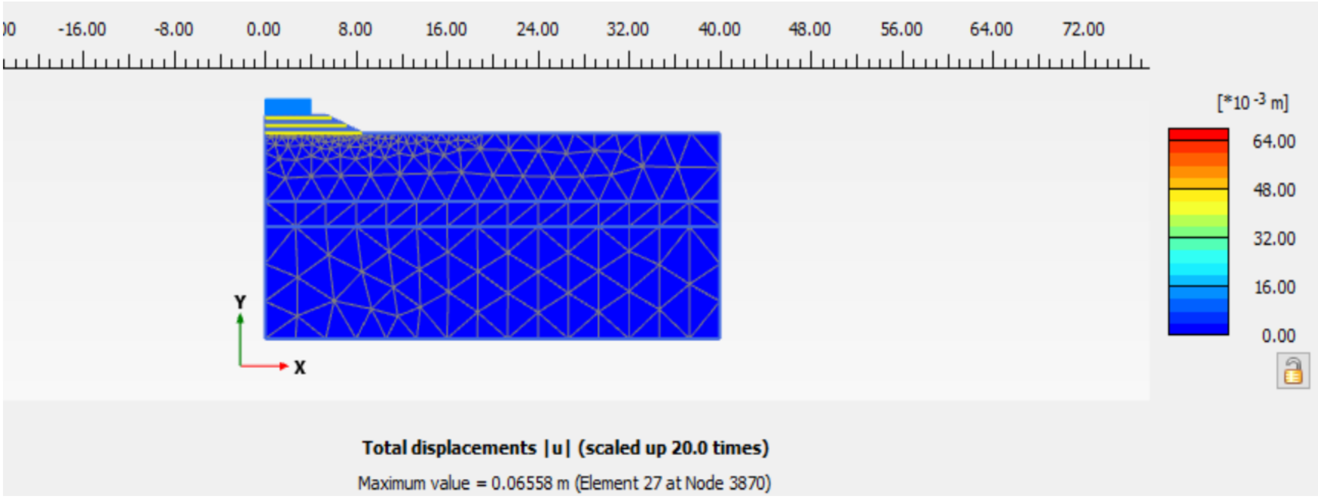


Figure 5.4.2c: Displacement with Geogrid

Figure 5.4.2d shows factor of safety of location 2. It is found that the factor of safety is 4.738 which is greater than 1. So, it is safe.

Step info				
Phase	Phase_28 [Phase_28]			
Step	Initial			
Calulation mode	Classical mode			
Step type	Safety			
Updated mesh	False			
Solver type	Picos			
Kernel type	64 bit			
Extrapolation factor	0.5000			
Relative stiffness	0.01555E-3			
Multipliers				
Soil weight			ΣM_{Weight}	1.000
Strength reduction factor	M_{sf}	-0.04943	ΣM_{sf}	4.738
Time	Increment	0.000	End time	70.00
Staged construction				
Active proportion total area	M_{Area}	0.000	ΣM_{Area}	1.000
Active proportion of stage	M_{Stage}	0.000	ΣM_{Stage}	0.000
Forces				
F_X	0.000 kN/m			
F_Y	0.000 kN/m			
Consolidation				
Realised $P_{Excess,Max}$	20.76 kN/m 2			

Figure 5.4.2. d: Calculation information of Factor of safety

5.5 Conventional Method

- **Location 1**

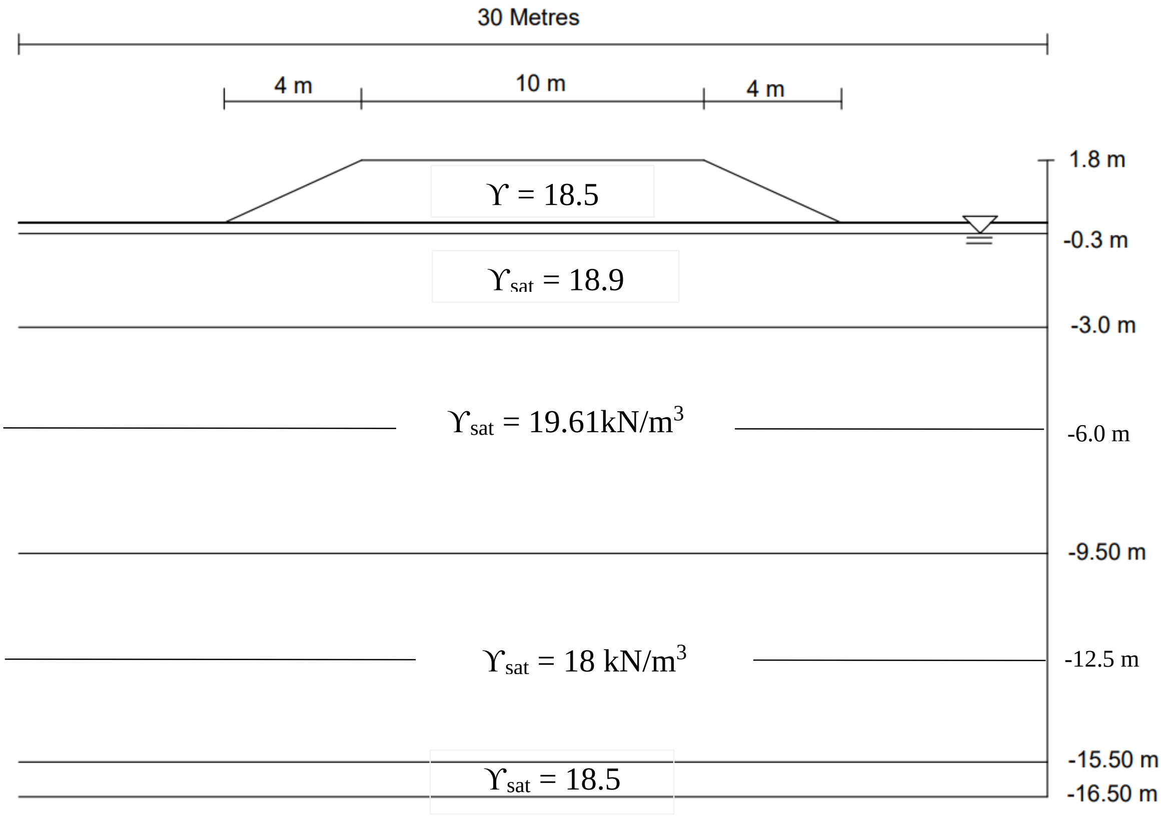


Figure 5.5. a: Section-view of Location-1

Now we are going to calculate the total settlement using the conventional method

$\gamma H = 18.5*1.8 = 33.3$ kN/m^2

$\Delta\sigma + \sigma_{01}' = 35.79$ kN/m^2

$\Delta\sigma + \sigma_{02}' = 62.415$ kN/m^2

$\Delta\sigma + \sigma_{03}' = 95.69263$ kN/m^2

$\Delta\sigma + \sigma_{04}' = 119.842$ kN/m^2

$\Delta\sigma + \sigma_{05}' = 144.1542$ kN/m^2

$\Delta\sigma + \sigma_{06}' = 152.904$ kN/m^2

For first layer:

$\sigma_{01}' = (0.3*18.9) + \{2.7*(18.9 - 9.81)\} = 30.24$ kN/m^2

For second layer,

$\sigma_{02}' = (0.3*18.9) + \{2.7*(18.9 - 9.81)\} + (19.61 – 9.81)*3 = 59.64$ kN/m^2

For third layer,

$\sigma_{03}' = (0.3*18.9) + \{2.7*(18.9 - 9.81)\} + (19.61 – 9.81)*3 + (19.61-9.81)*3.5 = 93.94$ kN/m^2

For fourth layer,

$\sigma_{04}' = (0.3*18.9) + \{2.7*(18.9 - 9.81)\} + (19.61 – 9.81)*3 + (19.61-9.81)*3.5 + (18-9.81)*3$

$= 118.51$ kN/m^2

For fifth layer,

$\sigma_{05}' = (0.3*18.9) + \{2.7*(18.9 - 9.81)\} + (19.61 – 9.81)*3 + (19.61-9.81)*3.5 + (18-9.81)*3$

$+ (18-9.81)*3$

$= 143.08$ kN/m^2

For sixth layer,

$\sigma_{06}' = (0.3*18.9) + \{2.7*(18.9 - 9.81)\} + (19.61 – 9.81)*3 + (19.61-9.81)*3.5 + (18-9.81)*3$

$+ (18-9.81)*3 + (18.5-9.81)*1$

$= 151.895$ kN/m^2

For Consolidation Settlement, $S_c = \frac{HCc}{(1+e0)}\log_{10}(\frac{\bar{\sigma}+\Delta\bar{\sigma}}{\bar{\sigma}0})$

For first layer,

$S_{c1} = \frac{3*0.254}{1+0.941}\log\frac{35.79}{30.24} = 0.028729$ m

For second layer,

$S_{c2} = \frac{3*0.081}{1+0.773}\log\frac{62.415}{59.64} = 0.002707036$ m

For third layer,

$S_{c3} = \frac{3.5*0.081}{1+0.773}\log\frac{95.758}{94.005} = 0.00128304$ m

For fourth layer,

$S_{c4} = \frac{3*0.077}{1+0.731}\log\frac{119.842}{118.51} = 0.00134601$ m

For fifth layer,

$S_{c5} = \frac{3*0.077}{1+0.731}\log\frac{144.279}{143.205} = 0.00089981$ m

For sixth layer,

$S_{c6} = \frac{1*0.16}{1+0.769}\log\frac{152.904}{151.895} = 0.0001308530616$ m

$S_c = S_{c1} + S_{c2} + S_{c3} + S_{c4} = 0.035095749$ m

S_c = 35.1 mm

- **Location 2**

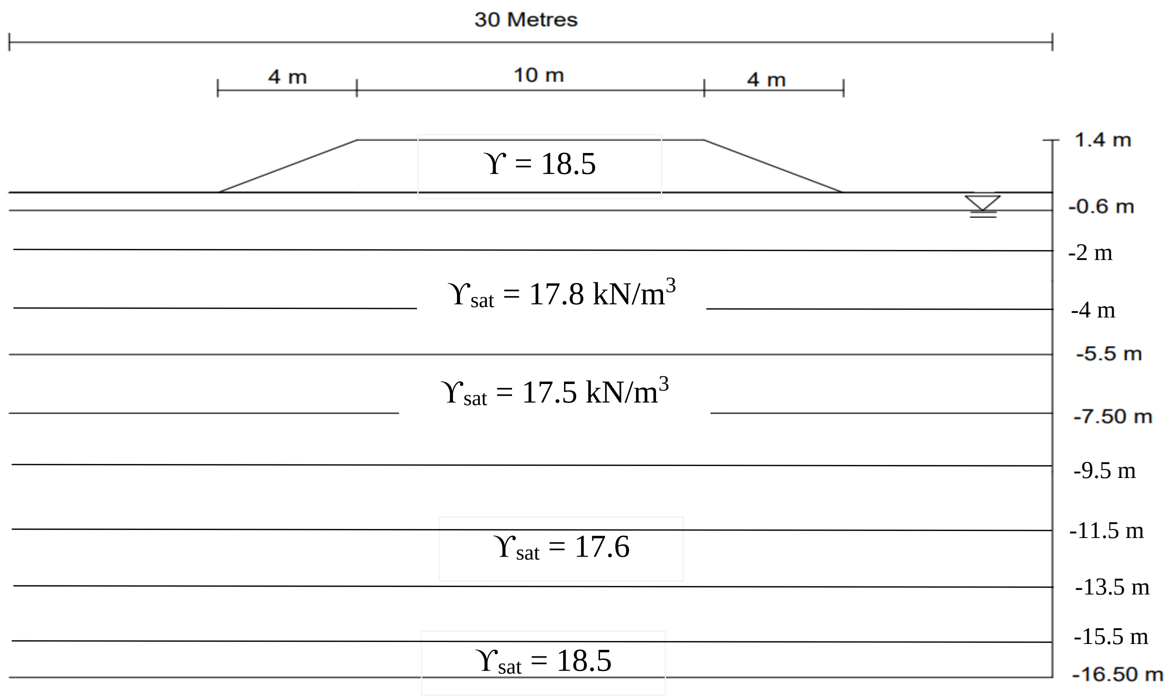


Figure 5.5. b: Section-view of Location-2

Now we are going to calculate the total settlement using the conventional method

$\gamma H = 18.5*1.4 = 25.9$ kN/m^2

$\Delta\sigma + \sigma_{01}' = 28.341$ kN/m^2

$\Delta\sigma + \sigma_{02}' = 41.0835$ kN/m^2

$\Delta\sigma + \sigma_{03}' = 52.1855$ kN/m^2

$\Delta\sigma + \sigma_{04}' = 66.937667$ kN/m^2

$\Delta\sigma + \sigma_{05}' = 82.1542$ kN/m^2

$\Delta\sigma + \sigma_{06}' = 97.4971$ kN/m^2

$\Delta\sigma + \sigma_{07}' = 112.91$ kN/m^2

$\Delta\sigma + \sigma_{08}' = 128.3664$ kN/m^2

$\Delta\sigma + \sigma_{09}' = 136.1058$ kN/m^2

For first layer:

$\sigma_{01}' = (0.6*17.8) + \{1.4*(17.8 - 9.81)\} = 21.866$ kN/m^2

For second layer,

σ_{02}' = (0.6*17.8) + {1.4*(17.8 - 9.81)} + (17.8 – 9.81)*2 = 37.846 kN/m^2

For third layer,

σ_{03}' = (0.6*17.8) + {1.4*(17.8 - 9.81)} + (17.8 – 9.81)*2 + (17.8-9.81)*1.5 = 49.831 kN/m^2

For fourth layer,

σ_{04}' = (0.6*17.8) + {1.4*(17.8 - 9.81)} + (17.8 – 9.81)*2 + (17.8-9.81)*1.5 + (17.5-9.81)*2

= 65.211 kN/m^2

For fifth layer,

σ_{05}' = (0.6*17.8) + {1.4*(17.8 - 9.81)} + (17.8 – 9.81)*2 + (17.8-9.81)*1.5 + (17.5-9.81)*2 + (17.6-9.81)*2

= 80.791 kN/m^2

For sixth layer,

σ_{06}' = (0.6*17.8) + {1.4*(17.8 - 9.81)} + (17.8 – 9.81)*2 + (17.8-9.81)*1.5 + (17.5-9.81)*2 + (17.6-9.81)*2 + (17.6-9.81)*2

= 96.371 kN/m^2

For seventh layer,

σ_{07}' = (0.6*17.8) + {1.4*(17.8 - 9.81)} + (17.8 – 9.81)*2 + (17.8-9.81)*1.5 + (17.5-9.81)*2 + (17.6-9.81)*2 + (17.6-9.81)*2 + (17.6-9.81)*2

= 111.951 kN/m^2

For eighth layer,

σ_{08}' = (0.6*17.8) + {1.4*(17.8 - 9.81)} + (17.8 – 9.81)*2 + (17.8-9.81)*1.5 + (17.5-9.81)*2 + (17.6-9.81)*2 + (17.6-9.81)*2 + (17.6-9.81)*2 + (17.6-9.81)*2

= 127.531 kN/m^2

For ninth layer,

σ_{09}' = (0.6*17.8) + {1.4*(17.8 - 9.81)} + (17.8 – 9.81)*2 + (17.8-9.81)*1.5 + (17.5-9.81)*2 + (17.6-9.81)*2 + (17.6-9.81)*2 + (17.6-9.81)*2 + (17.6-9.81)*2 + (17.6-9.81)*1

= 135.321 kN/m^2

For Consolidation Settlement, $S_c = \frac{HCc}{(1+e0)}\log_{10}(\frac{\bar{\sigma}+\Delta\bar{\sigma}}{\bar{\sigma}0})$

For first layer,

$S_{c1} = \frac{2*0.075}{1+0.657}\log\frac{28.341}{21.866}$ = 0.0101973 m

For second layer,

$S_{c2} = \frac{2*0.075}{1+0.657}\log\frac{41.0835}{37.846} = 0.003226987$ m

For third layer,

$S_{c3} = \frac{1.5*0.075}{1+0.657}\log\frac{52.185}{49.83} = 0.00136159$ m

For fourth layer,

$S_{c4} = \frac{2*0.06743}{1+0.821}\log\frac{66.93767}{65.211} = 0.000840541$ m

For fifth layer,

$S_{c5} = \frac{2*0.0921}{1+0.7961}\log\frac{82.1542}{80.791} = 0.0007452492$ m

For sixth layer,

$S_{c6} = \frac{2*0.0921}{1+0.7961}\log\frac{97.4971}{96.371} = 0.0005174268$ m

For seventh layer,

$S_{c7} = \frac{2*0.0921}{1+0.7961}\log\frac{112.91}{111.951} = 0.00037991$ m

For eighth layer,

$S_{c8} = \frac{2*0.0921}{1+0.7961}\log\frac{128.3664}{127.531} = 0.000290806$ m

For ninth layer,

$S_{c9} = \frac{1*0.0921}{1+0.7961}\log\frac{136.1058}{135.321} = 0.000257561$ m

$S_c = S_{c1} + S_{c2} + S_{c3} + S_{c4} + S_{c5} + S_{c6} + S_{c7} + S_{c8} + S_{c9} = 0.0178173$ m

S_c = 17.8 mm

5.6 Discussion

- **Comparison of field monitoring settlement data with PLAXIS 2D: Soft Soil Model & Hardening Soil Model**

	Settlement (mm)		
	Field Monitoring Data	**Hardening Soil Model**	**Soft Soil Model**
Location-1	16	40	17
Location-2	23	51	26

- **Soft soil model**

% Accuracy of settlement = $\frac{\textbf{Soft soil model} - \textbf{Field monitoring Data}}{\textbf{Field monitoring Data}}$

- **Hardening Soil Model**

% Accuracy of settlement = $\frac{\textbf{Hardening Soil model} - \textbf{Field monitoring Data}}{\textbf{Field monitoring Data}}$

		Hardening Soil Model	Soft Soil Model
% Accuracy of settlement	**Location-1**	150%	6.25%
	Location-2	121.7%	13.04%

From the above analysis, % accuracy of settlement for hardening soil model is 150% and 121.7% for Location-1 & 2 respectively in comparison with field monitoring data which is quite a high difference and thus explain the unsuitability of hardening soil model for clay or silty clay soil. On the other hand, soft soil model gives 6.25% and 13.04% for Location-1 & 2 respectively which shows a high accuracy of settlement value compared to the field monitoring settlement. Thus explain the better suitability of soft soil model over hardening soil model for clay or silty clay soil.

- **Comparison of settlement "with geogrid" and "without geogrid"**

Location-1

- **Soft soil model:**

	Nodes	Settlement, (mm)		
		Case 1: No geogrid	**Case 2: 1 layer of geogrid**	**Case 3: 3 layers of geogrid**
Location-1	(1.04, 1.8)	33	31	31
	(2, 1.8)	32	30	30
	(2.98,1.8)	29	28	28

- **Hardening Soil Model:**

	Nodes	Settlement, (mm)		
		Case 1: No geogrid	Case 2: 1 layer of geogrid	Case 3: 3 layers of geogrid
Location-1	(1.04, 1.8)	70	69	69
	(2, 1.8)	68	66	66
	(2.98,1.8)	63	62	62

% Difference of settlement = $\frac{\textbf{1 layer of geogrid} - \textbf{No geogrid}}{\textbf{No geogrid}}$

% Difference of settlement = $\frac{\textbf{3 layers of geogrid} - \textbf{No geogrid}}{\textbf{No geogrid}}$

% Difference of settlement	Nodes	Case 2 to Case 1		Case 3 to case 1	
		Soft soil Model:	Hardening Soil Model	Soft soil Model:	Hardening Soil Model
	(1.04, 1.8)	6.06%	1.43%	6.06%	1.43%
	(2, 1.8)	6.25%	2.94%	6.25%	2.94%
	(2.98,1.8)	3.45%	1.59%	3.45%	1.59%

From the above analysis, we can see there is slight difference after the addition of the geogrid in comparison to the “without geogrid” case for both the models, even though soft soil model handed better result for the field monitoring settlement analysis. The % difference of settlement is represented for case -2 and case-3 in comparison to case-1. Moreover, there is no change at all for different number of layers of geogrids use. This shows, there is almost no effect of geogrids on embankment settlement. This shows the need of advanced modelling for monitoring the effect of geogrid in embankment settlement.

Location-2

❑ **Soft soil model:**

	Nodes	Settlement, (mm)		
		Case 1: No geogrid	Case 2: 1 layer of geogrid	Case 3: 3 layers of geogrid
Location-2	(1.10, 1.2)	52	50	50
	(2.11, 1.2)	50	49	49
	(3.09,1.2)	47	45	45

❑ **Hardening Soil Model:**

	Nodes	Settlement, (mm)		
		Case 1: No geogrid	Case 2: 1 layer of geogrid	Case 3: 3 layers of geogrid
Location-1	(1.04, 1.8)	93.6	92.5	92.5
	(2, 1.8)	89	89	89
	(2.98,1.8)	84	84	84

% Difference of settlement = $\frac{\textbf{1 layer of geogrid} - \textbf{No geogrid}}{\textbf{No geogrid}}$

% Difference of settlement = $\frac{\textbf{3 layers of geogrid} - \textbf{No geogrid}}{\textbf{No geogrid}}$

	Nodes	Case 2 to Case 1		Case 3 to case 1	
		Soft soil Model:	Hardening Soil Model	Soft soil Model:	Hardening Soil Model
% Difference of settlement	(1.10, 1.2)	3.84%	1.18%	3.84%	1.18%
	(2.11, 1.2)	2%	0%	2%	0%
	(3.09,1.2)	4.26%	0%	4.26%	0%

From the above analysis, we can see there is slight difference or no difference at all after the addition of the geogrid in comparison to the “without geogrid” case for both the models, even though soft soil model handed better result for the field monitoring settlement analysis. The % difference of settlement is represented for case -2 and case-3 in comparison to case-1. Moreover, there is no change at all for different number of layers of geogrids use. This shows, there is almost no effect of geogrids on embankment settlement. This shows the need of advanced modelling for monitoring the effect of geogrid in embankment settlement.

- **Comparison of Safety Factor Analysis between “without geogrid” and “with geogrid”**

- **Soft Soil Model**

	Location No.	Case 1: With Geogrid	Case 2: Without Geogrid
Factor of Safety	Location-1	3.411>1	4.321>1
	Location-2	3.517>1	4.738>1

	Location No.	Case 2 to Case 1
% Difference of Factor of Safety	Location-1	26.68%
	Location-2	34.72%

From the above analysis, both the locations simulation for the factor of safety shows a less moderate percentage of increase “with geogrid” compared to “without geogrid”; 26.68% increase for Location-1 and 34.72% for Location-2. More advanced modelling is required to evaluate the influence of geogrid in embankment settlement.

- **Comparison of Conventional method with field monitoring data and PLAXIS-2D simulation**

	Settlement, (mm)		
	Conventional Method	**Field Monitoring Data**	**Soft Soil Model**
Location-1	35.1	16	17
Location-2	17.8	23	26

- **<u>Soft soil model</u>**

Difference of settlement = $\left|\textbf{Soft Soil Model- Conventional Method}\right|$

- **<u>Field Monitoring Data</u>**

Difference of settlement = $\left|\textbf{Field Monitoring Settlement – Conventional Method}\right|$

		Field Monitoring Data	**Soft Soil Model**
Difference of settlement (mm)	**Location-1**	19.1	18.1
	Location-2	5.2	8.2

From the above analysis, we can observe that the conventional method settlement value of Location-1(i.e. 35.1mm) is quite higher than the monitoring data and the PLAXIS-2D simulation. This value should have been lower than the settlement amount of Location-2 (i.e. 17.8mm) when compared with the field monitoring data and Plaxis simulation. In software simulation, there are numerous other variables considered while running any model such as OCR, coefficient of earth pressure at rest: Ko, flow parameters, stiffness, strength values, etc. On the other hand, conventional method involved few of the parameters for which Location-1 value resulted 19.1mm and 18.1mm larger than the conventional method. Thus, software simulation using soft soil model matched better with the monitoring data, while conventional method has shown some discrepancy.

CHAPTER 6: Conclusion

6.1 Conclusions on Completed Research Work

- Soft soil model is more efficient in the case of field monitoring data rather than Hardening soil model.
- Obtained almost similar settlement for soft soil model compared to field monitoring data.
- In further work of soft soil model and hardening soil model, it is observed that the addition of geogrid is not that much effective, whereas we know that geogrid reduces settlement significantly and uniformly. This shows more advanced modelling should be pursued to monitor this effect.
- By Comparing the cases of 'with and without geogrid' for safety factor analysis, it can be said that with geogrid it has better safety in slope stability.

6.2 Future work and recommendation

We have reviewed that it would add more value in this work if we focus on 'Sub loading t_{ij} model' in future. In Embankment's settlement estimation sub loading subloading t_{ij} model for FEM analysis is very much convenient and gives more accurate result.

In our further analysis, for better comparison with field monitoring data, we can use all the three models: hardening soil model, soft soil model and sub loading t_{ij} model. This will enhance the research work with a precise view of the better models among the three.

Though geogrid is a quick solution, but it has to be designed based on experience and soil condition. To minimize soil settlement, preloading and improving by PVD would be another alternative.

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