MODELING STUDIES TO ASSESS LONG TERM SETTLEMENT OF LIGHT WEIGHT AGGREGATE EMBANKMENT

by

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ABSTRACT

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Expanded clay and shale (ECS) is a low weight material derived from clay and shale, and has been used in large quantities throughout the world. Very few studies have been performed to investigate the behavior of ECS and how it can reduce the settlements of embankments when used as a fill material. This thesis presents research findings of numerical modeling to assess the long term settlements of ECS embankment on a soft clay deposit and supports both pavement and traffic loading. Observed settlements near the pavement of a light weight ECS embankment built on soft clay indicated that this material is adequate in reducing settlements when compared to a normal fill embankment.

Laboratory investigations were conducted on both embankment materials and these results showed that normal clayey fill with low plasticity have high compressible properties when compared to the ECS granular material. Numerical modeling followed by the hyperbolic formulation showed that the ECS embankment and normal embankment on soft clay could provide adequate information to interpret long term settlements. Analyses showed that the ECS embankment is efficient in minimizing the long term settlements when compared to normal fill embankment.

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INTRODUCTION

1.1 General

Settlement is a common problem on road embankments and the cause of failure in the road embankment is due to the movement of the pavement which results in either large settlement or sliding due to inadequate shear strength. Erosion damage may also take place but, since this is seldom severe enough to render the pavement unstable, it cannot be classified as failure and is not considered in this thesis. Normal fill can comprise of mix of different soils like silty sand, clayey sand or silty clay etc which are commonly found embankment material at various locations in Texas. Due to climatic variations, swelling or shrinkage related soil movements commonly occur in the soils underneath the infrastructures such as pavements, embankments and light to medium loaded residential & commercial buildings. In pavements, the soil movement results in settlements, surface cracking and thereby creating difficult driving conditions and also costly rehabilitation and maintenance for the TxDOT (Briaud et al., 1997).

Many companies are spending lot of time and effort in reducing pavement distress related to subgrade swell/shrinkage. Researchers are working to find suitable ground improvement techniques of stabilizing embankment soils over soft soil.

1.2 Research Objective

A locally available light weight aggregate (LWA) material, known as expanded clay shale or simply ECS, was used as a fill material for the construction of the south embankment of State Highway SH 360 in Arlington, Texas. The application of expanded clay and shale as a stabilizing fill material beneath the pavements is to mitigate the distress caused from consolidation settlements of underlying foundation soils and also cyclic swell-shrink movements.

This method is expected to improve the long-term performance of these pavements by reducing the overall settlements

Hence, the main objective of the proposed research is to evaluate the effectiveness of ECS material in stabilizing soft subsoils which are supporting transportation infrastructure including highways and embankments. The planned tasks for the study can be summarized as follows:

1. To review the available literature on settlements
2. Study normal fill (treated as Control section) and ECS fill (Test Section)
3. Simulate both type of fills in Finite Element Modeling (FEM using PLAXIS 2D 8.2) and Finite Difference Modeling (FDM using FLAC 2D 4.0)
4. Simulate the long term settlements at SH 360 using different constitutive models.
5. Studying hyperbolic method for interpreting long term settlements using measured elevation survey data
6. Compare numerical model results with hyperbolic method's prediction results and evaluate the performance of ECS in mitigating settlements of embankments

1.3 Thesis Organization

This thesis report consists of six chapters. The first chapter explains about objective of this research. The second chapter presents detailed review of available literature addressing different light weight material treatment and QA/QC aspects followed by description of instrumentation of these sections with inclinometers and Elevation survey. Chapter three explains laboratory procedures pertaining to tests conducted on control section and test section soil specimens. The results obtained from laboratory studies, which includes strength and other properties of both control and treated soil properties are presented. Chapter four includes different model studies used in numerical simulation in FEM (PLAXIS 2D) and FDM (FLAC 2D).

Chapter five presents a comprehensive analysis of the results from the field studies. This includes results obtained from vertical inclinometers and Elevation survey. Here it presents the details of field monitoring data using elevation survey and studying that data using hyperbolic method. This Chapter also presents the comparison of hyperbolic formulation with FEM and FDM analysis results.

Summary and conclusions from this research study, significance of the findings from laboratory, field studies and numerical modeling and future research needs are addressed in Chapter 6.

CHAPTER 2

LITERATURE REVIEW

A comprehensive literature search was conducted to determine the settlement problems nationwide, current knowledge of the causes of settlement, and current mitigation techniques.

2.1 Factors Causing Settlement

Many studies are reported in the literature (Hopkins 1969, Stewart 1985, Kramer and Sajer, 1991) that explains the mechanisms causing the settlement in embankment on the bridge transition. A joint study performed by University of Texas at Arlington and University of Texas at El Paso as well as TxDOT has summarized the following factors causing settlements in the embankments:

- **Consolidation settlement of foundation soil**: Consolidation of foundation soil under approach embankment is one of the most important contributing factors to bridge settlement (Hopkins, 1969; Wahls, 1990; Dupont and Allen, 2002). It usually occurs because of heavy traffic loads applied at the embankment surface and to the weight of the embankment itself (Dupont and Allen, 2002). Mostly cohesive soil represents a threat instead of cohesion-less soil. Cohesive soil like soft or high plastic clay represents more critical situation. Cohesive soils are more susceptible to high lateral and vertical movement. Settlements of soil are divided in to three different phases: Initial, primary and secondary consolidation (Hopkins 1969). The initial settlement is a short term settlement when load is applied on the surface of the soil (Hopkins 1969). The primary settlement mainly contributes to the total settlement of the soils (Hopkins 1973). There is a gradual escape of water due to the compression of loaded soil. Secondary phase occurs as a result of change in void ratio of the loaded soil after dissipation of excess pore pressure (Hopkins 1969).

- **Poor compaction**: Poor compaction of the embankment results in low density, high deformation and unnecessary maintenance cost (Lenke, 2006). Compaction mechanically increases the density, stability and reduces water seepage, swelling and contraction.
- **Poor drainage and soil erosion**: The dysfunctional, damaged and blocked drainage systems cause erosion and void development which can cause serious settlement problems (Jayawickrama et al., 2005). The rain water can seep in to the embankment fill via cracked pavement section or faulty joints and can soften the fill and create internal erosion.
- **Traffic volume**: Lenke (2006) noted that the settlement was found increasing with vehicle velocity, vehicle weight, heavy truck traffic and number of cycles of repetitive loading.
- **Types of Abutment**: A number of researchers Schaefer and Koch 1992; Briaud et al. 1997; Hoppe 1999; Abu-Hejleh et al. 2006 considered lateral movement of the bridge abutment and settlement of the embankment amongst the primary problem. The lateral movement affects the integral abutment bridges much more severely than non-integral abutment bridges. In integral bridges, as the temperature increases, the bridge super-structure moves the abutments toward the retained soil causing high lateral stresses, which can reach stress levels as high as the passive pressure limit (Schaefer and Koch 1992). As the temperature decreases, the abutments move away from the soil, creating a void between the abutment and backfill material. The presence of the void can intensify soil erosion, increasing the size of the void under the approach slab (Schaefer and Koch 1992; Wahls 1990, Greimann et al. 1986) reported that the lateral movement of the bridge that occurs in integral abutment bridges also affects the pile stresses and can potentially reduce the vertical load carrying capacity of the piles. The U-type abutment are the closed abutments having two side walls and front wall resting on spread footing below natural soil. The compaction of embankment fill is difficult in these abutments and is subjected to higher lateral movement. A great number of cantilever and stub abutments have been constructed in Texas. Stub abutments are economical but have maintenance problems (TxDOT Bridge Design Manual, 2001). The settlement at beginning of the bridge has been

noticed on stub while cantilever abutment has shown good performance but has constructional problem and high construction cost.

2.2 Remedial Measures to Reduce Settlement

Construction of structures like embankments and retaining walls over soft soils was always a challenging ground engineering problem due to high settlements experienced by these structures. The ground improvement techniques including stone columns, grouting, dewatering, preloading, vertical drains and in situ mixing of admixtures like lime and cement have been used to improve bearing capacity and reduce settlements of soft soils (Wahls, 1990; Bergado and Patawaran, 2000; Barksdale and Bachus, 1983; Porbaha, 2000).

Wang and Huang, 2006, compared four ground and embankment treatment techniques which included embankment reinforcement, light weight embankment, cement deep mixing piles and separating walls, for uneven settlement, horizontal displacement and stress distribution and found that light weight embankment has more effectiveness among them. There are different types of lightweight geomaterials available in the Market.

Miki (1996) specifies the following applications of the lightweight geomaterials:

- Reducing residual settlement of embankment built on soft ground
- Minimizing differential settlement between approach embankment and structure
- Minimizing deformation when constructing near adjacent structure
- Faster construction period
- Low maintenance infrastructure

The following section discusses different types of lightweight embankment fill materials.

2.2.1 Light-Weight Expanded Polystyrene, EPS

Polysterene also known as EPS is a foamed plastic and is frequently utilized as a lightweight packaging material and as a thermal insulating material is used in construction applications (Thompsett et al., 1995).

A case study near Oslo in 1972, used expanded polystyrene to raise the road level of a highly settled road. On top of the expanded polystyrene, a new 500 mm pavement was laid. During a period of next 12 years, only 80mm settlements were recorded and almost zero settlement since then (Thompsett et al., 1995).

Another example where the EPS material was used for an embankment is reported on the Lokkerberg Bridge near Swedish border. A miniscule 6 cm deformation was observed after 12 years and majority of that occurred during construction period (Frydenlund et al., 2001). Figure 2.1 shows the longitudinal profile of the EPS embankments of the Lokkerberg Bridge and Figure 2.2 presents the settlement information for the same bridge.

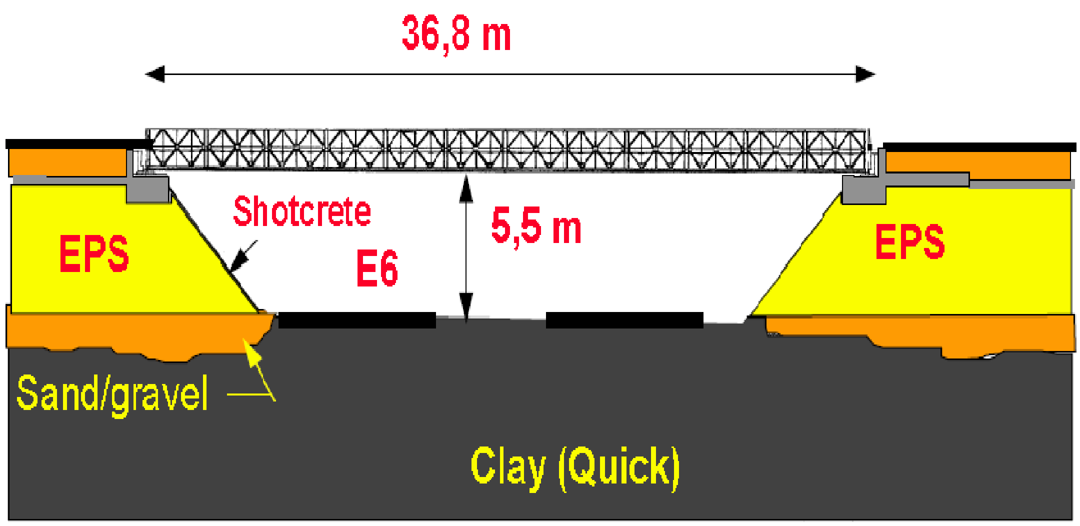


Figure 2.1 Longitudinal Profile of EPS Embankment at Lokkeberg bridge, Swedish Border - Fredyland and Aaboe, 2001

The use of expanded polystyrene blocks for lightweight fill has now become standard practice in road construction and road widening projects in Europe, Asia and North America.

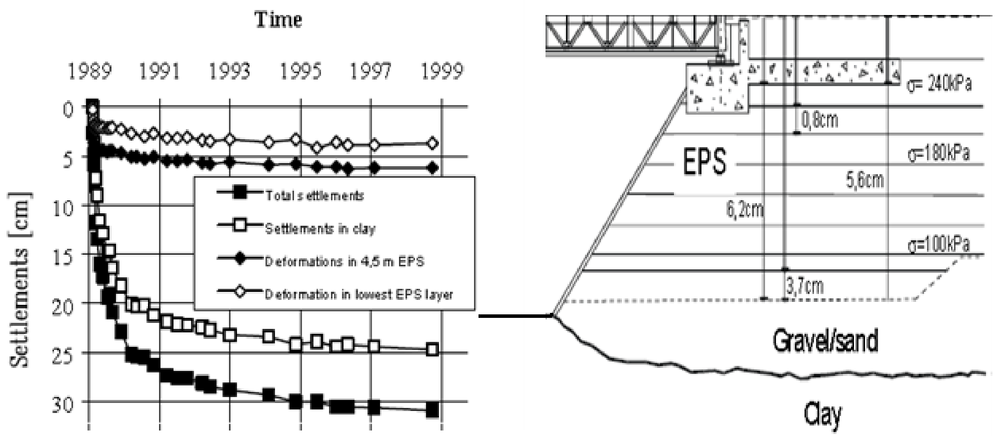


Figure 2.2 Deformations in EPS Embankment at Lokkeberg - Fredyland and Aaboe, 2001

Expanded polystyrene is resistant to common inorganic acids and alkalis but not resistant to organic solvents including petrol and diesel oil. For protecting the EPS various methods are used like clay fills, polyurethane coatings and impermeable membranes. (Thompsett et al., 1995).

The success rate of EPS as an embankment material is inconsistent. There have been five reported failures and the general causes have been due to water fluctuations, buoyancy and fires.

2.2.2 Light Weight Fill from a Mixture of Fly Ash and Bottom Ash

Fly ash and bottom ash are coal combustion by-products. These two by-products are generated in large quantities throughout the world. In USA, about million tons of coal combustion by-products are produced every year. Figure 2.3 illustrates the disposal method of Fly and bottom ash mixture. To reduce the environmental impact of unused Fly ash, different applications have been identified. One of the applications is to use it as a fill material in embankments (Patelunas 1986; Srivastava and Collins 1989).

Figure 2.3 Disposal pond of fly and bottom ash mixture generated at Wabash River Power Plant – Sungmin Yoon et al. 2009

A case study in Terre Haute, Indiana was conducted to demonstrate the effectiveness of ash mixture as fill material in embankment (Sungmin Yoon et al., 2009). The ash mixture was classified as sandy silt (ML) by USCS. After the embankment was constructed it was monitored for a period of one year. Figure 2.4 shows a cross sectional view of the demonstration embankment.

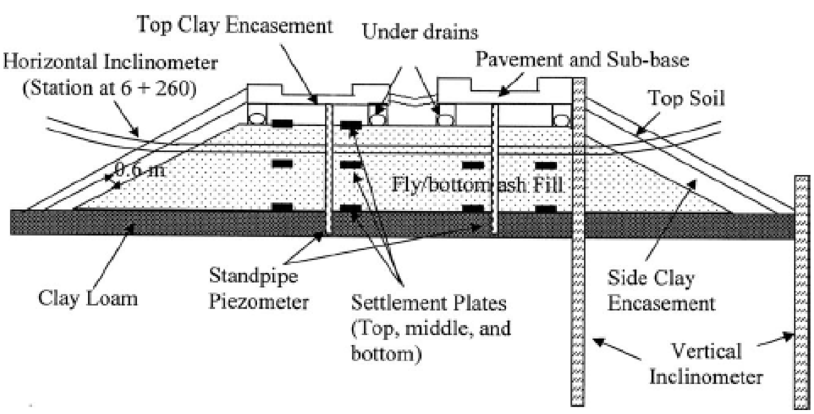


Figure 2.4 Cross section of an embankment at State Road 641,Terre Haute, Indiana - Sungmin Yoon et al. 2009

The study showed a maximum settlement of 80 mm at the bottom of the embankment and the settlement started to stabilize in 5 months after the embankment was constructed.. At the top of the embankment, a differential settlement of about 5 mm was observed while negligible lateral movements were observed at shoulders and toe of the embankment. Sungmin Yoon et al., concluded that the fly and bottom ash mixture used in the construction of the embankment is a viable alternative to conventional fill materials.

Environmental impact studies showed that the leaching of trace metals from fly and bottom ash is a main concern as it can contaminate the ground-water (Alleman et al., 1996). Longer term studies are necessary for more definite conclusions on the environmental impact of ash embankment construction.

2.2.3 Lightweight Tire Scrap

Heavy traffic volume across the world has an unintended by product- scrap tires and researchers are putting lots of effort in finding beneficial reuse of this material. Tuncer and Bosscher, 1994, studied the reuse of discarded tires as a fill and drainage material and observed that the tire chips constrained deformation modulus in elastic range is 1/100 that of pure sand, have high hydraulic conductivity and improves the frictional respose of sand in mixtures. Figure 2.5 presents a photograph of typical tire scrap.

Figure 2.5 Scrap tire beads - An Deng and Jin-Rong Feng, 2009

Aderinlewo and Okine 2008, observed that the tire scrapes have useful properties like high permeability, low thermal conductivity and is lightweight but also has the disadvantages of large settlement, potential for internal heating, and leachate effects.

In a case study in Portland, Maine the tire shreds were used as lightweight fill for construction of two 9.8-m high highway embankments (Humphrey et al. 1998). Tire shreds were chosen because they were cheaper and reused the 1.2 million scrap tires. Wick drains were used to accelerate consolidation of the foundation soils.

The tire shreds were placed on top of the geotextile on the prepared base. After placing the shreds, geotextile separator was placed on the sides and top of the tire shred zone and then the surrounding soil cover. Low-permeability soil was placed on the outside and top of the fill to limit inflow of air and water. The embankment was topped with granular soil and temporary surcharge.

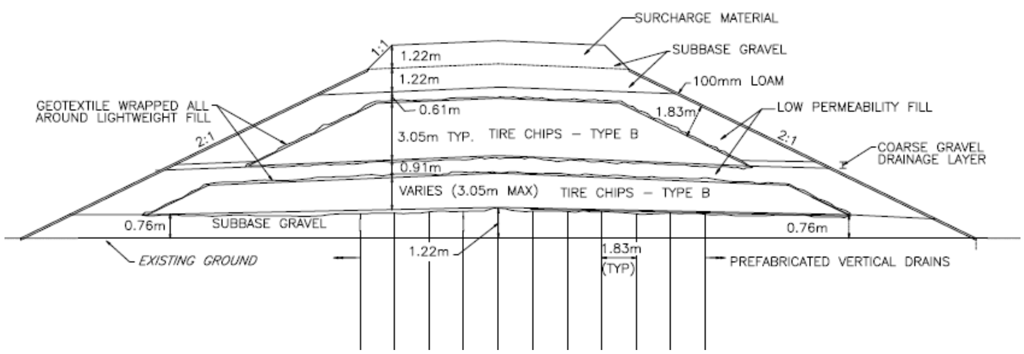


Figure 2.6 Cross section through embankment for the Portland Jetport Interchange - Humphrey et al., 1998

At each tire shred layer settlement plates were installed to monitor settlement. The study concluded that the tire shreds had a high factor of safety against slope instability and they are more economical alternative as a low weight fill material.

2.2.4 Lightweight Aggregate

In the decades following WWII, USA has enjoyed strong social, economic, industrial, and urban growth, which has naturally led to the building of new, modern communication infrastructures between centers of development and on their outskirts. This development boom led to building of roads and railway lines over weak geotechnical formations like soft soil deposits. This prompted substantial research in material science related with lightweight aggregates.

Lightweight Aggregate (LWA) is described as a range of special use aggregates that have an apparent specific gravity considerably below ordinary sand and gravel (ACI-213, 1967). Lightweight aggregates range from extremely light materials used for insulative and non-structural concrete (Vermiculite and Perlite) to expanded clay shale (ECS) used for high performance structural concrete. However the strength of this material is considerably low but has very good insulation properties, high elasticity, absorbs sounds and other vibrations (ESCSI 1994). Use of LWA is environmental friendly as they have lower transportation requirements and use raw materials that have limited structural applications in their natural state. This minimizes demands on finite resources such as quality natural sands, stones and gravels (Ries and Holm, 1993).

There are several types of LWAs, which include granulated / foamed blast-furnace slag, lightweight expanded clay aggregate (LECA), expanded clay or shale (ECS), furnace bottom ash (FBA), pulverized fuel ash (PFA), or the less common pumice (a volcanic material) and vermiculite. However, this study focuses on expanded clay and shale, which will be referred to as lightweight aggregate (LWA). The advantages of LWAs over conventional aggregates include high internal angle of friction, increased thermal insulating capacity, freeze-thaw resistance, and low unit weight. Lightweight aggregate fills are approximately half the weight of normal fills. It has been effectively used to solve numerous geotechnical engineering problems and to convert unstable soft soil into usable land (Holm and Valsangkar, 1993).

Several states use LWA in asphalt surface treatments. The advantages of using LWA in asphalt surface treatments are high skid resistance, extension of roadway life, increased freeze/thaw resistance, increased resistance to stripping, and considerable reduction of windshield, headlight, and paint damage from flying stones. One disadvantage mentioned is that LWAs can roll out of the asphalt under constant breaking or turning conditions (Gallaway, 1997). Texas was the only state that appeared to have conducted considerable research into LWA for highway uses. Moore (1970) investigated the use of LWAs in flexible bases. The study developed the pressure-slaking test and the modified pressure-slaking test for testing the aggregates suitability. The test consists of underwater pressure-cooking followed by severe agitation. The study concludes that a synthetic aggregate is acceptable for use in Texas flexible bases when the loss in gradation from the pressure-slaking test is less than 10 percent or less than 4 percent in the modified pressure-slaking test. In addition, the study concluded that synthetic aggregates suitable for flexible bases in Texas could be produced from clay in rotary kilns (Moore and William, 1970).

Studies were conducted in the Nordic countries concerning LWA use in road construction. However, many papers have not been translated in to English. According to one Finnish/Swedish research group, expanded clay (Exclay, a type of lightweight aggregate) has two main road-related uses, “either for frost insulation purpose in frost susceptible soils or for lightening of road embankments on soft soils to prevent settlement” (Gustavsson et al., 2002).

2.2.4.1 Lightweight Expanded Clay Aggregate

Due to abundance of clay which is a raw material for lightweight expanded clay aggregate (LECA), its use is becoming very popular around the world (Rossignolo at al., 2003).

Figure 2.7 LECA Granular - Arioz et al., 2008

LECA is produced by baking natural clay in a kiln at a temperature of1000-1200 ℃ which introduces swelling in it due to the expanding gases inside the mass (Mladenovic, et al. 2004). The high porosity contributes to the light weight nature of LECA.

Excellent frost insulation property of Expanded clay LWA makes it highly suitable to use in road pavements which have frost susceptible soils. The other important benefits of LECA are use as light weight fill for stability improvement and reduction of settlement.

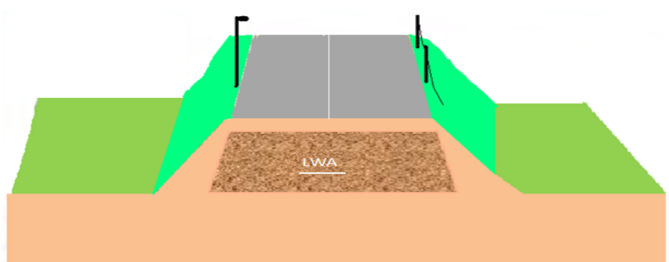


Figure 2.8 Schematic of lightweight embankment system to reduce settlement

2.2.4.2 LWA from Mining Residues, Heavy Metal Sludge, and Incinerator Fly Ash

Huang et al, 2006, experimented with the mining residues, heavy metal sludge and fly ash to create a new type of LWA. The composition of the mining residues which is extracted from a mine in Hualien, Taiwan resembles that of shale. The fly ash used is from the municipal waste incinerator in Taipei, Taiwan. Heavy metal sludge, was taken from a printed circuit board factory in Taiwan. These sludge materials comprise heavy metal hazardous wastes including Cr and Cu (Huang et al, 2006).

The study illustrated the manufacturing process which included raw material preparation, drying, pulverizing, mixing, pelletization, sintering and cooling (Huang et al, 2006). The analysis of composition showed that the chemical composition of the pellets matches the chemical composition of expandable LWA as shown in Figure 2.9.

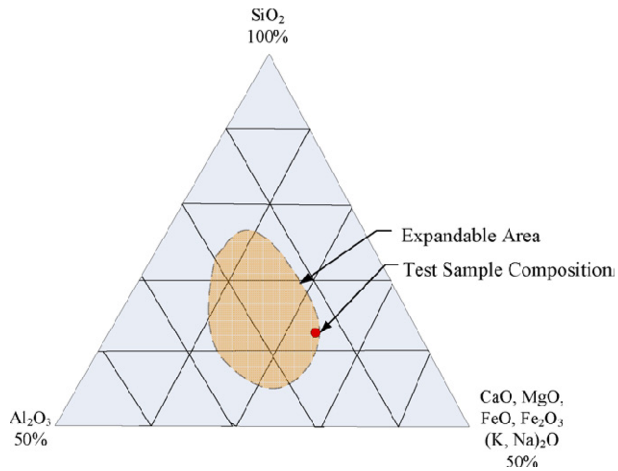


Figure 2.9 Ternary plot of LWA - Huang et al., 2006

The study concluded that above sintering temperature of 1150 ^{0}C, these aggregates become non-hazardous. This new method of utilizing hazardous waste products for creating LWA not only combines the benefits of light weight aggregates but also provide a safe ecological disposal of these waste products.

2.2.4.3 ECS as Lightweight Aggregate

Expanded clay and shale lightweight aggregate due to its property of high strength and low unit weight makes it a reliable and economical geotechnical solution for landfill leachate drainage systems, high-rise building, underground conduits and pipelines for insulation or in unstable soil, insulating backfill and insulating road base, retaining walls and embankments (Holm and Valsangkar 1993 & 2001)..

The benefits of ECS can be summarized as (www.escsi.org):

- Reduces dead loads
- Reduces lateral forces
- Reduces overturning forces
- Provides a high angle of internal friction.
- Free draining
- Water and acid insoluble
- High insulation value
- Chemically inert, and environmental friendly
- High strength and durability
- Readily available
- Controlled gradations
- Easy to handle and install
- Cost effective

ECS is prepared by expanding select minerals in a rotary kiln at temperatures over 1000 ℃ (Holm and Ooi, 2003). The production and ra w material selection processes are strictly controlled to ensure a uniform, high quality product that is structurally efficient, durable and inert, yet also lightweight and insulative (Holm and Valsangkar, 1993 & 2001).

Figure 2.10 ECS Lightweight Aggregate - Saride et al. 2008

The initial cost of ECS lightweight aggregate is usually higher than a comparable normal weight aggregate but the cost offsets when the net savings from long term use is taken into perspective. The cost savings are obtained in the following areas: labor, lower dead loads, better fire resistance resulting in reduced concrete thickness, and less reinforcing required in building frames, girders, piers, and footings (Holm and Valsangkar, 1993 & 2001). ECS is not only economical in long run but also a environmentally sustainable material. Taking these benefits into consideration, ECS has found a huge favor in construction industry today.

2.3 Numerical Modeling

A numerical simulation/modeling of different construction processes is important to ascertain the safety of current and planned construction (Zimmerman et al., 2005). Numerical modeling software offers a unified and generic framework for such computations such as analysis of deformations, stresses, thermal behavior and continuous stability assessment in soils, rocks and structures, including soil structure interaction. The softwares have evolved and now allows us to perform the modeling in 2D and/or 3D. All the stages of construction i.e. from the initial stage to the final construction can be simulated in a single environment (Zimmerman et al., 2005).

According to Joaquin and Sherwin (2005) there are three important steps in the computational modeling of any physical process: (i) problem definition, (ii) mathematical model, and (iii) computer simulation. The first step is to define the problem in terms of a set of relevant measurable parameters. The second step is to represent the idealization of the physical reality by a mathematical model. These mathematical models use numerical time-stepping procedure to obtain the models behavior over time. After establishing the suitable boundary and initial conditions, the third and final step is to proceed to its solution using computer simulation. (Joaquin and Sherwin, 2005).

2.3.1 Modeling for Settlements

The settlement of embankments on soft soils is traditionally an important geotechnical problem and has been extensively studied by a large number of researchers. Excessive settlement can render the highways unserviceable, increase the cost of maintenance, and even be detrimental to the stability of embankment. Therefore it's very important to accurately evaluate the embankment settlement during the highway construction design (Murray, 1971).

The soft soil properties of high compressibility and low permeability introduces large embankment settlement over time as excess pore pressure dissipates embankment loading is applied. (Biot, 1941; Bowles, 1979; Borges, 2004).

For predicting the behavior of embankment on soft ground, one of the key point is to simulate the consolidation process. The consolidation rate is mainly influenced by the foundation soil permeability. The permeability of soft ground varies during the loading and consolidation process, and significant changes occur before and after the soil yields (Tavenas et al. 1978, 1980).

Available methods for calculating embankment settlement are the layer-wise summation method (LSM), empirical formulation method, finite element method (FEM), etc.

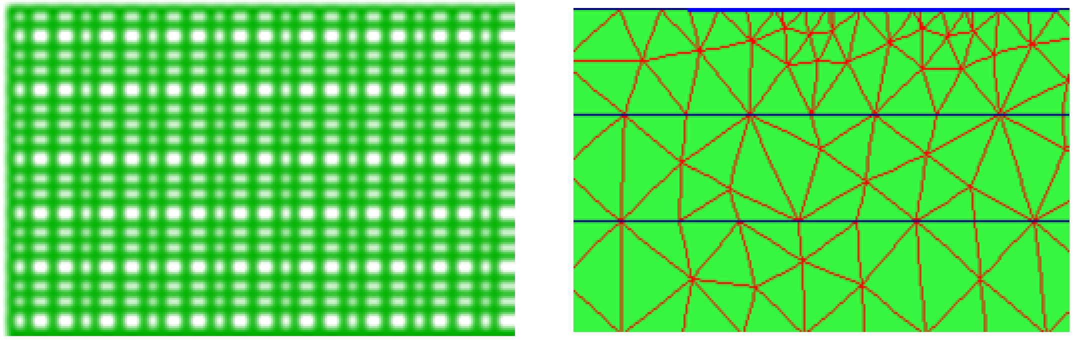
(Qian and Yin, 1996; Wang, 2004). Of these methods, FEM is a very powerful numerical tool for solving complicated 2D or 3D consolidation settlement problems. It can handle arbitrary boundary conditions, different loading schemes and it considers the coupling effects of loading and soil consolidation (Zienkiewicz and Taylor, 1989; Taiebat and Carter, 2001; Wang, 2004).

For modeling long term settlements in this case study, both FEM and FDM method have been used. PLAXIS software utilizes FEM while FLAC software utilizes FDM.

2.3.2 Differences between Finite Element Method and Finite Difference Methods

A finite difference method (FDM) discretization is based upon the differential form of the Partial Difference Equation to be solved. It utilizes a point-wise approximation to a solution. The domain is discretized into a grid of hexahedral cells or nodes. The solution will be obtained at each nodal point. Although FDM is easy to implement and the compute time for each step is fast, however the number of steps required for convergence is high. The other disadvantage is that the domain is not accurately represented if the domain is discontinuous or non-rectangular in shape (Fausett, 2003). Fig 2.11 (a) illustrates the FDM mesh.

A finite element method (FEM) discretization is based upon a piecewise representation of the solution in terms of specified basis functions. In FEM the discretization is not restricted to a grid of hexahedral cells or nodes, instead, a solution is approximated using interconnecting sub-regions or elements. These elements are typically simple geometrical figures as illustrated in Fig 2.11 (b). This flexibility of construction of elements in FEM allows it to accurately model the complex geometries. The downside is that FEM is difficult to implement but the opinions vary on this (Zienkiewicz and Cheung, 1965).



(a) (b)

Figure 2.11 Discretization methods (a) FDM (b) FEM

2.4 Instrumentation

The main objective of this section is to present a brief review on instruments used in this study. Instruments provide actual field data and helps in evaluating the movement and failures under actual field conditions. The instrumentation was used in this study to compare the performance of the LWA vis-a-vis normal fill. The choices of instruments depend on many factors like geological understanding of the area, subsurface material and the groundwater depth. The other factors include the transportation facility, the rate and magnitude of movement and type of movement i.e. horizontal or vertical movement or both. For large and fast movements relatively crude instrumentation can be used but if the movement is slow and small instrumentation accuracy and the repeatability of its measurement takes precedence (Machan and Bennett, 2008)..

2.4.1 Characteristics and Instrumentation Details

Field instrumentation is very important in geotechnical engineering and therefore, geotechnical engineers should have proper knowledge of instrumentation. But instrumentation is not answer to everything so its use must be prudent. The wrong type of instruments or wrong placement of instruments can provide wrong or confusing results which can divert the attention from real problems (US Army Corps of Engineer, EM 1110-2-1908).

The objectives of a geotechnical instrumentation plan are grouped into four categories: analytical assessment; prediction of future performance; legal evaluation; and development and verification of future designs.

2.4.1.1 Analytical Assessment

The following objectives are met during the analytical assessment (US Army Corps of Engineer, EM 1110-2-1908).

(1) *Verification of design parameters* – Observed data from instrumentation can be used to not only verify the selected design parameters but also used to modify and refine the future designs.

(2) *Verification of design assumptions and construction techniques* – The data obtained from satisfactory actual performance of new or modified design can help its chances of acceptance.

(3) *Analysis of adverse events* – Causes of various types of failures and deformations at the project site can be uncovered from the precious instrumentation data.

(4) *Verification of apparent satisfactory performance* – The instrumentation data of satisfactory performance as well as adverse events can prove to be valuable records for future design purposes and development of new and innovative technology.

2.4.1.2 Prediction of Future Performance

Instrumentation data analysis can be used for the predictions such as continued satisfactory performance or indicating a sign of potential future distress. The instrumentation data recorded during and after events like rainfall can be very useful for future performance (US Army Corps of Engineer, EM 1110-2-1908).

2.4.1.3 Legal Evaluation

Instrumentation data can be used in determining causes or extent of adverse events so that the merit of various legal claims can be evaluated (US Army Corps of Engineer, EM 1110-2-1908).

2.4.1.4 Development and Verification of Future Designs

Analysis of the performance of existing normal fill and LWA fill embankments and instrumentation data generated during operation, can be used to further the construction technology. Instrumentation data from existing embankment construction projects can provide safer and economical design for future construction of embankments (US Army Corps of Engineer, EM 1110-2-1908).

2.4.2 Different Types of Instruments Used in the Field Monitoring

There are various field instruments like Extensometers, Inclinometers, Piezometers, Soil Stain Meter, Pressure cells and settlement cells to measure soil movements, settlements, strains, deformations, pore pressures in the soil. They also warn us about the design assumptions suitable for the given field conditions. The following sections present the details of instruments used in the current study and the installation procedures followed.

2.4.2.1 Inclinometers

Inclinometers are used to measure the displacement normal to the axis of a pipe. The inclinometer probe which contains a gravity sensing transducer is inserted into the casing pipe. This transducer is designed to measure inclination with respect to the vertical axis. The casing pipe may be installed either in the borehole or in fill, in most cases in a near vertical alignment, to provide horizontal deformation. Due to the sensitive nature of the probes, they should be handled carefully during storage and transportation (Dunnicliff, 2003).

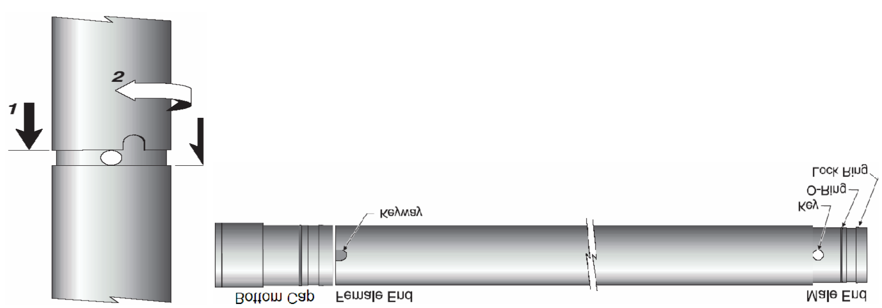


Figure 2.12 Details of inclinometer casing and assembling procedure - Slope Indicator, 1997

2.5 Surface Elevation Surveying

The surface surveying consists of establishing benchmark control stations and survey points at various locations of interest. The survey points are periodically measured to monitor for potential deformations at test section and control sections. For this technique, the control stations are established on stable ground. Surface monitoring is accurate to a resolution of approximately 0.25 in. (6 mm). However, long term deformation trends can be difficult to establish, due to personnel turnover, susceptibility to damage of control points, etc. This method provides essentially provides the relative movement of the pavement surface relative to its' initial position (i.e. settlement or heave).

2.6 Visual Monitoring

This approach consists of occasional on-site inspections. Field inspection is a good technique to identify signs of distress or change. However, visual observations are typically not accurate method in detecting small movements or long-term settlements. Periodic good resolution photographs may be a good idea for comparison.

2.7 Summary

This chapter discusses various causes of settlement, its remedial measures, different types of fill materials and finally the instrumentation used in determining the horizontal and vertical movement. It is observed that among all the different types of embankment fill materials, ECS is the most suitable for Texas since it is easily produced and very economical. For this study Vertical Inclinometers were used to determine horizontal movement and theodolite for vertical surface movement.

CHAPTER 3

EXPERIMENTAL STUDIES

Laboratory testing is an essential aspect of geotechnical engineering. Laboratory test are conducted using ASTM standard procedures. Material properties like cohesion unit weight, internal friction angle etc can be determined from different laboratory tests. Visual observation and field test results are the primary source of information before doing laboratory testing. Laboratory test results provide a clear understanding and knowledge of soil properties.

3.1 Quality Control for Laboratory Testing

Quality control for Lab Testing involves proper handling of samples while transporting and storage, Its important to maintain quality otherwise it can result in misleading test results. . In addition, the following guidelines given by Mayne, et al., (2002) for laboratory testing of soils was followed.

3.1.1 Tests Conducted

To study the behavior of LWA and Normal fill, Laboratory tests namely, sieve and hydrometer test, Atterberg limits tests, proctor's compaction tests, direct shear tests, Swell tests, and Consolidation tests were performed.

The soil used for various tests were collected from site and then wrapped in a plastic bags and transported to the laboratory.

3.1.1.1 Sieve Analysis and Hydrometer Analysis

Sieve analysis was done as per ASTM D22. There are two different procedures for dry and wet sieving. For our study, the sample collected was in the form of hard lumps and the sample was pulverized to very fine particles before doing dry sieve analysis. For normal fill material, first dry sieving was performed, where the sample was oven dried and allowed to cool. An oven dried sample of 1000 grams passing through 4.75, 2.0, 1.8, 0.6, 0.425, 0.25, 0.125, 0.075 mm sieves was taken. The pan was attached at the bottom of the sieve stack. The sample was poured on the top sieve and stirred for about 10 minutes. Soil retained at each sieve was measured and weighed. The weights of the sample on all the sieves were added to compare it with initial sample weight. The difference shouldn't be more than 1%.

Secondly, wet sieving was performed where oven dried sample was kept soaked in a tap water for about 2 hours. The sample was transferred to 200 sieve. The sample was washed thoroughly, discarding the material passing no. 200 sieve. The retained material collected from no. 200 sieve was oven dried and weighed it after it has cooled. Difference between dry weight before and after washing was around 50% recorded.

Hydrometer analysis was performed on the same sample which was finer than no. 200 sieve size. The lower limit of the particle size determined by this procedure is about 0.001mm. Hydrometer analysis test procedure was adopted as per ASTM D-422.

For ECS, the material had less than 5% fines, so wet sieve analysis was not done. An oven dry sample of 500 grams passing through 4.75, 2.0, 1.8, 0.6, 0.425, 0.25, 0.125, 0.075mm sieves was taken. Soils retained on various sieves were calculated and a particle distribution was obtained. Figure 3.1 shows the particle distribution curve for the ECS materials.

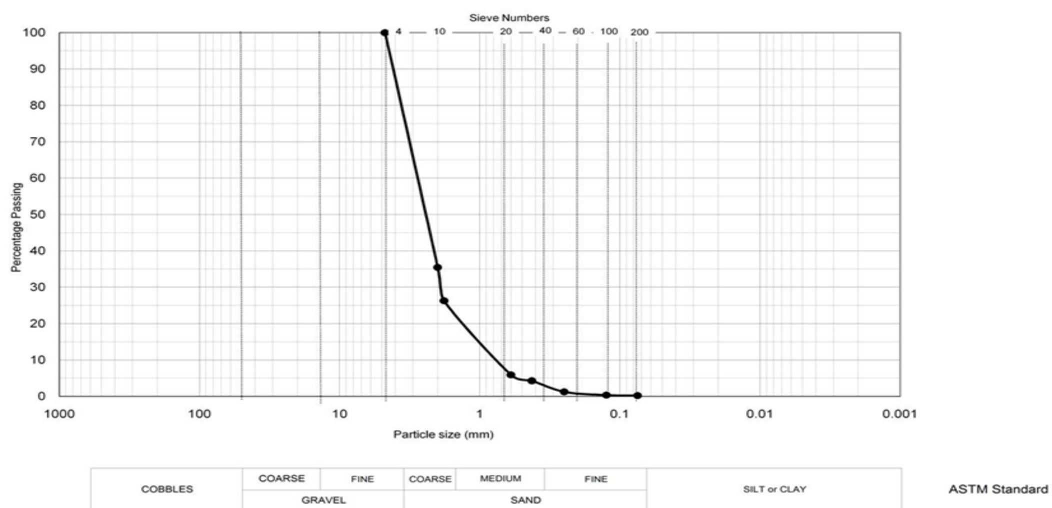


Figure 3.1 Particle Size Distributions for ECS

3.1.1.2 Atterberg Limits

For classification of soil, Atterberg Limits tests (Liquid Limit and Plastic Limit) were performed on the collected sample. The liquid limit test of a soil was performed using casagrande liquid limit apparatus. About 250 grams of air dried sample passing sieve no. 40 was used for both liquid and plastic limit test. Distilled water was preferred instead of tap water to avoid ion exchange between soil and water impurities, which may affect the soil plasticity. Place about 50 grams of soil paste in a cup, level off with the spatula the top surface symmetrically to give maximum depth of 1 cm. The grooving tool was used to straight groove through the soil paste along the diameter through the center of the hinge. The handle was turned at a rate of 2 revolutions per second and counted the number of blows until the two parts of the soil come in contact at the bottom of the groove. About 15 grams of soil was transferred in the container to determine the water content by oven drying. The test was repeated at least 3 to 4 times. The flow curve was plotted to represent the number of blows on logarithmic scale and corresponding moisture content.

The plastic limit test was done to determine plasticity of the soil. Here 30 grams of soil passing sieve no. 40 was used. Distilled water was used to mix water thoroughly in to the soil. 10 grams of plastic soil mass was used to form a ball and then roll in to thread with the fingers on the ground glass plate. Keep on rolling until the thread starts to crumble at a diameter of 3mm. The crumbled thread was kept in a container for moisture content determination. The processes were repeated 2 more times with fresh sample and the average of three moisture contents was obtained to calculate plastic limit. After determining liquid limit and plastic limit, the plasticity index was calculated and plotted on the plasticity index chart to know the type of soil. Figure 3.2 presents the test results obtained for normal fill.

Table 3.1 Summarizes the results obtained from both Grain Size Analysis and Atterberg's test

Material Type	% Gravel	% Sand	% Silt	% Clay	Liquid Limit	Plastic Limit	Plasticity Index	USCS Classification
NF-1 (North Side)	0.2	44	40	15.8	35	14	21	CL
NF-2 (North Side)	0.2	41	35	23.8	36	15	21	CL
ECS (South Side)	65% Coarse Sand 25% Medium Sand 10% Fine Sand				-	-	-	

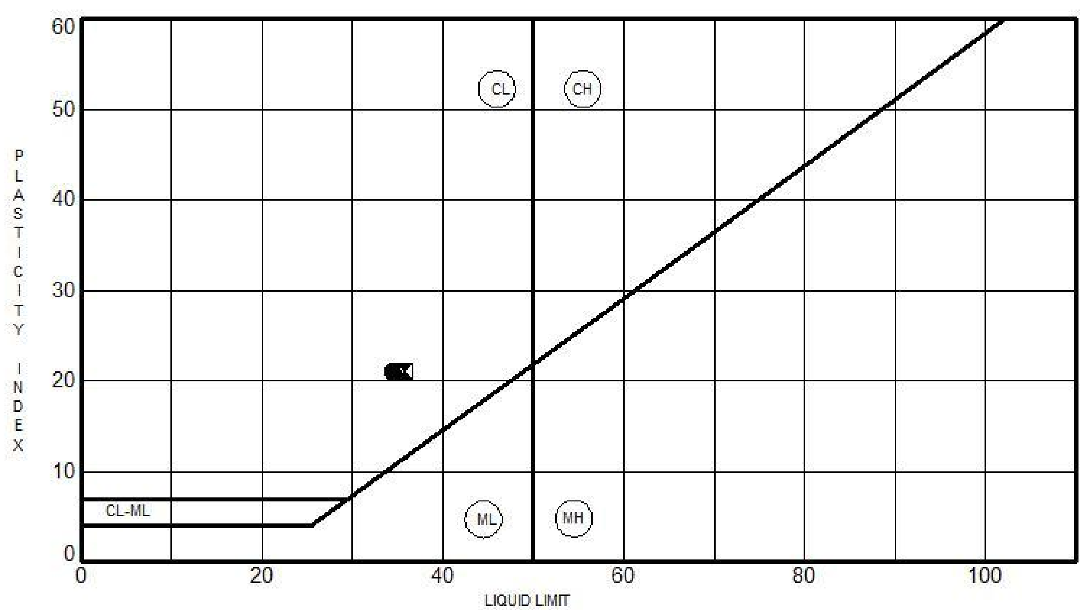


Figure 3.2 Plasticity Chart for Normal Fill

3.1.1.3 Compaction Test

Compaction characteristics were determined according to ASTM D698 (Standard Procter Test). Laboratory compaction tests are used to determine the relation between water content and dry weight and to find the maximum dry unit weight and optimum water content.

For each sample the required amount of sample was air dried and weighted and the mass of material required is around 3kg passing no. 4 sieve. A suitable amount of water was added to the dry soil to obtain the desired moisture content. The soil sample was evenly distributed so that the mold is about half full. The sample was compacted by applying 25 blows of the rammer dropped from a controlled height of 300mm and the rammer was positioned properly before releasing. The soil sample is evenly compacted in to 3 layers following the same procedure. The extension collar and the base plate were removed carefully and weight of soil and mold was taken. The soil was extracted from the mold using extractor and immediately three representative samples were taken from the soil to determine the water content. The process was repeated to obtain more compaction points. Figure 3.3 shows the compaction test data comprising of optimum moisture content and dry densities for the normal fill materials.

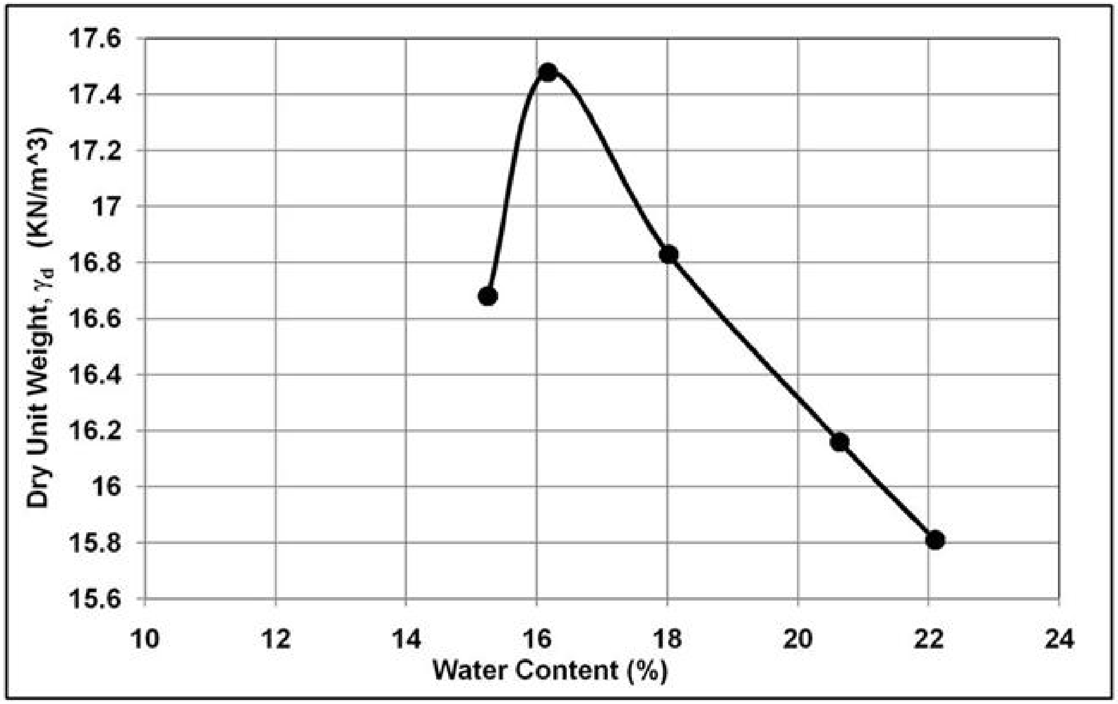


Figure 3.3 Compaction Curve for Normal Fill

3.1.1.4 Direct Shear Test

A series of direct shear tests were performed on ECS material according to ASTM D 3080 method. ECS samples were well compacted and were placed in a 2.5 inch shear box. Normal stresses of 50, 100 and 200kPa were applied on the samples and then the samples were sheared. Normal stresses were applied with the help of a loading ram. Shearing was applied with the help of a horizontal ram. This set up was connected to the computer where all the graphs were plotted between shear stress and shear displacement. Figure 3.4 and 3.5 shows the graph between measured shear stress and applied normal stress. The calculated peak friction angle of the ECS is 49.5º and the cohesion is 75 kPa. The same for normal fill are 85 kPa and 18 degrees respectively.

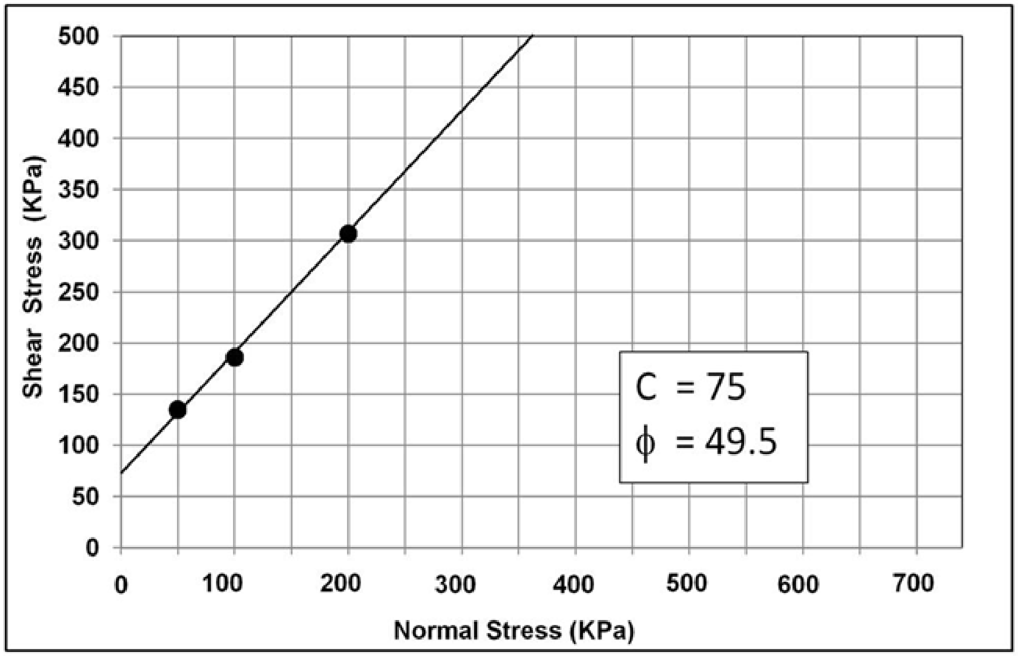


Figure 3.4 Direct Shear Test results for ECS material

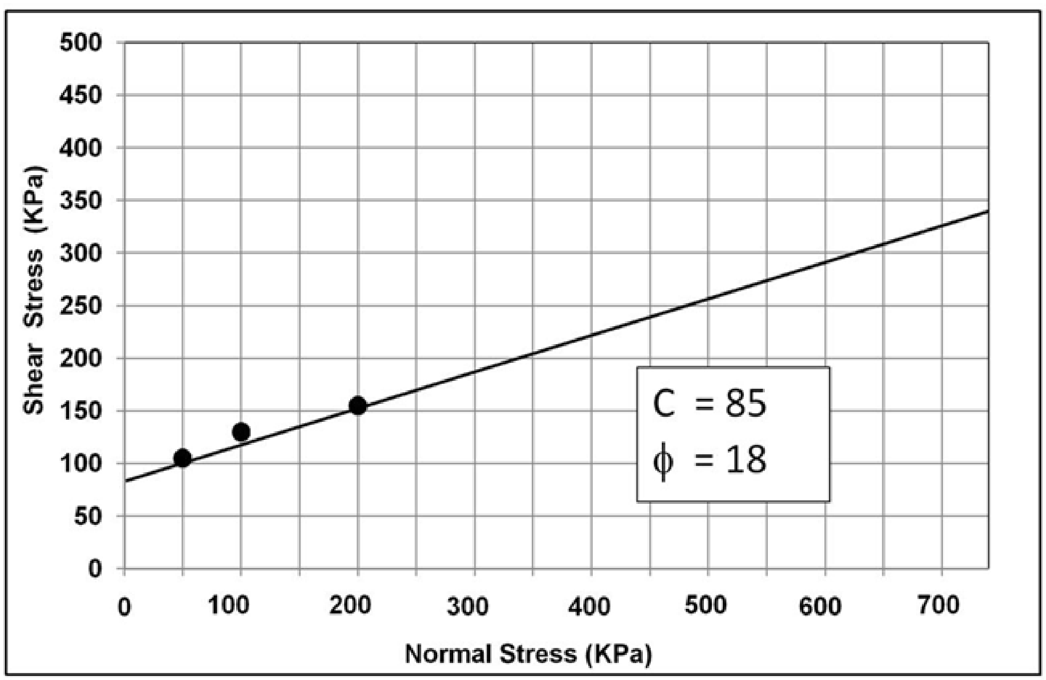


Figure 3.5 Direct Shear Test results for Normal Fill Material

3.1.1.5 Consolidation Tests

Consolidation tests on the two materials, Normal fill and ECS were performed. The test was conducted according to ASTM D2435. The soil sample is kept inside a stiff steel ring which blocks its lateral expansion and two porous stones allow the water drainage. The sample is fully submerged in water and remains fully saturated during the test. Sample was prepared in a 2.5 inch diameter and 1 inch in height ring and was then placed for testing in an oedometer. An initial sitting pressure of 35kPa was applied on the sample and then the sample was consolidated for 24 hours. After 24 hours of consolidation, the initial sitting was applied such that it shows no swelling. Dial gauge reading was noted under initial sitting pressure. The first increment of the load 1000psf was applied and the readings were recorded at certain time intervals for 60 minutes. The particular sequences of loading and unloading of applied pressure are 1000, 2000, 4000, 8000, 16000, 32000, 16000, 8000, 4000 and 1000 psf. This range of applied pressure completely covers the effective stresses that are needed for settlement calculations. This range encompasses the smallest and largest effective stresses in the field. This experimental result shows the response of different materials subjected to loading-unloading cycles. Here two materials – ECS and Normal fill, behaves differently during loading and unloading phases.

Figure 3.6, 3.7 and 3.8 shows the graphs between void ratio versus logarithm of effective pressure for the ECS material and the Normal fill. Compression index C_c for the ECS material was found to be 0.14. The lower compression index value for the ECS material was expected as it's a granular fill material which is around 0.02 to 0.05. Table 2 gives the values for the compression and recompression indices for the both materials. From the table, C_c for Normal fill material has a C_c value of 0.12 which indicates that it is low plastic soil and with high compressibility which may result in high settlements on the pavement surface and can cause undulations.

Table 3.2 Compressibility Coefficients

Compressibility Coefficients	Normal Fill	ECS
Compression index, Cc	0.12	0.14
Swelling Index, Cs	0.033	0.009

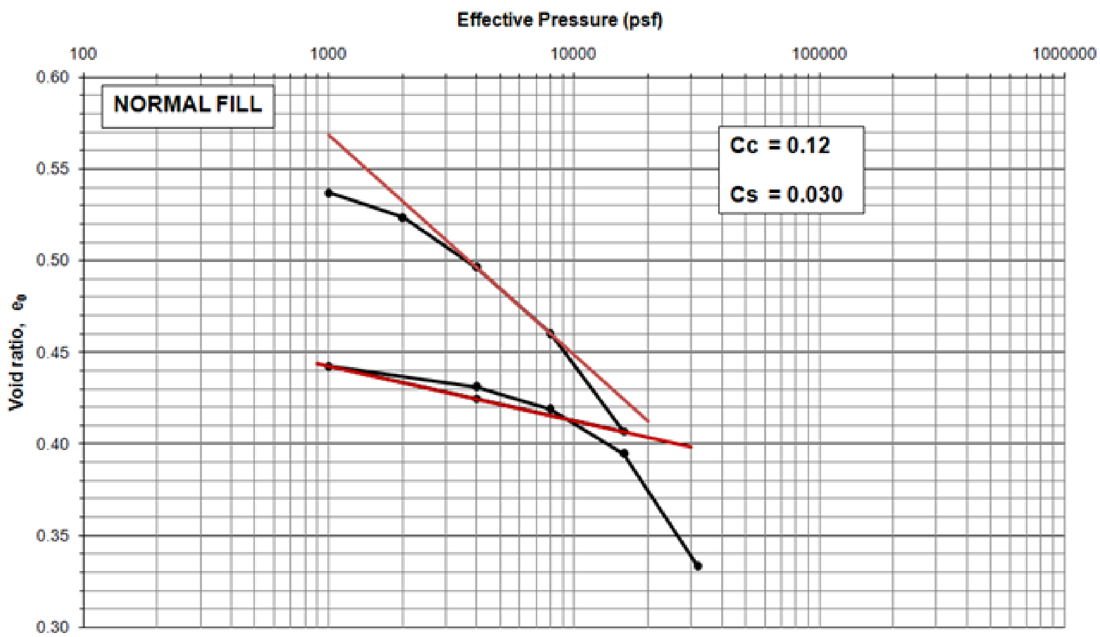


Figure 3.6 e – log (p) graph for Normal Fill material

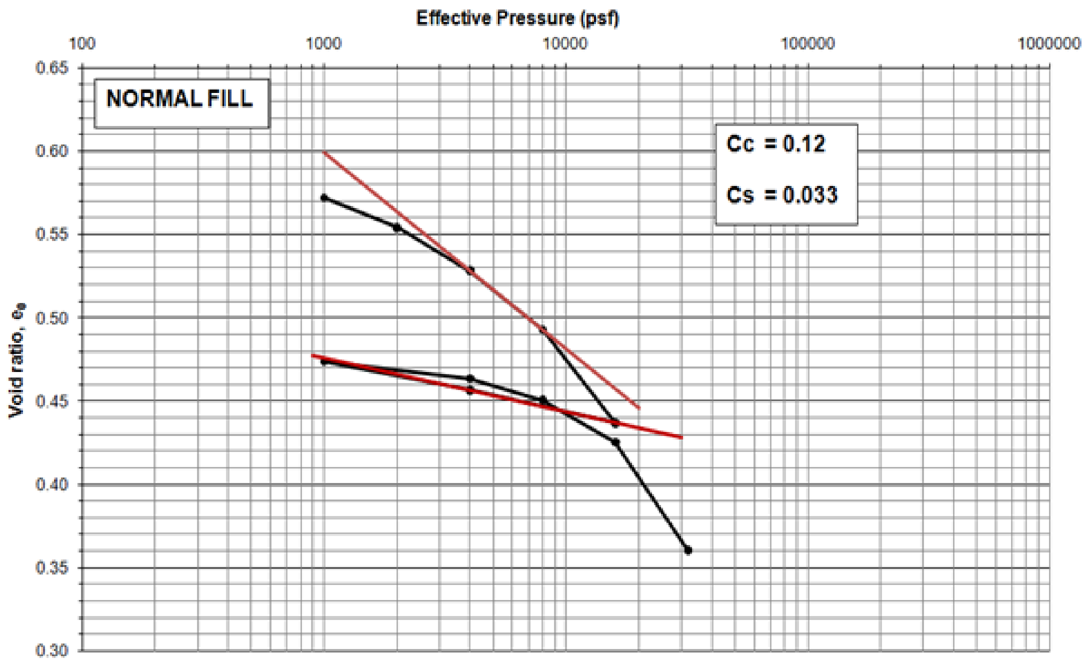


Figure 3.7 e – log (p) graph for Normal Fill material

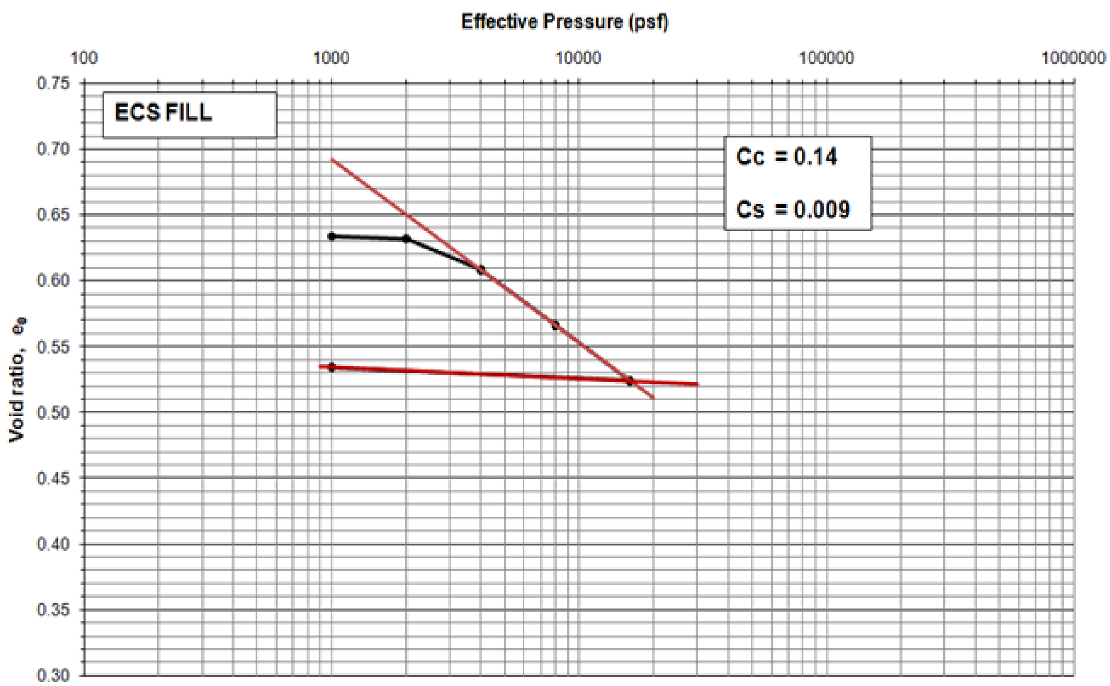


Figure 3.8 e – log (p) graph for ECS material

3.1.1.6 Vertical Swell Tests

One-Dimensional Free Swell Test measures the amount of heave in the vertical direction of a laterally confined specimen in a rigid ring. The swelling characteristics of these materials are important to understand the swelling related movements due to climate fluctuations. To understand the swelling nature of both of the materials, vertical free swell tests were conducted on both the ECS and normal fill samples as per as per the ASTM D4546. A conventional oedometer steel ring of size 64 mm (2.5 in.) in diameter and 25 mm (1 in.) in height was used. The inner face of the consolidation ring is lubricated to minimize the friction during free swell.

Readings were taken for 24 hours and the swell strain were calculated. The magnitude of the vertical free swell strain for the normal fill was found to be 2.5% and for the swell strain, it was observed to be negligible for the ECS material. This confirms that the normal fill soil is a moderate swelling material whereas the ECS is a non-swelling material.

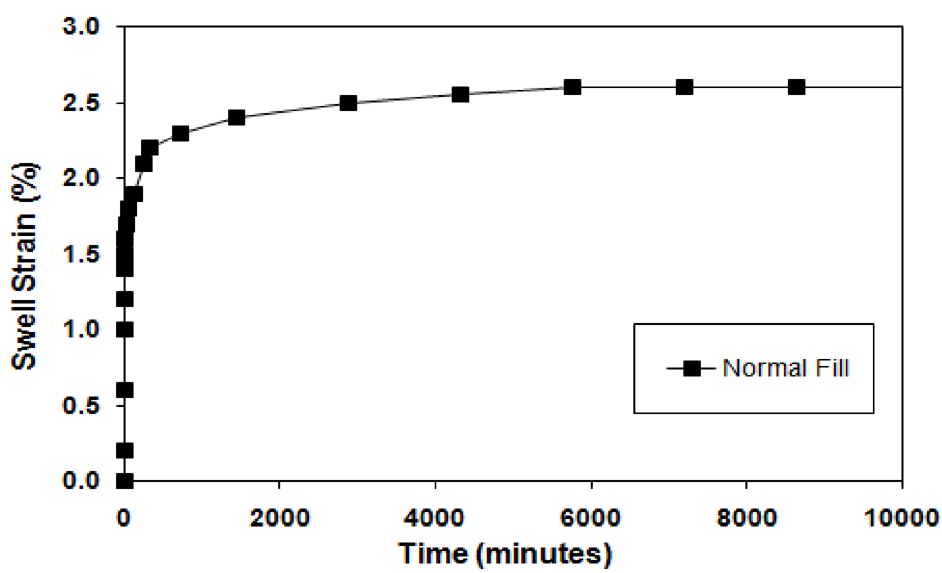


Figure 3.9 Free Swell test graph for Normal Fill

3.1.2 Selection of Design Properties

The lab test results cannot be trusted completely before doing a comprehensive assessment of quality and consistency of the data. Only if the data appears to be consistent with the expectations it should be accepted else the inconsistencies and poor data should be identified and eliminated and then retesting should be done (Geotechnical Design Manual, 2010).

Past knowledge and experience of the area surrounding the test site also comes in handy, since it helps the geotechnical designer to make educated engineering judgement while selecting an average, typical or design value for the selected property.

Normal Fill Material. The normal fill materials are likely to be clay with low plasticity (CL) and contain enough fines to be moderately moisture sensitive. Local material is not all weather material.

Direct shear strength testing on local material that meets gradation requirements results in undrained friction angles of 18 degrees and cohesion value of 85kPa.

The common normal fill material can be of CL, ML, SM, GM, SP-SM, GP-GM with unit weight of 17 to 21kN/m^3 and a friction angle between 15 to 38 degrees. Normal fill with fines content may sometimes be modeled as having an apparent cohesion value from 5 to 95kPa. If a cohesion value is used, the friction angle should be reduced so as not to increase the overall strength of the material. For long term analysis, the normal fill material should be modeled with less cohesive strength.

Low Weight Aggregate. These materials are often used as fill material for embankments. For design purposes, typical values of 6 to 11kN/m^3 for the unit weight and internal angles of friction of about 40 to 50 degrees as an undrained parameter should be used. ECS aggregates provide a practical, reliable and economical geotechnical solution (DeMerchant and Valsanger, 2002). Table 3.3 shows the general engineering properties of ECS (after ESCSI, 2004).

Table 3.3 General Properties for ECS (ESCSI, 2004)

Aggregate property	Measuring method	Test method	Commonly specifications for ECS	Typical for ECS aggregates	Typical design values for ordinary fills
Soundness Loss	Magnesium Sulphate	AASHTO T 104	<30 %	<6 %	<6 %
Abrasion Resistance	Los Angeles Abrasion	ASTM C 131	<40 %	20 – 40%	10 – 45%
Compacted Bulk Density	Density Test	ASTM D 698	<70 lb/ft^3	40– 65 lb/ft^3	100-130lb/ft^3
Stability	Direct Shear Test & Triaxial (CD)	ASTM D 3080 & Corps of engineers EM 1110-2-1906	According to project	35º - 45º	30º - 38º (fine sand-sand & gravel)
Loose Bulk Density	Loose	ASTM C 29	Dry <50 lb/ft^3 Saturated<65 lb/ft^3	Dry 30-50 lb/ft^3	89-105 lb/ft^3
pH	pH meter	AASHTO T 289	5 – 10	7 – 10	5 - 10

Soft Soils Material. Soft soils are characterized by very low strength, very high compressibility and having very important time-consolidation effects. These soils pose special challenges for the design of engineering transportation projects and therefore require careful consideration regarding settlement and stability of embankments.

Vertical settlement is also a major concern while constructing embankments on soft soils. Estimation of settlement and the time period for it to occur can be derived from laboratory consolidation tests. Secondary compression must always be evaluated when estimating long-term settlement. Compression index values based on vertical strain (C_c) typically range from 0.1 to 0.3 for organic silts and clays, and are generally above 0.3 to 0.4 for soft clay. The coefficient of secondary compression (C_α) is typically equal to 0.06 x C_c for soft clay respectively (Geotechnical Design Manual, 2010).

3.2 Summary

This chapter discusses different types of engineering tests to ascertain the properties of ECS and Normal fill materials. Typical tests that were performed are sieve analysis and hydrometer test, Atterberg limits test, compaction test, direct shear test, consolidation test and swell test. The quality and consistency of the laboratory test data was reviewed and determined if the results are consistent with expectations.

CHAPTER 4

NUMERICAL MODELING

The use of numerical modeling to predict soil deformation and stresses has been practiced for years. During the numerical analysis, detailed site-specific properties of the road and embankment systems were incorporated to simulate complex construction sequences. The main field of application of constitutive models is the execution of numerical calculations by means of appropriate methods such as Finite element or Finite difference methods.

4.1 Modeling Methods

Numerical modeling using linear model has a benefit of fast estimate of the material response but the downside is its limited accuracy. Therefore the application of linear model is limited to cases where the stress or deformation states of a soil mass are of interest. For getting reliable description of the soil behavior it is necessary to employ nonlinear models (GEO5 - Theoretical Manual, 2010).

The non-linear models can be divided into two groups. The first group of models originates from the Mohr-Coulomb failure criterion. The Mohr Coulomb model belongs to this group. According to Coulomb C.A., 1776, “the Mohr–Coulomb failure criterion represents the linear envelope that is obtained from a plot of the shear strength of a material versus the applied normal stress”. The second group of material models is represented by the Modified Cam-Clay model. This model is based on the concept of critical state of soil (GEO5 - Theoretical Manual, 2010). Accurately modeling the constitutive behavior of the soil is difficult because of the complexity involved in the selection of design parameters and the soil properties. Simplified non-linear models such as Mohr-Coulomb, or more advanced such as Modified Cam-Clay and the Hardening Soil Model can be used with some degree of accuracy (Ng and Lings, 1995). .

4.2 Finite Element Modeling Procedure

Numerical modeling enables the designer to study the effects of embankment loading, surcharge and the soil behavior in various conditions without resorting to simplified assumptions. Two dimensional finite element modeling of the different fill embankments is performed using PLAXIS 2D software.

A parametric study was performed to arrive at the critical parameters that define the behavior of the system. The parametric study comprised of all the elements which had an influence on the behavior of the system. The various aspects like lateral movements and the total settlements were studied. The standard units length (m), force (N) and time (days) were used. The geometry was drawn using geometric lines and standard fixities were then used to define the boundary conditions.

The properties of different soil material sets were created and assigned to material model. After the model was created and material models were assigned, finite mesh was generated using different mesh settings. The following sections present the details of the finite element model including the choice of material models, finite element mesh, and boundary and loading conditions that were adopted to simulate field conditions and obtain settlements of the embankments.

4.2.1 Choice of Constitutive Model and Material Properties

In the material set, type of material and type of model from material model box can be selected. In order to simulate the behavior of the soil, a suitable model and appropriate material parameters must be assigned to the geometry. In PLAXIS, soil properties are collected in material data sets and the various datasets are stored in a material database. The material properties used in the model for different material types are presented in Table 4.1 and Table 4.2.

The selected project is a bridge embankment constructed on soft soil, Arlington, Texas. The embankment is found on a 5.5m thick soft clay layer followed by 2.5m of well graded sand layer, followed by sand stone. Typically, soft clay has drained shear strength value ranges from 40 to 50kPa (Bowles 2000). Hence, the soft clay layer over sand was modeled with an effective cohesion of 45kPa. The embankment section included 4 in. thick hot-mix asphalt concrete (HMAC) and 11 in. of reinforced concrete pavement working platform above the existing ground.

A plate element was used to simulate the pavement layer, which is placed on top of the embankment. The properties of the pavement layer are obtained from the literature and they are entered in a material set as a Young's modulus value of 30GPa and a thickness of 0.35 m (for road). The material properties of subsoil and pavement layers which are used in the current modeling analysis are listed in Tables 4.1 and 4.2.

Table 4.1 Properties of subsoils used in Finite Element Analysis

Sub-Soil Properties	Soft Clay	Sand (well graded)	Sandstone
	Drained	Drained	Un-drained
Compression Index, Cc	0.34	-	-
Swelling Index, Cs	0.043	-	-
Initial Void Ratio, e_0	1.3	-	-
Young Modulus, E (MPa)	-	2.0E+7	2.13E+10
Poisson Ratio, μ	-	0.25	0.2
POP (kPa)	35 - 48	-	-
Unit weight, γ (kN/m^3)	18.85	17.65	26.47
Cohesion, c (kPa)	45	-	27200
Friction Angle, ϕ (deg.)	-	33	28
Dilation Angle, ψ (deg.)	-	-	-
Model	Soft-Soil Model	Mohr-Coulomb	Hardening Soil Model

Table 4.2 Pavement properties used in Finite Element Analysis

Pavement Properties	Concrete
EA	1.116E+11
EI	1.179E+09
μ	0.15

The numerical model has been analyzed with 2D plain strain model considering only half of the section due to the symmetry of the problem. A total width of 52m has been used which starts from the center of the embankment. The geometry of an embankment has been made using geometric line option with approximate dimensions as per the field dimensions. The LWA and Normal fill embankments with 1:2 (V:H) slopes were modeled in two dimensional plain strain method of PLAXIS. A maximum traffic load of 40kN was used for loading on the embankment.

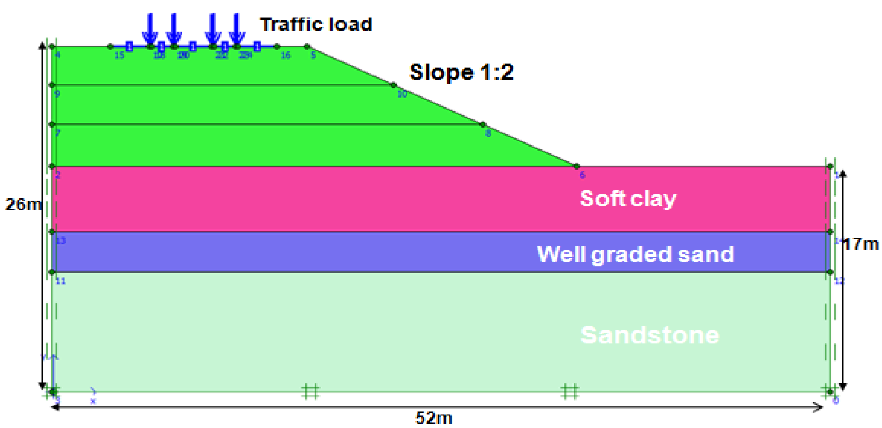


Figure 4.1 Geometry of embankment model in PLAXIS

Two cases were investigated in the finite element analysis to evaluate the influence of the following:

1. Vertical settlement on control section and test section using Mohr Coulomb Model (MCM) for embankment fill

2. Vertical settlement on control section and test section using Cam-Clay Model or Soft-Soil Model (SSM) for embankment fill.

Here the actual settlement behavior of the embankment over soft soil is expected to be between these two cases.

The soft clay, the sand and the embankment fill were modeled as elastic-perfectly plastic materials. No deformation below the sand layer was assumed. Mohr-Coulomb failure envelope was used as the failure criterion in first case and critical state based soft soil (similar to CAMCLAY) failure criterion was used in the second case. The material properties of the embankment materials for Mohr-Coulomb Criteria and Soft-Soil Criteria are provided in Table 4.3 and Table 4.4, respectively. The elastic modulus adopted for the normal fill was typically between 7 to 21MPa (Bowles 2000).

Table 4.3 Embankment Material Properties – Mohr Coulomb Model, MCM

Embankment Properties	ECS (Undrained)	Normal Fill (Undrained)
E (MPa)	50	21
μ	0.25	0.35
K (MPa)	33.33	23.3
G (MPa)	20	7.7
Unit weight (kN/m^3)	7.85	17.2
c(kPa)	75	85
ϕ (deg.)	49.5	18
ψ (deg.)	18	5
Model	Mohr-Coulomb (MCM)	Mohr-Coulomb (MCM)

Table 4.4 Embankment Material Properties – Soft Soil Model, SSM

Embankment Properties	ECS (Drained)	Normal Fill (Drained)
Cc	0.05	0.12
Cs	0.0	0.033
e_0	1.0	0.5
E (MPa)	50	21
μ	0.25	0.35
Unit weight (kN/m^3)	7.85	17.2
POP	58	52
Model	Soft Soil Model (SSM)	Soft Soil Model (SSM)

4.2.2 Mesh Generation

Two types of triangular elements are used in the PLAXIS, 6 noded triangular elements and 15 noded triangular elements. Advantages of higher order triangular elements is that they provide better representation of the description of continuous strain and stress variations and also provides good description of a continuous displacement field with relatively few elements. The disadvantages of higher order elements is that the failure loads may be dependent on the mesh and makes poor description of discontinuous stress and strain. In PLAXIS, the program automatically creates unstructured mesh as there is no possibility of making a so-called structured mesh. The mesh size cannot be set explicitly. The mesh is generated based on random seeds. The mesh size may be changed globally by means of global coarseness and locally by means of local coarseness. Figure 4.2 presents a typical mesh generated for the current analysis.

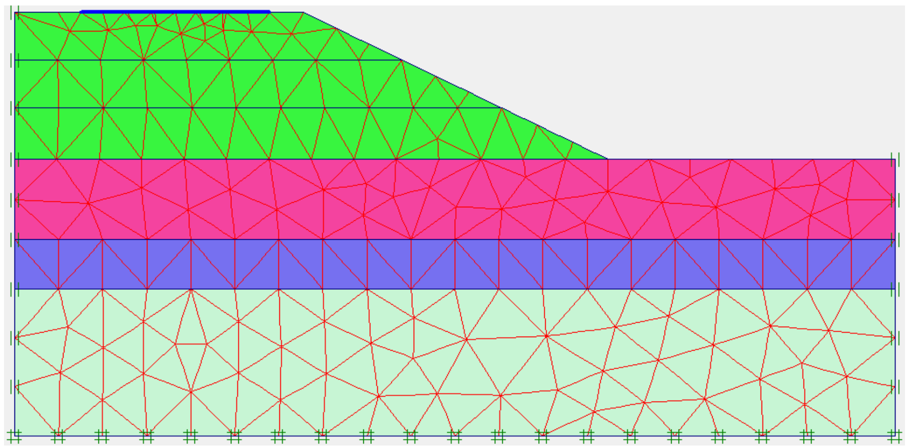


Figure 4.2 Typical Mesh Generation in PLAXIS

4.2.3 Initial and Boundary Conditions

In the initial conditions, water unit weight is set to 10kN/m^3. The water pressure is fully hydrostatic and is based on a general phreatic level. In addition to phreatic level, boundary condition for consolidation analysis can be additional input. The lines of consolidation need to be selected in vertical direction that means vertical boundaries must be closed to restrain the horizontal flow and no free outflow is allowed at that boundary.

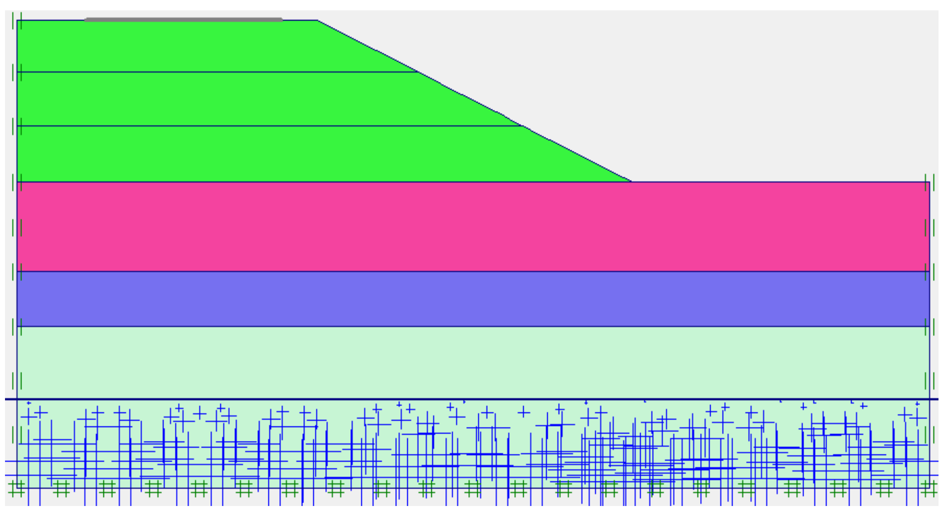


Figure 4.3 Active Water Level in PLAXIS

The water conditions can also be specified in the Geometry configuration mode using phreatic level by generating pore pressure using phreatic level. In the analysis, constant ground water level has been considered.

4.2.4 Initial Stresses

In initial stresses which are effective stresses, Over-Consolidation Ratio (OCR) and Pre-Overburden Pressure (POP) are used in analysis when using cam-clay model. Initial stresses are developed by the POP procedure (PLAXIS 8, User Manual). It is also possible to specify the initial stress state using the Pre-Overburden Pressure (*POP*) as an alternative to the over consolidation ratio. The Pre-Overburden Pressure is defined by:

$$POP = \sigma_p - \sigma'^{0}_{yy} \tag{4.1}$$

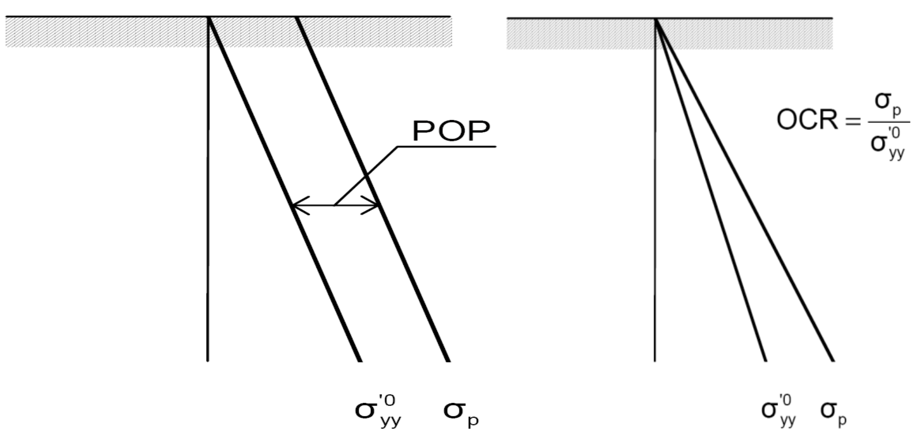


Figure 4.4 POP and OCR profile illustrations - PLAXIS 8, User Manual

Once the geometry of the model was developed, finite element model is complete. Initial situation and initial stress state should be stated before calculation. This was done in the initial conditions part of the input program. When using Mohr Coulomb model, the analysis require the generation of the initial stresses by means of K_o procedure. K_o procedure can be used to calculate initial stresses. The suggested K_o procedure is based on Jaky's formula (1-K_o* Sinϕ).

4.2.5 Calculation

After the generation of Phreatic level and initial stresses, the input is complete and calculations can be generated. These calculations are generally used to define the different phases of embankment construction. Figure 4.5 presents a snapshot of different phases of the construction process as implemented in FEM program.

The actual construction sequence was not reported in the reference and it was simulated by adding embankment fill in three layers with equal thickness (3m). Four loads coming in contact with road through tires were used to simulate the traffic loading. The loading consisted of 40kN having uniform maximum vertical contact stress over the contact area with ratio of 1:0.85 (width=19.27 cm, length=16.38 cm) placed on top of the pavement.

In the modeling analysis, the embankment construction consists of three phases, each taking 30 days. After the construction phase, consolidation period of 60 days was introduced to allow excess pore pressure to dissipate. The consolidation option in FEM software allows fully automatic time stepping procedure that takes the critical time step into account. Pavement construction and traffic loading were also taken into the consolidation analysis with different time intervals. The last phase in consolidation analysis was selecting minimum pore pressure where the default value of 1kN/m^2 was used for the pore pressure.

To calculate the global safety factor for the road embankment, the phi-c reduction option available in the PLAXIS was selected and used in the next phase.

Next Insert Delete...

Identification	Phase no.	Start from	Calculation	Loading input	Time	W
Initial phase	0	0	N/A	N/A	0.00 ...	
Gravity Loading	1	0	Consolidation	Staged Construction	15.0...	
1st stage Emb Loading	2	1	Consolidation	Staged Construction	30.0...	
Consol Period	3	2	Consolidation	Staged Construction	60.0...	
2nd Stage Emb Loading	4	3	Consolidation	Staged Construction	30.0...	
Consol Period	5	4	Consolidation	Staged Construction	60.0...	
3rd stage Emb Loading	6	5	Consolidation	Staged Construction	30.0...	
Consol Period	7	6	Consolidation	Staged Construction	60.0...	
Pavement Const	8	7	Consolidation	Staged Construction	30.0...	
Traffic Loading	9	8	Consolidation	Staged Construction	1000...	
Pore Pressure	10	9	Consolidation	Minimum pore pressure	17.0...	

Figure 4.5 Calculation Steps using FEM program

4.2.6 Results of Finite Element Analysis

On evaluating the total displacement, it can be seen that the failure mechanism is developing with excess pore pressure distribution. The settlement at the pavement surface and embankment were increasing considerably after the end of the construction of embankment and pavement. This is due to the dissipation of excess pore pressure in soft soil layer which causes consolidation in soils.

The vertical displacement (or settlement) contours for normal fill and ECS fill from the numerical analysis (using Mohr coulomb and Cam-Clay model for embankment) over 30 years of time are presented in Figure 4.6 and Figure 4.9 respectively. Figure 4.7, 4.8 and Figure 4.10, 4.11 shows maximum settlement that can occur after full dissipation of pore pressure at both embankment locations.

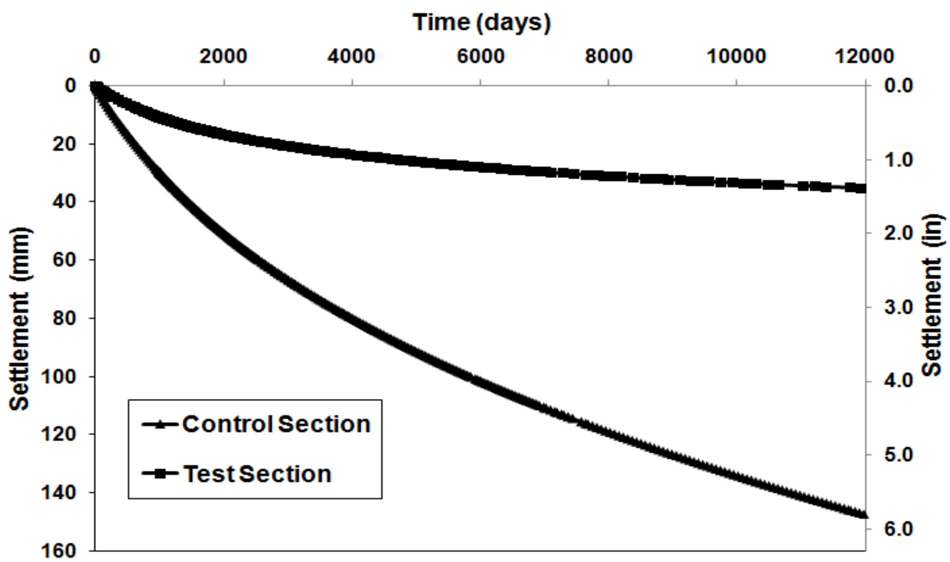


Figure 4.6 Settlement vs. Time plot in the pavement in FEM using MCM for Embankment

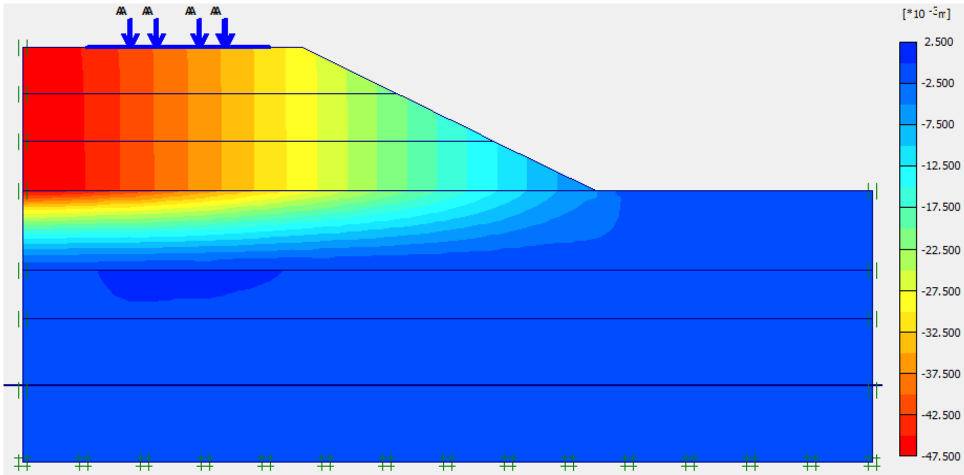


Figure 4.7 Settlement contours at the end of pore pressure dissipation in ECS using MCM for Embankment

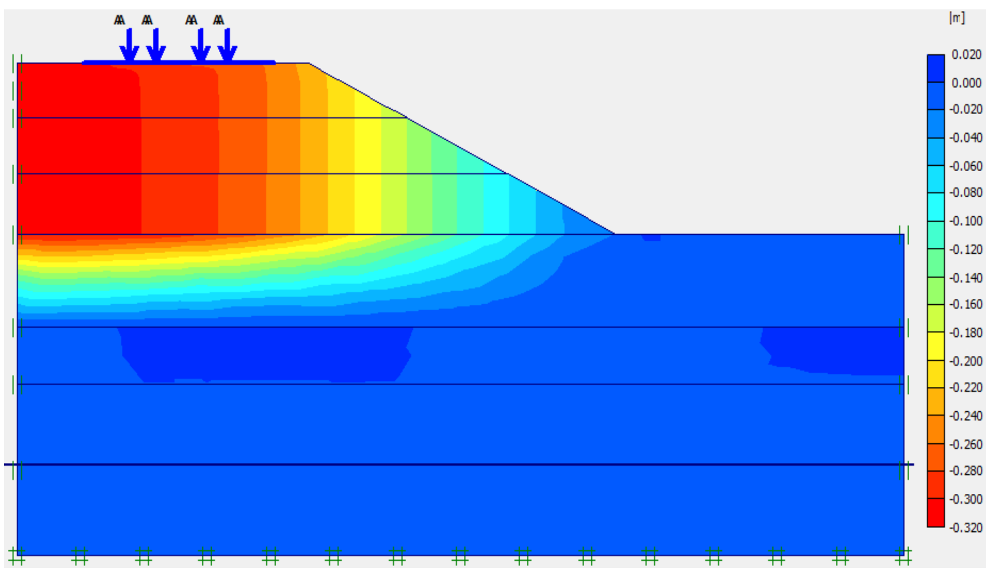


Figure 4.8 Settlement contours at the end of pore pressure dissipation in Normal Fill using MCM for Embankment

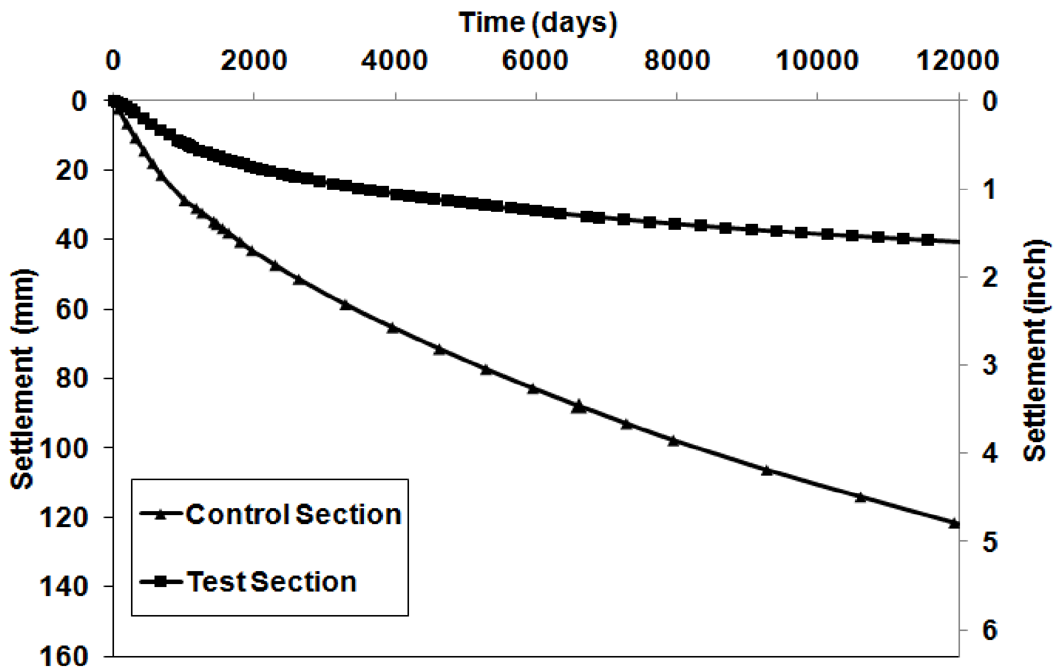


Figure 4.9 Settlement vs. Time plot in the pavement in FEM using SSM for Embankment

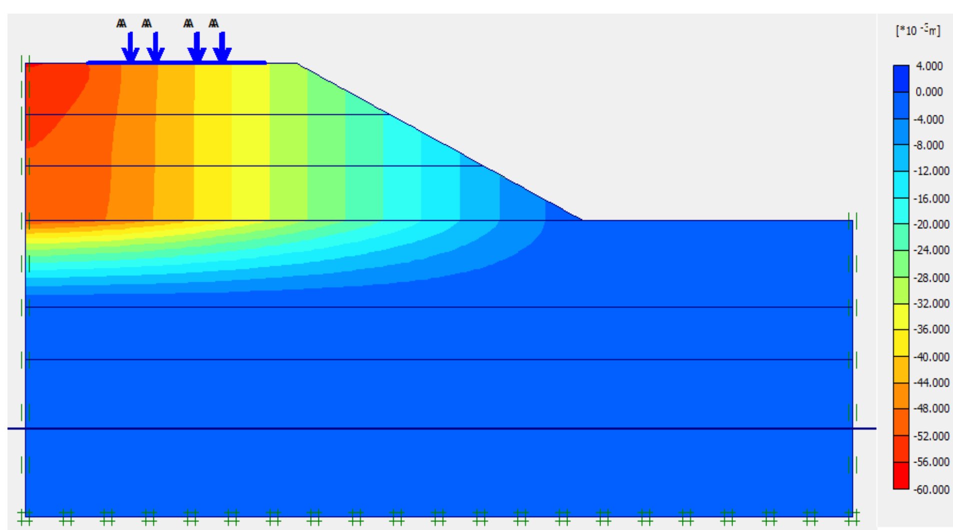


Figure 4.10 Settlement contours at the end of pore pressure dissipation in ECS using SSM for Embankment

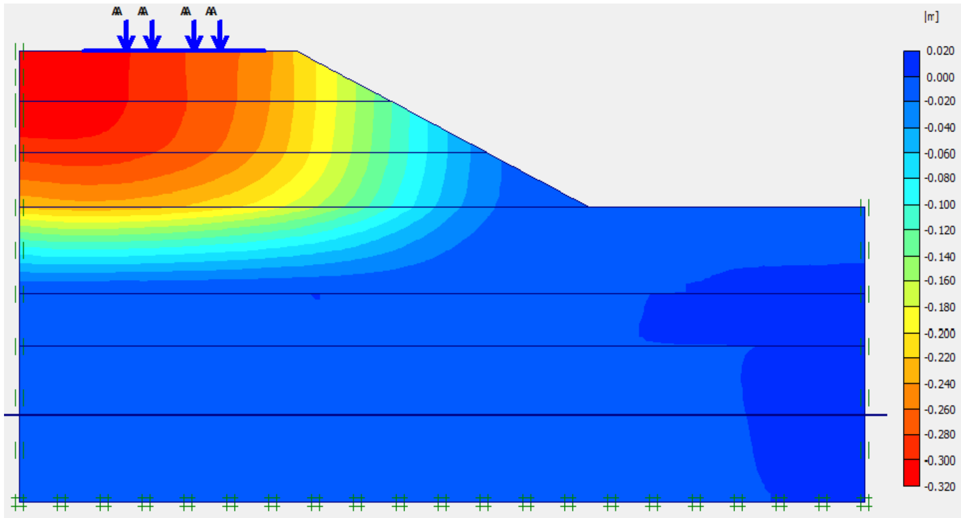


Figure 4.11 Settlement contours at the end of pore pressure dissipation in Normal Fill using SSM for Embankment

4.2.7 Vertical Stress in Finite Element Analysis

The vertical stress contours from the numerical analyses for the two cases are also presented in Figures 4.12, 4.13, 4.14 and 4.15. It is shown that more stress concentration occurs at bottom of the normal fill as the loads are transmitted from the top to the bottom. The degree of stress concentration is slightly reduced when ECS materials are used instead of normal fill. In addition, the ECS carry fewer loads than the normal fill due to the low weight and high strength properties.

From Figure 4.12 and 4.13, it has been observed that a maximum stress of 80kPa has been transmitted to the soft foundation soil in the case of the ECS embankment after the full dissipation of pore water pressure; whereas, a maximum vertical stress of 150kPa is recorded at the interface of the embankment base and soft foundation soil for the control embankment. This is expected since the ECS is a LWA material which has approximately half the unit weight of the normal fill materials and hence imparts lower thrust on the foundation soil.

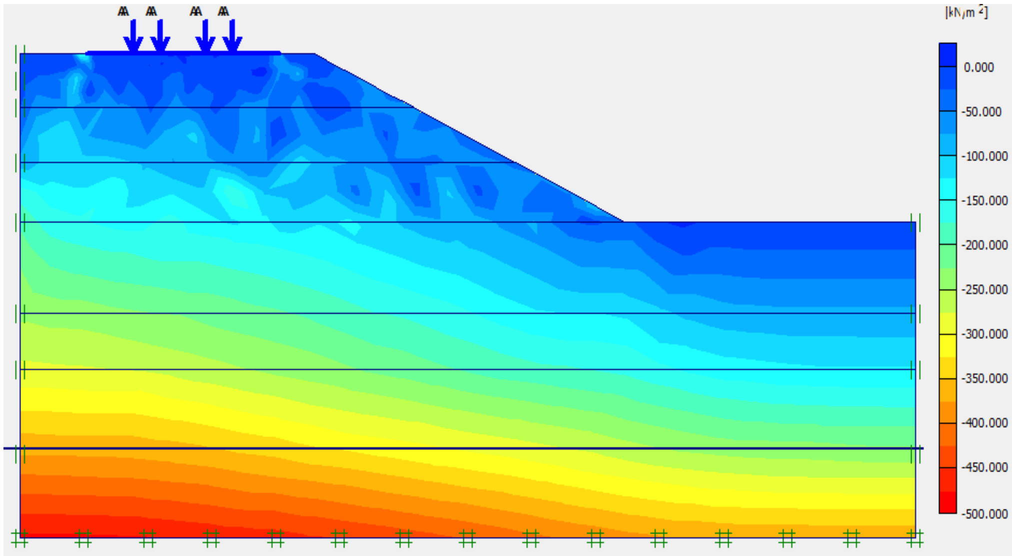


Figure 4.12 Stress distribution in Normal Fill after full dissipation of pore pressure using MCM for Embankment

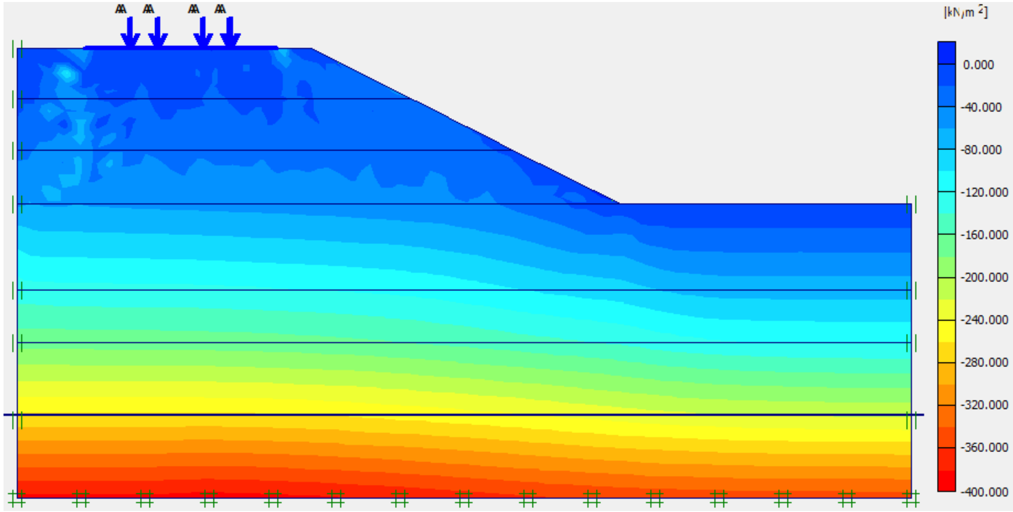


Figure 4.13 Stress distribution in ECS Fill after full dissipation of pore pressure using MCM for Embankment

From Figure 4.14 and 4.15, it has been observed that a maximum stress of 50kPa has been transmitted to the soft foundation soil in the case of the ECS embankment; whereas, a maximum vertical stress from 150kPa is recorded at the interface of the embankment base and soft foundation soil interface in the case of the control embankment.

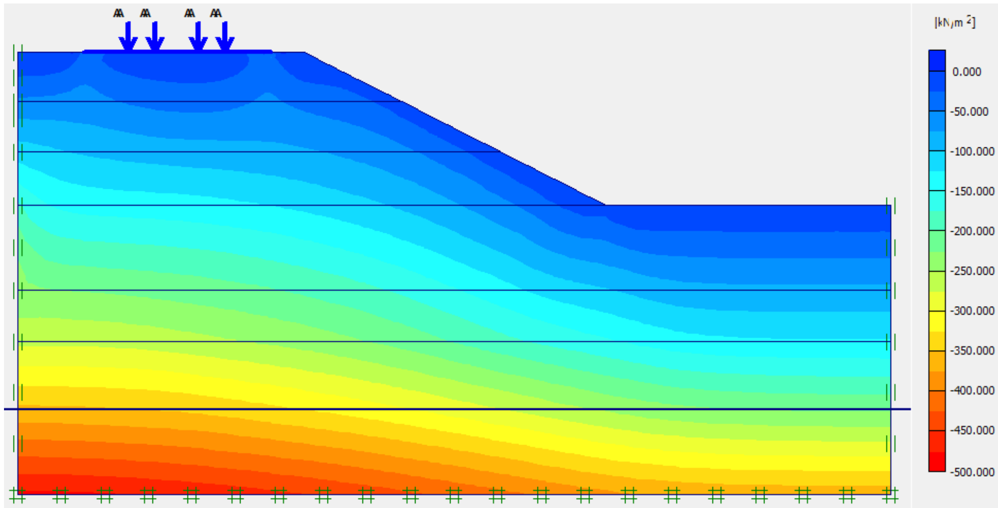


Figure 4.14 Stress distribution in Normal Fill using SSM for Embankment

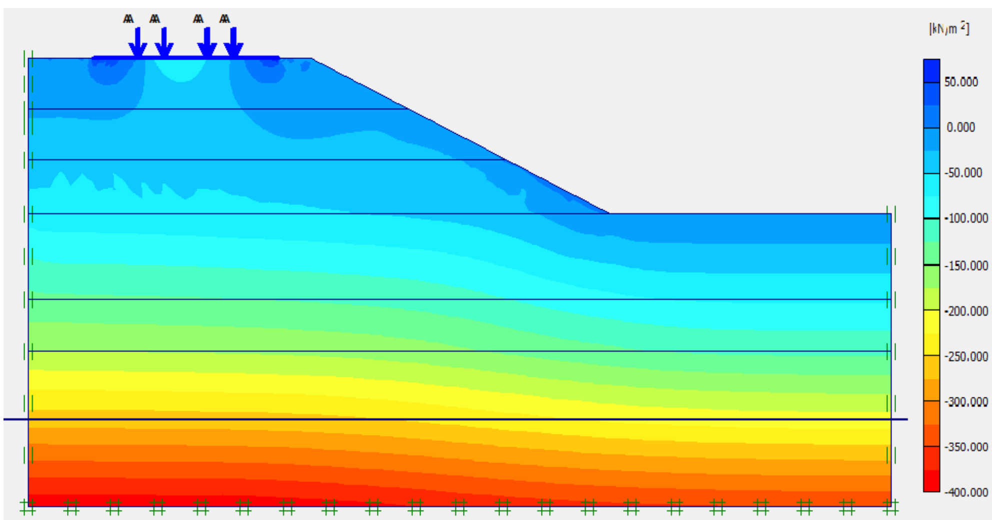


Figure 4.15 Stress distribution in ECS Fill using SSM for Embankment

4.3 Finite Difference Modeling Procedure

The finite difference method (FDM) is perhaps the oldest numerical technique used for the solution of sets of differential equations, given initial values and/or boundary values (Desai and Christian 1977). For FDM analysis, the computer code FLAC – Fast Lagrangian Analysis of Continua (FLAC, 2000) was selected because of its flexibility and wide acceptance. It easily incorporates key factors and phenomena which affects the behavior of ground conditions thus making it a better choice to perform the FDM. The FDM code is an explicit two-dimensional finite difference program that performs a Lagrangian analysis. Here, explicit means it uses a time stepping procedure to solve the problem without forming the stiffness matrix. The Lagrangian formulation enables the grid to move and deform with the material it represents since the incremental displacements are added to the coordinates (FLAC 4.0 Manual, 2000).

FLAC is a very good tool for assessing the effect of many different material properties. It is also a very robust tool because it can handle any constitutive model with no adjustment to the solution algorithm (FLAC 4.0 Manual, 2000).

The use of *FLAC* in problem solving for static mechanical analysis in geotechnical engineering by considering special aspects that is considered in model creating and solution.

- Grid generation
- Boundary and Initial Conditions
- Choice of constitutive model and material properties
- Interpretation of results

4.3.1 Grid Generation

In FLAC, grid generation is limited to simple and regular shaped regions. There is a Fish function which can be used to generate user defined grids with varying zones. The advantage of using FISH function is that it can easily adjust grid boundary and zone density using “SET” command.

The FLAC grid is configured by specifying the command “CONFIG axisymmetry” at the beginning of the data file for analysis. The structural element formulation only applies for plane-strain or plane-stress analysis. The first step in FLAC is to select system units which are in meters, kilograms and seconds and then the advanced constitutive modeling option is selected. In this software, the factor of safety interface option is available only for Mohr Coulomb Model in FLAC. Adjust total stress and ground water analysis can also be selected in FLAC simply by specifying the “CONFIG ats, gw” command at the beginning of the data file.

The model used in our case study takes advantage of half symmetry. The size of the model is 52 meters wide and 26 meters in height. Here the grid is composed of several zones consist of constant height but variable zone width. A reasonably fine grid should be selected to ensure that the settlement contours will be well-defined as it develops. It is best to use the finest grid possible when studying problems as shown in Figure 14.16.

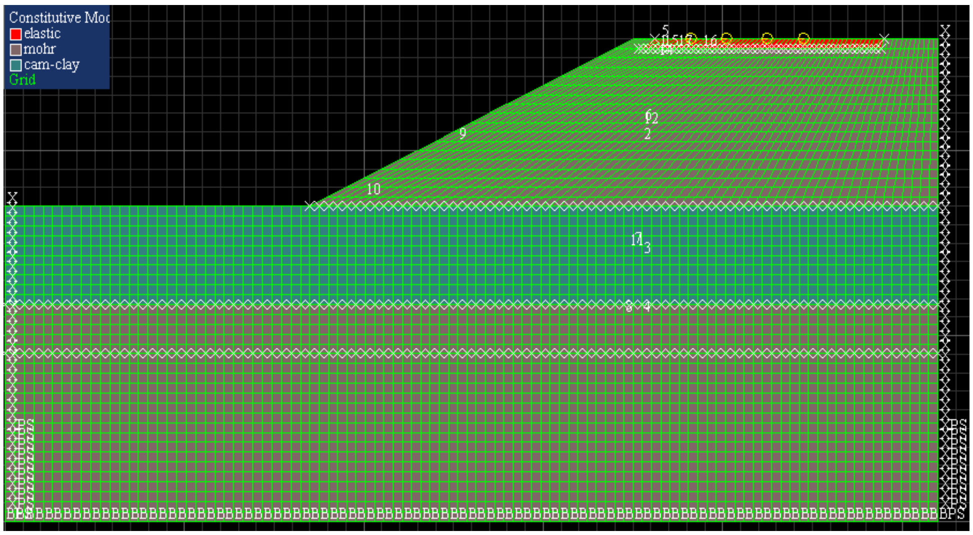


Figure 4.16 Fine grid generation in FDM

The basic grid used is named as Slope inside GENERATE command. The zones along the slope face are all quadrilateral-shaped. The grid is divided into different regions using mark command to define different soil types in different regions.

4.3.2 Boundary and Initial Conditions

If the geometry and loading in the embankment system is symmetrical, then we can make mechanical boundary conditions correspond to roller boundaries along the symmetry line and fixed displacements in the x and y direction, using the command "Fix x and y" at the model base. The acceleration of gravity is set to be 9.81 m/sec^2 (positive means to set it downwards).

There is an in-situ state of stress in the ground before starting any excavation or construction. An initial condition specified in the FLAC grid simulates that in-situ stress. This is an important step as it ultimately affects the subsequent behavior of the model. Ideally, information about the initial state comes from field measurements but, when these are not available, the model can be run for a range of possible conditions. A set of stresses is installed in the grid and then FLAC is run until an equilibrium state is obtained (FLAC 4.0 Manual, 2000). It is more difficult to give the initial stresses when materials of different densities are present. A layered system with a free surface, enclosed in a box with roller side boundaries and fixed base are used in analysis as shown in Figure 4.17. The ground water flow option in FLAC can be used to find phreatic surface by establishing initial pore pressure boundary condition before mechanical response. We apply pore pressure to raise the water level but in our case study, the constant water table is adopted in the middle of the sandstone layer

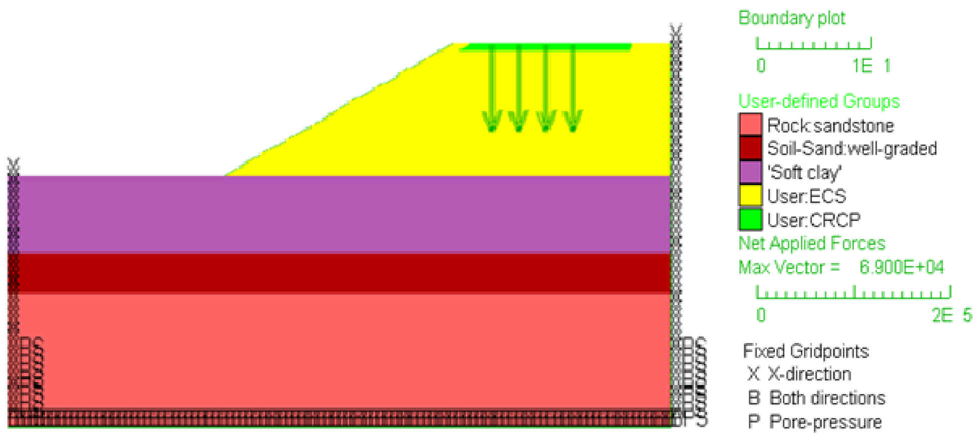


Figure 4.17 Geometry with selected Material layers and initial conditions

4.3.3 Choice of Constitutive Model and Material Properties

There are ten built-in material models in FLAC. Here we will discuss the selected one which has been considered for analysis.

(1) Elastic, isotropic.

(2) Mohr-Coulomb plasticity.

(3) Modified Cam-clay.

The elastic, isotropic model is valid for homogeneous, isotropic, continuous materials that exhibit linear stress-strain behavior. The elastic, transversely-isotropic model is appropriate for elastic materials that exhibit a well-defined elastic anisotropy (FLAC 4.0 Manual, 2000).

The Mohr-Coulomb plasticity model is used for materials that yield when subjected to shear loading, but the yield stress depends on the major and minor principal stresses only; the intermediate principal stress has no effect on yield. The Mohr-Coulomb model is applicable for most general engineering studies. Also, Mohr-Coulomb parameters for cohesion and friction angle are usually more-readily-available than other properties for geo-engineering materials. Mohr-Coulomb models are the most computationally-efficient plasticity models (FLAC 4.0 Manual, 2000).

The modified Cam-clay model accounts for the influence of volume change on deformability and on resistance to failure. In the Cam-clay model, models, tangential bulk and shear moduli are functions of plastic volumetric deformation. Few points on cam clay model are summarized below (FLAC 4.0 Manual, 2000):

1. The elastic deformation is nonlinear, with the elastic moduli depending on mean stress.
2. Shear failure is affected by the occurrence of plastic volumetric deformation; the material can harden or soften, depending on the degree of preconsolidation.
3. As shear loading increases, the material evolves toward a critical state at which unlimited shear strain occurs with no accompanying change in specific volume or stress.
4. There is no resistance to tensile mean stress.

These models assume an isotropic material behavior in the elastic range described by two elastic constants — bulk modulus (K) and shear modulus (G). The elastic constants, K and G, are used in FLAC rather than Young's modulus, E, and Poisson's ratio, v, because it is believed that bulk and shear moduli correspond to more fundamental aspects of material behavior than do Young's modulus and Poisson's ratio (FLAC 4.0 Manual, 2000).

The equations to convert from (E, ν) to (K, G) are

$$K = E/3(1 - 2\nu) \qquad (4.2)$$

$$G = E/2(1 + \nu) \qquad (4.3)$$

Models that include groundwater flow require the bulk modulus of the water, Kw. The physical value of Kw is 2GPa for purewater at room temperature, but the value selected should depend on the purpose of the analysis.

Vermeer and de Borst observe that values for the dilation angle are approximately between 0° and 20°, whether the material is soil, rock, or concrete and this property is typically determined from direct shear tests. The "permeability," k, required by FLAC is the coefficient of the pore pressure term in Darcy's law.

The modified Cam-clay model is defined by initial elastic moduli plus parameters that prescribe the nonlinear elasticity and hardening/softening behavior. The properties are given below and these properties are determined from the laboratory testing.

κ = slope of the elastic swelling line

λ = slope of the normal consolidation line

M = material constant

mpc = preconsolidation pressure

mv_1 = specific volume

p1 = reference pressure

v_0 = initial specific volume

v_λ = specific volume at reference pressure on normal consolidation line.

The material properties of the subgrade soil materials adopted in FLAC are provided in Table 4.5 and these results are used in performing the analysis. The material properties of the embankment materials are provided in FEM section in "Table 4.3" as same properties are adopted to perform analysis in FLAC.

Table 4.5 Properties of subsoil at the location.

Sub-Soil Properties	Soft Clay	Sand (well graded)	Sandstone
Cc	0.34	-	-
Cs	0.043	-	-
e_0	1.3	-	-
M	1	-	-
G (MPa)	26E+6	13.3E+6	88.3E+8
K (MPa)	57E+6	8.0E+6	88.7E+8
μ	0.3	0.25	0.2
mpc (KPa)	97.6	-	-
mv_1	2.3	-	-
Unit weight (kN/m^3)	18.85	17.65	26.47
c (kPa)	-	-	27200
ϕ (deg.)	-	33	28
ψ (deg.)	-	-	-
Model	Cam-Clay Model	Mohr-Coulomb	Mohr-Coulomb

4.3.4 Results in Finite Difference Analysis

Since FLAC models a nonlinear system as it evolves in time, the interpretation of results may be more difficult than with a conventional finite-element program that produces “a solution” at the end of its calculation phase.

The calculation is continued using step command and update interval of plot must be used to estimate the effects of time. The SOLVE command is used to find the equilibrium state (FLAC 4.0 Manual, 2000).

FLAC models the flow of fluid through the soil which is permeable in nature. In consolidation (a type of fluid/solid interaction) the slow dissipation of pore pressure causes displacements to occur in the soil and thus manifest in two mechanical effects. First, changing pore pressure causes change in effective stress, which affects the soil response. Second, the flow of water reacts to mechanical volume change by a change in pore pressure (FLAC 4.0 Manual, 2000). In our analysis we found that the convergence to the coupled drained and undrained analysis is very slow. In our analysis, there are certain variables that are of particular interest such as displacements and stresses. The construction of an embankment is assumed to occur instantaneously, pavement and traffic moving on top surface is also considered at the same time. An undrained analysis is first conducted to evaluate the settlement in the short-term after building of the embankment. The long-term response is then monitored after allowing drainage. The first stage of the simulation corresponds to the short time response of the system in which no flow is assumed to take place. The command SET flow off is specified. Loading of 40 KN by the traffic coming in contact with road through four wheels of vehicle is simulated on a 0.25m section of the model's top boundary. Once the full load is applied, the model is cycled to equilibrium using SOLVE command. During this stage, a pore pressure in soft clay beneath embankment develops as a result of volumetric deformations, but do not dissipate.

In the second stage, fluid flow is allowed to develop by issuing the commands 'SET flow on'. Water then drains through the model where the pore pressure is fixed at zero, and additional settlement takes place under the embankment. The 'SOLVE' command is used to perform the simulation and it requires parameters like consolidation age and steady state flow.

History command is used to track the important variables during analysis. Plots of these histories provide an understanding of the system. When certain zones in the model reach yield point, those zones are indicated by Plastic indicators. The maximum unbalanced force when reaches to its equilibrium state, it represents a stable model i.e. the maximum settlement has occurred in the model. Graphical output is saved in one of these formats: JPG, PCX, DXF (AutoCAD) etc (FLAC 4.0 Manual, 2000).

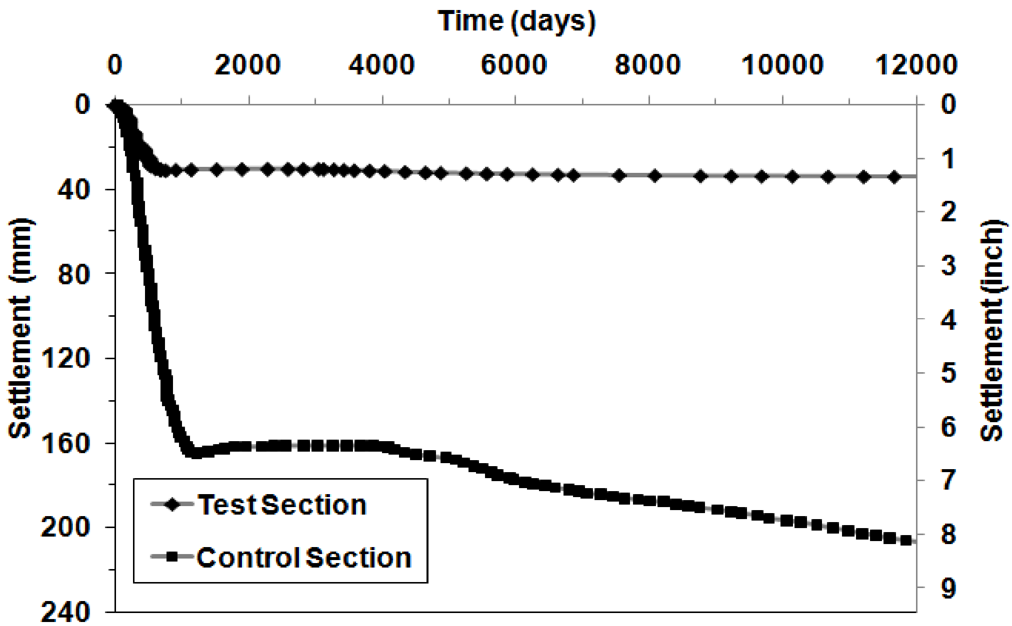


Figure 4.18 Settlement Results near pavement in FLAC using MCM for Embankment

The vertical displacement (or settlement) graph for normal fill and ECS fill from the numerical FLAC analysis using Mohr Coulomb Model in FLAC are presented in Figure 4.18, respectively.

Displacement graph and vertical displacement contours at the end of the undrained and drained numerical simulations are presented in Figures 4.18, 4.19 and 4.20. The vertical displacement histories recorded at different monitoring points indicate that the settlement at the surface of both types of an embankment near pavement increases from approximately 150mm to 200mm as a result of drainage. Note that the displacement graphs in Figure 4.18, and vertical displacement contours in Figures 4.19 and 4.20, correspond to the combined undrained and drained displacements. Figure 4.19 confirms that a steady-state flow has been reached by the end of the drained simulation. In Figure 4:20 confirms that a steady state flow conditions are still in progress.

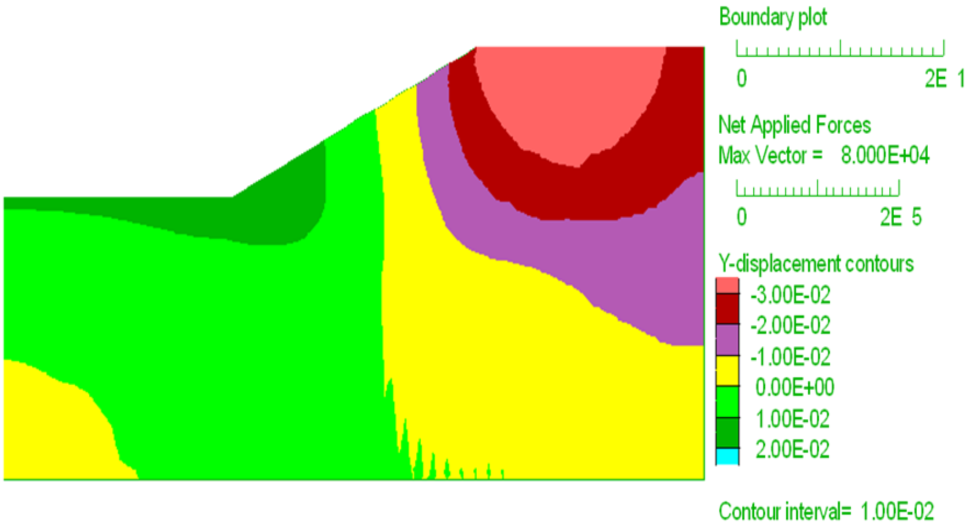


Figure 4.19 Maximum Settlement of ECS using MCM for Embankment

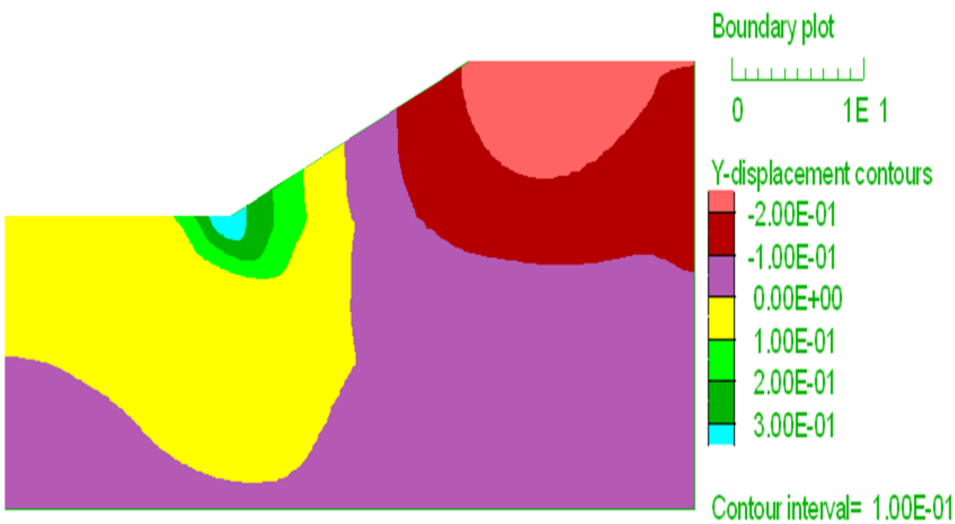


Figure 4.20 Maximum Settlement of Normal Fill using MCM for Embankment

4.3.5 Vertical Stresses in Finite Difference Analysis

From Figures 4.23 and 4.24, it has been observed that a stress distribution over whole section remains 250kPa which means stress are uniform over the ECS embankment and vertical stress between 200 to 300kPa is recorded at the interface of the embankment base and soft foundation soil in the control embankment at the end of drained conditions but the stress keeps on increasing over the depth shown in Figure 4.23. At undrained conditions shown in Figures 4.21 and 4.22, the stress of 150kPa has been transmitted to soft soil in case of ECS embankment whereas stress of 250kPa is recorded at the interface of embankment and soft soil in case of control embankment.

The higher stresses distributed to the soil below normal fill is due to the higher unit weight of the normal fill material. Due to traffic loading, ECS embankment has accommodated more stresses within the embankment section and transferred less stresses to the foundation soil. In drained condition shown in Figure 4.24, ECS section creates uniform distribution of stresses within itself and also to the foundation soil. In contrast, more stresses are being transmitted by the normal fill to the soft soil leading to large surface deformations.

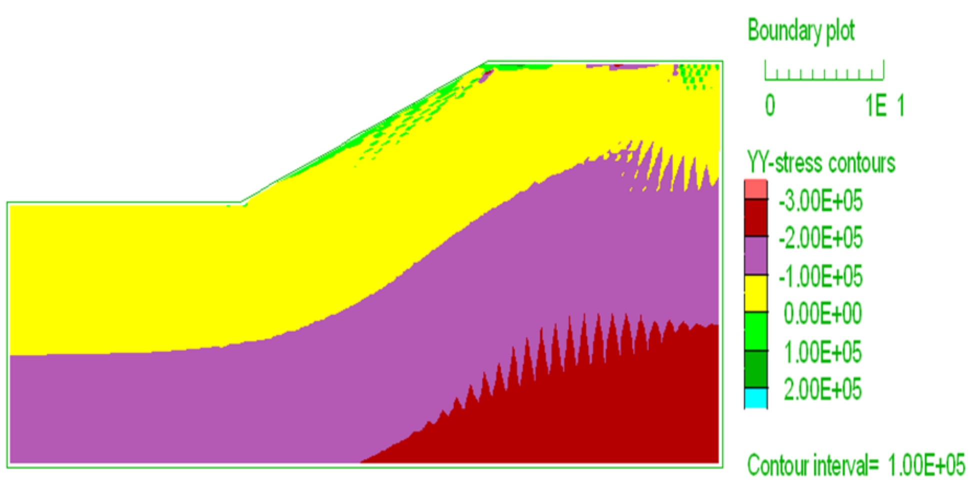


Figure 4.21 Stress distribution in ECS Fill at undrained condition using MCM for Embankment

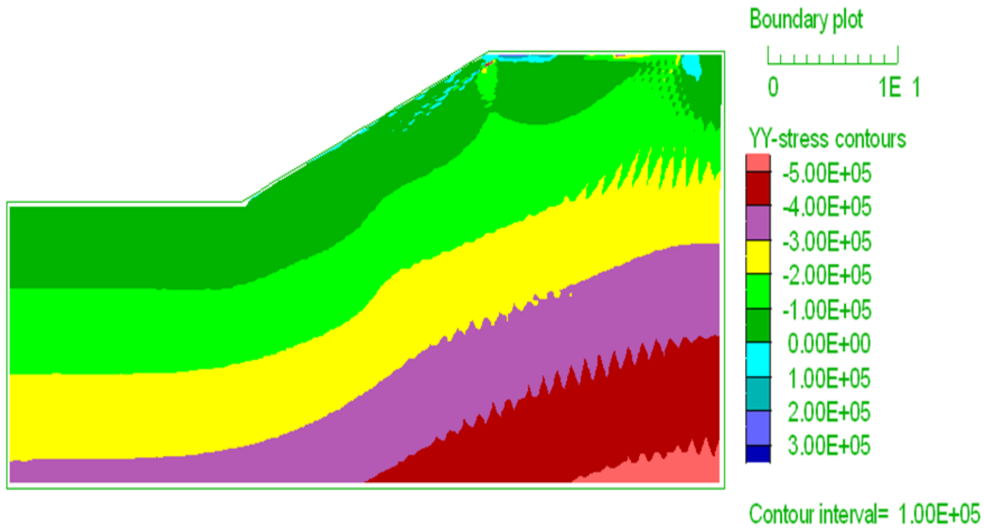


Figure 4.22 Stress distribution in Normal Fill in undrained condition using MCM for Embankment

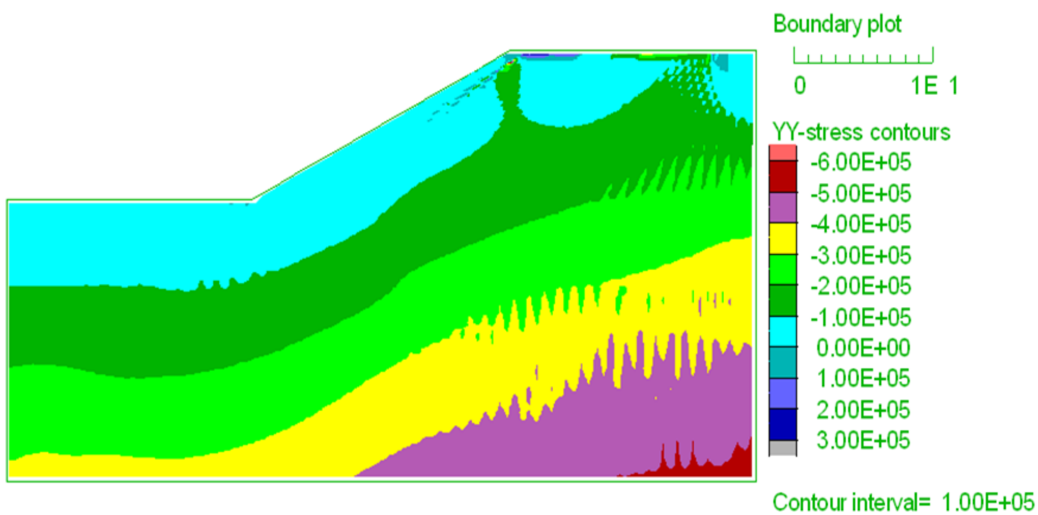


Figure 4.23 Stress distribution in Normal Fill in drained condition using MCM for Embankment

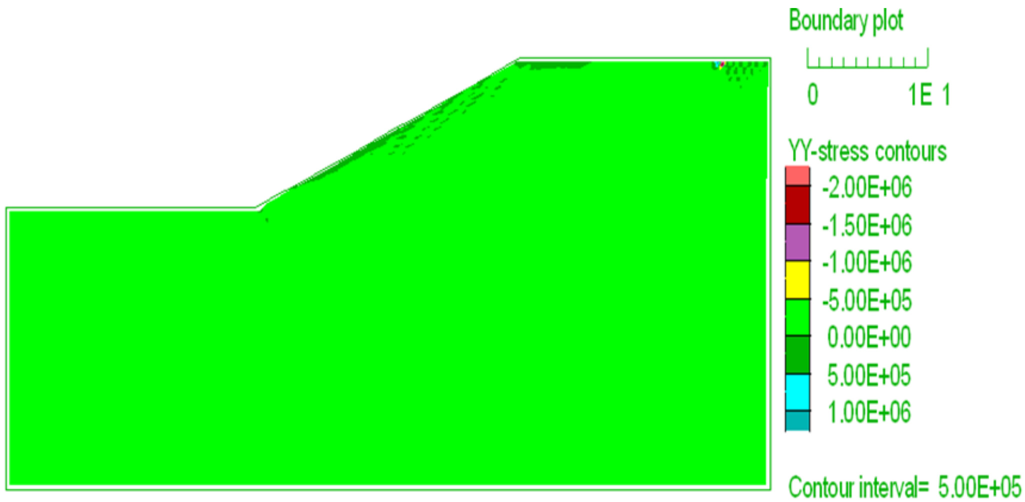


Figure 4.24 Stress distribution in ECS Fill in drained condition using MCM for Embankment

4.4 Summary

This chapter discusses the numerical analysis methodology using FEM (PLAXIS) and FDM (FLAC) based modeling of the present embankment section. Three different numerical analyses were carried out using the same subsoil properties. In FEM analysis, Mohr Coulomb and Cam-Clay model were used for embankment while in FDM analysis, only Mohr Coulomb model was used for embankment.

CHAPTER 5

ANALYSIS OF FIELD RESULTS AND COMPARISONS WITH NUMERICAL RESULTS

Here two full scale test embankments were constructed on SH-360 with different type of backfill materials. The area is underlain by 5 to 6m of soft compressible clay. Beneath the soft clay is well graded sand and it is underlain by sandstone which is around 9 to 10m thick. The cross sections of the bridge include two 12 ft travel lane in each direction, 8 ft shoulders on both sides of embankment. The bridge layout is presented in Figure below.

Figure 5.1 Satellite Imagery of SH-360

This case study contains fair information on soil properties, LWA materials, and five year field measurements of settlements done using elevation survey after construction and horizontal movement checked monthly using inclinometer. The finite difference software PLAXIS Version 8.0 and FLAC (Fast Lagrangian Analysis of Continua) Version 4.0, developed by Itasca Consulting Group, Inc., were adopted for this numerical analysis. The analysis is focused on the evaluation of the maximum settlement over different period of time. The analyzed section taken for the modeling purpose of SH 360 is given in Figure 5.2 presented in section 5.1.1.

5.1 Analysis of Field Results

Pavement in this area is typically constructed on embankments with a total height of 0.3 to 1.5m and exerts traffic load of 35 to 55kN on the ground surface. Without any ground improvement, a sudden drop in elevation has taken place at the joint of the approach embankment with bridge structure. Stability problem is always associated with settlement problems where there is soft ground beneath embankment as is the case in our study location.

After construction of embankment and pavement road, readings of vertical displacements using elevation surveys on the surface of road and horizontal movement using inclinometers were made at certain time intervals.

5.1.1 Instrumentation Data

Four vertical inclinometers were installed in the ECS embankment materials to monitor the horizontal deformations as shown in Figures 5.2. Lateral displacements in the fill materials are expected due to traffic vibration, strong bonding between pavement and fill material or erosion. The first inclinometer was installed at the center line of the two embankments near the center of the median. The second vertical inclinometer was installed on the inside slope of the southbound embankment. The third and forth vertical inclinometer was installed on the inside and outer slope of the northbound embankment.

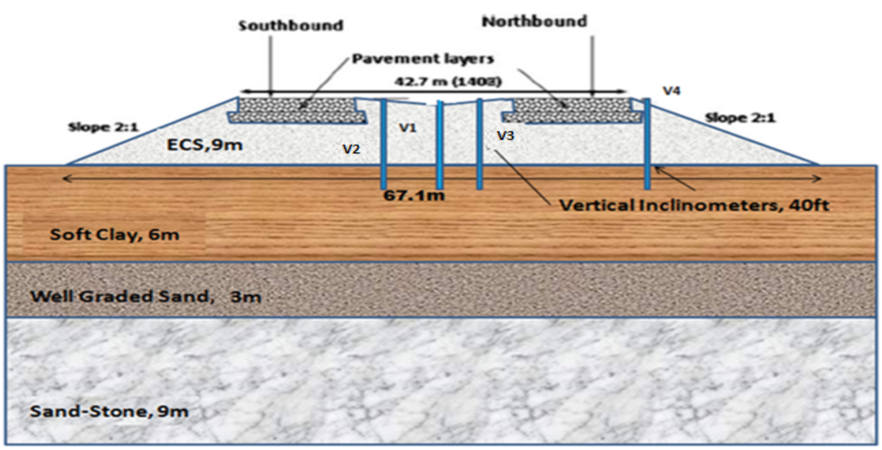


Figure 5.2 Profile View at ECS section

Displacement profiles are useful for determining the magnitude, depth, direction, and rate of ground movement thus helping in providing performance statistics of light weight embankment fill materials like ECS. . Hence, the ECS embankment section was instrumented with four vertical inclinometers, 12m in length and 70mm in diameter. Since ECS is a granular material, specialized field installation the inclinometers was followed. Here drilling was done using larger diameter auger with a central annular opening. Dry Bentonite chips were laid during removal of the auger. Water was poured into the Bentonite column and left for curing and then reaugered with a smaller diameter auger to install the inclinometer casing. This procedure ensured the proper installation of the inclinometers at the assigned site.

The Digitilt vertical inclinometer system consists of the inclinometer casing, a vertical inclinometer probe, a connecting cable and a readout unit. The probe for the vertical inclinometer consists of one accelerometer measuring tilt in the plane of the inclinometer wheels, which track the longitudinal grooves of the casing. The other accelerometer measures tilt in the plane perpendicular to the wheels. Inclination measurements are converted to lateral deviations after applying the necessary corrections as required (www.slopeindicator.com).

Changes in lateral deviation, determined by comparing data from current and initial surveys, indicate embankment movements in both directions. Plotting the cumulative changes at each measurement interval (0.6 m in this case) yields a high resolution displacement profile. The data collected for a period of five years after the embankment construction from four vertical inclinometers V1, V2, V3 and V4 are presented in Figures 5.4a, b, c and d respectively. This data was collected after the embankment construction and before the application of traffic load.. Hence, these small deformations are attributed to the construction induced settlement..

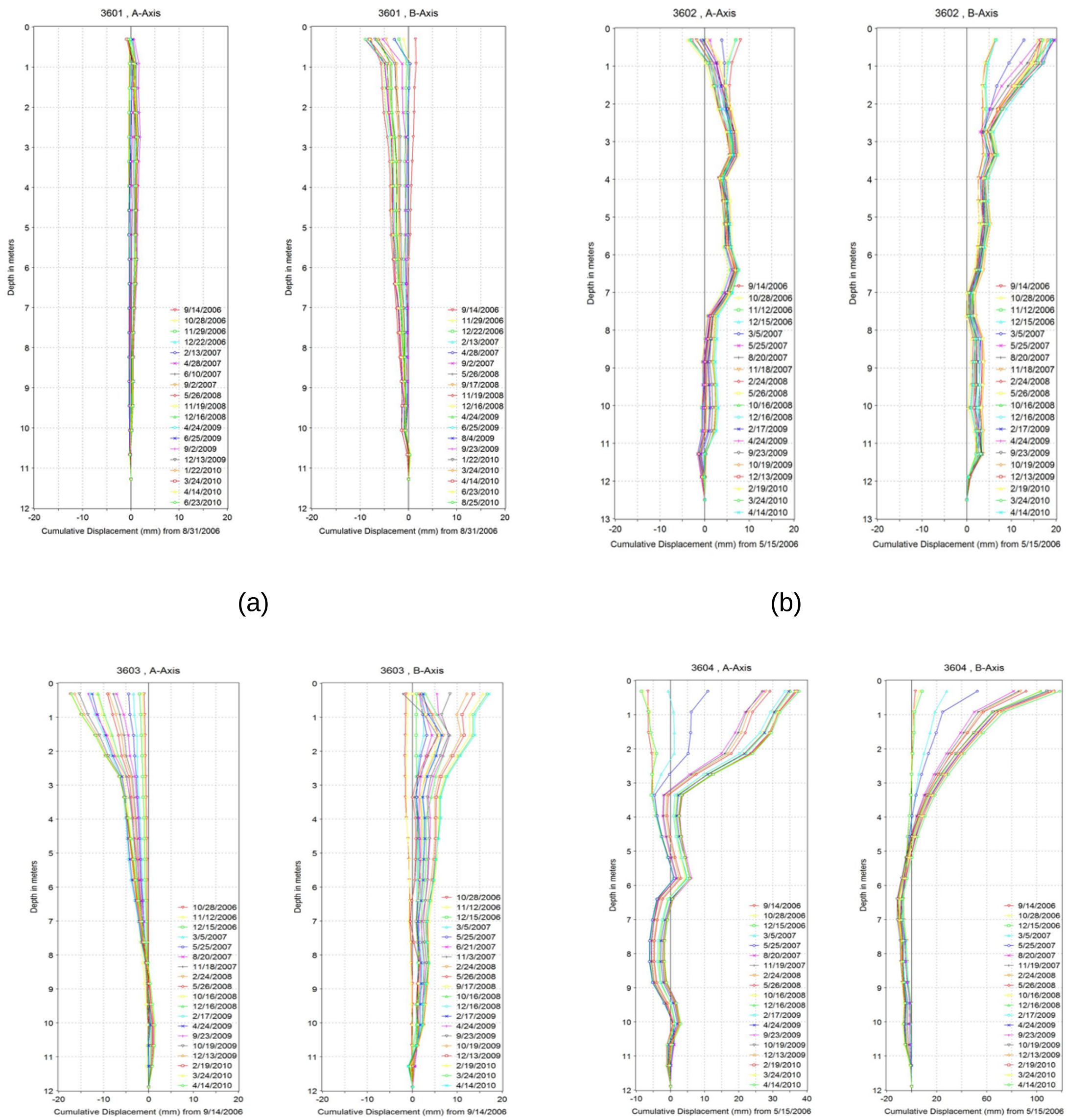
In the ensuing months after the construction of embankment further lateral movements were recorded due to the traffic loading and time dependent consolidation settlements of the foundation material. Due to the granular nature of the ECS fill consolidation settlements was minimal in the test section. While in the control section elevation survey confirmed that there was much higher settlement.

From Figure 5.4a, 5.4b, 5.4c and 5.4d, it can be seen that the lateral movements (deviations from the center line) in first, second and third of the inclinometers are within the permissible limits except the fourth one. The first inclinometer (V1) which is located at the center line of the median showed very less variation in its reading throughout the depth due to symmetrical loading from either side. The fourth inclinometer (V4), located at the outside slope of the northbound lanes, shows a slight rotation of the slope. The second and third inclinometer (V3 and V2), located at the inside slope of the northbound and southbound lanes, shows a slight rotation of the slope.

(a) (b)

Figure 5.3 (a) Horizontal movement seen near V2 location (b) Testing at V2 location

The readings from the four vertical inclinometers (V1, V2, V3 and V4 for ECS material) are presented in Figure 5.4 (a), (b), (c) and (d). Readings were taken once in month from Aug 2006-June 2007.After that the readings were taken with a gap of 2 to 5 months in-between up to June 2009 and then again the readings were taken once in a month from July 2009 to April 2010.Again from April 2010, the readings were taken with one month of gap. The results at V1 location indicated that in north-south direction, there were very small movements but have around 6mm from 0 to 1m depth in east-west direction. It’s seen that the horizontal displacement is not much concern at V1 location. Physical field observation has shown abnormal horizontal movement near V2 location shown in Figure 5.3 which may be a future concern but the inclinometer results at V2 is showing the movement within permissible limit. Continuous monitoring is recommended.



(a) (b)

(c) (d)

Figure 5.4 (a) V1, (b) V2, (c) V3, (d) V4, Vertical Inclinometer Readings: Cumulative displacement vs. Depth

At V4 vertical inclinometer, the movement is increasing from 90 to 120mm from 0 to 1m depth. The movement is more on the top and very less below 3m, typically lateral movements beyond 25 mm are considered problematic. Very high horizontal movement readings have been recorded during the time duration of May 2007 to June 2010. Possible reason of movement can be due to soil erosion, inadequate subsurface drainage systems, vibration of moving traffic and poor construction practices. Erosion can lead to severe problems like void development, failure of slope protection cover and loss of backfill material under the pavement and bridge. Field observations show that the worst erosion conditions are associated with directing surface water directly in to gaps between pavement and shoulders and directly on to and under the slope protection. Bad construction practices like poor compaction of the fill material can lead to severe settlement problems. Selection of the correct compaction moisture content is important in compacting poorly graded granular backfill. This parameter can be determined from optimum moisture content test by plotting the relationship between dry density and moisture content and selecting a target acceptable density.

Figure 5.5 Horizontal movements on ECS section

The Figure 5.5 above illustrates horizontal displacement at test section which has been filled with tar. It's seen that the horizontal displacement is increasing with time. At V2 and V3, the movements are under permissible limits and are not considered problematic. This section is under continuous monitoring to observe if any abnormal movements occur.

5.2 Pavement Surface Profiling

The objective of pavement surface profiling is to find the undulation on the pavement surface. Profiling as well as visual/digital photographic inspections have been carried out on both the ECS and normal fill embankments to understand the fill settlements and any heave related movements.

Elevation surveys have been taken once every two weeks for the first few months after the construction and thereafter monthly for the duration of the data collection. Surveys were conducted using a Total Station device. After establishing control points, surface profiling is carried out. This provides designers with information on movement of the pavement surface from its initial recorded position. These movements can be either positive which indicates heaving or it can be negative which indicates settlement.

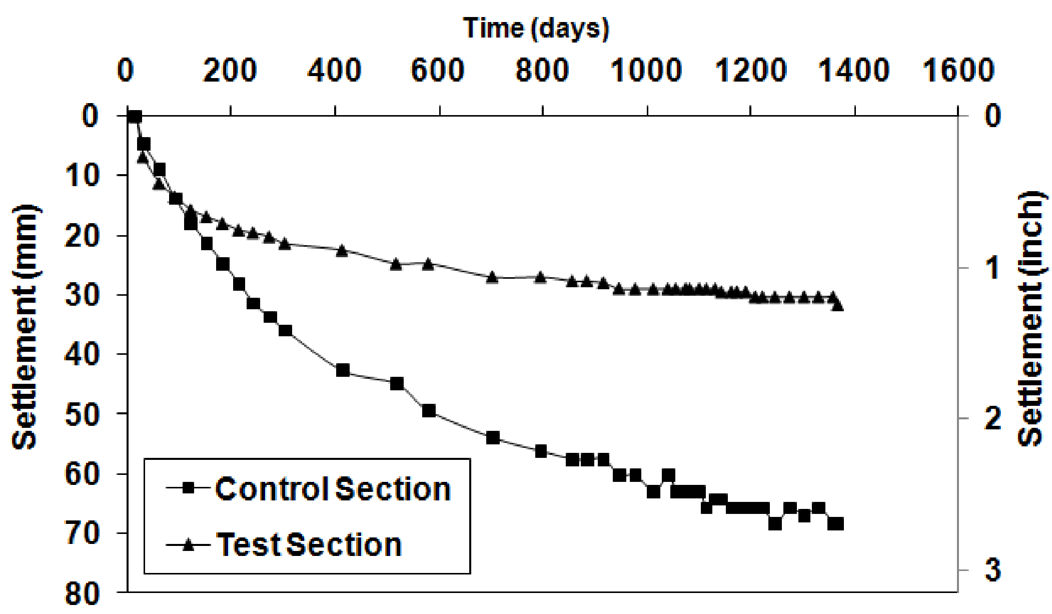


Figure 5.6 Settlement readings from selected survey points

5.3 Hyperbolic Formulation

The hyperbolic formulation method has proved applicable for long term settlement prediction in complex soil formations (Tan 1971, Kodamdaramaswamy and N. Rao 1980, N. Rao and Somayajulu 1981). The two constant mathematical relation mentioned below provides a practical solution for geotechnical problems like estimation of long term settlement. In this case the relationship between settlement δ and time t is assumed to follow a hyperbolic curve :-

$$\frac{t}{\delta} = B + At \tag{5.1}$$

Where A and B are the slope and the intercept of the straight line, respectively. If settlement δ versus time t obeys the hyperbolic relationship, the transformed hyperbolic plot of t/δ against t should be a straight line and the ultimate total settlement is obtained from the asymptotic line to the hyperbolic formulation. Taking the limits of the above Equation as t approaches infinity, the total settlement is given by 1/A, which is the reciprocal of the slope of the straight line (Kodamdaramaswamy and N. Rao 1980).

5.3.1 Long Term Settlements Using Field Elevation Data with Hyperbolic Formulation

The predicted settlement using hyperbolic formulation matches the measured settlement values. The hyperbolic relationship was an approximate fit of the survey elevation data over the entire range of settlement-time response. The straight line was prominent and obvious irrespective of various shapes of settlement versus time curves.

The hyperbolic formulation analysis was performed at both control section and test section where the following equations are determined using linear relationships:

Settlement at the control section

S = t / (0.0107 x t + 5.5982) mm

Settlement at the control section

S = t / (0.0295 x t + 4.8726) mm

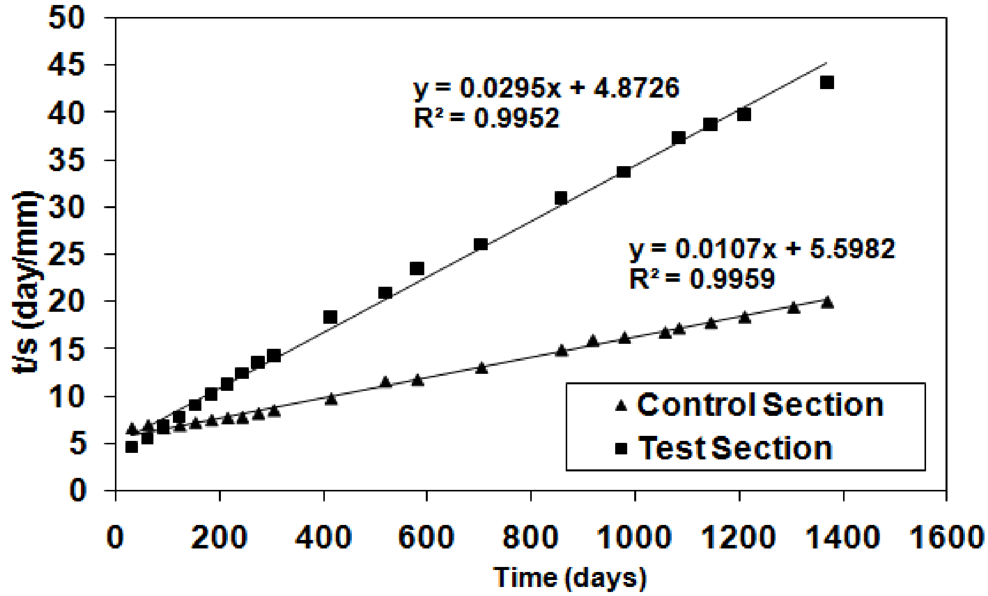


Figure 5.7 Hyperbolic plot of t (days) versus t/s (day/mm) corresponding to the settlement records.

5.4 Comparison of Field Results with Hyperbolic Formulation

Vertical displacement (or settlement) data from the elevation survey over the pavement surface were collected from the last four years of both normal fill and ECS fill. This data was then compared with hyperbolic formulation and the results matched closely with the measured settlements. Figure 5.8 and 5.9 shows a good match of hyperbolic formulation values with the measured elevation survey data.

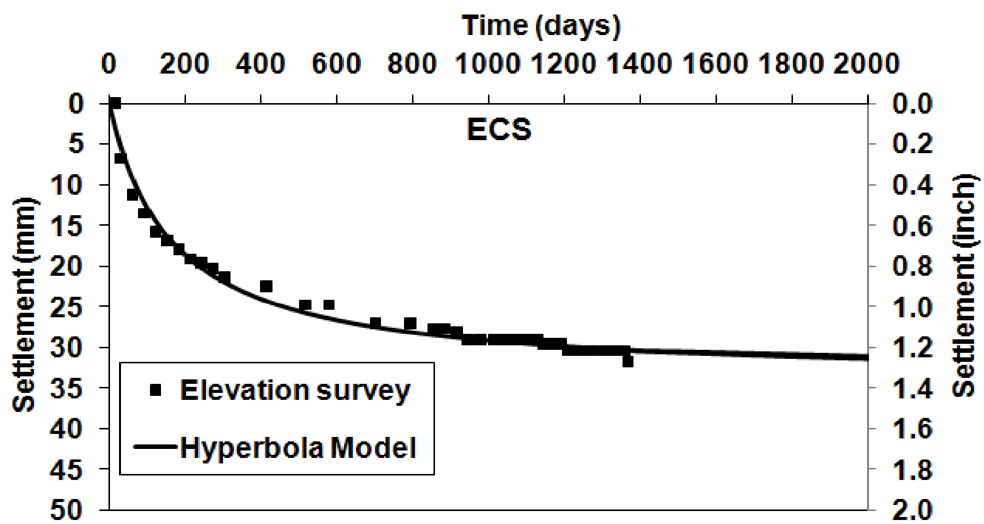


Figure 5.8 Comparison of Elevation Survey data with Hyperbolic Formulation for ECS

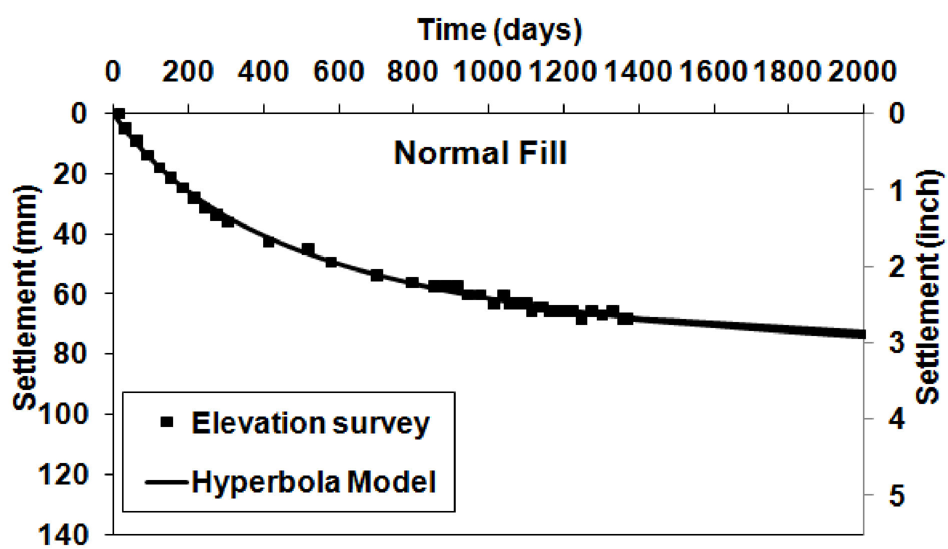


Figure 5.9 Comparison of Elevation Survey data with Hyperbolic Formulation for Normal Fill

5.5 Comparison of Hyperbolic Formulation Results with Numerical Modeling Results

Terashi (2003) indicated that nearly 60 percent of on-land application in Japan and perhaps roughly 85 percent of Nordic applications are for the settlement reduction and improvement of stability of embankment. The settlement reduction creates a more economical solution for embankment systems.

The behavior of treated sections has been comprehensively studied through field observation of full scale physical model, laboratory testing and numerical simulation. However, the cost of constructing and monitoring treated and untreated test embankments are quite high. An alternative method such as numerical experiment or simulation by means of appropriate methods such as finite element and finite difference techniques is essentially required. The numerical simulation of these two embankments systems were realized by means of Finite Difference and Finite Element methods using 2D analysis program. The aim of this study is to investigate the influence of geometric configurations using 2D numerical simulations of the two test embankments. Particular attention is given to the vertical displacements or settlement. Subsequent comparisons are made to study the long term settlement behaviors between the findings of 2D numerical simulations and those from the actual measured field data used by the hyperbolic formulation method of the two full scale embankments (test section and control section).

As a ground improvement technique, the ECS has been used as an alternative to increase bearing capacity, reduce settlements, and increase deep-seated slope stability of embankments over soft soil.

In addition to the treated embankment, other embankment section was also constructed without treatment and this section was considered as control section. Therefore, this site has one treated and one control sections for performance evaluation of ECS and normal fill materials.

After four years, the measured maximum settlements were approximately between 50 to 200mm at control section as shown in Figure 5.10 and Figure 5.11. It looks like the settlements had become relatively stable after approximately few years since construction.

Figure 5.10 Vertical displacement at SH 360 -Control Section

Figure 5.11 Close view of vertical displacement at SH 360 - Control Section

Four cases were analyzed to evaluate the long term influence of the (1) Vertical settlement on control section and test section using Mohr Coulomb Model for embankment fill (PLAXIS, FEM) (2) Vertical settlement on control section and test section using Cam-Clay Model for embankment fill (PLAXIS, FEM). (3) Vertical settlement on control section and test section using Mohr Coulomb Model for embankment fill (FLAC, FDM). (4) Extension of vertical settlement on control section and test section using hyperbolic formulation to predict long term settlements. These results are presented in the next section.

5.5.1 Vertical Displacement at Pavement Surface

The vertical displacements (or settlement) for normal fill and ECS fill from the numerical analysis were compared near the pavement surface because in the field, maximum settlement at normal fill was recorded near the pavement surface and very less settlement was seen at the pavement surface of ECS fill. The long term maximum settlement at pavement surface was then determined using hyperbolic formulation and these settlements are determined at 10, 20 and 30 years using the hyperbolic formulation results are used as a benchmark to compare the numerical analysis modeling results.

The numerical analysis, FEM and FDM results from Chapter 4 are compared with hyperbolic formulation results at 10, 20 and 30 years to check the long term settlements over the pavement and these results are summarized in Figure 5.12 and Figure 5.13. These results are also presented in Table 5.1, Table 5.2 and Table 5.3.

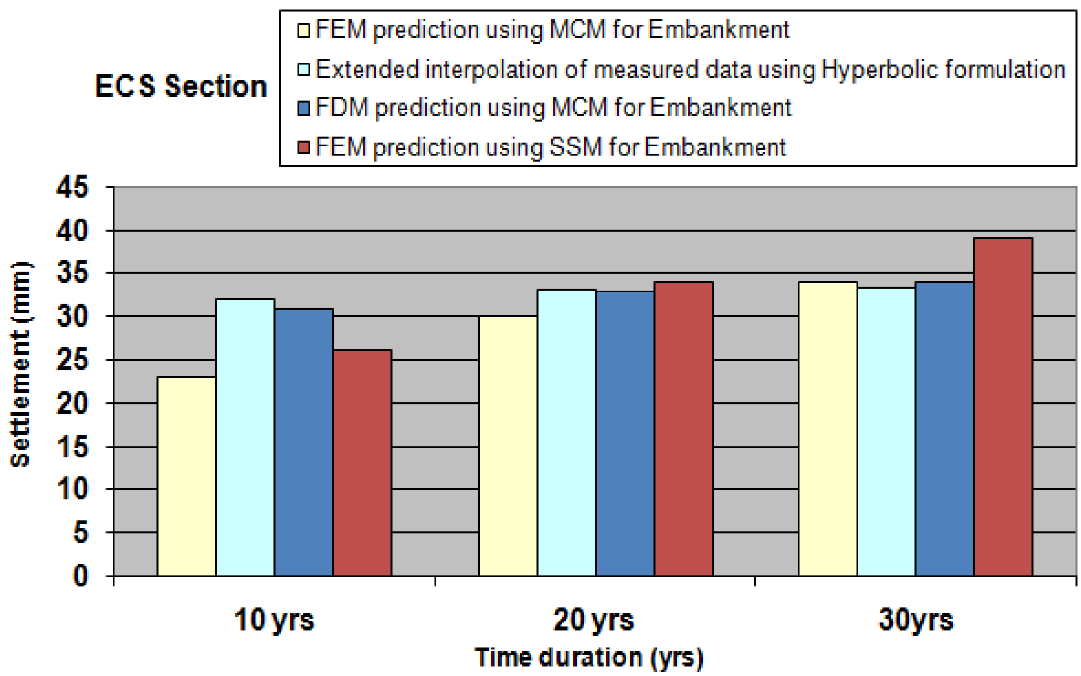


Figure 5.12 Comparison of predicted and measured long term settlement results using different methods at ECS Section

As expected, the first case with ECS fill modeled in PLAXIS (FEM) shows that the results are reasonably close to the long term interpretations based on hyperbolic formulation of the initial measured data. The second case with ECS using Cam-Clay Model yielded a good match and the same observation is noted for the third case with FDM modeling.

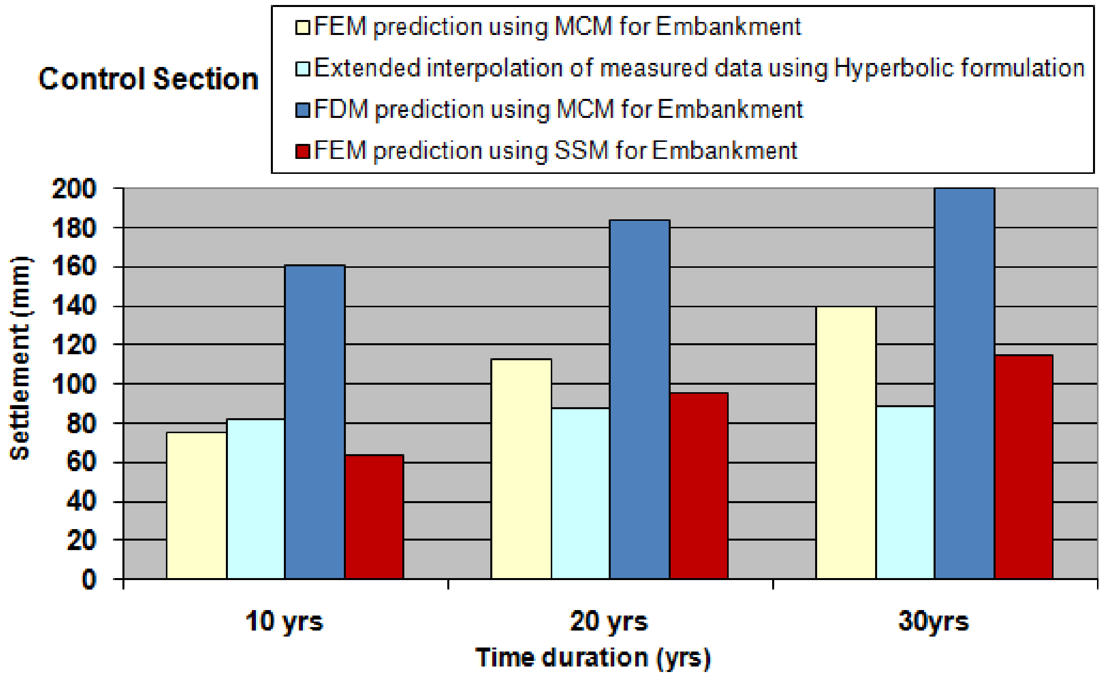


Figure 5.13 Comparison of predicted and measured long term settlement results using different methods at Control Section

The overall settlement observed in the control section is much higher than the ECS which is an expected outcome. Figure 5.13 compares different modeling test results on the control section with the hyperbolic formulation results. The first case with normal fill predicts moderately close results of the maximum settlement over the hyperbolic formulation method using Mohr-Coulomb Model. The second case with normal fill predicts reasonably close maximum settlement to the hyperbolic formulation using Cam-Clay Model. The third case was developed in FDM where normal fill over-predicts the maximum settlement at surface than as per case forth, hyperbolic formulation.

Table 5.1 Hyperbolic Formulation Results

Time (years) / Settlement (mm)	10	20	30
Control section	81.9	87.3	89.2
Test section	32.43	33.15	33.40

Table 5.2 PLAXIS, FEM Results using MCM / SSM for Embankment

Time (years) / Settlement (mm)	10	20	30
Control Section	75 / 64	113 / 95	140 / 115
Test Section	23 / 26	30 / 34	34 / 39

Table 5.3 FLAC, FDM Results using MCM for Embankment

Time (years) / Settlement (mm)	10	20	30
Control Section	161	184	200
Test Section	31	33	34

5.6 Summary

This chapter discusses instrumentation data and comparison of numerical model results with hyperbolic formulation.. Instrumentation data from vertical inclinometer installed at ECS location shows one of the locations has horizontal movement out of permissible limit. Possible reason of movement can be soil erosion, inadequate subsurface drainage systems, vibration of moving traffic and poor construction practices. All FEM and FDM methods provided closer predictions with the extended interpretations of measured settlements. The only exception is the FDM model's predictions for Control section and these variations are attributed to interface behavioral variations in the FDM. Overall, FEM modeling provided very good match of long term settlements of both Control and test sections.

CHAPTER 6

SUMMARY, CONCLUSIONS AND FUTURE RESEARCH RECOMMENDATIONS

6.1 Introduction

The main objective of the present research was to study the use of different constitutive models used in numerical analysis to model light weight embankment and local embankment fills over a soft subsoil and then recommend the use of the models for future designs of light weight embankment sections. The objectives of this research were accomplished and the results of this research are presented in Chapter 4 and Chapter 5. Some of the salient research findings of this research are summarized in the following section.

6.2 Summary and Conclusions

In this research project mainly two types of materials were studied for embankment to better check the settlement near pavement surface. Various test were performed on both test and control section of an embankment material to know the engineering properties which can be used in numerical analysis. A series of laboratory tests were performed which include tests like Atterberg limits, compaction, sieve analysis, hydrometer, direct shear test and free vertical swell. Numerical modeling using FDM and FEM models are used to determine the long term settlement behaviors of both control section and test sections. Mohr coulomb model were used for embankment material modeling in the FDM analysis while both Mohr-coulomb and Cam-Clay Models were adopted for FEM analysis. The hyperbolic formulation was studied and used on elevation survey data and then the method was used to predict the long term settlements. Based on the present experimental program, numerical analyses and hyperbolic formulation prediction results, the following conclusions are drawn:

1. Finite difference analysis need appropriate constitutive model for a particular soil type and takes considerable time for complete settlement analysis.

2. Finite element analysis is more powerful and gives better results that match with interpretations of long term settlements using the measured data.
3. Light weight material fill shows less settlement when compared to normal material fill settlements because the unit weight of LWA being one third to that of normal fill.
4. Vertical stresses are less in LWA embankment when compared to normal fill embankment and this implies stress induced settlements will be lesser in LWA embankment.
5. Swelling characteristics test data showed that untreated section swell was about 2.5% where as for the treated sample it was almost negligible.

Based on predicting settlements of embankment, there are a few uncertainties in the modeling which are mentioned below:

1. Uncertain time history of loading.
2. Economic problems that restrict testing and analysis.
3. Uncertain stratigraphy and water level fluctuations.
4. Movement due to shearing stresses.

6.3 Future Research Recommendations

Below are few recommendations for future research in the use of low weight materials as an embankment fill:

1. Developing a method to obtain reliable soil profiles of the study location.
2. Other low weight materials with different properties can be analyzed before using it as a fill material in field.
3. Studying other constitutive models on these materials
4. Tension in the soil need to be studies in FDM analysis to get more accurate prediction.
5. Documentation of any study location with all data accessible.
6. Doing the same work in 3D modeling and comparing the results with 2D.

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