**ADDIS ABABA UNIVERSITY**

**ADDIS ABABA INSTITUTE OF TECHNOLOGY**

**SCHOOL OF CIVIL AND ENVIRONMENTAL ENGINEERING**


# Finite Element Analysis of Geosynthetic Reinforced Pile-Supported (GRPS) Embankments

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**A Thesis in Railway Civil Engineering**


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Addis Ababa

**Advisor**

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A Thesis

Submitted in Partial Fulfillment of the Requirements for the Degree of Master of Science

The undersigned have examined the thesis entitled **‘‘Finite Element Analysis of Geosynthetic Reinforced Pile-Supported (GRPS) Embankments’’** presented by **HALELUYA MESFIN**, a candidate for the degree of **Master of Science** and hereby certify that it is worthy of acceptance.

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## UNDERTAKING

I certify that research work titled “Finite Element Analysis of Geosynthetic Reinforced Pile-Supported (GRPS) Embankments” is my own work. The work has not been presented elsewhere for assessment. Where material has been used from other sources it has been properly acknowledged / referred.

Haleluya Mesfin

## ABSTRACT

A large number of projects are now being carried out on soft ground due to the limitation of land availability for infrastructure projects in many countries. In addition, embankments are often required in many civil engineering construction projects involving infrastructure development to elevate the ground level. However, the undesirable characteristics associated with soft soils such as inadequate bearing capacity and large settlements/displacements, which can occur over a long time made embankment construction over soft ground is a challenging task. Several researches in the past have shown that Geosynthetic Reinforced Piled Embankment Systems (GRPES) are attractive solutions for such problems. Thus, a finite element model was carried out using the finite element software ABAQUS in order to investigate the behavior of GRPS embankments and improve the existing knowledge in the area. The role of pile supports and geosynthetic reinforcement was investigated using a two-dimensional numerical model with three different analysis cases i.e. when the soft soil is not improved - case 1, when a pile support is used in the soft soil - case 2, and when a pile support and geosynthetic reinforcement are used together - case 3. It is observed that there is 80% reduction in lateral displacement in case 2 than case 1 and 12% reduction in case 3 than case 2. The incorporation of piles to the soft soil has shown a 2.65% increase in vertical stress and a 96% reduction in settlements. Pile settlements were found to be low by about 83% compared to the corresponding subsoil. The incorporation of geosynthetic reinforcement while the piles are present showed a slight reduction in vertical displacement. In addition, the influence of increasing the number of geosynthetic layers caused a reduction in the lateral displacement.

**Keywords**

SOFT SOILS, GEOSYNTHETIC REINFORCED PILE-SUPPORTED (GRPS) EMBANKMENT, THREE DIFFERENT ANALYSIS CASES, FINITE ELEMENT MODEL, LATERAL DISPLACEMENT, STRESS, SETTLEMENT, GEOSYNTHETIC LAYERS

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## NOMENCLATURE

*The nomenclatures in this report include abbreviations and symbols used in equations, longer naming, and acronyms*

| | |
|---|---|
| GRPS | Geosynthetic Reinforced Pile-Support |
| GRPES | Geosynthetic Reinforced Piled Embankment Systems |
| FEA | Finite Element Analysis |
| AREMA | American Railway Engineering and Maintenance-of-Way Association |
| FEM | Finite Element Modeling |
| FE | Finite Element |
| CAE | Complete ABAQUS Environment |
| R | Shear resistance |
| 2D | Two Dimensional |
| 3D | Three Dimensional |
| M-C/MC | Mohr-Coulomb |
| NLGEOM | Non-Linear Geometry |
| $\tau$ | Shear strength |
| $\sigma$ | Normal stress |
| $\sigma_1$ | Maximum principal stress |
| $\sigma_1$ | Minimum principal stress |
| $\phi$ | Angle of internal friction |
| c | Cohesion/ Intercept of the failure envelope |
| $\sigma^{\circ}_c$ | Yield strength under axial compression |
| $\rho$ | Density |
| $\nu$ | Poison's ratio |

| | |
|---|---|
| $E$ | Elastic modulus |
| $E_{eq}$ | Equivalent elastic modulus |
| $A_w$ | Plan area of pile wall |
| $A_p$ | Area of the pile head |
| $E_p$ | Elastic modulus of the pile |
| $E_s$ | Elastic modulus of the soil |
| $\psi$ | Dilation angle |

# CHAPTER 1 INTRODUCTION

## 1.1 General

One of the key factors that play a pivotal role in nation's development is the presence of a reliable and efficient transportation system. Transportation is a critical ingredient that calls for the growth and development creating utilization of a nation's ability to utilize its natural resources, distribution of goods, integration of the manufacturing and agricultural sectors and it greatly supplies education, medical and other infrastructural facilities (Mustapha, 2011).

Laying special stress to the on hand traffic problems and development hindrances residing in the capital as well as on the national level, the Ethiopian Railway Corporation launched a railway network system construction. This results in supporting the fast growing economy of the country through constructing modern railways infrastructure which is cost effective and that transports bulk freight within short period of time and enhance public mobility.

The Ethiopian government planned to construct about 5000km railroad network investing billions of birr. The new railroad network is planned to link strategically selected locations, where railway stations are to be established. With this regard, due to the high investment cost this means of transportation is consuming due consideration should be given to lessen underutilization of the means in terms of safety, comfort, economy & efficiency. Therefore, studies should be carried out to see the sights of the major causes of inefficiency in problematic areas and appropriate solution must be provided.

The presence of weak soil in a construction site is one of the causes that diminish the proper utilization of structures. As a result of this, to make the site efficient ground improvement can be introduced. A journal by Dr. S. K. Tiwari and N. K. Kumawat states that the engineering techniques of ground improvement are removal and replacement, pre-compression, vertical drains, in-situ densification, grouting, stabilization using admixtures and reinforcement. The purpose of these techniques is to increase bearing capacity of soil and reduce the settlement to a considerable extent.

## 1.2 Background of the study

Different constructions brought into play sites that are fitting and not fitting. Since all constructions involve soil, ground improvement is vital when facing poor soil conditions. Ground improvement is the primary application of many constructions, permitting construction on soils by changing their characteristics. Ground improvement techniques have therefore become a major area of geotechnical engineering and a large number of treatment methods have been developed to suit a wide range of ground conditions and foundation problems (Serridge, 2006).

Engineers face many difficulties when designing and constructing embankments over soft ground. This is due to the undesirable characteristics of soft soils such as low bearing capacity, insufficient shear strength, high compressibility and excessive settlements. As a result, embankments will undergo large deformations and lateral movements, which can cause delays in construction and even failure of the embankment.

Out of several techniques available for improving the weak strata, granular piles/gravel piles/stone columns have been used to a large extent for several applications. This technique is a very efficient method of improving the strength parameters of soil like bearing capacity and reducing consolidation settlement. It offers a much economical and sustainable alternative to piling and deep foundation solutions. Ground improvement when implemented through stone column technique aids in a much stable solution to construction in weak cohesive soils (Mani & Nigee.K, 2013).

The vital issue during infrastructure construction is the application of ground improvement methods for unsuitable soils as discussed above to realize the nation’s target in generating ultimate benefit. Therefore, it is important to make use of a ground improvement technique that is paramount. It is in this light that this thesis is initiated to review, model and the effect of the geosynthetic reinforcement in a GRPS embankment is discussed in detail using three different analysis cases.

## 1.3 Statement of the problem

Though embankments are used frequently in civil engineering construction works whenever there is a need to elevate the ground surface, they cannot be constructed on soft soils due to compressibility and shear strength. Construction on soft ground has increased drastically as a result of the rapid growth of population and the lack of suitable land. For the reason that the undesirable characteristics of soft soils such as low bearing capacity, insufficient shear strength, high compressibility and excessive settlements the embankments will undergo large deformations, lateral movements, which can cause delays in construction and even failure of the embankment unless improved. Therefore, it is important to study one of the potential and that has been applied ground improvement method i.e. Geosynthetic Reinforced Pile Support.

## 1.4 Significance of the study

As globalization is a phenomenon which is here to stay, and which will deepen in extent and effect in the future, it is in the interest of the developing countries that research is undertaken on the application of the best technique to improve the weak strata. The situation in the Ethiopian construction is following an increasing trend in recent years. This research work is conducted to contribute for producing optimum results in the construction sector.

## 1.5 Objective of the study

### 1.5.1 General Objective

- The general objective of this study is to optimize the performance of a track structure by applying pile-support geosynthetic and reinforcement

### 1.5.2 Specific Objective

- To study the effect of constructing an embankment over soft soil without improving the soft strata (no pile supports or geosynthetic reinforcement), then supporting it with piles, and finally with pile supports and geosynthetic layer/s

- To assess the lateral displacement, stress, settlement, and influence of increasing geosynthetic layers of the above three cases for typical section of railway based on the result of Finite Element Analysis (FEA)

## 1.6 Scope and delimitation of the study

### 1.6.1 Scope of the study

The domestic construction industry has been using many types of ground improvement techniques. However, the research focuses on a technique which is applied to railway. The study mainly deals with ground improvement technique using geosynthetic reinforced pile support for an embankment section.

This study mainly dwells on improving the weak subgrade using the above mentioned technique to provide an increased bearing capacity and a considerable reduction in settlement. To this end, the main focus of the study include: the use of this technique, the weak soil strata to be improved, and the three cases mentioned on the objective.

### 1.6.2 Delimitation of the study

Since the modeling is done for a railway track structure, the research work is delimited to a railway project. Even though the study is delimited to railway projects, the reviewed literatures comprise the use of the technique in the railway, road or other projects that are used as appropriate in the analysis.

## 1.7 Methodology of the study

This research started with unstructured literature review during proposal preparation: to get an in-depth knowledge of the subject area attributed to the ground improvement method chosen; to assess the level of existing knowledge and identify gaps to select thesis topic; and identify suitable research methods for data collection and analysis. Conceptual and contextual literature reviews were followed once the main research work is started.

In pursuance of the presented objectives, the following methods were employed:

- A comprehensive literature review is made about constructing an embankment over a soft soil

- Models were developed with a design input of an embankment over a soft soil without any improvement, only with pile supports, and with geosynthetic reinforced pile support. Finite Element method of analysis is applied here and ABAQUS computer software was used for modeling and analysis.

- Analysis results trying to identify the optimum solution were reported from the model analysis.
- Interpretation of the results
- Finally conclusion and recommendation were given on all over the research results.

The data are from American Railway Engineering and Maintenance of Way Association (AREMA) manual and other acknowledged publications.

The entire procedure is summarized on the flow algorithm below (Figure 1-1):

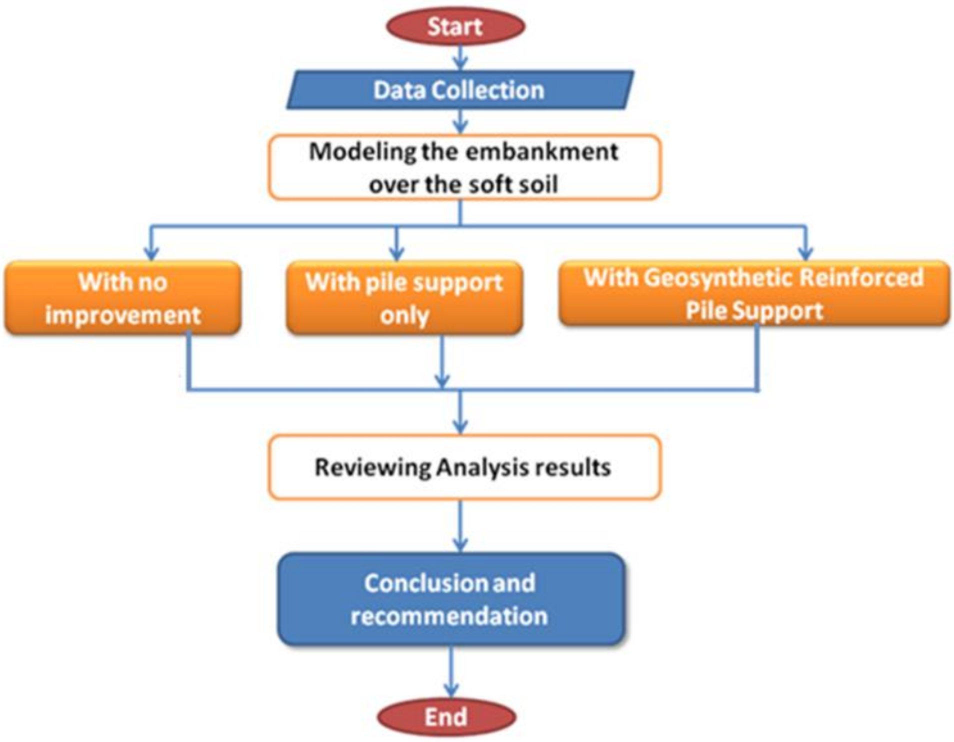



**Figure 1-1 Algorithm for conducting Numerical Analysis of Geosynthetic Reinforced Pile- Supported Embankment**

## 1.8 Organization of the Thesis

This study is divided into four chapters. A brief outline of this study is given below:

**Chapter 1** is the introductory as stated above which contains the general background, statement of the problem, significance and objectives of the study, scope and delimitation of the study, methodology, and here organization of the thesis.

**Chapter 2** is comprehensive literature review which begins with a general description of various track components and its functions. This is followed by the reviewing of embankments, track substructure performance improvement i.e. railway subgrade and ground improvement techniques in detail. Then embankments constructed over soft soil and Geosynthetic Reinforced Pile-Support is reviewed which includes gravel piles and significance of geosynthetic reinforcement. The next topic reviewed is all about numerical analysis focusing on the commercial software ABAQUS.

**Chapter 3** is finite element modeling and analysis and is divided into three subtopics; finite element modeling using ABAQUS, Numerical modeling and analysis of results.

Last chapter**, Chapter 4** summarizes the main findings of this research study; conclusions have been made from the present work based on the result of the FEM analysis. Finally, recommendations are made for further research that can be carried out in order to expand the knowledge base in the area of GRPS. This chapter is followed by a list of references.

# CHAPTER 2 LITERATURE REVIEW

## 2.1 General

As the lifeblood in nations' development the presence of a reliable and efficient transportation system plays a pivotal role. Raju (2001) in his paper presents that although railways are one of the oldest modes of transportation systems, modern railway organizations desire to increase the axle load and train speed for economic and environmental reasons. Railway track structures undertake a fundamental role within the transportation infrastructure of a country and contribute significantly in sustaining a healthy economy. With the current increase in demand for passenger train travel along with higher required axle loads for freight vehicles it is essential that a better understanding of how the track bed behavior influences track performance is sought. Kennedy (2011) in his dissertation states that this will help to reduce future track maintenance costs.

## 2.2 Track Components and Functions

The purpose of a railway structure is to provide safe and economical train transportation. This requires the track to serve as a stable guide way with appropriate vertical and horizontal alignment. To achieve this role, each component of the system must perform specific functions satisfactorily in response to traffic loads and environmental factors imposed on the system (Selig & Waters, Track Geotechnology and Substructure Management, 1994).

Traditionally rail tracks are laid on a bed of ballast material and the track components of ballasted track can be grouped into two main components; the superstructure and substructure. The superstructure consists of rails, fastening system and sleepers while the substructure consists of ballast, subballast and subgrade. Figure [2-1 (a) and (b)] show the components of a typical ballasted track from the longitudinal and transverse directions respectively. Within this thesis only ballasted track is considered. Moreover, although each of the superstructure components are important to the stability of the overall track structure, within this thesis as Justin Kennedy concentrated on the substructure components as they typically contribute the most to track deterioration (Kennedy, 2011).

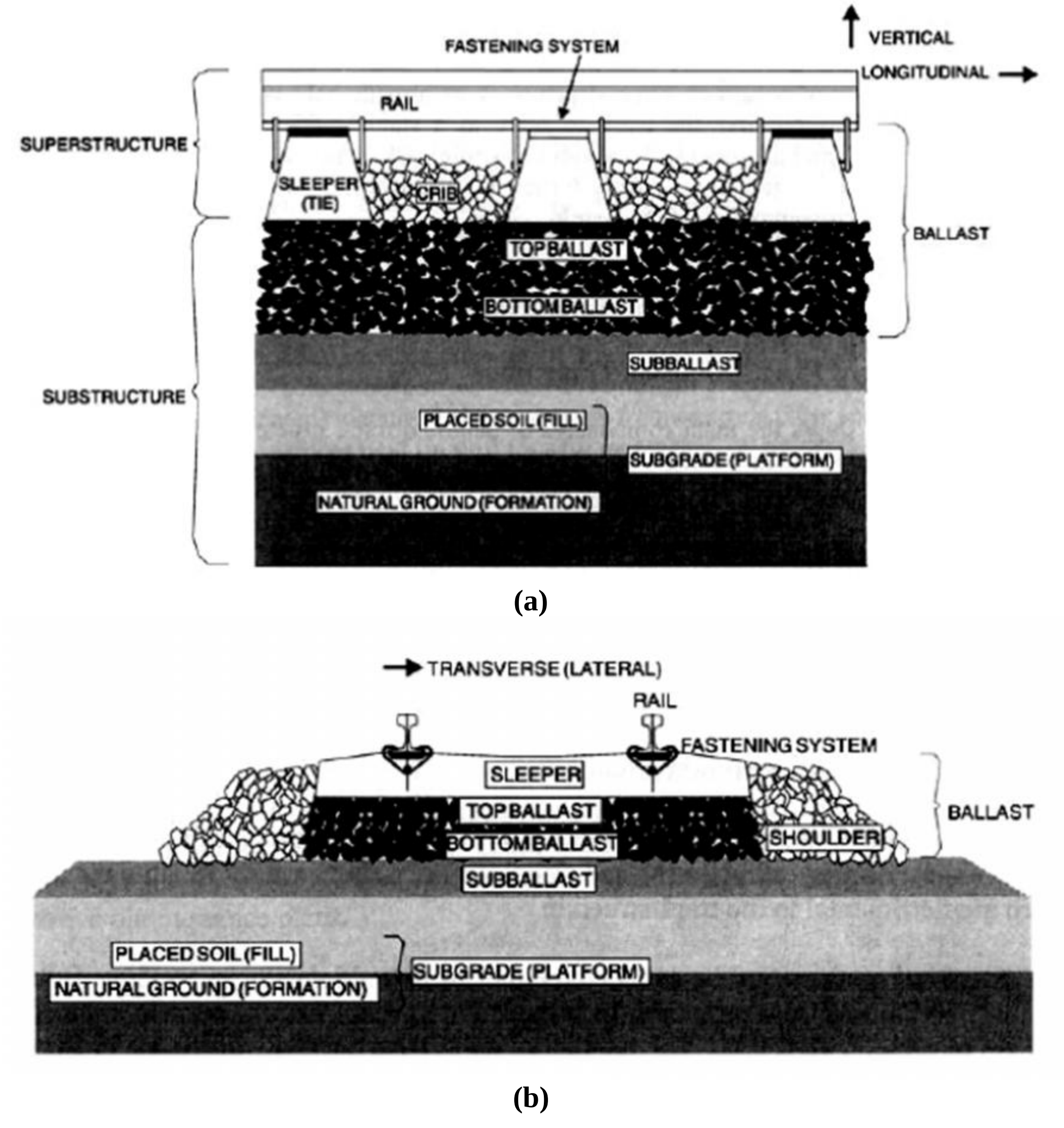



**Figure 2-1: (a) Longitudinal cross section of ballasted track structure (Selig and Waters, 1994), (b) Transverse cross section of ballasted track structure (Selig and Waters, 1994)**

Rails guide the train wheels and transfer the vertical and horizontal components of the wheel loads to the underlying spaced sleeper sections. Rail pads are used as a fastening system between the sleeper and rail to retain the rails against the sleepers and maintain the correct track gauge. In addition, the rail pads resist the vertical, lateral, longitudinal, and overturning movements of the rail by anchoring in the superstructure to the ballast (Selig & Waters, Track Geotechnology and Substructure Management, 1994).

The sleepers act to distribute the applied load over the ballast and are generally either wooden (timber) or pre-stressed/reinforced concrete.

Ballast is a crushed granular material placed as the top layer of the substructure in which the sleepers are embedded. Crib ballast is placed between the sleepers and in the shoulders beyond the sleeper ends. A wide range of ballast materials can be found on the tracks, such as granite, basalt, limestone, slag and gravel. The primary purpose of the ballast layer is to distribute the applied loads from the sleepers down to the underlying soil layers and protect the subgrade from high stresses. The voids in the ballast layer provide essential drainage of water falling onto the track and also allow the maintenance requirement of rearranging ballast particles to adjust track geometry (Selig & Waters, Track Geotechnology and Substructure Management, 1994).

Subballast is the granular layer of material separating the ballast and the subgrade and generally, subballast materials consist of broadly graded sand-gravel mixtures or broadly graded crushed natural aggregates or slags. The subballast layer further reduces the stress at the bottom of the ballast layer to a tolerable level for the top of the subgrade, offering a cheaper option to the otherwise thicker ballast layer. Another important function of the subballast layer is to act as a separator and prevent interpenetration between the subgrade and the ballast. By acting as a separator the subballast layer prevents the upward migration of fine material emanating from the subgrade, the attrition of subgrade by ballast and can provide drainage of water either flowing to the subgrade form the ballast or vice versa.

The subgrade is the foundation for the track structure and it can either consist of the existing natural soil, which is most likely to be a fine grained soil with silt and clay components, or placed fill. The main function of the subgrade is to provide a stable foundation for the track structure and therefore the subgrade has a significant influence on track performance and maintenance. The strength and stiffness of the subgrade ultimately dictates the amount of load that can be applied to the track and consequently controls the required depth of overlying granular material (Kennedy, 2011).

## 2.3 Embankment

Embankment is a very important part of every road and railway structure. It has to support the road pavement or railway structure and shall further distribute the forces, applied onto the pavement or railway structure, over the subsoil without exhibiting unacceptably large deformations. When constructing an embankment one should take into account a great number of variable properties of both the construction materials to be applied and the subsoil.

The stability of the embankment determines the performance of the overlying road pavement or railway structure. As the embankment consists of soil, in road and railway engineering soil is an essential construction material.

The stability of the embankment is influenced by many factors. Some of these factors have to be accepted and the structural design of the road pavement or the railway structure has to include these factors. However, the negative effects of other factors can be limited through a solid structural design and an adequate construction of the road or railway.

The construction of an embankment on weak subsoil always leads to settlements. The magnitude of these settlements depends on the magnitude of the loading, i.e. the deadweight of the embankment, and the deformation characteristics of the subsoil. As both the magnitude of the loading and the deformation characteristics of the subsoil usually exhibit some variation, the settlements of the subsoil nearly always are non-uniform. Non-uniform settlements (Figure 2-2) result in unevenness of the railway and therefore they have a negative effect on the driving comfort and even on the traffic safety.

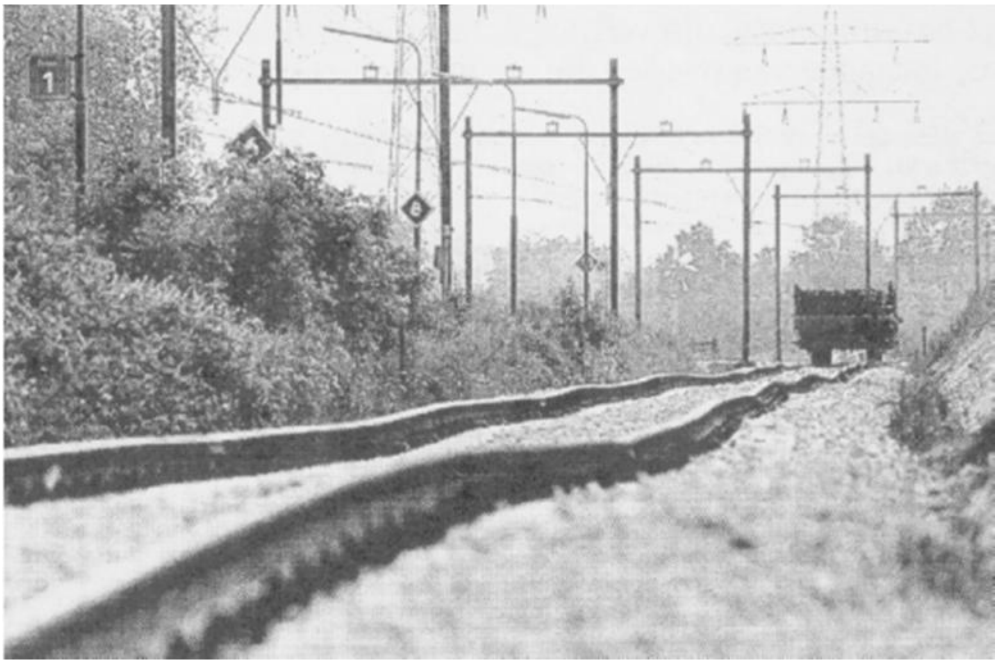


**Figure 2-2 Deterioration of a railway track**

The reason for the occurrence of settlements is clear. The extra deadweight load of the embankment must be borne by the grain skeleton of the subsoil, so by the normal stresses and shear stresses that develop in the contact points between the grains. When the shear stresses become too great a reorientation of the grain skeleton takes place, in such a way that the volume of the pores decreases and the number of contact points between the grains thus increases to such an extent that equilibrium is obtained. Settlements therefore are the consequence of a decrease of the pore content in the subsoil. These settlements are called the primary settlements.

Except primary settlements also secondary settlements occur in the subsoil. These secondary settlements result from creep of the grains themselves due to the extra deadweight load of the embankment. These secondary settlements are also called secular effect. In this case the development of the settlement with time is shown in Figure 2-3. (Molenaar & Houben, 2003).

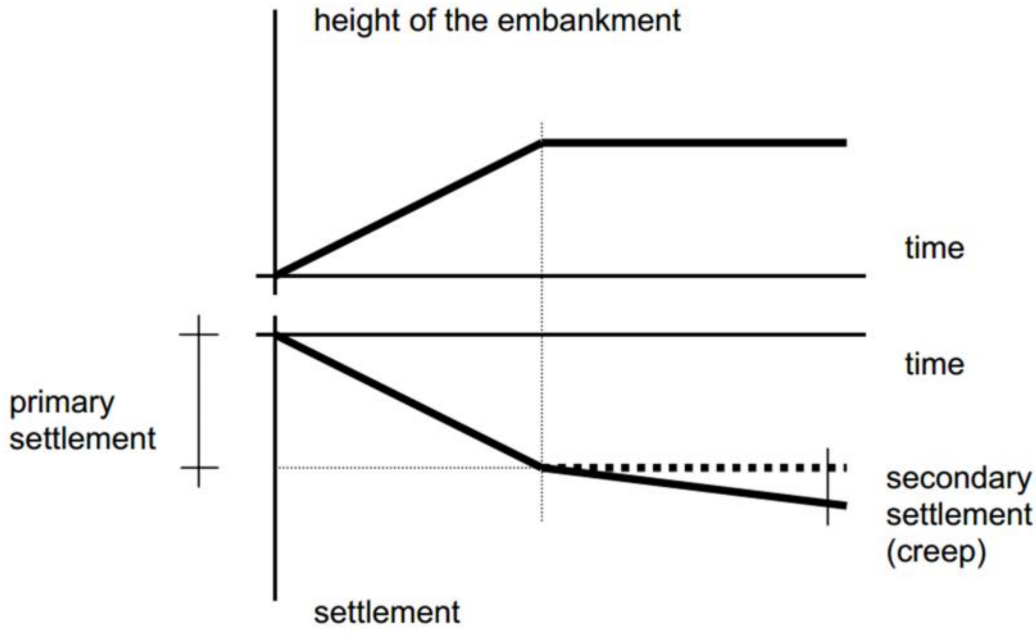



**Figure 2-3 Subsoil settlements with time in the absence of water**

## 2.4 Track Loading

The loads imposed on the track structure can be classified as either mechanical or thermal and they are applied to the track structure in the form of repeated vertical, lateral and longitudinal forces resulting from traffic and changing temperatures. Lateral and longitudinal forces are more complex and harder to predict than vertical forces, which act perpendicular to the plane of the rails (Selig & Waters, Track Geotechnology and Substructure Management, 1994).

## 2.5 Track substructure performance improvement

The strength and stiffness of the subgrade generally dictates the behavior of the track substructure under repeated loading and defines the settlement contributions from each layer. For a weak subgrade layer the subgrade will be the main source of settlement and the general solution to stabilize the track is to increase the depth of the overlying granular layers to reduce the subgrade stresses and reach acceptable track geometry. However, if the subgrade is too weak or has too low a stiffness that the track with any depth of granular material will not meet the minimum required track geometry under a particular load, then improvement of the subgrade is required. In addition, subgrade improvement may also be required when the train speed on a particular line approaches a threshold value for the track, namely the track 'critical velocity' (Banimahd, 2008). This phenomenon is associated with low velocity surface wave propagation. Subgrade improvement methods include modifying the subgrade properties in-situ (grouting, lime slurry stabilization, electrical treatment), modifying the subgrade properties by reconstruction and replacement (compaction, admixture stabilization), strengthening with asphalt concrete, and slip stabilization (drainage, retaining structure). For a relatively firm subgrade the ballast layer will be the main source of the settlement, although the depth of ballast is still dictated by the strength and stiffness of the subgrade (Kennedy, 2011).

## 2.6 Railway subgrade

The subgrade, which is the foundation of the track structure, highly variable in composition and behavior, and often comprising local natural soils, must function appropriately by remaining stable under traffic loads regardless of its makeup. Since it is hidden from view and difficult to access or modify after construction, it should be given ample attention. And knowing the failure modes of the subgrade helps in producing a suitable support for the track structure (Selig, Railway Subgrade Engineering, 2014).

### 2.6.1 Failure modes

Dr. Ernest T. Selig (2014) summarizes the most significant forms of failures, their causes and features as follows:

**Table 2-1 Subgrade failure modes and causes**

| Type | Causes | Features |
|---|---|---|
| Massive shear failure | -weight of train and track structure | -high embankment and cut slopes |
| | -inadequate soil strength | -(usu.) increase in water content |
| Progressive shear failure | -repeated overstressing | -squeezing near surface |
| | -fine-grained soils | -heaves in crib and/or shoulder |
| | -high water content | -depression under ties |
| Excessive plastic deformation | -repeated loading | -differential subgrade settlement |
| | -soft or loose soils | -ballast pockets |
| Excessive shrinkage and swelling | -highly plastic soils | -rough track surface |
| | -changing moisture content | |
| Attrition with mud pumping | -repeated loading | -muddy ballast |
| | -contact between ballast and subgrade | -inadequate ballast |
| | -clay rich rocks or oils | |
| | -water presence | |
| Consolidation | -embankment weight | -increased static soil stress as from new construction |
| | -saturated fine grained soils | |
| Frost heave and thaw softening | -periodic freezing temperature | -occurs in winter/sprong |
| | -free water | -rough track surface |
| | -frost susceptible soils | |
| Liquefaction | -repeated loading | -large displacement |
| | -saturated silt and fine sand | -more severe with vibration |
| | | -can happen in subballast |
| Slope erosion | -running surface water and/or subsurface water | -soil washed or blown away |
| Soil collapse | -water transport of soil grains | -ground settlement |

## 2.7 Ground Improvement

A review on Soft Soil Stabilization using Stone Columns states that there are a number of methods available to improve ground conditions such as stone columns, jet grouting, compaction grouting, short pile, dynamic compaction, lime stabilization etc. Before using any of these methods, it is required to know the ground improvement in detail. Mokhtari et al defined ground improvement in simple words as “the process of enhancing the quality of soil” (Mokhtari & Kalantari, 2012).

Due to the advancement in geotechnical engineering it is possible for us to modify the poor foundation soils to the strength and compressibility characteristics to suit the foundation of our choice. Ground improvement, is the modification of existing site foundation soils to provide better performance under design and/or operational loading conditions. Ground improvement techniques are used increasingly for new projects to allow utilization of sites with poor subsurface conditions. Previously, these poor soils were considered as economically unjustifiable or technically not feasible and are often replaced with an engineered fill or location of the project is changed.

In short, ground improvement is executed to increase the bearing capacity, reduce the magnitude of settlements and the time in which it occurs, retard seepage, accelerate the rate at which drainage occurs, increase the stability of slopes, mitigation of liquefaction potential, etc (Hirkane, Gore, & Salunke, 2014).

## 2.8 Benefits/Objectives of Ground improvement Techniques

Since all constructions involve soil, a ground improvement is a necessity where the railway lines cross as part of the alignment where the existing soil deposits are unfavorable or poor (Raju, 2001).

A ground improvement brings up a track substructure as long as railway track system is concerned. According to Berrak Teymur various techniques for improving soil are used to:

- Reduce the settlement of structures
- Improve the shear strength of soil and thus increase the bearing capacity of shallow foundations
- Increase the factor of safety against possible slope failure of embankments and earth dams
- Reduce the shrinkage and swelling of soils

## 2.9 Classification of Ground improvement techniques

Hirkane et al (2014) classified the ground improvement techniques based on the nature of strata and purpose of improvement that can be adopted:

**A. For cohesive soils:**

- Vertical Drains
- Vacuum Dewatering
- Stone columns

- In-situ deep mixing

**B. For cohesionless soils:**

- Compaction piles
- Vibro-compaction
- Stone-columns
- Dynamic compaction
- Compaction by deep blasting
- Grouting

Hirkane et al (2014) also presented the final choice among the methods available will depend on overall economy in the total foundation cost.

Based on the methods applied Babu (2012) presented the classification of the techniques that are practical now days as follows in Table 2-2:

**Table 2-2: Ground improvement techniques based on methods applied**

| Ground improvement techniques | | |
|---|---|---|
| **Ground Reinforcement** | **Ground Improvement** | **Ground Treatment** |
| Stone Columns | Surface Compaction | Soil Cement |
| Soil Nails | Drainage/Surcharge | Lime Admixtures |
| Micropiles | Electro-osmosis | Flyash |
| Jet Grouting | Compaction grouting | Dewatering |
| Ground Anchors | Blasting | Heating/Freezing |
| Geosynthetics | Dynamic Compaction | Vitrification (use of heat to melt and then solidify harmful chemicals in a solid mass) |
| Fibers | | |
| Lime Columns | | |
| Vibro-Concrete Column | | |
| Mechanically Stabilized Earth | | |
| Biotechnical | | |

## 2.10 Embankment construction over soft soil

### 2.10.1 General

Various techniques have been used in practice in order to overcome these problems and improve the soft ground for embankment construction. Traditional ground improvement techniques are mostly consolidation based methods such as preloading, vertical drains, vacuum consolidation or staged construction. There are also various methods to reduce the load from the embankment using lightweight fill materials and to provide additional support by over excavating the existing soft soil and replacing it with suitable materials, changing the embankment geometry by reducing the slope of the embankment, grout injection, reinforcing the embankment with geogrids or providing additional support by adding column supports. All the above mentioned ground improvement methods have their advantages and disadvantages depending on the situation they are being used. For large embankments over deep soft clay deposits, removal of existing soft soil is not practical and use of a soil with lower density does not reduce the loads transferred to the soft ground. When shear strength and stiffness gain due to consolidation are unpredictable and availability of land is insufficient to change the embankment geometry, the most reliable and convenient solution among these techniques is the use of column supports to carry the embankment load. Column supports can be hard columns such as piles (Jenck et al. 2009; Han et al. 2012) or semi hard columns such as deep cement mixed columns (Huang and Han 2009) and stone columns (Deb et al. 2007). With pile-supports, a larger proportion of the embankment load will be transferred to the piles rather than the soft foundation soil reducing the stresses applied on the foundation soil layer substantially. As a result, the pore pressure increments and the settlements in the foundation soil layer can be reduced.  Pile supports are effective in difficult or extremely poor ground conditions such as landfills, Brownfield sites and dumps where engineering behavior of soils are not well known and extracting soil properties by means of routine laboratory tests is difficult. In these situations, since majority of the embankment load is transferred to the piles, detailed knowledge about mechanical properties of ground is not required (Ariyarathne, 2014).

Since pile casting can be an expensive process, this method may not be economical as the other ground improvement methods mentioned here. Therefore, the selection of this method is based on the time constraints of the project, the reliability as well as the soil properties. Since construction can start immediately after installing the piles, the time savings when using pile-supported embankments can outweigh the additional construction cost for time sensitive projects. So, this method is suitable for such soft soils and when the soil strength is reducing this method becomes more suitable (Ariyarathne, 2014).

### 2.10.2 Description of Geosynthetic Reinforced Pile-Supported Embankments

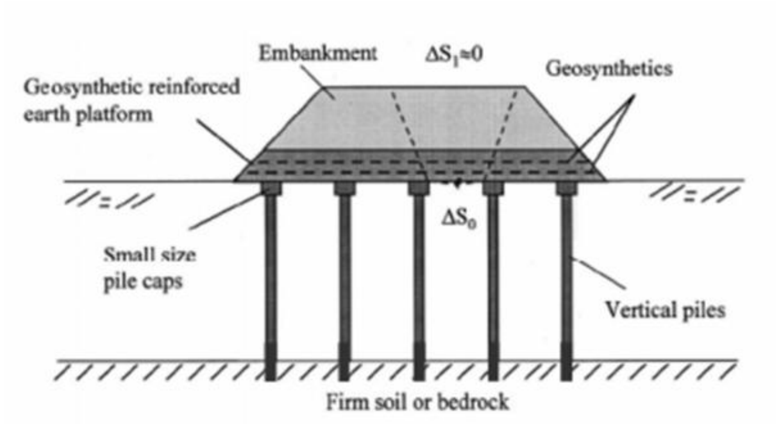
According to Satibi et al. (2007) piled embankment is an embankment, which is supported by piles embedded in the soft soil. Geosynthetic layers are often included for the embankment reinforcement. A piled embankment is constructed by installing piles with a certain grid formation in a soft soil up to a certain depth, which is generally reaching a competent stratum such as firm soil or bedrock. If geosynthetic reinforcement is included, the geosynthetic material is laid on top of a thin layer of embankment material. It is not usually laid directly on top of pile caps. After constructing the geosynthetic layers, the embankment fill is raised up to the required height. Finally, the construction such as railway or road pavement is built on top of the embankment.

#### *2.10.2.1 Soil*

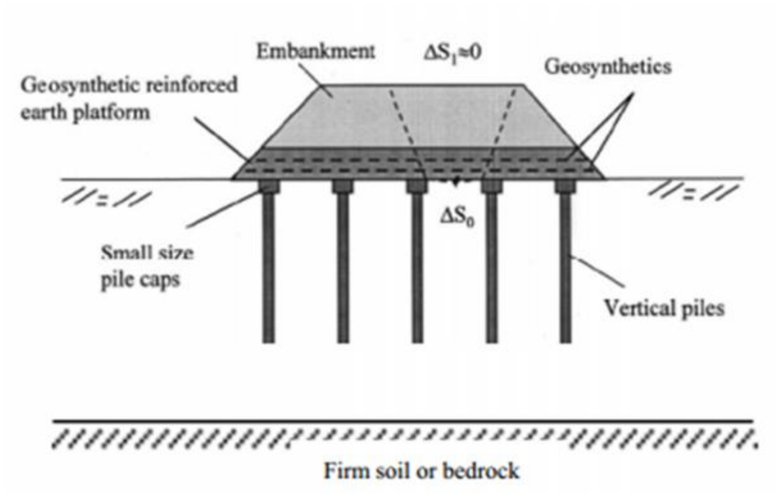
Satibi et al. (2007) also presents when an embankment needs to be built on soft soil, large soil deformation will take place if no measure is taken. Therefore, they suggest the application of a piled embankment as ground improvement when the ground condition at the construction site is soft clay since soft clay is typically very compressible and has a low strength.

#### *2.10.2.2 Piles*

The soft soil cannot take the external loads from the traffic and embankment without having large deformations. Hence, in a piled embankment, the loads are transferred to the much stiffer piles according to Satibi et al. 2007). The piles used for this purpose are generally prefabricated (driven) or cast in place displacement piles (jacked in or screwed piles). The piles are preferably embedded to a competent stratum (end bearing piles Figure 2-4 (a)). However, when the soft clay layer is thick, it is often that the piles cannot reach the competent stratum. These are called floating piles Figure 2-4 (b).




**(a)**




**(b)**

**Figure 2-4 Piles-supported embankment (a) on end bearing piles (b) on floating piles**

### *2.10.2.3 Embankment*

As stated by Satibi et al. (2007) to build infrastructure such as roads, a stiff layer or platform is needed. For this reason, embankments are built. This embankment needs to be stiff and strong. In order to reach this, it is required that the embankment must consist of good quality materials (sand and gravel or crushed stones) with wide grain size distribution and the internal friction angle should be higher than 30°.

### *2.10.2.4 Geosynthetics*

Satibi et al. (2007) discusses concerning the advantages of utilizing geosynthetic reinforcement for embankments on piles such as reducing the need of large pile caps and the need of raking piles. Based on the practical experience, the geosynthetic type used varies. The geosynthetics can have uni- or biaxial tension with the tensile strength varying between 20 and 1100 kN/$m^2$. And often up to 3 layers of geosynthetics are applied.

### 2.10.3 Mechanisms of Load Transfer

Design of pile-supported embankment in conjunction with geosynthetic reinforcements are commonly adopted in present construction practice. The embankments fill mass between the two consecutive piles has a tendency to move downwards due to the presence of soft soils under the fill. Shear resistance, R, in fill mass above the pile cap restrain the movement of fill mass to some extent. The development of shear resistance (R) reduces the pressure acting on the geosynthetic. However, it increases the load applied onto the piles. The load transfer mechanism from the fill mass on to the pile caps is known as "*Soil Arching Effects*" (Han and Gabr, 2002, cited by Rao (2006)). As summarized by McNulty (1965) and cited by Rao (2006), arching is the ability of the material to transfer load from one point to another in response to relative displacement between the locations. Geosynthetic reinforcement enhances the load transfer from the fill soil to piles, reducing the total and differential settlements. The reduction of differential settlements at the base of the embankment is reflected at the surface of the embankment. Installation of geosynthetic reinforcement increases the load transfer and reduces the area replacement ratio of the columns (piles) as observed by Russell and Pierpoint (1997), Han and Wayne (2000), (Han and Gabr, 2002). Test conducted by Terzaghi (1936) and McNulty (1965) affirm that the shear stress induced by soil arching increases with the displacement and fill thickness above the yielding soil portion (Han and Collin, 2005) cited by Rao (2006)).

Satibi (2007) presented the way of piled embankments work as described in Figure 2-5. The external load, for example from the traffic and the embankment above the soil arch is transferred to the piles via the soil arching mechanism. The embankment load below the soil arch will be barred by the geosynthetic and will be directed to the piles via geosynthetic tension. The piles transfer the load to the deeper and stiffer soil stratum. Thus, the soft soil experiences little force and therefore compaction because of the force are transferred through the geosynthetic and the piles.

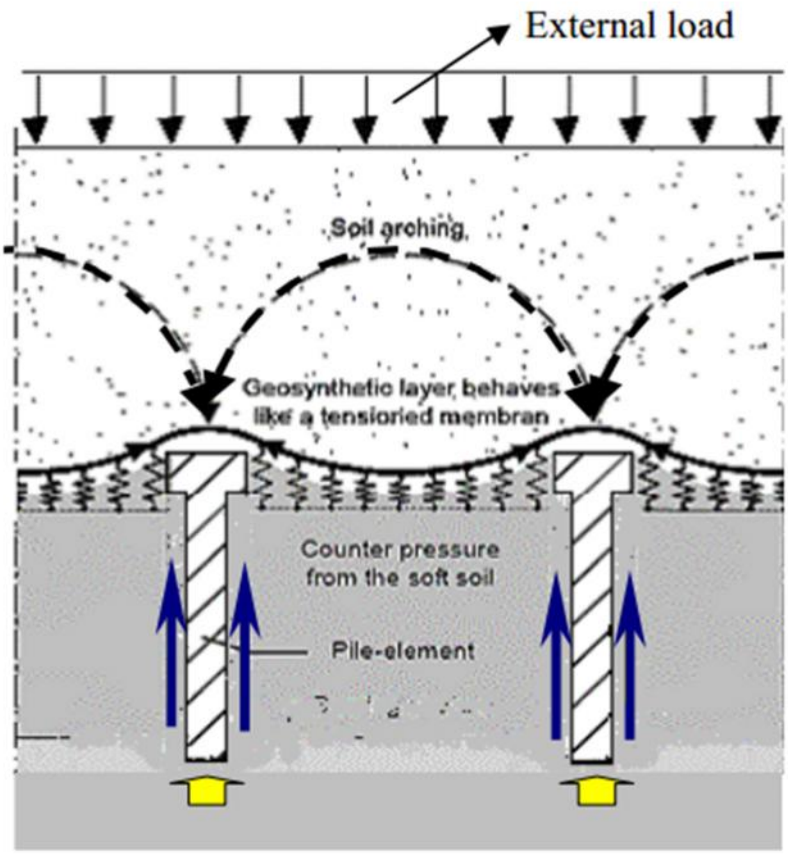



**Figure 2-5: The Idea of Piled Embankment**

## 2.11 Ground improvement using gravel pile

### 2.11.1 General

A proper competent stable ground can be attained, for a weak ground/soil, by employing ground improvement techniques. A paper by Tiwari and Kumawat (2014) addressed the provision of the techniques for conditions as mechanical properties are not adequate to bear the superimposed load of infrastructure to be built, swelling and shrinkage property more pronounced, collapsible soils, soft soils, organic soils and peaty soils, karst deposits with sinkhole formations, foundations on dumps and sanitary landfills, handling dredged materials for foundation beds, handling hazardous materials in contact with soils, using of old mine pits as site for proposed infrastructure.

Though it may not be immediately apparent, ground improvement methods have made considerable advances when a project site come across any of the above difficult conditions. The possible alternative solutions were avoiding the particular site which is not recommended, remove and replace the unsuitable soils or modify existing ground which is costly, enable cost effective foundation design, reduce the effects of contaminated soils, ensure sustainability in construction projects using ground improvement techniques (Tiwari & Kumawat, 2014).

### 2.11.2 Gravel/Granular Piles

Stone columns, also known as granular piles, consist of stone aggregates (generally 20 mm to 75 mm) compacted into a vertical hole (generally of diameters of 0.6 m to 1.0 m and depths of 15 m to 20 m) according to Jayalekshmi et al. (2015).

Krishna et al. (2006) presented granular piles/ stone columns/ geopiers are methods of ground improvement applied by partially substituting the in-situ soil to provide increased bearing capacity, significant reduction in settlement, free drainage, increase in liquefaction resistance, etc.

Granular piles/stone columns are constructed in soft soils by making circular holes and filling them with granular materials such as natural stone, sand or stone chips. When about 10 to 35% weak soil is removed and replaced with granular material in the form of piles, the load carrying capacity of the ground increases and settlement decreases significantly and the ground becomes useable to support the structure (Rakesh & P.K., 2012).

### 2.11.3 Construction of Granular Pile

The improvement of a soft soil with granular pile can be accomplished using various techniques that involve excavation, replacement and compaction. Rao (1982) and Ranjan and Rao (1983) developed a simple method, particularly useful in developing countries, which is technically viable and uses indigenously developed equipment. A spiral auger is used to make the borehole utilizing manual labor. After reaching the desired depth, the borehole is thoroughly cleaned and the stone aggregate is placed in the borehole in layers of 300 – 500 mm followed by sand layer of 50 – 100 mm. A cast iron hammer weighing 125 kg and diameter less than the diameter of the borehole, operated by a power winch having a fall of 750 mm is used to compact the sand/stone aggregate layer. During the course of compaction, the sand fills the voids of the stone aggregates followed by the lateral and downward displacement of the charged material till full compaction of the surrounding soil is reached.

The construction technique is economical since it doesn't require skilled labor. And it can be applied in developing countries where man power is cheaply available (Rakesh & P.K., 2012).

According to Hwang and Tu (2002) the vibrator during construction first generates a lateral vibratory force to liquefy the saturated sandy soils and then penetrates down to the design depth under its self-weight and external hydraulic pressure or compressed air pressure. When the vibrator is pulled up by around 0.5-1.0m, the gravel material is poured into a vertical transport pipe using a lifting bucket which raises the gravel on the ground to a hopper mounted on the top of pipe. The gravel then goes through the pipe, and the space between the vibrator and the bottom of the construction hole is filled with gravel. When the space is completely filled, the vibrator squeezes the surrounding ground downward and laterally so as to form a compact gravel pile. The vibrator is pulled up when the vibratory compaction pressure reaches a limiting value or the vibro-compaction time reaches 2 minutes. The gravel pile is completed by repeating the steps in Figure 2-7 of pulling up the vibrator, feeding the gravel and squeezing the surrounding soil in between the design depth to the ground surface.

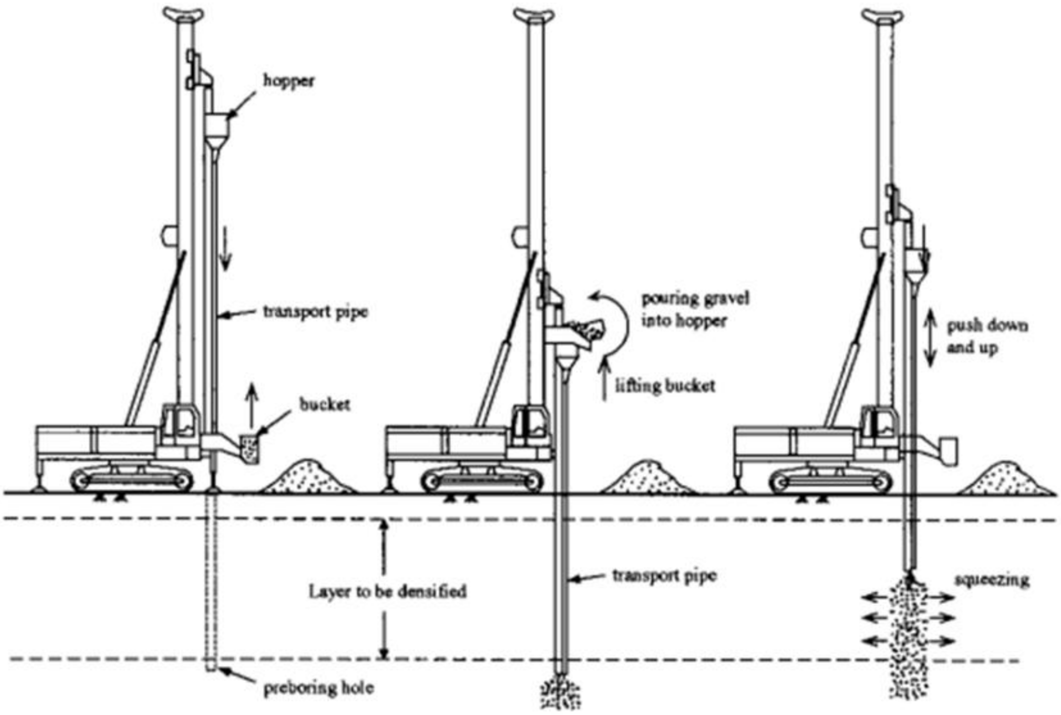



**Figure 2-6: construction sequences of gravel pile with a bottom feeder design**

### 2.11.4 Methods for granular pile construction

Various methods for installation of granular piles have been used all over the world depending on their proven applicability and availability of equipment in the locality (Madhav, 1995).

- **Vibro-Compaction Method:** is used to improve the density of cohesion less, granular soils using a vibroflot which sinks in the ground under its own weight and with the assistance of water and vibration (Baumann and Bauer, 1974; Engelhardt and Kirsch, 1977). After reaching the predetermined depth, the vibroflot is then withdrawn gradually from the ground with subsequent addition of granular backfill thereby causing compaction. The process is repeated in stages, as shown in Figure 2-8, forming a compacted column of granular piles.

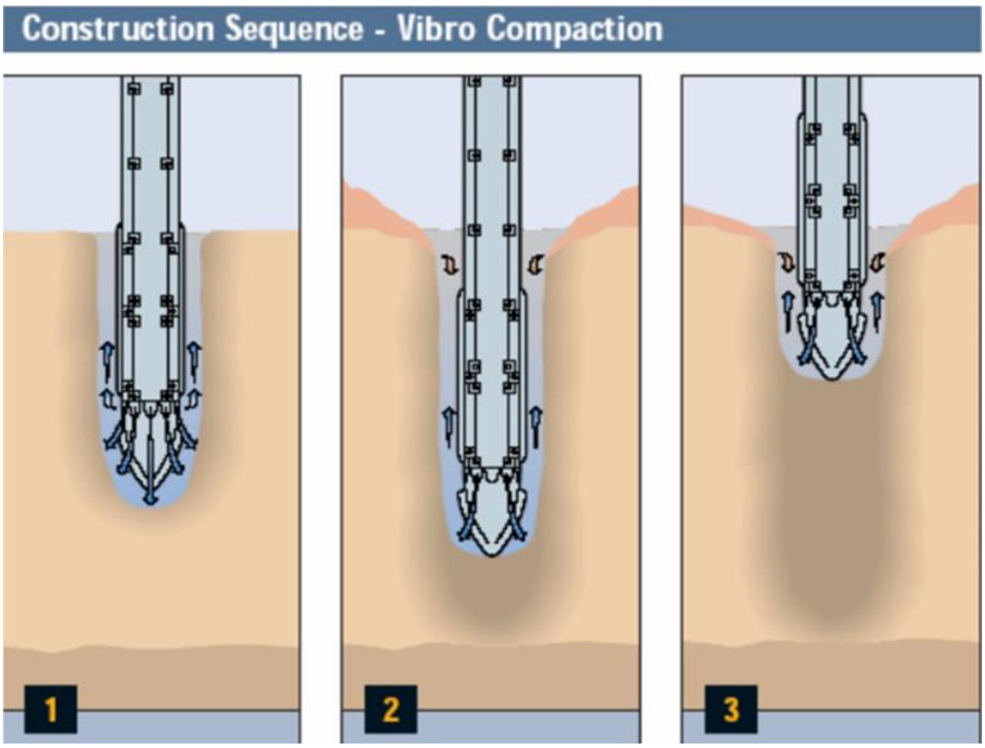



**Figure 2-7: Vibro-Compaction Method (Baumann and Bauer, 1974)**

***Stage 1:*** At full water pressure, the vibrator penetrates to design depth and is surged up and down as necessary to agitate sand, remove fines and form an annular gap around the vibrator.

***Stage 2:*** Once at depth, the water pressure is reduced and with the vibrator remaining in the ground, sand infill is added from ground level and compacted at the base of the vibrator.

***Stage 3:*** When the required compaction resistance is achieved, the vibrator is raised and more sand infill added and compacted as before. This procedure is repeated until compaction point is built up to ground level.

- **Vibro-Compozer Method:** is used for stabilizing soft clays in the presence of high ground water level (Aboshi et al. 1979; Aboshi and Suematsu, 1985; Barksdale, 1981). The resulting pile is usually termed as sand compaction pile. These are constructed by driving the casing pipe to the desired depth using a heavy vertical vibratory hammer located at the top of the pipe. The casing is filled with specified volume of sand and the casing is then extracted and partially redriven using the vibratory hammer starting from the bottom. The process is repeated until a fully penetrating compacted granular pile is constructed.

- **Cased Borehole Method:** In this method the piles are constructed by ramming granular materials in prebored holes instages using a heavy falling weight (usually of 15 to 20 kN) from a height of 1.0 to 1.5 m. The method is a good substitute for vibrator compaction considering its low cost. However, disturbance and subsequent remolding by the ramming operation may limit its applicability to sensitive soils.

- **Vibro-Replacement Method:** This method is used to improve cohesive soils. The equipment used is similar to that for vibro compaction. The method can be carried out either with the wet or dry process. In the wet process, a hole is formed in the ground by jetting a vibroflot down to the desired depth with water. When the vibroflot is withdrawn, it leaves a borehole of greater diameter than the vibrator. The uncased borehole is flushed out by and filled in stages with imported gravel. The main difference between the wet process and dry process is the absence of jetting water during the initial formation of the hole in the former.

### 2.11.5 Function of Gravel/Granular Piles

Granular piles provide primary functions of reinforcement and drainage, and in addition, improve the strength and deformation properties of soft soil in post installation and reconsolidation phase. They increase the unit weight by replacement, drain rapidly the excess pore pressures generated, act as strong and stiff elements and carry higher shear stresses. They are installed in wide variety of soils, ranging from loose sands to soft clays and organic soils. Granular piles are cost effective; more so, in end bearing conditions (Madhav, 1995).

Improvement of a soft soil by stone columns is due to three factors. The first factor is inclusion of a stiffer column material (such as crushed stones, gravel, and so on...) in the soft soil. The second factor is the densification of the soft soil surrounding the stone columns during the installation of stone column. The third factor is the vertical drainage provided by stone columns (Guetif et al. 2007). Therefore, the insertion of stone columns into weak soils is not just a replacement operation and stone column can change both the material properties and the state of stresses in the treated soil mass (Reihani & dehghani, 2014).

#### *2.11.5.1 Benefits of Pile-Supported Embankments*

- There is no need to wait long periods for consolidation process to complete before starting the embankment construction.
- The embankment can be constructed in a single stage.
- Suitable for fast-tracked construction when there are time constraints for the completion of a project
- The pressure applied on the subsoil can be significantly reduced and as a result significant reductions in total and differential settlements can be achieved within the embankment fill.
- Much more reliable than the other soft ground improvement techniques, when ground conditions are unpredictable

- Less maintenance is required

## 2.12 Significance of Geosynthetic Reinforcement in Embankment Construction

Embankments constructed on soft soils are reinforced using geosynthetic layers to improve the stability (Humphrey & Holtz, 1987). Single or multiple layers of geosynthetic reinforcement is installed in pile-supported embankment systems in order to increase the load transfer to the piles and reduce the required area replacement ratio of piles (Lawson, 1992).Geosynthetics are generally produced using polymer materials. These materials usually exhibit elastic-viscoplastic behavior which can depend on load, loading rate, time and temperature. Geosynthetics can be geotextiles, geogrids or geomembranes.

Geotextiles can be described as textiles made out of synthetic fibers. These synthetic fibers are either woven using standard weaving machinery, or are matted together in a random manner. Former are referred to as woven geotextiles and the latter is called nonwoven geotextiles. Geotextiles are used in practice for separation, reinforcement, filtration and drainage. Geogrids are exclusively made as reinforcement materials. They are not tightly woven or non-woven textile fabrics like geotextiles. They are plastics which are formed into an open grid-like configuration. Geomembranes are impervious thin sheets which are manufactured out of rubber or plastic material. They are mainly used for linings and covers of liquid or solid storage facilities (Koerner, 1994). Pile-supported embankments can be reinforced using both geotextiles and geogrids.

While providing a support for an embankment over a soft soil in addition to the pile support used geosynthetic material is also applied in order to achieve, according to Priyanath Udayanga Ariyarathne, the following benefits (Ariyarathne, 2014):

- Piles do not have to be closely spaced
- Pile caps can be made smaller
- No need of battered piles at the edges of the embankment
- Lateral soil movements are reduced
- Less land usage

## 2.13 Design Methods

The design of a GRPS embankment involves calculating the amount of embankment load transferred to piles through soil arching and the portions carried by the geosynthetic layer and foundation soil. It is conservative to assume that the foundation soil does not carry any load which is the case for most of the available design methods at present. With this assumption, the tension in the geosynthetic reinforcement is calculated based on an assumed deformed shape.

### 2.13.1 Parameters Used in the Design and Analysis of GRPS

Some of the key variables and parameters used in the design and analysis of a GRPS embankment are presented by (Sloan, 2011) and are summarized below:

- Performance parameters: The allowable total and differential settlements, factors of safety for stability and lateral spreading, and geosynthetic reduction factors of safety for damage, degradation and creep.

- Subsurface parameters: Properties of the foundation soil such as the unit weight, Poisson's ratio, lateral earth pressure coefficient, compressibility and the coefficient of friction between soil and piles, and depth to the bearing layer for pile design and the depth to the ground water table.

- Loading Conditions: Traffic load, embankment load and loads from other structures if applicable.

- Material parameters: Properties of piles (compressive strength, flexural strength, stiffness, allowable load), properties of the load transfer platform (unit weight, friction angle, elastic modulus and Poisson's ratio), properties of the embankment fill (unit weight, friction angle, elastic modulus, lateral earth pressure coefficient and Poisson's ratio) and properties of the geosynthetic layers (ultimate tensile strength, stiffness, allowable design strength and the interaction coefficient between soil and geosynthetic)

- Geometric parameters: Pile spacing, pile diameter, layout of piles (square, rectangular or triangular) as shown in Figure 2-9, pile tributary area, height of the embankment, side slope inclination of the embankment, number of geosynthetic layers and pile cap properties if used (width or diameter, thickness and shape)

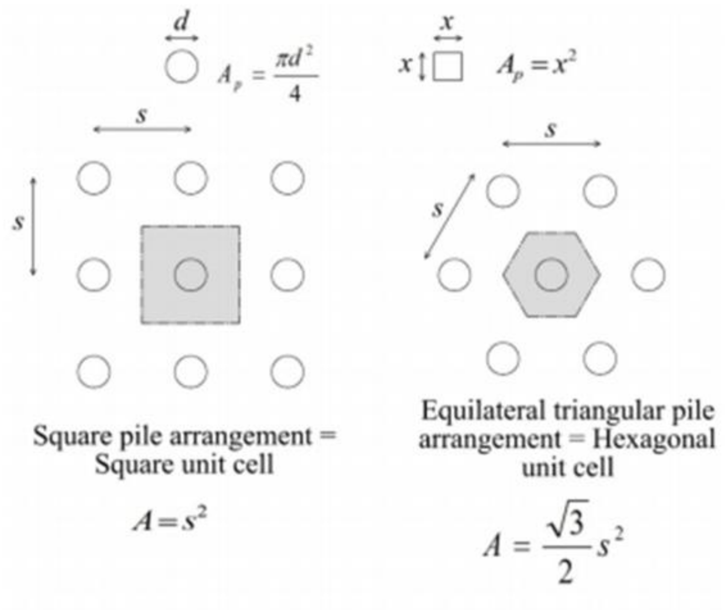



**Figure 2-8: Unit cells for square and equilateral triangular pile arrangements (Sloan, 2011)**

## 2.14 Numerical Analysis

### 2.14.1 General

Numerical analysis is the area of mathematics and computer science that creates, analyzes, and implements algorithms for solving numerically the problems of continuous mathematics. Such problems originate generally from real-world applications of algebra, geometry, and calculus, and they involve variables which vary continuously. These problems occur throughout the natural sciences, social sciences, medicine, engineering, and business. Beginning in the 1940's, the growth in power and availability of digital computers has led to an increasing use of realistic mathematical models in science, medicine, engineering, and business; and numerical analysis of increasing sophistication has been needed to solve these more accurate and complex mathematical models of the world. The formal academic area of numerical analysis varies from highly theoretical mathematical studies to computer science issues involving the effects of computer hardware and software on the implementation of specific algorithms (Atkinson, 2007).

Because piled embankments are complex soil-structure interaction problems, numerical methods are considered a powerful tool to reduce the uncertainties and have been used for the piled embankment design.

## 2.15 ABAQUS

### 2.15.1 General

Khabbazian et al. (2010) presented ABAQUS as high performance, quality and ability software for finite element analysis (FEA) to solve more kinds of challenging simulations than any other software.

ABAQUS/CAE (Complete ABAQUS Environment) is used to create the model graphically, run the analysis, and then view the results. ABAQUS/CAE contains nine modules that divide the modeling tasks into functional units. These modules are: Part, Property, Assembly, Step, Interaction, Load, Mesh, Job, and Visualization (Mazursky, 2006).

# CHAPTER 3 FINITE ELEMENT MODELING

## 3.1 Finite Element Modeling Using ABAQUS

### 3.1.1 General

The **Finite Element Method** (**FEM**) is a numerical technique for finding approximate solutions to boundary value problems for partial differential equations. It is also referred to as **Finite Element Analysis** (**FEA**). FEM subdivides a large problem into smaller, simpler, parts, called finite elements. The simple equations that model these finite elements are then assembled into a larger system of equations that models the entire problem. FEM then uses variational methods (mathematical analysis that deals with maximizing or minimizing functional i.e. a function from a vector space into its underlying scalar field, or a set of functions of the real numbers) from the calculus of variations to approximate a solution by minimizing an associated error function.

The subdivision of a whole domain into simpler parts has several advantages:

- Accurate representation of complex geometry
- Inclusion of dissimilar material properties
- Easy representation of the total solution
- Capture of local effects

FEM is best understood from its practical application, known as **Finite Element Analysis (FEA)**. FEA as applied in engineering is a computational tool for performing engineering analysis. It includes the use of mesh generation techniques for dividing a complex problem into small elements, as well as the use of software program coded with FEM algorithm. In applying FEA, the complex problem is usually a physical system with the underlying physics expressed in either PDE or integral equations, while the divided small elements of the complex problem represent different areas in the physical system.

FEM Solution Process

i. Divide structure into pieces (elements with nodes) (discretization/meshing)

ii. Connect (assemble) the elements at the nodes to form an approximate system of equations for the whole structure (forming element matrices)

iii. Solve the system of equations involving unknown quantities at the nodes (e.g. displacements)

iv. Calculate desired quantities (e.g. strains and stresses) at selected elements

### 3.1.2 Basic Modeling Decisions

A GRPS embankment system is a three-dimensional problem, which requires three-dimensional numerical modeling for its ideal representation as suggested by Kempton et al. (1998). However, the widespread use of three-dimensional numerical modelling is limited due to the large time consumption, complexity of the modelling process and the requirement of high performance computers. In order to perform two-dimensional modelling, the piles have to be converted into the plane-strain condition. Because in plane-strain analysis, it is not possible to model the three dimensional nature of a pile and this will lead to modelling the piles as continuous walls spanning in the longitudinal direction. The actual properties of a pile are smeared in the plane-strain direction per meter width in order to obtain equivalent properties for the pile wall.

One way of of modelling the piles in two-dimensional plane-strain condition is the use of an equivalent elastic modulus for the pile walls, keeping the thickness of the wall the same as the pile diameter . This is done by taking the normal stiffness of the piles into consideration as shown in Figure 3-1. As mentioned by Huang et al. (2009) cited by Ariyarathne (2014). Equivalent elastic modulus of the walls can be found by;

$$\boldsymbol{E_{eq}A_w = E_p.A_p + E_s\,(A_w - A_p)} \qquad (3\text{-}1)$$

where, $A_w$ = is the plan area of pile wall

$A_p$ = is area of the pile head

$E_p$ = Elastic modulus of the pile

$E_s$ = Elastic modulus of the soil

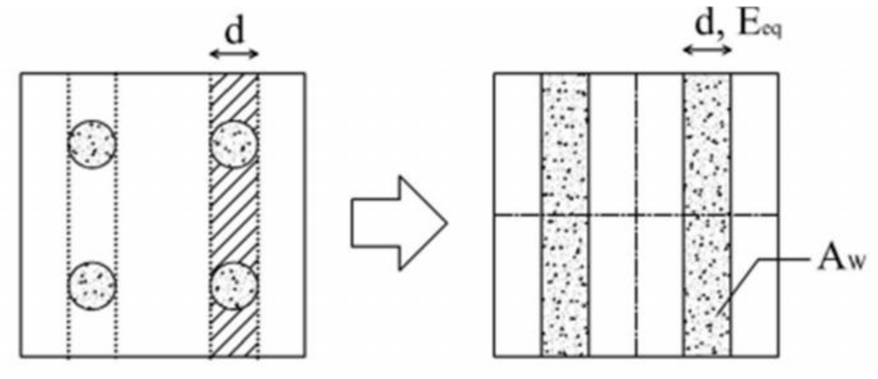



**Figure 3-1: Two-dimensional idealization method**

Therefore, for the embankment, granular fill, geosynthetic, pile and subsoil, 2D, deformable, shell element was chosen. A shell element means that the thickness of the element is significantly smaller than the other dimensions. On the property module an elastic, isotropic material was chosen, where Young's Modulus and Poisson's ratio were then defined. Then a section was created, where a homogeneous solid section was chosen. An instance of the shell elements was created and assembled into the global coordinate system. The first analysis step is Initial, which is the default that is already there. For the model, a Load step was added. A static, general load was chosen, and the NLGEOM command was turned off because NLGEOM is used for large displacements analysis and buckling. As loads and boundary conditions are step dependent, meaning that the analysis step in which they are performed must be specified, boundary conditions were created in the Initial step. Mechanical, displacement/rotation boundary conditions were chosen. Next, the load was created in the Load step. A mechanical pressure was chosen for the load, and the pressure was assumed to be uniform for simplicity.

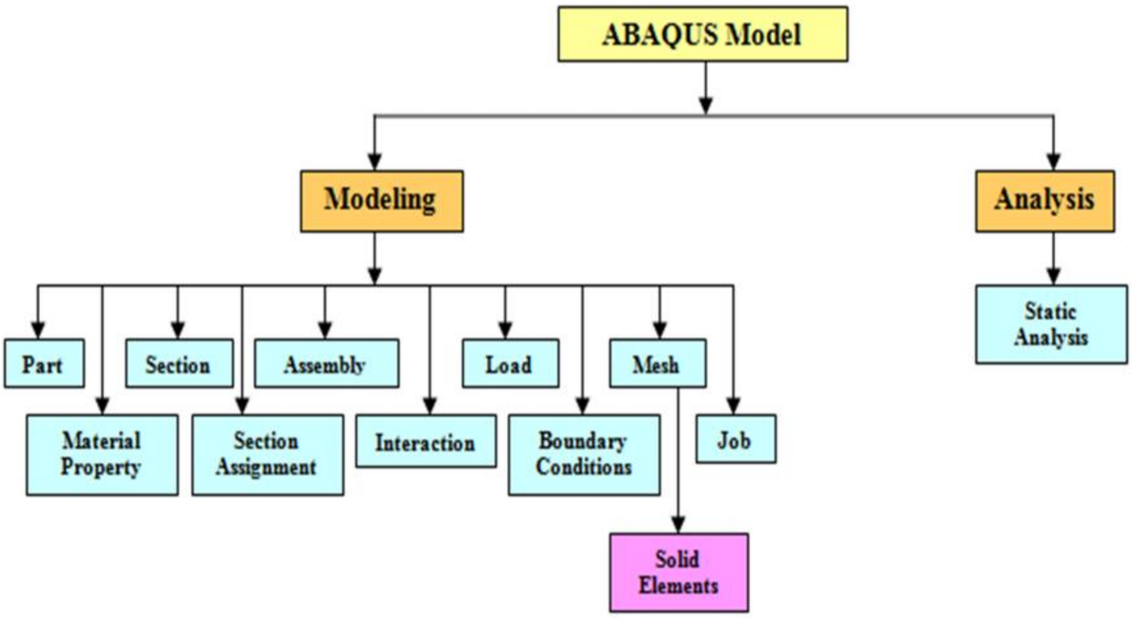



**Figure 3-2: Algorithm for ABAQUS modeling and analysis modules**

The main limitation of FEM analysis is to choose a material model, which can accurately capture the material mechanical behavior. At present, the theory of linear elastic analysis to study stress-strain conditions in a pavement is widely used in mechanistic pavement design. In linear elastic analysis, each layer is characterized by Young's modulus and Poisson's ratio. However, it can only approximate the variation of stiffness especially for granular materials. Hence, FEM analysis of granular material should take into account the nonlinear stress-strain relationship.

#### *3.1.2.1 Elastoplastic material models*

Most common engineering materials exhibit a linear stress-strain relationship up to a stress level known as proportional limit. Plastic behavior happens when stress exceeds the yield point. Beyond this limit, the stress-strain relationship will become nonlinear, where materials undergo significant increase in strain for very small increase in stress. Such increase in strain is termed plastic flow of the material. Additionally, nonlinear stress-strain relationships in an elastoplastic model can cause changes of material stiffness at different load levels. In an elastoplastic material, the total strain of a material is considered as the sum of recoverable elastic strain and permanent plastic strain components.

According to Kelly (2013), in the one-dimensional (uniaxial test) case, a specimen will deform up to yield and then generally harden. It is shown in Figure 3-3 that in the perfectly plastic case, once the stress reaches the yield point (A), plastic deformation ensues, so long as the stress is maintained at Y. If the stress is reduced, elastic unloading occurs. In the hardening case, once yield occurs, the stress needs to be continually increased in order to drive the plastic deformation. If the stress is held constant, for example at B, no further plastic deformation will occur; at the same cannot occur in the perfectly-plastic case, where there is one of plastic deformation or elastic unloading.

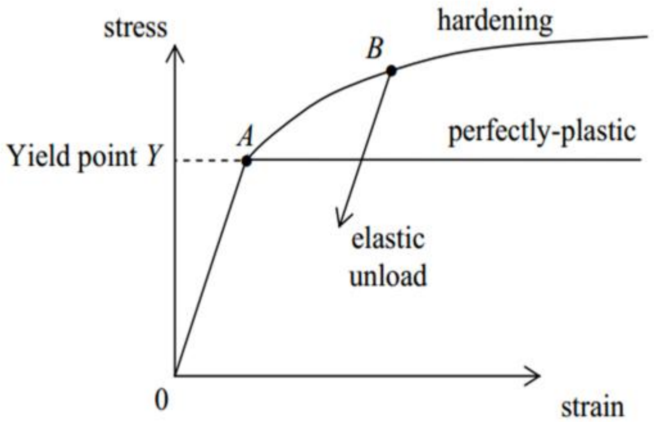



**Figure 3-3: Typical uniaxial stress-strain curve (Kelly, 2013)**

The Mohr-Coulomb approach can be used when experimental data are not directly available from the friction angle and cohesion values of the materials to be modeled. One approach to matching Mohr-Coulomb parameters is by performing a triaxial test under tension and compression. The Mohr Coulomb yield surface can be defined as:

$$\tau = \sigma \tan \phi + c \qquad (3\text{-}2)$$

Where ϕ is the angle of internal friction and c denotes the cohesion of the material. A geometrical interpretation of Coulomb's criterion can be established by considering Mohr's circles involving the principal stresses $\sigma_1$, $\sigma_2$ and $\sigma_3$ as shown in Figure 3-4.

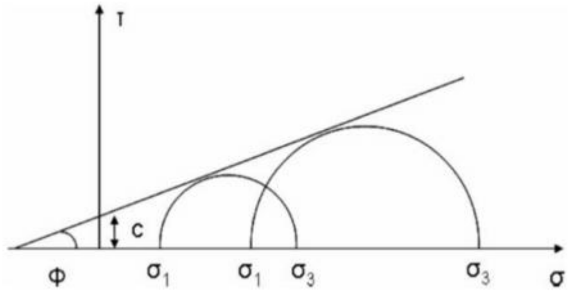



**Figure 3-4: Graphical representation of Mohr Coulomb failure criteria (Mijangos & O'Kelly, 2009)**

The Mohr-Coulomb model can be rewritten in terms of the principal stresses as follows:

$$\sigma_1 - \sigma_3 + (\sigma_1 + \sigma_3)\sin\phi - 2c\cos\phi = 0 \qquad (3\text{-}3)$$

## 3.2 Numerical Modeling

This section presents the details of the numerical model and the procedure adopted for simulating the above mentioned embankment using the finite element method.

### 3.2.1 Site Conditions and Embankment Geometry

Since main objective of this study is to carry out two-dimensional finite element modeling of a GRPS embankment and discuss the effects of adding pile supports and geosynthetic reinforcement to the embankment behavior; general embankment geometry was selected for the analysis with two foundation soil layers. The cross section of the embankment along a pile row in the longitudinal direction is shown in Figure 3-5.

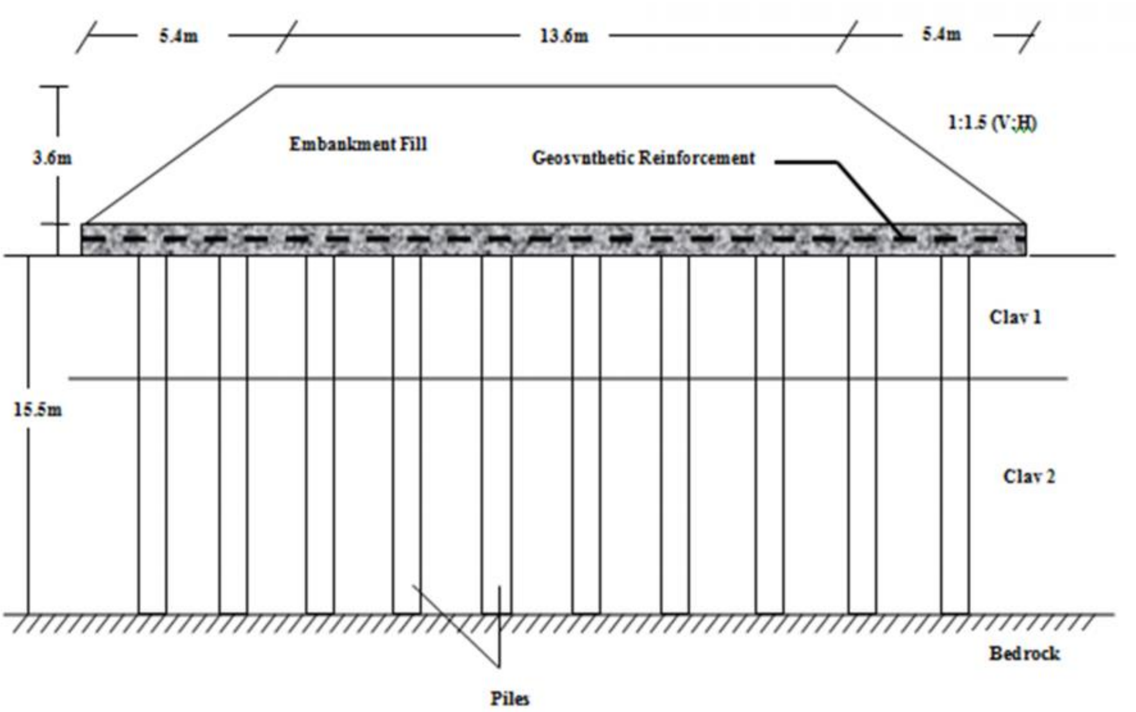



**Figure 3-5: Cross-Section of the Embankment over a Soft Soil**

Selected soil profile consists of two soil layers resting on bedrock. The first layer is a clay layer having a thickness of 6.5m and it is followed by another clay layer of 9m thickness with different material property which is resting on bedrock.

The embankment spans in the longitudinal direction and has a crest width of 13.6m and a base width of 24.4m. The side slopes are 1:1.5 (Vertical: Horizontal). The total height of the embankment fill is 3.6m. The fill material consisted mainly of gravel mixed with sand, with an average unit weight of 20kN/$m^3$. The piles were arranged in a square pattern with a spacing of 2m. Table 3-1: summarizes the information about the piles. Above the clay layers one layer of the geosynthetic was sandwiched between two, 0.3-m-thick, gravel layers to form a 0.6-m-thick composite-reinforced bearing layer as shown in Figure 3-6. The geosynthetic layer is placed 0.3m above the pile heads in order to prevent damage during construction and due to the pile edges. Piles used here are end bearing piles which are supported by bedrock.

**Table 3-1: Information of a Geosynthetic Reinforced Pile-Supported Embankment**

| Embankment Height (m) | Pile Length (m) | Pile Spacing (m) | Pile Diameter (m) | Cushion Structure |
|---|---|---|---|---|
| 3.6 | 15.5 | 2.0 | 1.0 | 0.6m gravel cushion + a layer of geosynthetic |

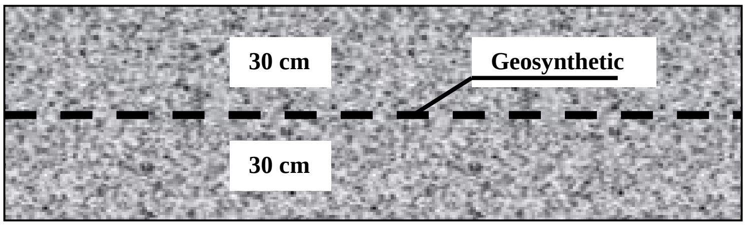



**Figure 3-6: Structure of the Gravel Cushion with a Layer of Geogrid**

### 3.2.2 Material Model and Parameters

The behavior of the track substructure should be modeled in order to take in to account the properties of the subgrade materials when developing a model of the track system. As discussed in the above literature, the resilient modulus, the plastic strain and the failure criteria either the Drucker-Prager or Mohr-Coulomb criteria of the subgrade material are important parameters to be considered in track subgrade modeling.

The pile and the geosynthetic materials were both modeled as elastic materials in order to simplify the model. This is due to the assumption that geosynthetic material was observed to show a very minimal damage and plastic strain from previous experiences of Shi (2009). But the embankment, the gravel bed and the subgrade materials are modeled as elastoplastic materials.

The material parameters used for the numerical modeling are shown in Table 3-2.

**Table 3-2: Material properties used in the finite element simulations (Cui & Zhuang, 2015)**

| Material | Density ($kg/m^3$) | Elastic Modulus, E(MPa) | Poison's ratio, *v* | C (kPa) | Angle of internal friction, φ |
|---|---|---|---|---|---|
| Embankment (gravel mixed with sand) | $2x10^3$ | 60 | 0.15 | 40 | 20 |
| Gravel bed | $1.8x10^3$ | 12.5 | 0.35 | - | 20 |
| Geosynthetic Reinforcement | Tensile stiffness kg = 1.0 MN/m; Poisson's ratio v = 0.3 (assumed) | | | | |
| Clay 1 | $1.9x10^3$ | 6.4 | 0.3 | 37.3 | 20.4 |
| Clay 2 | $2x10^3$ | 9.2 | 0.3 | 75.6 | 27.3 |
| CFG Pile (Cement Fly-ash Gravel) | $2.3x10^3$ | 22000 | 0.1 | - | - |
| Pile wall | $2.3x10^3$ | $E_{eq}$ = 4326 | 0.1 | - | - |

The material model used for modeling the coarse grained fill layer and the embankment fill layer is a linear elastic-perfectly plastic material model with Mohr-Coulomb failure criterion where E is the elastic modulus, c’ is the effective cohesion, φ' is the effective friction angle, ψ is the dilation angle and *ν* is the Poisson’s ratio.

Piles were modeled with linear elastic material properties (the Young’s modulus and Poisson’s ratio).

A geosynthetic layer with a higher stiffness was used in this model in order to observe the influence of adding a geosynthetic layer to a conventional embankment. The geosynthetic layer was modeled with linear elastic material properties similar to the piles.

### 3.2.3 Fundamental assumptions and simplification

The following assumptions and simplification are made in establishing the model for static analysis of the embankment over soft soil.

- Track is modeled in 2D to minimize computational costs in 3D model
- Only vertical static loads are considered in the model
- All the elements are modeled as solid elements
- Load is applied on the embankment: this is to know the subgrade deformation

### 3.2.4 Two-Dimensional Modeling

Due to the symmetry of the embankment along its middle vertical axis, only a half of the embankment was modeled. The total horizontal length of the model is taken to be 36.6m, in order to minimize boundary effects. The geosynthetic layer was placed inside the first fill layer at a 0.3m height above the pile heads. Generally a gravel platform is used to contain the geosynthetic layer.

The interaction between the geosynthetic layer and soil was taken into account during this analysis. A value of 0.8 was selected for the interface friction coefficient between the granular fill and geosynthetic layer which is a realistic value. The pile-soil interface behavior was not considered in this analysis due to convergence problems occurred in the contact algorithm during the analysis. According to Priyanath, satisfactory results can be obtained from numerical modeling of GRPS embankments without considering the interface friction between soil and piles (Ariyarathne, 2014).

The aim of this paper is to investigate the track substructure response without applying any improvement to the substructure and under the application of pile supports and then geosynthetic. The track system was modeled consisting only of the embankment and subgrade materials. Consequently, a distributed pressure was introduced on the embankment according to the recommended pressure on the subgrade. According to AREMA [section 10.2.2.6] it is recommended that the calculated pressure be smaller than 25psi on a good subgrade. The value should be reduced for subgrade of poor quality.

On the basis of simplified consideration, to the assumption that when load is applied above a sleeper, the sleeper below the load is applied supports 50% of the wheel load and each of the neighboring sleepers support another 25%. Stress measurement and finite element analysis application, however, have shown that the wheel load distribution along successive sleeper is as shown in Figure 3-7 and cited by Profillidis (1995):

- Sleeper under wheel load………………….40%
- First neighboring sleeper…………………..23%
- Second neighboring sleeper………………..7%

Therefore, when a wheel load is applied over a sleeper, its effect is negligible beyond the second successive sleeper.

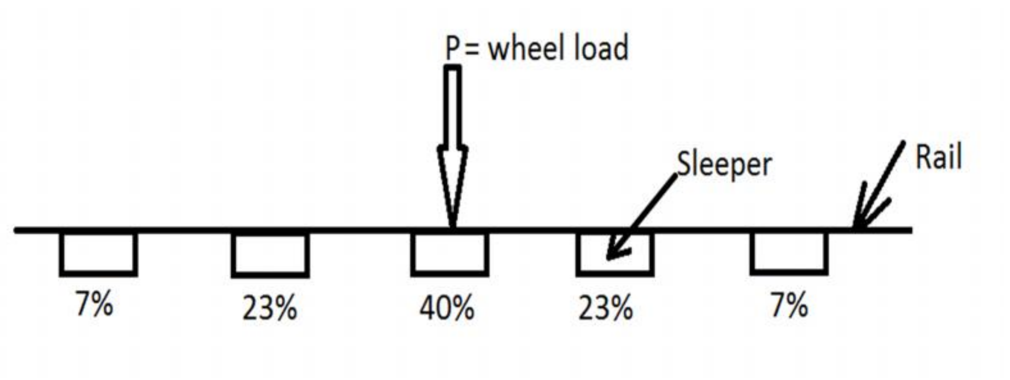



**Figure 3-7 Wheel load distribution along successive sleepers**

**Table 3-3 Modeling parameters for rail and Sleeper**

| Model components | Density (kg/m$^3$) | Elastic Modulus, E(MPa) | Poisson's ratio |
|---|---|---|---|
| Rail | 7850 | 210 | 0.3 |
| Sleeper | 2400 | 70 | 0.2 |

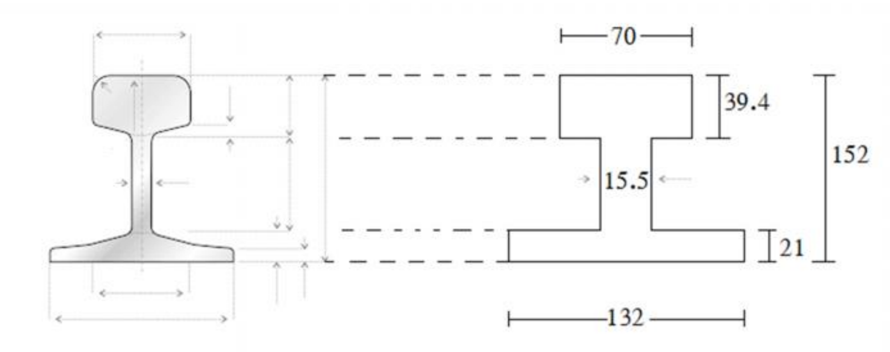



**Figure 3-8 Cross section of rail and sleeper**

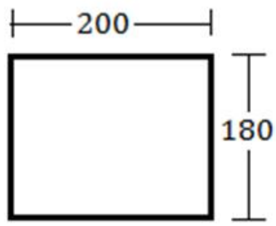



**Figure 3-9 Sleeper cross section**

**Table 3-4 Modeling parameters for rail and Sleeper**

| Model components | Stiffness ($K_p$) | | Damping ($C_p$) | |
|---|---|---|---|---|
| Rail pad | Soft pad | Stiff pad | Soft pad | Stiff pad |
| | 100 KN/mm | 600 KN/mm | 43 Ns/mm | 93 Ns/mm |

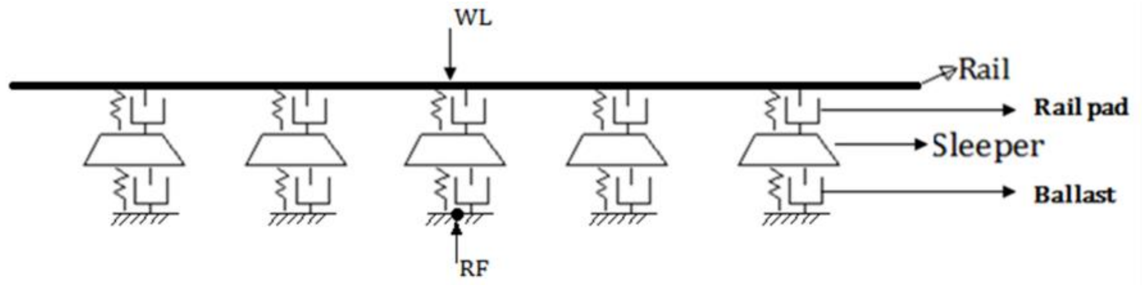



**Figure 3-10: Model of the rail-sleeper system**

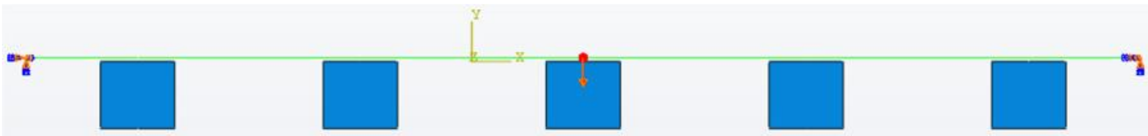
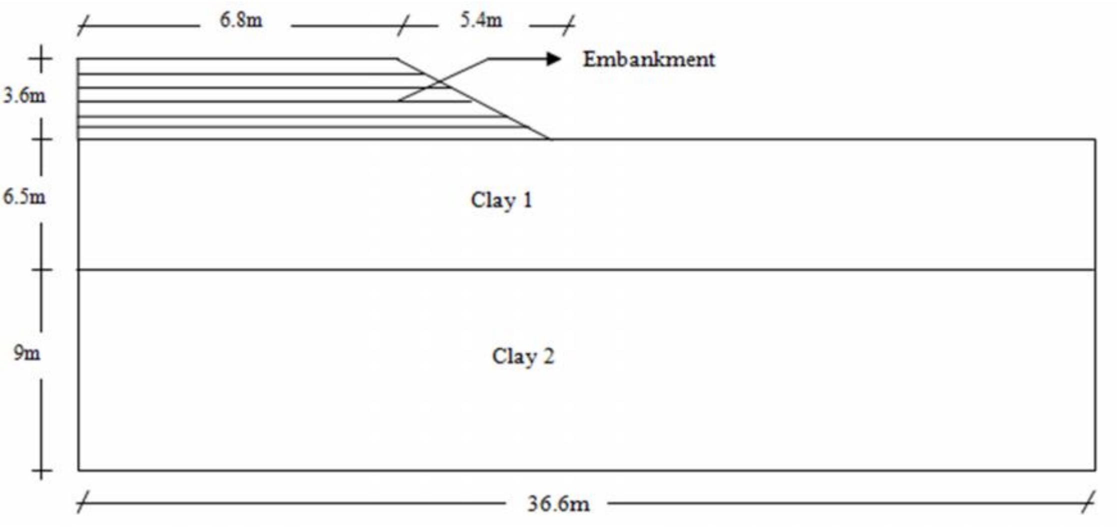


**Figure 3-11: Abaqus model of rail-sleeper system**

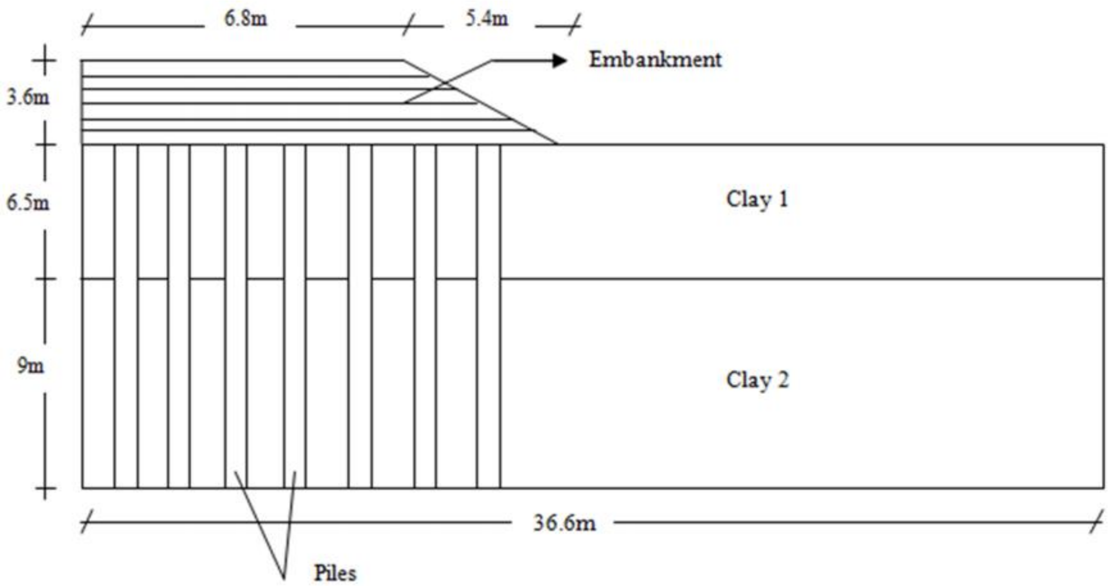
The reaction force (RF) due to the wheel load of 25t applied is found to be 0.1N. Changing this to a distributed pressure, i.e. $P=F/A_e$ (where $A_e$ effective area), gives 123kPa. As mentioned above this is less than the allowable pressure of 25psi=172kPa which must be lowered for poor subgrade quality.

#### 3.2.4.1 Different Cases Selected for the Analysis

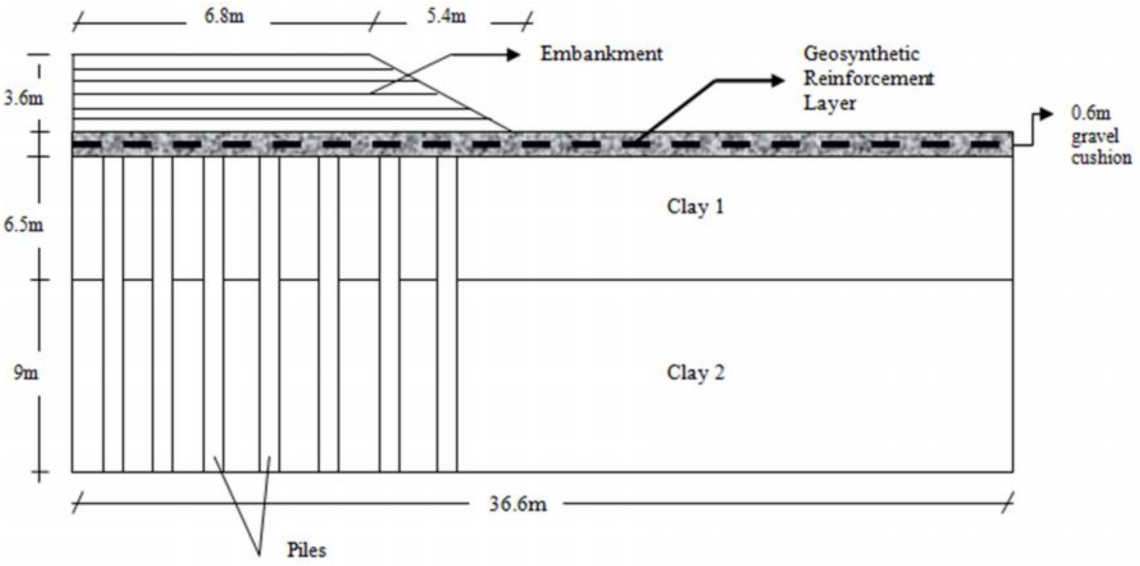
Three different analysis cases were selected for numerical modeling as mentioned in the introduction. The first one is the embankment problem without pile supports or geosynthetic reinforcement (Case 1); the second case is where pile supports are added to Case 1; and Case 3 is the one with both pile supports and geosynthetic reinforcement as shown in Figure 3-12.


6.8m
5.4m
Embankment
3.6m
6.5m
Clay 1
9m
Clay 2
36.6m

**Case 1**


6.8m
5.4m
Embankment
3.6m
6.5m
Clay 1
9m
Clay 2
36.6m
Piles

**Case 2**




**Case 3**

**Figure 3-12: Different cases used for the analysis**

## 3.3 Analysis of Results

Results obtained from the two-dimensional numerical modeling are presented and discussed in the following sections.

### 3.3.1 Comparison of the Three Cases

The behavior of the embankment without pile supports or geosynthetic reinforcement (Case 1), with only pile supports (Case 2) and with both pile supports and geosynthetic reinforcement (Case 3) are compared in this section. The comparison is carried out using lateral displacements of the embankment, settlements, vertical stress distribution in piles and sub soil, and influence of the number of geosynthetic layers.

#### *3.3.1.1 Lateral Displacements*

The lateral displacements in the embankment toe are compared for the three cases. Figure 3-13 shows the lateral displacement of the embankment toe with time. According to this figure, it is clear that the lateral deformation of the toe is considerably higher in Case 1 when compared to Cases 2 and 3. This very high deformation in Case 1 is reduced by 80% when piles are added to the system. With the addition of a geosynthetic layer (Case 3), the deformation in Case 2 is reduced by 12%. Therefore, it is clear that the addition of pile supports to a conventional embankment problem can reduce the lateral deformations by a significant amount and by adding a geosynthetic layer to the system, this can be reduced further.

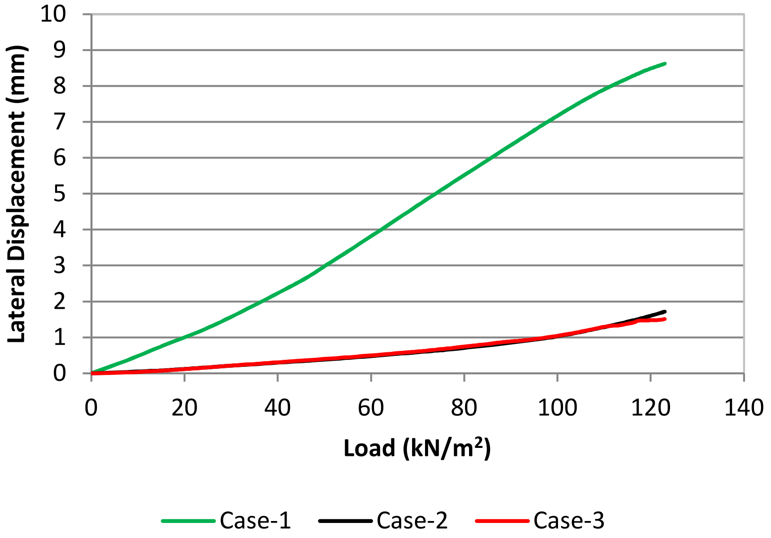



**Figure 3-13: Lateral deformation of the embankment toe**

### *3.3.1.2 Vertical Stresses*

The addition of a geosynthetic reinforcement layer has increased the vertical stress on the piles by 2.65%. Therefore, by considering Figure 3-14, it can be shown that the addition of a geosynthetic layer to a pile-supported embankment slightly increases the vertical stress transferred to piles.

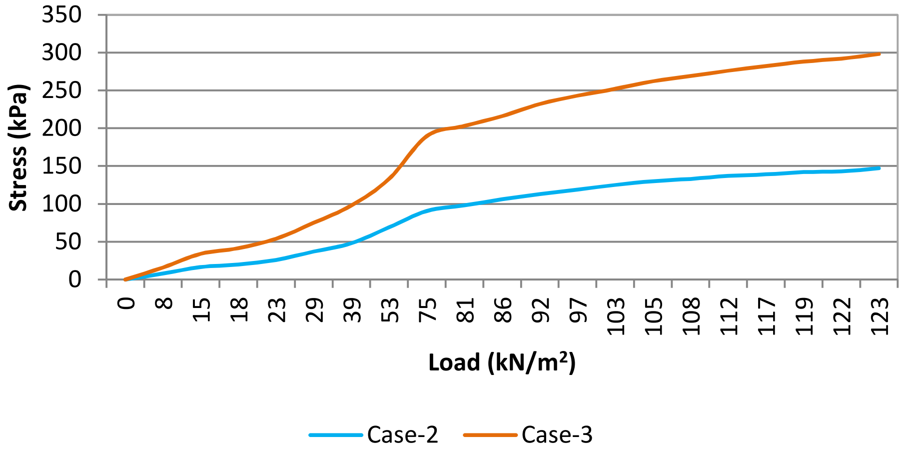



**Figure 3-14: Vertical stress on the pile head (center pile)**

The vertical stress variation on the subsoil for Cases 2 and 3 in Figure 3.15 keeps increasing. However, this increase is not as much as that of the piles i.e. with a difference of 75%. This is because much of the stress is transferred to the piles as a result of soil arching. Due to the presence of soft foundation soil between piles, the embankment fill in between piles will tend to settle more than the fill just above the piles. This load transfer mechanism is termed soil arching. The shape of the arch is shown in Figure 3-16 below.

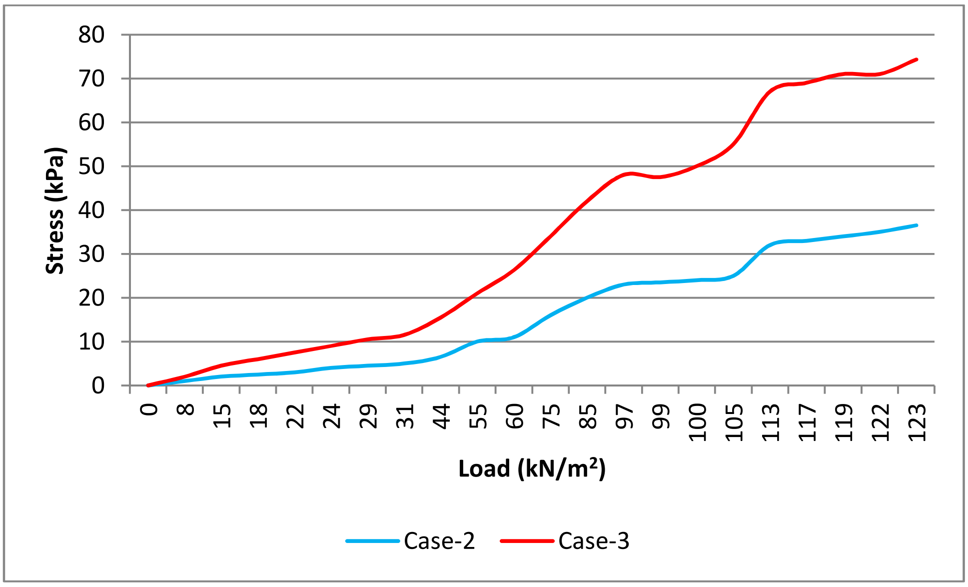



**Figure 3-15: Vertical stress on the subsoil**

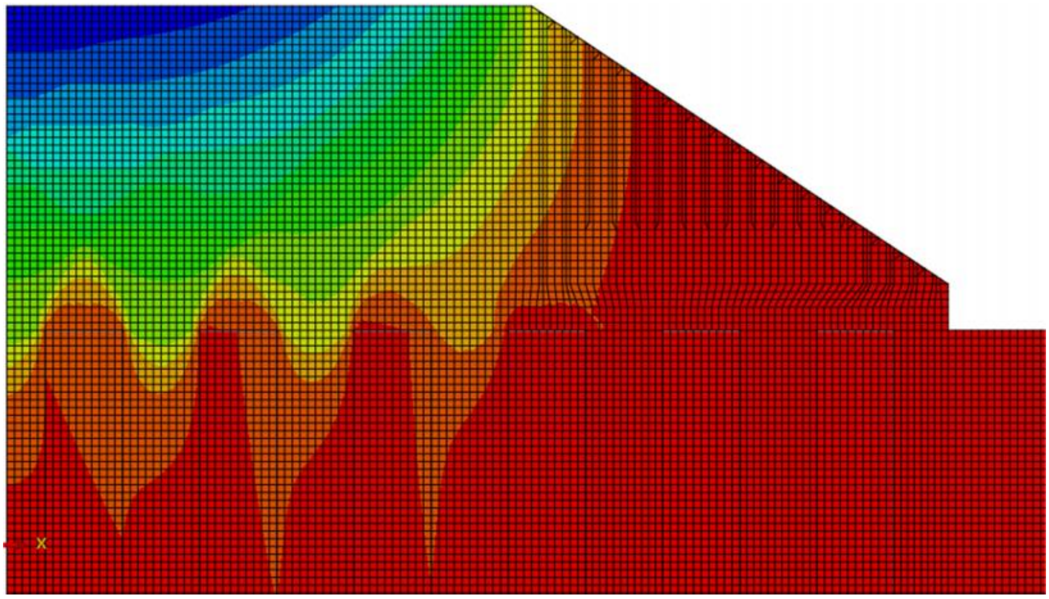


**Figure 3-16: Shape of the arch when piles are incorporated**

#### *3.3.1.3 Settlement*

##### *3.3.1.3.1 Settlements of Piles and Soft Soil*

According to Figure 3-17, settlements are extremely high when piles or geosynthetic reinforcement are not present. When piles are incorporated into the system, the settlements are significantly reduced. This settlement improvement due to the addition of a geosynthetic layer is negligible. However, in Cases 2 and 3, the maximum settlement is about 96% lesser than that of Case 1.

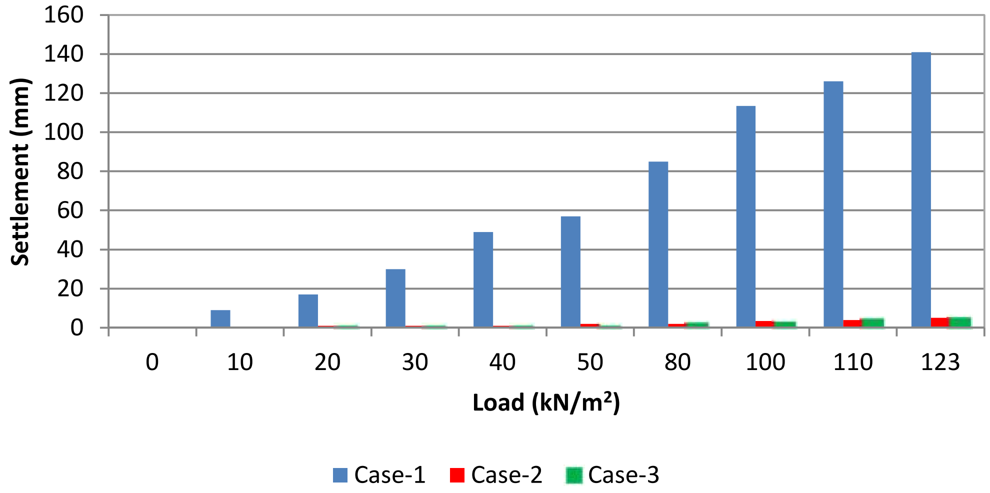



**Figure 3-17: Settlement of the subsoil**

Pile settlements are extremely low (about 83% less) when compared to the settlements in the subsoil (Figure 3-18). One reason for this is the higher stiffness of piles when compared to the foundation soil. Another reason is that the sub soil to be improved is assumed to be supported by bedrock in this study (implied by a fixed boundary condition) including the piles, which will restrain downward pile movements.

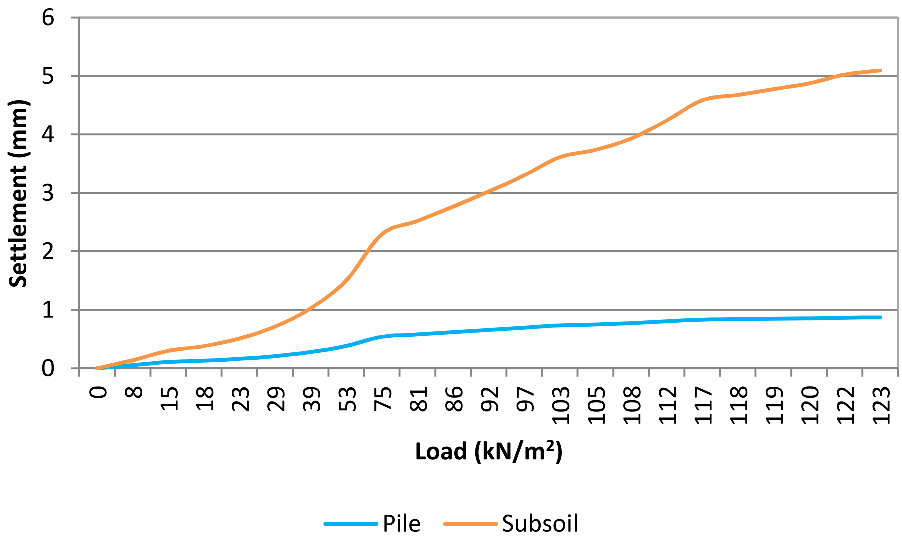



**Figure 3-18: Pile settlements vs. settlements in the sub soil**

A negligible vertical displacement (for pile 6, i.e. far from the embankment center or which is almost near the embankment toe compared to pile 1 near the embankment center in Figure 3-19) is observed in Figure 3-20.

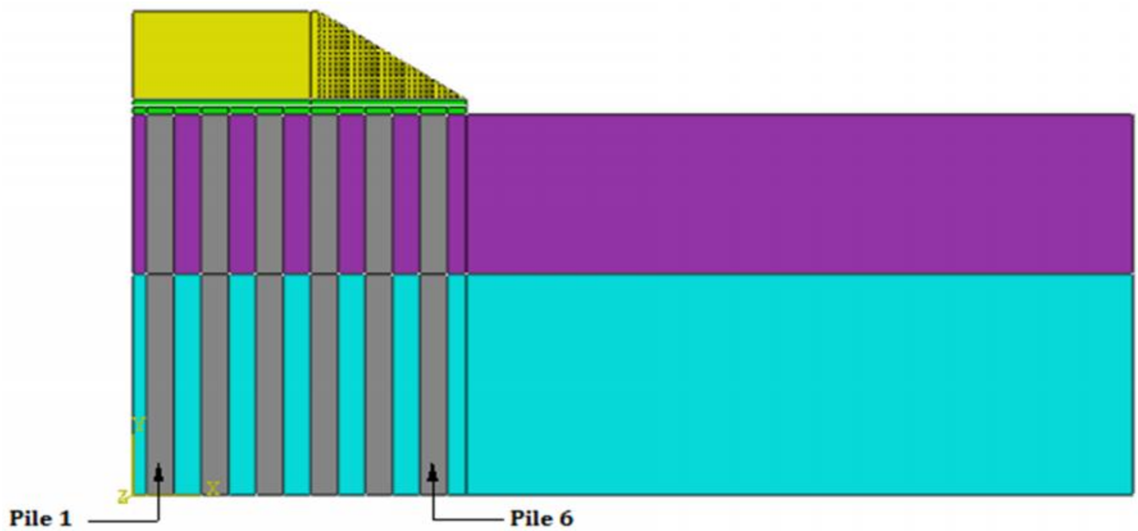



**Figure 3-19: Pile 1 and Pile 6**

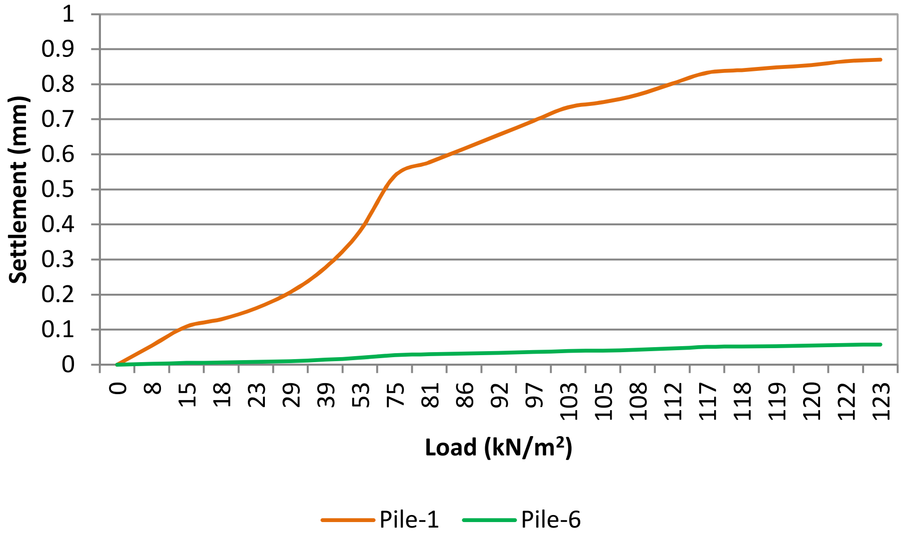
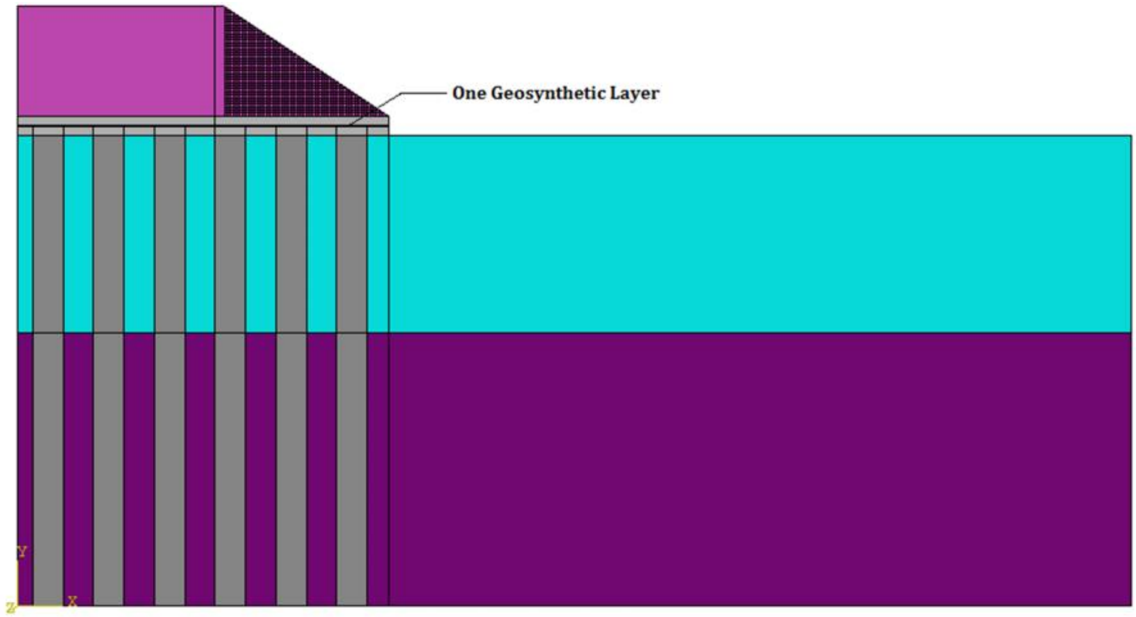



**Figure 3-20: Settlement of piles**

### *3.3.1.4 Influence of the Number of Geosynthetic Layers*

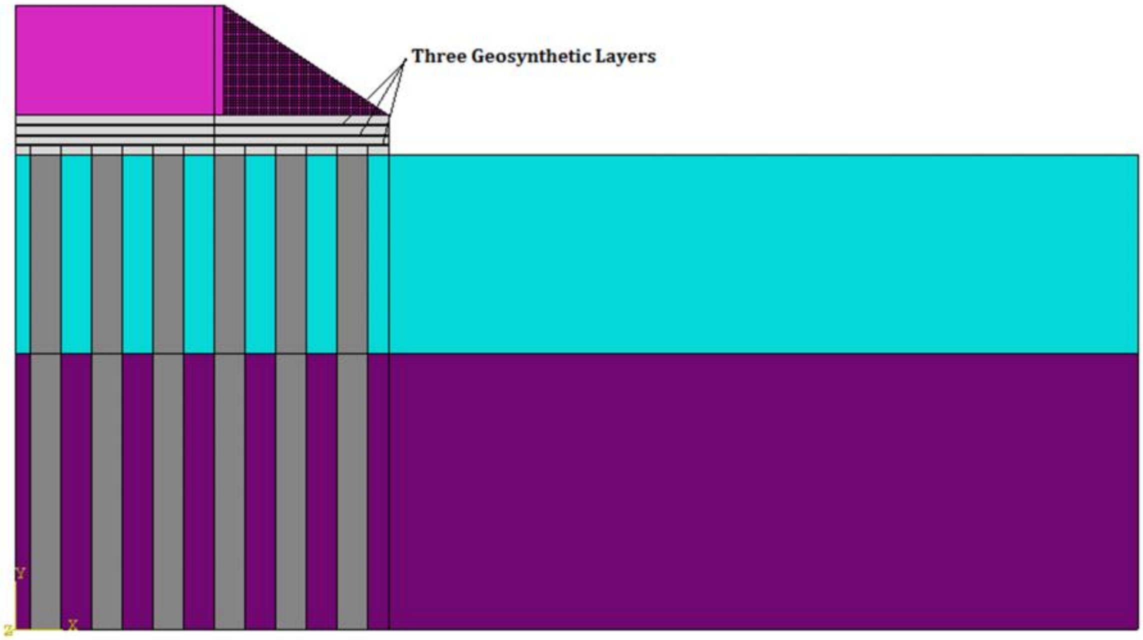
#### *3.3.1.4.1 Lateral Displacement*

GRPS embankments can be constructed with a single layer of geosynthetic reinforcement or a number of reinforcement layers. Behavior of the selected GRPS embankment was studied with the addition of multiple layers of geosynthetic reinforcement. The study was carried out up to three geosynthetic layers as shown in Figure 3-21 (a), (b), and (c). The vertical spacing of the reinforcement layers are taken to be 0.3 m with the first layer placed 0.3 m above the pile heads. The same soil-geosynthetic friction coefficient of 0.3 was used for all the layers in this study.




**(a) One Geosynthetic Layer**


**(b)Two Geosynthetic Layers**




**(c)Three Geosynthetic Layers**

**Figure 3-21: Position of geosynthetic layers**

When the number of geosynthetic layers is increased, the lateral displacement is reduced as shown in Figure 3-22. The reason for this type of variation is, when more than one geosynthetic layer is used the stiffness of the load transfer platform is increased and there is more resistance to lateral movements. This variation is similar to that of the single reinforcement layer with increased stiffness.

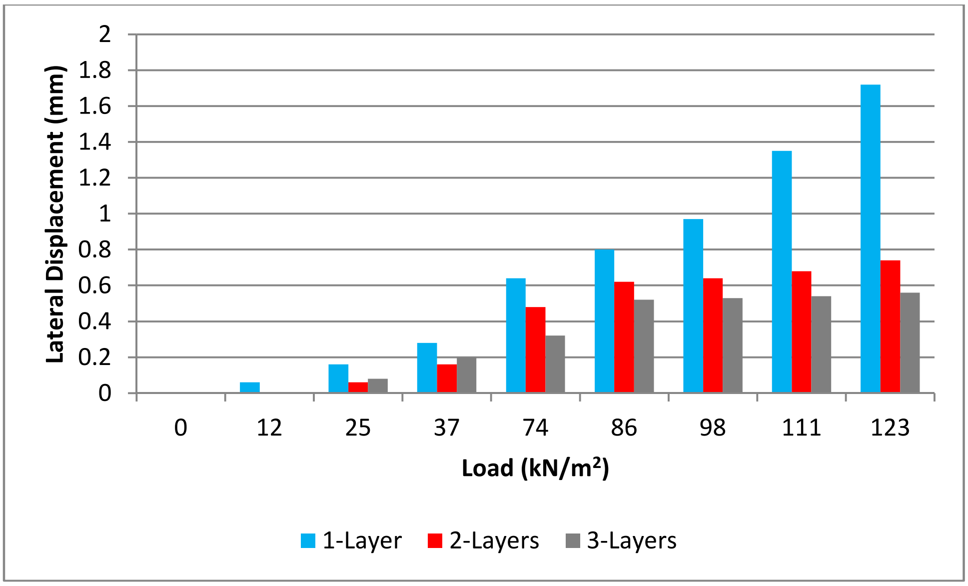



**Figure 3-22: Lateral displacement of the toe with the number of geosynthetic layers**

# CHAPTER 4 CONCLUSIONS AND RECOMMENDATIONS

## 4.1 Conclusion

A finite element interface model of analysis was presented to predict the behavior of a geosynthetic reinforced pile-supported embankment over soft soil under static loading. An embankment over soft soil was numerically modeled first with no improvement made for the soft subsoil, then only with pile support, and finally with pile support and geosynthetic reinforcement in this study using the finite element modeling software ABAQUS\Standard. All the numerical models analyzed here are two-dimensional plane-strain models, which consider a cross-section of the embankment for modeling while assuming the behavior of piles as pile walls. Three different cases were compared where Case 1 has no pile supports or geosynthetic reinforcement, Case 2 has only pile-supports and Case 3 has both pile-supports and geosynthetic reinforcement.

- The addition of pile supports to a conventional embankment problem can reduce the lateral deformations by a significant amount and by adding a geosynthetic layer to the system this can be reduced further.

- The addition of a geosynthetic layer to a pile-supported embankment slightly increases the vertical stress transferred to piles.

- Settlements are extremely high when piles and geosynthetic reinforcement are not present. When piles are incorporated into the system, the settlements are significantly reduced and further reduced by a small amount by adding a geosynthetic layer.

- The observed vertical displacements on the subsoil were very high compared to the displacement on pile heads for Cases 2 and 3 due to the load transfer by soil arching. In the comparison of vertical displacements on foundation soil and piles a significant reduction is observed which brought the concept of soil arching.

- The observed vertical stress on pile heads were very high compared to the stress on subsoil for Cases 2 and 3 due to the load transfer by soil arching. In the comparison of vertical stresses on subsoil and piles for the three cases, a significant reduction is observed in the stress on subsoil when pile supports are added and with the addition of the geosynthetic layer, the vertical stress reduced further. As a result of this behavior, the vertical stress on pile heads increased for Case 3 in comparison to Case 2.

- Increasing the number of geosynthetic layers influenced the lateral displacement to reduce.

## 4.2 Recommendations and further study

### 4.2.1 Recommendation

In this study, finite element model simulation were performed on embankments over soft soil with no improvement, with pile support, and with pile support and geosynthetic reinforcement under static loading conditions to observe the stress, and deformation behavior of a track subgrade. Three types of FE model simulations were performed on similar geometry and material properties, except the soft soil improvement types were altered to demonstrate the effects of pile supports and geosynthetic reinforcement. This study provided implications of use of GRPS in various practical situations in a railroad substructure if supported with further detailed studies.

### 4.2.2 Further study

Though there is an improvement in the stress and vertical displacement due to the introduction of the pile supports and geosynthetic reinforcement, the improvement result obtained is can be more satisfactory if the some points are considered further. These include:

- Three-dimensional numerical modeling
- Dynamic loading for serviceability assessment

The effect of increasing the spacing of the piles may also be studied further.

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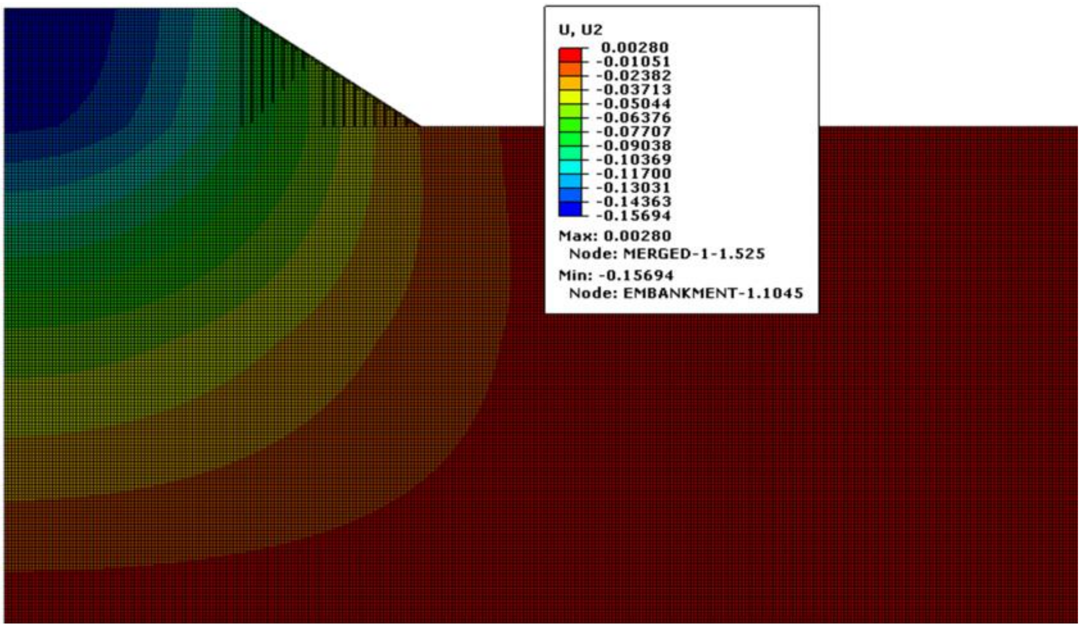
# APPENDICES

## Appendix A

<u>Contour Plots of Vertical Displacement Results</u>

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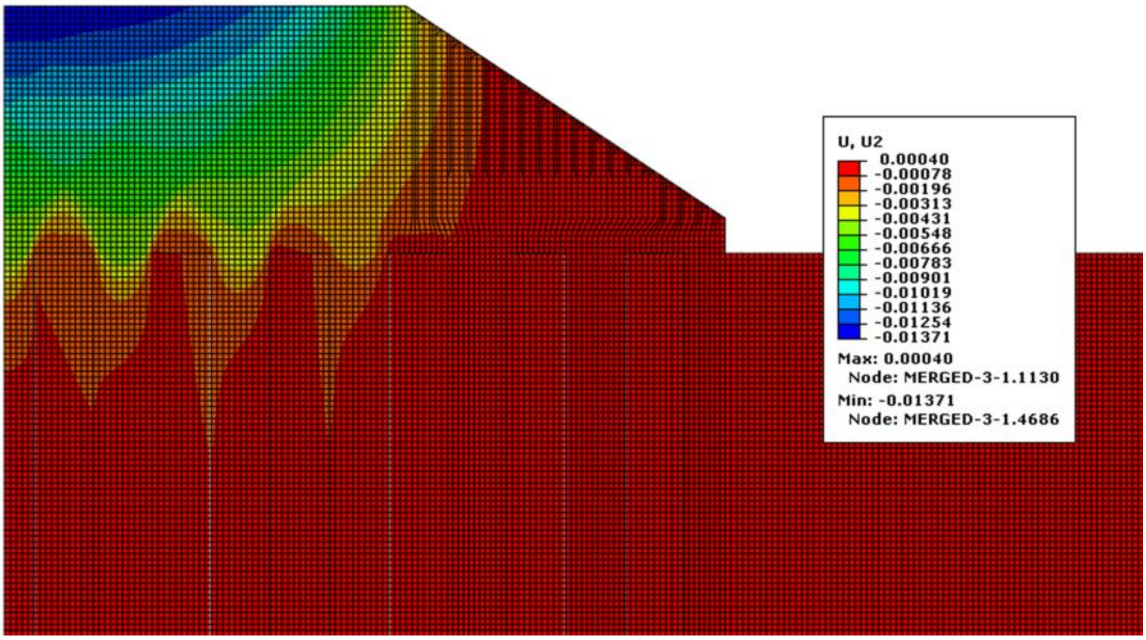
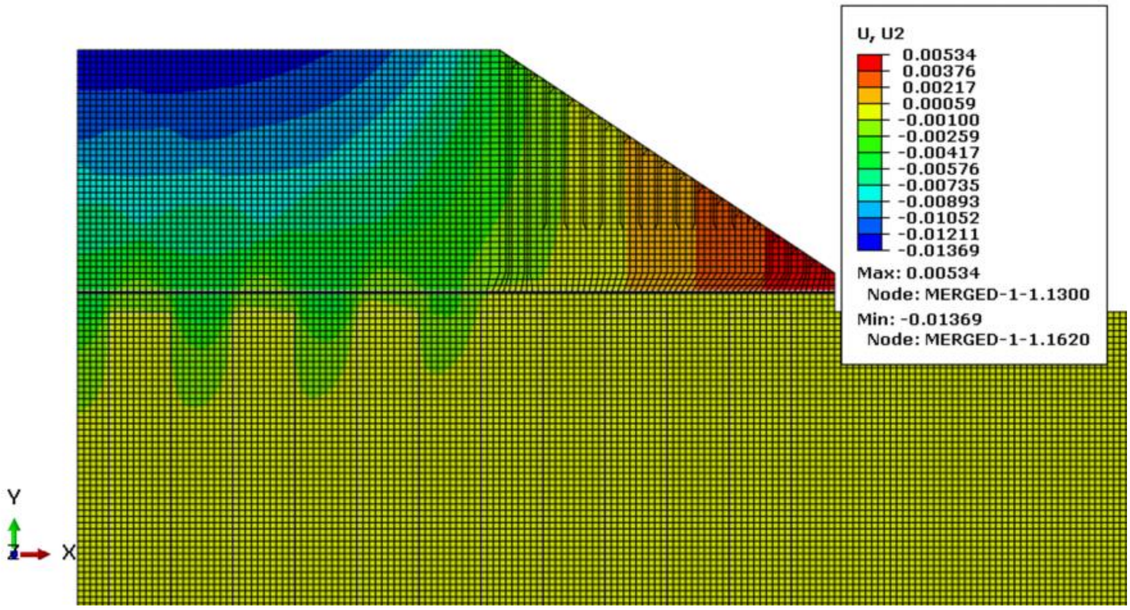



**A1-2: Case 2**


U, U2
0.00534
0.00376
0.00217
0.00059
-0.00100
-0.00259
-0.00417
-0.00576
-0.00735
-0.00893
-0.01052
-0.01211
-0.01369
Max: 0.00534
Node: MERGED-1-1.1300
Min: -0.01369
Node: MERGED-1-1.1620
Y
X

**A1-3: Case 3**