Effect of Reinforcement In Slope Stabilization of Road Embankment of Dhaka-Feni Express Way

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PROJECT REPORT APPROVAL

The thesis titled ― Effect of Reinforcement in Slope Stabilization of Road Embankment of Dhaka-Feni Express Way submitted by Md. Abdul Jabbar, Nabil Mahmood and Md. Tariqul Islam Rabbi. St. No. 145410, 145442, 145454 respectively, have been found as satisfactory and accepted as partial fulfillment 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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We also place on record, our sense of gratitude to one and all, who directly or indirectly, have contributed to this venture.

ABSTRACT

Slope stability is the potential of soil covered slopes to withstand and undergo movement. Stability can be determined by the balancing of shear stress and shear strength. A stable slope can be initially affected by preparatory factors, making the slope conditionally unstable. The factors which can trigger a slope failure can be climatic events which can then make the slope unstable, leading to mass movements. Mass movements can be caused by increase in shear stress, such as loading, lateral pressure, and transient forces. This thesis mainly focuses on the effect on reinforcement of a highway road embankment section in order to stabilize the slope. We are mainly considering the impact of water level fluctuation on the stability of slope due to which soil water coupling occurs and also considering the variation in dimension of slopes. In these analyses we have done our numerical analyses with the Finite Element Method computer program FEM-tij 2D and a comparison of the analyzed results. The finite element method needs additional information regarding the potential performance of a slope but just basic parameter information is needed when we using traditional methods. A 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.

In this study, advanced approaches are used for soil water coupling in FEM-tij 2D modeling of slope stability which is being evaluated. Conditions for failure in the soil slope is analyzed and necessary steps are taken in order to prevent it. In our case we have used soil nailing to analyze conditions considering the soil slope with and without nailing including both drained and undrained condition.

TABLE OF CONTENTS

Contents

Chapter 1
Introduction

1.1 General

Slope stability is the process of calculating and assessing how much stress a particular slope can manage before failing. If slopes become unstable it causes a vital problem which is the safety concern of the public and also the properties. Considerable amount of research has been done regarding the slope instability problem about how to counter and resolve this issue. These major researches have led to indispensable advancements in understanding the multiplex behavior of soil.

Over the last few years extensive study has been done to understand the different conditions of the slope and analyze its failure criteria. In order to avoid the failure of slope, slope stabilization must be done. Considering the importance of slope stability to their work, it's beneficial for civil engineers to understand how to properly evaluate slope stability and leverage various techniques to achieve slope stabilization.

This thesis provides stabilization of slope through the usage of soil nailing. Failure criteria is analyzed considering conditions with and without soil nailing, also keeping in considerations of the different water levels and soil water interaction for Dhaka-Feni highway at a particular embankment. In this research we have used an elastoplastic constitutive model for soils, called the extended subloading tij-model (Nakai et al., 2011) used in finite element analysis.

1.2 Background

Bangladesh consists of about 21,900 km of road and about 65% of these roads incorporates rural roads. The remainder of these roads is composed of both National and Regional roads in about equal proportions. The construction of this infrastructure is made by local materials which consists alluvial sands and silts and sometimes contain varying amount of clay, mica and organic material. Sizeable sections of the network are subjected to flooding for about three months annually on average. Any soil volume is continuously affected by hydrological conditions prevailing; present water is either influencing or completely governing the actual soil properties. At the scale of bank slopes and embankments, the road embankments are influenced by external water loads, development of pore pressures, and hydrodynamic impact from internal and external water flow. This internal and external water flow causing soil-water coupling. Due to soil water coupling slope stability greatly affected and in many cases slope failure may occur.

Dhaka-Feni Highway is one of the most prominent road networks of Bangladesh if we consider from an economic perspective. This highway plays as a vital transportation artery as it facilitates the movement of vehicles from Dhaka-Chittagong by linking the two cities being apart from each other by a distance of 250 km. Therefore, it is extremely necessary that we make sure about the safety of the road by preventing the failure of the embankment.

1.3 Objective of the study

The aim of this study is to identify

1. To determine the optimum tensile strength of the reinforcement in road embankment considering the allowable factor of safety and displacement.
2. To determine the length of reinforcement considering the stability of the model road embankment.
3. Analyses of stability based on various slope.
4. To introduce massive use of soil reinforcement in Bangladesh.
5. To reduce mass land use.

1.4 Scope of the study

The scope of the study may be:

1. Analyze slope stability of road embankment with different slopes.
2. Determination of optimum water level to which the slope remains stable.
3. Finding economic solutions for slope stabilization.

Chapter 2
Literature Review

2.1 Ground Investigations

Before any further examination of an existing slope, or the ground onto which aslope is to be built, essential borehole information must be obtained. This information will give details of the strata, moisture content and the standing water level. Also, the presence of any particular plastic layer along which shear could more easily take place will be noted.

For the study in this thesis, we used the field data from the soil test report of the Dhaka-Chittagong highway performed at IUT Geotechnical laboratory in the previous year.

2.2 Geotechnical Parameters

Before a geotechnical analysis can be performed, the parameters values needed in the analysis must be determined.

2.2.1 Unit Weight

Unit weight of a soil mass is the ratio of the total weight of the soil to the total volume of the soil. Unit weight, γ, is usually determined in the laboratory by measuring the weight and volume of a relatively undisturbed soil sample obtained from the field. Measuring unit weight of soil directly in the field might be done by sand cone test, rubber balloon or nuclear densitometer. We will use unit weights presented in a report by IUT Geotechnical laboratory.

2.2.2 Cohesion

Cohesion, c, is usually determined in the laboratory from the Direct Shear Test. Unconfined Compressive Strength Suc can be determined in the laboratory 8 using the Triaxial Test or the Unconfined Compressive Strength Test. There are also correlations for Suc with shear strength as estimated from the field using Vane Shear Tests. Our soil test report has already determined the cohesions for this project.

2.2.3 Friction Angle

The angle of internal friction, φ, can be determined in the laboratory by the Direct Shear Test or by Triaxial test. For our analysis we will use values determined by IUT Geotechnical laboratory soil test report.

2.2.4 Young's Modulus of Soil

Young 's soil modulus, E_s, may be estimated from empirical correlations, laboratory test results on undisturbed specimens and results of field tests. Laboratory test that might be used to estimate the soil modulus is the Triaxial test. For our analysis we will use values determined by IUT Geotechnical laboratory soil test report.

2.3 Type of soil

Geotechnical engineers classify soils, or more properly earth materials, for their properties relative to foundation support or use as building material. These systems are designed to predict some of the engineering properties and behavior of a soil based on a few simple laboratory or field tests.

2.3.1 Sand

Soil material that contains 85% or more sand; the percentage of silt plus 1.5 times the percentage of clay does not exceed 15 (CSSC; USDA).

2.3.2 Clay

Soil material that contains 40% or more clay and 40% or more silt (CSSC; USDA).

2.3.3 Silty clay

Soil material that contains 40% or more clay and 35% or more silt (CSSC; USDA).

2.3.4 Silt

Soil material that contains 80% or more silt and less than 12% clay (CSSC; USDA).

2.3.5 Sandy clay

Soil material that contains 7 to 27% clay, 28 to 50% silt, and less than 52% sand (CSSC; USDA).

2.4 Basic Requirement for Slope Stability Analysis

Whether slope stability analyses are performed for drained conditions or undrained conditions, the most basic requirement is that equilibrium must be satisfied in terms of total stresses. All body forces (weights), and all external loads, including those due to water pressures acting on external boundaries, must be included in the analysis. These analyses provide two useful results: (1) the total normal stress on the shear surface and (2) the shear stress required for equilibrium.

The factor of safety for the shear surface is the ratio of the shear strength of the soil divided by the shear stress required for equilibrium. The normal stresses along the slip surface are needed to evaluate the shear strength: except for soils with $\varphi = 0$, the shear strength depends on the normal stress on the potential plane of failure.

In effective stress analyses, the pore pressures along the shear surface are subtracted from the total stresses to determine effective normal stresses, which are used to evaluate shear strengths. Therefore, to perform effective stress analyses, it is necessary to know (or to estimate) the pore pressures at every point along the shear surface. These pore pressures can be evaluated with relatively good accuracy for drained conditions, where their values are determined by hydrostatic or steady seepage boundary conditions. Pore pressures can seldom be evaluated accurately for undrained conditions, where their values are determined by the response of the soil to external loads.

In total stress analyses, pore pressures are not subtracted from the total stresses, because shear strengths are related to total stresses. Therefore, it is not necessary to evaluate and subtract pore pressures to perform total stress analyses. Total stress analyses are applicable only to undrained conditions. The basic premise of total stress analysis is this: the pore pressures due to undrained loading are determined by the behavior of the soil. For a given value of total stress on the potential failure plane, there is a unique value of pore pressure and therefore a unique value of effective stress. Thus, although it is true that shear strength is really controlled by effective stress, it is possible for the undrained condition to relate shear strength to total normal stress, because effective stress and total stress are uniquely related for the undrained condition. Clearly, this line of reasoning does not apply to drained conditions, where pore pressures are controlled by hydraulic boundary conditions rather than the response of the soil to external loads.

2.5 Analyses of Drained Conditions

Drained conditions are those where changes in load are slow enough, or where they have been in place long enough, so that all of the soils reach a state of equilibrium and no excess pore pressures are caused by the loads. In drained conditions pore pressures are controlled by hydraulic boundary conditions. The water within the soil may be static, or it may be seeping steadily, with no change in the seepage over time and no increase or decrease in the amount of water within the soil. If these conditions prevail in all the soils at a site, or if12the conditions at a site can reasonably be approximated by these conditions, a drained analysis is appropriate. A drained analysis is performed using:

• Total unit weights

• Effective stress shear strength parameters

• Pore pressures determined from hydrostatic water levels or steady seepage analyses.

2.6 Analyses of Undrained Conditions

Undrained conditions are those where changes in loads occur more rapidly than water can flow in or out of the soil. The pore pressures are controlled by the behavior of the soil in response to changes in external loads. If these conditions prevail in the soils at a site, or if the conditions at a site can reasonably be approximated by these conditions, an undrained analysis is appropriate. An undrained analysis is performed using.

- Total unit weights
- Total stress shear strength parameters

2.7 Short-Term Analyses

Short term refers to conditions during or following construction—the time immediately following the change in load. For example, if constructing a sand embankment on a clay foundation takes two months, the short-term condition for the embankment would be the end of construction, or two months. Within this period of time, it would be a reasonable approximation that no drainage would occur in the clay foundation, whereas the sand embankment would be fully drained.

2.8 Long-Term Analyses

After a period of time, the clay foundation would reach a drained condition, and the analysis for this condition would be performed as discussed earlier. Analyses of Drained Conditions, because long term and drained conditions carry exactly the same meaning. Both of these terms refer to the condition where drainage equilibrium has been reached and there are no excess pore pressures due to external loads.

2.9 Pore Water Pressures

For effective stress analyses the basis for pore water pressures should be described. If pore water pressures are based on measurements of groundwater levels in bore holes or with piezometers, the measured data should be described and summarized in appropriate figures or tables. If seepage analyses are performed to compute the pore water pressures, the method of analysis, including computer software, which was used, should be described. Also, for such analyses the soil properties and boundary conditions as well as any assumptions used in the analyses should be described. Soil properties should include the hydraulic conductivities. Appropriate flow nets or contours of pore water pressure, total head, or pressure head should be presented to summarize the results of the analyses.

2.10 Soil Property Evaluation

The basis for the soil properties used in a stability evaluation should be described and appropriate laboratory test data should be presented. If properties are estimated based on experience, or using correlations with other soil properties or from data from similar sites, this should be explained. Results of laboratory tests should be summarized to include index properties, water content, and unit weights. For compacted soils, suitable summaries of compaction moisture–density data are useful. A summary of shear strength properties is particularly important and should include both the original data and the shear strength envelopes used for analyses (Mohr–Coulomb diagrams, modified Mohr–Coulomb diagrams). The principal laboratory data that are used in slope stability analyses are the unit weights and shear strength envelopes. If many more extensive laboratory data are available, the information can be presented separately from the stability analyses in other sections, chapters, or separate reports. Only the summaries of shear strength and unit weight information need to be presented with the stability evaluation in such cases.

2.11 Circular Slip Surface

Inherent in limit equilibrium stability analyses is the requirement to analyze many trials slip surfaces and find the slip surface that gives the lowest factor of safety. Included in this trial approach is the form of the slip surface; that is, whether it is circular, piece-wise linear or some combination of curved and linear segments. Slope/W has a variety of options for specifying trial slip surfaces. The position of the critical slip surface is affected by the soil strength properties. The position of the critical slip surface for a purely frictional soil (c = 0) is radically different than for a soil assigned untrained strength (φ = 0). This complicates the situation, because it means that in order to find the position of the critical slip surface, it is necessary to accurately define the soil properties in terms of effective strength parameters.

2.12 Factor of Safety

In slope stability, and in fact generally in the area of geotechnical engineering, the factor which is very often in doubt is the shear strength of the soil. The loading is known more accurately because usually it merely consists of the self-weight of the slope. The FoS is therefore chosen as a ratio of the available shear strength to that required to keep the slope stable. For highly unlikely loading conditions, accepted factors of safety can be as low as

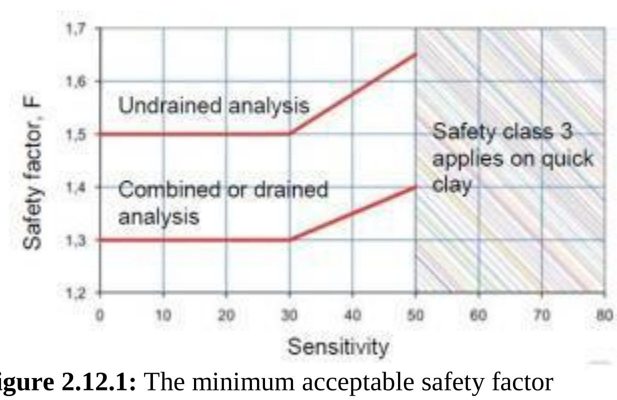

Figure 2.12.1: The minimum acceptable safety factor

1.2-1.25, even for dams e.g. situations based on seismic effects, or where there is rapid drawdown of the water level in a reservoir. According to TK Geo 11(Swedish Transport Administration requirements and guidelines) allowable limit for factor of safety is 1.5 for undrained analysis and 1.3 for combined or drained analysis.

2.13 Traffic load

Traffic load refers to the action of the traffic on the carriageway or railway structure. Action distribution shall be taken into consideration using an elastic theoretical based method. Where there are low permeable soils the traffic load is to be reduced for drained and combined analysis. Normally the traffic load can be ignored for combined analysis and drained analysis in the above conditions. Account must be taken of the vehicles and other equipment used in the execution phase.

Design using partial factors. The characteristic surface load for traffic shall be:

- 15 kN/m2 for design situations where the critical failure surfaces are short
- 10 kN/m2 for design situations where the critical failure surfaces are long
  Design using characteristic values. The characteristic surface load for traffic shall be:
- 20 kN/m2 for design situations where the critical failure surfaces are short
- 13 kN/m2 for design situations where the critical failure surfaces are long

2.14 Numerical analysis

Slope stability analyses can be performed using deterministic or probabilistic input parameters. FEMtij-2D can model isoperimetric soil types, complex stratigraphic and slip surface geometry, and variable pore-water pressure conditions using a large selection of soil models.

Chapter 3
Methodology

Many different solution techniques for slope stability analyses have been developed over the years. Analyze of slope stability is one of the oldest types of numerical analysis in geotechnical engineering. In this project we will use Finite Element Method for our analysis. FEMtij-2D a geotechnical finite element analysis software for 2D static analysis developed by Professor Dr. Hossain MD. Shahin at Nagoya Institute of Technology, Nagoya.

3.1 Finite Element Modeling

The finite element program FEMtij-2D was used for evaluating the stability of embankment slope. The road embankment cross-section utilized for the numerical model is presented in figure 3.1.1 and 3.1.2 that shows embankment without nailing and with nailing respectively.

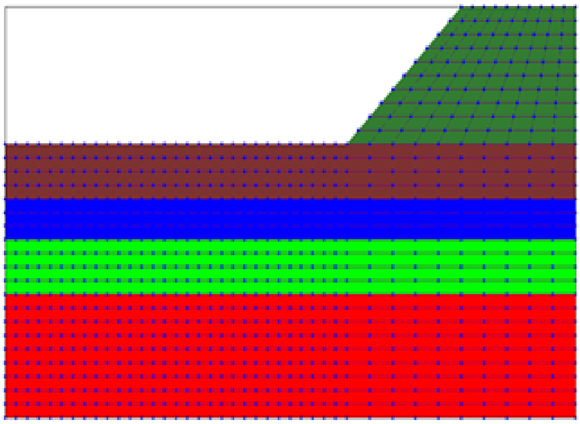

Figure 3.1.1: Left half of the embankment cross-section mesh (Without Nailing)

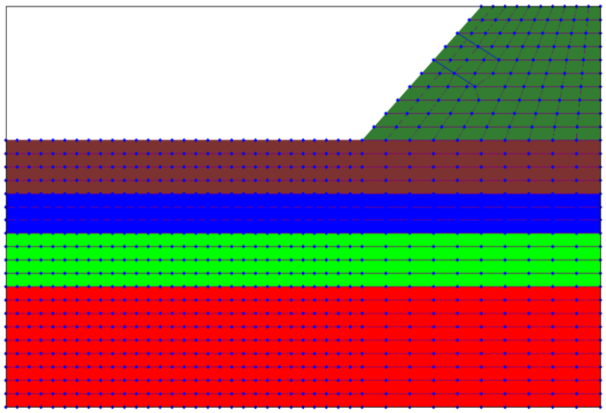

Figure 3.1.2: Left half of the embankment cross-section mesh (With Nailing)

3.2 Mesh Generation and Boundary Conditions

In this modeling, 4-node rectangular elements were used; see figure 3.1.1 & 3.1.2. The powerful 4-node element provides an accurate calculation of stresses and failure loads. The two vertical boundaries are free to move vertically only supported as roller support at the left and right side of the embankment as shown in figure 3.2.1, whereas the horizontal boundary at the bottom is considered to be pinned support.

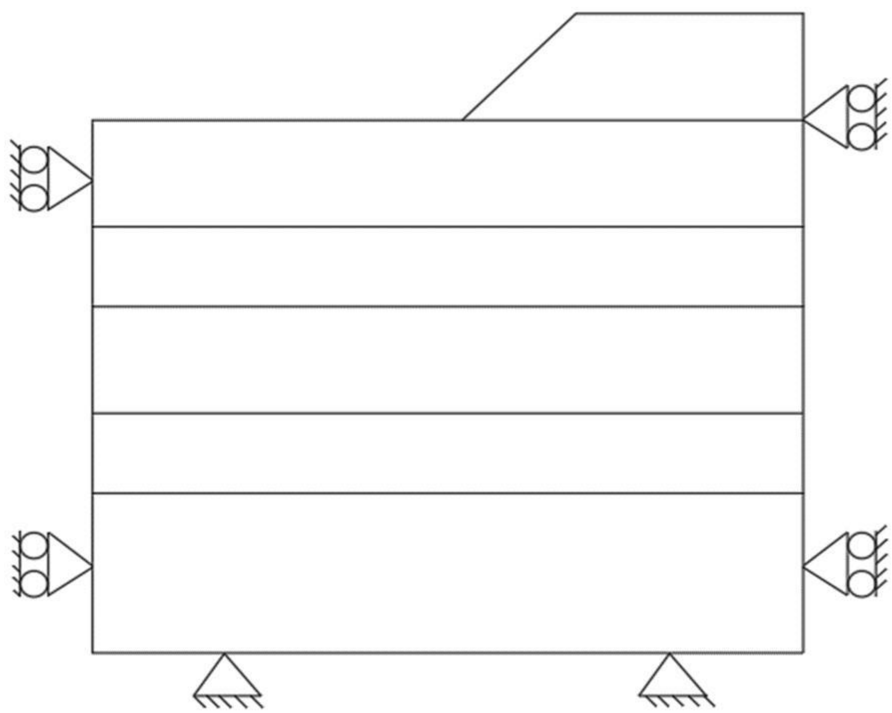

Figure 3.2.1: Boundary conditions used in analysis

3.3 Material Model

The Mohr–Coulomb model was used for this analysis. This model involves five parameters, namely Young 's modulus, E, Poisson 's ratio, ν, the cohesion, c, the friction angle, φ, and the dilatancy angle, ψ. In this case dilatancy angle was assumed to be zero, since it is close to zero for clay and for sands with a friction angle less than 38° (Lenita, T.). This work applied only for two-dimensional plain-strain problems. The Mohr–Coulomb constitutive model used to describe the soil (or rock) material properties. The Mohr–Coulomb criterion relates the shear strength of the material to the cohesion, normal stress and angle of internal friction of the material. The failure surface of the Mohr-Coulomb model can be presented as:

$$f = \frac{I_1}{3}\sin\phi + \sqrt{J_2}\left[\cos\sigma - \frac{1}{3}\sin\sigma\sin\phi\right] - C\ \cos\phi$$

where| ϕ is the angle of internal friction, C is cohesion

$$I_1 = (\sigma_1+\sigma_2+\sigma_3) = 3\sigma_m$$

$$J_2 = \left(\frac{1}{2}(S^2{}_x+S^2{}_y+S^2{}_y\right) + \tau^2{}_{xy}+\tau^2{}_{yz}+\tau^2{}_{zx})$$

$$\sigma = \frac{1}{3}\sin^{-1}\left[\frac{3\sqrt{3}J_3}{2J_2^{3/2}}\right]$$

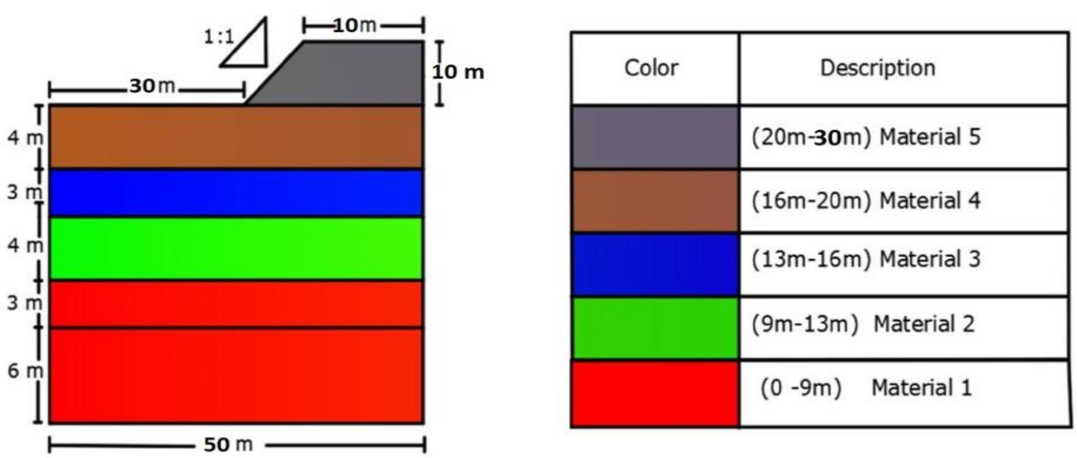

Figure 3.3.1: Materials used in different layers.

3.4 Drainage Boundary Conditions

The model used here has 2 types of drainage boundary conditions. The left, right and the bottom of the cross-section was modelled as undrained boundary and the top level where the water level fluctuates is modelled as drained boundary to represent the actual situations.

3.5 Simulated Models

We have used 4 different models for determining the suitable condition for embankment design. Different models have different slopes and water level conditions. Figure 3.5 shows the specifications for the simulated models for this analysis.

Case (Without Nailing)	Water Level	Slope(V:H)
Case1	No Water	1:1
Case2	Up to Surface	1:1
Case3	Up to half of road embankment	1:1
Case4	Bottom of road embankment	1:1

Case (With Nailing)	Water Level	Slope(V:H)
Case1	No Water	1:1
Case2	Up to Surface	1:1
Case3	Up to half of road embankment	1:1
Case4	Bottom of road embankment	1:1

Table: Specifications of different cases simulated

Chapter 4
Results and Discussion

4.1 Initial Ground Condition

The base of the embankment has different soil layers. The vertical stress increases as depth increases as the results show in figure 4.1.1. These figures show the initial condition of the ground when the embankment was not created and the water level was up to the third layer of soil from the bottom of the embankment. Also, the figure 4.1.2 shows the coefficient of earth pressure at rest k0 and void ratio plot for the initial ground condition.

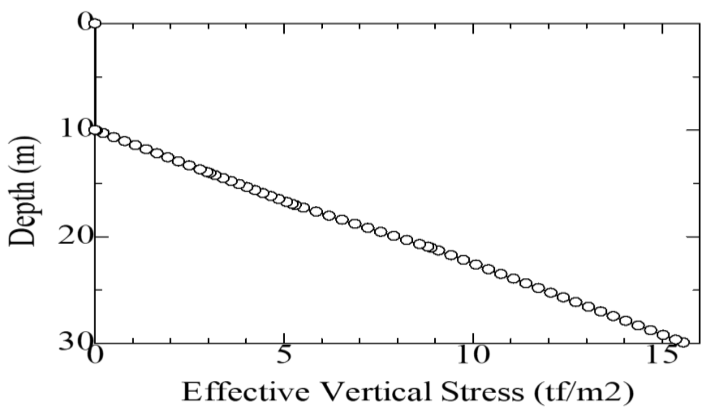

Figure 4.1.1: Vertical stress distribution

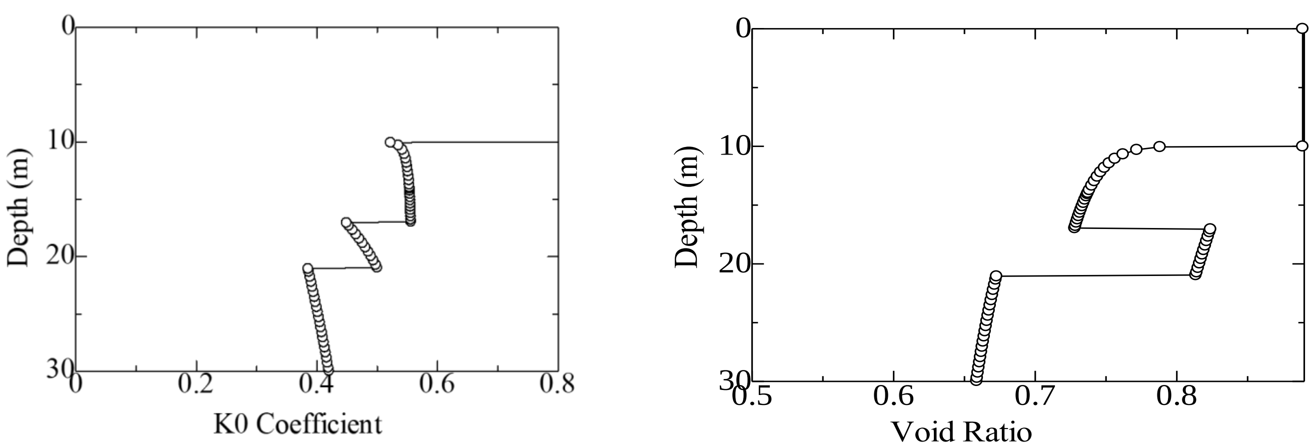

Figure 4.1.2: K0 coefficient and Void ratio plot for initial ground

4.2 Analysis of Embankment

The analyzed embankment consists of 1:1 slope. This case represents half of the actual slope of the Dhaka-Feni expressway that was built. We haven't considered any water level rise in the making of this embankment rather than that we applied load in the cases those are discussed later with various water levels.

4.2.1 Vertical Stress (Embankment)

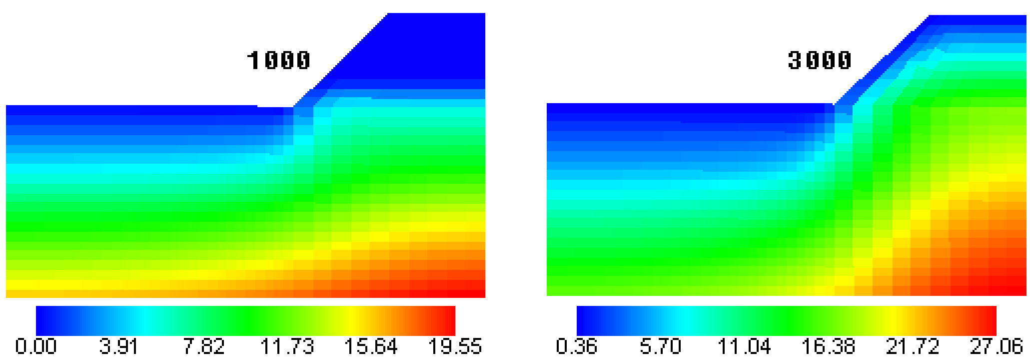

Figure 4.2.1: Distribution of vertical stress at different stages: unit (tf/m^2)

As the construction of the embankment proceeds the vertical stress distribution in the ground as well as the embankment also changes. Figure 4.2.1 shows the change of the vertical stress at two different stages. The first figure is the condition when one third of the embankment construction is complete and the second figure is for the completed embankment. Vertical stress increases as the embankment soil filling proceeds. Vertical stress is greater below the embankment. As we can see that the maximum vertical stress is 19.55 tn-f/m^2 when one third of the embankment is constructed and its maximum vertical stress is 27.06 tn-f/m^2 when the embankment is completed.

4.2.2 Shear Strain (Embankment)

Soil particles hold up itself with the shear strength between the soil particles. Shear strain diagram shows the most strained zones where the probable shear failure may occur and for the case of embankment the toe of the embankment is the most critical zone. Here in the figure 4.2.2 we can observe that the maximum shear strain is .07 when one third of the embankment is completed and it has a maximum shear strain of .11 when the full construction of the embankment is completed. It is important to notice that the maximum strain happened to be occur just a bit right of the toe of the embankment.

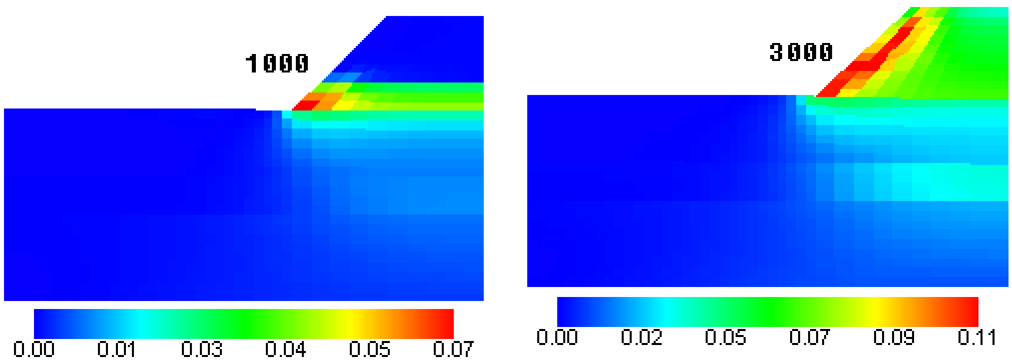

Figure 4.2.2: Distribution of shear strain

4.2.3 Displacement Vector (Embankment)

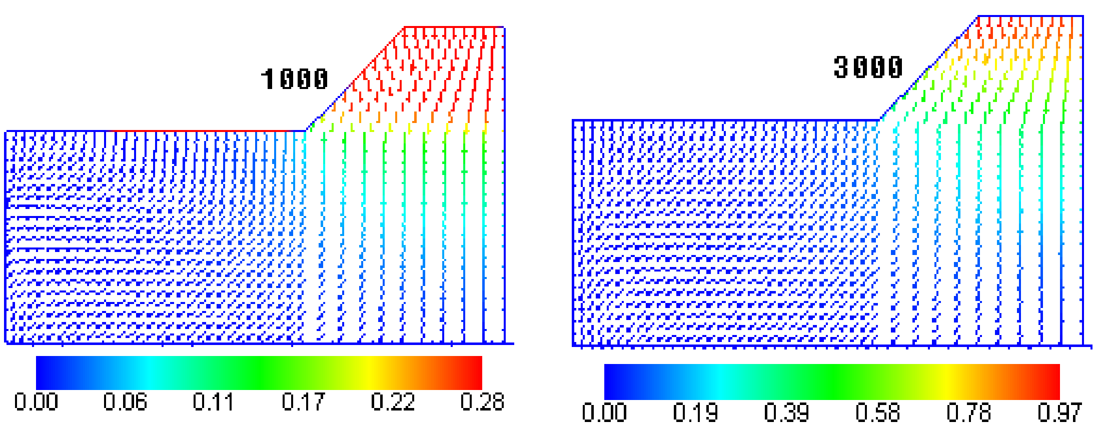

Figure 4.2.3: Distribution of vertical displacement vector (meter)

In figure 4.2.3 the displacement vector for case of embankment shows that the embankment is stable and the soil particles are settling downwards. Stabilization of the soil particles is also understood from the displacement vector. It has a maximum value of 27.06 cm in the embankment just at the top of the base level.

4.3 Analysis of Case No. 1 (Without Nailing, No Water Table)

Case no. 1 consists of 1:1 slope and the no water table considered for the analysis in the embankment. The main focus of this analysis is to find the situation of the soil under loading when there is no water in the embankment. The detailed results are discussed below.

4.3.1 Vertical Stress (Case 1)

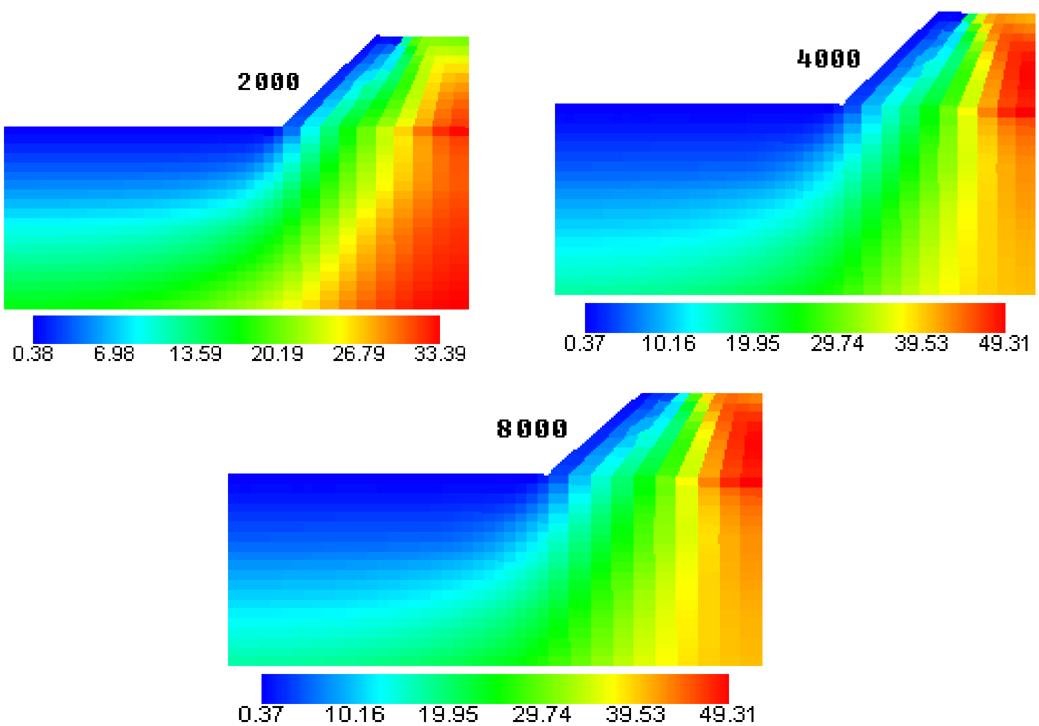

Figure 4.3.1: Distribution of vertical stress at different stages (tf/m^2)

As there is no water and loading are given on the embankment actually a significant amount of change in the vertical stress distribution noticeable. At 2000 steps of loading the bottom of the bas has the maximum vertical stress developed which is about 33.39 tf/m^2. The figure 4.3.1 shows the change of the vertical stress distribution of the cross-section during half and full stage of the construction of the embankment. At 8000 loading steps it shows the vertical stress is 49.32 tf/m^2 and this time the maximum value of stress is developed in the side of the embankment.

4.3.2 Shear Strain (Case 1)

In this case we can see that with the increase of loading the strain is also increasing. In case 2 the maximum value of strain is 0.07 that is achieved at 4000 loading steps and it stays the same till 8000 steps as it there is no water. Figure 4.3.2 shows these values

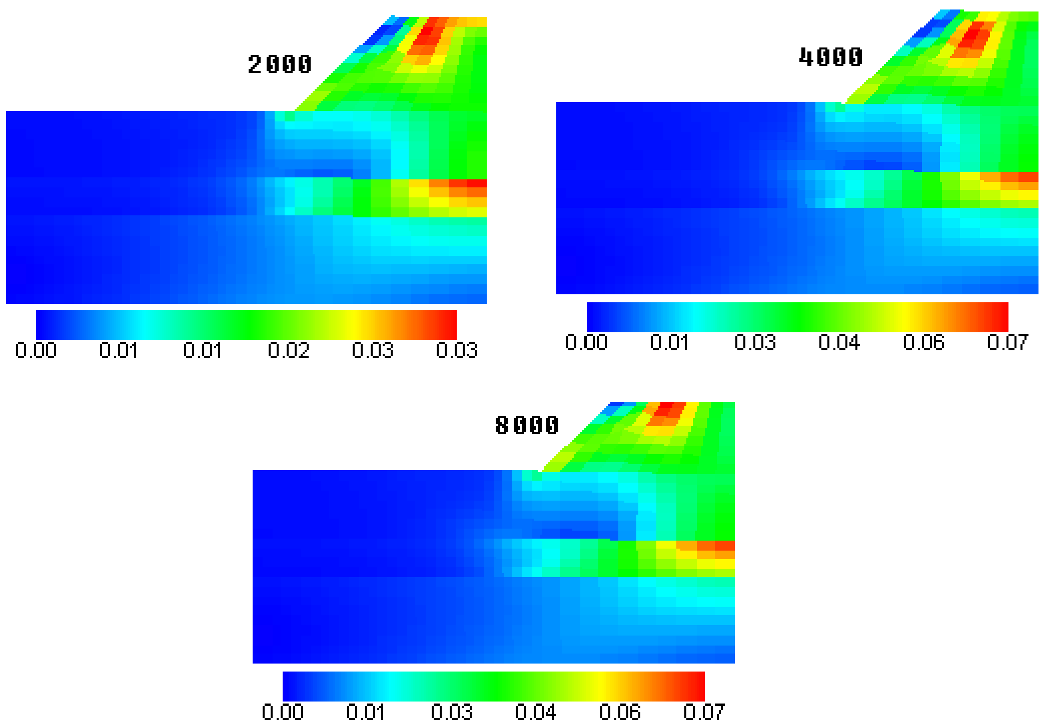

Figure 4.3.2: Distribution of shear strain (case 1)

4.3.3 Pore Water Pressure (Case 1)

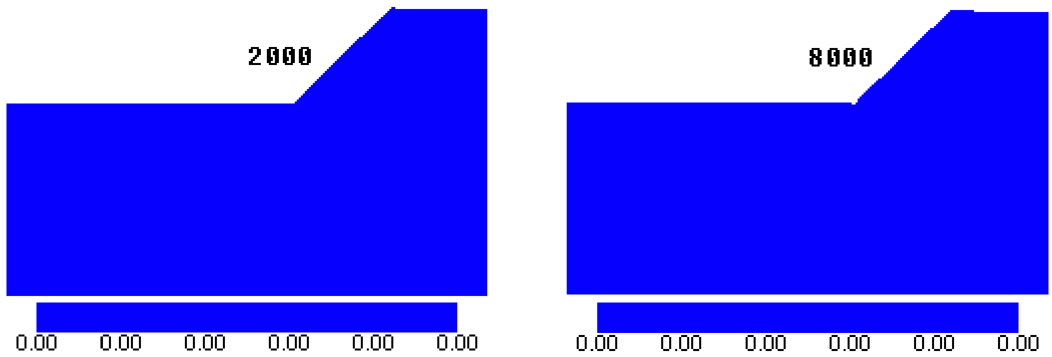

Figure 4.3.3: Distribution of pore water pressure with time (tf/m^2)

In this simulation as there is no water table so the pore water pressure is zero. Despite of the pore water pressure is zero, the figure 4.3.3 are shown for the research equity for every analysis.

4.3.4 Displacement Vector (Case1)

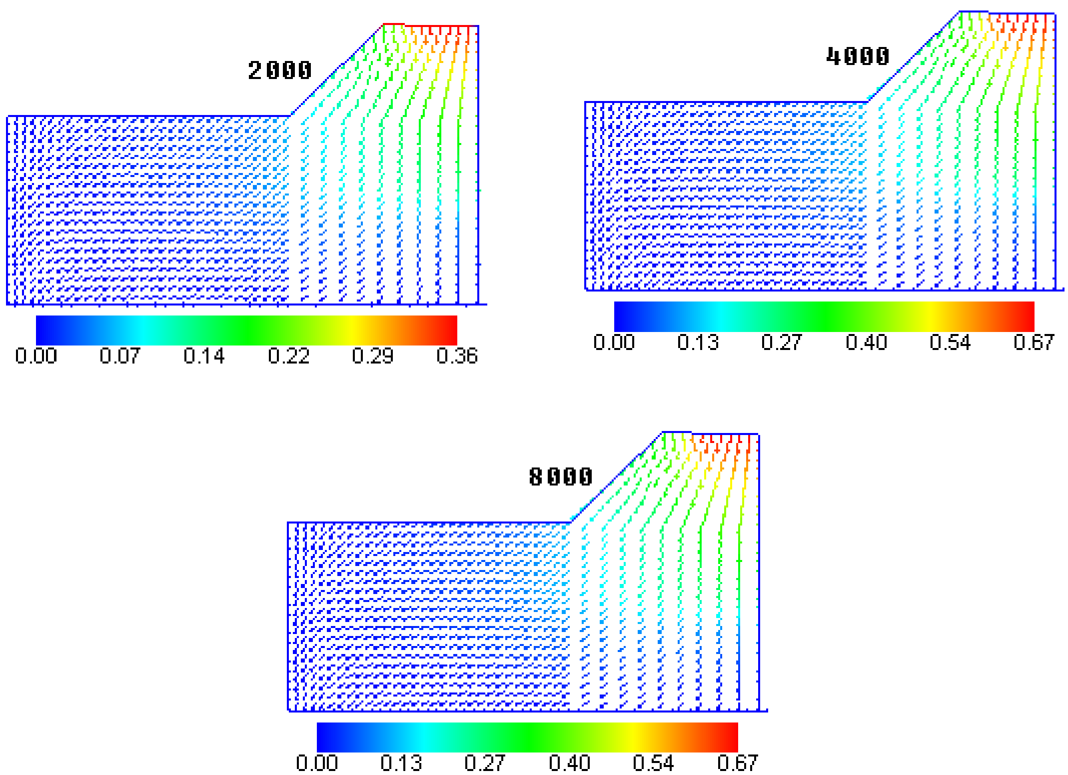

Figure 4.3.4: Displacement vector with time (meter)

Figure 4.3.4 presents displacement vector that shows the gradual increase in the displacement value of the slope when the load risen up to maximum value that at 8000 loading steps. The slope will tend to become unstable as the load is rising. In this case we can see that the value of displacement vector at 2000 loading step is 0.33 m, when it the value is 0.62 m at maximum 8000 loading steps.

4.3.5 Surface Settlement (Case1)

There is no surface settlement for consolidation purpose as there is no water involved in this case. And the extracted figure from software which relate the settlement with time give overlapping line of different times.

4.4 Analysis of Case 2 (Nailing, No Water)

4.4.1 Vertical Stress (Case 2)

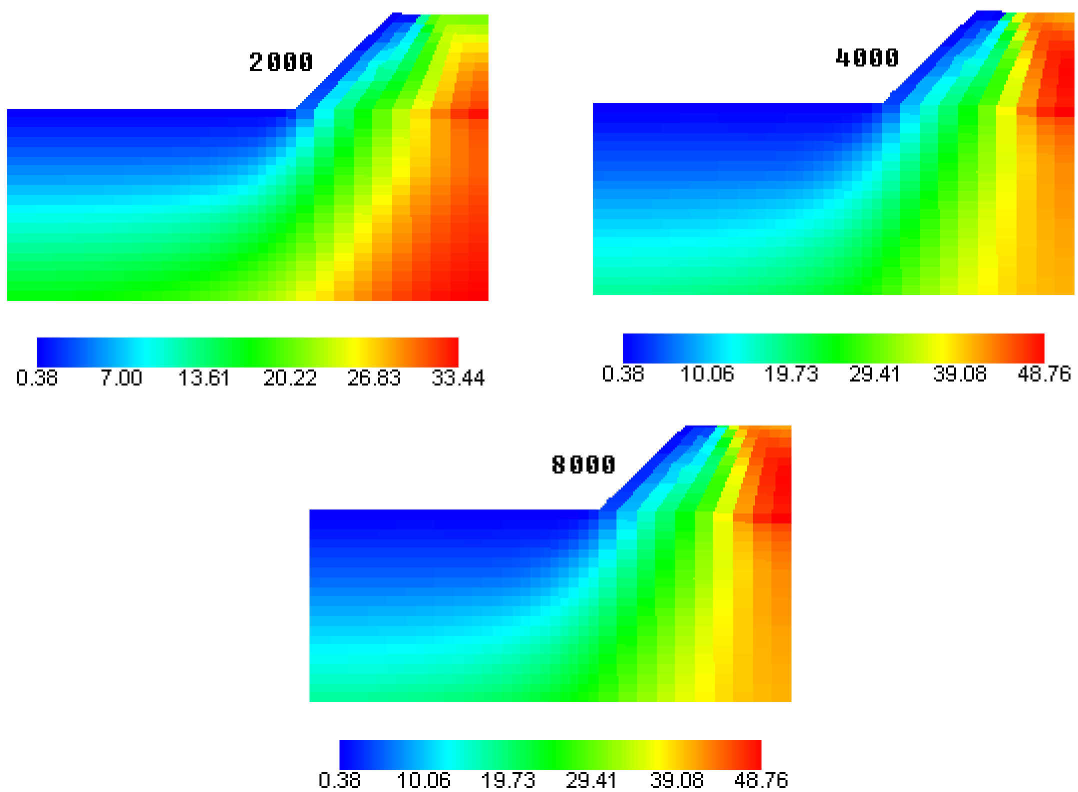

Figure 4.4.1: Distribution of vertical stress at different stages (tf/m^2)

In the figure 4.4.1 we can see here that the vertical stress distribution in the ground is a bit lower from the previous case that was without nailing. Figure 4.4.1 shows the changes of the vertical stress at three different stages. Vertical stress increases as the loading is increased, but in this case the soil stress is lower than the case without nails. Vertical stress is greater at lower side of the embankment. As we can see that the maximum vertical stress is 48.76 tf/m^2 at 4000 steps and 8000 steps.

4.4.2 Shear Strain (Case 2)

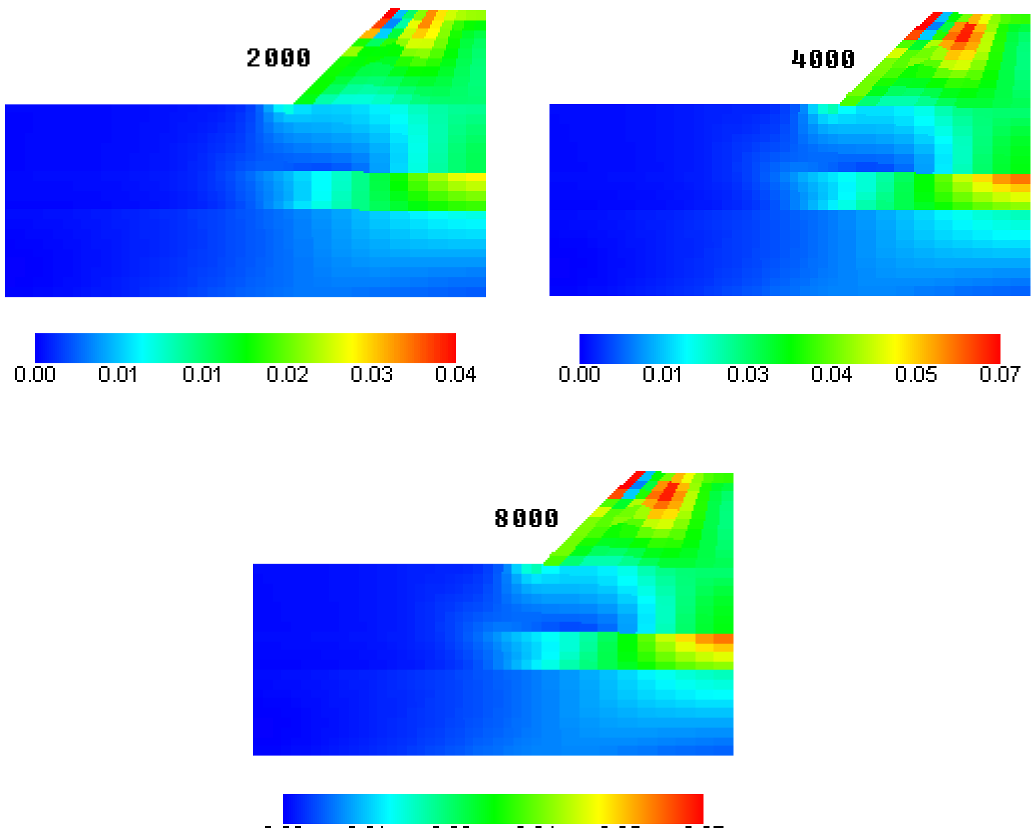

Figure 4.4.2: Distribution of shear strain (case 2)

Here we can see that with the increase of loading the strain is also increasing. In case 2 the maximum value of strain is 0.07 that is achieved at 4000 loading steps that is almost similar to the case 1 and it stays the same till 8000 steps as it there is no water.

4.4.3 Pore Water Pressure (Case 2)

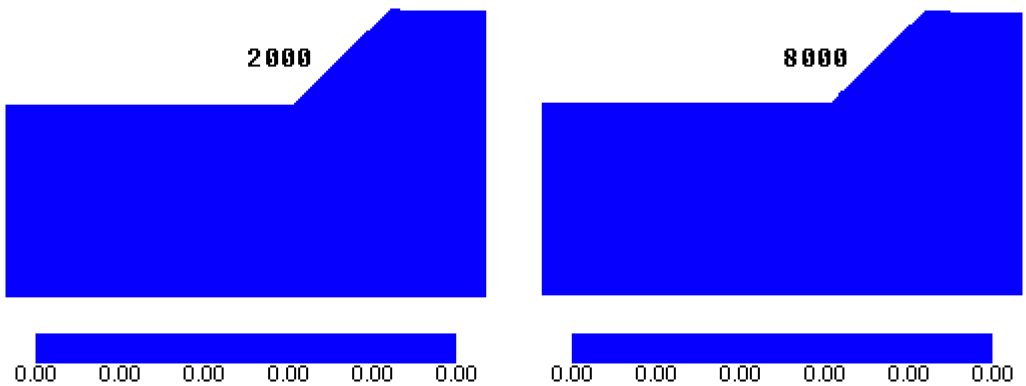

Figure 4.4.3: Distribution of pore water pressure with time (tf/m^2)

As the case 1 & 2 are the simulations where there is no water table so the pore water pressure is also zero here like previous case. Despite of the pore water pressure is zero, the figures are shown for the research equity for every analysis.

4.4.4 Displacement Vector (Case 2)

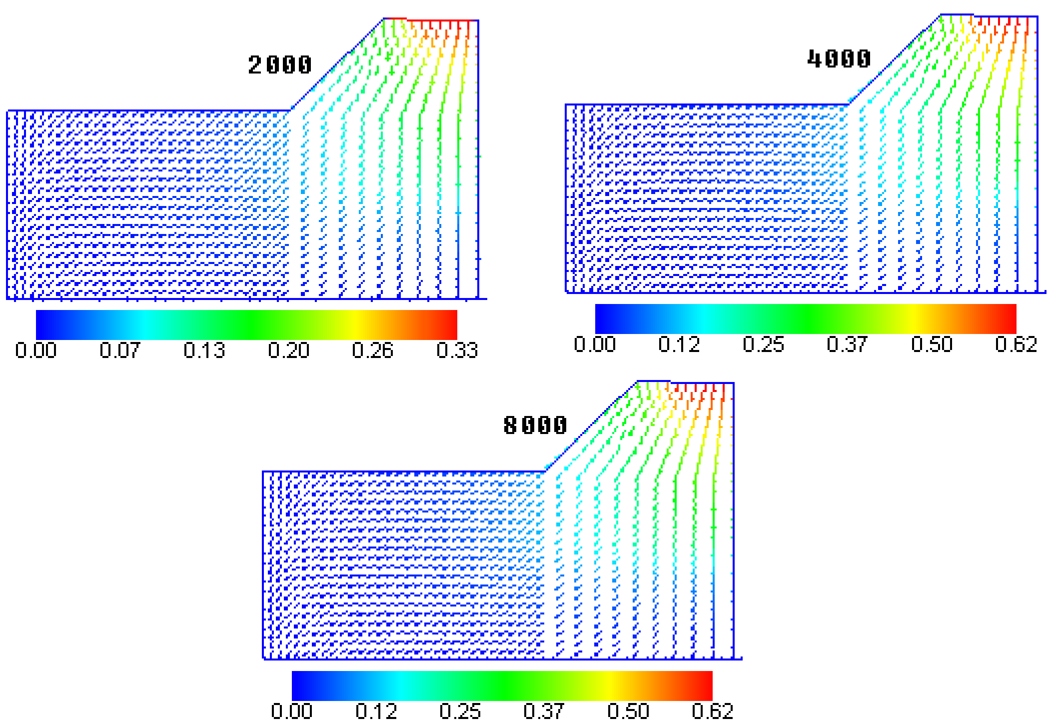

Figure 4.4.4: Displacement vector with time (meter)

Figure 4.3.4 presents displacement vector that shows the gradual increase in the displacement value of the slope when the load risen up to maximum value that at 8000 loading steps. The slope will tend to become unstable as the load is rising. In this case we can see that the value of displacement vector at 2000 loading step is 0.33 m, when it the value is 0.62 m at maximum 8000 loading steps.

4.5 Analysis of Case 3 (No Nailing, 0m Water)

Case 3 has been set up with no nailing and 0m the water level is kept at the surface level of the embankment to check if there is any change due to water level existence in the embankment. There is significant level of change noticed in this case that differs from the previous case that is case1. Detailed results discussed below.

4.5.1 Vertical Stress (Case 3)

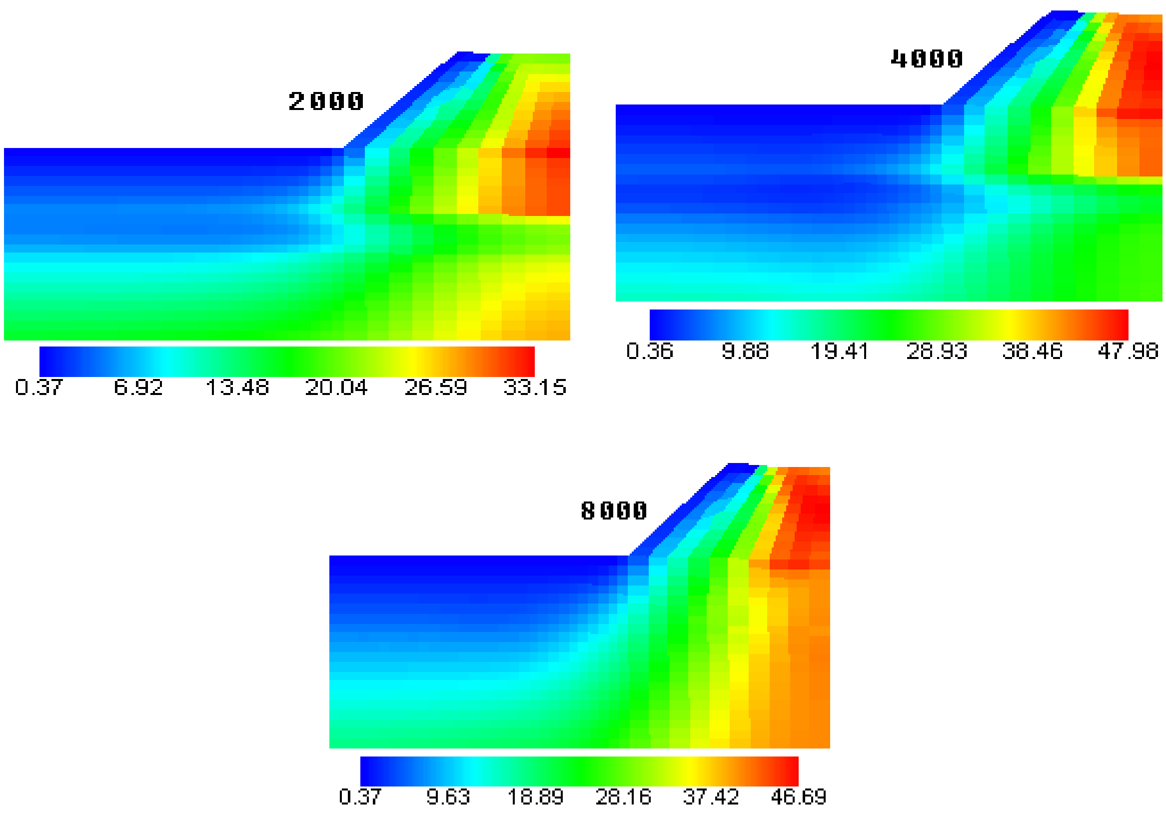

Figure 4.5.1: Distribution of vertical stress at different stages (tf/m^2**)**

Due to the presence of water in the slope of the embankment this time there is some amount of decrease in the vertical stress of the cross-section. Vertical stress has been decreased by around 5.5% compared to the case 1. Figure 4.5.1 shows the change of the vertical stress in the cross-section. Maximum value of vertical stress is at the bottom of the embankment and its value is 47.98 tf/m^2 which decreased with consolidation steps to 46.69 tf/m^2. Although the slope hasn't failed when water level is introduced but it has been confirmed that there will be decrease in the stress bearing capacity when water exists.

4.5.2 Shear Strain (Case 3)

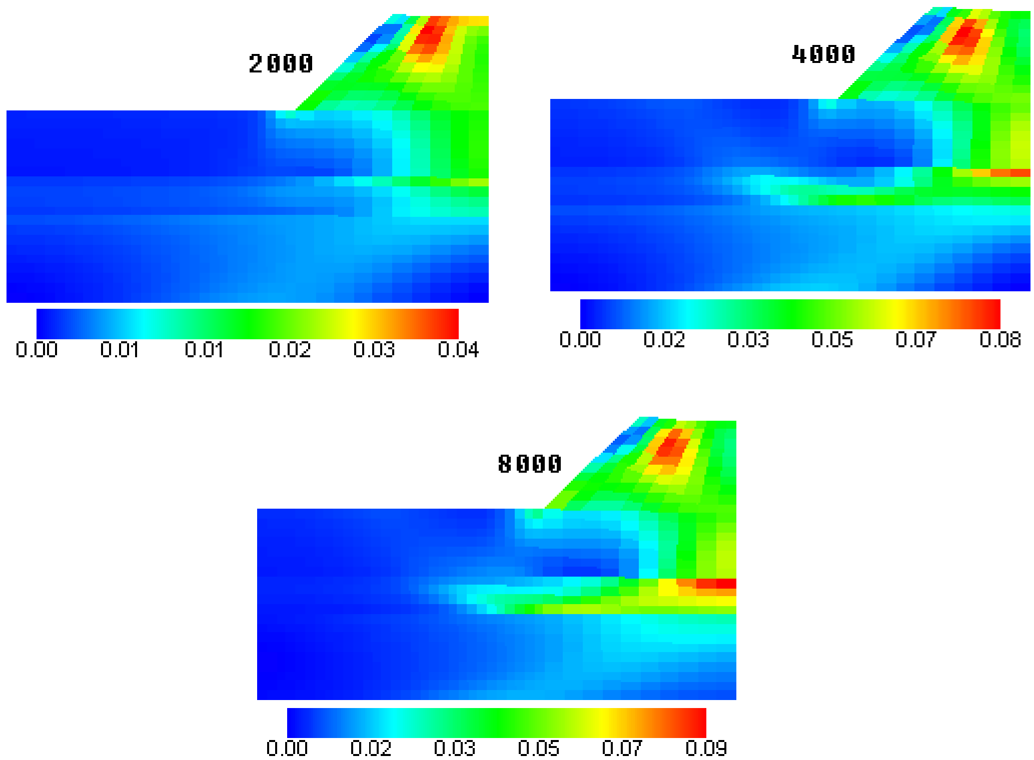

Figure 4.5.2: Distribution of shear strain (case 3)

In this case we have considered water table at the surface level and we can see that with the increase of loading the strain is also increasing. In case 3 the maximum value of strain is 0.08 that is achieved at 4000 loading steps and it increases to a value of 0.09 at 8000 steps as it there is consolidation due to presence of water. Figure 4.5.2 shows these values.

4.5.3 Pore Water Pressure (Case 3)

There was no water in the previous case (case 1). But there is significant pore water pressure development due to the introduction of water table in the embankment. It should be noticed that there is no excess pore water pressure present in the embankment and also the first layer of base. Figure 4.10 show the change of pore water pressure. Below the embankment there is the maximum pore water pressure developed within the base. The maximum pore water pressure is developed in 4000 loading steps that is 21.16 tf/m^2. And the pore water pressure is dissipated slowly with the further increase of loading steps and it becomes 0 at 8000 loading steps.

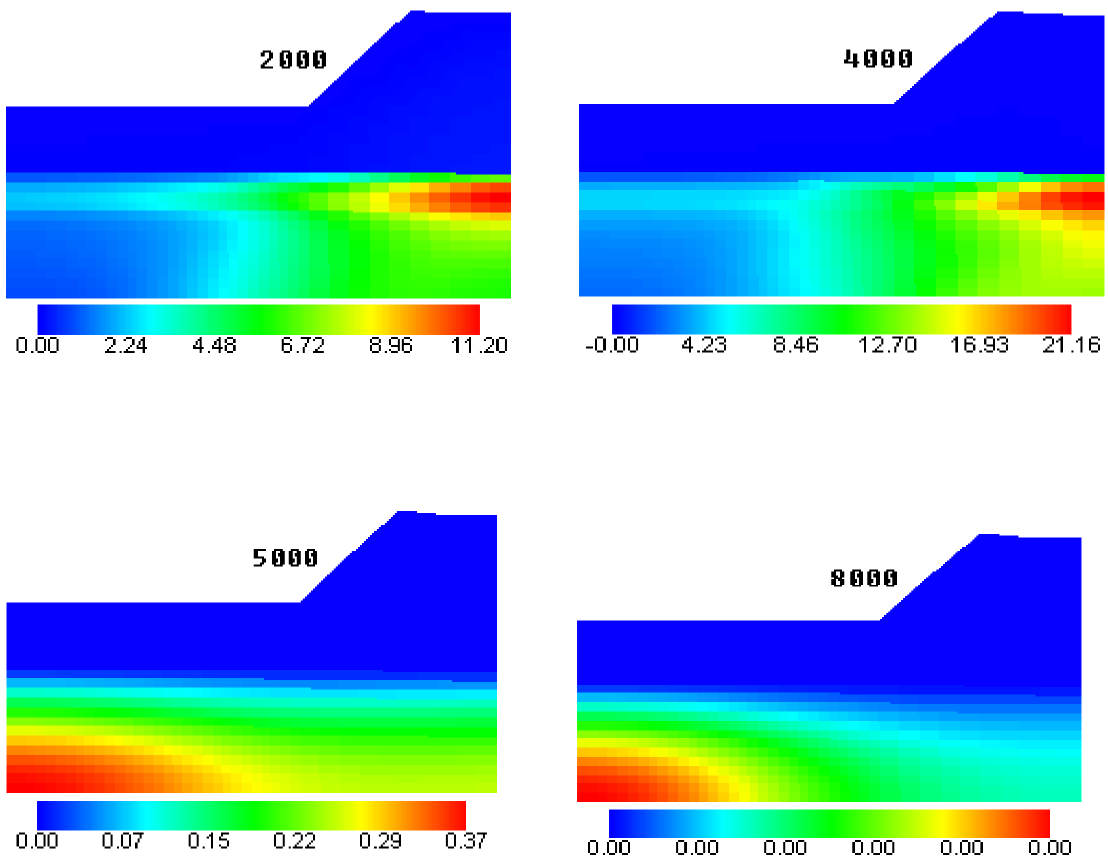

Figure 4.5.3: Distribution of pore water pressure with time (tf/m^2**)**.

4.5.4 Displacement Vector (Case 3)

Case 3 has a worse case of stabilization than case 1 as in this case there is presence of water in the embankment in this case. Displacement vector diagram shows that presence of water table in the slope increases the value of displacement vector as there is consolidation involved. The maximum displacement vector value is 0.79 m at complete loading and consolidation which was 0.67 in the case of without water.

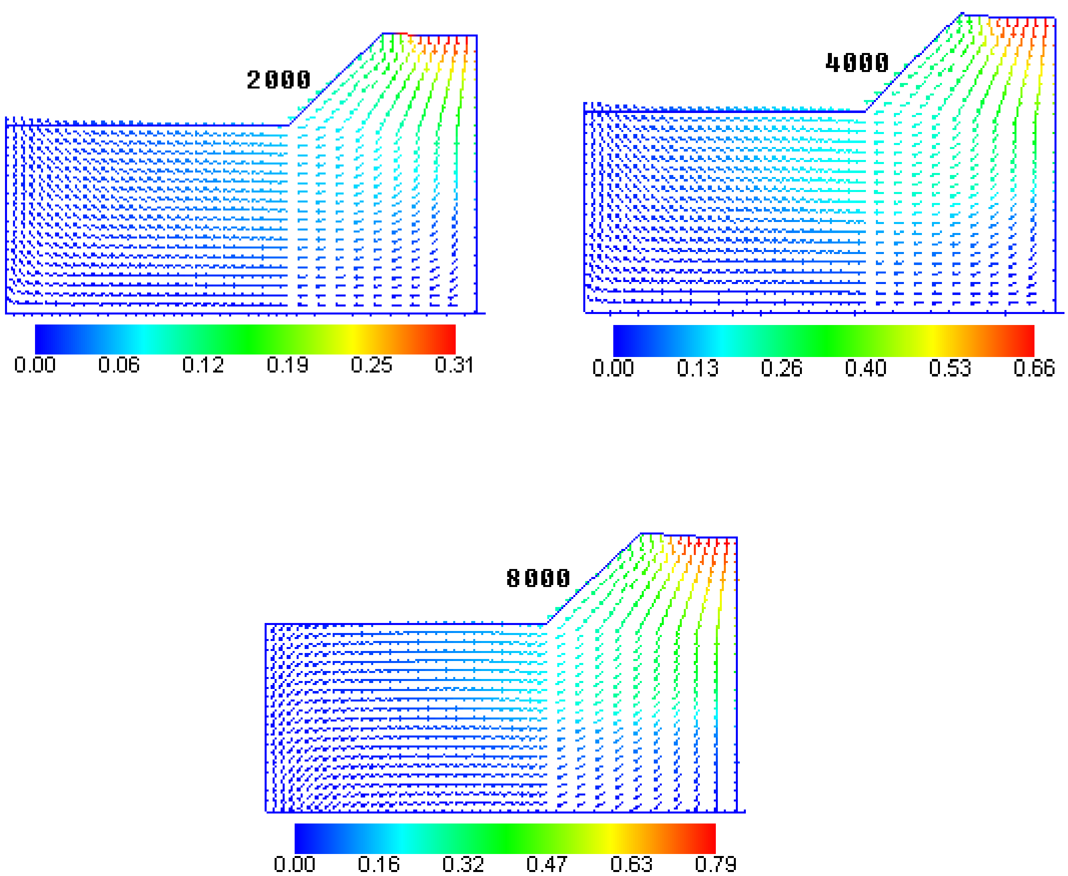

Figure 4.5.4: Displacement vector with time (meter)

4.5.5 Surface Settlement (Case 3)

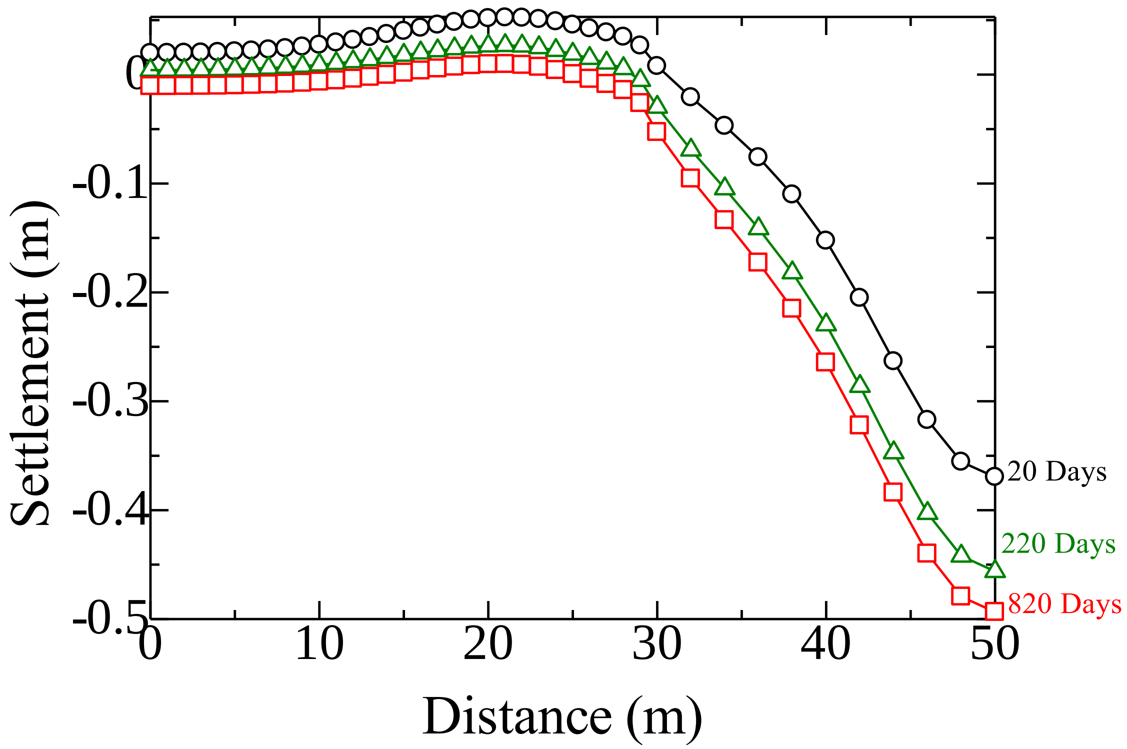

Figure 4.5.5: Plot for surface settlement at different stages (Case 3)

Plot of the surface settlement shows the settlement of the surface of the base. Due to the existence of the water table we can have the plot of settlement with respect to distance in different times. Figure 4.5.5 shows the surface settlement surface settlement of 20, 220 & 820 days. Due to consolidation at 820 day of loading the settlement is 52cm that is a high value.

4.6 Analysis of Case 4 (Nailing, 0m Water)

The case 4 has been introduced with nailing in the soil as reinforcement to increase the stability of the embankment. The case is kept the same 0m water table to compare with the previous case 3 that it loaded without the nailing. There is significant level of change noticed in this case that differs from the previous case that is case 4. And that is the improvement due to using soil nailing. Detailed results discussed below.

4.6.1 Vertical Stress (Case 4)

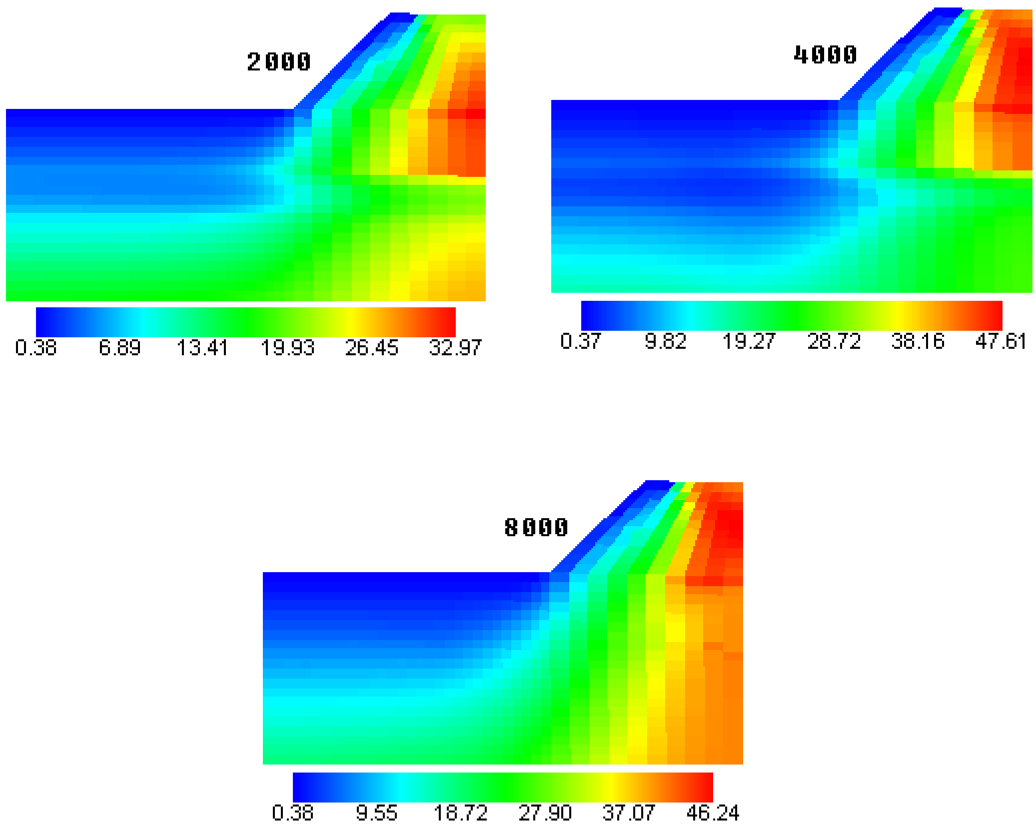

Figure 4.6.1: Distribution of vertical stress at different stages (tf/m^2)

Due to the presence of water in the slope of the embankment this time there is some amount of decrease in the vertical stress of the cross-section. Vertical stress has been decreased by some amount compared to the case 3. Figure 4.6.1 shows the change of the vertical stress in the cross-section. Maximum value of vertical stress is at the bottom of the embankment and its value is 47.98 tf/m^2 which decreased with consolidation steps to 46.69 tf/m^2. All the stresses of the cases are lower than the case 4. Although the slope hasn't failed when water level is introduced but it has been confirmed that there will be decrease in the stress bearing capacity when water exists.

4.6.2 Shear Strain (Case 4)

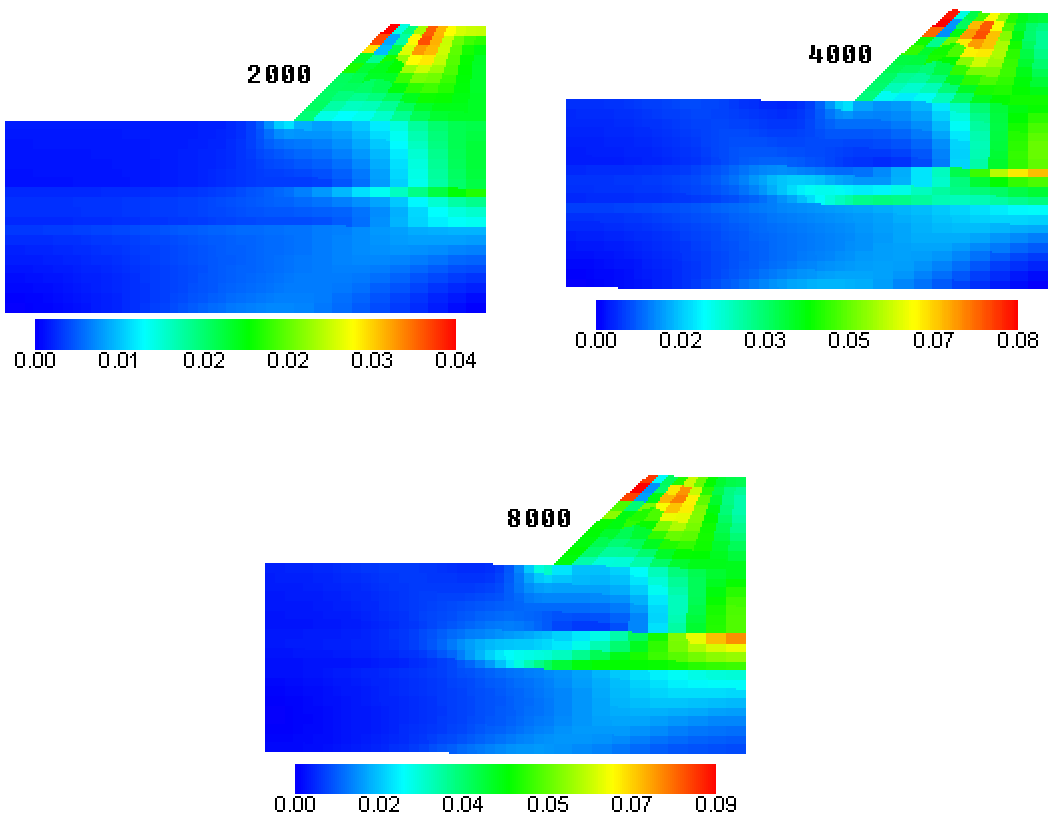

Figure 4.6.2: Distribution of shear strain (case 4)

The figure 4.6.2 has shown that this case has no change in the shear strain value from the case 4. We can see that with the increase of loading the strain is also increasing. In case 4 the maximum value of strain is 0.08 that is achieved at 4000 loading steps and it increases with the time. It is 0.09 at the 8000 steps as it there is consolidation due to water. Figure 4.6.2 shows these values of increasing strain.

4.6.3 Pore Water Pressure (Case 4)

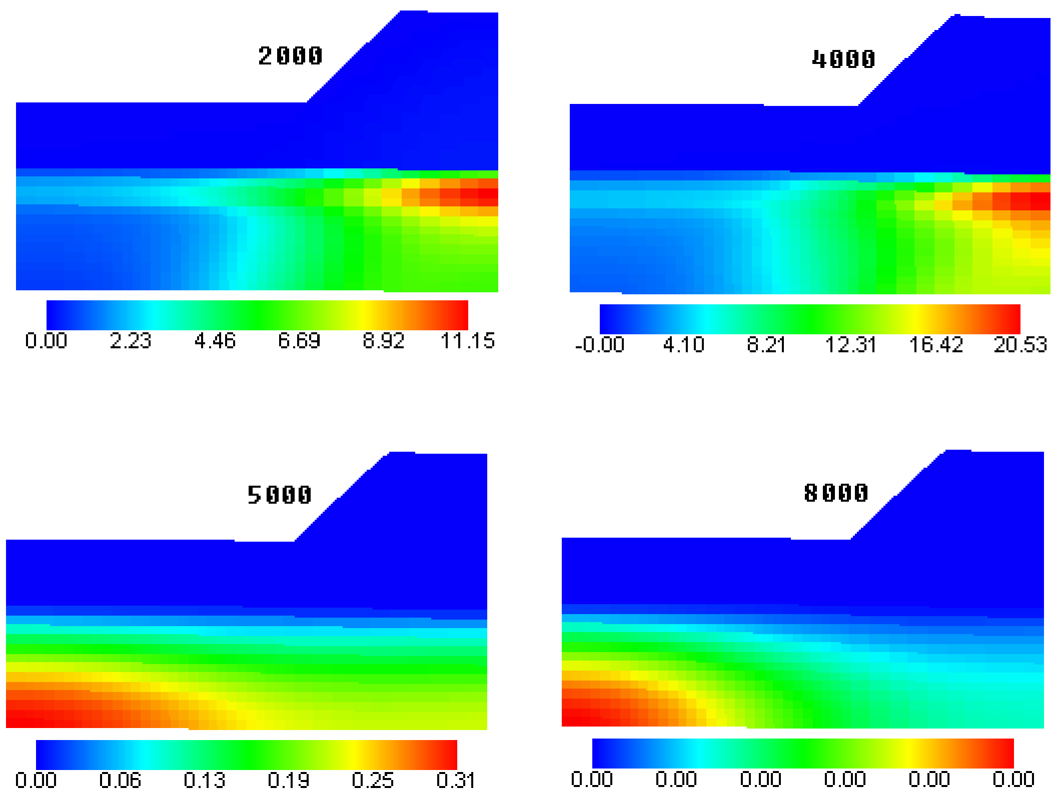

Figure 4.6.3: Distribution of pore water pressure with time (tf/m^2)

In the figure we can see that there is significant pore water pressure development due to the introduction of water table in the embankment. And with the nailing in the soil the pore water pressure has some decreased value in this case. It should be noticed that there is no excess pore water pressure present in the embankment and also the first layer of base. Figure 4.6.3 show the change of pore water pressure. Below the embankment there is the maximum pore water pressure developed within the base. The maximum pore water pressure is developed in 4000 loading steps that is 20.53 tf/m^2 that less than the similar case without nailing. And the pore water pressure is dissipated slowly with the further increase of loading steps and it becomes 0.31 tf/m^2 at 5000 loading steps which is also less then case 4 and 0 at 8000 loading steps.

4.6.4 Displacement Vector (Case 4)

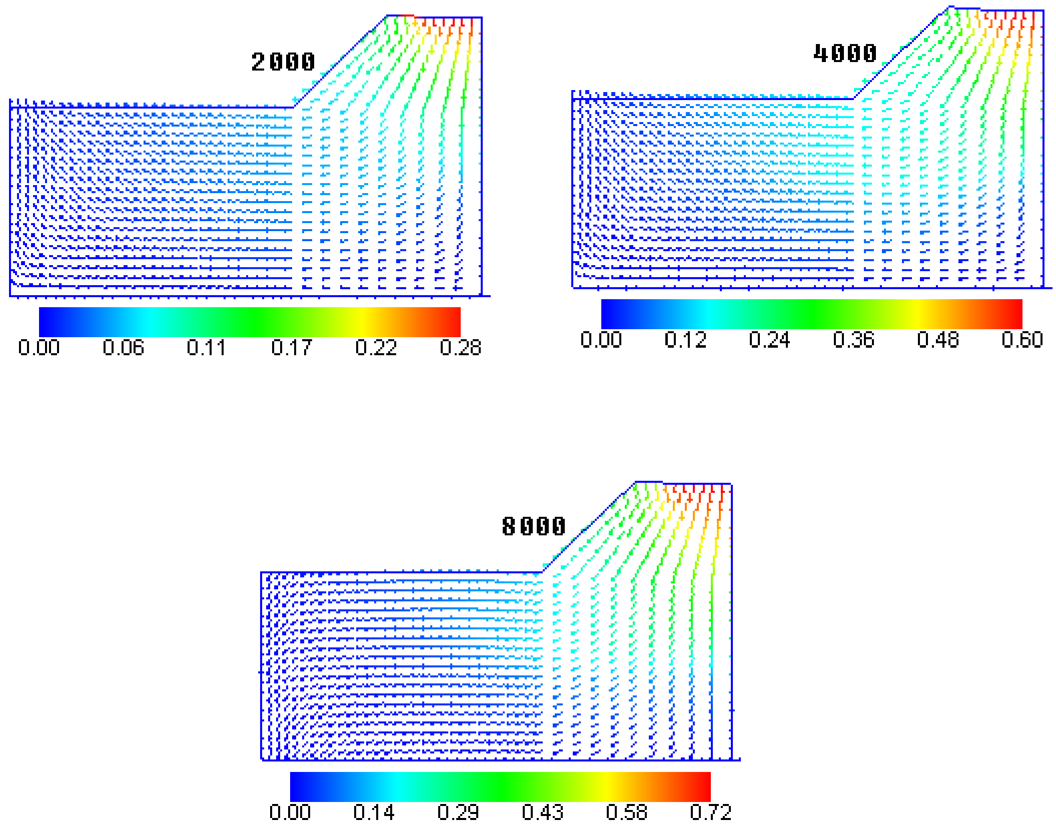

Figure 4.6.4: Displacement vector with time (meter)

The figure 4.6.4 presents the situation of displacement vector after the nailing is given. This case shows the gradual increase in the displacement value of the slope when the load risen up to maximum value that at 8000 loading steps. The slope has shown some improvement as we have use nailing to stabilize the slope. In this case we can see that the value of displacement vector at 2000 loading step is 0.31 m which was 0.33m for the case without nailing, at 4000 loading step value is 0.60m which was 0.66m and at 8000 loading step the value is 0.72 when it was 0.79m in the case 3. So, nailing is improving the soil.

4.6.5 Surface Settlement (Case 4)

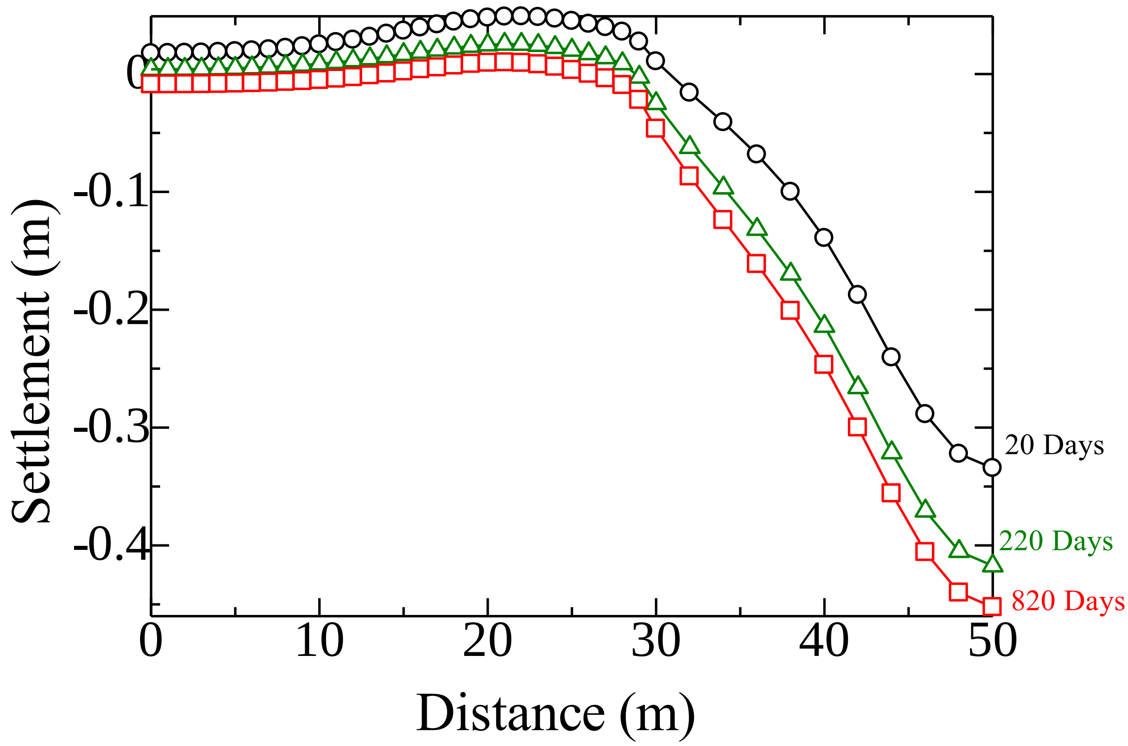

Figure 4.6.5: Plot for surface settlement at different stages (Case 4)

Here the plot of the surface settlement shows the settlement of the surface of the base. Due to the existence of the water table we can have the plot of settlement with respect to distance in different times. Figure 4.6.5 shows the surface settlement surface settlement of 20, 220 & 820 days. Due to consolidation at 220 days of consolidation the value is around 40 cm which was around 46 cm and at 820 days of consolidation the settlement is 45 cm that is also lower value than case 3 that is around 52 cm. Nailing is effectively handling the soil settlement.

4.7 Analysis of Case 5 (No Nailing, 5m Water)

This is another case without soil nailing but this time the water level is brought down to 5m below from the surface of embankment. Case 5 consists the same property of material as before only the change is the water table. Analysis results for case 5 is discussed below in details.

4.7.1 Vertical Stress (Case 5)

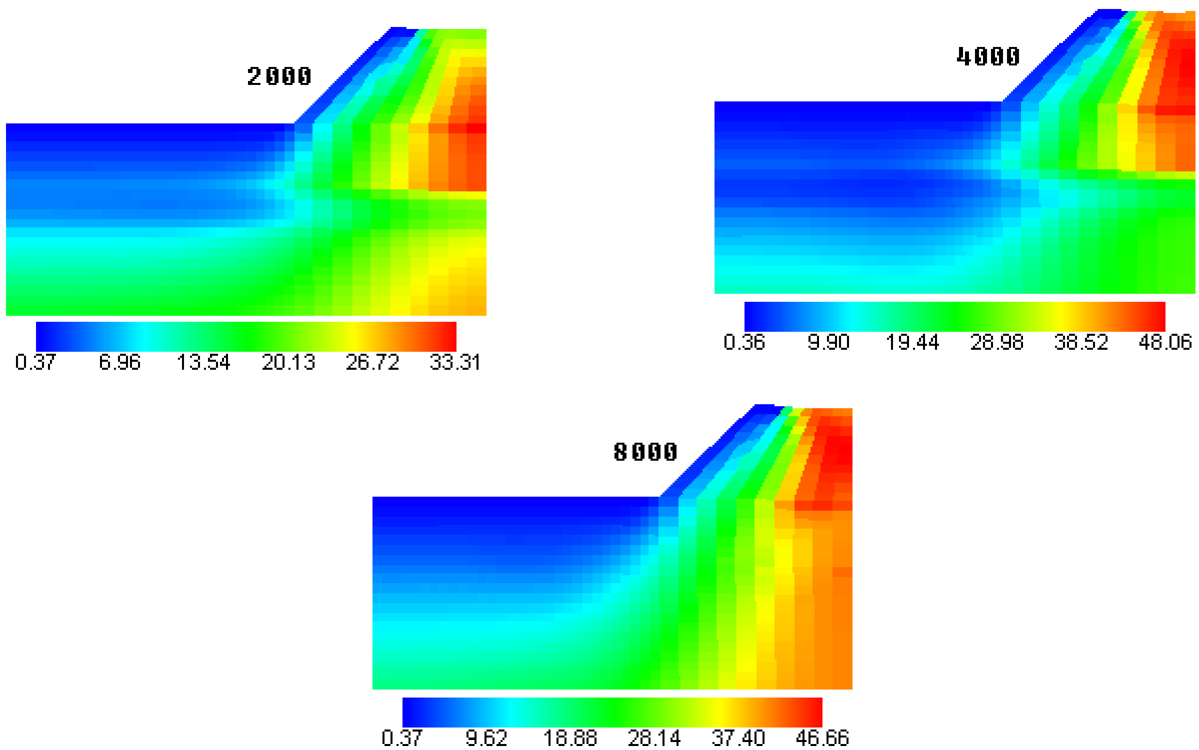

Figure 4.7.1: Distribution of vertical stress at different stages (tf/m^2**)**

Here the water level is at 5m below from the embankment surface and for the presence of water at that level the slope of the embankment this time there is some amount of decrease in the vertical stress of the cross-section at 2000 and 4000 steps than the case 3. Figure 4.7.1 shows the change of the vertical stress in the cross-section. Maximum value of vertical stress is at the bottom of the embankment and its value is 48.06 tf/m^2 which decreased with consolidation steps to 46.66 tf/m^2.

4.7.2 Shear Strain (Case 5)

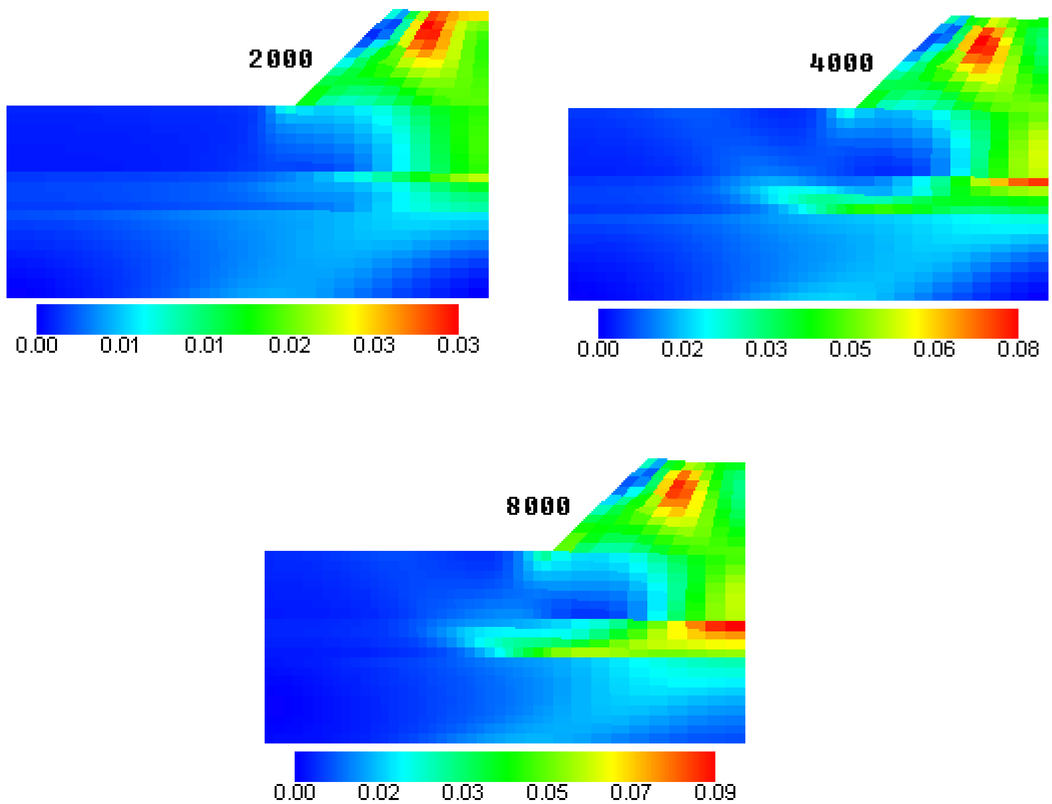

Figure 4.7.2: Distribution of shear strain (case 5)

The figure 4.7.2 shows the value of shear strain of the soil profile. This case we have considered water table at 5m below from the surface level and we can see that with the increase of loading the strain is also increasing. In this case the maximum value of strain is 0.08 that is achieved at 4000 loading steps and it increases to a value of 0.09 at 8000 steps as it there is consolidation due to presence of water which is almost similar to case 3 that have water table at the level of surface. Figure 4.7.2 shows these values of strain at different loading steps.

4.7.3 Pore Water Pressure (Case 5)

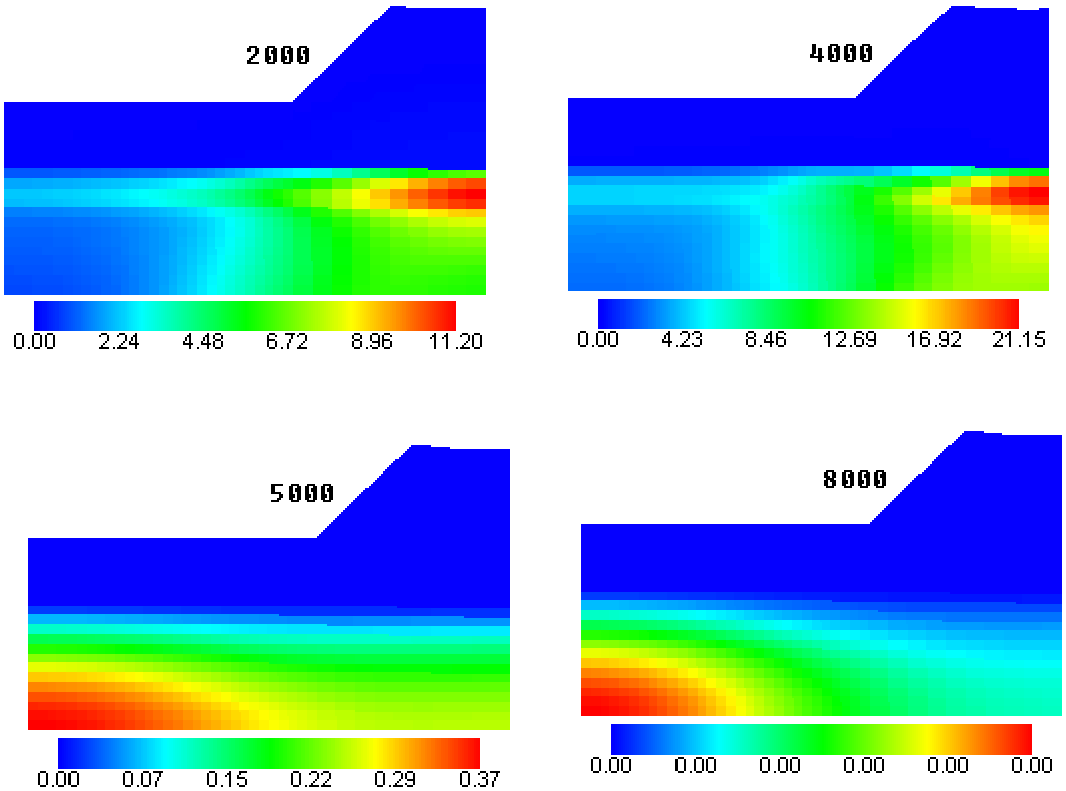

Figure 4.7.3: Distribution of pore water pressure with time (tf/m^2)

There it is the case of embankment without nailing that have a water table 5m below from the surface of the embankment. There is significant pore water pressure development due to the introduction of water table in the embankment and it same as the profile of pore water pressure in the case 3. It should be noticed that there is no excess pore water pressure present in the embankment and also the first layer of base. Figure 4.7.3 show the change of pore water pressure. Below the embankment there is the maximum pore water pressure developed within the base. The maximum pore water pressure is developed in 4000 loading steps that is 21.15 tf/m^2. The pore water pressure is dissipated slowly with the further increase of loading steps and it becomes 0 at 8000 loading steps.

4.7.4 Displacement Vector (Case 5)

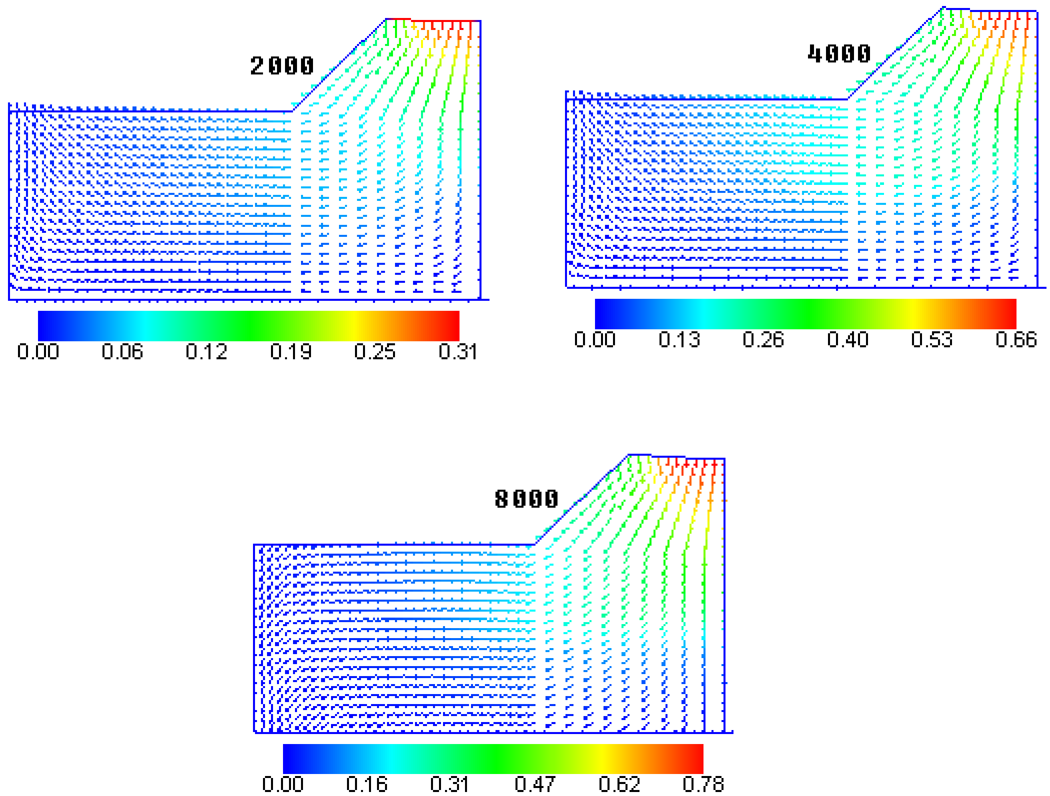

Figure 4.7.4: Displacement vector with time (meter)

Figure 4.7.4 presents displacement vector that shows the gradual increase in the displacement value of the slope when the load risen up to maximum value that at 8000 loading steps. The slope will tend to become unstable as the load is rising. In this case we can see that the value of displacement vector at 2000 loading step is 0.31 m, when it the value is 0.66 m at 4000 loading steps and it is 0.79 m at maximum 8000 loading steps.

4.7.5 Surface Settlement (Case 5)

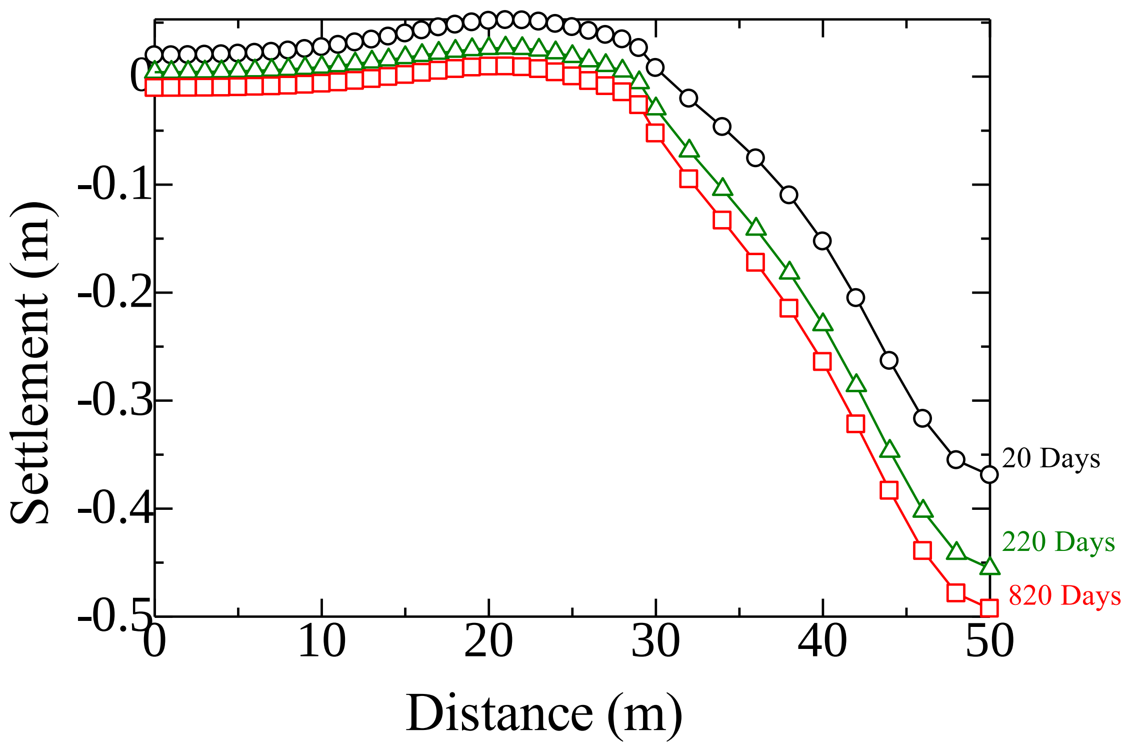

Figure 4.7.5: Plot for surface settlement at different stages (Case 5)

The figure 4.7.5 shows the surface settlement of the base. Due to the existence of the water table it shows almost same results like case 3. We have the plot of settlement with respect to distance in different times. In the figure it shows the surface settlement surface settlement of 20, 220 & 820 days. Due to consolidation at 820 day of loading the settlement is around 50 cm.

4.8 Analysis of Case 6 (Nailing, 5m Water)

The case 6 is the nailed embankment version of the case 5 that has shown improvement and the water level is kept at 5m from the top 0f the surface level of the embankment to check if there is any change due to nail given in the soil embankment. Detailed results are shown below.

4.8.1 Vertical Stress (Case 8)

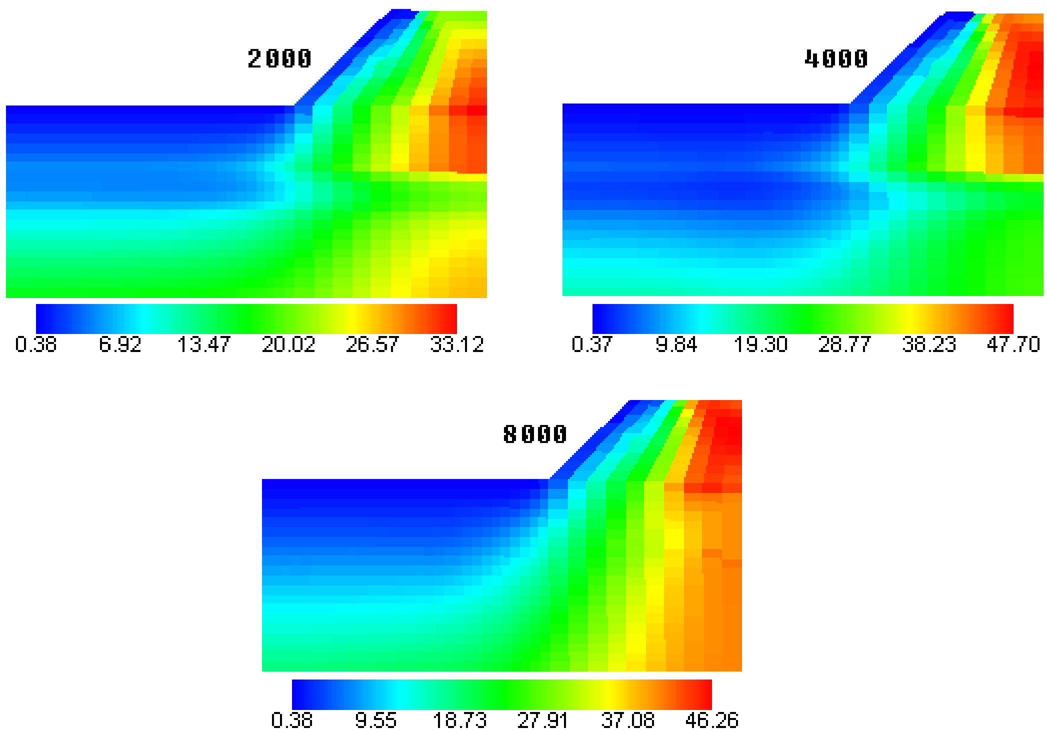

Figure 4.8.1: Distribution of vertical stress at different stages (tf/m^2)

Because of presence of water in the slope of the embankment this time there is some amount of decrease in the vertical stress of the cross-section. Vertical stress has been decreased by some amount compared to the case 5. Figure 4.8.1 shows the change of the vertical stress in the cross-section. Maximum value of vertical stress is at the bottom of the embankment and its value is 47.70 tf/m^2 which decreased with consolidation steps to

46.26 tf/m^2. All the stresses of the cases are lower than the case 5. So there is improvement due to nailing.

4.8.2 Shear Strain (Case 6)

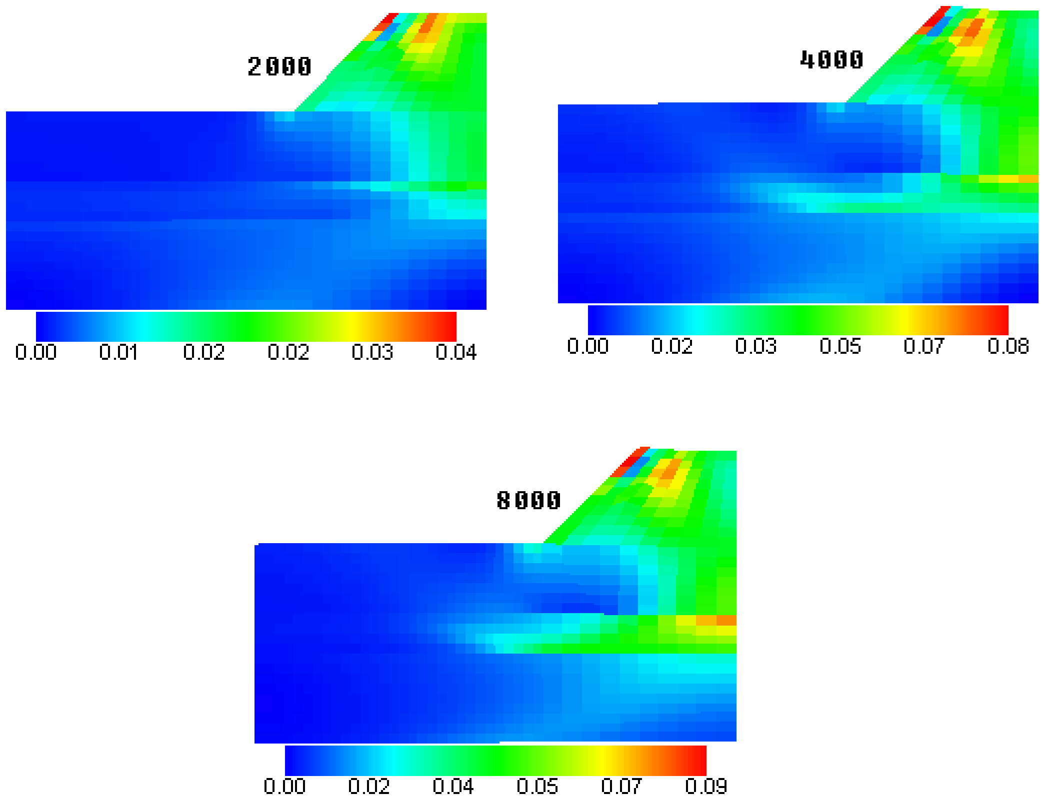

Figure 4.8.2: Distribution of shear strain (case 6)

The figure 4.8.2 has shown that this case has no change in the shear strain value from the case 4. We can see that with the increase of loading the strain is also increasing. In case 4 the maximum value of strain is 0.08 that is achieved at 4000 loading steps and it increases with the time. It is 0.09 at the 8000 steps as it there is consolidation due to water. Figure 4.8.2 shows these values of increasing strain can be observed.

4.8.3 Pore Water Pressure (Case 6)

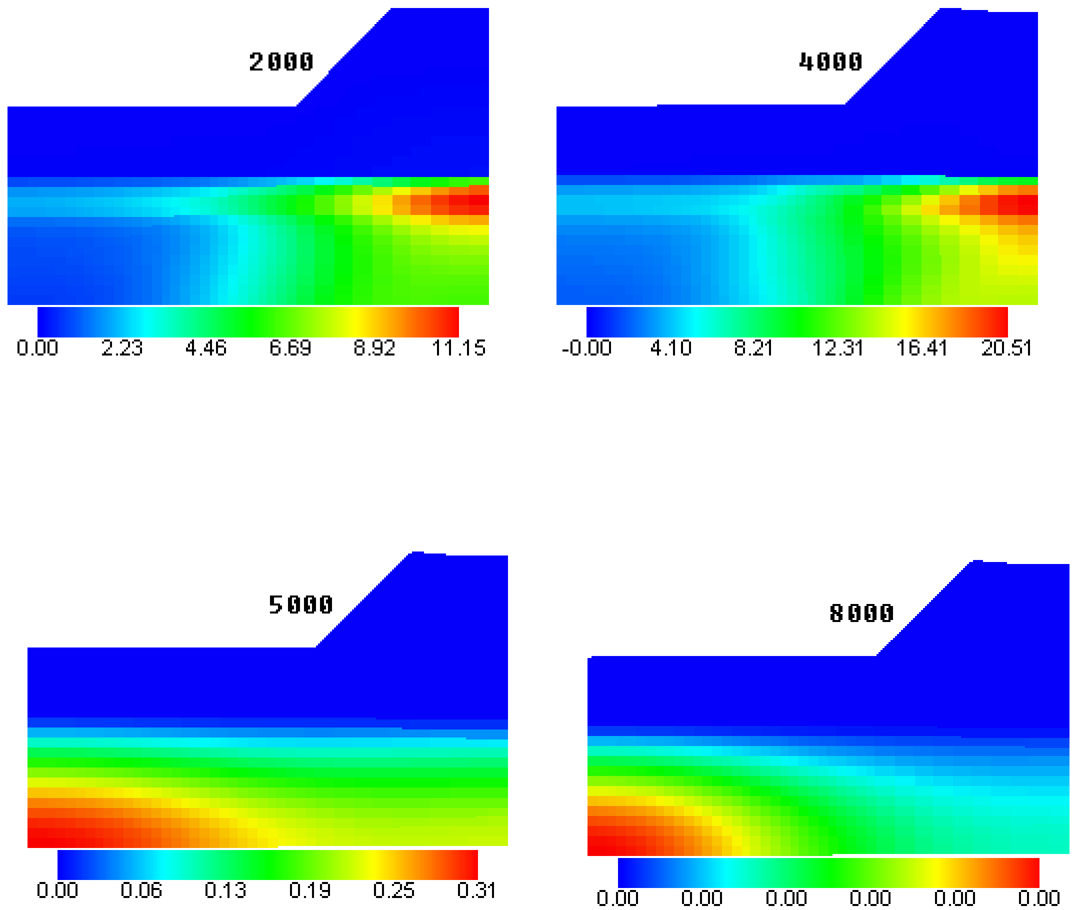

Figure 4.8.3: Distribution of pore water pressure with time (tf/m^2)

Figure 4.8.3 shows that there is significant pore water pressure development due to the introduction of water table in the embankment. And with the nailing in the soil the pore water pressure has some decreased value in this case. It should be noticed that there is no excess pore water pressure present in the embankment and also the first layer of base. Figure 4.8.3 show the change of pore water pressure. Below the embankment there is the maximum pore water pressure developed within the base. The maximum pore water pressure is developed in 4000 loading steps that is 20.51 tf/m^2 that less than the similar case without nailing. And the pore water pressure is dissipated slowly with the further increase of loading steps and it becomes 0.31 tf/m^2 at 5000 loading steps which is also less then case 4 and 0 at 8000 loading steps.

4.8.4 Displacement Vector (Case 6)

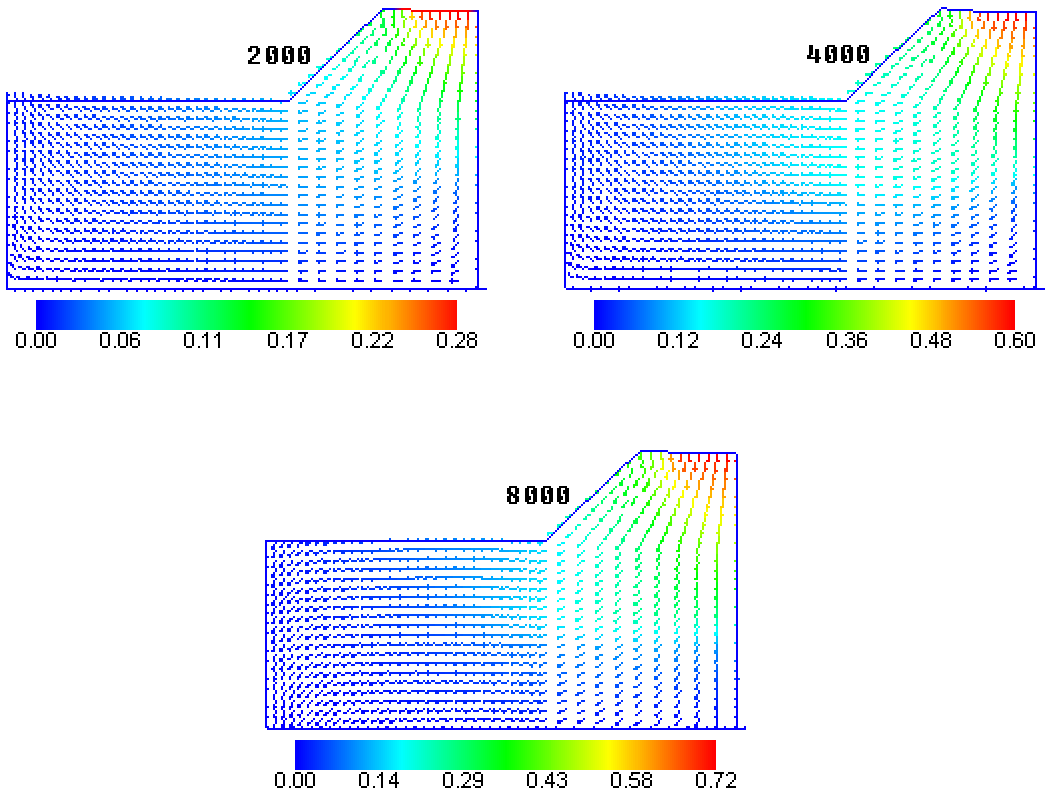

Figure 4.8.4: Displacement vector with time (meter)

The figure 4.8.4 presents the situation of displacement vector after the nailing is given. This case shows the gradual increase in the displacement value of the slope when the load risen up to maximum value that at 8000 loading steps. The slope has shown some improvement as we have use nailing to stabilize the slope. In this case we can see that the value of displacement vector at 2000 loading step is 0.28 m which was 0.31m for the case without nailing, at 4000 loading step value is 0.60m which was 0.66m and at 8000 loading step the value is 0.72 when it was 0.78m in the case 5. So, nailing is improving the soil.

4.8.5 Surface Settlement (Case 6)

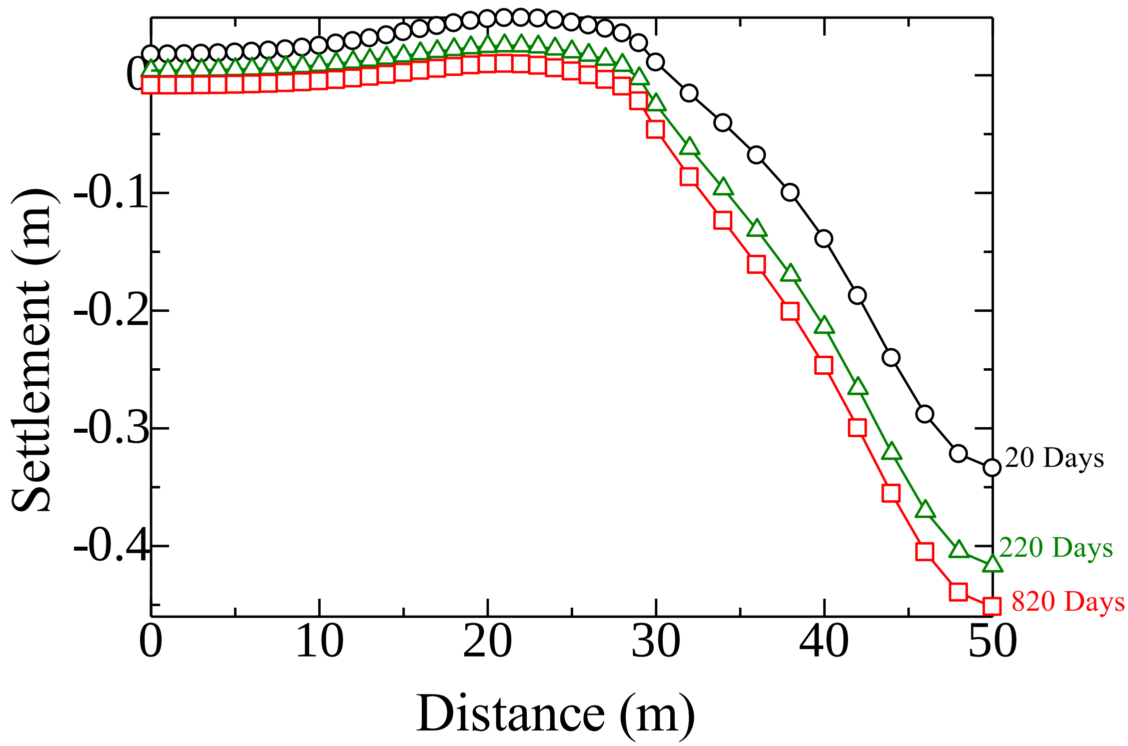

Figure 4.8.5: Plot for surface settlement at different stages (Case 6)

The figure 4.8.5 is the plot that shows the settlement of the surface of the base. Due to the existence of the water table we can have the plot of settlement with respect to distance in different times. Figure 4.6.5 shows the surface settlement surface settlement of 20, 220 & 820 days. Due to consolidation at 220 days of consolidation the value is around 41 cm which was around 47 cm and at 820 days of consolidation the settlement is 47 cm that is also lower value than case 5 that is around 50 cm. Nailing is effectively handling the soil settlement.

4.9 Analysis of Case 7 (No Nailing, 10m Water)

The Case 7 consists of a slope of 1:1 (V:H) and it has a water table 10 m below from the embankment surface. It has a better stabilization than the previous two cases that was water at 0 m and at 5m from the top surface. The pore water pressure development has also been decreased significantly for this slope. Analysis results for case 7 is discussed below in details.

4.9.1 Vertical Stress (Case 7)

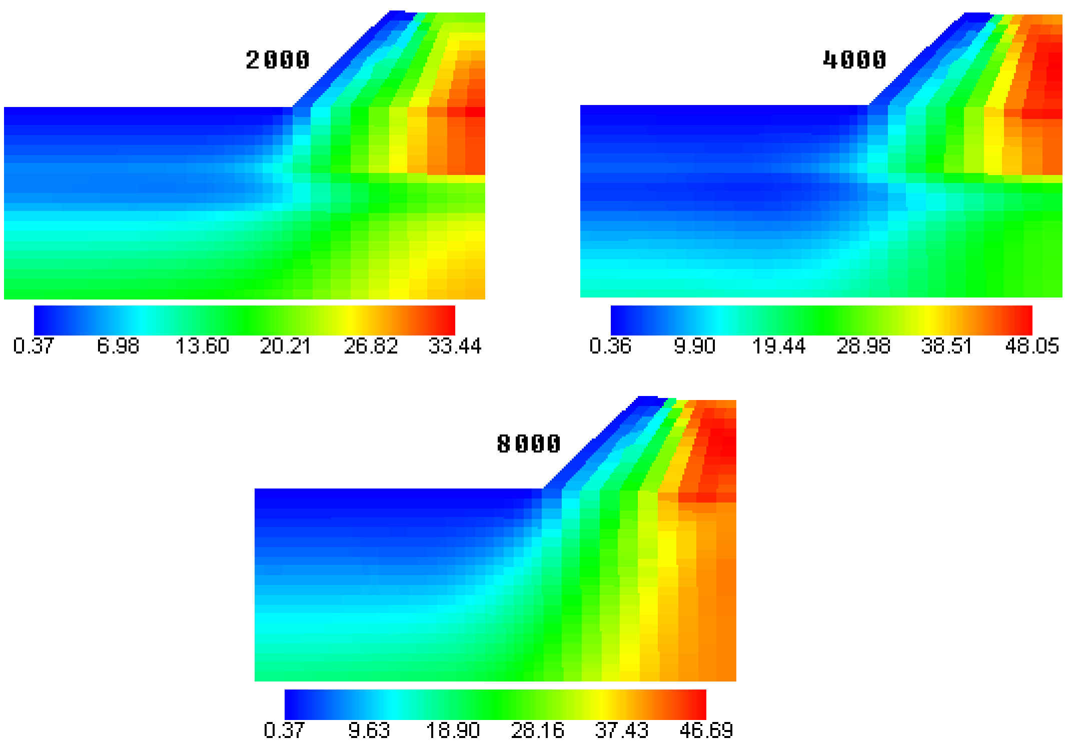

Figure 4.9.1: Distribution of vertical stress at different stages (tf/m^2)

The water level in this case is situated at 10 m below from the embankment surface and for the presence of water at that level the slope of the embankment this time there is some amount of decrease in the vertical stress of the cross-section at 2000 and 4000 steps than the case 5. Figure 4.9.1 shows the change of the vertical stress in the cross-section. Maximum value of vertical stress is at the bottom of the embankment and its value is

48.05 tf/m^2 which decreased with consolidation steps to 46.69 tf/m^2 that is a bit higher value than case 5.

4.9.2 Shear Strain (Case 7)

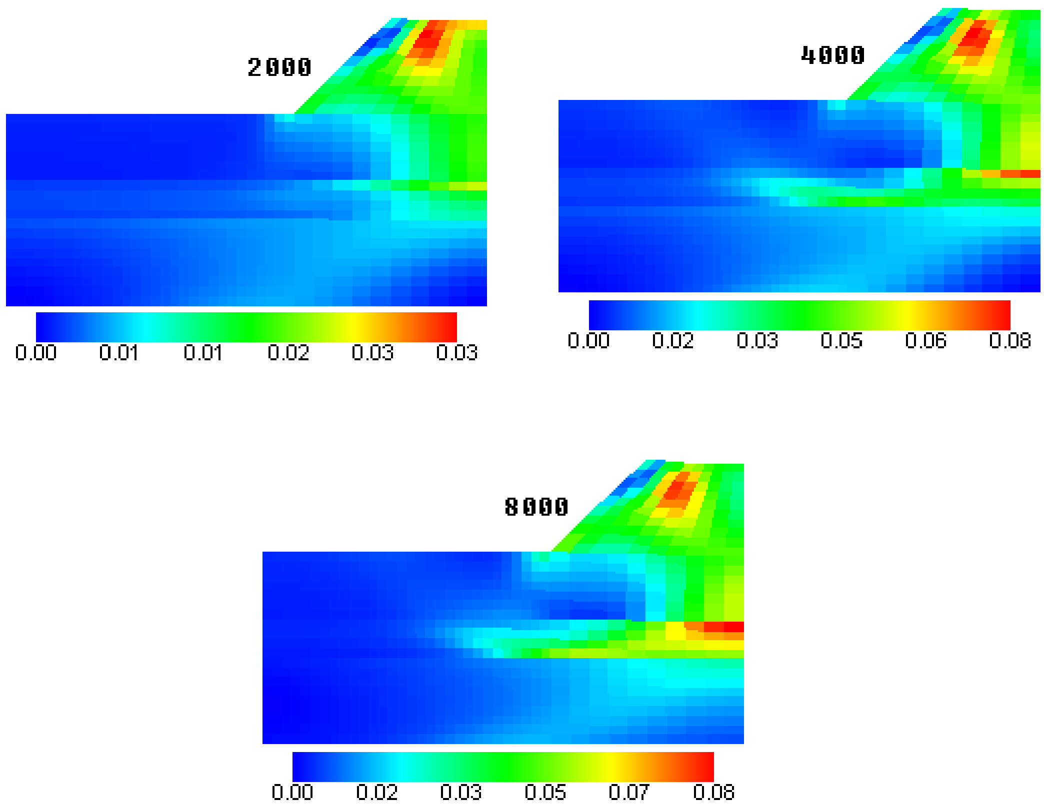

Figure 4.9.2: Distribution of shear strain (case 7)

The figure 4.9.2 shows the value of shear strain of the soil profile. This case we have considered water table at 5m below from the surface level and we can see that with the increase of loading the strain is also increasing. In this case the maximum value of strain is 0.08 that is achieved at 4000 loading steps and stays the same till 8000 steps as it there is consolidation due to presence of water which is almost similar to case 5 that have water table at the level of surface. Figure 4.9.2 shows these values of strain at different loading steps.

4.9.3 Pore Water Pressure (Case 7)

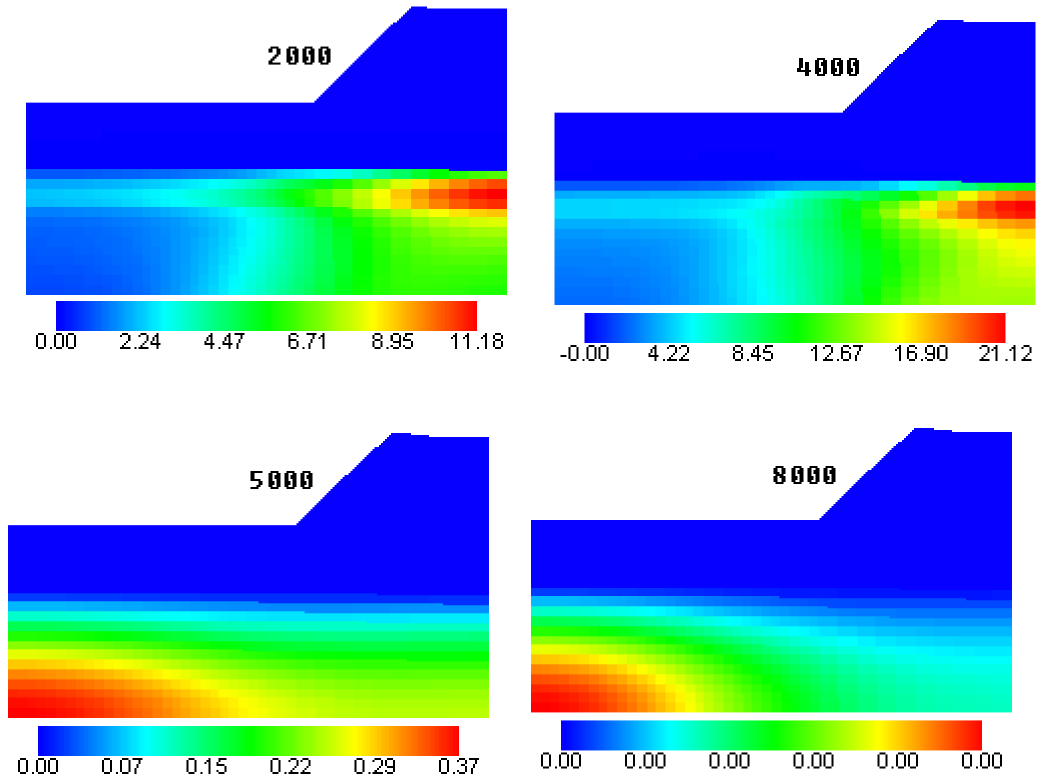

Figure 4.9.3: Distribution of pore water pressure with time (tf/m^2)

This is the case of embankment without nailing that have a water table 10 m below from the surface of the embankment. There is significant pore water pressure development due to the introduction of water table in the embankment and it same as the profile of pore water pressure in the case 5. It should be noticed that there is no excess pore water pressure present in the embankment and also the first layer of base. Figure 4.9.3 show the change of pore water pressure. Below the embankment there is the maximum pore water pressure developed within the base. The maximum pore water pressure is developed in 4000 loading steps that is 21.12 tf/m^2 which is a bit less than case 5. The pore water pressure is dissipated slowly with the further increase of loading steps and it becomes 0 at 8000 loading steps.

4.9.4 Displacement Vector (Case 7)

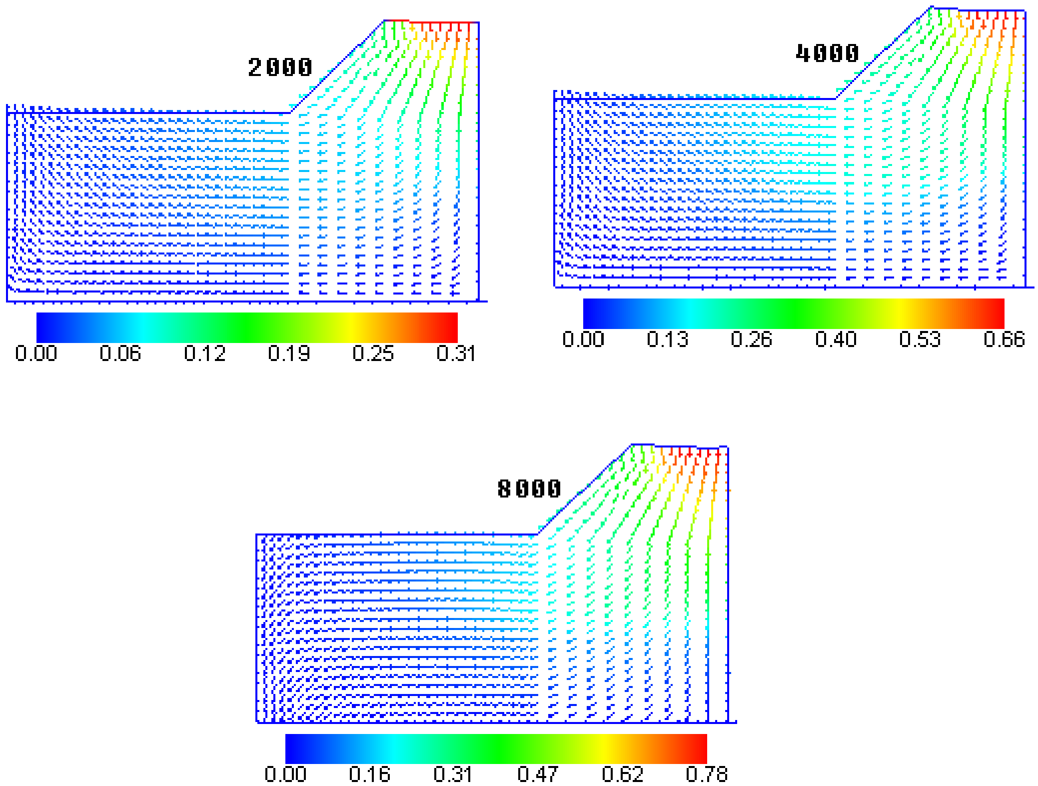

Figure 4.9.4: Displacement vector with time (meter)

Figure 4.9.4 presents displacement vector that shows the gradual increase in the displacement value of the slope when the load risen up to maximum value that at 8000 loading steps. The slope will tend to become unstable as the load is rising. In this case we can see that the value of displacement vector at 2000 loading step is 0.31 m, when it the value is 0.66 m at 4000 loading steps and it is 0.79 m at maximum 8000 loading steps.

4.9.5 Surface Settlement (Case 7)

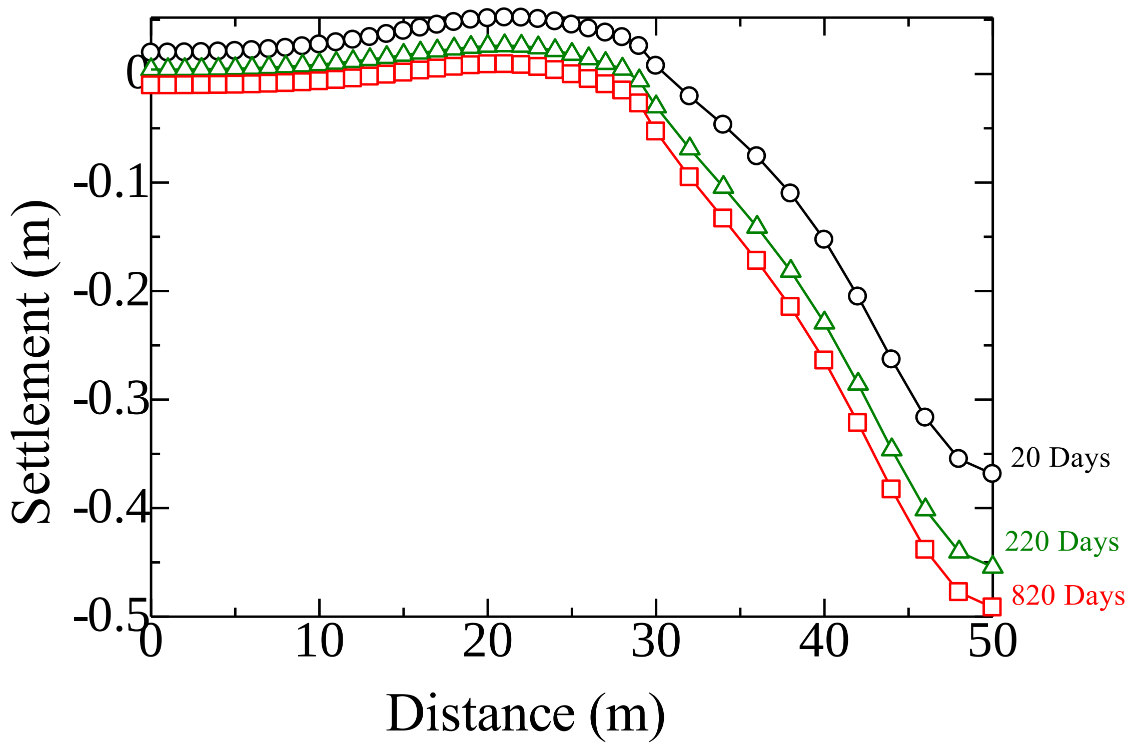

Figure 4.9.5: Plot for surface settlement at different stages (Case 7)

The figure 4.9.5 shows the surface settlement of the base. Due to the existence of the water table it shows almost same results like case 5. We have the plot of settlement with respect to distance in different times. In the figure it shows the surface settlement surface settlement of 20, 220 & 820 days. Due to consolidation at 820 day of loading the settlement is around 51 cm.

4.10 Analysis of Case 8 (Nailing, 10m Water)

Case 3 has been set up with a slope of 1:1 (V:H) and the water level is kept at the surface level of the embankment to check if there is any change due to water level existence in the embankment. There is significant level of change noticed in this case that differs from the previous case that is case1. Detailed results discussed below.

4.10.1 Vertical Stress (Case 8)

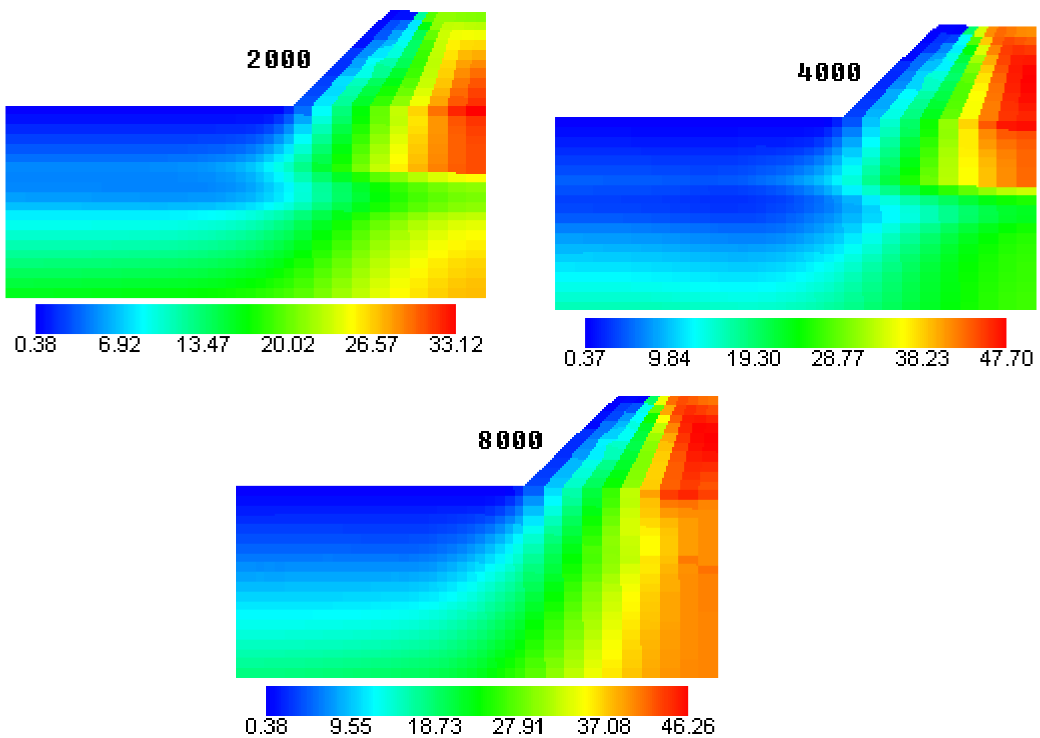

Figure 4.10.1: Distribution of vertical stress at different stages (tf/m^2)

Because of presence of water in the slope of the embankment this time there is some amount of decrease in the vertical stress of the cross-section. Vertical stress has been decreased by some amount compared to the case 7. Figure 4.10.1 shows the change of the vertical stress in the cross-section. Maximum value of vertical stress is at the bottom of the embankment and its value is 47.70 tf/m^2 which decreased with consolidation steps to 46.26 tf/m^2. All the stresses of the cases are lower than the case 7. So, there is improvement due to nailing.

4.10.2 Shear Strain (Case 8)

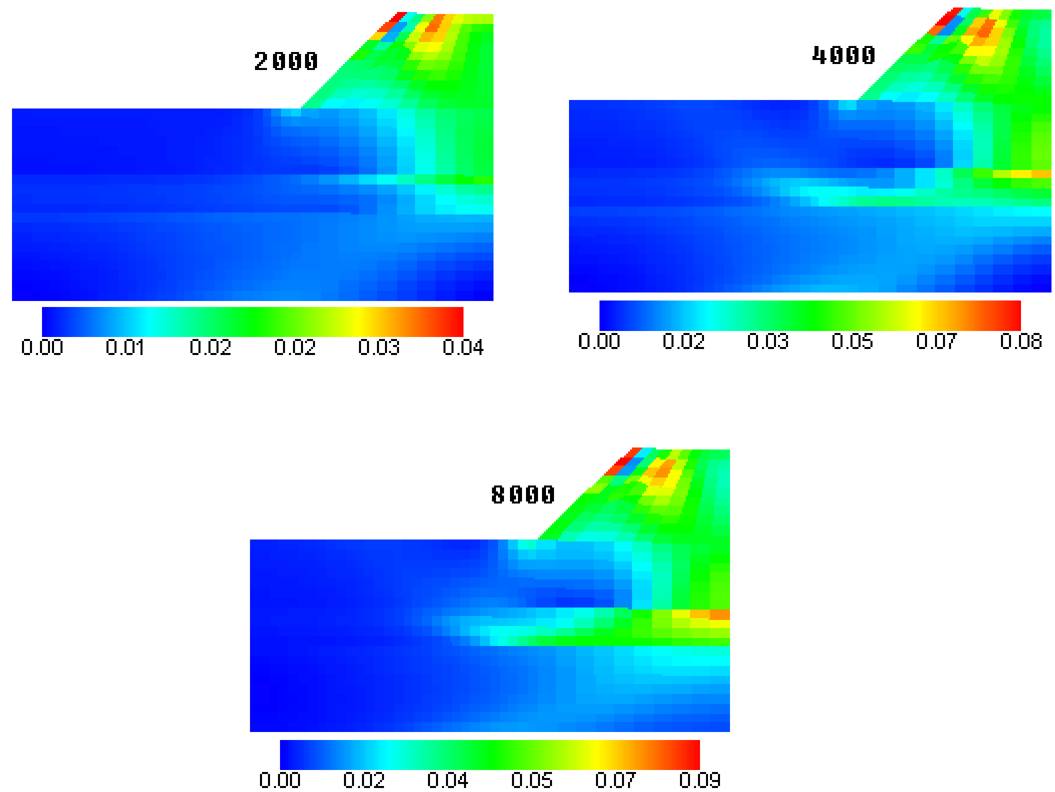

Figure 4.10.2: Distribution of shear strain (case 8)

The figure 4.10.2 has shown that this case has no change in the shear strain value from the case 4 & 6. We can see that with the increase of loading the strain is also increasing. In this case the maximum value of strain is 0.08 that is achieved at 4000 loading steps and it increases with the time. It is 0.09 at the 8000 steps as it there is consolidation due to water. Figure 4.10.2 shows these values of increasing strain can be observed.

4.10.3 Pore Water Pressure (Case 8)

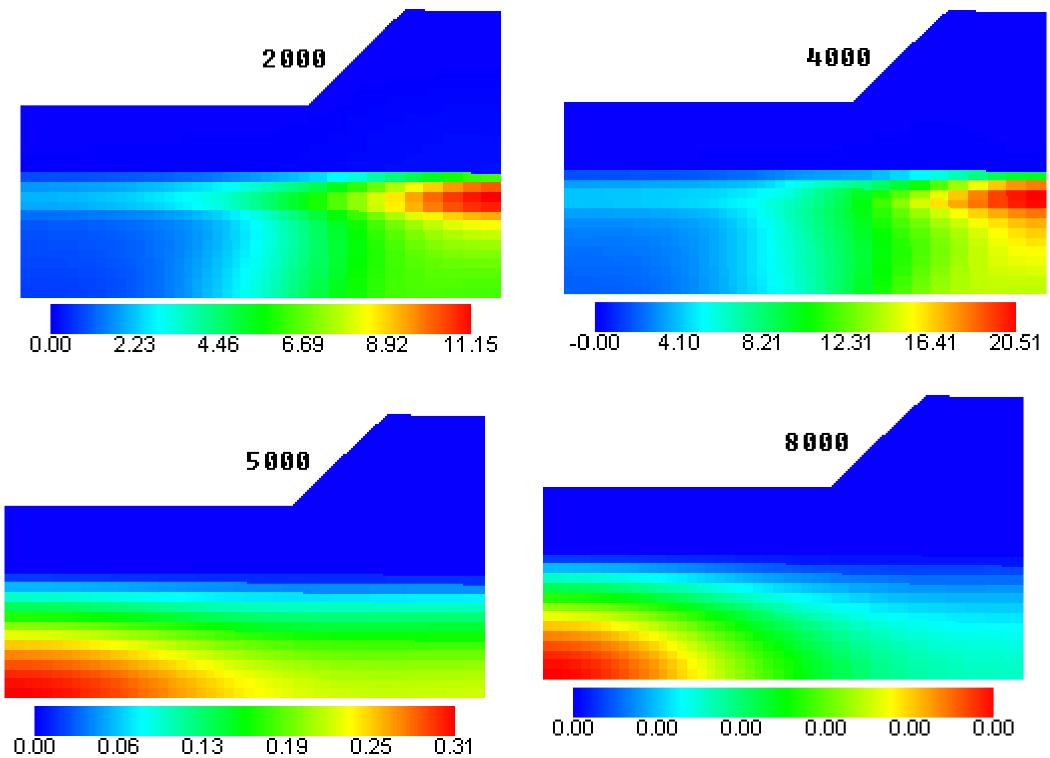

Figure 4.10.3: Distribution of pore water pressure with time (tf/m^2)

Figure 4.10.3 shows that there is significant pore water pressure development due to the introduction of water table in the embankment. And with the nailing in the soil the pore water pressure has some decreased value in this case. It should be noticed that there is no excess pore water pressure present in the embankment and also the first layer of base. Figure 4.10.3 show the change of pore water pressure. Below the embankment there is the maximum pore water pressure developed within the base. The maximum pore water pressure is developed in 4000 loading steps that is 20.51 tf/m^2 that less than the similar case without nailing. And the pore water pressure is dissipated slowly with the further increase of loading steps and it becomes 0.31 tf/m^2 at 5000 loading steps which is also similar to case 6 and 0 at 8000 loading steps.

4.10.4 Displacement Vector (Case 8)

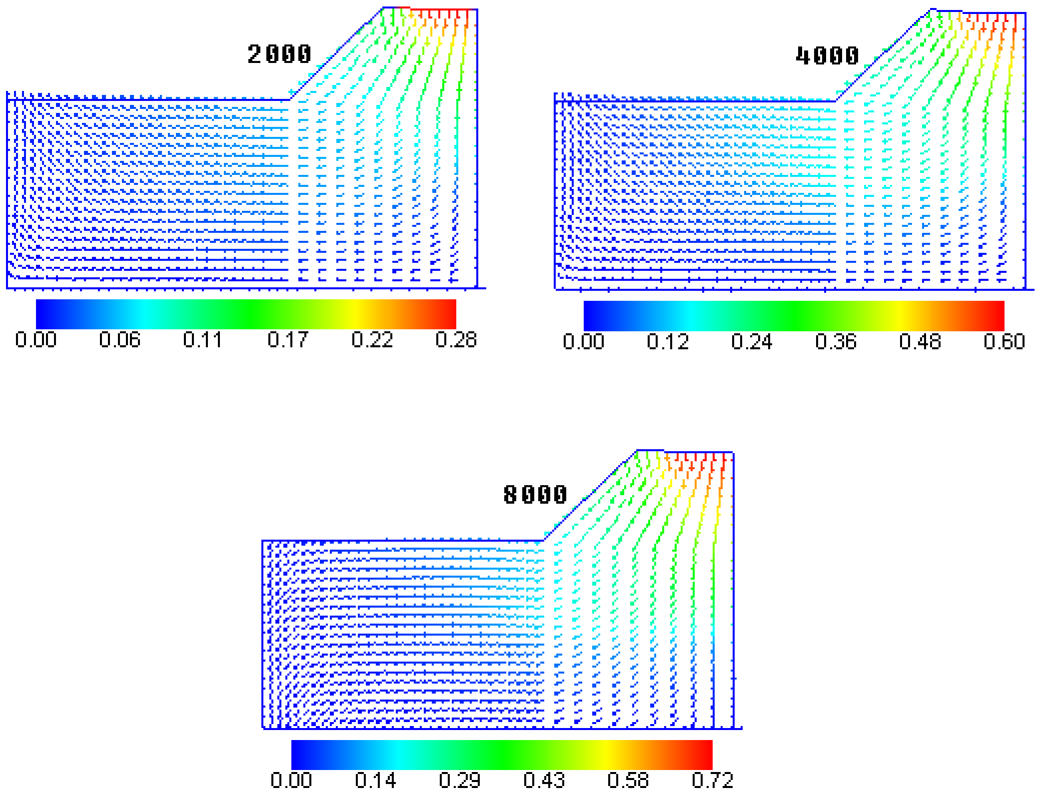

Figure 4.10.4: Displacement vector with time (meter)

The figure 4.10.4 presents the situation of displacement vector after the nailing is given. This case shows the gradual increase in the displacement value of the slope when the load risen up to maximum value that at 8000 loading steps. The slope has shown some improvement as we have use nailing to stabilize the slope. In this case we can see that the value of displacement vector at 2000 loading step is 0.28 m which was 0.31m for the case without nailing, at 4000 loading step value is 0.60m which was 0.66m and at 8000 loading step the value is 0.72 when it was 0.78m in the case 5. It is similar to case 6 also. So, nailing is improving the soil.

4.10.5 Surface Settlement (Case 8)

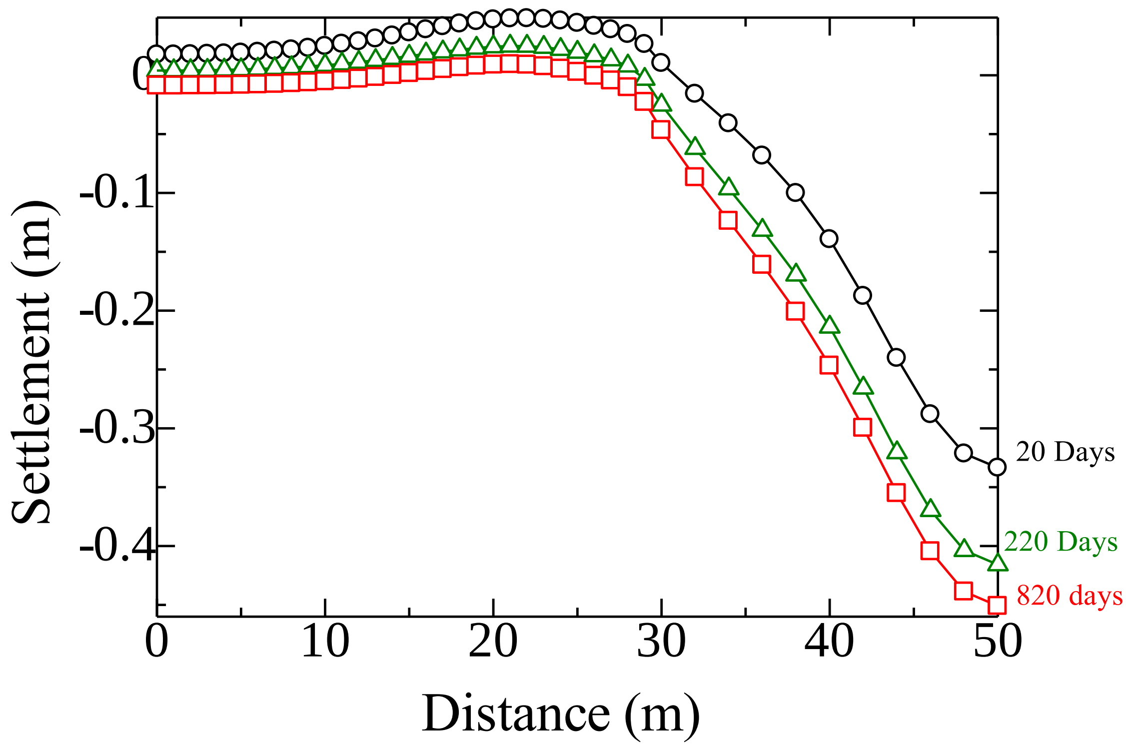

Figure 4.10.5: Plot for surface settlement at different stages (Case 2)

The figure 4.10.5 is the plot that shows the settlement of the surface of the base. Due to the existence of the water table we can have the plot of settlement with respect to distance in different times. Figure 4.6.5 shows the surface settlement surface settlement of 20, 220 & 820 days. Due to consolidation at 220 days of consolidation the value is around 41 cm which was around 47 cm and at 820 days of consolidation the settlement is 47 cm that is also lower value than case 5 that is around 50 cm. But it is similar to case 6. Nailing is effectively handling the soil settlement.

Chapter 5
Conclusion

5.1 General

Slope instability has become a vital concern as it can cause disastrous changes in the surrounding area where it occurs. Huge traffic load can cause increased surface settlement and the displacement of soil also increases which causes huge problems for the safety of the people. Nailing the embankment helps us to reinforce it and make it stable. Through our calculation we have proven that this method is effective enough to make the soil stable by decreasing the displacement and settlement under large traffic load conditions and also under the consideration of drained and undrained conditions with nailing. Thus, we can conclude that soil nailing is an effective process to reinforce the embankment of Dhaka-Feni Express-Way.

Natural slope instability is a major concern in the area where failures might cause catastrophic destruction on the surrounding area. The failures might be triggered by internal or external factors that cause imbalance to natural forces. An internal triggering factor is the factor that causes failure due to internal changes, such as increasing pore water pressure and or imbalanced forces developed due to external load.

The analyses performed in this study given the specific conditions of soil properties, slope geometry, and pattern of water-level changes showed that the water-level fluctuations resulted in lowering of stability and development of vertical deformations at the slope crest. The stability decrease is to assign to the fact that the groundwater table is increasing for each water level fluctuation.

Despite the fact that the absolute magnitudes of differences identified in this study were sufficient vertical displacement, pore water pressures, shear strain etc. these do nonetheless demonstrate important dissimilarities concerning the ability to capture/simulate real soil-water interactions and changes.

The results obtained in this study are reflecting effects of soil nailing in improving the soil overall stability. Since from the analyzed 8 cases shows that soil nailing can be effective in improving the vertical stress, shear strain, pore water pressure handling and settlement of the embankment of the road.

5.2 To be further considered

Our research of the nailing is done with 3m nails in the embankment. Future research using the results getting from different stability analyses will focus on the following areas as this study did not address them:

- Using different nail size that can be 4m, 5m and 6m.
- Stability analysis for different slopes considering frequent water level rise.
- Determination of allowable water level for stable slope.
- Factor of safety will be determined.
- Required ground reinforcement strategy will be applied for slope protection.

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