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The Behavior of Ordinary and Geosynthetic Encased Stone Columns in Collapsible Soil: A Numerical Study

Author, Henok Lemma Belayneh

Addis Ababa Science and Technology University

Department of Civil Engineering

College of Architecture and Civil Engineering

Po. Box;1000-21281 Addis Ababa, Ethiopia Email; enoch.geotech@gmail.com

Abstract

This paper examines the basic assumptions, procedures and results of the numerical analysis in simulating the behaviour of un-encased versus geotextile encased stone column for collapsible soils to support an embankment dam, using commercially available finite element method software PLAXIS 3D.

Input parameters of numerical modelling were estimated using different correlation techniques based on SPT N values of soil. Soil data for this thesis were adopted from the SPT results. The data for stone column and geosynthetic material were adopted from a research work of Iman Hossein Pour Babei.

The results of this study show that, geosynthetic encased stone column (GESC) has a remarkable influence on decreasing settlements and accelerate the consolidation time. The use of GESC decreases the amount of settlement of unreinforced soil from 338mm to 257mm. It also accelerates the consolidation end time from 9719 days to 653 days.

The research also recommends to conduct further study on the response to bulging effect of GESC in relation to geotextile stiffness.

KEYWORDS; Finite Element Analysis, Consolidation, Collapsible soil, PLAXIS, SPT, GESC

Chapter One

Introduction

1.1 General

Collapsible soils are problematic soils that make the construction of foundations extremely difficult in its natural state. Such soils are moisture sensitive; and exhibit relatively rapid volume compressions on the action of loading and wetting. If the foundations rest on collapsible soil, several ground improvement techniques needs to be considered in order to avoid foundation failure.

Stone columns are widely used globally as structural elements due to the simplicity, short duration, and cheap cost of their construction. The purposes of this technique are to increase the bearing capacity and reduce the total settlement of soft soils. Stone columns derive their strength and stiffness primarily from the confinement stress provided by the surrounding soil (Guetif etal. 2007).

A cited literature revealed that when these stone columns are used to improve collapsible soil, the materials (stone pieces) of stone column get into the surrounding soil due to inadequate lateral confinement, particularly at depths closer to the ground. However, to overcome these limitations, stone columns can be encased using geosynthetics (Al-Obaidy, 2000).

The main purpose of this study is to investigate the behaviour of encased stone column in collapsible soil numerically using commercially available finite element software PLAXIS 3D.

1.2 Objective of the Study

1.2.1 General Objective

The main objective of this study is to evaluate the behaviour of stone columns (ordinary and geosynthetic encased) in collapsible soil by numerical analysis.

1.2.2 Specific Objectives

- To examine the collapse potential of soil.
- To determine input parameters for numerical simulation from SPT results.

- To validate the FEM model with actual field test results.
- To evaluate the improvement achieved in treating collapsible soil with ordinary and geosynthetic encased stone column.

1.3 Scope of the Study

This research is intended to assess the behavior of GESC in collapsible soils under a 5m embankment dam by using PLAXIS 3D, FE method software. The study is supported by different types of literatures and series of laboratory and field experiments.

However, the findings of the research are limited to one soil sample which is collapsible soil that is found in SNNPR, Hawassa near the airport. Therefore, findings of this thesis should be considered as indicative rather than definitive for application.

1.4 Structure of the Study

This paper is organized in six chapters.

The first chapter gives a brief description of the thesis background, statement of the problem, objectives, scope and limitation of the study. Chapter two presents literature review on collapsible soil, stone column and related studies on OSC and GESC. Chapter three, briefly discuses about the materials, methods and procedures. The fourth chapter deals with modelling. This chapter concerns about finite element modelling and model validation. The fifth chapter discussed about analysis, results and discussions. The last chapter, chapter six presents the conclusions and recommendations drawn from this research.

Chapter Two

Literature Review

2.1 Collapsible Soils

Collapsible soils are mainly wind-deposited sand and/or silts and volcanic dust deposits. These deposits are characterized by relatively low unit weights, low natural moisture contents, high void ratios and are cohesion less or slightly cohesive (Das, 2007).

Collapsible soils exhibit high apparent strength in natural state. But when there is an increase in saturation ratio or loading condition, collapse occurred as the bonds between the grains breakdown (Khaled, 2012).

2.1.1 Examination of Collapse Potential of Soil

A. Geological Reconnaissance

Geological and geomorphological reconnaissance together with the experience of the geotechnical engineer can be very helpful in predicting collapsible potential of soil deposits (Beckwith,1995).

B. Indirect Evaluation of Collapse Potential

Several collapse criteria have been proposed for predicting the collapse behavior of collapsible soils. Qualitative and semi-quantitative correlations between collapse potential and various index properties have been developed and reported as listed below.

- Clonjer criterion, 1959: Clonjer presented a criterion for examination of collapse of the soils based on dry density of soils.

Table 2.1 Clonger criterion for collapse potential of soil

Dry density(gr/cm^3)	**Collapse Potential**
<1.28	High collapse potential
1.28 – 1.44	Medium collapse potential
>1.44	Low collapse potential

- Denisov criterion, (1951): used the coefficient of subsidence; **K= e/e$_{LL}$** for defining the collapse of the soil. Where e_{LL} is air void ratio in liquid limit and e is air void ratio in natural state.

In this criterion; if K= 0.5 – 0.75: highly collapsible soil;
if K= 1.0: non-collapsible loam;
if K= 1.5 – 2.0: non-collapsible soil.

- Priklonski (1952) examined the collapsibility of soil by liquidity Index (K_D) as

$K_D = \frac{Wn-PL}{PI}$

Where W_n = natural moisture content

PL = plastic limit

PI = plasticity index

In this criterion; if $K_D < 0$: highly collapsible soils;

if $K_D > 0.5$: non-collapsible soils;

if $K_D > 1.0$: swelling soils.

- Clevenger (1958) also proposed a criterion for collapsibility based on dry density. If the dry density is less than 12.6 kN/m^3, then the soil is liable to undergo significant settlement. On the other hand, if the dry density is greater than 14.1 kN/m^3, then the amount of collapse should be small, while at intermediate densities the settlements are transitional.
- In Soviet Building Code (1962), the soil is considered to be susceptible to collapse upon wetting if the in-situ degree of saturation (S) is less than 60 percent and for some constant K>-0.10.

Where, $K = \frac{e_0 - e_L}{1+e_0}$ and: e_o is the natural voids ratio;

e_L is the voids ratio corresponding to liquid limit

- Feda (1964) examined collapsible soils as follows:

$$K_L = \frac{W_o}{S_r} - \frac{PL}{PI}$$

Where: W_o= natural water content,

S_r = natural degree of saturation,

PL = plastic limit, and PI = plasticity index

For Sr < 100 %, if $K_L > 0.85$, the soil is collapsible.

- Handy (1973) suggested that collapsibility could be determined either by the percentage clay content or from the ratio of liquid limit to saturation moisture content.

Table 2.2 Handy criterion for collapse potential of soil

Clay Content	**Collapse Potential**
<16%	Highly collapse
16% - 24%	Collapse

25% - 32%	Probably Collapse
>32%	Non-collapse

- Zur and Wiseman (1973) used the ratio D_o / D_{LL}; where D_o is the in-situ dry density and D_{LL} is the dry density of soil at full saturation and at moisture content equal to the liquid limit.

  If $D_o / D_{LL} < 1.1$; soil prone to collapse

  If $D_o / D_{LL} > 1.3$; soil prone to swell

C. Direct Evaluation of Collapse Potential

Direct evaluation of collapse potential and measuring the severity of collapse strain can be examined either in laboratory by performing oedometer test or in field by performing plate load test.

2.2.2 Collapsible Soils in Ethiopia (According to ERA manual Site Investigation Manual - 2013)

In Ethiopia, collapsible soils are present in the southern part of the Omo River and in the central and southern part of the rift valley. Often, their existence around Zeway, Shashemene, and Hawassa is manifested by the occurrence of ground cracks and potholes during heavy rains or floods due to hydro-compaction. In the Afar region, collapsible soils are present in the form of sand dunes.

2.1.2 Treatment of Collapsible Soils

Several methods have been used for treating collapsible soils. This method can be categorized as; dynamic compaction of soil at natural moisture content, prewetting, dynamic compaction after prewetting the soil, chemical Stabilization, soil replacement and stone columns.

2.2 Stone Columns

Stone columns are a common ground improvement method employed to treat soft soils, and act as reinforcement elements to reduce settlement and increase bearing capacity (Hughes & Withers, 1974; Balaam & Booker, 1981; Sondermann & Wehr, 2004).

Those materials which are used in stone column method have high permeability by comparison of the soft soils. Thus, stone columns behave like drain holes and help to speed up the rate of consolidation process.

2.2.1 Ordinary Stone Column

The primary purposes for using stone column technique are to improve bearing capacity, reduce settlement, and enhance drainage and stability (Ambily and Gandhi, 2007; Zhang et al., 2015).

Thus, the presence of granular columns will accelerate the dissipation of the excess pore water pressure in soft soils which in turn accelerates the consolidation process and reduces both the total and post-construction settlements.

2.2.2 Encased Stone Column

Ordinary stone columns usually derive their bearing capacity from passive resistance provided by the surrounding foundation soil pressing against the lateral bulging of stone columns as a result of axial load application. When embedded in soft clay, stone columns may bulge due to lack of confinement offered by the surrounding soft soil. Furthermore, the soft clay may enter the voids between granular material of column to cause clogging and reduce the permeability of granular columns for drainage (Al-Obaidy 2000).

In order to avoid these consequences, additional confinement can be provided by using geosynthetic encasement. This will help to isolate the granular soil inside the column so that it does not mix with the surrounding soil and increase the stiffness of the columns (Murugesan and Rajagopal 2009).

2.3 Related Laboratory and Numerical Studies

Many scholars conducted laboratory study on different problematic soils that are improved by encased stone columns. Among those scholars Ayadat & Hanna (2005), Ali et al. (2010), Demir & Sarici (2016) and Tandel et al. (2013) have studied on collapsible soil, kaolin clay, soft clay and clay soil types respectively.

Elsawy et al (2010) had studied a FEM analysis on the behavior of ordinary and encased stone columns. L. keykhosropur et al(2012) had also examined the influence of various parameters on the performance of geosynthetic encased stone columns through 3D numerical modeling. A numerical study was undertaken by S.R Lo et al(2010) to examine the reinforcing role of stone columns in soft clay.

Particular Features of this Study

This paper studies the behavior of GESC in collapsible soil under an embankment dam and adjoins other uses of stone column. It argues the stone column is not only act as reinforcing material for increasing the overall strength and stiffness of soft soil, but it also promotes consolidation through effective drainage. This is because the materials which are used in stone column method have high permeability in comparison to the host soils. In the analysis of FEM a new calculation option is introduced, namely a consolidation analysis.

Chapter Three

Materials, Methods and Procedures

3.1 Introduction

A representative soil sample is taken from Ethiopian Rift valley system specifically, located in SNNPR, Hawassa near to the Airport.

3.2 Materials

Soil data for this thesis were adopted from the Standard penetration test (SPT) and the soil profiles for Bore hole 7 with sandy silt was chosen. The proposed site is for Factory Building Project. However, for this research purpose a hypothetical embankment dam is proposed to build on such problematic soil type using geosynthetic encased stone column. As the geotechnical report of the soil investigation revealed, the soil exhibits high potential for collapse.

3.2.1 Geological and Geotechnical Overview of the Study Area

In the Ethiopian Rift systems, the Quaternary Sediments are mostly of lacustrine origin. Lacustrine beds are inter-bedded in plio-pleistocene ignimbrites in the lakes district and on the rift shoulders.

The geological and geotechnical investigations of the study area are outlined below.

3.2.1.1 Site Geology

The upper most layer of the project site is loose to medium dense, light brown to white, non-plastic sandy SILT/ silty SAND with gravel (Resedimented Pyroclastic Material). Beneath the top geological formation in the project site dense, light brown, non-plastic sandy SILT soil layer is found (Resedimented Pyroclastic Material).

The lowest profile found by investigation in the project site is dense to very dense, dull white, sandy SILT soil layer (Volcanic Ash/resedimented pyroclastic material).

3.2.1.2 In-situ Tests

The in-situ test conducted in the drilled borehole is Standard Penetration Test (SPT) using a standard hammer, under an impact of an automatic sliding hammer. Summary of the SPT test results are given in the following table.

Table 3.1 Standard Penetration Test results

BH ID	Depth	Field Description of soil	SPT N-Value (450mm)	SPT N – Value (300mm)
BH 7	1.50	Loose to medium dense,light brown to white, non-plastic sandy SILT/ silty SAND with gravel (Resedimented Pyroclastic Material)	4/4/5	9
	3.00		3/3/3	6
	4.50		2/2/3	5
	6.00		2/3/4	7
	7.50		9/10/13	23
	9.00		10/13/15	28
	10.50	Dense, light brown, non-plastic sandy SILT (Resedimented Pyroclastic Material)	9/12/16	28
	12.00		R	50

	13.50	Dense to very dense, dull white, non-plastic sandy SILT (volcanic ash/Resedimented Pyroclastic Material)	18/21/R	50
	15.00		R	50
	16.50		13/17/19	36
	18.00		17/21/R	50

3.2.1.3 Sampling

Representative soil samples were collected from soil investigation report. Results of Borehole ID 7 laboratory tests are presented below in table 3.2.

Table 3.2 Laboratory test results of soil samples

Sample Depth(m)	**3.50-4.00**	**6.50-7.00**	**9.50-10.00**	**13.50-14.00**
Atterberg Limits				
Liquid limit (%)	NP	NP	NP	NP
Plastic limit (%)	NP	NP	NP	NP
Plasticity index (%)	NP	NP	NP	NP
Specific Gravity				
G_s	2.58	2.53	2.55	2.57
Particle size distribution				
Gravel (%)	7	5	1	1
Sand (%)	52	49	48	45
Silt (%)	39	45	50	52
Clay (%)	2	3	1	2
Fine passing No. 200 (%)	41	49	51	54
Dry density and Moisture content				
N.M.C (%)	1.12	3.41	9.61	6.12

γ_{dry} (gm/cm^3)	1.24	1.33	1.32	1.25
γ_{dry} (kN/m^3)	12.14	13.12	12.98	12.21
O.M.C (%)	9.32	10.90	11.21	13.23
Void ratio	1.08	0.89	0.92	1.06

3.2.2 Examining the Collapse Potential of Studied Soil

Soil characterization and behavior are the most important parameters in the design of foundations. Therefore, the site investigation and characterization are crucial to use one of the various methods appropriate for a specific project. In this paper the collapse potential of soil is examined based on the criteria's which are suggested by different scholars and presented in section 2.1

The first criterion that predicts the collapse potential of studied soil was stated by Handy (1973). In this criterion the soil is identified as collapsible by using its clay content. If the clay content is less than 16% the soil is highly collapsible and if it's between 16% to 24% the soil is collapse. The test results shown that the soil have clay content less than 16%. Thus, by using this criterion the soil is highly collapsible.

The second criterion that considered the tested soils are collapsible was Clonjer criterion (1959). Clonjer stated a criterion for examination of collapse of the soils based on dry density of soils. Based on this criterion if the dry density of the soil is less than 1.28 gm/cm^3 the soil has high collapse potential and if it's in between 1.28 gm/cm^3 to 1.44 gm/cm^3, it has medium collapse potential. By using this criterion, the tested soil has a medium collapse potential.

The third criterion that estimates the magnitude of collapse potential of the studied soil was the chart that was prepared by Holtz and Hilf, (1961). The chart classifies the soils as; soils have been observed to collapse and soils have not generally been observed to collapse. Thus, by using this chart the studied soil sample is considered as soils that have been observed to collapse.

The fourth criterion that justify the susceptibility of collapse potential of the studied soil was stated by Clevenger (1958). This criterion identifies the susceptibility of soil to collapse by using the natural dry density and if the natural dry density is less than 12.6 kN/m^3 the soil is liable to undergo significant settlement significant settlement will while if the natural dry density is greater than 14.1 kN/m^3 small settlement will be occur. Thus, as the test results shown the soil is liable to undergo small settlement.

The fifth parameter that justify the studied soil sample has the potential to collapse is the chart that is prepared by Moghadan et al. (2006). This chart uses the relations between dry unit weight and percentage finer than 0.075mm (sieve no. 200) to evaluate the collapse potential of soils. Thus, based on this chart the soil has moderate to high collapse potential.

Generally, most of the criteria that that are suggested by different scholars to predict the susceptibility of collapse potential of the studied soil from moderate to high range.

3.3 Methods and Procedures

The Standard penetration test (SPT) is a common, simple and inexpensive in situ testing method for defining subsurface materials. The data of which can be used for defining the geotechnical properties of soils. When laboratory data is not available, it is common practice to correlate the SPT values with many soil parameters. This research is based on the SPT data.

3.3.1 Corrections to SPT N Value

The most applicable correction method that accounts for the effect of energy delivered, overburden stress and ground conditions is the standardized SPT corrections method.

The SPT values obtained during investigation shall be adjusted to N_{55} or N_{70} standard energy ratio value using the following formula (Bowles, 1988).

$$N'_S = C_N * N * \eta_1 * \eta_2 * \eta_3 * \eta_4$$

Where; N'_S= Adjusted N (N_{55} or N_{70}) ;

C_N=adjustment for effective overburden pressure $C_N = \frac{P_o^{``}}{P_o^{`}}$ (kpa) $= \sqrt{\frac{95.76}{P_o^{`}}}$

N = SPT values (unadjusted) and

η_i = Adjustment factors for energy ratio, rod length, sampler and borehole diameter correction.

Another correction factor N_{60} also stated as per ASTM geotechnical engineering

$$N_{60} = \frac{C_E C_B\ C_S C_R}{0.60}$$

Where; N_{60}= Corrected N values corresponding to 60% Energy Efficiency

That is, The Energy Ratio (ER) = 60%

C_E, C_B, C_S and C_R =are energy ratio, rod Length, sampling method and borehole diameter correction factors.

$N_{measured}$ = Raw SPT Value from Field Test

3.3.2 Mohr – Coulomb Model Parameters

The Mohr-Coulomb model idealizes the soil as elastic perfectly plastic material and has five basic parameters.

The soil behavior before failure is computed by Hooke's law of elasticity (defined by Young's modulus and Poisson's ratio). The failure of soil is based on Mohr-Coulomb failure criteria (defined by angle of internal friction and cohesion). The irreversible plastic strains that are generated from shearing are computed using non-associated flow rule (defined by angle of dilation).

3.3.3 Basic Adopted Parameters for Simulating in PLAXIS

- The modulus of elasticity (E) values is summarized in Joseph E. Bowles (1996) for different types of soils based on N_{55} value.

Thus, for sandy silt soil, E_S = 300(N+6)

Where the N value is estimated as $N_{55.}$

- The angle of internal friction was mathematically approximated by Kulhawy and Mayne (1990)

$$\varphi = \tan^{-1}[N60/(12.2+20.3\frac{\sigma'}{P_a})]^{0.34}$$

Where, σ' is the overburden pressure and P_a is the atmospheric pressure.

- Poisson`s ratio is assumed to be 0.3 and for cohesion.

C_u is correlated by N_{60} value presented by equation3.5. However, due to undrained condition it assumed becomes zero.

$$C_u=0.06P_aN_{60}$$

- The value of angle of dilation for silty sands ψ depends on the angle of internal friction and can be estimated as;

$$\psi = \Phi - 30°,\ \text{for}\ \Phi>30^0$$

Based on the above equations the input parameters are listed in the following table.

Table 3.3 Summary of input parameters.

Borehole 7									
Depth(m)	**SPT N - 55 value**	**SPT N – 60 Value**	**γ(kN/m³)**		**E(kN/m²)**	**υ**	**C_u(kN/m²)**	**φ(⁰)**	**Ψ**
			Unsat.	**Sat.**					
1.50	9	6.60	12.14	17.24	4500	0.3	_	36.56	6.56
3.00	6	4.68	12.14	17.24	3600	0.3	_	31.56	1.56
4.50	4	4.35	12.14	17.24	3000	0.3	_	29.49	0.00
6.00	5	6.10	13.12	17.74	3300	0.3	_	30.72	0.72
7.50	16	20.03	13.12	17.74	6600	0.3	_	40.40	10.40
9.00	18	25.67	13.12	17.74	7200	0.3	_	41.67	11.67
10.50	17	25.67	12.98	17.70	6900	0.3	_	40.73	10.73
12.00	17	45.83	12.98	17.70	6900	0.3	_	45.44	15.44
13.50	27	45.83	12.21	17.27	9900	0.3	_	45.04	15.04

15.00	18	45.83	12.21	17.27	7200	0.3	_	44.27	14.27
16.50	18	33.00	12.21	17.27	7200	0.3	_	40.39	10.39
18.00	14	45.83	12.21	17.27	6000	0.3	_	42.91	12.91

Chapter Four

Model Validation

4.1 Introduction

As part of this thesis, validation of the finite element method was done by comparing with the real-life situation or field recorded data to insure the model created is accurately modelled.

A full-scale load test reported by Iman Hossein Pour Babaei was used to validate the finite modelling approach. Excess pore water pressures were measured by piezometers located at depths of z = 3m, 6m, and 8m (shown in figure 4.1). The commercial finite element package PLAXIS 3D (three dimensional FEA software) was used for the finite element modelling.

4.2 Measured and Computed Result

Measured and computed excess pore water pressure at the location of piezometers are presented below to evaluate the degree of the accuracy of the FEM.

It can be seen that the numerical analysis predicted the measured excess pore water pressure reasonably well, in particular the 3D model adequately simulated the maximum excess pore pressures in the construction stages. It also shows the dissipation of excess pore pressure during consolidation period.

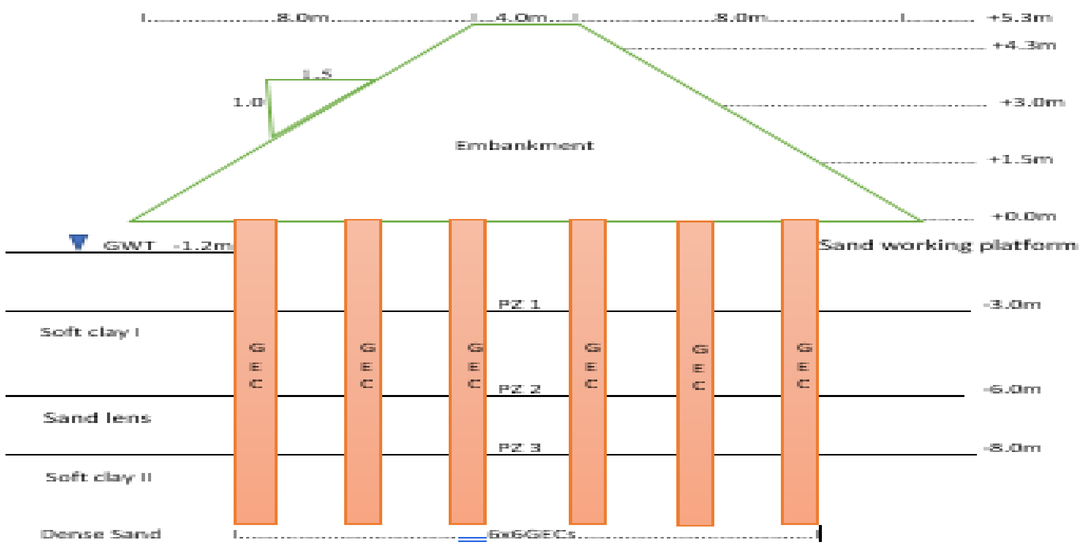

Figure 4.1 Embankment side column arrangement & instrumentation layout.

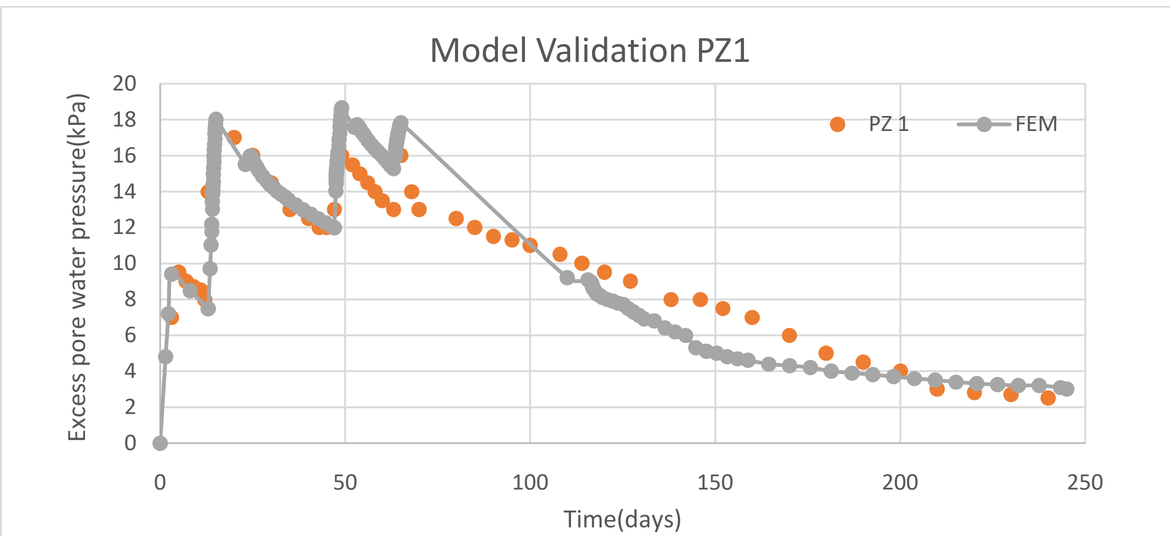

Figure 4.1 Measured and FEM computed excess pore water pressure at 3m depth.

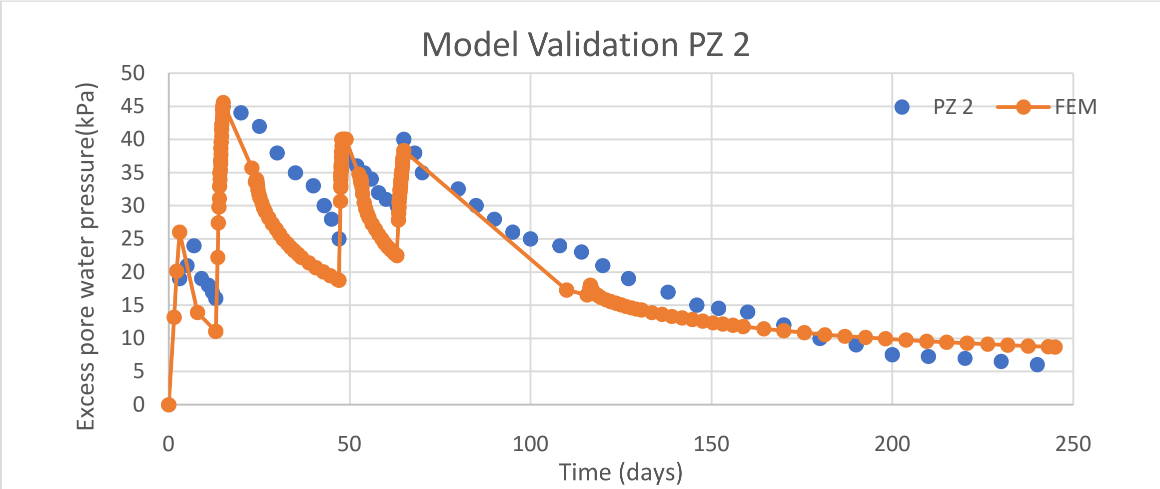

Figure 4.2 Measured and FEM computed excess pore water pressure at 6m depth.

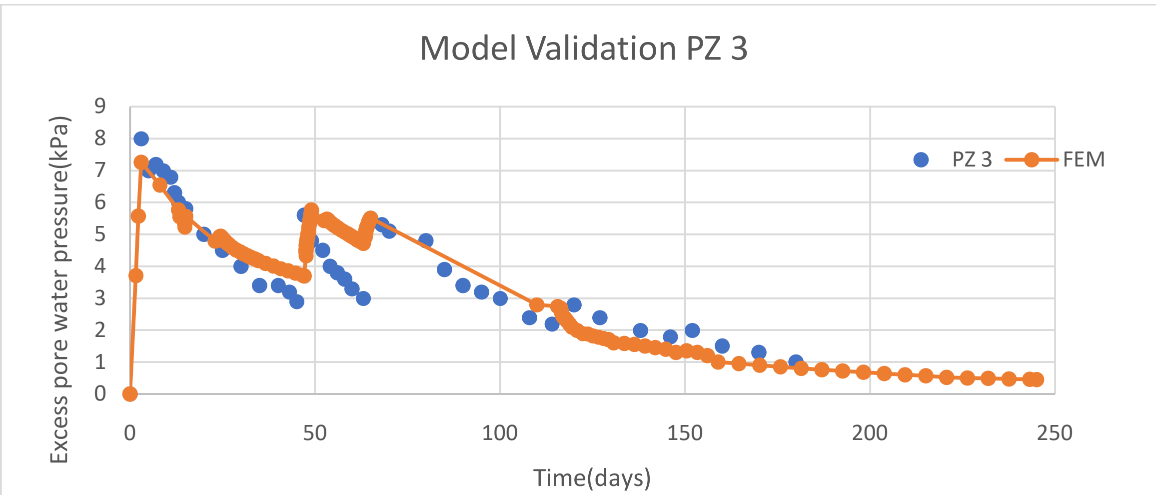

Figure 4.3 Measured and FEM computed excess pore water pressure at 8m depth.

Chapter Five

Analysis, Results and Discussions

5.1 Introduction

In this thesis, the elastic- perfectly plastic model of Mohr-Coulomb (MC) constitutive model failure criterion was adopted. This model is chosen because many scholars suggest this model as the best soil model to predict the realistic behaviour of structures, like embankment dams that are imposed lower working load. Among those scholars (Zukri A. and Nazir R. (2018)), clearly summarizes the numerical modelling techniques of soils improved by stone columns that has done by many researchers. Moreover, input parameters for this model can be determined from different test results (like SPT or CPT) using reasonable correlation formulas.

5.2 Finite Element Analysis

5.2.1 Materials and Parameters

The material properties of soil strata were taken from SPT results, and to minimize computational time soils that have nearly the same properties are merged to a single layer as shown in Table 5.1.

The properties of the stone column, geosynthetic material and embankment fill are taken from the work of Iman Hossein Pour Babaei. The geosynthetic encasement was assumed to be isotropic elastic material and its tensile stiffness J_{enc} =1750kN/m.

Table 5.1 Parameters for soil strata, embankment fill and stone column.

Material	Embankment	Layer 1 (0-4.5m)	Layer 2 (4.5-10.5m)	Layer 3 (10.5-13.5m)	Layer 4 (13.5-18.0m)	Stone Column
Model Type	Mohr-Coulomb (drained)	Mohr-Coulomb (undrained)	Mohr-Coulomb (undrained)	Mohr-Coulomb (undrained)	Mohr-Coulomb (undrained)	Mohr-Coulomb (drained)
γ_{unSat}(kN/m^2)	24.0	12.14	13.12	12.98	12.21	18
γ_{Sat}(kN/m^2)	28.0	17.24	17.74	17.70	17.27	20
E'(kPa)	50000	3700	6000	6900	7575	80000
c'(kPa)	0.0	0.0	0.0	0.0	0.0	0.0
φ'(^{0})	38	32.54	38.28	43.10	43.15	38
υ'(-)	0.3	0.3	0.3	0.3	0.3	0.3
ψ(^{0})	8	2.54	8.28	13.10	13.15	8

5.2.2 Numerical Procedures

The embankment fill was built in four stages as shown in Table 5.2. The first three stage was conducted in 2 days and allowed to consolidate for a time interval of 30 days. For the loading type parameter, because the consolidation is for predefined period, staged construction is chosen. Finally, the last stage was constructed for 2 days by utilizing minimum pore water pressure option to assess the consolidation end time. Hence a total of 8 calculation phases are defined.

Table 5.2 Phases for loading defined for numerical model.

Calculation step	Calculation type	Loading type	Embankment height(m)	Time interval (days)	Event
Initial phase	K_o procedure		-	-	Initial stress state
Phase_1	Consolidation	Staged construction	1.5	2	Construction
Phase_2	Consolidation	Staged construction	1.5	30	Consolidation
Phase_3	Consolidation	Staged construction	3.0	2	Construction
Phase_4	Consolidation	Staged construction	3.0	30	Consolidation
Phase_5	Consolidation	Staged construction	4.0	2	Construction
Phase_6	Consolidation	Staged construction	4.0	30	Consolidation
Phase_7	Consolidation	Staged construction	5.0	2	Construction
Phase_8	Consolidation	Minimum excess pore pressure	5.0	-	Consolidation

Because of the symmetry of the embankment, the right part of the embankment has been considered as shown in figure 5.1. The position of the points of locations for FEM analysis can be seen in Figure 5.1.

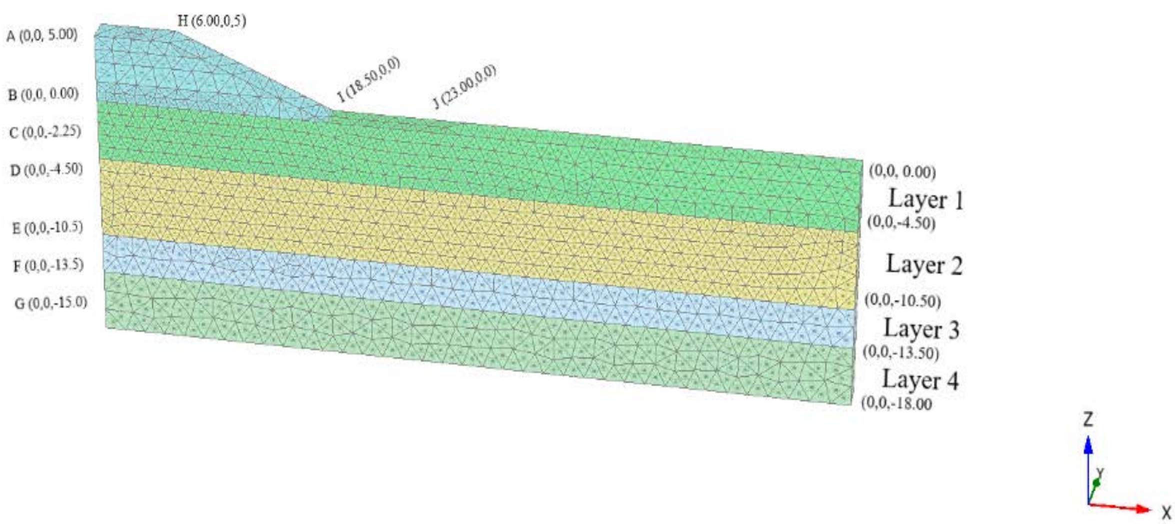

Figure 5.1 Mesh Connectivity and Points of location for FEM analysis.

5.3 Discussion

5.3.1 General

Three different models, by varying reinforcement type have been considered to analyze the effect of consolidation end time, excess pore water dissipation and settlement with respect to time as listed below.

<u>Model R</u> (varying reinforcement type)

- MUnS: Unreinforced Soil
- MOSC: 9 Ordinary Stone Columns with a diameter of 1.0m, at 2.5m spacing and 10m height
- MGESC: 9 Geosynthetic Encased Stone Columns with a diameter of 1.0m, at 2.5m spacing and 10m height.

5.3.2 Consolidation End Time Analysis

Figure 5.2 shows the consolidation end time for unreinforced soil, ordinary stone columns and geosynthetic encased stone columns beneath an embankment dam. It can be seen from the figure, the geosynthetic encased stone column reduces the consolidation end time in comparison to the unreinforced soil from 9719 days (approximately 26.6years) to 653days. Therefore, the geosynthetic encased stone column has a substantial influence on reducing the consolidation end time over the unreinforced soil and ordinary encased stone columns.

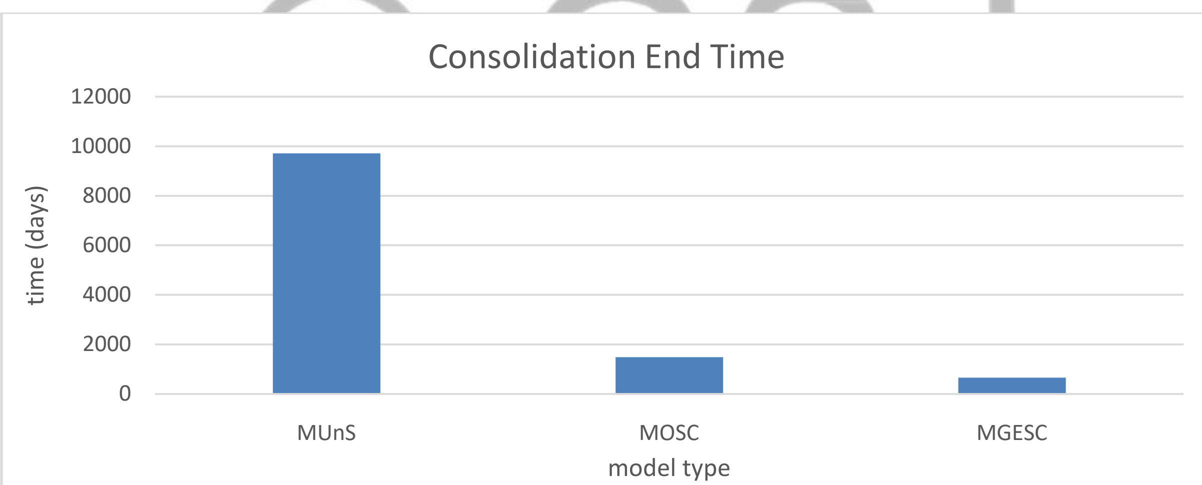

Figure 5.2 Consolidation end time of Model R.

5.3.3 Analysis of Excess Pore Water Pressure

The amount of excess pore water pressure was evaluated at four points (points C, D, E and F) for each model as shown on Figures 5.3 to 5.6.

It arrived at maximum amount after completion of each step of embankment construction and also decreases progressively with time until it reaches consolidation end time.

Generally, the higher excess pore water pressure exists at lower layers compared to the ground level.

The figure clearly illustrates the model with geosynthetic reinforced stone column has a substantial impact on increasing the amount of excess pore water pressure.

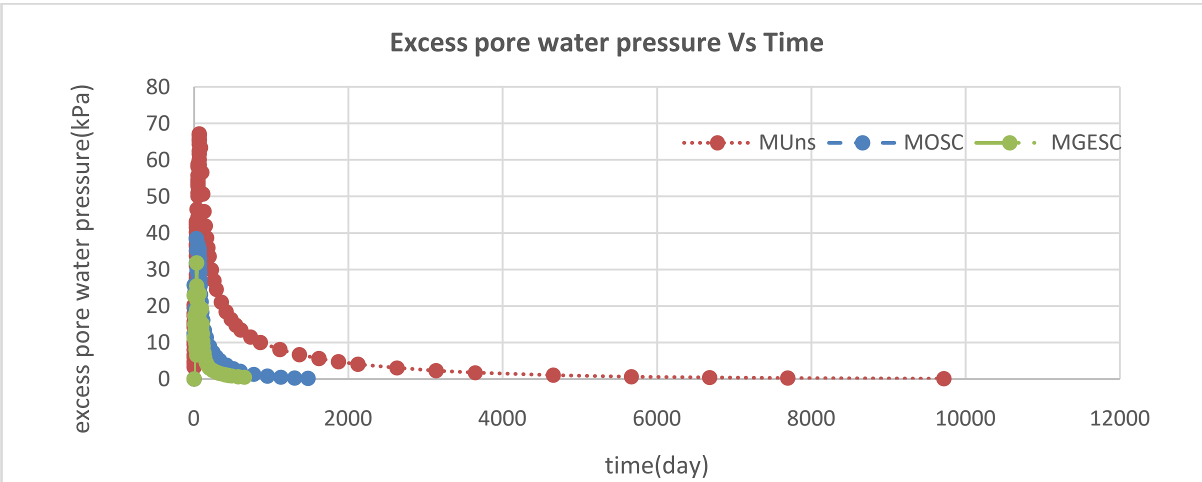

Figure 5.3 Model R: Excess pore water pressure versus Time at point C (0,0,-2.25).

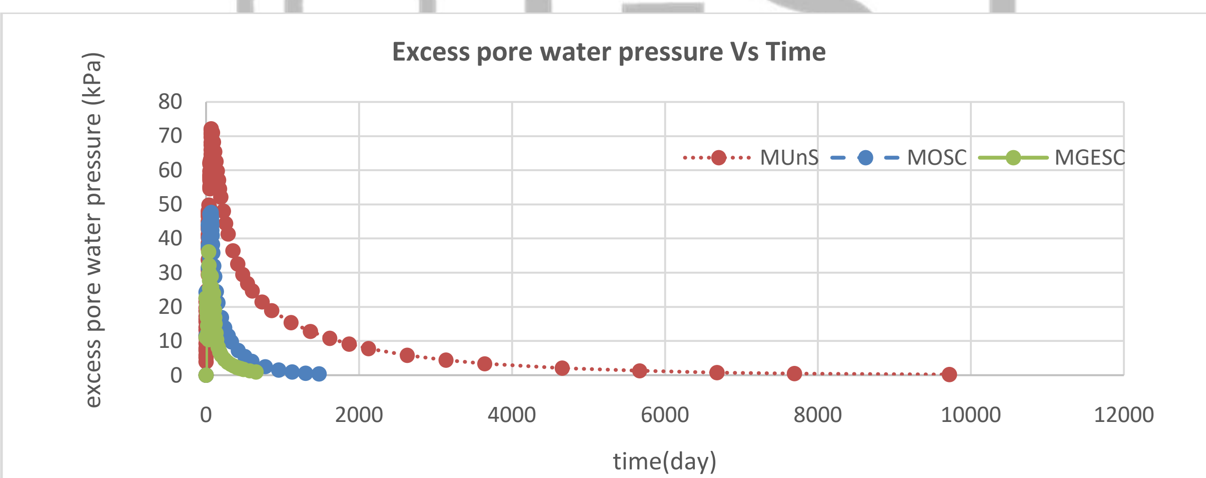

Figure 5. 4 Model R: Excess pore water pressure versus Time at point D (0,0,-4.50).

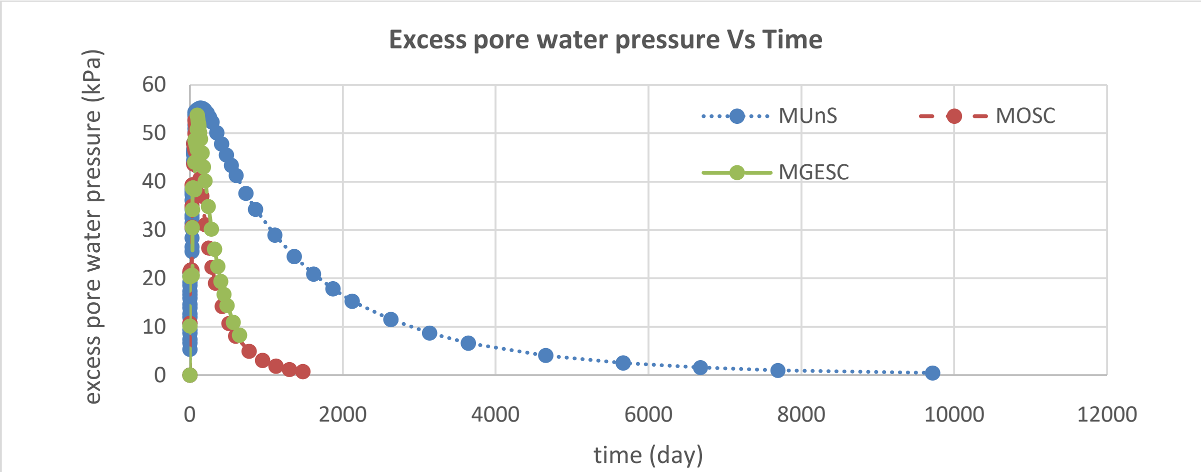

Figure 5.5 Model R: Excess pore water pressure versus Time at point D (0,0,-10.5).

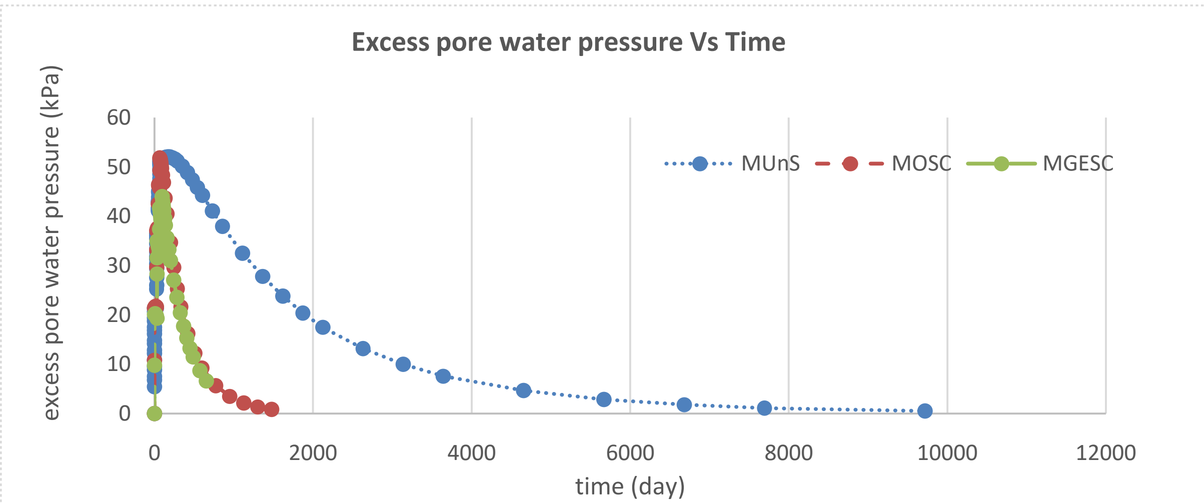

Figure 5.6 Model R: Excess pore water pressure versus Time at point E (0,0,-13).

5.3.4 Settlement Analysis with respect to Time

Settlement versus time relationships at three points (points A, H and I) for unreinforced, OSC and GESC treated soils were analyzed at the consolidation end time (shown on figures 5.7 to 5.9).

Among the selected points the maximum settlement occurred at point A, the middle of the embankment dam, while the settlement at point I shows heaving (swelling) effect.

The great benefit of using geosynthetic encased stone column can be seen from these models. The results obtained from these figures indicates that, the unreinforced soil has maximum settlement, which is 338mm and this amount decreased to 257mm for soils treated by GESC.

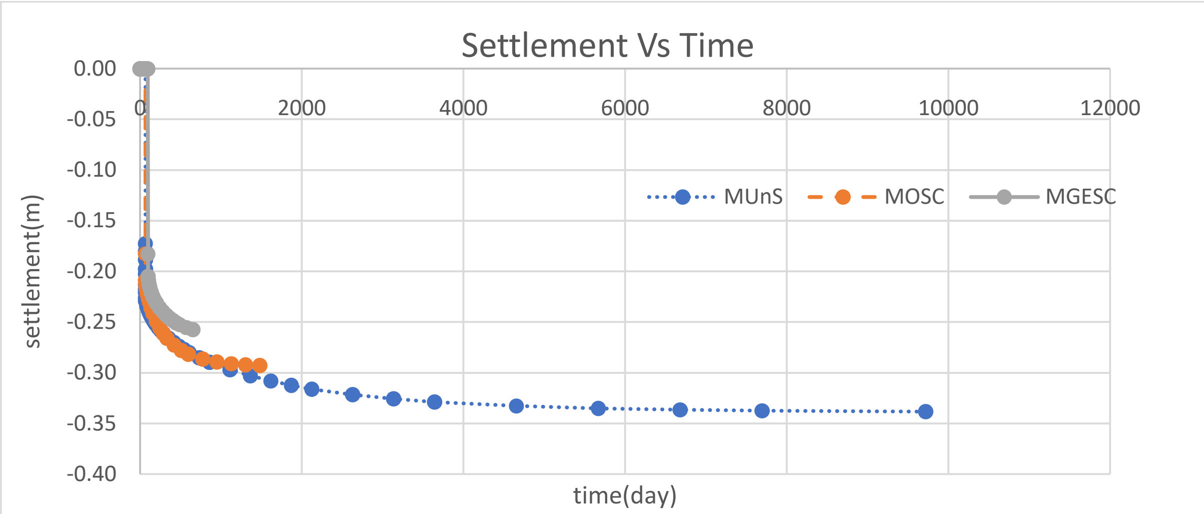

Figure 5.7 Model R: Settlement versus Time at point A (0,0,5)

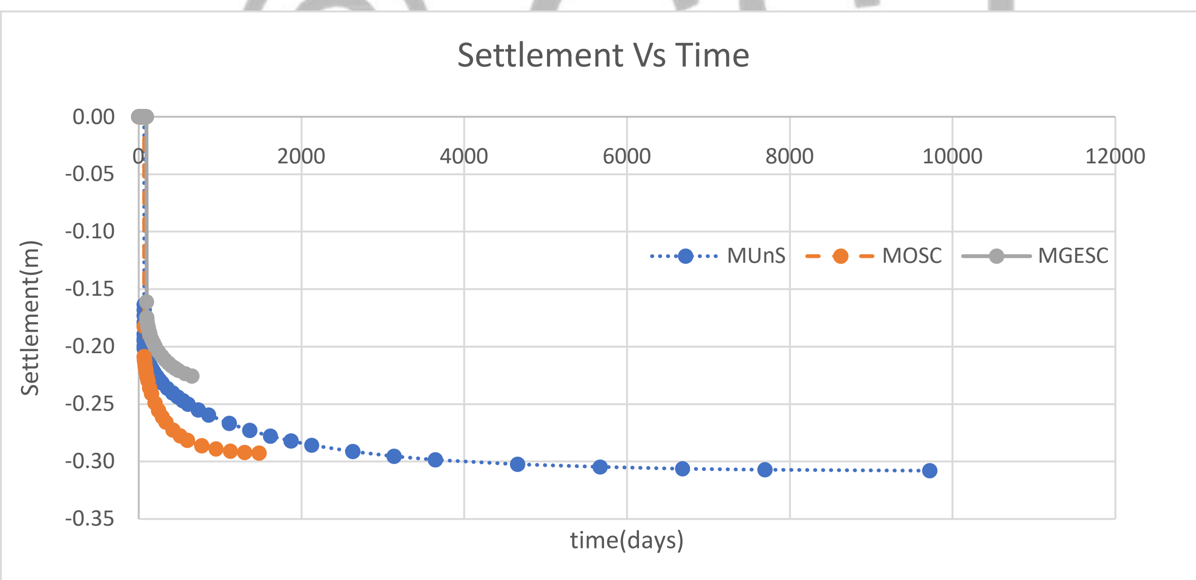

Figure 5.8 Model R: Settlement versus Time at point H (6,0,5)

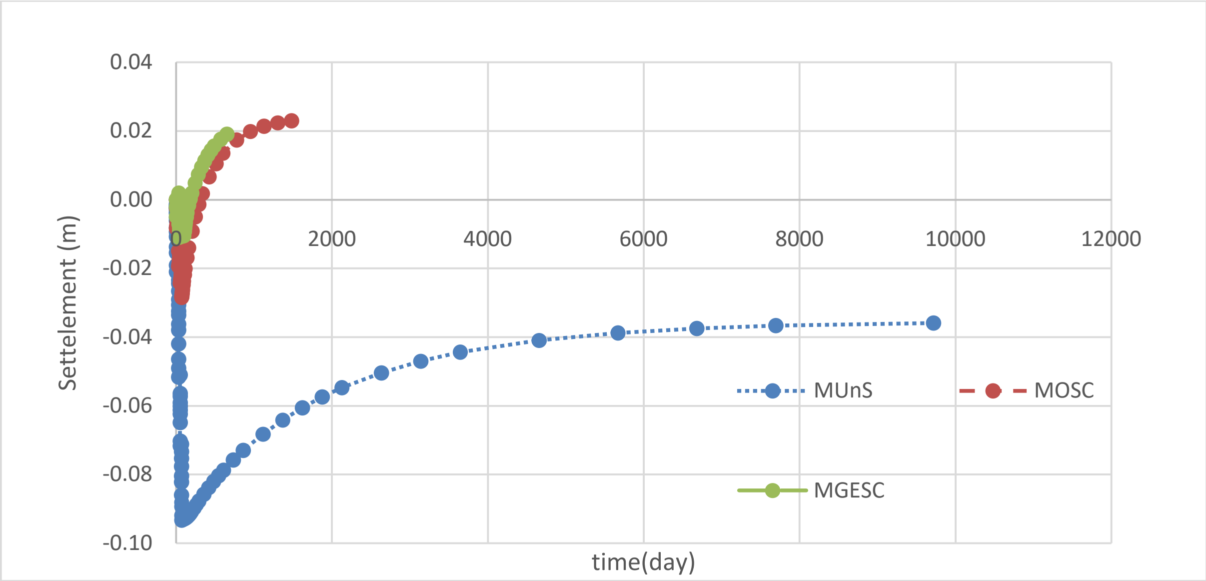

Figure 5.9 Model R: Settlement versus Time at point I(18.5,0,0)

Chapter Six

Conclusions and Recommendations

6.1 conclusions

This thesis has investigated numerical simulation of the behavior of geosynthetic encased stone column by PLAXIS 3D and argued that the GESC is the best method to speed up the consolidation time and control excessive settlement.

From the finite element analysis, the following conclusions can be drawn;

- The results of the finite element analysis showed satisfactory agreement with the field measurements that are conducted by Iman Hossein Pour Babaei. The numerical results predicted well the measured excess pore water pressure and settlement of the embankment dam. The FEM consolidation analysis also clearly shows the expected behaviour of staged-constructed dam.

- The GESC has a substantial influence over OSC in reducing the consolidation end time and settlement. It speeds up the consolidation end time by 55.86% and decreases the settlement by 12.29%.

6.2 Recommendations

This research has investigated the behaviour of geosynthetic encased stone column in collapsible soil numerically. It is believed that the present research will be a good commencement for our country to conduct more studies on this topic.

Hence, there are areas in which further study of GESC can be carried out to improve further the understanding of this ground improvement technique.

Areas for future research include:

- Experimental and analytical study under cyclic loading on the behaviour of stone columns in collapsible soils.
- The study of slope stability problem for embankment dam that is supported by GESC. Even if settlement and consolidation time are the primary concern for stone column reinforced ground, slope stability is also an important case and need to be studied specially when slip surface develops underneath the column toe.
- Evaluation of parametric study on using GESC such as column diameter, length, spacing. column length to diameter ratio, spacing to diameter ratio, area replacement ratio and other parameters.
- The study in response to bulging effect of the GESC in relation to the geotextile stiffness, column height and diameter.

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