Design and Analysis of Reinforced Road Embankment on Soft Soil by using Finite Element Modeling Software PLAXIS

Rajkumar Yadav1 Ankesh Shrivasthava2 Sooraj Jain3
1M.E Scholar 2,3Assistant Professor
1,2,3Samrat Ashok Technological Institute, Vidisha, India

Abstract— During construction of roads on soft soils a certain bearing capacity of the sub base is required to prevent unnecessary differential settlements of the road structure. For subsoil with insufficient bearing capacity and, stabilization is necessary. The bearing capacity can be increased by excavation of the soft material, chemical stabilization by using chalk or by using geosynthetics. In Gujarat, Bhavnagar proposed approach road between creek kalatalav sanesh to Bhavnagar, soil of this site is black cotton soil and generally CBR of black cotton soil is less than 2%. For developing a good and durable road network in black cotton soil areas, the nature of soils shall be properly understood. In the present paper, reasons for poor condition of roads in B.C soils and measures to be taken for construction and improvement of roads on BC soils are presented.

Key words: Reinforced Road Embankment, Finite Element, PLAXIS

I. INTRODUCTION

For several years geosynthetics are used to stabilize the sub base for unpaved roads like rural roads or access roads. For paved roads till now the use of geosynthetic reinforcement is not discussed due to the very small displacements of the surface and due to the poor long term tensile strength of geosynthetics used in roads. For paved roads geosynthetics should be used which have a very positive stress-strain behavior and good long-term behavior.

Bhavnagar is a city in the state of Gujarat and it is situated 198 km from the state capital Gandhinagar. This proposed Creek Kalatalav Sanesh road is the approach road to Bhavnagar District and it is the approach road for nearby Salt Industries also.

The proposed road is formed in such conditions in most cases Rural Roads of Madhya Pradesh, where the ground water table is high and flood level is also high during the monsoon. Also sub grade strength was found to be CBR<2% with poor soil properties. The road used to demand high maintenance and repair cost.

Generally, lands with black cotton soils are fertile and very good for agriculture, horticulture, sericulture and aquaculture. Good irrigation systems exist, rainfall is high and people are affluent in these areas. Though black cotton soils are very good for agricultural purposes, they are not so good for laying durable roads. Good road network is a basic requirement for the all-round development of an area. Unfortunately, poor road network is hampering the full-fledged development of the otherwise prosperous areas.

For developing a good and durable road network in black cotton soil areas, the nature of soils shall be properly understood. Black cotton soils absorb water heavily, swell, become soft and lose strength. Black cotton soils are easily compressible when wet and possesses a tendency to heave during wet condition. BC soils shrink in volume and develop cracks during summer. They are characterized by extreme hardness and cracks when dry. The stability and performance of the pavements are greatly influenced by the sub grade and embankment as they serve as foundations for pavements. On such soils suitable construction practices and sophisticated methods of design are to be adopted. Considering the site condition and poor soil properties, we suggested a composite that will help to offer the required design strength for road pavement construction, necessary for this project.

The solution included proper drainage with High Strength Geo-grid was provided for improving the ground conditions and uniform stress transfer to subsoil. Selection of product and solution was done meticulously to mitigate the present problem of coastal road.

II. LITERATURE REVIEW

Paravita Sri Wulandari & Daniel Tjandra Analysis of geotextile reinforced road embankment using PLAXIS 2D. (Science Direct 2015), the purpose of this study is to determine the optimum tensile strength of geotextile as the reinforcement in road embankment considering the allowable factor of safety and displacement. The stability analysis of the road embankment has been done by finite element method using PLAXIS 2D. In this study, three types of sequence modeling were conducted. First, the stability of road embankment without any reinforcement was analyzed. Second modeling was to determine the length of geotextile reinforcement considering the stability of the model road embankment. The last sequence was to investigate the stability of the model reinforced embankment with various tensile strength of geotextiles reinforcement. The result of this study showed that the optimum tensile strength of geotextiles was strongly influenced by the factor of safety. The 4 m of embankments with slopes as 2H : 1V, was investigated in this study. The water level was at 0.00 level. A nominal surcharge of 6.13kN/m³ has been used for modeling the traffic load. A layer of geotextile PET woven is placed between the base of embankment and the sand mat. The tensile strength of geotextile reinforcement was varied from 100 up to 1000 kN/m.

In an earlier study, (M. Rouainia 2005) investigated the influence of structure degradation on the behaviour of embankments on soft soil. Using the fully instrumented trial embankments which had been constructed on estuarine clay deposits on the Pacific Highway along the east coast of Australia, the Kinematic Hardening Structure Model (KHSIM) proposed by (D. Muir Wood 2000) which is proficient in addressing the initial structure and destructuration process in soft clays was implored to validate the settlement of the embankment by the finite element method. The parameters which were used in the
KHSM were calculated from a set of field and laboratory data from soft soils in that region and in other to predict the consolidation behaviour of the soft deposit, coupled analysis of excess pore pressure and displacements was used in the finite element implementation. The left and right boundaries of the finite element mesh that was used to describe the soils below the embankment were restrained in the horizontal directions, the bottom boundary was set to be undrained and the top boundary was set to be free drained to zero pore pressure. As the embankment is symmetric, just half of the embankment was simulated in the model and allowed to consolidate for 100,000 days. The settlements at the ground surface as well as the excess pore pressures beneath the embankment along some reference points were compared with the field measured data to access the level of accuracy of the finite element model.

From the results, the displacements predicted by the finite element analysis using the KHSM were very good in the overall. However, the excess pore pressures were under predicted as a result of variable boundary in-situ conditions. It was concluded that since in soft clay materials, the excess pore pressures and settlements are connected, using models that account for destructuration processes were necessary to correctly study geotechnical problems with related materials (M. Rouainia 2005).

(Marcin Cudny 2003) investigated the possibilities of a multi-laminate constitutive model which accounts for structural anisotropy and destructuration effects with the 2D finite element code Plaxis to simulate the behaviour of a test road embankment constructed on soft soil deposits at Haarajoki, Finland. A spatially distributed anisotropic overconsolidation was introduced in the multi-laminate model which combines the strength anisotropy with the characteristic mechanical process of destructuration. The longitudinal and cross sections of the Haarajoki test embankment. The construction of the embankment was completed within 3 weeks and was done in multi-stages of 0.5 m using a gravel fill. Excess pore pressures of about 3 to 10 kPa were measured in the deposit before the construction began. The part of the embankment without ground improvement (cross section 35840) was analysed in the study.

III. DESIGN, MYTHOLOGY & MATERIALS

A. General Information

The test embankment at Bhavnagar is a city in the state of Gujrat, situated 198 Km. from the state capital Gandhinagar. The proposed road is formed between the Creeks, where the ground water table is high and flood level is also high during the monsoon. Also sub grade strength was found to be CBR<2% with poor soil properties. The road used to demand high maintenance and repair cost. Which organize a competition with tasks involving evaluation of settlements, horizontal displacements and pore pressure? The 2.0 m high embankment is constructed as a noise barrier and is founded on soft soil deposits. The deposits in this area are characterized by a high degree of anisotropy and natural inter-particle bonding (Yildiz et al. 2009) which influences its stress-strain behavior. Experiments such as oedometer test (both Incremental Loading (IL) and Constant Rate of Strain (CRS)) and triaxial tests have been conducted by Road Administration Consulting Laboratory. Since clay is highly anisotropic, there is large variation with respect to depth which is also evident from laboratory results from oedometer IL tests. Hence depths exhibiting similar stress-strain behavior and initial conditions are grouped in a single layer and this process of layering samples with respect to depth would be discussed below

B. Field Observations

It has been observed that even on ground improved areas, primary consolidation continues significantly after three years of embankment construction. The strain effects are found to be minor below a depth of 10 meters. The effect of three years consolidation period is clearly observed on layers closer to the embankment. The increase of un-drained shear strength and pre-consolidation pressure are small for subsoil that was slightly over-consolidated before embankment construction. Hence it is clear from field observations that the most critical region should exist around 2-4 m for soft soil layer combined with dry crust behavior. The 2m thick dry crust layer can distort the prediction accuracy since even advanced models are unable to capture the stress strain behaviour for dry crust soils due to difficulty in its parameter derivation.

C. Embankment

An illustration of the embankment cross-section is shown in Figure 3.1. The embankment is 2.0 m high and 20 m long. The crest of the embankment is 12 m wide with slopes constructed at a gradient of 1:2. The phreatic level is at the ground surface. The embankment material varies from Moorum to Sandy Soil with a moisture content of 2.5 % and maximum dry density of 1.7 K/\text{m}^3. Size distribution includes 18 % of 32 to 64 mm fraction and 20 % for more than 64 mm. The saturated density of the embankment material is 21KN/m3. Mirafi georaid has been spread on the leveled ground over the entire embankment. The embankment is underlain by heavily over consolidated crusted clay layer. The dry crust layer is 2 m thick and is followed by 10.5 m thick soft clay deposit. Deposits that is more permeable than clay such as Hard Clay and Sandy Soil can be found beneath the soft clay layer.

![Fig. 1: Geometry of Pavement](image)

D. Soil Profile

The water content for soft clay varies between 70 to 120 % with maximum water content in the shallow layer of soft soil and decreases with depth. The water content for soft soil is equal to or, in most cases, greater than their liquid limit. Soft soil around 3m depth exhibit the highest plasticity index and tends to decrease with depth. The dry crust layer,
however, exhibits a much lower range of water content ranging between 35 to 55%. The loss of water can be due to desiccation from high temperature variation. This leads to crack propagation and high permeability properties in the first initial layers. From desiccation, there is swelling in soil leading to reduction in void ratio and subsequent increase in its unit weight. The unit weight for dry crust layer varies between 16.86 to 17.68 kN/m³ whereas for soft soil, the unit weight varies between 13.72 - 16.21 kN/m³ increasing with depth. The organic content varies between 1.2 % - 2.2 %. The sensitivity of the soil varies between 20 to 65 depending on the depth.

### Table 1: Properties of Different Soil

<table>
<thead>
<tr>
<th>Type</th>
<th>Drain ed</th>
<th>UnDrai ned</th>
<th>UnDrai ned</th>
<th>Drain ed</th>
</tr>
</thead>
<tbody>
<tr>
<td>$γ_{sat}$ [kN/m³]</td>
<td>17.00</td>
<td>16.00</td>
<td>15.00</td>
<td>16.00</td>
</tr>
<tr>
<td>$γ_{sat}$ [kN/m³]</td>
<td>20.00</td>
<td>19.00</td>
<td>18.00</td>
<td>20.00</td>
</tr>
<tr>
<td>$k_s$ [m/day]</td>
<td>0.500</td>
<td>0.000</td>
<td>0.000</td>
<td>1.000</td>
</tr>
<tr>
<td>$k_s$ [m/day]</td>
<td>0.500</td>
<td>0.000</td>
<td>0.000</td>
<td>1.000</td>
</tr>
<tr>
<td>$e_{sat}$ [-]</td>
<td>0.500</td>
<td>0.000</td>
<td>0.500</td>
<td>0.500</td>
</tr>
<tr>
<td>$c_u$ [-]</td>
<td>1E15</td>
<td>1E15</td>
<td>1E15</td>
<td>1E15</td>
</tr>
<tr>
<td>$E_{sat}$ [kN/m²]</td>
<td>30000</td>
<td>1500.00</td>
<td>10000.00</td>
<td>3000.00</td>
</tr>
<tr>
<td>$G_{sat}$ [kN/m²]</td>
<td>11538</td>
<td>572.51</td>
<td>375.94</td>
<td>1153.80</td>
</tr>
<tr>
<td>$E_{sat}$ [kN/m²]</td>
<td>40384</td>
<td>2079.14</td>
<td>1481.64</td>
<td>4038.46</td>
</tr>
<tr>
<td>$c_{ref}$ [kN/m²]</td>
<td>1.00</td>
<td>2.00</td>
<td>2.00</td>
<td>1.00</td>
</tr>
<tr>
<td>$E_{inc}$ [kN/m²]</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$γ_{ref}$ [m]</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
<td>0.000</td>
</tr>
<tr>
<td>$C_{conc}e$ [kN/m²]</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$T_{str}$ [kN/m²]</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
<td>0.00</td>
</tr>
<tr>
<td>$R_{inter}$ [-]</td>
<td>1.00</td>
<td>1.00</td>
<td>1.00</td>
<td>0.65</td>
</tr>
<tr>
<td>Interface permeability</td>
<td>Neutra l</td>
<td>Neutra l</td>
<td>Neutra l</td>
<td>Neutra l</td>
</tr>
</tbody>
</table>

### Design and Analysis of Reinforced Road Embankment on Soft Soil by using Finite Element Modeling Software PLAXIS

PLAXIS is a finite element package that has been developed specifically for the analysis of deformation and stability in geotechnical engineering projects. The simple graphical input procedures enable a quick generation of complex finite element models, and the enhanced output facilities provide a detailed presentation of computational results. The calculation itself is fully automated and based on robust numerical procedures.

1) **Proposed Work Analysis by using PLAXIS:**

The geometry of the sample is meshed in PLAXIS, open source software for finite element mesh generator. The mesh is generated with respect to second order shape function due to coupled flow analysis. Single element is modeled for all drained test simulations. The generated mesh file is incorporated in PALXIS as an input geometry file. Due to axial symmetry on the left edge and confining ring on the right edge (cell pressure in case of triaxial test), lateral movement is restricted on these boundaries.

Frictional effect from confining ring is not considered in simulation due to limited scope of this software. Hydraulic pressure is set to zero on the upper boundary to allow vertical drainage. Vertical displacement is restrained on the lower edge.

**E. Geogrid**

Geosynthetics is the term used to describe a range of generally polymeric products used to solve civil engineering problems. The term is generally regarded to encompass eight main product categories: geotextiles, geogrids, geonets, geomembranes, geosynthetic clay liners, geofoam, geocells (cellular confinement) and geocomposites. The polymeric nature of the products makes them suitable for use in the ground where high levels of durability are required. Properly formulated, however, they can also be used in exposed applications. Geosynthetics are available in a wide range of forms and materials, each to suit a slightly different end use. These products have a wide range of applications and are currently used in many civil, geotechnical, transportation, geo-environmental, hydraulic, and private development applications including roads, airfields, railroads, and embankments, retaining structures, reservoirs, canals, dams, erosion control, sediment-control, landfill liners, landfill covers, mining, aquaculture and agriculture.
Fig. 3: Model with using Geogrid

2) Loading Condition:
Initially an isotropic pressure of 5 mPa is assigned for total length of 12 m over entire concrete asphalt pavement as vehicle load, here total depth of pavement surface is about 300 mm considered with the following properties.

<table>
<thead>
<tr>
<th>Plate no.</th>
<th>Data set</th>
<th>Length [m]</th>
<th>Nodes</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Concrete Asphalt</td>
<td>12,000</td>
<td>703, 547, 521, 469, 449, 417, 301.</td>
</tr>
</tbody>
</table>

Table 2: Properties of Concrete Asphalt

<table>
<thead>
<tr>
<th>n o.</th>
<th>Identification</th>
<th>EA [kN/m]</th>
<th>EI [kNm²/m]</th>
<th>w [kN/m]</th>
<th>v</th>
<th>Mp [kNm/m]</th>
<th>Np [kN/m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Concrete</td>
<td>1.58 E11</td>
<td>1.179 E9</td>
<td>1.80</td>
<td>0</td>
<td>0.15  E15</td>
<td>1E15 5</td>
</tr>
</tbody>
</table>

Table 3: Properties of Concrete

3) Mesh Data:
Mesh shows stress variation and ultimate effect position of the concerning stress and deflection, also shows the failure region of the embankment.

4) Calculation Phases:
It shows the calculation for consolidation of 5 days to 200 days with effect of c-φ reduction of the embankment for the depth of ground water table/ phreatic level.

In this case we use staged construction for determination of failure pattern of the embankment.

<table>
<thead>
<tr>
<th>Phase</th>
<th>S. N.</th>
<th>Star t pha se</th>
<th>Calculatio n type</th>
<th>Load input</th>
<th>Fir st ste p</th>
<th>La st ste p</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial phase</td>
<td>0 0</td>
<td>-</td>
<td>-</td>
<td>0 0</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Embankment top 2m Consolation</td>
<td>1 0</td>
<td>Consolation analysis</td>
<td>Staged construction</td>
<td>1 11</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Consolation 200 Days</td>
<td>2 1</td>
<td>Consolation analysis</td>
<td>Staged construction</td>
<td>12 19</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Embankment More than 2 m 5 Days</td>
<td>3 2</td>
<td>Consolation analysis</td>
<td>Staged construction</td>
<td>20 21</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Consolation Till</td>
<td>4 3</td>
<td>Consolation</td>
<td>Minimum pore</td>
<td>22 30</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4: Construction Stage

5) Soil Reinforcing Mechanisms in Walls and Slopes:
Figure 5 shows a steep slope in a dry cohesionless soil with a face inclined at βs to the horizontal, where βs is greater than the internal angle of shearing resistance. Without the benefit of soil reinforcement the slope would collapse, however, by the incorporation of suitable soil reinforcement the slope may be rendered stable. Investigation of the basic reinforcing mechanisms reveals that the soil in the slope comprises two distinct zones. These are shown in Figure 5 as the active zone and the resistant zone. Without reinforcement the active zone is unstable and tends to move outwards and downwards with respect to the resistant zone. If soil reinforcement is installed across the active and resistant zones it can serve to stabilize the active zone.

Figure 5 shows a single layer of reinforcement with a length Lj embedded in the active zone and length Lj embedded in the resistant zone. A practical reinforcement layout would contain multiple layers of reinforcement; however, the single layer shown in Figure 5 is adequate to illustrate the basic mechanisms involved. The precise reinforcing mechanism will be affected by the properties of the reinforcement. Flexible reinforcements provide stability to a reinforced mass of soil by transferring destabilizing forces from the active zone to the resistant zone where they are safely absorbed. In this process purely axial tensile loads are absorbed, or dissipated, by flexible reinforcement. Provided reinforcement develops an adequate bond, and has adequate tensile stiffness, it will absorb tensile strains developed in the soil in the active zone. These tensile strains are transferred from the soil, to the reinforcement, through the mechanism of soil/reinforcement bond. The tensile strains developed in the reinforcement in the active zone give rise to a corresponding tensile force in the reinforcement in the active zone.

If the total length of the reinforcement is limited to Lj then the transfer of load from soil to reinforcement in the active zone would not prevent collapse of the active zone. To achieve this reinforcement extends a length Lj into the resistant zone. Provided the reinforcement has sufficient tensile strength to sustain tensile loads absorbed from the active zone it will shed these into the soil in the resistant zone. As in the active zone, load is transferred from reinforcement to soil by the mechanism of soil/reinforcement bond. The tensile load in the reinforcement over the length Lj is not constant but decreases towards the free end of the length.
Lej remote from the slope face, as load is shed into the soil. At the free end of the reinforcement in the resistant zone the tensile load in the reinforcement is zero. Flexible reinforcement is incorporated in fill during construction. Consequently, the layers of reinforcement are horizontal.

Flexible reinforcement is also inserted into cut sections during construction (in the form of soil nails) at inclinations close to the horizontal. This inclination is convenient since it coincides with the general inclination of the tensile strains developed in the soil in the active zone.

Fig. 5: Reinforcing Mechanisms in Walls and Slopes

IV. RESULTS AND DISCUSSION

Design of geotextile reinforced embankment is performed by determining the factor of safety with sequentially modified reinforcement until the target factor of safety is achieved. Generally, the failure pattern for geotextile reinforced embankment using 10 m length of geotextile and 100 kN/m of tensile strength. The result of geotextile reinforced embankment analysis using PLAXIS 2D by considering the factor of safety and displacement along the base of embankment is summarized. Factor of safety tends to increase with increases in tensile strength of geotextile reinforcement. In this study, the optimum tensile strength of geotextile reinforcement is 100kN/m by considering the factor of safety. The displacement was analyzed along the base of embankment in order to determine the optimum tensile strength of geotextile. Increasing the tensile strength of geotextiles decreased the displacement along the base of embankment. Since the displacement do not have the significantly effect by increasing the tensile strength of geotextile, this parameter could be neglected in determining the optimum tensile strength of geotextile in this study.

A. Test Results

The geometry of the Pavement embankment is meshed in Plaxis, as shown in Figure 6. The entire model as per the geometry is modeled and boundary conditions have provided to save computation time. A length of 26 m from embankment line has been considered to prevent boundary effects on material velocity and groundwater flow. Meshing is done using elements with second order shape functions due to incorporation of flow analysis. The mesh is locally refined in the embankment and the dry crust layer.

Fig. 6: Initial mesh Condition without applying load

Effective mesh geometry same for both condition i.e. without geogrid and with geogrid. In below figure density of mesh increases with depth due to self-weight and surcharge loading of materials.

Due to symmetrical boundary, lateral movement is restrained on the left and right boundary (center line of embankment). Both lateral and vertical movements are restrained on the bottom boundary. The unit weight of water is taken as 9.81 kN/m$^3$. Hence a pressure head of 104.40 kN/M$^2$ is set at the bottom boundary. The phreatic level is set on the ground level. A total of 7 post processing points have been chosen. Porepressures at depth of 2, 3.5, 5 and 12m below embankment are recorded along with settlements on centre line, 2, 4, 5m and 12 m below embankment. Prior to embankment loading, the in-situ stress equilibrium needs to be checked.

B. Results from Simulation

1) Deformation /Settlement:

Figure 7 shows the comparison between settlements along embankment center line with respect to time for both conditions i.e. with geogrid and without geogrid. The simulation shows a good fit with field measurement, however, the modified creep index has been changed. Initially the simulation is run for 5 days to 200 days to check the parameter's sensitivity on deformation and pore pressure values. The parameters µi permeability and dry crust stiffness, dominate the overall simulation. Due to large value of modified creep index, the initial deformation during embankment construction is more in normal embankment comparative to By Using Geogrid as shown in Figure 8.

Fig. 7: Extreme displacement/Settlement = 296.00 mm (Failure)
The settlement prediction from Finite Element model at 2 m, 4m and 9m right of embankment. A Heavy deviation observed is attributed to the stiffness of top dry crust layer. The rate of load distribution, with respect to area, from embankment to dry crust layer and deformation therewith relies heavily on the stiffness of this layer. Due to difficulty in testing dry crust samples, information from laboratory results can be insufficient and requires examination of several other data. From the above information it shows that embankment without using geogrid going to show failure after consolidation of 200 days. In other hand embankment using geogrid with calculated spacing (BS 8001-2002), is safe and showing acceptable settlement.

2) Effective Stress:
Stress is a measure of the average force per unit area of a surface within a deformable body on which internal forces act. The concept of effective stress is one of Karl Terzaghi’s most important contributions to soil mechanics. It is a measure of the stress on the soil skeleton (the collection of particles in contact with each other), and determines the ability of soil to resist shear stress. It cannot be measured in itself, but must be calculated from the difference between two parameters that can be measured or estimated with reasonable accuracy.

Effective stress ($\sigma'$) on a plane within a soil mass is the difference between total stress ($\sigma$) and pore water pressure ($u$): $\sigma' = \sigma - u$. Here in this case the pore water pressure ($u$) is zero.

Hence effective stress ($\sigma'$) is equal to total stress ($\sigma$) [$\sigma' = \sigma$]. On the basis of above criteria the effective stress increases with respect to the depth up to the maximum distribution of the load i.e. the effect of the stress is limited to the depth of the soil. Here the stress is increases up to the 42m and after it the effect of stress being constant. The results of effective stresses on to the wall are shown in Figure 9 without using geo-grid. The total effective principal stresses are about -142.00 kN/m². The stress is increases with the depth; negative sign shows the downward effect of stresses to the vertical direction. The effective stress on to the top is about -9.39 kN/m², which is same as without geogrid embankment and increases with the depth of the embankment, and highest stress on to the embankment is about -128.00 kN/m² located just at the second last layer of the pavement.

3) Active Pore Pressure (Same for Both Cases):

Hence active pore pressure ($u$) is equal to total stress ($\sigma$) [$u = \sigma$]. On the basis of above criteria the active pore pressure increases with respect to the depth up to the maximum distribution of the load i.e. the effect of the stress is limited to the depth of the soil. Here the stress is increases up to the 42m and after it the effect of stress being constant. The results of active pore pressures on to the wall are shown in Figure 11 without using geo-grid. The total active principal stresses are about -105 kN/m². The stress is increases with the depth; negative sign shows the downward effect of stresses to the vertical direction. The effective stress on to the top is about -9.39 kN/m² and increases with the depth of the embankment, and highest stress on to the embankment is about -128.00 kN/m² located just at the second last layer of the pavement. In second case with using Geo- grid at constant interval as per given in BS-8006-2002, the total effective principal stresses is going to reduce are about-128.00 kN/m². The stress is increases with the depth; negative sign shows the downward effect of stresses to the vertical direction. The effective stress on to the top is about -9.39 kN/m², which is same as without geogrid embankment and increases with the depth of the embankment, and highest stress on to the embankment is about -128.00 kN/m² located just at the second last layer of the pavement.
4) *Total Mean Stress*: The total stress ‘σ’ is equal to the overburden pressure or stress, which is made up of the weight of soil vertically above the plane, together with any forces acting on the soil surface (e.g. the weight of a structure). Total stress increases with increasing depth in proportion to the density of the overlying soil. The total vertical stress shows in Figure 14, which shows the total stress on to the Embankment is -187.37 kN/m² vertically. The stress is increases with the depth; negative sign shows the downward effect of stresses to the vertical direction.

The total vertical stress reduces after using Geo-Grid with calculating Spacing, shows in Figure 15, which shows the total stress on to the Embankment is -88.70 kN/m² vertically.

5) *Total mean stress at cross section at mid-section*: The cross section of the Embankment shows the effect of the total stress. The total mean stress is shown in Figure 16 the maximum stress is acts to the bottom; the graphical representation in Figure 17 shows the effect of stresses with respect to the depth.

6) *Displacements*: Displacement, in Newtonian mechanics, specifies the change in position of a point in reference to a previous position. In simple terms, it’s the difference between the initial position and the final position of an object. Figure 18 shows the total displacement pattern in soil. The direction of displacement is indicated by red arrows and the length of each arrow represents the magnitude of the displacement at the corresponding point. Figure 18 shows the extreme total displacement is of about 33.14 mm. without using geogrid layers, above figure shows that the maximum displacement occurs at the top of the facing. And bottom maximum displacement.

Figure 19 represents the vertical displacement of the soil. The vertical displacement reduced to is 2.84 mm. with the change in failure pattern of the embankment. The highest vertical displacement also occurs at the top portion it is observed that the most of the displacement occurs at the fill soil.
Displacement in natural soil or the foundation soil is found to be relatively small. Therefore, use of higher strength soils as back fill soil may reduce the displacement.

Fig. 18: Total Displacement (Utot) 33.14 mm

Fig. 19: Total Displacement (Utot) 2.84 mm using geo-grid

Analysis of Geogrid Behavior:
Figure 20 to 28 represents the distribution of total, horizontal and vertical displacements of geogrids respectively. The model results indicate the displacements generally increase toward the top of the Embankment, which is quite logical and identical with the actual field situation.

Figure-29 shows the Time vs. Displacement of different points selecting at same level for both models, which shows that at a constant loading, as the displacement is reached more quickly at model without reinforcement rather than using geo grid

Fig. 20: Total Displacement of 3 No. Geo grid 0.975 mm

Fig. 21: Total Axial Force of 3 No. Geo grid 2.23kN/m

Fig. 22: Total Horizontal Displacement (Ux) of 3 No. Geo grid -0.075 mm

Fig. 23: Total Displacement of No. 2 Geo-grid 0.136mm

Fig. 24: Total Axial Force of No. 2 Geo grid 7.0 kN/m

Fig. 25: Total Horizontal Displacement (Ux) of No. 2 Geo grid -0.155 mm

Fig. 26: Total Displacement of No. 1 Geo-grid 1.02 mm
V. DISCUSSIONS OF TEST RESULTS

In the present research work an analytical has been proposed to evaluate the performance of a road embankment with and without soil reinforcement, consist a natural soil deposit and well graded moorum as a backfill material, loose gravel as a drainage material, and SR2/UX-1700 geogrid layer as a reinforcing material were used. The analytical model was generated using a finite element software program PLAXIS.

The top of the wall is subjected to IRC class ‘A’ loading which shows the worth condition for the analysis.

As we can clearly observe from the plotted Figures and graph of the soil reinforce embankment and selected points on to it. All figures and graphs can be sub divide in to four categories. These are stresses, displacements, axial forces and strains.

Based on results obtained from the present investigation, the following conclusion can be made on the behaviour of the soil including geogrid as a reinforcing material.

1) The complete geometry of 13m height and 25m wide Road embankment can be constructed by applying horizontal and vertical fixities boundary conditions, and using uniformly graded Moorum as backfill material according to BS8002-1992, geogrid SR-2/UX 1700 as a reinforcing element by using software PLAXIS 8.2.

2) After generating the finite element mesh, software PLAXIS gives number of nodes for calculating the displacement and stress-strain values for the whole structure.

3) Calculation program shows that the Road Embankment is stable against total horizontal and vertical sliding failure, after using Geo-Grid and Actual Displacement d’= 20.76 mm is much less than the Failure Displacement Normal Model of d=264.5 mm, and gives the adequate F.O.S. required for the stability of the wall.

4) Geometry figures 3.3.2 shows the total 04 number. Reinforcing elements have been used in Road Embankment, with constant spacing as per BS 8002 which makes Road stable.

5) The calculation phases are done in seven steps in which four phases are calculating staged construction and remaining phase calculating the C-Phi Reduction with multiplier construction, and given results make structure is safe in all phases.

6) Stresses figures shows the maximum principal stresses is much less than after using geo-grid as a reinforcing material instead of road constructed without geo-grid.

VI. CONCLUSIONS & RECOMMENDATIONS

The finite element analysis performed in this study has indicated that geotextile reinforcement may be an effective method of improving the performance of embankments constructed over High Water Table Zone. The stabilizing effect of the geotextile was seen to increase as the geotextile modulus increased. The effect was greatest for shallower deposits. The effect of geotextile reinforcement was compared with alternative construction techniques which involved the use of light weight fill or berms alone and in conjunction with geotextile reinforcement. In particular, it was found that the combined use of geotextile reinforcement and light weight fill may be a very effective means of improving the performance of embankments over hilly terrain. In conclusion, the safely designing and construction of wall is due to the combination of the following factors:

- Improper installation of geogrid in the wall would lead to slacks and berms within it.
- Hence care should be taken for proper installation in the process of laying, installation and construction.
- The length of top layer of geogrid should always extend beyond the failure envelop line.
- This will assure the development of proper shear strength through frictional resistance between geogrid and soil.
- The backfill soil, if have less permeability and is poorly draining then proper drainage should be provided in facing of the wall or in far end of the wall.
drainage is not provided it will cause destruction to the structure.

- Sandy soil with cohesion shows less wall movements than cohesionless soil. Hence it is recommended that some cohesion can be added in backfill soil by some additives or by soil stabilization.

- Increased value of grid strength will end up with smaller displacement therefore geogrid with higher strength is recommended to use.

All the above steps make Road Embankment stable and allow going further in the actual construction of the embankment, on the basis of the outcome results. It has been proved that the analysis by the software of a proposed work before actual construction plays a vital role, and gives basic and important idea to completing the work economically and short duration.

Within the frame work of this thesis, an attempt has been made to use reinforcement geosynthetics as longitudinal reinforcement for embankment design. However, it is necessary to emphasize that the scope of this research work does not exhaust all aspects of reinforced earth structures with reinforcement geogrid.

REFERENCES


[16] N. Racana, R. gourves and M. grediac, blaise Pascal University, “mechanical behavior of soil reinforced by geocells”.


[18] PLAXIS 8.2 design manuals a “finite element code for rock and soil analysis”.


