STABILITY OF EXCAVATED SLOPES DUE TO GENERATION OF NEGATIVE EXCESS POREWATER PRESSURE

A MAJOR - II

PROJECT REPORT

SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS

FOR THE AWARD OF THE DEGREE

OF

MASTER OF TECHNOLOGY
IN
GEOTECHNICAL ENGINEERING

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AUGUST-2020

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CANDIDATE'S DECLARATION

I, Ankur Sharma, Roll No. 2K18/GTE/22, student of M. Tech. (Geotechnical Engineering), hereby declare that the project Dissertation titled "Stability of excavated slopes due to generation of Negative Excess Porewater Pressure" which is submitted to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the dergree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma, Fellowship or other similar title or recognition.

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Date: 31st August 2020

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CERTIFICATE

I hereby certify that the Project Report titled "Stability of excavated slopes due to generation of Negative Excess Porewater Pressure" which is submitted by, Ankur Sharma, Roll No 2K18/GTE/22, Geotechnical Engineering, Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by the student under my supervision. To the best of my knowledge this work has not been submitted in part or full for any degree or Diploma to this University.

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ABSTRACT

Negative excess pore water pressure or depressed pore water pressure or pore water tensions are formed after a saturated soil in undrained condition is excavated. Because of this short and long term instabilities could arise following the dissipation of this depressed pore water pressure. Just after excavation this depressed pore water pressure starts decreasing slowly depending upon the drainage conditions prevailing in the soil and reach a certain equilibrate or steady state seepage values after a long period of time. Because of this, there are variations in average effective stress values in the zones below and adjacent to this excavated slope, which in turn pose serious threat to the stabilities of such slopes in the long run. In this paper a benchmark geotechnical excavation was investigated in a finite element software MIDAS GTS NX to determine the generation and dissipation of this depressed pore water pressure. Also, the effect of different in-situ stress values on the stability of slopes was studied. A fully-coupled stress seepage analysis was undertaken to simulate the real time dependent analysis. Results show that with higher removal of stresses or higher unloading, higher negative excess pore water pressures were generated which in the short term increased both the effective stress and factor of safety but with gradual dissipation of the same might lead to delayed failure of the slopes.

ACKNOWLEDGEMENT

It gives me immense pleasure in expressing my gratitude to all those people who supported us and have had their contribution in making this major project possible. We would like to express our gratitude to our mentor as well as our HOD who gave me the golden opportunity to do this project on the topic **"Stability of excavated slopes due to generation of Negative Excess Porewater Pressure."**

I owe a debt of gratitude to my guide **Prof. AK Gupta** for incorporating in the idea of this Major-2 Project, helping me in undertaking this project and for being there whenever I needed his assistance. I am immensely grateful to all those who have helped me in making this project better, by providing me their valuable comments and guidance.

I am grateful to **Prof. Nirendra Dev, Head of Department,** Department of Civil Engineering, Delhi Technological University (Formerly Delhi College of Engineering) and all other faculty members of our department, for their astute guidance, constant encouragement and sincere support for this project work.

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CONTENTS

LIST OF TABELS

LIST OF FIGURES

LIST OF ABBREVIATIONS

FOS	FACTOR OF SAFETY
PWP	PORE WATER PRESSURE
NEPWP	NEGATIVE EXCESS PORE WATER PRESSURE
SRM	STRENGTH REDUCTION METHOD
INCR	INCREMENT

LIST OF SYMBOLS

K_O	Earth pressure coefficient at rest
σ_v	Unloaded total vertical stress
σ_h	Unloaded total horizontal stress
σ_v'	Removed effective vertical stress
$\Delta\sigma_1$	Change in maximum principal stress
$\Delta\sigma_3$	Change in minimum principal stress
h	Depth of excavation
γ_t	Total unit weight of soil
γ_w	Unit weight of water
u	Total pore water pressure
Δu	Changes in pore water pressure
u_O	Initial pore water pressure
Δu_e	Changes in excess pore water pressure
u_t	Pore water pressure at equilibrate ground water conditions
t	Time
A	Skempton’s pore water pressure parameter
c_h	Horizontal coefficient of consolidation
c_v	Vertical coefficient of consolidation
E_{50}^{ref}	Secant stiffness in drained triaxial test
E_{oed}^{ref}	Tangent stiffness for primary oedometer loading
E_{ur}^{ref}	Unloading/Reloading stiffness

CHAPTER 1

INTRODUCTION

1.1 GENERAL

Whenever an excavation is being done, it causes an unloading effect over the underlying soil, which in turn leads to the expansion of the soil, provided the soil is being excavated in the undrained condition. Such unloading effect under undrained conditions leads to pore pressure reductions in the soil. These reductions in ore water pressure values from the initial or before excavation are termed as Negative Excess Pore water Pressure.

These negative excess pore water pressure then dissipate with time till they reach a certain steady state seepage value. With this dissipation, the pore water pressure starts increasing and subsequently leads to decrease of average effective stresses which in turn starting posing serious threats to the stability of those excavated slopes.

1.2 EMBANKMENT VS EXCAVATION

When an embankment is made over a clay layer, the load of the embankment causes pore water pressures in the foundation to increase. Then as the times goes by, these excess pore water pressures dissipate try reaching values which are eventually being controlled by groundwater conditions. As the process of dissipation of this excess pressure moves forward, the strength of the clay increases and consequently so does the factor of safety of the embankment increases. It is depicted in Fig 1.

Similarly when an excavation is being made in a clay layer, the pore water pressures do changes, but instead of increasing, this time they decrease due to the removal of the excavated material. After this reduction in pore water pressure they start increasing depending on the groundwater conditions. And as the pore water pressure increases, effective strength starts decreasing and so does the factor of safety of the excavation. This is represented in fig 2.

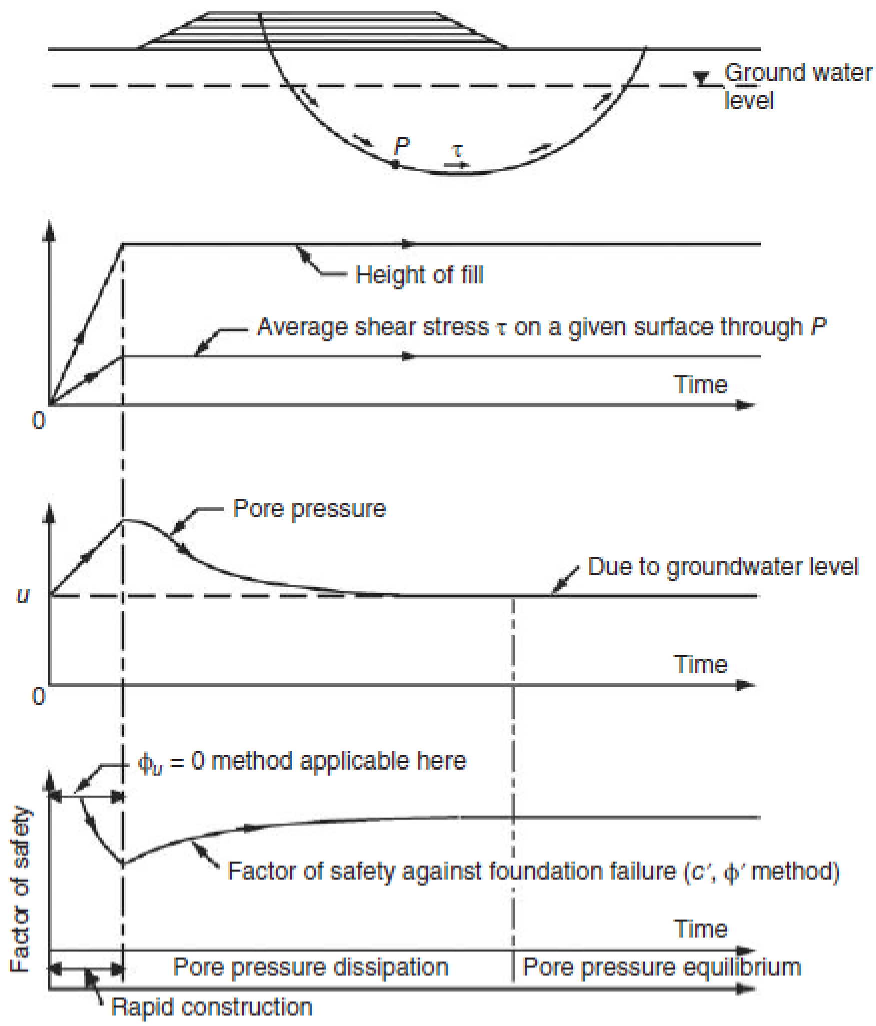

Fig. 1.1 Variation of shear stress, pore water pressure and factor of safety with time in an embankment over a clay layer (Source: Bishop and Bjerrum, 1960)

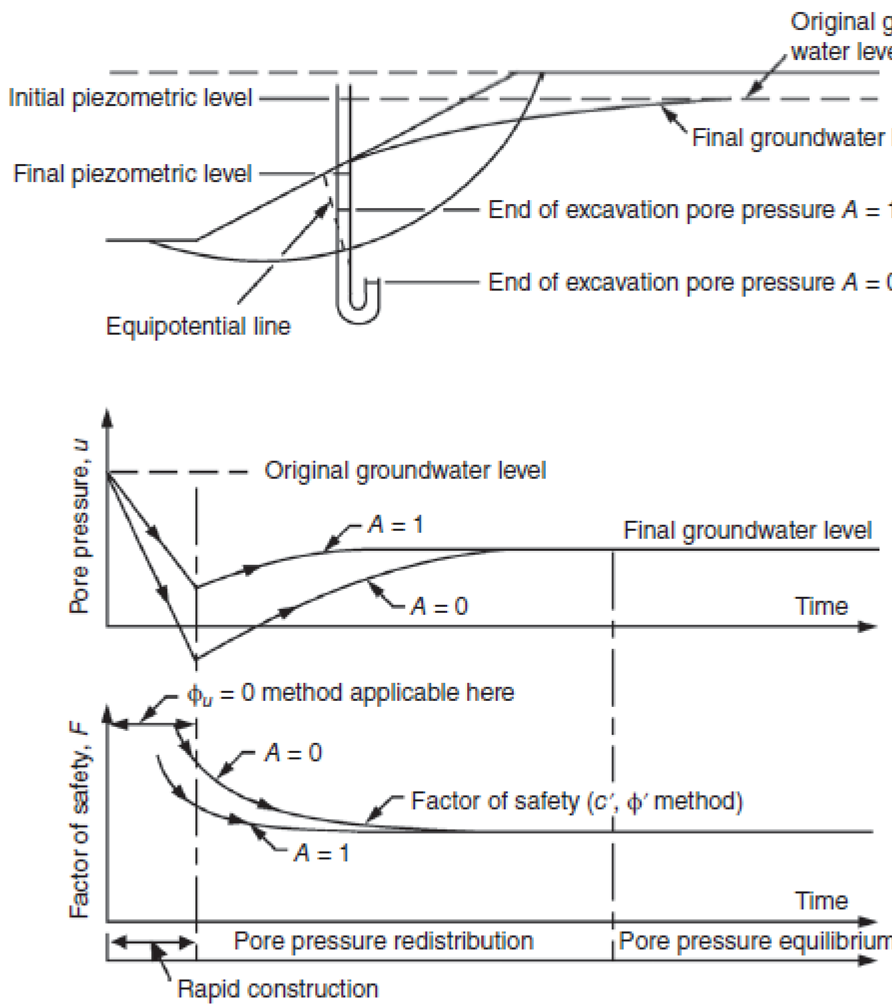

Fig. 1.2 Variation in pore water pressure and factor of safety with time in an excavation in a clay bed (Source: Bishop and Bjerrum, 1960)

In Fig. 2, the two curves denoted by A=1 and A=0 gives us the values for maximum and minimum changes in the pore water pressures in the soil strata below the excavation.

If the height of the excavation remains constant and there is no external load being applied, then the factor of safety keeps on decreasing continuously. The decreasing FOS attains it minimum value when the pore water pressures in the excavated soil attains its equilibrium value corresponding to groundwater seepage conditions. Therefore, in these cases, the long term stability of the excavation or any construction is more important than the conditions just after completion of the construction activity.

Hence, the gist of this is as follows:

a) If the stability just after the End-of-Construction is to be analyzed, it can be done by either using drained or undrained strengths depending upon the permeability of the soil.

b) And if long term stability, i.e. after a long period of time after the end of construction is to be analyzed, after the consolidation or swelling process is done, it can be done by using drained parameters and pore water pressures analogous to steady seepage conditions.

1.3 DRAINED VS UNDRAINED CONDITION

Generally, sand are considered to be always in drained conditions whereas clays are considered always to be in undrained conditions. But in reality the distinction between undrained and drained conditions doesn't depends solely on the type of soil but majorly on the rate of loading and rate of drainage. If the rate of loading is more than the rate of drainage then it is considered to be in drained condition where as if the rate of loading is less than the rate of drainage than it is considered to be in undrained condition.

1.4 EFFECTIVE STRESSES

It is the stress which is being transferred in the soil mass by grain-to-grain contact which tends to force the soil solids to come in closer contact with each other, resulting in increased denseness, stability, strength and reduced void ratio and mobilization of shear strength.

Since the pressure is transferred by grain-to-grain contact, it is termed also as intergranular pressure. Also, being a non-physical parameter, it cannot be computed directly and is computed by subtracting the neutral stress from the total stress.

1.5 PORE WATER PRESSURE

It is the pressure which is being transferred by the pore fluid and is equal to the height or weight of the fluid (water in our case) column above the concerned section in the soil mass

This water pressure acts all around the soil solids, hence not allowing the soil solids to come in closer contact to each other. This pressure also doesn't have any shear component. This pressure is also known as neutral pressure.

1.5.1 Positive Excess Pore water Pressure

This positive excess pore water pressure is also simply termed as excess pore water pressure. When a surcharge is applied instantly and water table level is at ground level, hat surcharge is being carried by the pore water, which in turn increases the pore water pressure, which, depending upon the permeability conditions of the soil, leads to seeping out of that pore water from the voids of the soil, subsequently transferring the surcharge over the soil solids.

1.5.2 Negative Excess Pore water Pressure.

Negative excess pore water pressure is a type of pressure which usually comes into effect when an over consolidated cohesive soil for e.g. clay, undergoes undrained shearing.

Excavation of a slope or a cut in analogous to unloading of the ground, which leads to expansion under undrained conditions and hence causing pore water pressure reductions and the amount of reduced value from initial pore water pressure is termed as Negative Excess Pore water Pressure.

These pore water pressure because of unloading are negative in comparison to the initial or final equilibrium values. These negative excess pore water pressures keeps on dissipating until steady state seepage conditions are acquired. This dissipation leads to increase in pore water pressure which in turn decreases the average effective strength in the slope and this reduction in turn leads to the failure of the slope in the long term.

Also, this dissipation of negative excess pore water pressure is accompanied by swelling of the soil. The process of swelling is generally considered to be the reverse of consolidation process, in which the volume of the soil gradually increases, instead of decreasing as is in the case of consolidation. Hence, dissipation of this excess pore water pressure in many cases may lead to upward movement of the soil, also called Heaving.

1.6 FULLY-COUPLED STRESS-SEEPAGE ANALYSIS

An analysis which couples ground stress analysis and seepage phenomenon can be categorized in different ways, depending upon the coupling. The generalized way is to first get the pore water pressure distribution by performing a seepage analysis initially and then showing it in the total stress or effective stress relating equation of stress analysis. This type of analysis is called sequential analysis. This analysis is used to know the static stress state for the given steady groundwater flow. But since the changes in stresses during stress analysis does not affect the seepage phenomenon in return, hence there doesn’t exist a two-way coupling.

Fully coupled stress seepage analysis, on the other hand, is a two way couple analysis in between the seepage and stress analysis. It can depict the changes in pore water pressure, deformation and stress with time.

1.7 STRENGTH REDUCTION METHOD (SRM)

In strength reduction method the strength values of the soil are reduced by a factor until the loss of stability or failure of the structure occurs. The reciprocal of this reduction factor is identified as the factor of safety associated with the soil model under investigation.

1.8 OBJECTIVE OF THE STUDY

The main objective of our study is to analyze the phenomenon of generation and dissipation of pore water tension upon excavation of a soil under undrained condition and its effect on the stability of the slopes both in the short and long term. Also, the effect of different in-situ stress conditions on the magnitude of this pore water tension and the corresponding time taken for the failure of slope is studied.

CHAPTER 2

LITERATURE REVIEW

Bishop and Bjerrum (1960) conducted a triaxial test on analyzing the generation of excess pore water pressure on undrained compression of a soil sample. He also studied undrained excavation. He studied the how shear stress changed with the generation and dissipation of these depressed pore water pressures and observed that the shear strength in the studied zone decreased gradually with gradual dissipation of these depressed pore water pressures and the total pore water pressures increased.

Vaughan et. al. (1973) studied the effect of rate of equilibration of pore water pressures on the stability of cuts made in over consolidated clay. He assumed the process of dissipation of this negative excess pore water pressure to be following the Terzaghi's theory of 1D consolidation. He analyzed his work using average pore pressure ratio,r_u, which is the ratio of pore water pressure and depth of soil above the point concerned. This ratio kept on increasing towards an equilibrium value. He studied pore water pressure equilibration in M1 Motorway, 9 years after its construction in 1964, which had a London clay soil properties and found out that there was still significant amount of negative excess pore water pressure there even after so many years. He attributed this behavior to the low permeable London clay soil. Hence he concluded that the delayed rate of equilibration played a significant role in the delayed failure of such slopes and deemed the theory of delayed failure was due to drop in drained strength of soil with time as unnecessary and not significant. He also hold out that the depressed pore water pressures after excavation are predominantly formed both adjacent and below the excavation pit.

Eigenbrod (1974) studied the changes the in pore water pressures generated due to slope excavation in plain strain over consolidated clays and compared them with many field studies. He observed that the time difference between the dissipation of the excess pore water pressure generated due to excavation and the time between then end of excavation and failure of the slope of the excavation was of the same order and emphasizing that many delayed failures could occur due to late equilibration of these generated excess pore pressures. He simulated the unloading effect over the excavated surface by applying forces equal in magnitude but opposite in direction to that of the forces which were there before excavation. He concluded that that the process of dissipation of pore water tension and the consequential decrease the effective stress in the slope was the major reason in the delayed failures of the excavated slopes.

Skempton (1984) analyzed a number of field case studies which had Brown London clay as the soil material and showed stiff fissured behavior. He concluded that the main reason after the delayed failures of such slopes was the slow dissipation of theses depressed pore water pressures. He also observed that the strength at the long-term slip surfaces corresponded to the fully softened condition or fissured strength and the in situ strength was greater than the residual strength but smaller than the peak strength, indicating that progressive failure also played a role in the failure of such slopes.

Yu. Qi. L. et al (2005) tried determining the “dissipation rule” for the generated negative excess pore water pressure after excavation. He derived analytical formulas for calculating this excess pressure and took into consideration the Terzaghi’s one dimensional consolidation theory and effective stress principle to calculate the effective stresses induced in such excavated slopes. He took in to consideration a foundation pit excavated and supported by a retaining wall and analyzed the dissipation of this pressure both outside and inside of this foundation pit. He concluded that this negative excess pore water pressure posed a serious threat to these retaining walls and also one could make full use of this excess pore water pressure for any rapid construction provided suitable safety provisions are made available for its safety in the long run.

Potts. et al (2009) performed a series of coupled finite element analysis of cut slopes with the properties of brown London clay and assuming strain-softening behavior. He analyzed the phenomenon of Progressive Failures in the slopes. Progressive failure means non uniform spreading of shear strength along a potential surface of rupture. He observed that for a slope to show delayed failure due to progressive failure, the soil must show brittle behavior or have brittle properties and nonlinear strains should be spread along any soon to be rupture surface. He also observed that to limit equilibrium method was not suitable for taking into effect the phenomenon of progressive failure and finite element analysis must be taken into account for its study. He considered a elasto-plastic model of Brown London clay with varying values of effective friction angle, cohesion and permeability. In his study, he attributed the main cause of delayed failure of the selected slope to be progressive failure, promoted by swelling.

Lollino. et al (2011) studied a case of a landslide occurring on a cut slope at Lucera, Italy. He performed a numerical analysis to the analyze the movement of slope with and the generation of excess pore water pressures after quarrying. He too attributed negative excess pore water pressure generation to the temporary stability gained by the cut in the long term due to undrained loading condition and its dissipation to the delayed failure of the slope. He also propounded that limit equilibrium method was not suitable for such analysis because it cannot determine the pore water pressures and its variation at any intermediary point of time during the process of consolidation and can only be used in assessing the stability of slope soon after the excavation and or when equilibrium has been achieved in the long term. That’s why he used a coupled-consolidation analysis to study the behavior of excavated slopes on a time-dependent manner. He said that such numerical procedures that uses soil-water coupling effects must be used to study the extent of time that a slope takes to fail owing to changes in pore water pressures with time.

Conte. et al (2012) proposed a simplified analytical method for analyzing the stability of clayey slopes when subjected to pore water pressure changes. His main aim was to overcome the current shortcomings while analyzing the effect of pore water pressure changes on the stability of slopes. In many of the current geotechnical engineering applications, the effect between the two is studied in an uncoupled manner with first calculating the pore water pressure changes at the potential or pre-defined failure surface and then using them for assessing the slope stability via limit equilibrium method. In his study, he gave a simplified method for analyzing the stability of slopes subjected to fluctuations in groundwater by considering infinite slope model.

After studying a number of research works, the common gist which can concluded is that soils when excavated in undrained conditions leads to the generation of negative excess pore water pressure or pore water tension or depressed pore water pressure. This excess pore water pressure imparts additional strength to the slope in short term but with time, as the excess pore water pressure dissipates, so does the strength of the slope reduces and thus leads to the failure of the slope in the long term. Also, many works considered this generation of negative excess pore water pressure as a phenomenon just reverse of consolidation and hence extended Terzaghi's theory of consolidation in this phenomenon also. The maximum amount of research has been done to study this phenomenon of generation of pore water tension after excavation but it's dissipation with time and its effect on the stability of slopes is studied to a lesser extent. Therefore, in this study, I have investigated an open pit mine excavation under undrained unloading condition via a fully-coupled stress seepage analysis by finite element method to analyze the effect of generation and dissipation of this excess pore water pressure on the stability of excavated slopes with varying values of earth pressure coefficients (K_O).

CHAPTER 3

METHOD AND WORKING

3.1 METHODOLGY

After excavating and dewatering a pit, assuming ground water table to be at the top surface i.e. a fully saturated soil and hydrostatic pore water pressure distribution on the ground, following will be the unloaded stress i.e. horizontal and vertical stresses (Eigenbrod, 1975):

$$\sigma_v = \gamma_t h \qquad (1)$$

$$\sigma_h = K_O \sigma_v' + u = h[K_O(\gamma_t - \gamma_w) + \gamma_w] \qquad (2)$$

The change in the pore water pressure Δu due to this unloading phenomenon can be calculated for saturated soils by the following equation (Skempton, 1954):

$$\Delta u = \Delta\sigma_3 + A(\Delta\sigma_1 - \Delta\sigma_3) \qquad (3)$$

These principal stresses were taken to be acting on the vertical and horizontal edges prior to excavation. Because of unloading, rotation of principal stresses occur. But these rotations were not that substantial and hence the stress changes $\Delta\sigma_v$ and $\Delta\sigma_h$ in vertical and horizontal directions were taken to be same to the changes in principal stresses $\Delta\sigma_1$ and$\Delta\sigma_3$. Hence, the changes in the pore water pressure due to excavation will be:

$$\Delta u = \Delta\sigma_3 + A(\Delta\sigma_h - \Delta\sigma_v) \qquad (4)$$

And the pore water pressure just after unloading u_O will be:

$$u_O = u - \Delta u \qquad (5)$$

This initial pore water pressure u_O immediately after unloading tends to equilibrate till the equilibrium ground water states are reached.

The difference in between the pore water pressures just after unloading u_O and the equilibrate ground water state u_t gives us the excess pore water pressure Δu_e in comparison to the last stage:

$$\Delta u_e = u_O - u_t \tag{6}$$

The dissipation of excess pore water tension is considered to be the opposite of that in consolidation and hence follows the conventional Terzaghi's one dimensional consolidation theory. Also, this one dimensional theory was extrapolated to two and three dimensional forms as follows (Yu. Qi. L. et al, 2005):

$$c_h \frac{\partial^2 u}{\partial x^2} + c_h \frac{\partial^2 u}{\partial y^2} + c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t} \tag{7}$$

In the present study, to analyze the effect of the generation and dissipation of this depressed pore water pressure, a fully coupled stress seepage analysis and strength reduction method finite element method was adopted in a finite element software named MIDAS GTS NX.

3.2 MIDAS GTS NX

MIDAS GTS NX is a 2D finite element program used for the evaluation of soil-structure interactions. GTS NX can be used to perform step-by-step analysis of excavation, banking, structure placement, loading and other factors that directly affect design and construction. The program supports various soil constitutive model, varying water level option and analytical methodologies to simulate real phenomena.

Settings for all types of field conditions can be simulated using non-linear analysis methods (such as linear/non-linear static analysis, linear/non-linear dynamic analysis, seepage and consolidation analysis, slope safety analysis) and various coupled analysis (such as seepage-stress, stress-slope, seepage-slope and nonlinear dynamic-slope coupled analysis).

3.3 NUMERICAL MODELLING

For our analysis, we have taken a case of an excavation of an open pit mine. The height of the excavation is 70 m which was excavated in the time period of 180 days. The benchmark excavation is shown in Fig.3.

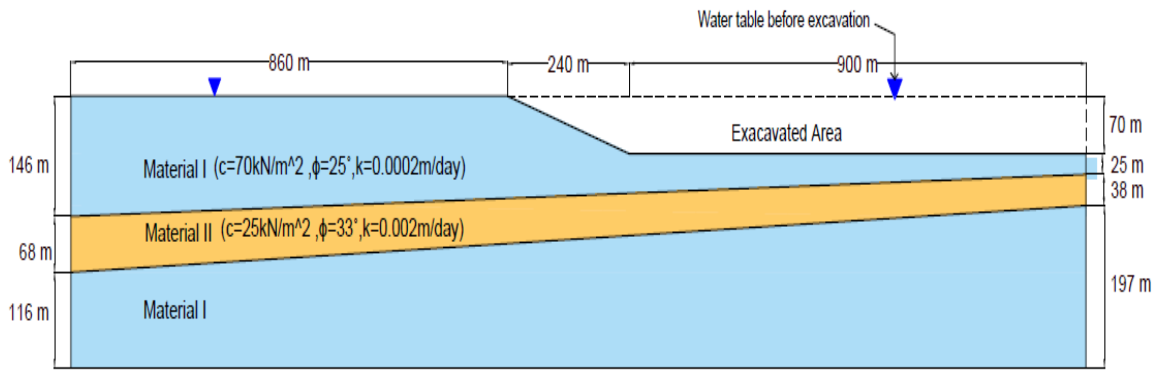

Fig.3.1 Geometry model

3.3.1 MATERIAL AND ITS PROPERTIES.

The soil material taken is to follow the Hardening Soil (HS) model. The hardening soil model is considered to be an advanced soil model through which we can compute more realistic responses of soil in relation to non-linearity, inelasticity and stress dependency. Both the materials followed 2D Plain strain behavior. The properties of the material are tabulated in Table 3.

3.3.2 GEOMETRY

A 2D open pit mine was modelled with dimensions given in fig. 3.1 and properties of the material given in Table 3.Initially, a full rectangular soil was modelled and then afterwards a part of the soil as per dimensions of the excavation was removed to simulate the unloading phenomenon.

PROPERTIES	MATERIAL (I)	MATERAIL (II)
Dry unit weight (kN/m^3)	22	21
Saturated unit weight (kN/m^3)	23	23
E_{50}^{ref} (MPa)	300	200
E_{oed}^{ref} (MPa)	300	200
E_{ur}^{ref} (MPa)	900	600
Cohesion (kN/m^2)	70	5
Friction angle (degrees)	25	33
Permeability (m/day)	0.0002	0.002
K_O	1,1.5,2	1,1.5,2

Table 3.1 Properties of the materials (Tolooiyan et al. 2019)

3.3.3 BOUNDARY CONDITIONS

To ensure that the effect of stress changes were captured fully, the geometries of the mesh were extended to large amount of lengths in both vertical direction measured from the ground surface and in horizontal direction in the direction of the toe and the crest of cut. The vertical boundaries were fixed for vertical movement but horizontal movement was allowed whereas the bottom layer was fixed in both vertical and horizontal direction and the upper layer was free to move. Regarding the hydrostatic boundary conditions, seepage was not allowed from the bottom layer but was allowed from the vertical boundaries. Also the groundwater was taken to be at the top surface i.e. the soil was considered fully saturated.

3.3.4 MESHING OF THE MODEL

A 2D finite element model was meshed sufficiently fined so as to capture all the changes in various attributes of the soil with size of each node being 8 units. To generate this mesh, 2D AUTO-FACE option was selected under the Generate Mesh tab. In MIDAS GTS NX, one can generate the mesh with a particular shape. We can select either a triangle, a quadrilateral or a combination of both. Here, for accurate and time efficient calculation of pore water pressure and stress at nodal points, we opted for a combination of both the shapes.

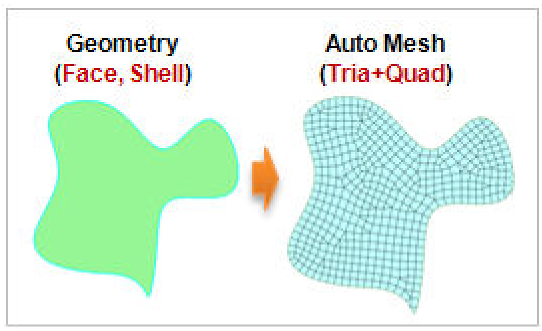

Fig 3.2 Type of Mesh

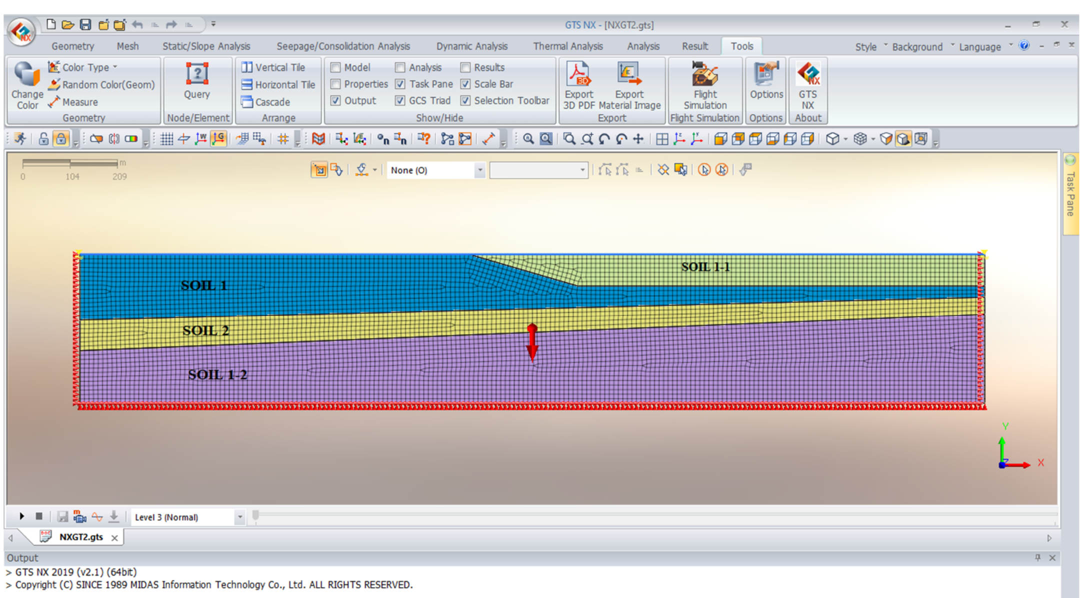

Fig 3.3 Initial mesh configuration with boundary supports and self-weight

Self-weight of the soil was also attributed to the soil which is acts at the center of the mesh. After this, the portion to be excavated was deactivated in stage set option to simulate the unloading or excavation process. The excavated mesh model is shown in

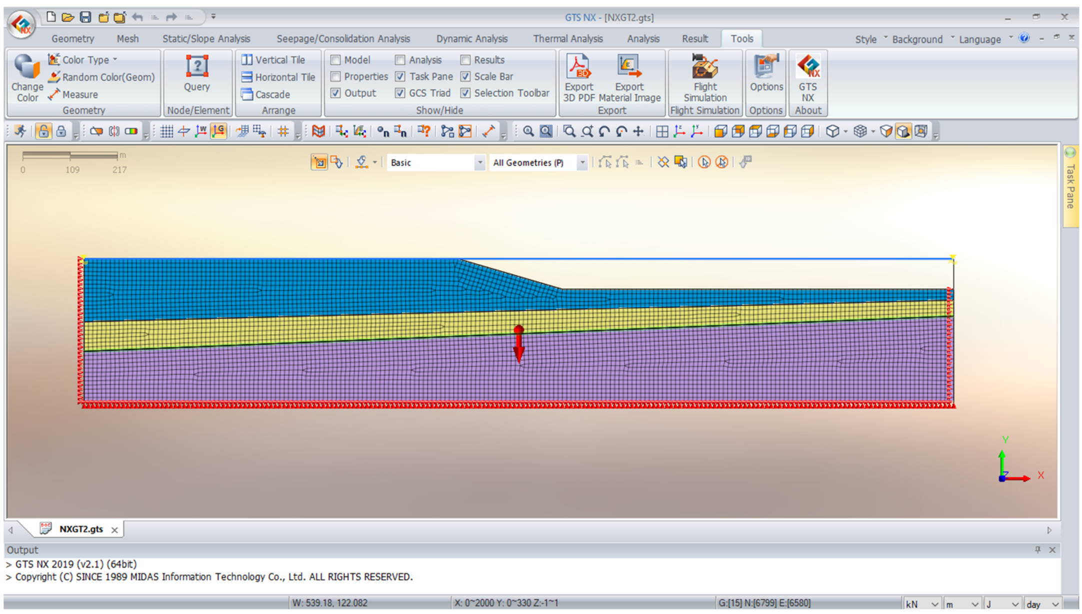

Fig 3.4 Final excavated mesh configuration

3.4 ANALYSIS

After meshing the model and assigning self-weight and suitable boundary conditions, Construction Stage Analysis solution type was used to simulate the different stages of construction in this excavation process.

3.4.1 Construction Stage Analysis

Construction stage analysis was taken into use to simulate the construction process undertaken while excavating the soil model take into account. In this stage, we can add multiple stages, each having its own set of loading/unloading conditions, boundary conditions and elements which can both be added or removed at each stage. Under this stage, to account for transient seepage phenomenon while doing stress or pore water analysis, Fully-Coupled Stress-Seepage analysis was adopted. In our study, following three construction stages were considered:

- Stage 1 : Initial Stage
- Stage 2 : Excavation Stage
- Stage 3 : Final Stage

1) INITIAL STAGE: This Initial State is generally the very first stage of any construction stage analysis. In this first stage, in-situ stresses in the soil strata were calculated. MIDAS GTS NX uses self-weight to calculate these initial in-situ stress by initializing the earth pressure at rest K_O value. This procedure is known as K_O and this method uses the constant K_O as defined by $K_O = \sigma_h / \sigma_v$ to determine the horizontal stress from the vertical stress to set it as the in-situ stress.

Using this method, the vertical stress σ_v is found first using self-weight analysis and that value is then used to compute the horizontal stress using $\sigma_h = K_O \sigma_v$. Here, the shear stress maintains its value, calculated from the analysis result.

In this stage, the boundary conditions as defined earlier were activated. Also, the self-weight was activated, which acts at the centroid of the model geometry, as shown in Fig 3.3.The water table as modelled at the ground surface was also activated. Also, to nullify any displacement caused by the overlying soil, which soon will be excavated in later stages, that displacement was cleared in this stage itself, by activating Clear Displacement option. It sets the displacement of the analysis result in this initial stage as 0. It is used to set the initial conditions of the in-situ state. The stress is not reset to 0.

2) EXCAVATION STAGE: In this stage, the soil element named as SOIL1-1, as depicted in Fig 3.3 was deactivated. Also, various hydrostatic boundary conditions as defined earlier were also activated in this stage. For no seepage from the bottom layer, the Surface Flux through that layer was set as 0 m^3/day. This excavated model is shown in Fig 3.4.

3) FINAL STAGE: In this stage, everything is kept same as the earlier stage except a new boundary conditions is added, namely Review boundary condition. The upper

surface is attributed this Review boundary condition. Under this boundary condition, iterative calculations is conducted when the exact seepage line is hard to find. Also, for this, the time period for this stage of construction is defined as 3000 days and this process is divided into 100 steps. Also, stability analysis via strength reduction method (SRM) was also checked to get FOS values after every step.

The above laid analysis was run for three different values of K_O. These different values of horizontal earth pressure at rest K_O were taken to see its effect on the generation and dissipation of negative pore water tension. There were stages two at which the results were taken into account:

A fully coupled stress seepage analysis was taken into consideration which is two way coupled analysis between seepage and stress analysis with the help of which one can find pore water pressure, stress and deformation changes with time.

3.4.2 Analysis Control

After setting the construction stage, Analysis Control option was used. Under this option, basic options, automatic settings and various advanced analysis options can be checked and changed depending on the selected analysis type. For our analysis, the following options were selected to take them into consideration while the software runs the given model:

- Automatically consider water pressure
- Include in-situ analysis with Self weight and Apply K_O condition
- Consider geometry nonlinear effects
- Update pore pressure with deformation

After this, the model with all the attributed properties and construction stages was run to get the output.

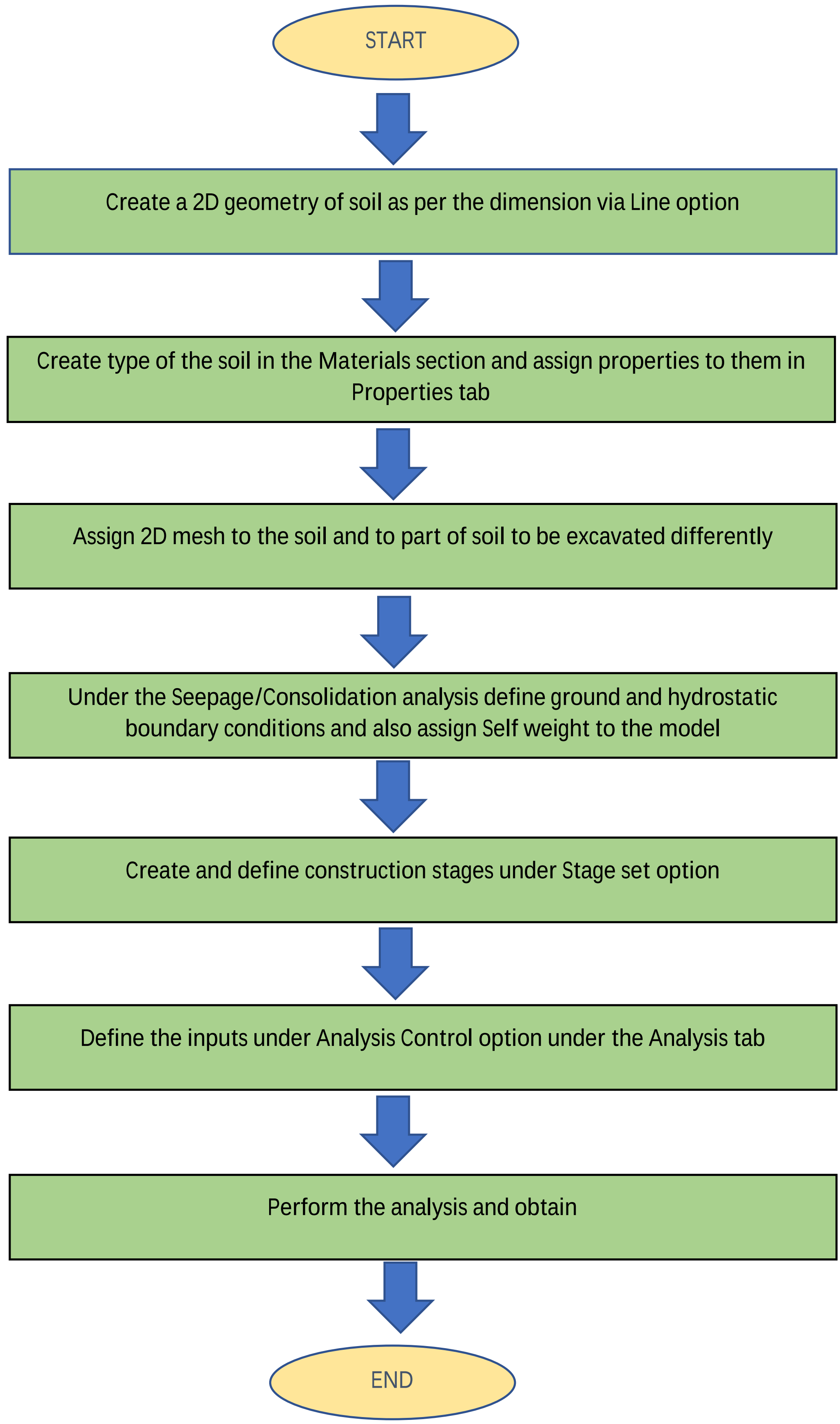

Fig 3.5 Flowchart regarding working in MIDAS GTS NX

CHAPTER 4

RESULTS AND DISCUSSIONS

4.1 EFFECT OF DEPTH OF EXCAVATION

The 70 m soil was excavated in the time period of 180 days. In the software, this time period was divided into ten stages, each increment (INCR) in time denoting 18 days. The pore water tension at different stages of excavation for K_O=1 is shown in Fig. 4

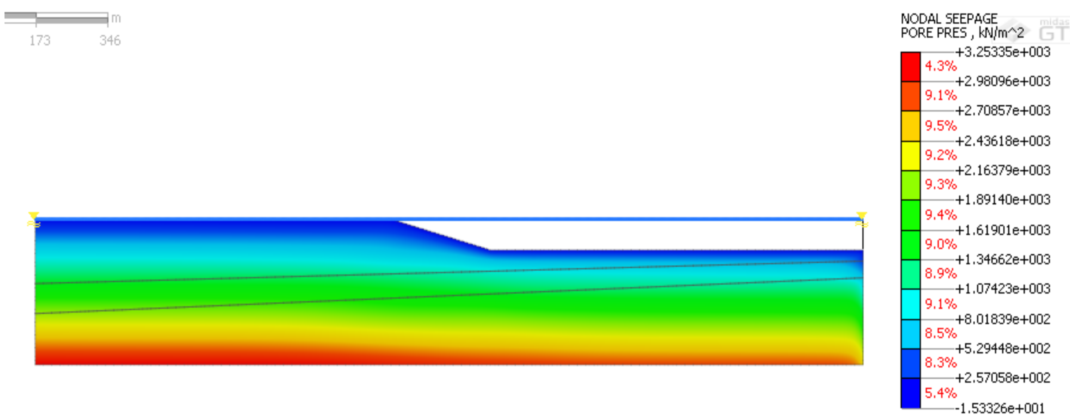

Fig. 4.1 Pore pressure at INCR=1 (Initial stage of excavation)

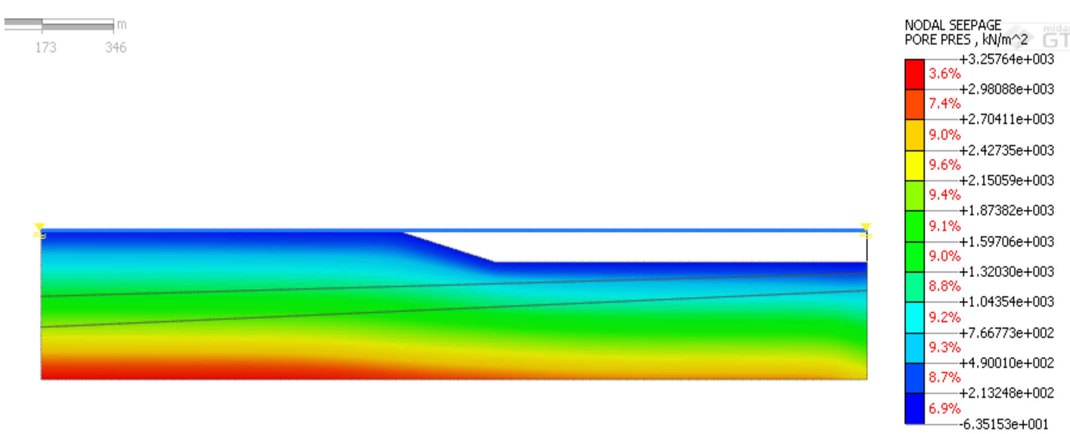

Fig 4.2 Pore pressure at INCR=10 (After the end of excavation)

4.2 EFFECT OF K_O

Here, for different values of K_O i.e. for K_O = 1, 1.5, 2, pore water pressures just after excavation is shown.

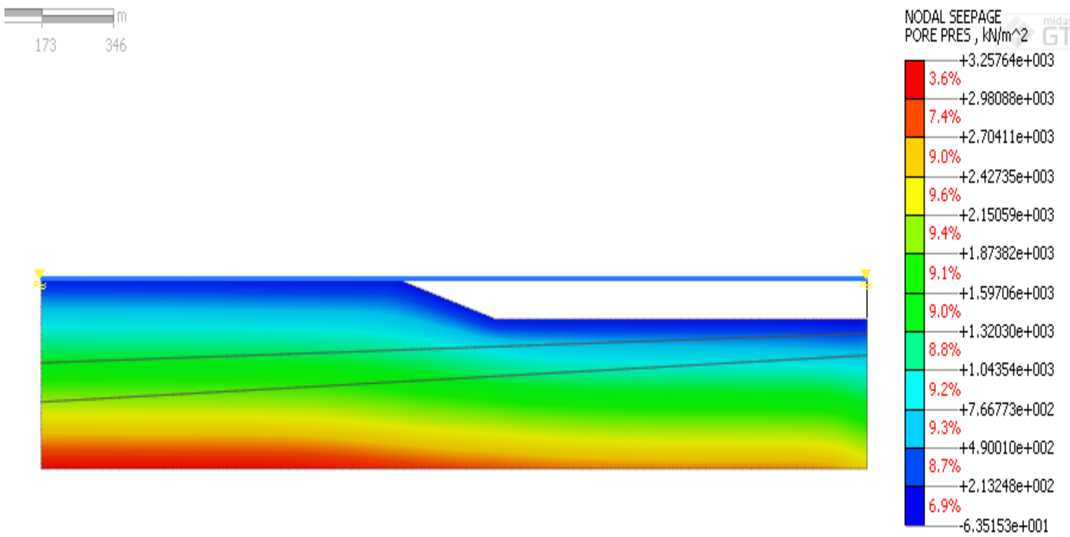

Fig.4.3 Pore pressure just after excavation for K_O = 1

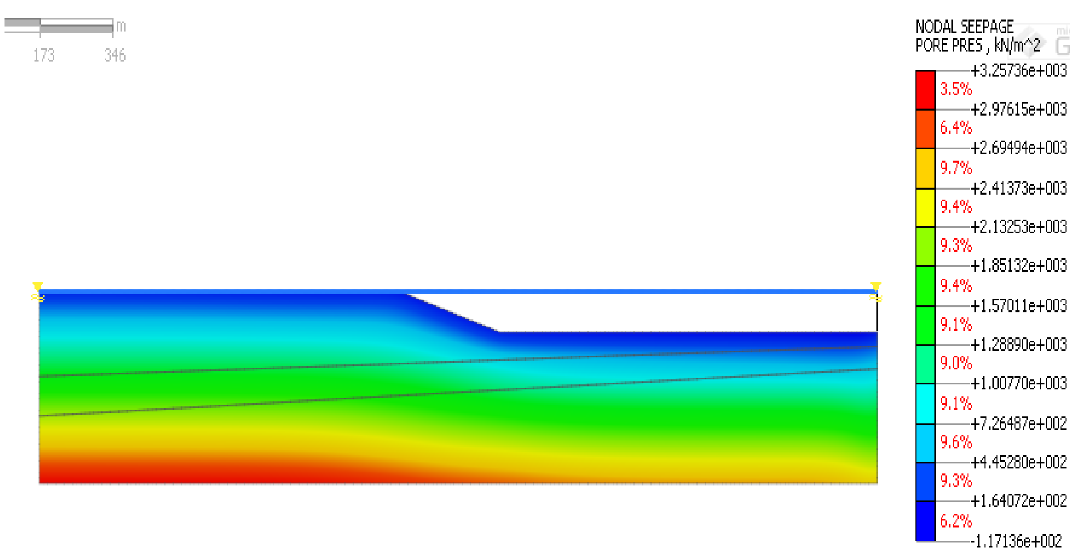

Fig 4.4 Pore pressure just after excavation for K_O = 1.5

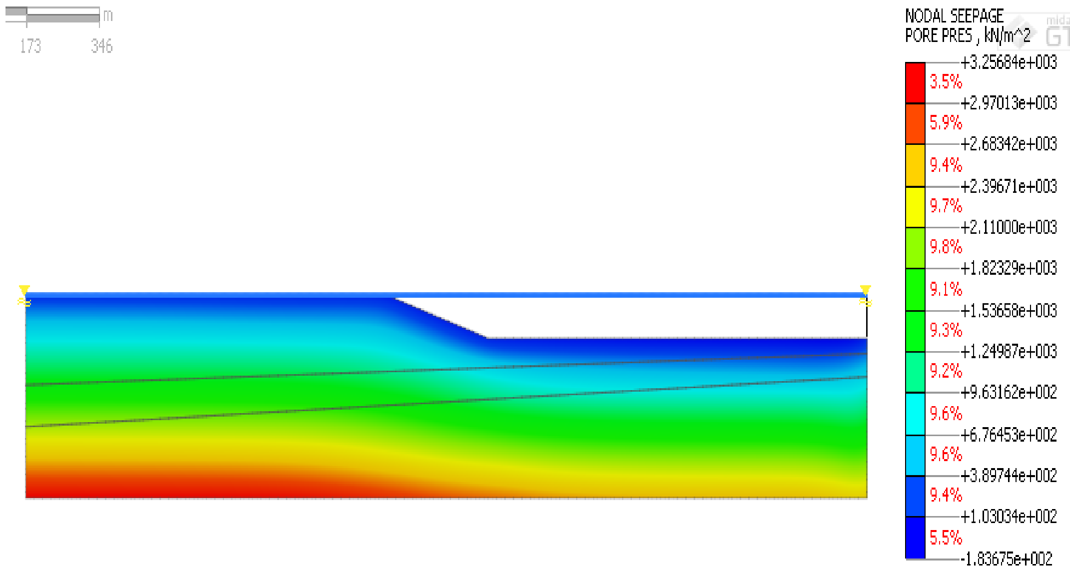

Fig 4.5 Pore pressure just after excavation for K_O = 2

4.3 DISSIPATION OF NEPWP

After the excavation was completed, the excavated slopes were left idol for a period of 3000 days to analyze the dissipation behavior of this pore water tension with time. Here, this time period of 3000 days was divided into 100 steps thereby each increment step resonating with 30 days.

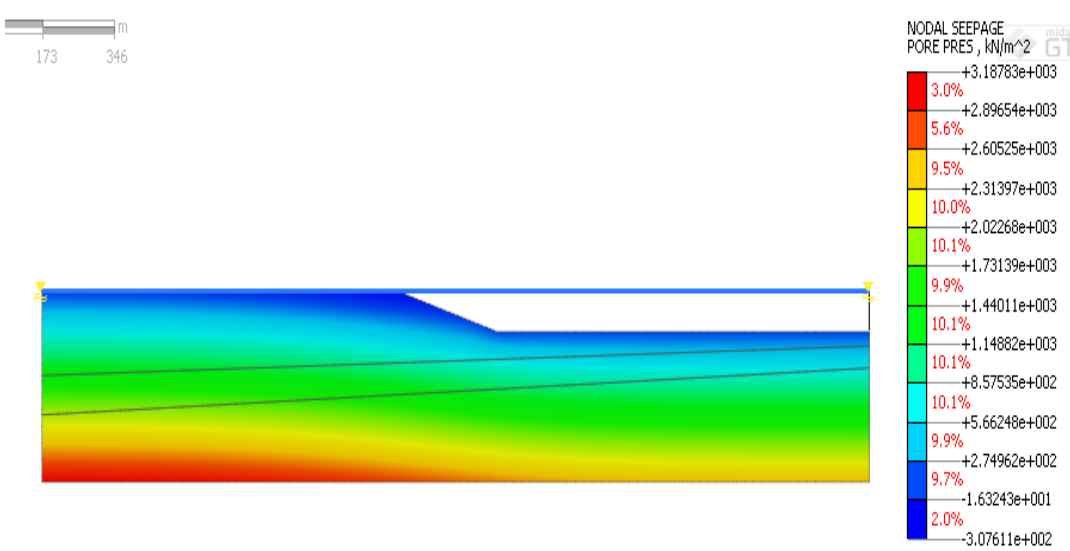

Fig 4.6 Pore pressure after 3000 days for K_O = 1

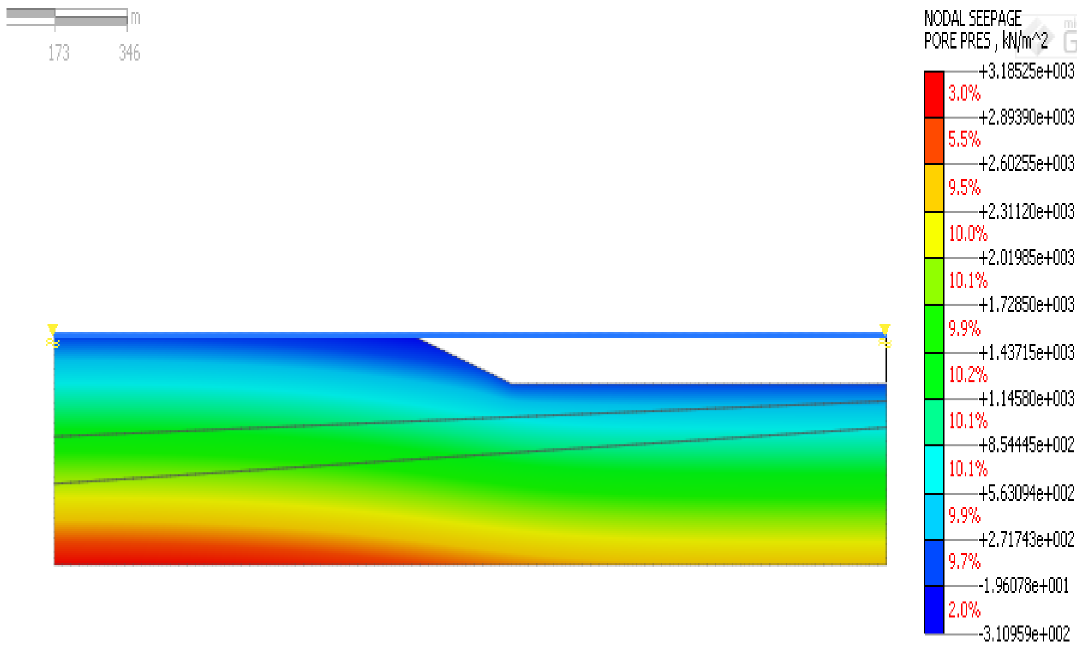

Fig. 4.7 Pore pressure after 3000 days for $K_O = 1.5$

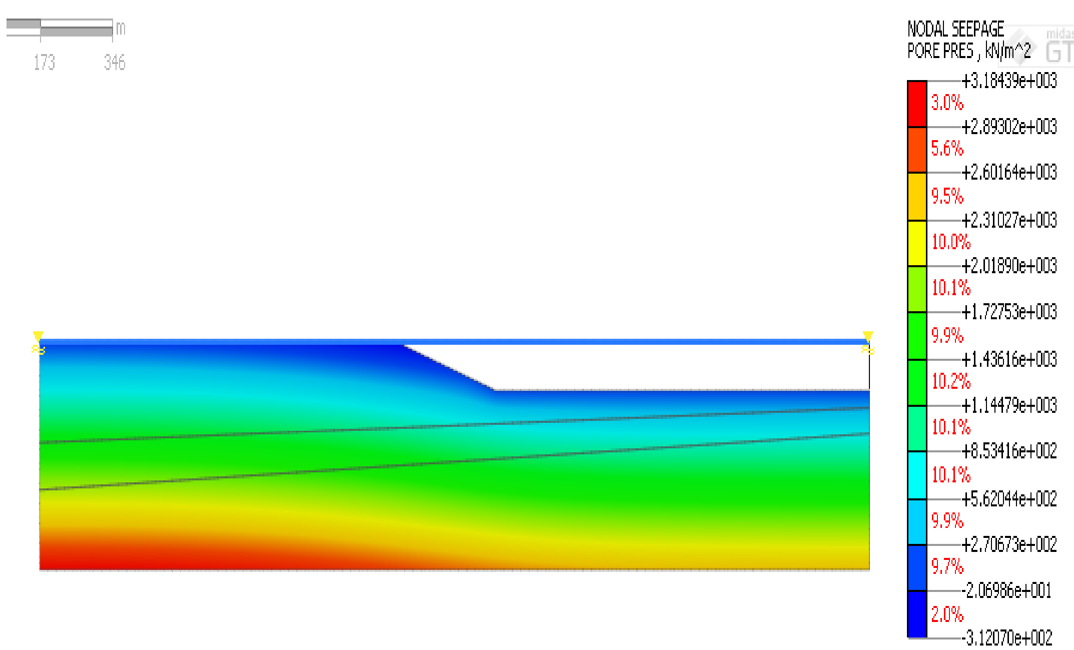

Fig. 4.8 Pore pressures after 3000 days for $K_O = 2$

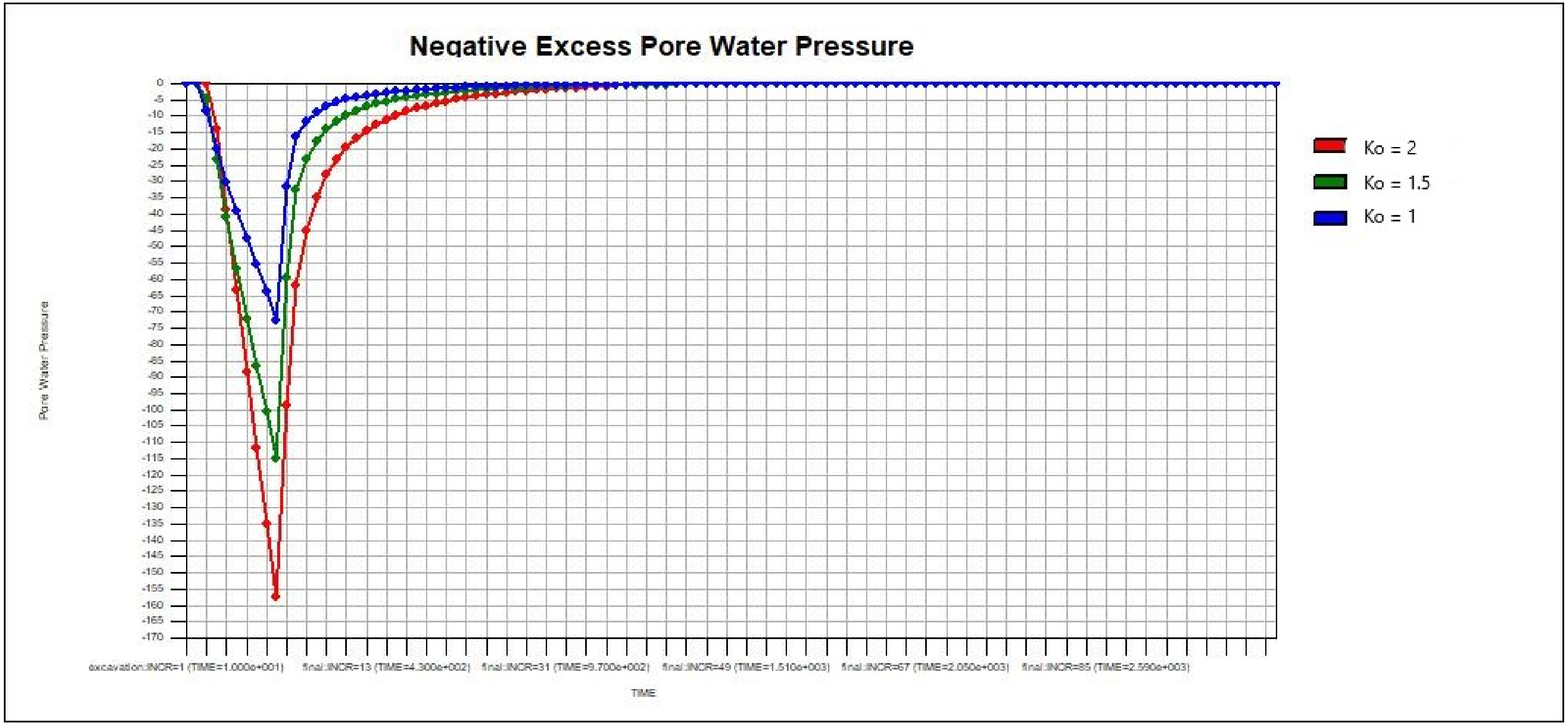

Fig 4.9 Overall trend of excess pore water generation and dissipation with time

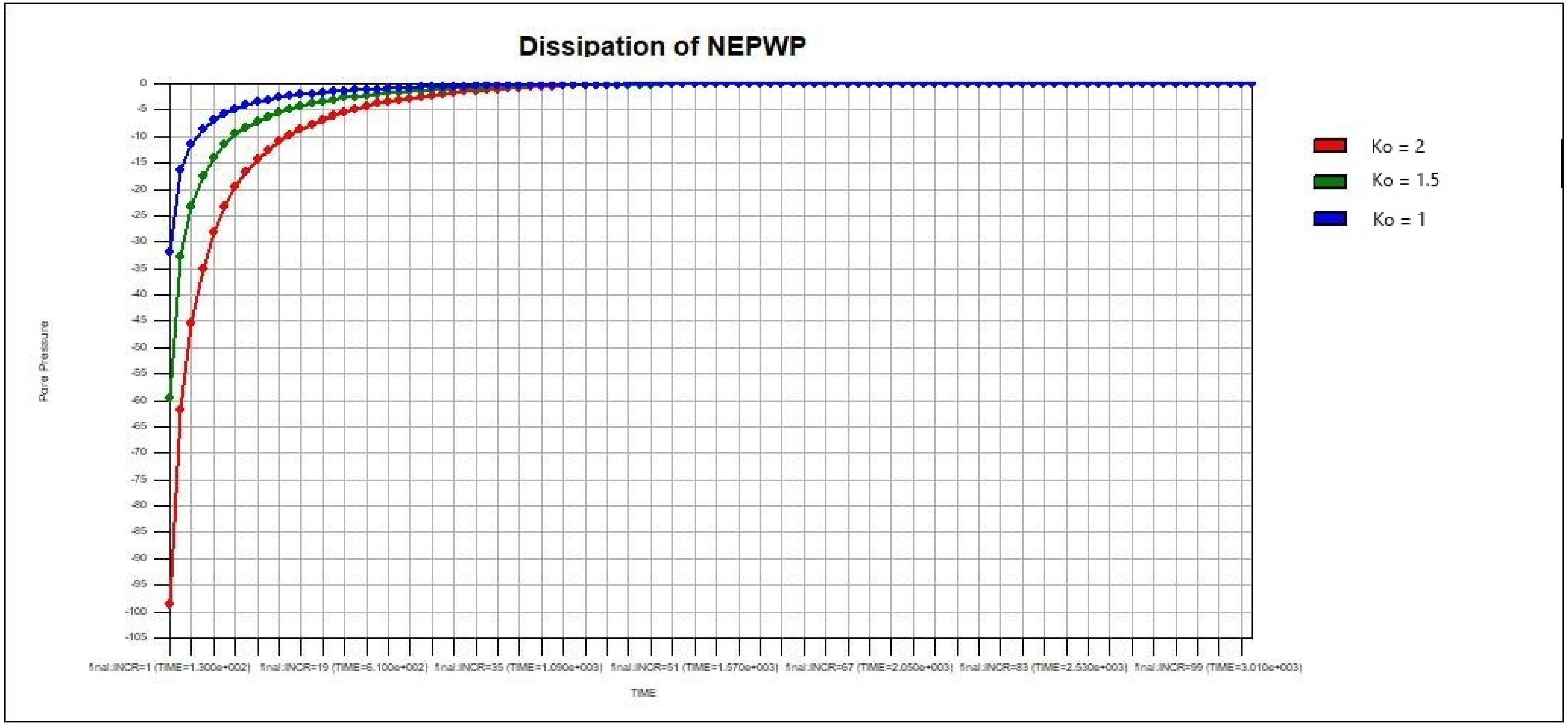

Fig. 4.10 Pore water pressures dissipation trend after the excavation

4.4 FACTOR OF SAFETY

Factor of safety in MIDAS GTS NX was calculated through in built strength reduction method which was opted for while defining the construction stage analysis. Fig. 4.11 shows how the factor of safety was varying with time of dissipation of negative excess pore water pressure.

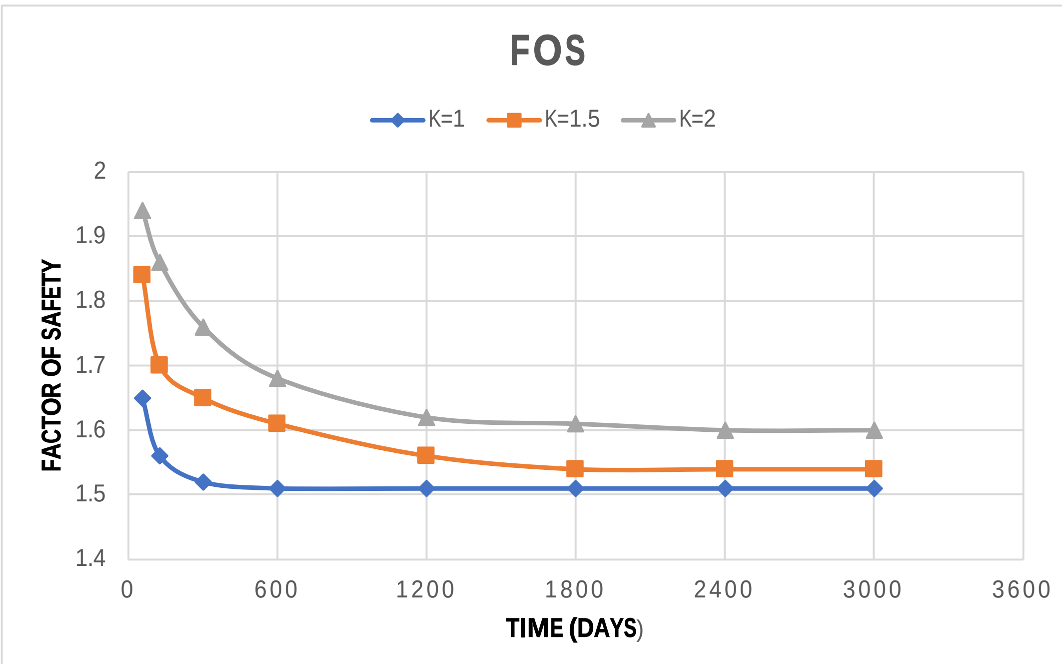

Fig 4.11 Factor of safety at different time values

	$K_O = 1$	$K_O = 1.5$	$K_O = 2$
Total Reduction (%)	8.48%	16.3%	21.25%

Table 4.1 Reduction in FOS over time

4.5 DISCUSSION

It can be seen from the distribution of the generated pore water tension is not even and it can be majorly attributed to the importance of stress removal levels. Also, the distribution of this excess pressure is generally confined in the zones below and adjacent to the excavation pit.

Also, it is noted that with higher values of K_O, higher values of pore water tension were reported due to greater amount of stress removals which are governed by the values of K_O. Hence, this value can be even higher in the case of deep excavations.

From Fig. 4.10 it can be seen that the maximum dissipation of the generated pore water tension has happened till the INCR of 20 to 30 i.e. by 600-900 days which constitutes 20-30% of the total time taken for the analysis i.e. 3000 days.

Now, from our present analysis, it can be seen that the values of K_O play a significant role in the stability of the excavated slopes in a saturated soil having low permeability or given less time for drainage. From the Fig. 4.11 it can be seen that the major reduction in the factor of safety of the excavated slope for all three values of K_O happens by 600-900 days. In this span of time, the pore water tension also dissipated at the maximum rate and simultaneously the effective stress also decreased.

Table 4.1 shows us the amount of reduction in the factor of safety of the slope over a span of 3000 days. From that it can be seen that for greater K_O values, the percentage reduction in FOS was also the highest. It is because higher K_O generates higher in situ stresses and upon removal of soil in excavation, higher amount of stresses get removed, which eventually lead to generation of more depressed pore water pressures and consequently leading to higher values of FOS in the short term. Now, after dissipation of this depressed pore water pressure, the case with high value of stress removal shows higher reduction the values of factor of safety.

CHAPTER 5

CONCLUSION

1) A soil when excavated leads to increase in the shear strength due to the increase in the height of the slope but leads decrease in the mean stress. Also, when an excavation is done in saturated soil in undrained condition, it leads to the formation of depressed pore water pressures called negative excess pore water pressures and these depressed pore water pressures are dependent on the values ofK_O. With higher values of K_O, greater are the stress removals upon unloading and hence greater is the generation of pore water tensions.

2) The phenomenon of formation of pore water tensions and their dissipation with time plays a significant role in the stability of an excavated cut over a period of time. The results of our study shows us that the dissipation of pore water tension if very fast in the initial phases of dissipation while it tends to decrease with time after some time. Getting to the final equilibrate values of steady state seepage may take a long time depending upon the type of drainage of the in situ soil and its geometry. About 20-30% is time by which the initial phase of fast dissipation of pore water tension is over.

3) Just after the end of excavation, the values of factor of safety is a higher in comparison to that obtained after a long period of time or when steady state condition has reached. Also. It can be concluded from our study that the amount of reduction in the values of FOS is a function of K_O. Hence, it can be said that the stability of excavated slope or cuts or walls depends upon factors such as the amount of stress removal, drainage conditions in the soil, rate of equilibration of depresses pore water pressure.

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