USE OF LIGHTWEIGHT FILL MATERIALS IN CONSTRUCTION OF ROAD EMBANKMENTS ON SOFT PEATY CLAY

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DECLARATION

The work included in this thesis in part or whole has not been submitted for any other academic qualification at any institution.

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UOM Verified Signature

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Abstract

Number of proposed highways in Sri Lanka are to be constructed over sites underlain by soft peaty clay, due to scarcity of land. In order to ensure that the in-service settlements of these roads are small enough and the road could function satisfactorily, number of special ground improvements techniques are to be adopted. An alternate approach that could be considered is the use of a lightweight fill material in the construction. Extremely lightweight fill material such as expanded and extruded polystyrene blocks were used in a number of developed countries in the construction of road embankments over soft ground and in landslide repair. However, these materials are to be imported to the country and would be very expensive. As such, from a local point of view, a process involving the use of such materials would not be economically competitive. In order to find an economically feasible solution, the lightweight fill materials should be developed with the locally available inexpensive raw materials.

As such, lightweight fill materials were developed locally by mixing with different proportions of tyre chips with lateritic soil, sawdust with lateritic soil and paddy husk with lateritic soil. Tyre chips were obtained by shredding discarded motorcar tyres. Sawdust was obtained from wood mill waste and paddy husk was obtained from rice mill waste. The developed fill material should of sufficiently low density and workable. Different mix proportions were tried out to get several suitable mixes. The developed material should be sufficiently incompressible and should possess adequate shear strength. Further detailed tests were conducted on selected mixes to establish their engineering characteristics in relation to strength and stiffness.

The effectiveness of the use of lightweight fill material in the embankment construction was studied in detail by the finite element package CRISP. The set criterion was that the in-service settlement of the road should be less than 50mm.This was achieved through the preloading process. In this study a comparison was done for two different approaches; one constructed with lateritic soil and the other incorporating the developed lightweight fills in the preloading process. The placement of the fill layers, the settlement of the peaty clay, the effect of the removal of the preload and the application of the pavement and

traffic load was studied with a fully coupled Modified Cam clay constitutive model. Parametric analyses were also done varying the thickness of the embankment and the peaty clay. The process was found to be helpful in reducing the construction period and consumed fill volume. The advantages were more prominent with the increase of embankment height and soft layer thickness.

INTRODUCTION

1.1 Difficulties in construction on soft ground

There are large number of sites underlain by soft peaty clays in Colombo and its suburbs. The thicknesses of deposit vary from 0.5m to as high as 15m and are present close to the surface level. The ground water table is also very close to the surface level. These areas are now being used for construction of buildings as well as for the development of new infrastructure facilities such as highways due to the scarcity of land.

Soft peaty clays are of very low shear strength and are extremely compressible due to their very high water contents and void ratios. Construction done on peaty clays would be subjected to very large settlements due to their high compressibility. This poses various problems during and after construction. Catastrophic failures are also possible due to very low shear strength. Problems due to these characteristics can be overcome by either transferring the loads to a harder stratum underneath through piles or improving the peat to the desired level by suitable ground improvement techniques.

Use of piled foundations is not a cost effective option for highways but it is ideal for cases such as multistoried buildings that are occupying a relatively small area. In the case of new highways that are of extensive plan areas, and with buildings with smaller intensity loading, the improvement of ground to an appropriate level before the construction would be more cost effective.

1.2 Improvement of peaty clay

Ground improvement methods alter the properties of the sub soil to enable the construction to be done subjected to the relevant constraints. Number of ground improvement techniques are available for the improvement of compressibility and shear strength of soft peaty clays. These techniques can be classified as densification techniques and solidification techniques. Preloading, vacuum pre consolidation and dynamic compaction are some of most widely used densification techniques. Addition of a cementations material and mixing over the full depth of the layer in order to solidify the peaty clay can be regarded as a solidification technique. These ground improvement techniques have advantages and disadvantages depending on the prevailing condition in a given project.

1.2.1 Removal and replacement method

Replacement of the peat with better material is done either after removal using mechanical equipment or by direct displacement. Mechanical excavation has been carried out to depths of about 5.5m but it is costly to remove such high thickness of peaty soil and replace with better material. Usually up to around a thickness of 3m partial or total excavations of the peat and backfilling with granular material may be economical.

1.2.2 Stage construction

This technique involves determining the rate of construction that will allow the soil to consolidate and increase in strength sufficiently to maintain an adequate factor of safety against bearing capacity failure for the corresponding increment of construction loading. By proceeding in constructional steps the foundation soil eventually becomes sufficiently strong to support the full construction loading. Because the soil settles during the construction phase the method is usually applied to earth embankments rather than to rigid foundation.

1.2.3 Pre consolidation by preloading

The most widely used method to improve the properties of peaty clay is the pre-consolidation by preloading. The peaty clay has to be consolidated under the weight of a sufficiently large surcharge. The load has to be there for a sufficiently long period. In this method the pressure applied by the surcharge should be greater than the pressure that the ground will experience due to the proposed structure. In order to prevent shear failure the required surcharge may have to be applied in stages.

Laboratory tests that have been conducted to assess the improvement of strength and stiffiness of peat through pre-consolidation had shown that strength and stiffness properties could be significantly improved by the pre-consolidation process.

To speed up the consolidation process further, prefabricated vertical drains could be installed prior to the preloading. Prefabricated vertical drains take advantage of the higher permeability of soils in horizontal direction. Once the drains are installed vertically pore water is pushed out laterally or vertically due to pressure gradients created by preloading. Pore water that moves laterally will meet vertical drains and will move out of the soil through the vertical drain. The purpose of vertical drain is to shorten the drainage path for the pore water to be discharged. At early stages vertical sand drains were used. At present prefabricated vertical drains are the most widely used type.

1.2.4 Electro-osmosis consolidation

In this method the water is taken out of the soft clay by the application of a DC voltage difference. As there is no loading involved it would not cause shear failures. In a field application, electrically conductive vertical drains are to be used in an appropriate grid.

1.2.5 Deep mixing method

This method is based on the chemical reaction between clay and chemical agents. In common practice quicklime or cement is used as a chemical agent. In the field a large machine with rotating blades is used for supplying the chemical agent into the soil and for insitu mechanical mixing of soil and agent. When, for example quicklime is used as a chemical agent, it absorbs the pore water, changes into hydrated lime and reacts with clay particles to form a pozzolanic reaction product. The cost of soil improvement by this method is relatively higher than the conventional methods. However the strength increase and reduction of settlement is amazingly large and quick.

1.2.6 Dynamic consolidation

In this method the soft compressible peaty clay is compacted and consolidated by dropping a heavy weight from large heights after placing a granular fill layer. The heavy tamping has been found to reduce air voids and to include general enforced settlement. This method, which was originally used for the densification of loose sandy strata, has been successfully used on peaty soils. The craters formed by the impact of the dropped weight are then filled with sand or any other good quality cohesion less material, and the compaction process repeated. Some level of mixing and replacement of the peaty clay also take place in this process.

1.2.7 Vacuum consolidation

Vacuum consolidation is an effective means for improving saturated soft soils. In this method the site is covered with an airtight membrane and a vacuum is created underneath it by using a vacuum pump. Usually prefabricated vertical drains are installed in an appropriate grid in the sub soil.

Instead of increasing the effective stress in the soil by increasing the total stress using conventional mechanical surcharge, vacuum assisted consolidation preloads the soil by reducing the pore pressure while maintaining a constant total stress. Thus it eliminates the risk of shear failure due to loading. It can be also used in combination with a surcharge.

1.3 Another possible solution – Use of a lightweight fill

Another possible approach to solve this problem is the use of a fill material of lower density in constructions done on soft peaty clays. This would be particularly useful in the construction of embankments and in approaching fill behind bridge abutments. Lightweight fill material imposes lower loads on the underlying soft soils resulting in lower settlements and reducing the possibility of shear failure. There are records of the use of lightweight fill in several applications in U.K. Some of the lightweight fill material used in developed countries are Pulverized fuel ash, expanded and extruded polystyrene blocks and, expanded shale and clay.

1.3.1 Expanded and Extruded Polystyrene blocks

An ultra-lightweight fill material with densities in the range of 20-40 kg/m^3 can be formed from expanded polystyrene beads or extruded polystyrene foam (Clarken 1986). Polystyrene has been used in Norway since the early 1970's. At present they are used for the construction of highway embankments in settlement sensitive areas (Aaboe 1986) and in the case of repair of landslides in highways (Brown 2002). When they are kept under water, full saturation could increase the density to a still very low value of 100kg/m^3. However, both forms of polystyrene are expensive with prices being around 50US$/m^3. There are different grades of polystyrene with compressive strength at 10% strain ranging from 25kN/m^2 to 250kN/m^2. Hartlen (1985) reported that the typical tangent deformation modulus is about 5MN/m^2 for polystyrene with a density of 20kg/m^3.

No degradation of the material has been reported due to immersion in water. A number of hydrocarbons such as petrol and tar products will cause polystyrene to dissolve and collapse. They could also catch fire easily. As such, sufficient coverage by earth fills or concrete is needed to provide protection from fire or petrol spillages.

1.3.2 Pulverized Fuel Ash

Pulverized fuel ash is a waste product from coal powered electricity-generating stations. It can be placed and compacted using conventional plant to produce a fill of lower bulk density than conventional fill materials, typically between 1300 and 1700 kg/m^3. The strength of Pulverized fuel ash increases with time in the presence of moisture due to pozzolanic action, reducing self-settlement and lateral pressures on retaining structures. (Riorden and Seaman 1994)

1.3.3 Expanded Clay

Expanded clay is a light weight aggregate produced by firing clay in rotating kilns. The resulting product comprises porous spherical pellets of diameter 5-20mm with air filled cells. Expanded clay has traditionally been used in the UK as an aggregate for lightweight concrete, fire resistant material, etc. The bulk density of the expanded clay after compaction is approximately 250 to 350 kg/m^3. However, due to absorption of water through the open cells in the particles the overall bulk density can increase to 700 – 800 kg/m^{3}over a number of years. (Riorden and Seaman 1994)

1.4 Need to develop a material locally and objective of the project

As it was not possible to produce any of the above lightweight fill materials economically under Sri Lankan conditions an attempt was made here to develop an alternate cost effective light weight fill materials using materials available locally. The primary objective of this research is to develop lightweight fill materials that can be used in construction on soft peaty clays. Attempts are made to develop three types of lightweight fills by mixing different percentage of tyre chips with lateritic soil, saw dust with lateritic soil and paddy husk with lateritic soil.

The developed lightweight fill material should be of sufficiently low density and of low compressibility. If it is too compressible, it will induce large settlement in the constructed embankment and its usefulness could be reduced. As such, the compressibility of the lightweight fill material should also be of the same order as compacted conventional fill. Also developed lightweight fill material should be sufficiently strong so as not to be subjected to shear failures easily. In the construction of embankments or in the use as fill behind retaining walls it is important that the lightweight fill material has shear strength parameters comparable with that of compacted soil fill. Therefore suitable laboratory tests were done to evaluate the engineering characteristics mainly on; compressibility, swelling and the shear strength.

To evaluate the effectiveness of the applications of the proposed lightweight fill material in construction of embankments on SriLankan peaty clay, a construction process using a lightweight fill material is proposed. The process is numerically simulated by the SAGE CRISP finite element package. Soft peaty clay is modeled by modified cam clay and other fill materials are modeled by the Elastic Perfectly Plastic model. Critical state model parameters for soft peaty clay are derived using the results of the laboratory tests.

Parametric analyses are done by varying the soft layer thickness and the height of the embankment to evaluate the effectiveness of the proposed process.

1.5 Arrangement of thesis

Chapter 1: -it includes the nature and problem of peaty clays available in SriLanka and the suitable ground improvement techniques and other possible solutions used elsewhere such as lightweight fill material. It also out lines the objectives and brief methodology of the study.

Chapter 2: - Presents the literature review. In this chapter general description of lightweight material, engineering properties and its application are presented.

Chapter 3: -Presents the development of different lightweight fill mixes with; lateritic soil; tyre chips, lateritic soil: sawdust, lateritic soil: paddy husk for the production of lightweight fill material. The chapter also discusses about workability, achieved dry density, compressibility and swelling characteristics and shear strength characteristics of fill material.

Chapter 4: - of the report presents the proposed process for the construction of embankments. Chapter discusses the need to find a suitable model to represent the consolidation behavior of peat. Various numerical procedures adopted during the simulation process is also presented in this chapter.

Chapter 5: - Presents the details of the proposed construction process the numerical procedure adopted and the simulations done with the CRISP program.

Chapter 6: - Presents the result obtained by the simulation of the proposed construction process using the finite element package CRISP.

Chapter 7: -Summarises the findings of this research project and suggests areas for further research and development.

LITERATURE REVIEW

2.1 Artificial Fills

In recent years, the use of artificial fills has become more common for a variety of reasons. In and around urban areas, finding large quantities of fills of suitable quality has become a problem, and sometimes they are found to be contaminated. Environmental regulations pertaining to filling operations have also made the use of normal fills more costly and problematic.

Artificial fills are defined here as those that are manufactured or processed in some way, and include foam plastics such as Styrofoam and Styropor, Elastizell, Geocell, and Solite. An additional feature of all of the materials described is that they are all lighter in unit weight than inorganic and natural fills and so a weight credit is associated with their use. When dealing with the construction over weak soils, this advantage can be of paramount importance.

If a hole is dug and the material that is removed is immediately used to fill the hole, the state of stress below is unchanged. It follows if a lightweight backfill is used, there will accrue a weight credit for a proposed structure. In an extreme case, if the weight of the backfill plus the structure is equal to the weight of the soil removed, then no settlement of the structure can occur. This principle is used extensively in Mexico City to produce so-called floating foundations on the highly compressible volcanic clays underlying the region.

A recent example of a case where foam plastic backfill would have provided a better and less expensive solution to a soil problem was the construction of a major arterial highway in Staten Island, New York. The method specified in the contract for subgrade stabilization was to excavate unsuitable material and backfill with compacted 1.5-inch broken stone. The purpose of using 1.5 inch broken stone instead of sand fill was to have larger voids, there by generating a larger weight credit (Monahan 1993).

Recently a research project was completed that establishes some important "geo-engineering' properties of one form of rigid foam plastic, expanded polystyrene (EPS). Negussey and Jahanandish (1993) tested and compared some engineering properties of EPS to soil, comparing EPS samples of two densities to samples of soft inorganic clay and uniform silica sand. There results indicate that the engineering properties of EPS can be quantified in a manner similar to earth material. However, they are of extremely low densities in the order of $15kg/m^3$.

EPS blocks were used to reconstruct an embankment slide that had occurred near Durango, Colorado, USA. The job was designed and supervised by the Colorado Department of highways (Monahan 1993). Approximately 40,000 cubic yards of EPS blocks was used as a fill for a new shopping mall in Syracuse, NY (Horvath, 1992).

Norwegian experience with EPS construction has encompassed more than 100 road projects since 1972 (Flaate, 1987,1989). Canadian experience includes the use of EPS backfill behind seven newly constructed bridge and overpass abutments in soft foundation areas near Vancouver (DeBoer, 1988).

2.2 Waste Material as Fills

As urban and industrial developments occurs in a given region, good fills within reasonable haul distances become more scarce and expensive. At the same time, the process of urbanization and industrialization generate larger and larger quantities of what was once considered waste but is now being viewed as recycleable material. Among these products are; ash, glass, rubber (tyres), wood chips, aluminum and other metal containers, and a broad category one might call construction rubble.

2.2.1 Civil Engineering Applications of Recycled Rubber from scrap tyres

Scrap tyre chips and their granular counterpart, crump rubber and have been successfully used in a number of civil engineering applications. Tyre chips consist of tyre pieces that are roughly shredded into 2.5 to 30cm lengths. They often contain fabric and steel belts that are exposed at the cut edge of the tyre chip. Tyre chips have been researched extensively as lightweight fill for embankments and retaining walls (Tweedie et al., 1998; Bosher et al, 1997;Masad et al, 1996; Upton and Machan, 1993; Humphrey and Manion, 1992), trench backfill for buried pipes (Gaseroiek et al, 1995), soil reinforcement (Foose et al. 1996), and have also been used as drainage layer for roads and septic tank leach fields (Humphrey, 1999). According to Humphrey, (1999) some of the beneficial properties of tyre chips in civil engineering applications include low material density, and high bulk permeability. In many cases, scrap tyre chips may also represent the least expensive alternative to other fill materials.

The hydraulic conductivity of secondary tire shreds was measured using large-scale constant head permeameter (Reddy and Saichek 1998b) .The hydraulic conductivity of tire chips under no vertical stress was too high to measure in the permeameter. However when a vertical stress of 163kPa was applied, the tire chips compressed approximately 53% and the resulting hydraulic conductivity was reduced to 0.793 cm/s as the normal stress increased, the compressibility increased and the hydraulic conductivity decreased. Under the maximum vertical stress of 1006kPa, the tyre chips compressed by 65% and the hydraulic conductivity decreased to approximately 0.01 cm/s. Thus, even at high normal stresses, the tyre chips possess a high hydraulic conductivity.

The compression of shredded tyre chips under repeated loading and unloading conditions that represent typical construction loading conditions was determined by cyclic compression testing (Reddy and Saichek 1998b). This testing was performed by placing tyre chips in a PVC compression mould approximately 0.36m in height and 0.3m in diameter with an initial unit weight of 416.5 kg/m^3. For this testing vertical stress increased in small increments of $1.2kN/m^2$ until a maximum stress of $31.8kN/m^2$ was reached. This load was then unloaded completely in small increments. During the first cycle of loading, the tyre compressed by 31% and when unloaded a residual compression of 12% was observed. During the second cycle of loading, the tires compressed to 32% and the residual compression increased to 20%. In the subsequent third and fourth compression cycles, the rate of compression and rebound is decreased, but the residual compression remained constant at approximately 22%.

Humphrey et al. (1993), Foose et al. (1996), and Gebhardt (1997) conducted direct shear tests on tyre chips and shreds ranging in size from 38 to 140 mm. If no peak shear stress is observed, the shear stress at 10% strain was taken as the peak value. The shear strength parameters for Mohr-Coulomb failure envelopes were calculated for tyre chips. Friction angles from these studies ranged from 19^0 to 38^0 with cohesion of 0 to 11.5kPa at normal stress between 0 and 83 kPa. Previous studies (Foose et al, 1996) interpreted the results according to accepted soil mechanics principles. However, Gebhardt (1997) suggested a power function to describe the relationship between direct shear strength (τ) and normal stress (σ) as $\tau = \sigma^{0.79}$.

Bressette (1984), Ahmed (1993), Benda (1995), Masad et al (1996), Wu et al. (1997) and Lee et al. (1999) conducted triaxial tests on tire chips of sizes changing from 2 to 51 mm. Tests were conducted under compression loading by most researchers. Wu et al. (1997), however, conducted compression unloading tests where the confining pressure σ_3 was reduced in increments from the initial consolidation pressure while simultaneously increasing the deviator load to keep σ_1 constant. A linear stress strain response up to 30% strain was observed from all compression-loading test at confining pressures between 35 and 350 kPa. Since strain softening was not observed, deviator stress at 10 to 20 % axial strain were selected as the shear strength for the determining Mohr Coulomb parameters shear strength parameters. Friction angles ranging from 6 to 57, and cohesions varying from 0 to 82 kPa were obtained. The initial tangent modulus of the stress strain curves, analogous to Young's modulus, ranged between 300 to 2500 kPa with higher values obtained at higher confining pressure.

2.2.2 Importance of Soil-Tyre mixtures in Civil engineering application

One potential problem when using tyre chips in earthwork applications is spontaneous combustion. For example, two tyre chip embankments spontaneously combusted in the state of Washington (Nightingale and Green 1997). However many other embankments which have been built are in operation without any evidence of this phenomenon. Moreover, spontaneous combustion is unlikely in fills containing tyre chips mixed with soil due to oxygen limitation, and no fires in soil tyre chips fills have been reported. Another issue frequently considered is the potential for ground water contamination by leachate from tyre chips fills. A recent study conducted by Edil et al. (1998) showed that contaminants of concern are leached only in very minimal quantities from tyre chips fill.

Edil and Bosscher (1994) and Humphrey and Sandford (1993), have shown that preloading can control the compressibility of tyre chips. Edil and Bosscher (1994) recommended a soil cap at least 1 m thick be placed over tyre chips fill to limit settlement under traffic loads or surcharge.

Humphrey and Sandford (1993) suggest that a soil cap 0.6-1.8m thick should be placed on top tyre chips embankments to prevent excessive deflection of overlying layers. Bosscher et al. (1997) report that the compressibility of tyre chips can be reduced significantly by adding 30 – 40 % sand by volume.

Humphrey and Sandford (1993), Foose et al. (1996), and Bernal et al. (1996) reported that pure tyre chip mixtures have a friction angle of 20^0 –35^0 and cohesion of 3-11.5 kPa based on large scale direct shear tests. Triaxial test conducted by Wu et al. (1997) on small tyre chips (< 40mm long) indicate that the friction angle can be in excess of 40^0 and that the cohesion intercept is negligible. Existing field evidence also supports that there is a high friction angle from pure tyre chips (Edil and Bosscher 1994).

Ahmed (1993), Humphrey et al. (1993), Edil and Bosscher (1994), Foose et al (1996), and Bernal et al. (1996) have reported that sand can be reinforced using tyre chips. These studies have shown that adding tyre chips increases the shear strength of the sand, with friction angles as large as 65 being obtained for mixtures of dense sand containing 30% tyre chips by volume. Foose (1993) shows, however, the strength decreases when the tyre chip content increases beyond 30% because the sand tyre chip mixture behaves less like reinforced soil and more like a tyre chip mass with sand inclusion.

2.2.3 Environmental Studies of Recycled Rubber

Recycling materials conserves valuable natural resources and reduces the amount of waste entering landfills. One issue to consider, however, is the suitability of using recycled material in subsurface applications. In addition to being technically beneficial and economically viable, a material must be environmentally benign to be considered acceptable (Tikalsky et al., 1998). Acceptable limits for metals and organics in leachate from shredded tyres are discussed in ASTM D 6270-98 (2002), which provides guidelines for using scrap tires in civil engineering applications. According to several studies, Toxicity Characteristics Leaching Procedure (TCLP) test results show that tire shreds are not a hazard to human health (Downs et al., 1997) Laboratory test results (Downs et al., 1997 Twin city testing, 1990) have shown that metals are leached at low pH and that organics are leached at high pH. This indicates that unbound shredded tires should be used in environments where the soil and ground water are at a fairly neutral pH.

Field studies have also been conducted to investigate the effect of leachate from tyre shreds placed above and below the water table (Downs et al., 1997; Humphrey et al., 1997;Edil and Bosscher, 1992). Concentration of iron and manganese exceeded secondary drinking water standards in both cases. Since secondary standards apply to the aesthetic and not health, in the quality of water, this is a minor concern unless state or local regulations require leachate to meet more stringent concentration levels. Levels of organic compounds in the leachate from tyre shreds located above the water table were below detection limits but those below the water table were detectable.

Downs et al., (1997), Humphrey et al., (1997), ASTM D 6270-98 (2002) specifies that applications involving tyre shreds should be confined to locations above the water table until it is determined that these detectable levels are not a threat to human health.

The Wisconsin study (Monahan 1993), in addition to comparable determination of engineering properties, also investigated the environmental suitability of shredded rubber. Parameters tested in hypsometer (leachate) studies included COD, BOD, CL, SO4, pH, alkalinity, hardness, TDS, Ba, Fe, Mn, Zn, Pb, and Na.While cautioning that there is need for additional studies, the test result "indicates that the shredded automobile tire samples show no likelihood of being a hazardous waste" (Edil and Bosscher, 1992). This study being a general investigation of shredded tires in highways applications, also includes studies of models, test embankments, and pavement design, and lists recommendations for design and construction specifications

2.2.4 Other general Studies of Recycled Rubber

The use of shredded rubber tyres and other waste materials as fills is a very sensible approach to the problem because of the large volume of material that can be recycled. A full session, encompassing six papers, was held at the 72nd Annual Transportation Research Board meeting, entitled "use of light weight waste material for embankments over soft soils", with emphases on shredded rubber tires and wood chips as fill (TRB, Washington, DC 1993)

Comprehensive research programs dealing with the use of shredded tires have been completed at the university of Maine, the university of Wisconsin, and Purude University. Projects using both shredded tyres and woodchips have been conducted in Oregon, Washington, and Minnesota (Monahan 1993). .

The Maine research study (Monahan 1993) determined the properties of gradation, sepecific gravity, absorption, compacted density, shear strenth, compressibility and coefficient of lateral earth pressure at rest. The latter property was investigated because the focus of the study was for the possible use of shredded tires as retaining wall backfill.

The Maine, Wisconsin, and Purdue studies (Monahan 1993) reported compacted unit weights of tire chips to be significantly lower than soil fills. With values ranging from about 320 – 640 kg/m^3 for pure tire chips. The Wisconsin studies also tested varying mixtures of soil and tire chips, as well as layered systems.

In more recent civil engineering applications, tires were shredded into smaller pieces and the resulting bulk material was used as subgrade fill, replacing heavier conventional fill material such as gravel. Vertical stresses in fills are thereby reduced. In a highway embankment in southern Oregon, an ancient landslide was re-mobilized after the embankment was raised and widened as part of an improvement project (Read, et al, 1991). To decrease load on the embankment, and to reduce slide movement, 580,000 units shredded tires were used as subgrade fill in place of existing soil. Subsequent observation and testing indicates that the embankment is performing adequately. Upton et al. (1992), commenting on the condition of the above road, stated that "tests indicate the pavement section over the shredded tire fill meets 20-year design life criteria [for pavements]", demonstrating the integrity of roads constructed over shredded tire fills. Edil and Bosscher (1992) reported similar findings for a test embankment in Wisconsin that was constructed using sand/gravel and shredded tire layers of varying thickness, and subjected to low-volume truck traffic for several months.

The combined use of wood fiber and geotextile reinforcement was reported in the construction of a lightweight fill across a swamp area in Washington. Staged construction techniques, planned on the basis of careful instrumentation monitoring, were used to maintain stability during the controlled rate of construction. The wood fiber used was of the fresh classification. Environmental regulations required that the wood fiber not extend to a level below mean high water. Additionally, a topsoil thickness of about 2 ft was placed over the wood fiber to guard against fire and exposure to oxygen. Despite heavy logging traffic, no serious pavement distress has been noted in the 5 years service period. Water sampling of leachate, and physical inspection of the wood fiber near the surface indicate no pollution or wood chips deterioration. A cost saving of approximately $500,000 was realized over the net lowest viable option (Allen and Killian, 1993).

In Minnesota, a job was completed using wood fiber, shredded rubber tires, and geotextile in an embankment design that was constructed to cross weak peat soils. A Geotextile was placed at the bottom of a 5 ft excavation, and wood chips were placed at a height of 1 ft above the water table, as required by the Minnesota Pollution Control Agency. Shredded tyres were then placed to a height 3 ft above wood chips. The tire layer was covered with geotextile, and the fabric was sewn together with the lower fabric to form an enclosing bag (Monahan, 1993).

2.2.5 Concluding Comment

Extremely lightweight fill material such as polystyrene has been used in developed countries with encouraging results. Importing them to be used locally would not be economically feasible. On the other hand material such as tyre chips had also being used. Having considered the studies done with such materials it is envisaged that a lightweight fill material appropriate for Sri Lanka could be developed by mixing commonly available lateric fills with tyre chips and other lightweight waste products such as sawdust and paddy husk.

2.3 Use of finite element techniques for modelling construction of embankments.

Finite element technique is suitable for analyzing the problem of construction of embankment on soft ground, because, the construction procedure, stress-strain characteristics and time dependent properties of the soft ground and compacted fill materials can be explicitly modeled. However, the accuracy of a finite element analysis not only depends on the constitutive models and the parameters used but also on the numerical techniques adopted, such as the methods of applying embankment load and simulating the variation of permeability of the soft ground with stress levels.

In finite element analysis, the incremental embankment load is applied by one of the following methods.

(1) Applying a surface loading

(2) Increasing the gravity of all or part of the embankment elements

(3) Placing a new layer of embankment elements.

If the embankment load is treated as a surface load, the stiffness of the embankment and the lateral spreading force from the embankment fill are completely ignored. Applying the incremental load by increasing the gravity of the whole embankment is much better than applying the surface loading, but still the sequence at which the load applied to the soft ground is not closely simulated. Since soft ground is not an elastic material, the response depends on the sequence of loading. Furthermore, the stiffness of the embankment may not be modeled properly. Applying the incremental load by placing a new layer of elements is more realistic. However in the case where the fill is of specified thickness, the nodal coordinates of the embankment above the current top surface of the embankment must be updated to account for the deformation during the construction process. Otherwise, the applied total fill thickness will be more than the actual value because of the settlement during the construction process. Most computer programs used for analyzing the behaviour of embankments on soft ground do not model construction well. CRISP computer program (Britto and Gunn 1987) is superior in many aspects.

For accurate prediction of the behaviour of embankments on soft ground, another key point is simulation of the consolidation process. The consolidation rate is mainly influenced by the permeability of the foundation soil. The permeability of soft ground varies during the loading and the consolidation process, and a significant change occurs before and during the yielding of soil (Tavenas and Leeroueil 1980; Tavenas et al. 1983). However, most finite element models do not consider the significant change in soft ground permeability before and after the soil yields, (Tavenas and Leeroueil 1980) and therefore could not simulate the whole consolidation process. It is important to consider the foundation soil permeability variation during the construction and consolidation process.

The actual embankment construction of placement and compaction of the fill material layer-by-layer is simulated in the finite element analysis. If the thickness of the placed layer is added on in each incremental construction step, it does not account for the settlement of compacted layer surface due to settlement of soft soil. Therefore ideally, the coordinates of the surface nodes should be corrected prior to the placement of the next layer. The process of construction of embankments on soft ground is a large deformation problem (Asaoka et al. 1992). When the finite element method is used for analyzing this geotechnical problem, the large deformation phenomenon can be approximately considered by updating the normal coordinates during the incremental analysis. Realistic computations of the variation of displacements and pore pressure with time, in clay foundations under stage-constructed embankments, require the use of numerical analyses with reliable constitutive models with coupled consolidation.

In the CRISP finite element program there is a facility to update the coordinates but it does only in the vertical direction and not in the horizontal direction. But in the embankment problems especially in constructions over peat there is a considerable amount of heaving with horizontals displacements causing problems during the analysis. Lot of researchers have done embankment analysis in CRISP without updating coordinates and got reliable results. In this analysis this problem is minimized in a way whereby embankments are analysed without updating coordinates and settlement in peat is found. Settlements in fill materials are found separately and incorporated in the final settlement.

DEVELOPMENT OF LIGHTWEIGHT FILL MATERIALS

3.1 Introduction

Use of lightweight fill materials such as Polystyrene blocks, Pulverised fuel ash, expanded clay which are used in developed countries, would not be economically feasible in Sri Lanka as they are to be imported to the country. As such, it is important to develop lightweight fill materials with locally available raw materials so that it would provide an economically competitive solution. Lateritic gravelly clay is the most widely used fill material in Sri Lanka. It is commonly available and less expensive than granular fills of sand or quarry dust.

As such, lateritic gravelly clay was selected as the base material and attempts were made to develop various lightweight fill materials by mixing with different proportions of tyre chips, saw dust and paddy husk. Tyre chips were obtained by shredding the discarded motorcar tyres; saw dust obtained as a by-product from wood mills and paddy husk is obtained from grinding mill waste. The developed fill material should of sufficiently low density and workable. Different mix proportions were tried out to get several suitable mixes. The developed material should be sufficiently incompressible and should possess adequate shear strength. Further detailed tests were conducted on selected mixes to establish their engineering characteristics in relation to strength and stiffness.

3.2Development of a lightweight fill material with tyre chips

Large numbers of discarded tyres are added annually to the existing stockpiles. Current disposal and stacking methods of waste tyres are not acceptable due to the possibility of fire and health hazards. Therefore discarded tyres should be used for other applications. One possible application is to use shredded tyres to make a lightweight fill material to be used in construction of embankments over weak or compressible soils. The main raw material tyre chips are widely available and can be obtained at a reasonable low cost.

Case histories of the use of tyre chips alone as a fill material and related issues were discussed in chapter 2.It is believed that the best results could be obtained by developing a mix of tyre chips with soil. Lateritic gravelly soils are the most widely used fill material in Sri Lanka. Lateritic fill material is widely available in the Country and is less expensive than other granular fill material such as sand or quarry dust. Therefore it is proposed to mix tyre chips with the lateritic fill material to develop the lightweight fill.

A lightweight fill material made by mixing of tyre chips of density 320 kg/m^3 and lateritic fill material density of $1850 kg/m^3$ could be quite useful in Civil Engineering constructions in Sri Lanka. Hence it is necessary to decide on a suitable mix proportion so that the developed fill material will have the desired engineering properties and is also economical. Primarily it should be of sufficiently low density. Tyre chips were mixed with lateritic fill in different proportions to get a workable mix of sufficiently low density. After several trials two mix proportions 1:2 (tyre chips: soil) and 1:3 (tyre chips: soil) by weight were used for further testing. These two samples were of sufficiently low density and were of reasonable workability. As such, they were used for further testing. Due to the lower specific gravity of the tyre chips its volume in the mix was much greater than the volume of soil. Basic properties such as density were obtained for a mix of proportion 1:1.However, the detailed engineering parameters could be obtained with this mix due to the problems of workability.

3.2.1 Testing of mixes and density analysis

3.2.1.1Particles Size distribution

Particle size distributions of the two mixes and lateritic soil were analyzed using the standard sieve analysis. The results obtained are presented in Figure 3.1 for the two-mixes and the lateritic soil.

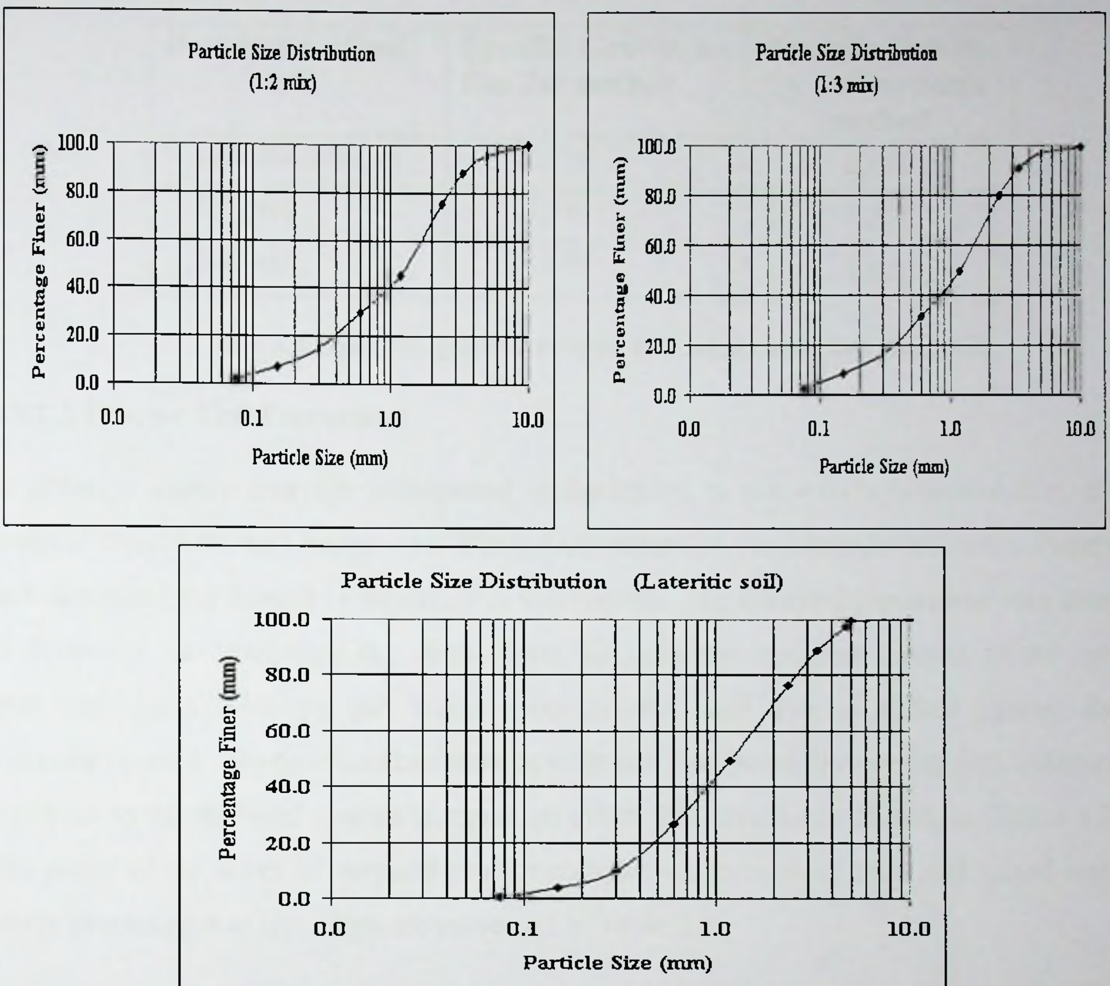


Figure 3.1Particle Size Distribution of Tyre chips mixes and Lateritic soil

3.2.1.2 Specific Gravity

Two methods, namely the density bottle method and the gas jar method, were used to determine the specific gravity of the lightweight fill mixes and lateritic soil. The Gas jar method is normally used when the mix particles are too large for the 50ml density bottle method. It is suitable for particles up to 37.5mm, and larger stones should be broken down to less than this size. It can also be used for fine gained soils. The specific gravity values obtained by the two different methods were different for the mixes. This was due to the fact that a uniform mix could not be used in the density bottle. The values obtained are summarized in Table 3.1.

Description of soil	Specific Gravity by Gas Jar method	Specific Gravity by density bottle method
Lateritic soil	2.700	2.640
1:3 mix	1.945	1.875
1:2 mix	1.695	1.640

Table 3.1 Specific gravity of tyre soil mixes and lateritic soil

3.2.1.3 Proctor Test Properties

In order to ensure that the constructed embankment is not overly compressible, the material should be well compacted. The determination of the optimum moisture content and maximum dry density is important in this context. The standard proctor test was done to determine the maximum dry density and the optimum moisture content of the two proposed mixes. The dry unit weight obtained for each trial is plotted against the moisture content. The optimum moisture content and maximum dry density thus obtained confirms to the standard proctor compaction effort. The results are shown in Figure 3.2. The result of the series of standard proctor compaction tests on lateritic soil mixed with varies percentages of tyre chips are presented in Table 3.2.

Description of mix	Optimum moisture Content (%)	Maximum dry density (kg/m^3)
Lateritic soil	17.55	1780
1:3 mix	17.40	1443
1:2 mix	17.00	1288
1:1 mix	14.20	1173

Table 3.2-Compaction properties tyre soil mixes and lateritic soil

It is clear from the data of Table 3.2 that the optimum moisture content decrease as the percentage of tyre chips in soil increases. This data also reveals that the dry density of the mix decreases as the percentage of tyre chips increases. This behavior is attributed to the fact that tyre chips are a lightweight material as compared to the soil grains. The data indicate that the dry density of the tyre chips mixed with lateritic soil is about two third the dry density of typical soils. Such values show a good potential for the use of tyre chips produce a lightweight fill.

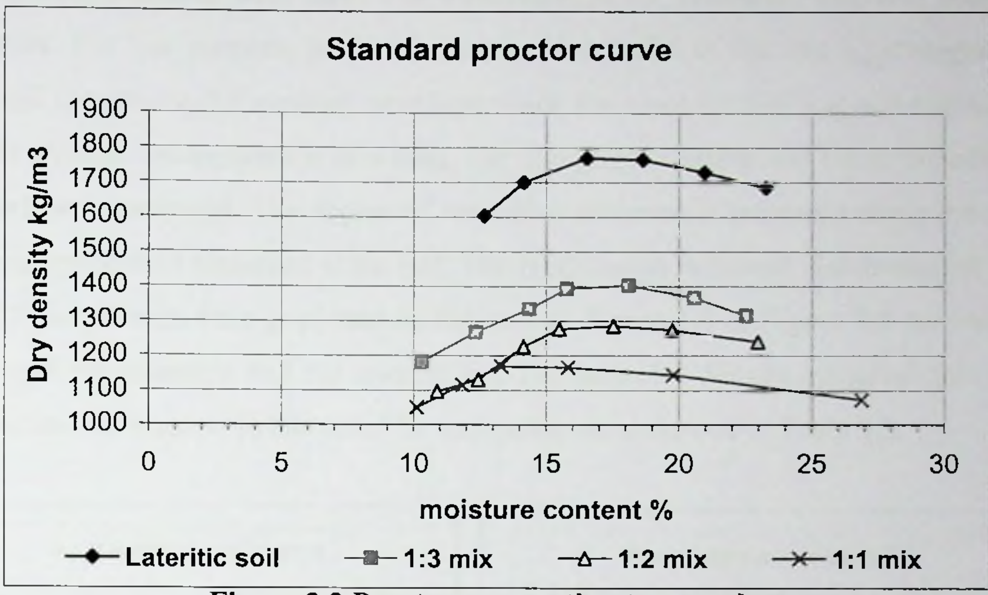


Figure 3.2 Proctor compaction test results

3.2.1.4 Saturation and swelling characteristic of the lightweight fill

The proposed lightweight fill will be used on grounds underlain by soft soil. These sites are often low lying and water table is quite high (almost at the ground level). Therefore, the possibility of lightweight fill getting saturated subsequently exists. Compacted lightweight fill could be subjected to some swelling and increase density during the saturation.

The process of the swelling is the gradual increase in volume of a soil with the absorption of water. When a soil sample is compacted at a given water content, it will be in an unsaturated state. Once the sample is submerged, it would absorb water and sample will get saturated. Some swelling may take place during this process. Therefore it is necessary to find the time required for saturation and the amount of swelling taking place. The amount of swelling occurring during saturation varies with the soil and if the swelling is very high it could be problematic and could have adverse effects on the final performance. Therefore, selected mix designs were checked for their swelling characteristics.

Swelling characteristics of the lightweight fill was studied by placing a sample in the conventional consolidation approaches. The sample was submerged and the free swell occurred was measured with time. The time required for saturation was also found in these tests. For this purpose, prepared compacted samples of the two lightweights fill mixes and the lateritic fill material were kept inside the consolidation ring and submerged in water without the application of a load. The dial gauge reading was taken periodically and swell was computed. The degree of saturation achieved is estimated using the mass and moisture content measured at the end. The final density achieved also measured. The swell of sample with time is plotted in Figure 3.3, Figure 3,4 & Figure 3.5 for the two lightweight fill materials and the lateritic fill. The saturated density achieved, the swell recorded and the time period required for saturation are presented in Table 3.3.

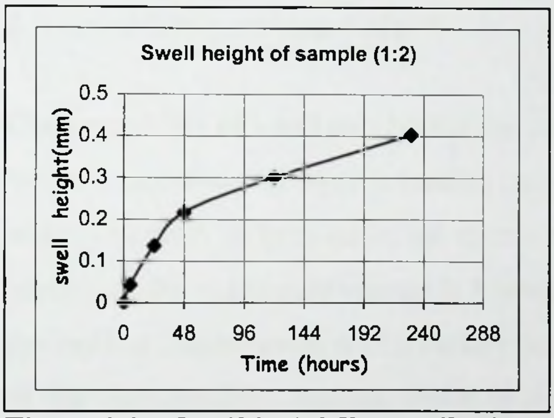


Figure 3.3 – Swell in 1:2 Tyre soil mix

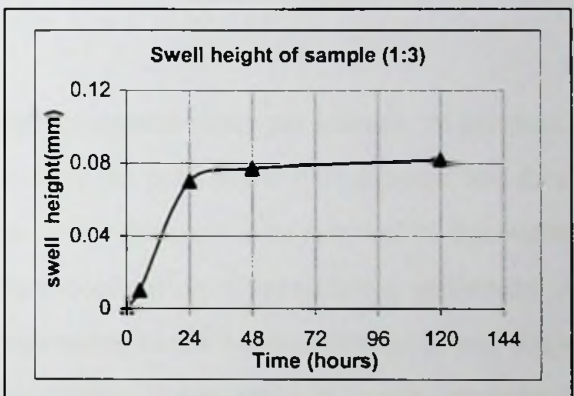


Figure 3.4 – Swell in 1:3 Tyre soil mix

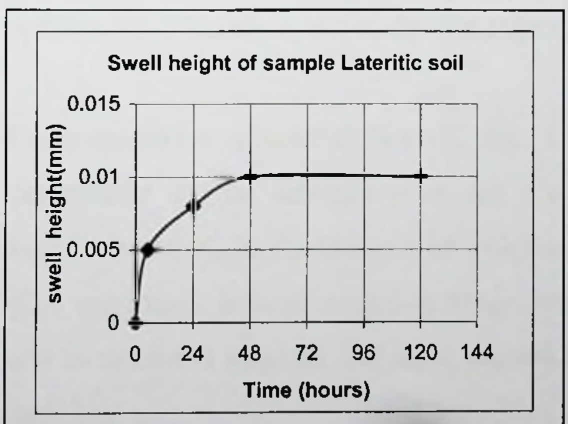


Figure 2.7 – Swell in Lateritic fill

Time (hours)	0	5	24	48	120	240	Saturated bulk Density kg/m^3
Swell in 1:2 fill (mm)	0	0.041	0.136	0.217	0.301	0.403	1641
Swell in 1:3 fill (mm)	0	0.01	0.07	0.077	0.082	-	1761
Swell in lateritic fill (mm)	0	0.005	0.008	0.01	0.01	-	2100

Table 3.3-Swelling characteristics of fill

3.2.2 Compressibility characteristic of fill material

Conditions of loading cause soil layers underneath to undergo certain amount of compression. This compression is due to;

1. Elastic settlement of soil and
2. Consolidation settlement of soil

Compressibility of a soil may be defined as the volumetric strain per increase of pressure. When a saturated soil layer is loaded, immediately the pore water will increase and then water will drain away to relive the excess pore water pressure. The removal of this water leading to the volumetric change is known as consolidation. Consolidation settlement is the vertical displacement of the surface corresponding to the volume change at any stage of the consolidation process. Prior to consolidation there could be some immediate compression. The developed mix should ideally show low immediate and consolidation settlements. Therefore, the study of compressibility characteristics is very important

Compressibility characteristics of the lightweight fill mixes and lateritic fill are determined in the laboratory in the Oedometer. Properties such as coefficient of consolidation **(C_V)**, coefficients of volume compressibility (m_v) and compression index (C_C) are found. It is necessary to obtain these properties in both; as compacted samples and in saturated samples. As such, consolidation tests were carried out on both types of samples.

3.2.2.1 Consolidation tests on as compacted fill

The lateritic soil and tyre chips-soil mixture were statically compacted at optimum water content in a consolidation ring to get a sample having density equal to the maximum proctor density. Vertical stress was increased up to 320kN/m^2 in stages in the standard one-dimensional consolidation setup. The **e-log** σ curves for the lateritic soil and tyre chips mixtures are shown in Figure3.6, Figure3.7 and Figure3.8 respectively. The values of the coefficient of consolidation (Cv) and coefficient of volume compressibility (mv) vary with vertical stress are shown in Figure 3.9.

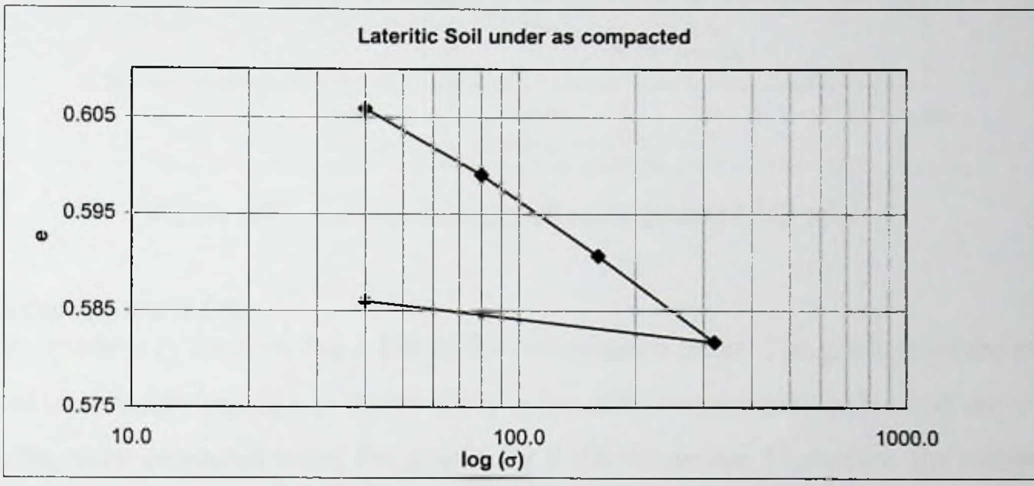


Figure 3.6-Compressibility of as compacted lateritic soil

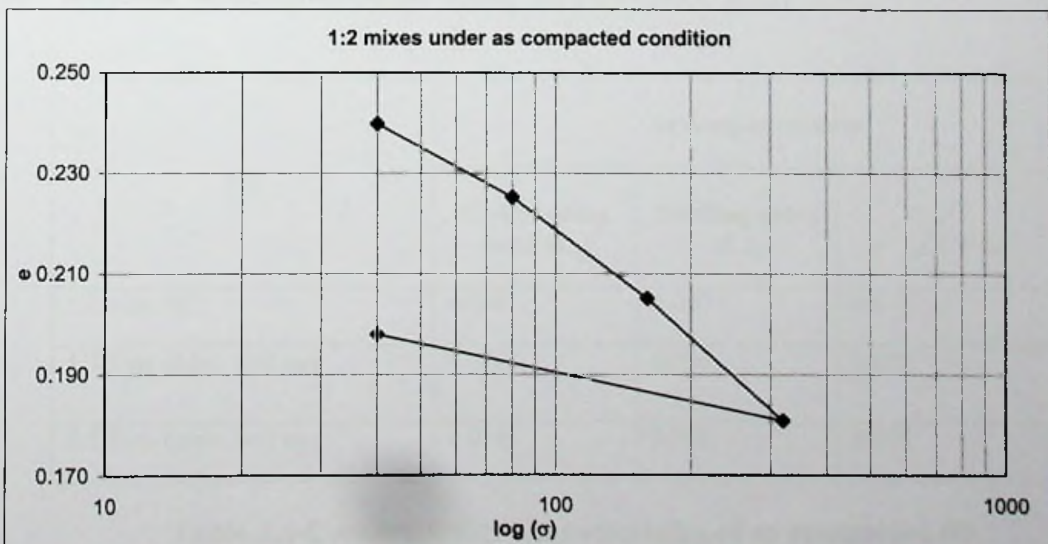


Figure 3.7-Compressibility of as compacted 1:2 mix

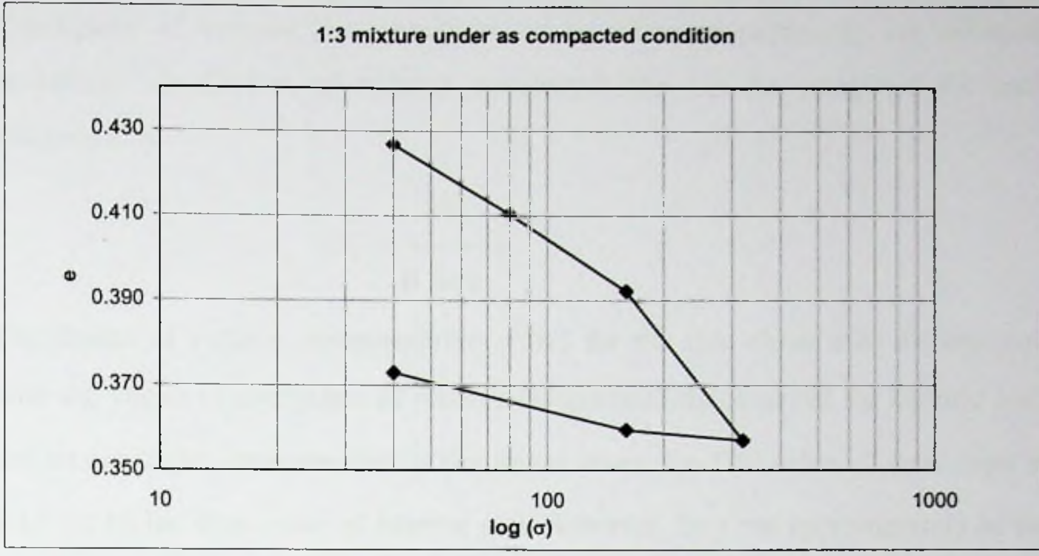


Figure 3.8 – Compressibility of as compacted 1:3 mix

Compression index

The gradient of the e vs. log σ plot is the compression index. The gradient of the swelling line is an unloading line is the swelling index. The compression index and the swelling index were computed using the graphs for different mixes. Thereafter, the compression ratio is computed. The values are presented in Table 3.4.It could be seen that with addition of the tyre chips the compressibility of the mix increased from that of a lateritic soil. However, the compressibility values are sufficiently small.

Fill	As compacted state		
	Compression index (C_c)	Swelling index (C_s)	$C_c/1+e_0$
Lateritic fill	0.029	0.007	0.018
1:3 Tyre chips: Soil mix	0.089	0.026	0.062
1:2 Tyre chips: Soil mix	0.074	0.028	0.058

Table 3.4-Comprssibility characteristics of as compacted fill

Coefficient of volume compressibility

Coefficient of volume compressibility is an alternate parameter for estimating the settlement. Coefficient of volume compressibility can be computed for each load increment by

$$m_v = \frac{\Delta H}{H\,\Delta\sigma v}$$

Coefficient of volume compressibility (mv) for the tyre chips mixture was compared with the values of coefficient of volume compressibility observed for lateritic soil. Since the mix is highly compressible at the initial stage, the mv value of tyre chips mixture little bit higher than value of lateritic soil. However, they are approximately of the same order at high vertical stress levels.

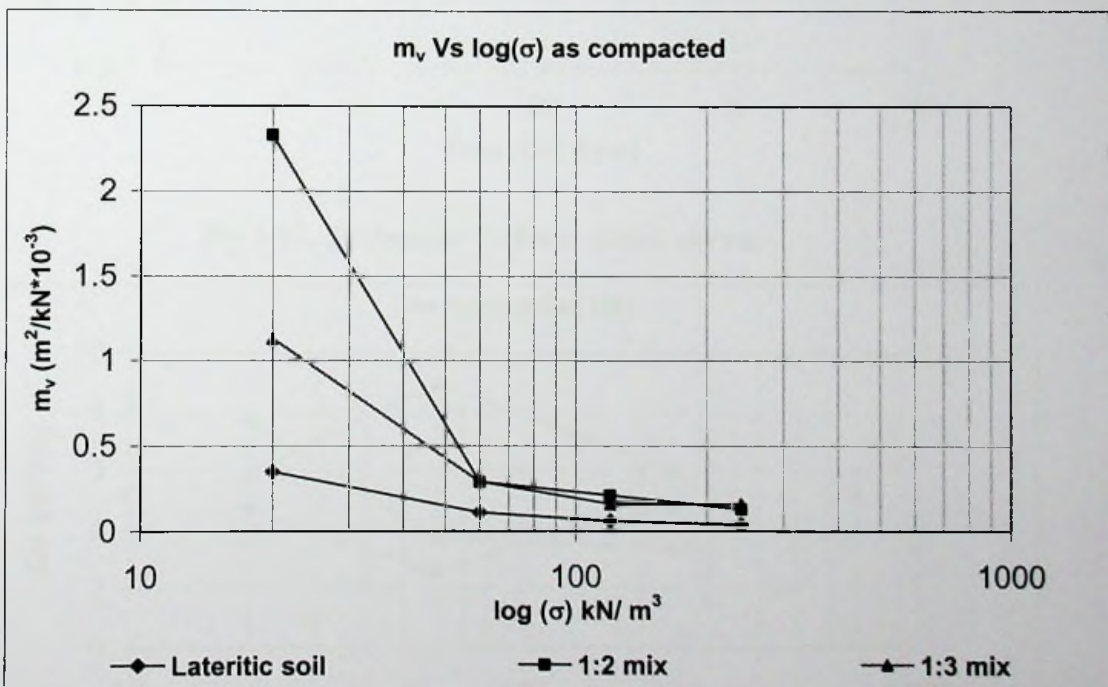


Figure 3.9 Effect on tyre mixing on m_v for lateritic soil

Coefficient of consolidation

When it is required to predict the time rate of settlement of soil in the field, it is necessary to know the coefficient of consolidation Cv for the soil. Magnitude of Cv may be different for the different load increment. The higher initial immediate compression is illustrated by a typical Settlement Vs Root (time) graph shown in Figure 3.10. A high value of Cv implies that the rate of consolidation is high

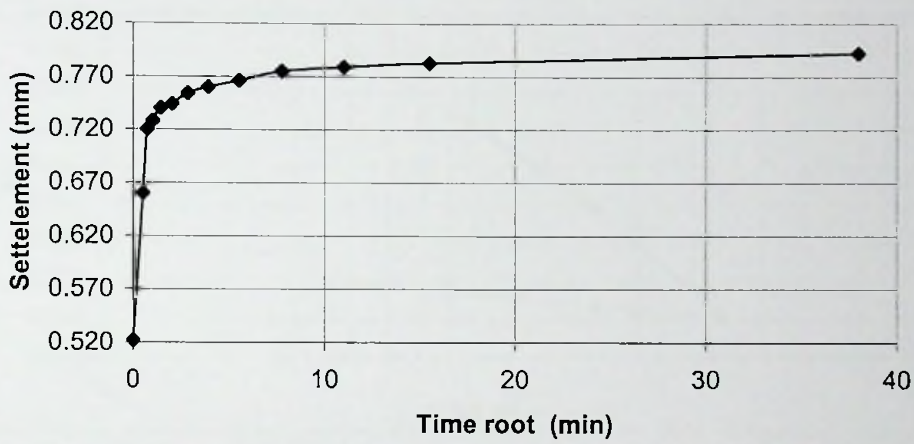


Fig 3.10. Settlement Vs Root (time) curves

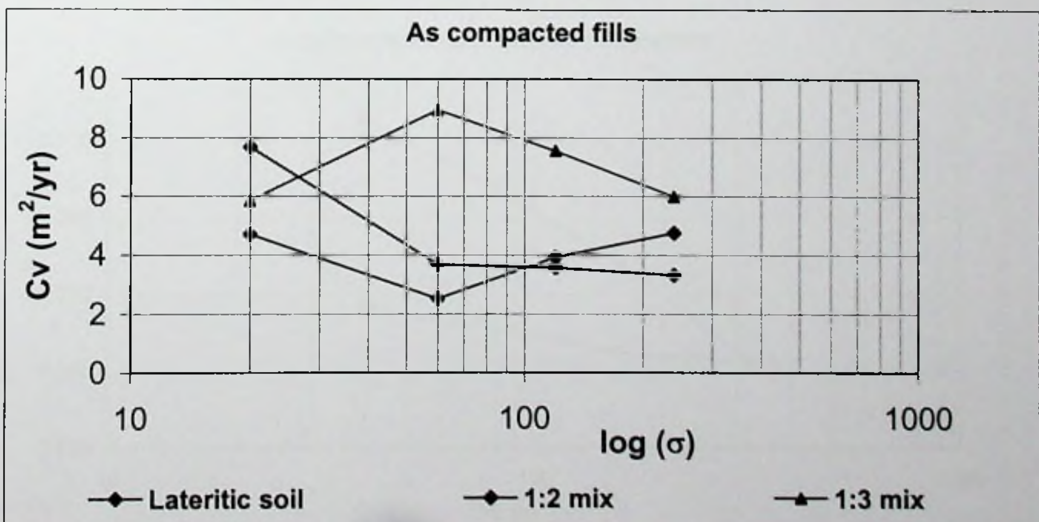


Figure 3.11 Effect on tyre mixing on Cv for as compacted lateritic soil

3.2.2.2 Consolidation tests under saturated condition

After saturating the lightweight fill material in the oedometer the consolidation test was done in stages. The **e-log σ** curves for the lateritic soil and tyre chips mixtures are shown in Figure 3.12, Figure 3.13 and 3.14 respectively. The values of the coefficient of volume compressibility (mv) and coefficient of consolidation (Cv) vary with vertical stress are shown in Figure 3.15 and Figure 3.16 respectively.

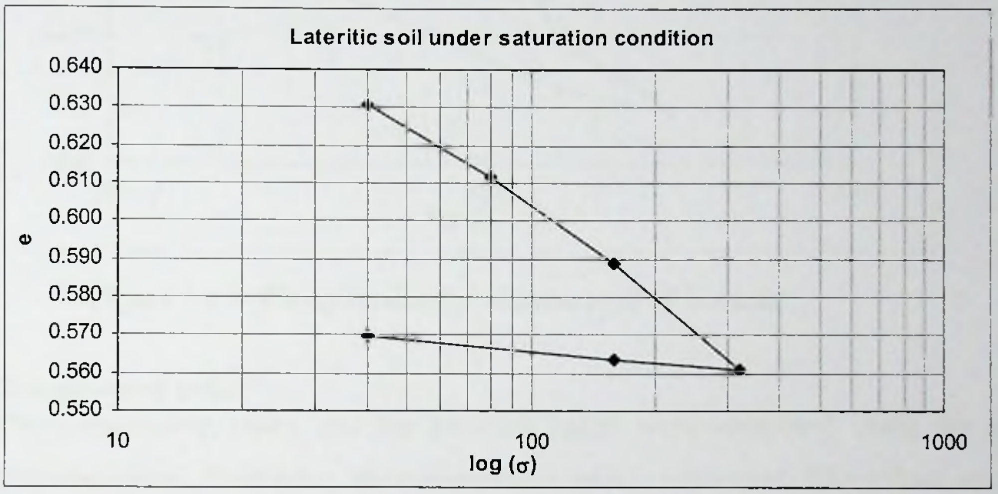


Figure 3.12 –Compressibility of saturated of lateritic soil

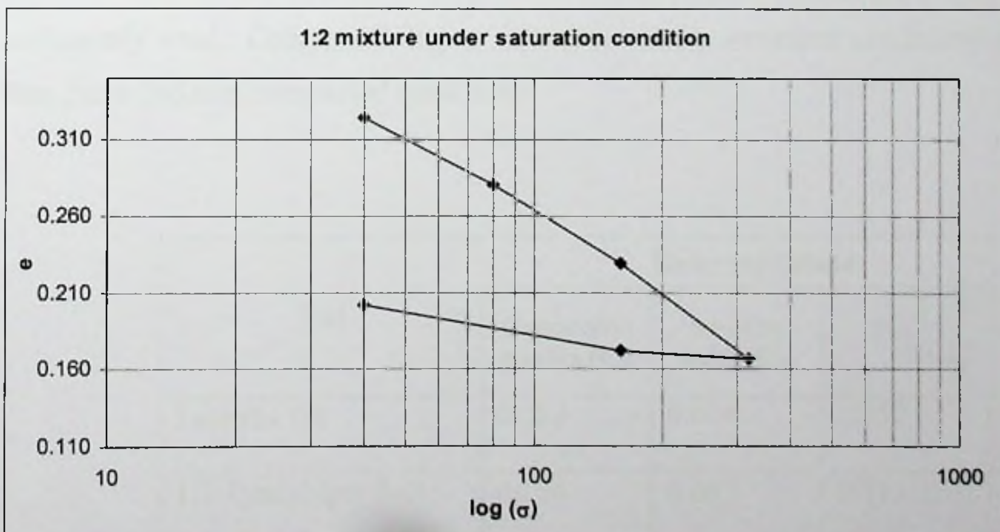


Figure 3.13 –Compressibility of saturated of 1:2 mix

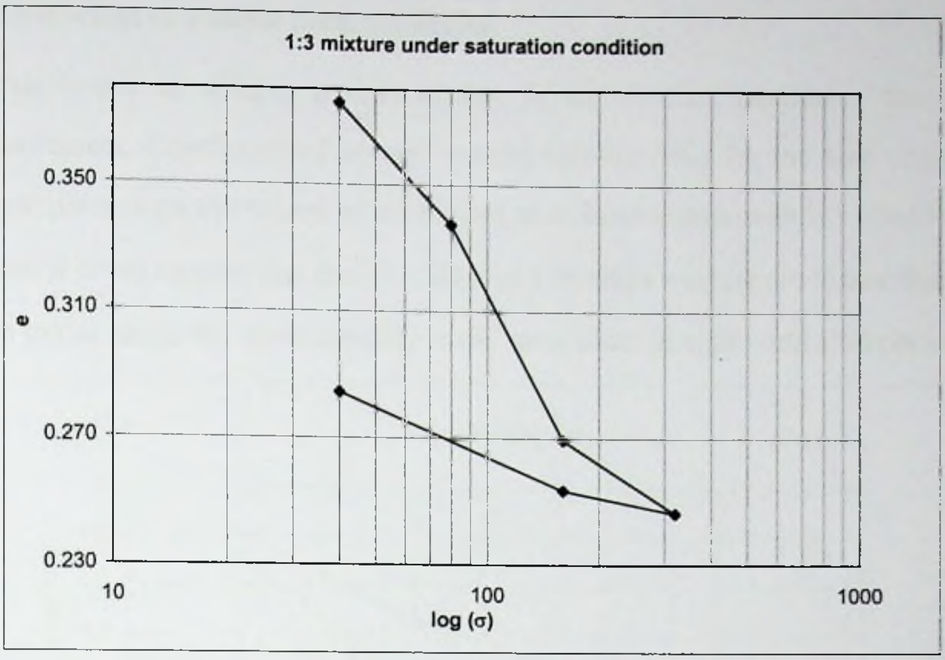


Figure 3.14 –Compressibility of saturated of 1:3 mix

Compression index

The compression index and the swelling index were computed using the graphs for different mixes. Thereafter, the compression ratio is computed. The values are presented in Table 3.5.It could be seen that with addition of the tyre chips the compressibility of the mix increased from that of a lateritic soil. However, the compressibility values are sufficiently small. Compressibility of the mixes under saturated conditions are greater than those under as compacted conditions.

Fill	Saturated state		
	Compression index (C_c)	Swelling index (C_s)	$C_c / 1+e_0$
Lateritic fill	0.084	0.014	0.052
1:3 Tyre chips: Soil	0.156	0.063	0.113
1:2 Tyre chips: Soil	0.187	0.058	0.143

Table 3.5-Comprssibility characteristics of saturated fill

Coefficient of volume compressibility

Coefficient of volume compressibility is an alternate parameter for estimating the settlement. Coefficient of volume compressibility (m_v) for the tyre chips mixture was compared with the values of coefficient of volume compressibility observed for lateritic soil It could be seen that the m_v values of tyre chips mixture are higher than lateritic soils at initial stages but approximately in the same order at high vertical stress levels.

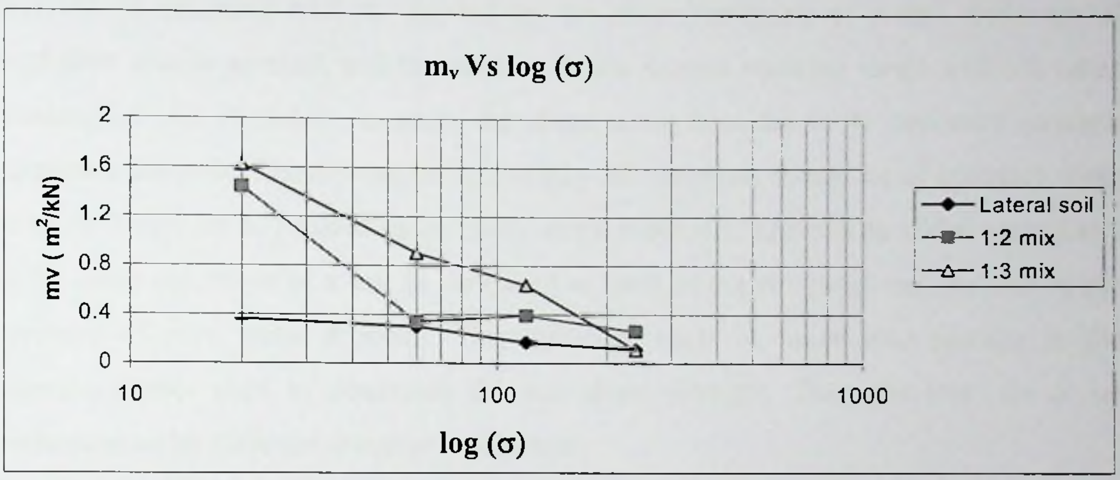


Figure 3.15 Effect on tyre chip mixing on mv for saturated lateritic soil

Coefficient of consolidation

Variation of Cv value against vertical stress for lateritic soil and different percentage of tyre chips mixture are shown in Figure 3.16.

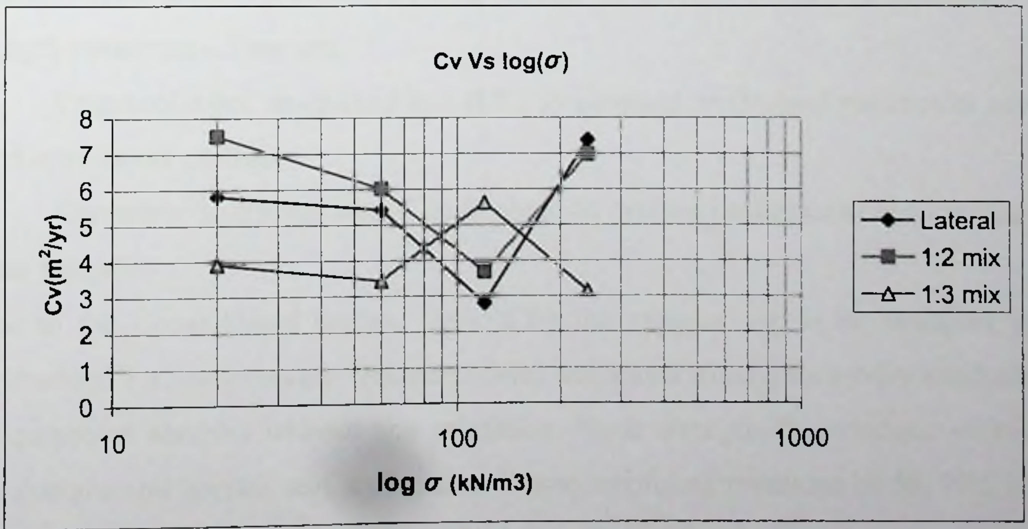


Figure 3.16 Effect on tyre chip mixing on Cv for saturated lateritic soil

3.2.3 Determination of Shear strength characteristic of fill material

The strength of a material may be broadly defined as the ability to resists imposed forces. It is often measured as the maximum stress the material can sustain under specified loading and boundary conditions.

In the soils, attention has been directed on measurement and use of shear strength. This is due to the fact that the; stability of a slope, bearing capacity of foundations, lateral earth pressure on retaining wall etc depend on the shear resistance of a soil. Soils are of negligible tensile strength and the stresses in the normal working range will not cause crushing of soil particles. As such, the shear strength is the most important strength parameter for soils. The developed lightweight fill materials should be of adequate shear strength if they are to be used successfully as fill materials. There is an added complexity in the shear behaviour of a soil as compared to most of the structural material due to the presence of pore water pressure. This influence must be taken into account in the laboratory tests used to determine the soil shear strength. Thus the tests are to be performed under different drainage conditions.

The shear strength parameters of the selected mix designs were determined through the triaxial test. Triaxial test is one of the most reliable methods now available for the determination of shear strength parameters of a soil.

Shear strength parameters should be found both under as compacted condition and under saturated conditions. Two types of tests should be done to obtain the necessary shear strength parameters. They are;

(1) Unconsolidated un-drained test **(UU)** to obtained un-drained parameters need for short-term stress analysis.

(2) Consolidated drained test **(CD)** to obtained drained parameters need for long term stress analysis.

Prior to the Consolidated drained tests **(CD)** the samples are to be saturated by the application of a backpressure. Unconsolidated undrained triaxial tests were conducted on as compacted samples without any saturation. Shear strength characteristic of selected mix designs and lateritic soil are measured using confining pressures of 50, 100, 150, & 200kN/m^2. The specimens were statically compacted to the required densities in a mould to the size of the standard Triaxial specimen of 38mm diameter & 85mm height.

3.2.3.1 Unconsolidated undrained triaxial tests

Figure 3.17, Figure 3.19 and Figure 3.21 show deviatric stress versus axial strain data obtained on latteritic soil and 1:3 and 1:2 tyrechips mixture. For lateritic soil, there is an increase in deviatoric stress as confining pressure increases. In addition, definite peaks are seen in the deviatoric stress-strain curves, which are typical of lateritic soil. The latteritic soil specimens failed along a distinct shear plane. For the tyre chips mixture, there was also an increase in deviatoric stress as confining pressure increases, but there was no distinct peak value in the curves. The failure stress was taken as the stress corresponding to 20% strain.

The Mohr-Coulomb failure envelopes obtained for lateritic soil and 1:3, 1:2 tyre chips mixtures are shown in Figure 3.18, Figure 3.20 and Figure 3.22. This type of graph is useful for determining the friction angle (Φu) and the cohesion intercept (C_u) values. The data are presented in Table 3.6.

Soil type	Cu (kN/m^2)	Φu
Lateritic soil	108	0^0
1:3 Tyre chips: Soil	50	13^0
1:2Tyre chips: Soil	22	19^0

Table 3.6 Undrained shear strength parameters

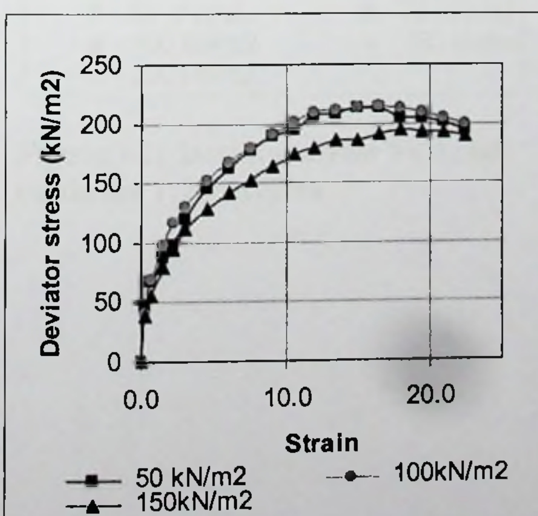


Figure 3.17 Deviator stress Vs Axial strain for Lateritic soil

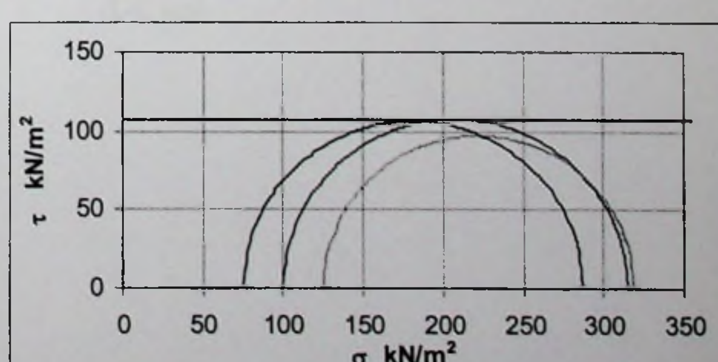


Figure 3.18Mohr circle Plot for Lateral soil-UU test

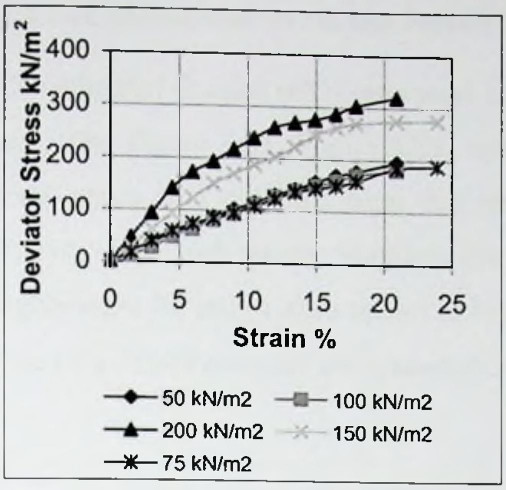


Figure 319Deviator stress Vs Axial strain for 1:3 mixtures

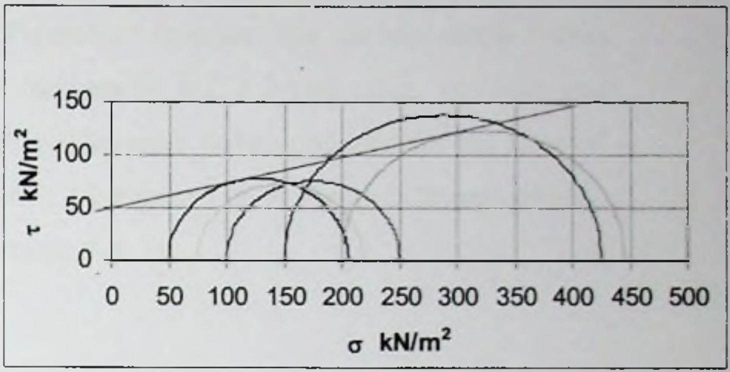


Figure 3.20Mohr circle Plot for 1:3 mixtures-UU test

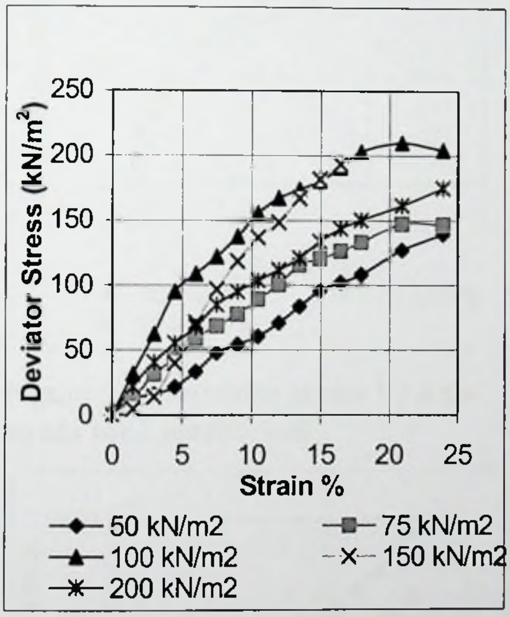


Figure 3.21 Deviator stress Vs Axial strain for 1:2 mixtures

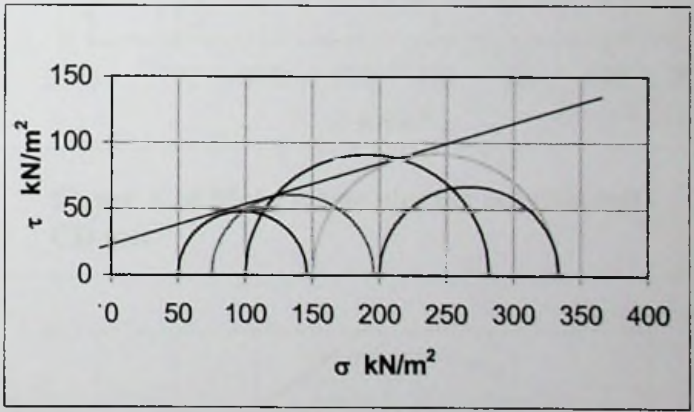


Figure 3.22 Mohr circle Plot for 1:2 mixtures --UU test

3.2.3.2Consolidated drain test results (CD)

Consolidated drained (CD) tests were carried out for selected lightweight fills & lateritic soil. The Figure 3.23, Figure 3.25, and Figure3.27 represent the deviator stress versus axial strain data obtained from that test for lateritic soil 1:3 tyre chips: soil mix and 1:2tyre chips: soil mixes. Similarly the Mohr-Coulomb failure obtained for the selected lightweight fill mixes were shown in Figure 3.24, Figure 3.26 & Figure 3.28 respectively. The C, ϕ values obtained are presented in Table 3.7.

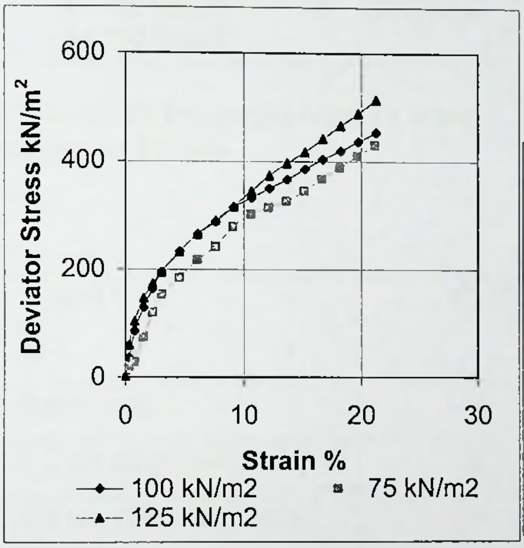


Figure 3.23 Deviator stress Vs Axial strain for Lateritic soil

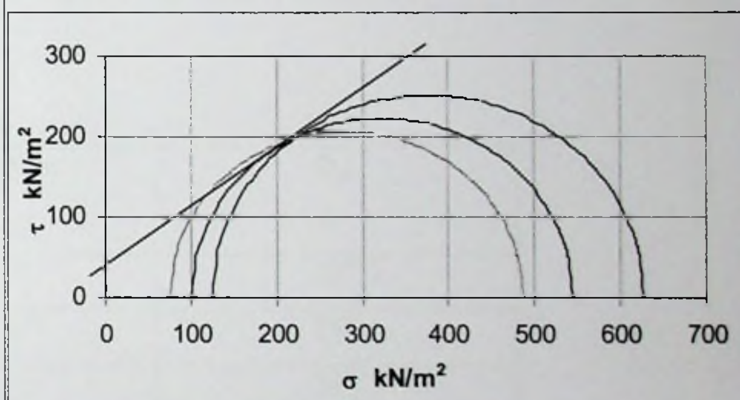


Figure 3.24 Mohr circle plot for lateritic soil - CD test

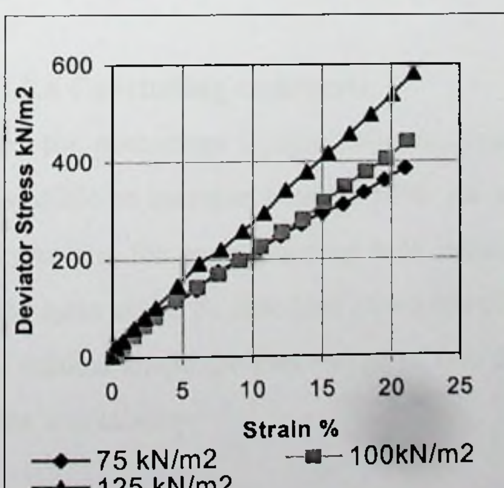


Figure 3.25 Deviator stress Vs Axial

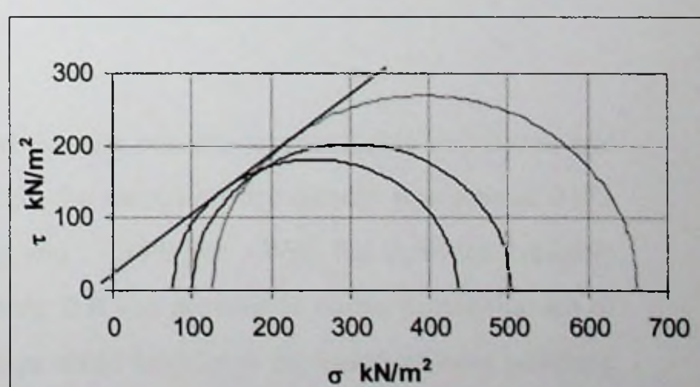


Figure 3.26 Mohr circle plot for 1:3 mix -CD test

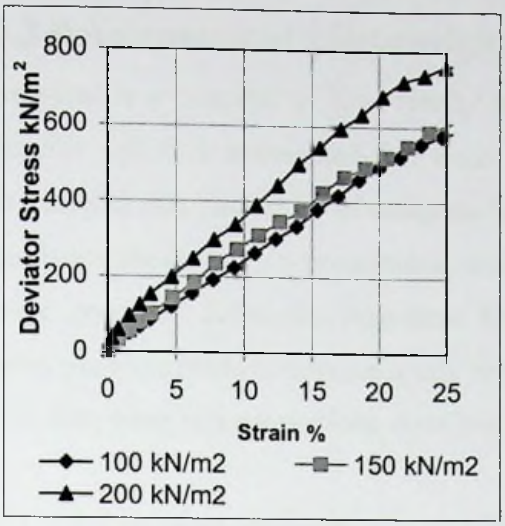


Figure 3.27 Deviator stress Vs Axial strain for 1.2 mix

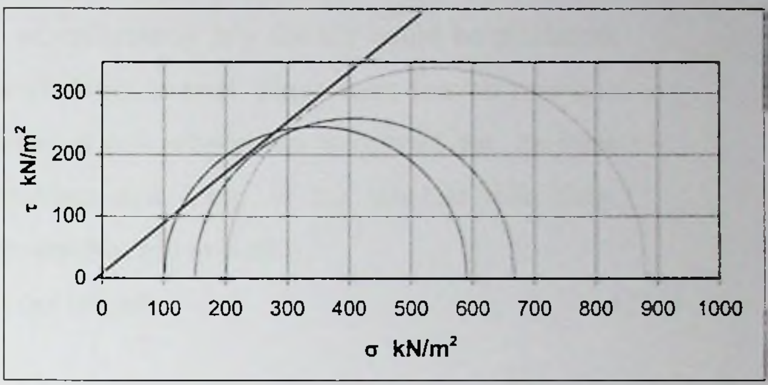


Figure 3.28 Mohr circle Plot for 1.2 mix -CD test

Soil type	C_d (kN/m^2)	Φ_d
Lateritic soil	40	33^0
1:3 Tyre chips: Soil mix	18	37^0
1:2 Tyre chips: Soil mix	10	38^0

Table 3.7 Drained shear strength parameters

3.2.4 Concluding comments

As the percentage of tyre chips increased the mix became less workable and it was not possible to increase beyond 50%. As such, the minimum dry density was around 1173 kg/m^{3}and the corresponding bulk density was 1340kg/m^3. With the facilities available the tyres could be shredded into a fibre form. If it was possible to obtain pieces that are of a cubical shape the percentage of tyre chips could have been increased without affecting the workability.

3.3 Development of a lightweight fill material with sawdust

Sawdust is a material of low density and by mixing a large portion of saw dust with lateritic soil it is anticipated that a mix of sufficiently low density could be produced. Developed mix should be of adequate workability as well. Since there was no previous guidance about the mix proportions, number of mix proportions were tried out. As there were concerns about the long-term behaviour and decay of the sawdust with time, attempts were made to develop a mix with sawdust ash as well.

The following mix proportions were tried out initially.

(1) 1:3 Sawdust: Soil mix

(2) 2:3 Sawdust: Soil mix

(3) 1:1 Sawdust: Soil mix

(4) 3:2Sawdust: Soil mix

(5) Sawdust

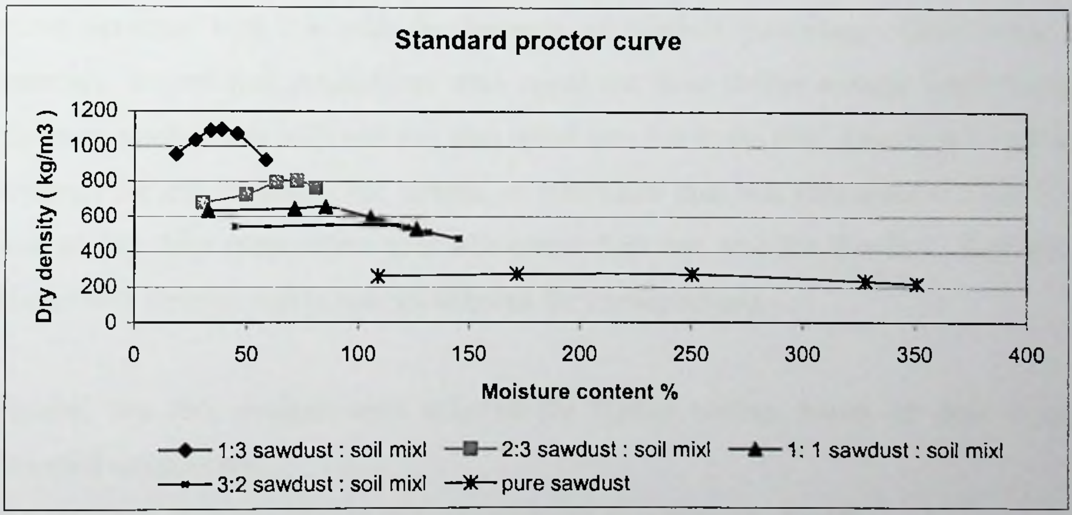


Figure 3.29 Proctor compaction test results

3.3.1 Testing of mixes and density analysis

All the above mix proportions were by weight of the respective materials. Density achievable with these mix proportions was determined through standard proctor compaction test and the bulk density, maximum dry density & optimum moisture content of the above mix designs obtained are presented in **Table 3.8**

Description of the mix	Optimum Moisture Content (%)	Maximum Dry Density (kg/m^3)	Corresponding Bulk Density (Kg/m^3)
1:3 Sawdust: Soil mix	36.55	1091.25	1490.10
2:3 Sawdust: Soil mix	69.70	806.75	1369.05
1:1 Sawdust: Soil mix	82.95	657.25	1202.43
3:2 Sawdust: Soil mix	103.55	553.10	1125.83
Pure saw dust mix	220.60	275.45	883.09
1:1 Sawdust ash : Soil mix	77.25	842.50	1493.33
Lateritic soil	17.55	1776	2087.68

Table 3.8 – Compaction properties of sawdust soil mixes and lateritic soil

It was generally observed that with the increase of the percentage of saw dust the achieved density decreased. The optimum moisture content increased and the Proctor curve becomes very flat with the increase of sawdust percentage. Considering their densities, several mix proportions were opted out from further testing. Lightweight fill mixes of sawdust ash with soil was also opted out, due to the high density and inability to compact the mix properly. The sample, of 100% saw dust was also opted out due to poor workability. Mix proportions of 1:3 Sawdust: Soil mix and 2:3 Sawdust: Soil mix had higher bulk density values and not selected for further testing.

Finally, two mix designs were selected for further testing, based on their densities. Selected samples are;

(1). 60% saw dust mixed with 40% lateritic soil by weight (**3: 2 mix**)

(2). 50% saw dust mixed with 50% lateritic soil by weight (**1: 1 mix**)

The standard proctor curves for the two mixes are presented in Figure 3.29 and Figure 3.30.

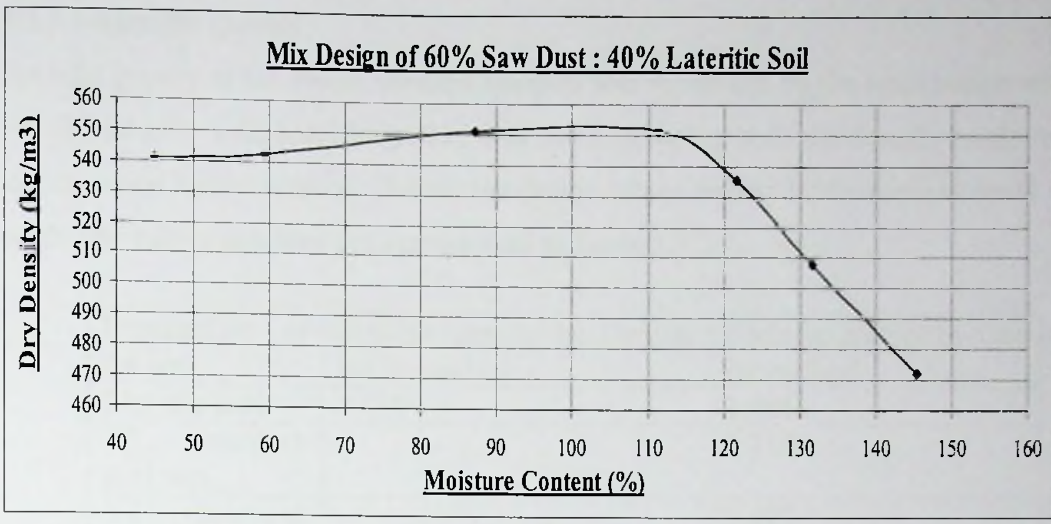


Figure 3. 30 – Standard Proctor Curves for 3:2 Sawdust: Soil mix

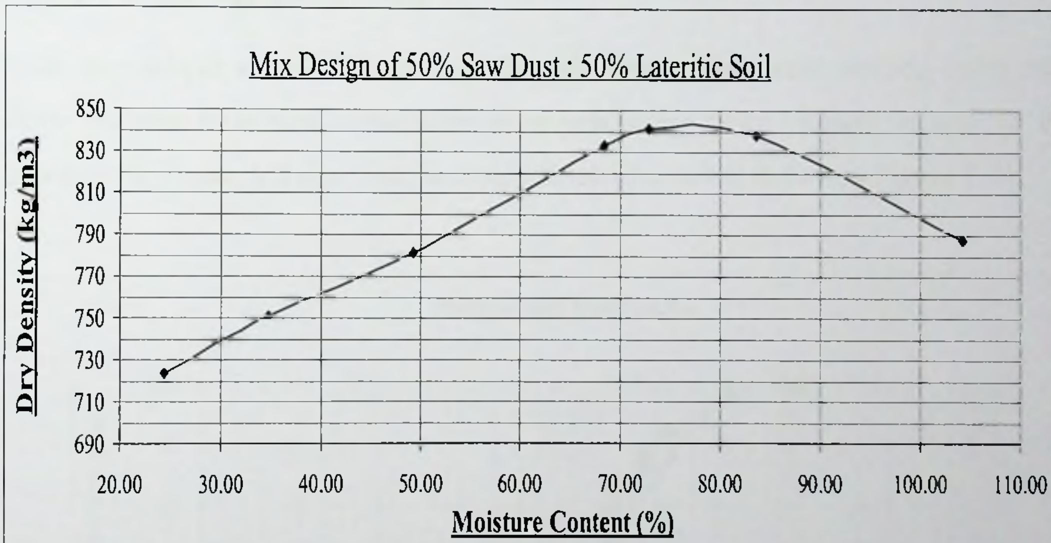


Figure 3.31 – Standard Proctor Curves for 1:1 Sawdust: Soil mix

3.3.1.1 Specific gravity

Specific gravity of the above selected samples was found out by the open beaker method. For the lateritic soil, specific gravity was obtained using both the density bottle method and the open beaker method. But for the design mixes density bottle method could not be used. The values obtained are summarized in Table 3.9

Description of the soil	Specific gravity by Density bottle method	Specific gravity by Gas jar method
Lateritic soil	2.650	2.695
1:1 Sawdust: Soil mix	--	1.213
3:2 Sawdust: Soil mix	--	1.027

Table 3.9 – Specific gravity of sawdust soil mixes and lateritic soil

3.2.1.2 Particle size distribution

Also the particle size distribution of the mix proportions were studied using standard sieve analysis. Graphical representation of percentage finer Vs particle size for the 1:1 Sawdust: Soil mix, 3:2 Sawdust: Soil mix & lateritic soil is shown in Figure 3.31.

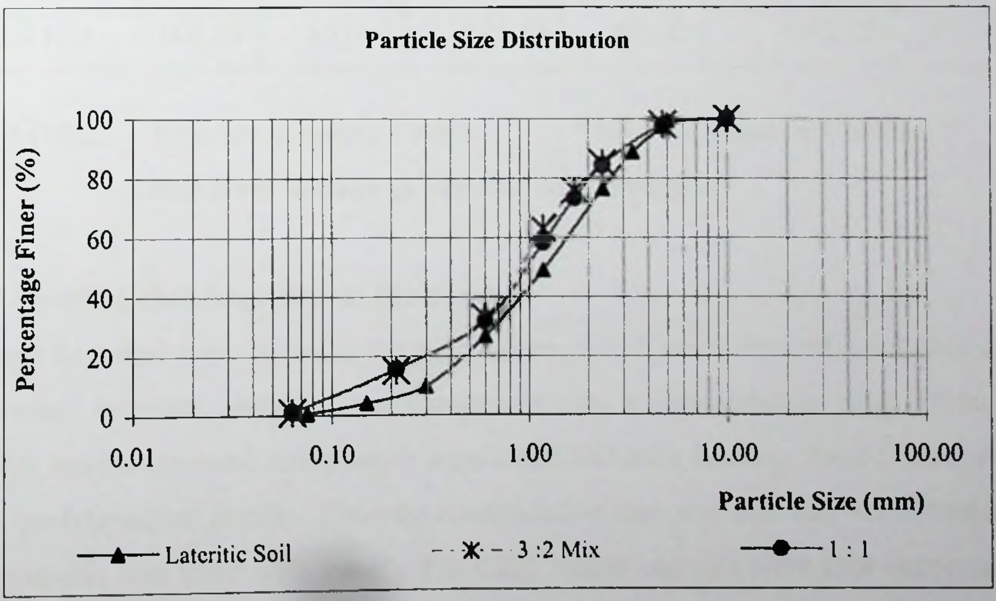


Figure 3.32 – Particle Size Distribution

As it was impossible to determine the Liquid Limit or Plastic Limit of the developed mixes it was not attempted. As the sawdust percentage increased the water absorption of the mix increased and optimum moisture content increased, causing an increase of the bulk density. This in turn would increase the weight applied on the underlying soft soil. As such, it was decided to use moisture content much lower than the optimum in the development of lightweight fill. Since the proctor curve was quite flat in this region, reduction of moisture content will not cause much of a change in the dry density achieved, but the bulk density would be much lower.

The selected water contents and the obtained densities are presented in Table 3.10. However, the fill material compacted at the lower moisture content could absorb more water when submerged and could cause an increase in density or swell. A series of tests were conducted to understand this behaviour.

Description of the Mix	OMC (%)	Max'm Dry Density (kg/m^3)	Max'm Bulk Density (kg/m^3)	Selected MC (%)	Dry Density Relevant to selected MC (kg/m^3)	Air voids %	Corresponding Bulk Density (kg/m^3)
1:1 mix	82.95	657.25	1202.43	60.00	640.50	6	1024.8
3:2 mix	103.55	553.10	1125.83	60.00	542.15	13	867.44

* **OMC** – Optimum Moisture Content   * **MC** – Moisture Content

Table 3.10– Details of Selected Mix Designs

3.3.2 Swelling characteristics of the mixes

Soil and Saw dust were mixed at the proportions, which were selected for further testing at selected moisture contents, and compacted into a consolidation ring. Three other samples were compacted into Proctor mould & California Bearing Ratio (CBR) moulds at the predetermined density. Then the consolidation ring was attached to oedometer and the apparatus was filled with water. The CBR mould samples were kept immersed in a water bath. Swell, the increase of the height of the sample was measured periodically and swell data obtained are presented in Table 3.11 and Table 3.12

Description of the test sample	Initial MC (%)	Initial Sample Height (mm)	Percentage Swell% After 24 hrs (%)	After 48 hrs (%)	After 72 hrs (%)	After 96 hrs (%)	After 120 hrs (%)	After 144 hrs (%)
CBR Mould	56.86	127.00	11.02	17.24	19.49	19.95	19.95	19.95
Consolidation ring	60.00	14.67	10.77	14.91	17.21	18.57	18.64	18.64
CBR Mould	77.25	127.00	08.43	14.96	18.76	19.17	19.29	19.29
Proctor Mould	82.95	115.00	08.32	10.72	17.05	19.09	19.09	19.09

Table 3.11– Swell Characteristics of 1:1 Sawdust: Soil mix

Description of the test sample	Initial MC (%)	Initial Sample Height (mm)	Percentage Swell% After 24 hrs (%)	After 48 hrs (%)	After 72 hrs (%)	After 96 hrs (%)	After 120 hrs (%)	after 144 hrs (%)
CBR Mould	60.00	16.67	10.02	13.86	18.15	20.62	20.92	20.92
Consolidation ring	68.78	127.00	12.01	17.22	19.88	21.52	21.52	21.52
CBR Mould	90.25	127.00	08.43	14.96	18.70	20.87	21.26	21.26
Proctor Mould	103.55	115.00	08.32	11.64	19.32	21.59	21.59	21.59

Table 3.12 – Swell Characteristics of 3:2 Sawdust: Soil mix

Above results shows that the initial compacted moisture content of the sample does not have a significant effect on the swelling values of the mix. Therefore to achieve a good workability of samples it was decided to mix with 60% moisture content and to carry out another test to find out swell characteristic of the sawdust mixes and lateritic soil. Data obtained from the tests are tabulated in Table 3.13 and Figure 3.33, and Figure 3.34 shows respective swell characteristics of the sawdust mixes and lateritic soils.

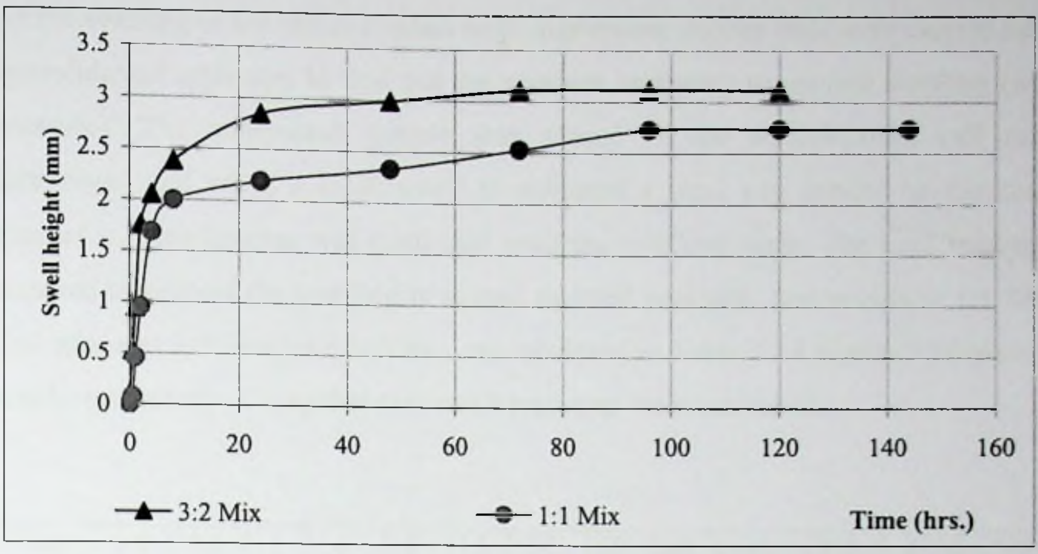


Figure. 3.33 – Swell Characteristics of sawdust mixes

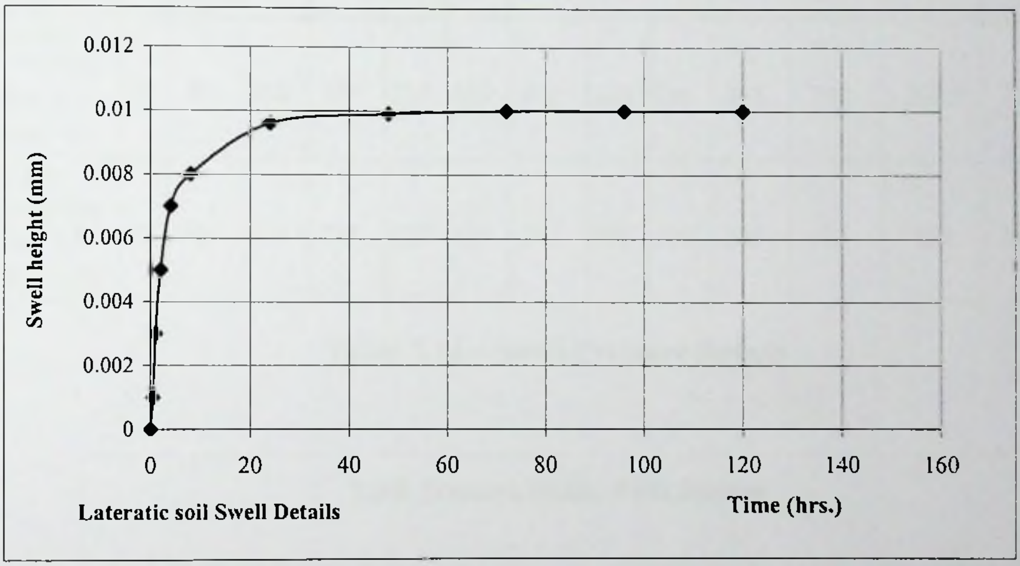


Figure. 3.34 – Swell Characteristics of Lateritic Soil

Time (hrs)	0	.5	1	2	4	8	24	48	72	96	120	144
Swell of 3 :2 mix (mm)	0	0.095	0.525	0.950	1.755	2.05	2.378	2.845	2.989	3.105	3.115	3.115
Swell of 1 : 1 mix (mm)	0	0.075	0.455	0.951	1.685	1.997	2.188	2.329	2.526	2.724	2.734	2.734
Swell of Lateritic Soil (mm)	0	0.000	0.001	0.003	0.005	0.007	0.008	0.010	0.010	0.010	0.010	0.010

Table 3.13 – Swell Details of selected sawdust mixes and Lateritic soil

As the swelling of the mixes appears to be significant, further tests were carried out in the consolidation apparatus to find out the pressure necessary to prevent swelling (ie; swell pressure). The compacted sample was placed in the consolidation cell and was submerged and when a small swell is indicated a load was applied to the hanger to prevent it. This process was continued until the swelling stops. The total load that was required to prevent the swelling is termed as swell pressure. Test results of 1:1 Sawdust: Soil mix and 3:2 Sawdust: Soil mix are tabulated in Table 3.14;Figure 3.34 presents the results graphically. Using this data swell pressures were estimated.

Time (min) (hrs.)	0	0.5	1	4	8	15	30	60 (1)	240 (4)	1440 (24)	2880 (48)	4320 (72)	5760 (96)	7200 (120)	8640 (144)
Load Applied for 1:1 mix (g)	80	90	162	208	217	226	233	240	248	261	262	263.5	264	264	264
Load Applied for 3:2 mix (g)	80	105	175	215	227	237	243	255	265	303	313	318	320	324	325

Table 3.14 – Swell Pressure Details

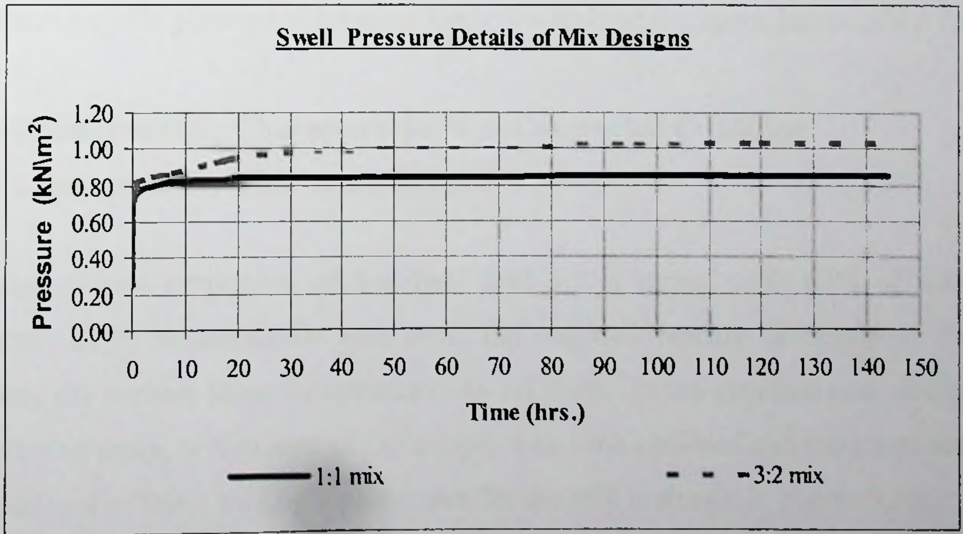


Figure 3.35 – Graphical Representation of Swell Pressure With Time

Test results for 1:1 mix as follows

Final applied load to stop swelling	=	265.00	g
Force on the sample	=	0.00259965	kN
Area of the mould	=	0.00306796	m^2

$$\text{Pressure required to stop the swelling} = \frac{\text{Force on the sample}}{\text{Area of the mould}}$$

$$= 0.85 \quad kN/m^2$$

Test results for 3:2 mix as follows

Final applied load to stop swelling	=	325.00	g
Force on the sample	=	0.00318825	kN
Area of the mould	=	0.00306796	m^2

$$\text{Pressure required to stop the swelling} = \frac{\text{Force on the sample}}{\text{Area of the mould}}$$

$$= 1.04 \quad kN/m^2$$

According to above test results, pressure required to prevent the swelling is very small. This is smaller than the stress due a fill of few inches thick. Therefore we can safely assume that the swelling occurs in light weight fill materials below the water table would be prevented by the pressure induced by the self weight of the upper layers of the fill.

3.3.3 Compressibility Characteristics in as Compacted Condition

3.3.3.1 Compressibility Characteristics of 3: 2 mix

The selected mix proportion of Sawdust: Soil 3:2 is mixed with 60% of water and compacted in to consolidation ring until the required density is obtained. Then by increasing the vertical stress incrementally to 160kN/m^2 in the standard one- dimensional consolidation setup, settlements of the sample with time obtained and the properties were computed in the. The e Vs Log σ plot drawn for the mix is shown in Figure 3.36.

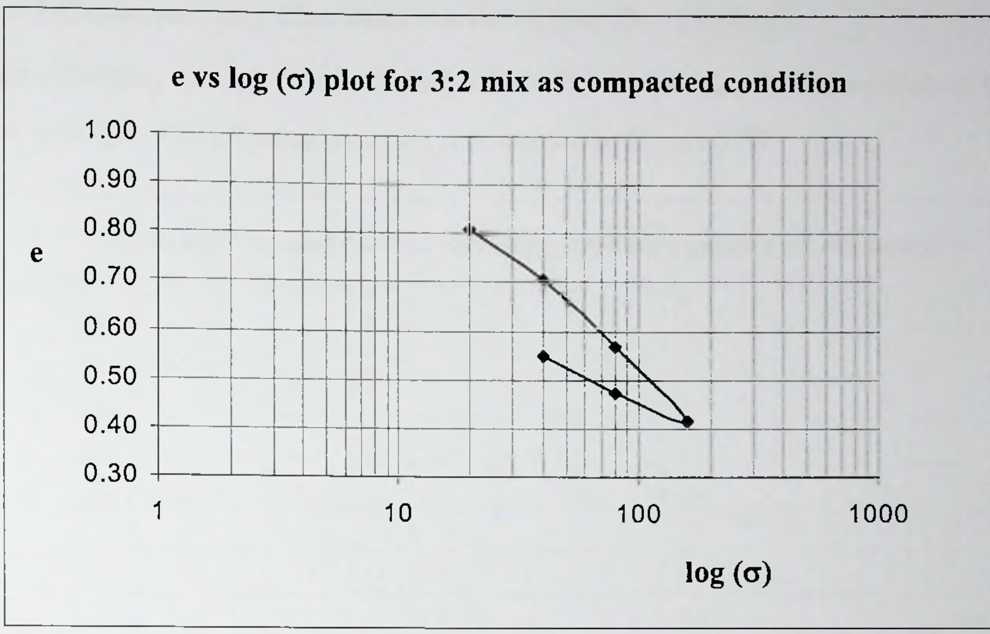


Figure 3.36 e vs. log (σ) plot for 3:2 Sawdust: Soil mix at as compacted state

3.3.3.2 Compressibility Characteristics of 1:1 Sawdust: Soil mix

The 1: 1 mix proportion is mixed with 60% of water and compacted in to consolidation ring at the required density and thereafter the one dimensional consolidation test was carried out. Data obtained is presented in the e Vs Log σ plot in Figure 3.37.

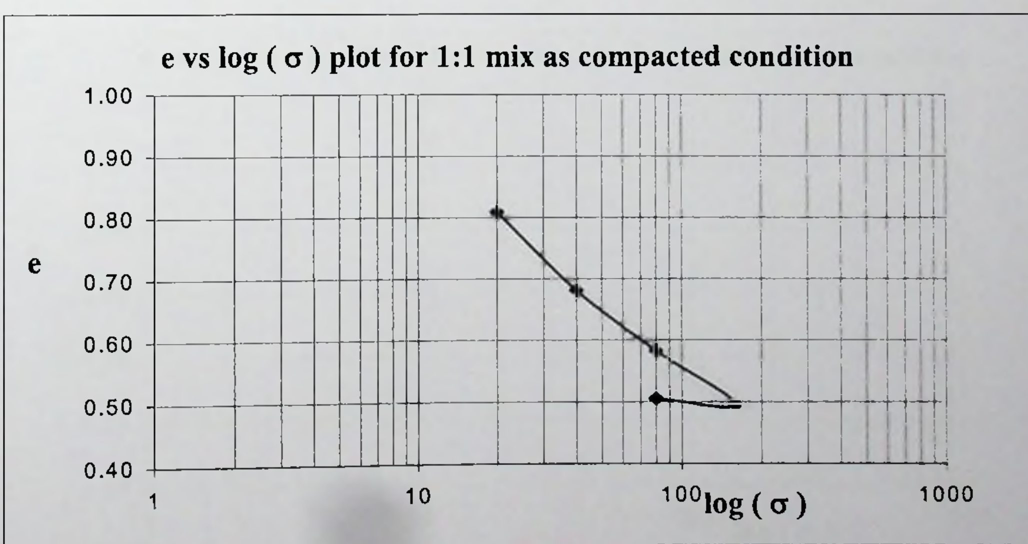


Figure 3.37 e vs. log (σ) plot for 1:1 Sawdust: Soil mix at as compacted state

3.3.4 Compressibility Characteristics under Saturated Condition

3.3.4.1 Compressibility Characteristics of 3:2Sawdust: Soil mix

After saturating the 3:2 Sawdust: Soil mix in the Oedometer the consolidation test was done in stages. The e Vs log σ curves is presented in Figure 3.38

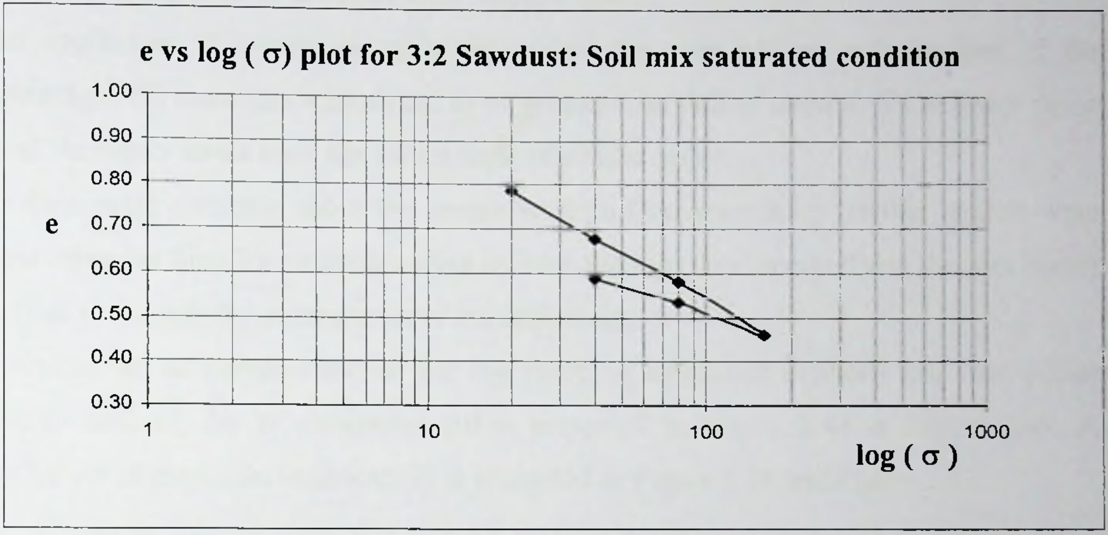


Figure 3.38 e vs. log (σ) plot for 3:2 Sawdust: Soil mix at saturated state

3.3.4.2 Compressibility Characteristics of 1:1 Sawdust: Soil mix

After saturating the 50%: 50% mix sample in the Oedometer, the consolidation test was carried out as earlier the e Vs log σ curve is represented by Figure 3.39.

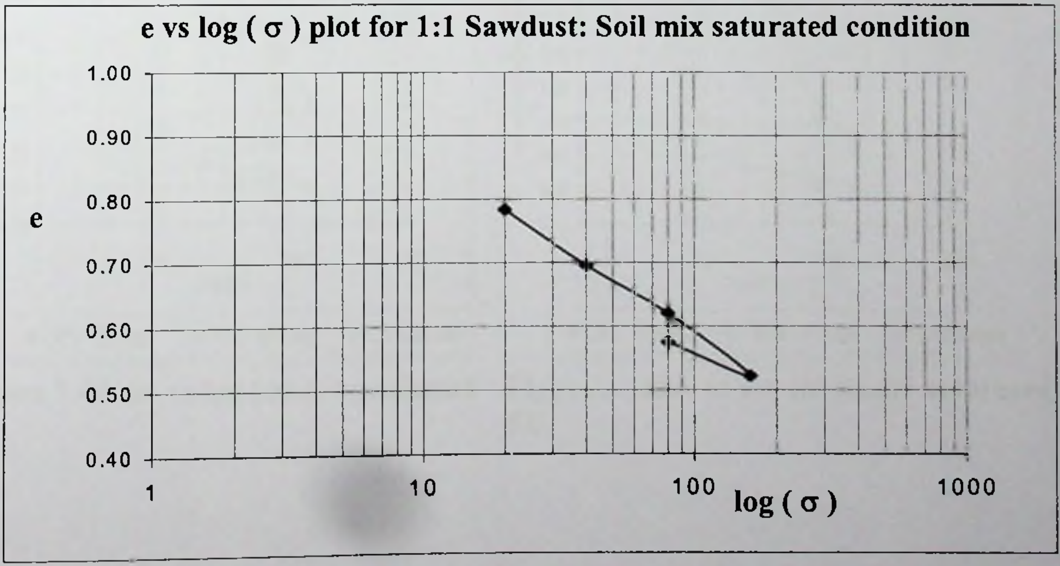


Figure 3.39 e vs. log (σ) plot for 1:1 Sawdust: Soil mix at as saturated state

3.3.4.3 Comments on the Compressibility Characteristics

The variations of coefficient of volume compressibility values are presented in Figure 3.40 & Figure 3.41. Similarly the variation of coefficient of consolidation with the stress levels of fill materials under as compacted condition and under saturated conditions are presented in Figure 3.42 & Figure 3.43 respectively.

The coefficient of volume compressibility and the compression index values of the lightweight fill materials were found to be greater than that of lateritic fill at lower stress level. At higher stress level the values were of similar order.

As there were concerns about the somewhat high Compressibility, further studies were done using the time Vs settlement plots by both the log (time) method and the root (time) method to identify the components of the settlement.

A typical set of curves obtained for log (time) (Casagrande method) and root (time) (Taylor method) for as compacted fill is presented in Figure 3.44 & Figure 3.45. A similar set of curves for saturated fill is presented in Figure 3.46 and Figure 3.47.

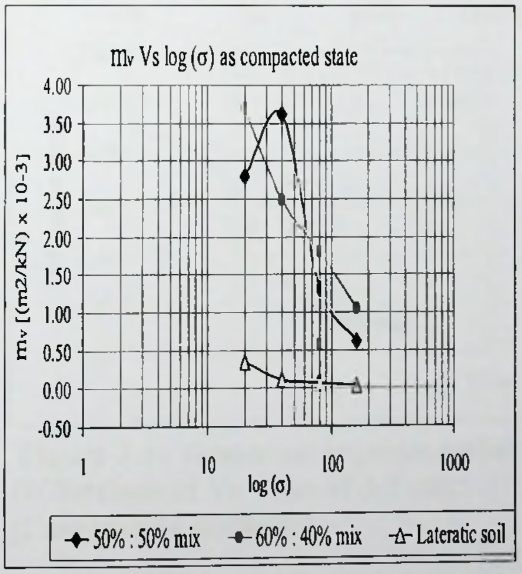


Figure 3.40 mv vs log (σ) as compacted fill

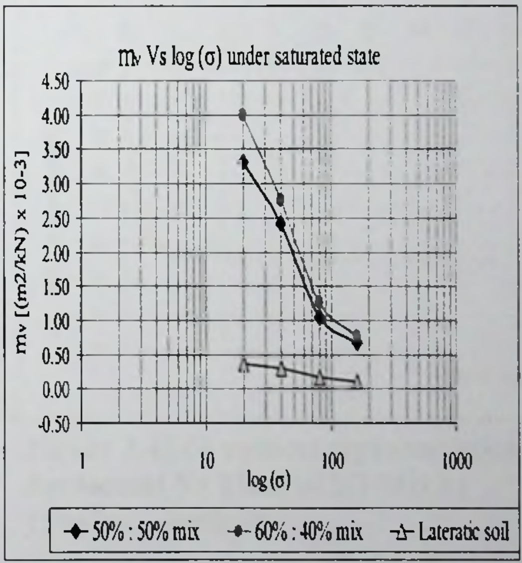


Figure 3.41 mv vs log (σ) under saturated fill

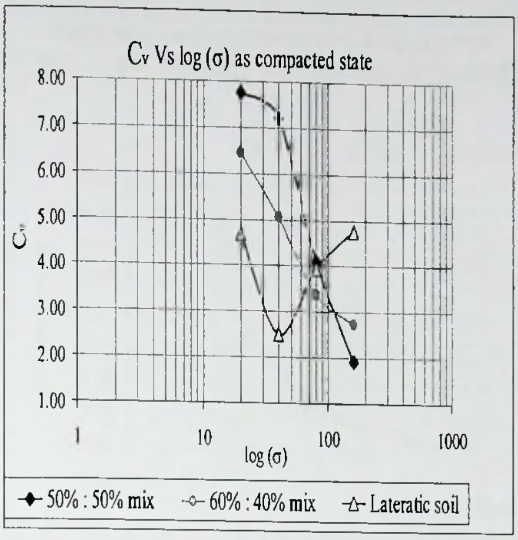


Figure 3.42Cv vs log (σ) as compacted fill

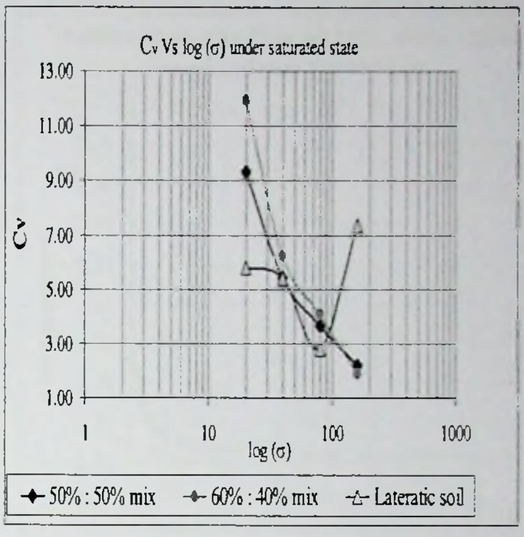


Figure 3.43 Cv vs log (σ) under saturated fill

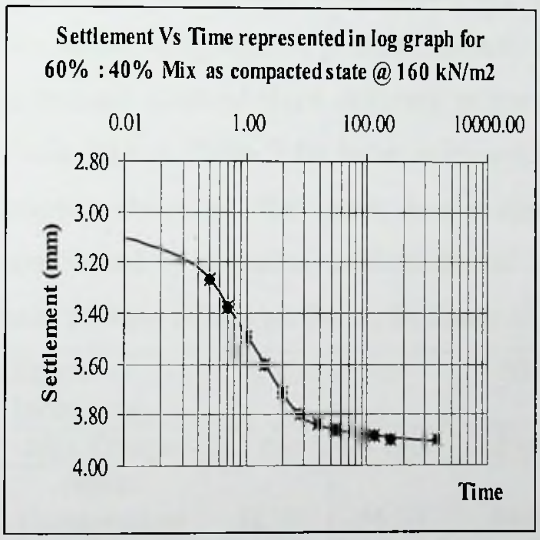


Figure 3.44 Graphical representation Of Settlement Vs Time of 3:2 mix (Casagrande methods)

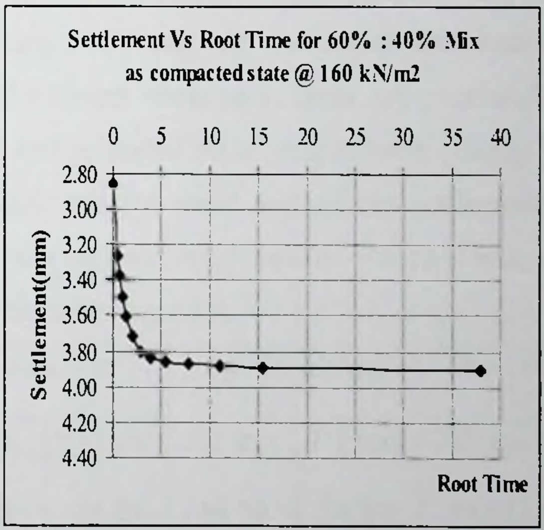


Figure 3.45 Graphical representation of Settlement Vs Time of 3:2 Mix as (Taylor methods.)

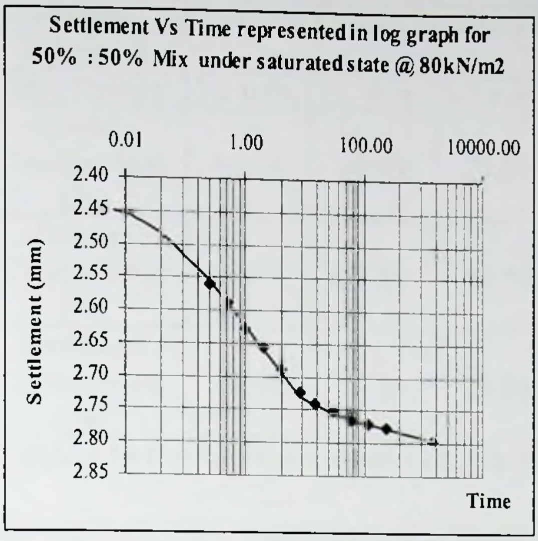


Figure 3.46 Graphical representation Of Settlement Vs Time of 1: 1 Mix Under Saturated state (Casagrande methods)

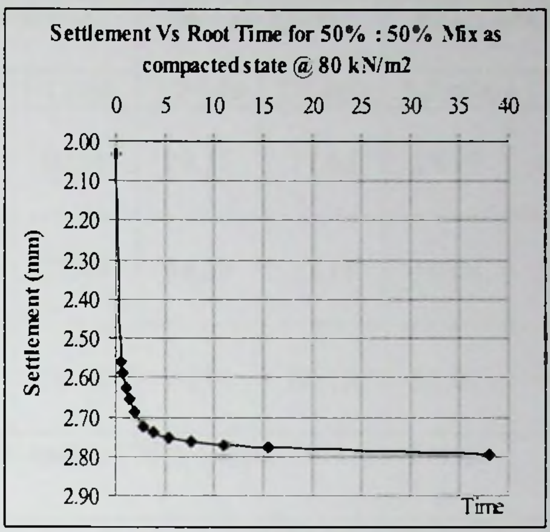


Figure 3.47 Graphical representation Of Settlement Vs Time of 1:1 Mix under saturated state (Taylor method)

The graphs for the other load increments were also of a similar shape. The percentages of the initial compression, primary consolidation settlement and secondary consolidation settlement obtained from different mixes at different stress increments are presented in Table 3.15 & Table 3.16 for as compacted and saturated mixes respectively (Using log graphs). Based on the above data it appears that the main part of the settlement is contributed by the initial settlement and it will be completed rapidly. Hence it will not have adverse effects on the performance of the lightweight fill.

Stress Increment	0 - 20 kN/m^2		20 - 40 kN/m^2		40 - 80 kN/m^2		80 - 160 kN/m^2	
Mix Design	3:2 mix	1:1 mix	3:2 mix	1:1 mix	3:2 mix	1:1 mix	3:2 mix	1:1 mix
Initial Compression (%)	42.91	55.01	58.00	56.28	45.39	58.59	39.65	66.77
Primary Compression (%)	36.96	39.48	25.10	36.36	38.49	38.44	56.40	33.12
Secondary Compression (%)	20.13	05.51	16.90	07.36	16.12	02.97	03.95	00.11

Table 3.15 Compression details of 3:2 mix and 1:1 mix as compacted State

Stress Increment	0 - 20 kN/m^2		20 - 40 kN/m^2		40 - 80 kN/m^2		80 - 160 kN/m^2	
Mix Design	3:2 mix	1:1 mix	3:2 mix	1:1 mix	3:2 mix	1:1 mix	3:2 mix	1:1 mix
Initial Compression (%)	40.58	50.99	27.09	60.91	29.31	71.65	28.35	61.69
Primary Compression (%)	26.74	31.89	49.90	22.44	44.39	22.16	50.04	29.20
Secondary Compression (%)	32.68	17.12	23.01	16.65	26.30	06.19	21.61	09.11

Table 3.16 Compression details of 3: 2 Mix and 1:1 Mix Under Saturated state

Fill	As compacted state			Saturated state		
	Compression index (Cc)	Swell index (Cs)	$\frac{Cc}{(1 + e_0)}$	Compression index (Cc)	Swell index (Cs)	$\frac{Cc}{(1 + e_0)}$
Lateritic soil	0.029	0.0047	0.0178	0.084	0.0095	0.0526
1:1 mix	0.301	0.0381	0.1579	0.290	0.1768	0.1522
3:2 mix	0.497	0.2527	0.2576	0.347	0.2256	0.1811

Table 3.17 Compressibility characteristics of sawdust soil mixes

Table 3.17 shows the details compression index **(Cc)** and swell index **(Cs)** of the selected mix designs and lateritic fill. Compression index values shows that the compressibility of lightweight fill materials is greater than that of lateritic fill. But the compression index values are still quite low and developed lightweight fill is seen to be of adequate stiffness.

3.3.5Determination of Shear strength characteristic of fill material

3.3.5.1Unconsolidated undrained test results

Shear Strength characteristics of Sawdust: Soil mixes

The Figure 3.48 and Figure 3.49 represent the deviator stress versus axial strain data obtained for unconsolidated un-drained test results of 3:2 Sawdust: Soil mix and 1:1 Sawdust: soil mix respectively. The Mohr-Coulomb failure obtained for the design lightweight fill mix of 3:2 Sawdust: Soil mix and 1:1 Sawdust: Soil mixes at unconsolidated un-drained state are shown in Figure 3.50 and Figure 3.51.

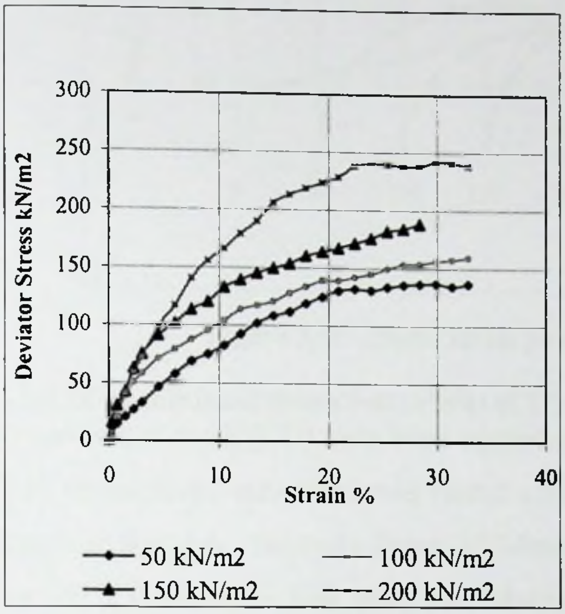


Figure 3.48- Deviator stresses Vs Strain for 3:2 mix

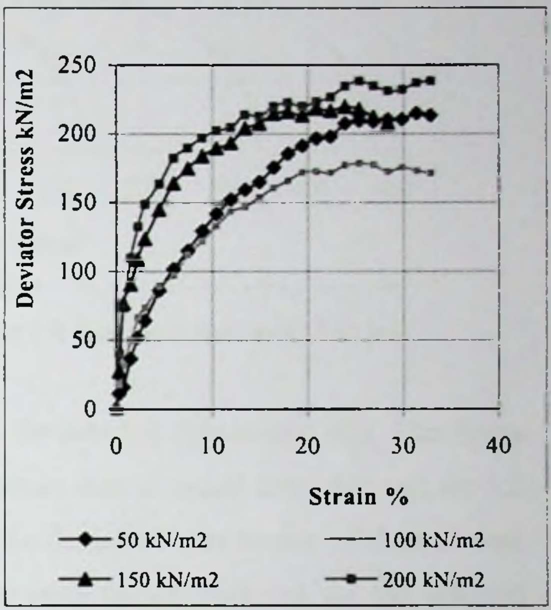


Figure 3.49 - Deviator stresses Vs Strain for 1:1 mix

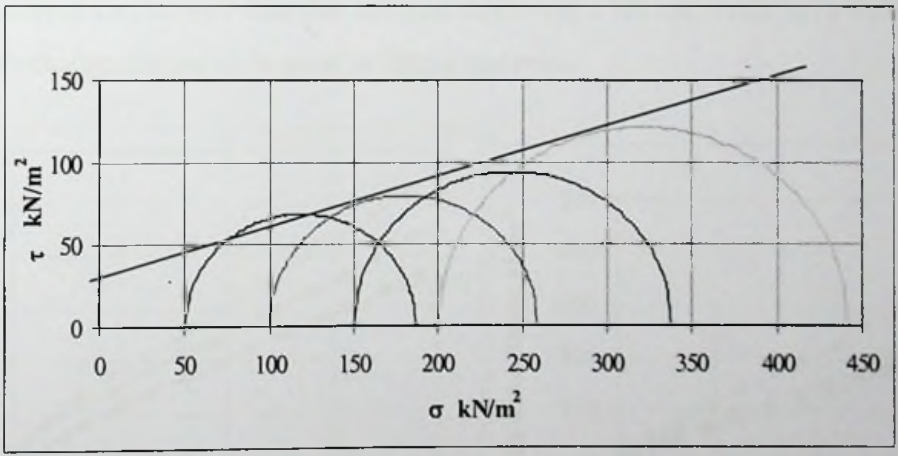


Figure 3.50 – Mohr circle plot for 3:2 Sawdust: Soil mix -UU test

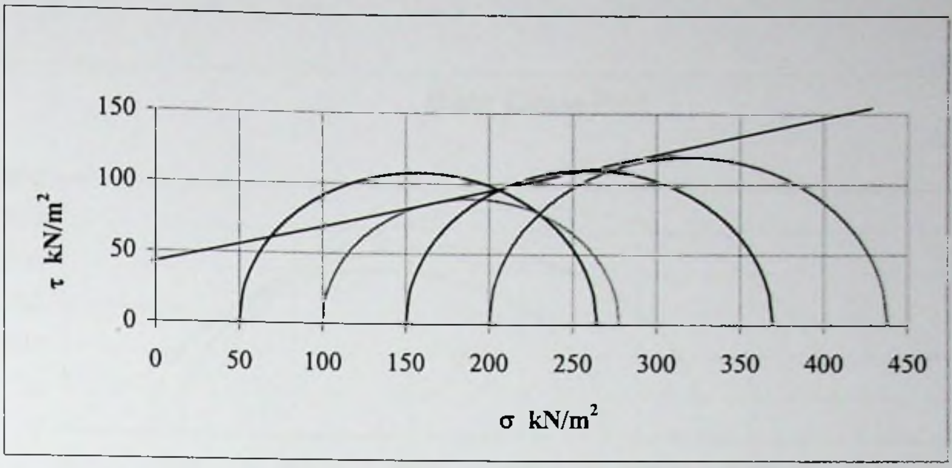


Figure 3.51 – Mohr circle plot for 1:1 Sawdust: Soil mix –UU test

3.3.5.2Consolidated drain test results (CD)

Consolidated drain (CD) tests were carried out for selected lightweight fills. The figure 3.52 represent the deviator stress versus axial strain data obtained from that test for 3:2 Sawdust: Soil mix. Similarly figure 3.53shows the deviator stress versus axial strain data for the 1:1 Sawdust: Soil mix. The Mohr-Coulomb failure obtained for the selected lightweight fill mix designs were shown in Figure3.54 & Figure3.55.The obtained shear strength parameters are summarised in Table 3.18. Based on the obtained shear strength parameters it can be said that the selected lightweight fill materials have sufficient shear strength characteristics to be used as filling material.

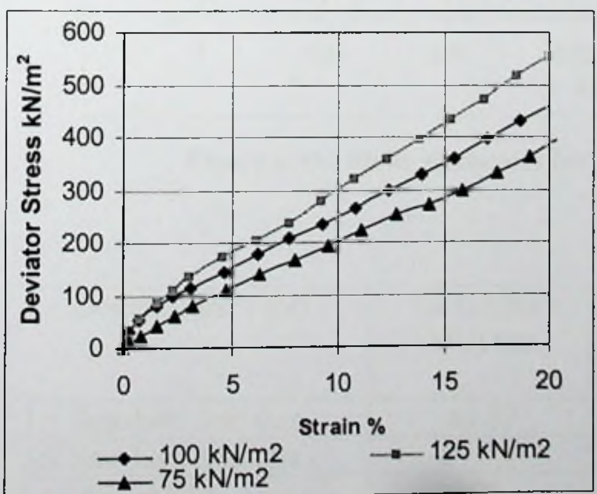


Figure 3.52 Deviator stresses Vs Strain for 3:2 Sawdust: Soil mix

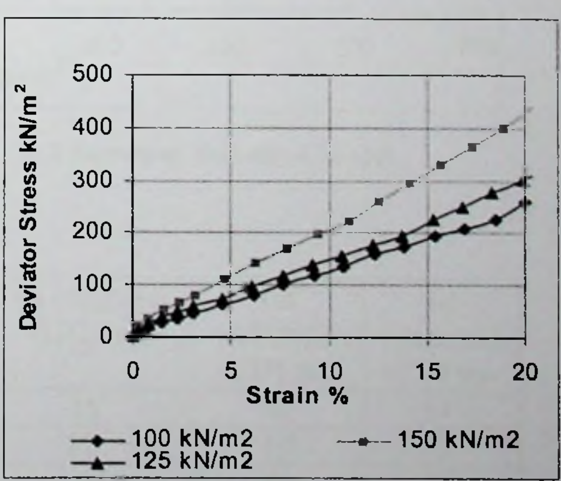


Figure 3.53 Deviator stresses Vs Strain for 1:1 Sawdust : Soil mix

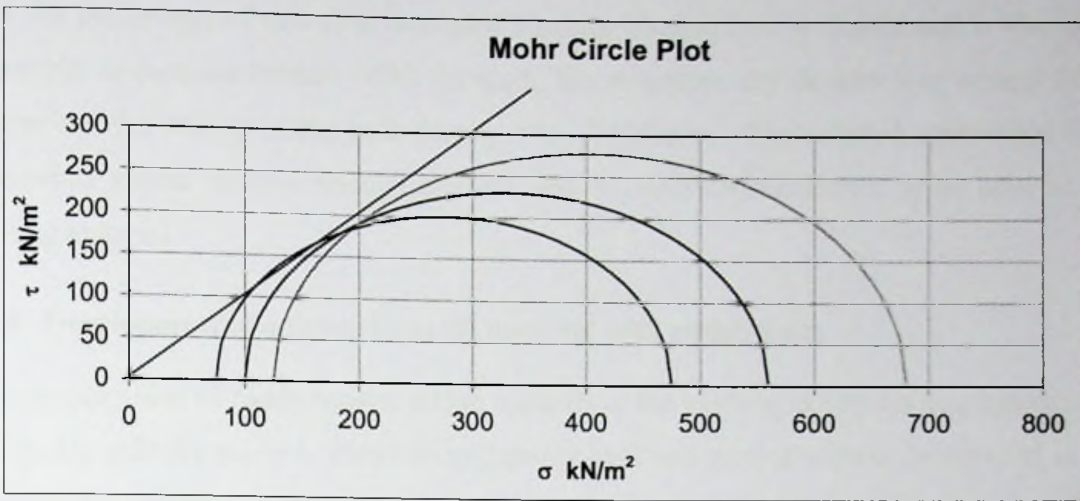


Figure 3.54 Mohr circle plot for 3: 2 Sawdust: Soil mix –CD test

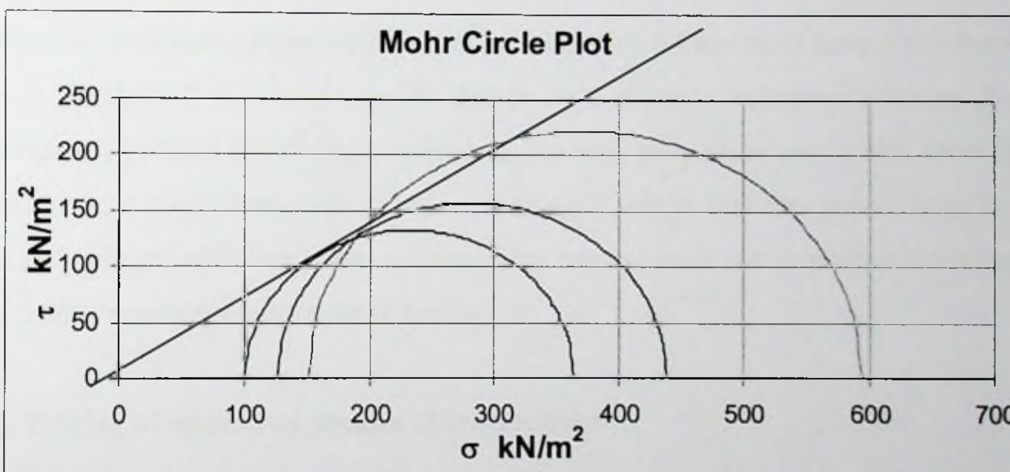


Figure 3.55– Mohr circle plot for 1: 1 Sawdust: Soil mix-CD test

Description of soil	Cu (kN/m²) (UU) test	ϕu (degrees) (UU) test	Cd (kN/m²) (CD) test	ϕd (degrees) (CD) test
1:1 Sawdust: Soil mix	46.80	15^0	0	34^0
3:2 Sawdust: Soil mix	27.77	17^0	10	37^0
Lateritic soil	106	0^0	40	33^0

Table 3.18 Shear strength parameters of sawdust: soil mixes

3.3.6 Concluding comments

As the percentage of saw dust increased the mix became less workable and it was not possible to increase beyond 60%. As such, the minimum dry density was around 553 kg/m^{3}and the corresponding bulk density was 1125kg/m^3. The selected lightweight fill materials appear to have enough strength and stiffness characteristics to be used as a filling material.

3.4 Development of a lightweight fill material with paddy husk

Large quantities of Paddy husk is added annually to the existing stockpiles as a byproduct of paddy refining process. These wasted paddy husk are burned without being used in a productive manner. These wasted paddy husk can be used for other applications such as to make a light weight fill material to be used in embankments over weak or compressible soils.

Paddy husk is of density around 205kg/m^3 and lateritic fill materials have a dry density of around 1760kg/m^3. It is necessary to decide on a suitable mix proportion so that the developed fill material will have desired engineering properties and is also economical. In this project paddy husk was mixed with lateritic soil in different proportions to get a workable mix of sufficiently low density. After several trials mixes proportion some mix proportions were selected for detail testing.

3.4.1 Testing of mixes and density characteristics

The standard proctor test was done to determine the maximum dry density and the optimum moisture content of the different mix proportions of paddy husk and soil. The optimum moisture content and maximum dry density thus obtained confirms to the standard proctor compaction effort. The dry unit weight obtained for each trial is plotted against the moisture content. The results are shown in Figure 3.56.The result of the series of standard proctor compaction tests on lateritic soil mixes with different percentages of paddy husk are presented in Table 3.19.

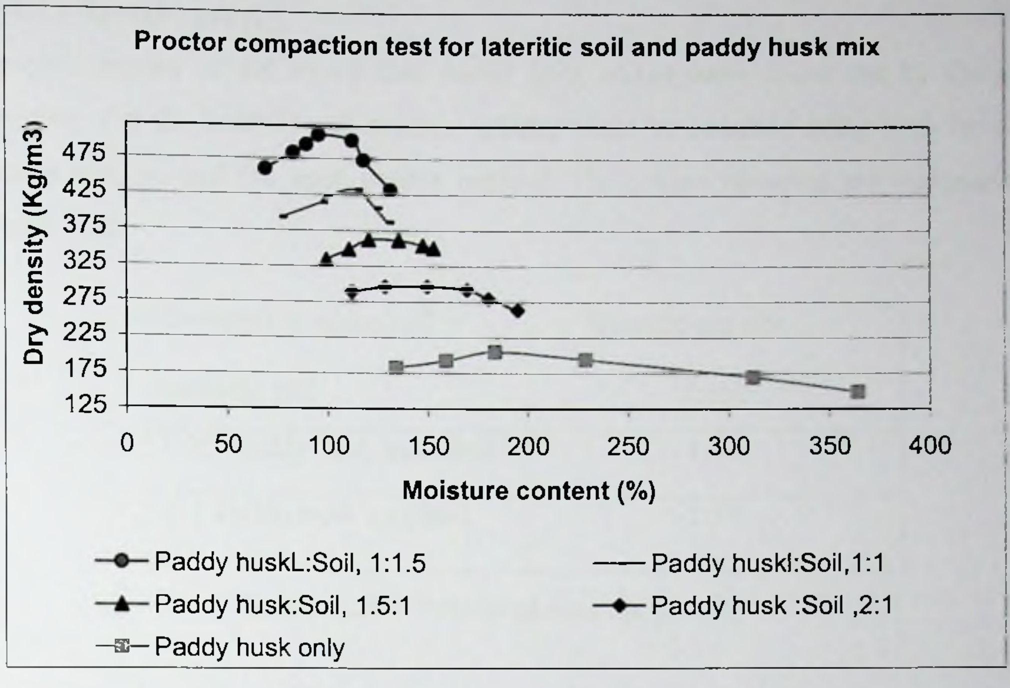


Figure 3.56 Proctor compaction test results of sawdust mixes

Description of the mix	Optimum Moisture Content (%)	Maximum Dry Density (kg/m^3)	Corresponding Bulk Density (kg/m^3)
100% lateritic soil	14.56	1763.30	2020.04
1:1.5 (Paddy husk: Soil)	98.00	508.00	1005.84
1:1 (Paddy husk: Soil)	112.94	432.50	916.90
1.5:1 (Paddy husk: Soil)	125.00	362.26	815.08
2:1(Paddy husk: Soil)	140.83	296.43	713.89
100% paddy husk	183.61	204.70	580.50

Table 3.19 Proctor compaction test results on lateritic soils with paddy husk

It is clear from the data of table that the optimum moisture content increases as the percentage of paddy husk increases. This data also reveals that the dry density of the mix decreases as the percentage of paddy husk increases. This behaviour is attributed to the fact that paddy husk is a material of much lightweight as compared to the soil grains. Thus it is clear that paddy husk has a good potential to be used in the production of a lightweight fill.

3.4.1.1 Specific gravity

Specific gravity of the above soil, paddy husk mixes were found out by the gas jar method. For the lateritic soil, specific gravity could be obtained using both the density bottle method and the open beaker method. The values obtained are summarized in Table 3.20.

Description of the soil	Specific gravity
Lateritic soil	2.650
1:1.5 Paddy husk mix: Soil	1.42
1: 1 Paddy husk mix Soil	1.37

Table 3.20 – Details of specific gravity

3.4.1.2 Particles Size distribution

Particle size distributions of the mixes were analyzed using the standard sieve analysis. The results are presented in Figure 3.57 for the mix proportions. The particle size distribution of the lateritic soil used is also presented in Figure-3.57

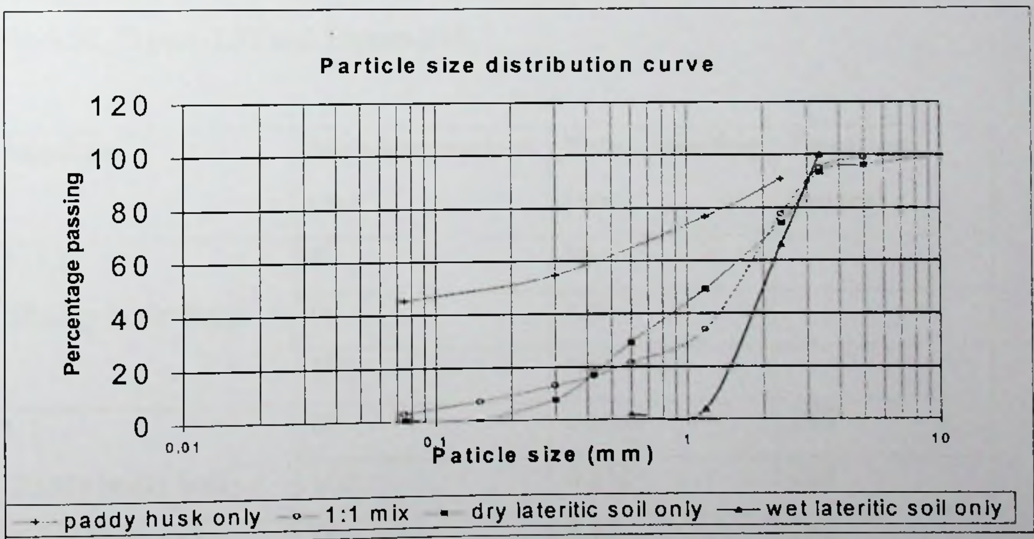


Figure-3.57: Particle size distribution curve for lateritic soil, soil-paddy husk mixes

3.4.1.3 Swelling characteristics of the mixes

The proposed lightweight fill will be used on grounds underlain by soft soil. These sites are often low lying and water table is quite high (almost at the ground level). Therefore, the possibility of lightweight fill getting submerged subsequently exists. Compacted lightweight fill could be subjected saturation at this stage and some swelling may also take place during the process. Therefore it is important to understand the changes that would take place when the compacted lightweight fill is submerged. The study of this aspect is very important specially in the context the soil mixed at moisture content much lower than the optimum.

Swelling and saturation characteristics of the lightweight fill were studied by placing a sample of the lightweights fill mixes in the conventional consolidation apparatus at different compacted moisture contents. In this process only the porous plate was kept on top of the sample. Since the top loading plate was also of some appreciable weight compared to that of a porous plate, it was not kept there leaving room for free swell. Soil specimens compacted at different initial moisture contents were submerged and the free swell occurred was measured with time. The time required for completion of swelling and saturation was also found in these tests. Swell data obtained are presented in Table-3.21, Figure-3.58 and Figure-3.59.

Mixture	**Moisture content (%)**	**Total swelling (mm)**	**Duration (hours)**
1: 1.5 (Paddy husk: Soil)	80	No	
	90	No	
	100	No	
1:1 (Paddy husk: Soil)	80	0.114	288
	100	0.110	288
Paddy husk only	180	0.14	282

Table-3.21 Swelling data for the mixes

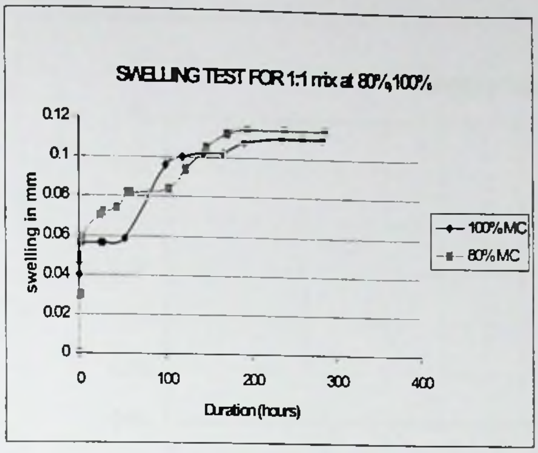


Figure-3.58: Swelling for 1:1 mix

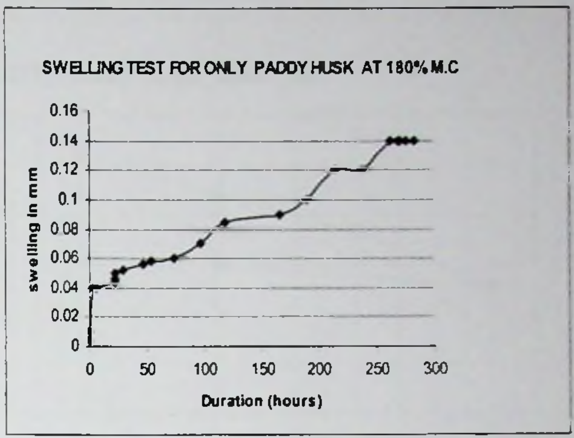


Figure-3.59: Swelling of paddy husk

No swelling was seen with 1.5:1 (soil: paddy husk) mix, but 1:1 (soil: paddy husk) showed some small swelling. When the compaction moisture content was lower, the swelling was slightly higher.

3.4.2 Compressible characteristics of the developed lightweight fill material.

In order to determine the compressibility characteristic of the developed lightweight fill a series of consolidation tests were conducted. Considering the possibility of saturation of the lightweight fill, tests were conducted on both the as compacted fill and saturated fill. Consolidation characteristic of mixes were determined in the laboratory in an Oedometer. Properties such as coefficient of consolidation (Cv), coefficient of volume compressibility (mv), and compression Index (Cc) are determined in the Oedometer.

3.4.2.1Consolidation tests on as compacted fill

The lateritic soil and paddy husk-soil mixture was statically compacted in a consolidation ring to get a sample. Vertical stress was increased up to 80kN/m^2 in steps in the standard one-dimensional consolidation setup. The **e-log σ** curve for the paddy husk mixture is shown in Figure 3.60 and Figure3.61. The variations of the values of the coefficient compressibility (mv) with vertical stress are shown in Figure 3.62 and the variations of the values of the coefficient of consolidation (Cv) with vertical stress are shown in Figure 3.63.

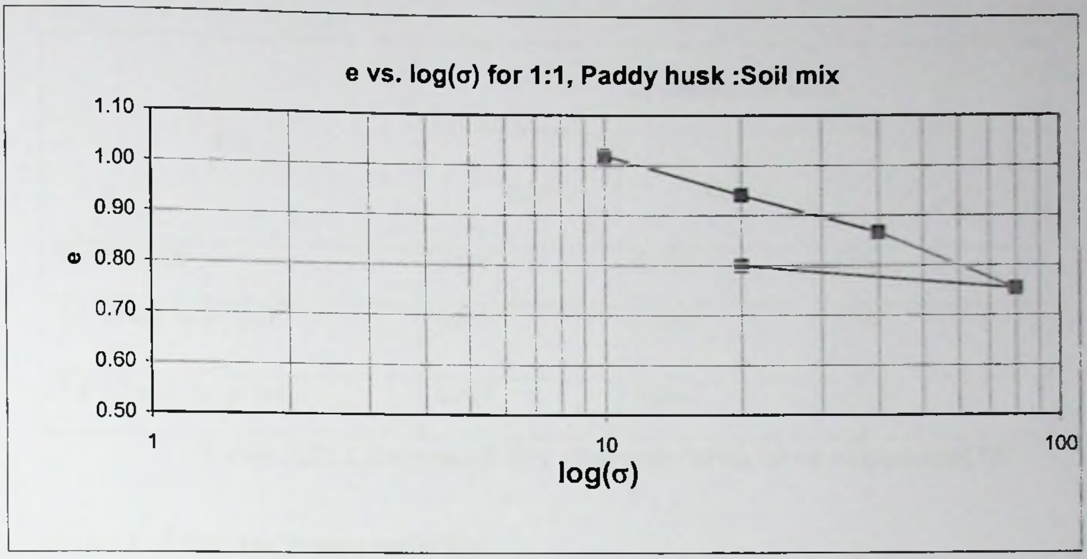


Figure 3.60 e vs log (σ) plot for 1:1 mix at as compacted state

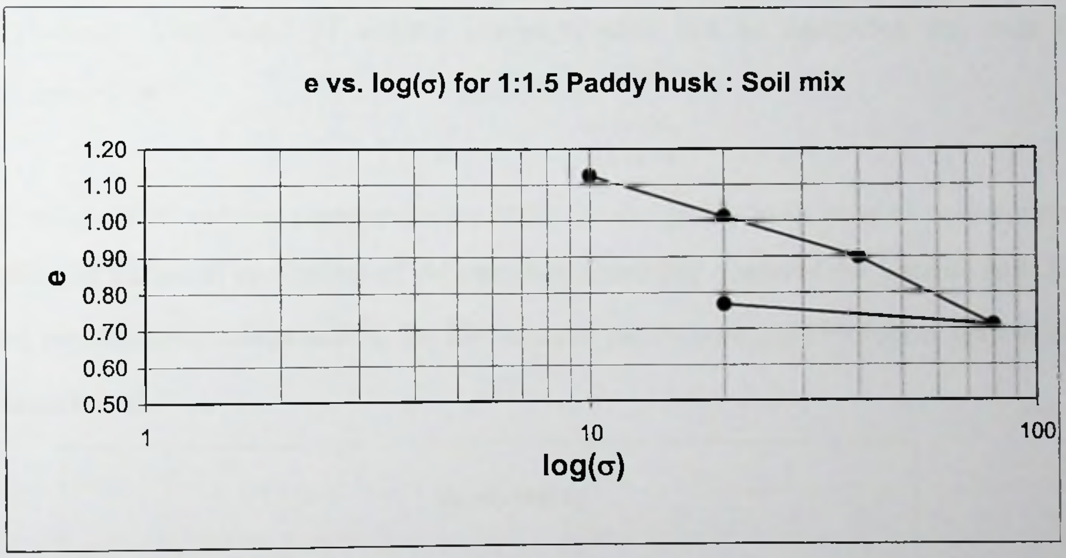


Figure 3.61 e vs. log (σ) plot for 1:1.5 mix at as compacted state

Compression index

In **e-log** σ curve the gradient of steeper slope is Cc and is termed a compression index. Higher Cc value indicates that the material is compressible.

Fill	As compacted state		
	C_c	C_s	$C_c / 1+e_0$
Lateritic fill	0.029	0.007	0.018
1:1 Paddy husk: Soil	0.382	0.100	0.182
1:1.5 Paddy husk: Soil	0.249	0.066	0.116

Table 3.22 Compressibility characteristics of as compacted fill

Coefficient of volume compressibility

Coefficient of volume compressibility is an alternate parameter for estimating the settlement. Coefficient of volume compressibility can be computed for each load increment by

$$m_v = \frac{\Delta H}{H \, \Delta\sigma v}$$

Coefficient of volume compressibility (mv) for the paddy husk mixture was compared with the values of coefficient of volume compressibility observed for lateritic soil. Since the mix is highly compressible, the mv value of paddy husk little bit higher than value of lateritic soil

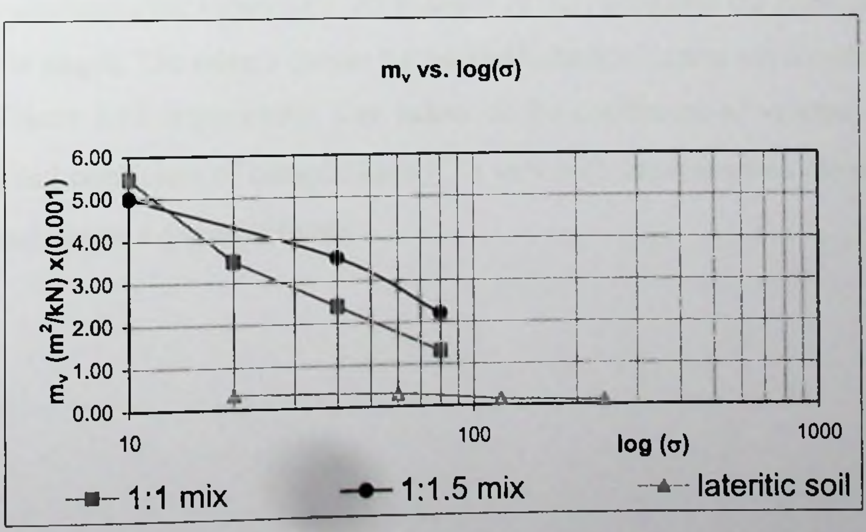


Figure 3.62: Effect on paddy husk mixing on mv for lateritic soil as compacted

Coefficient of consolidation

When it is required to predict the time rate of settlement of soil in the field, it is necessary to know the coefficient of consolidation Cv for the soil. Magnitude of Cv may be different for the different load increment .The Cv values for the paddy husk: soil, 1:1, 1:1.5 mixes are presented in figure 3.63

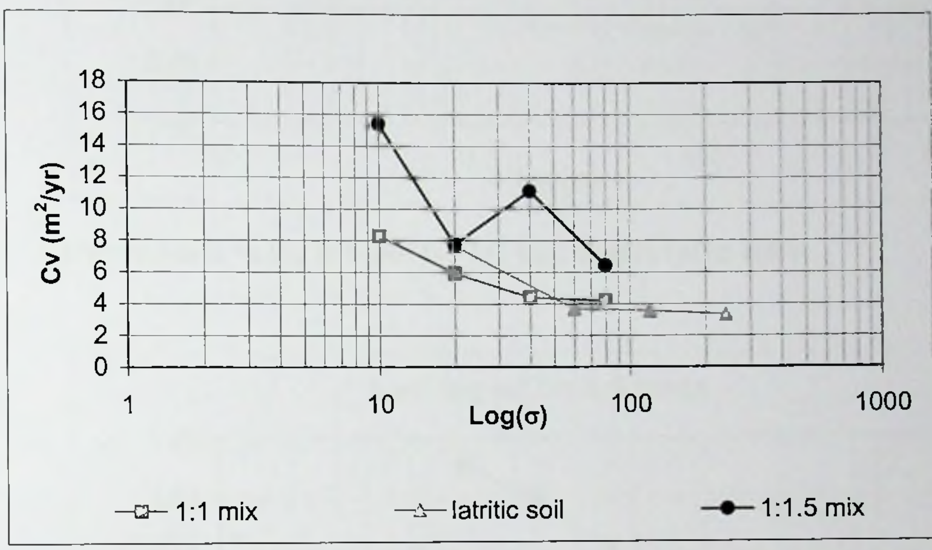


Figure 3.63 Effect on paddy husk mixing on Cv for lateritic soil as compacted

3.4.2.2 Consolidation tests under saturated condition

After saturating the lightweight fill material in the oedometer the consolidation test was done in stages. The **e-log** σ curves for the paddy husk mixtures are shown in Figure 3.64, and Figure 3.65 respectively. The values of the coefficient of volume compressibility (mv) and coefficient of consolidation (Cv) vary with vertical stress are shown in Figure 3.66and Figure 3.67 respectively.

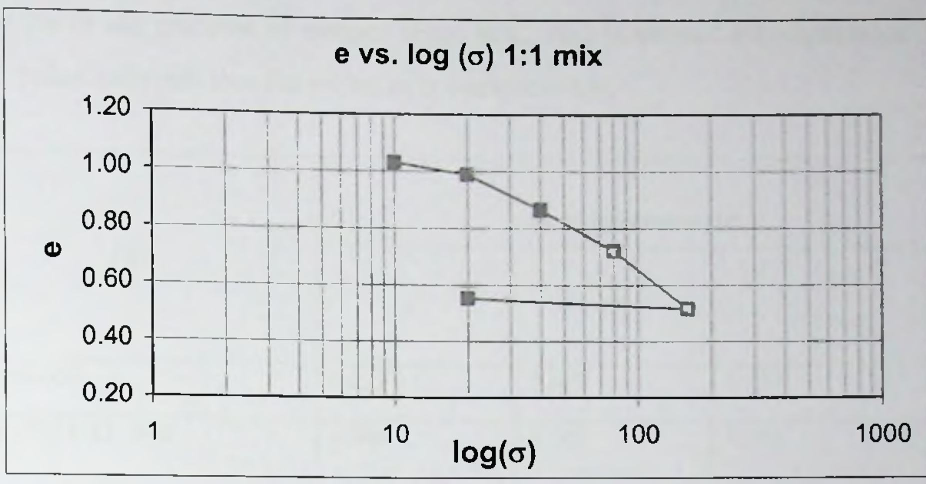


Figure 3.64e vs log (σ) plot for 1:1 mix at saturated state

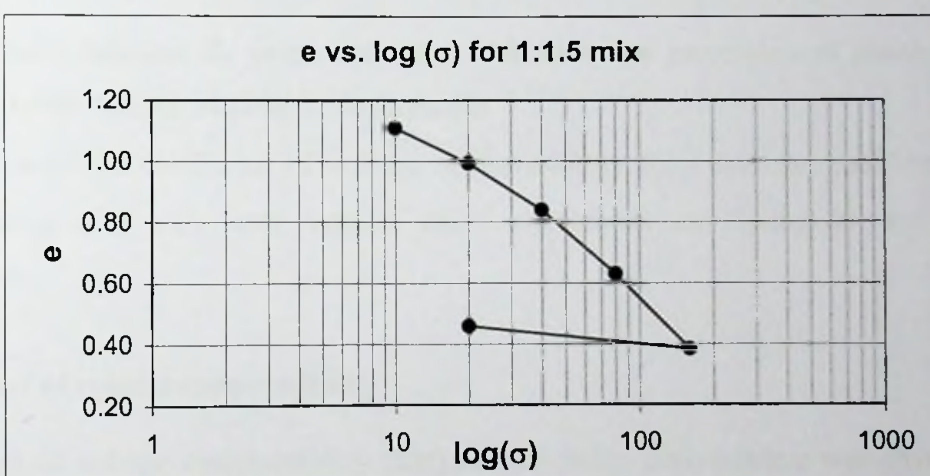


Figure 3.65 e vs log (σ) plot for 1:1.5 mix at saturated state

Compression index

The void ratios of mixes are greater than those of lateritic soil at higher stress its converse. The gradients of e vs. log σ plot are greater in the mixes. This implies that the mixes are more compressible that the lateritic soil. The values of compression index obtained are presented in Table-3.23

In **e-log** σ curve the gradient of steeper slope is C_c and is termed a compression index. Higher C_c value indicates that the material is compressible.

Fill	**As compacted state**		
	C_c	C_s	$C_c/1+e_0$
Lateritic fill	0.029	0.007	0.018
1:1 Paddy husk: Soil	0.409	0.041	0.195
1:1.5 Paddy husk: Soil	0.501	0.051	0.236

Table-3.23 Compressibility characteristics of saturated fill

Above results indicates C_c values increased with increased percentage of paddy husk higher C_c value indicate material is compressible.

The values of the coefficient of volume compressibility (m_v) and the coefficient of consolidation (C_v) vary with vertical stress are shown in Figure3.66 and 3.67 respectively.

Coefficient of volume compressibility

Coefficient of volume compressibility (mv) for the paddy husk mixture was compared with the values of coefficient of volume compressibility observed for lateritic soil Due to increased initial Compressibility of mixes ΔH is very high. Therefore mv value of paddy husk mixture little bit higher than value of lateritic soil.

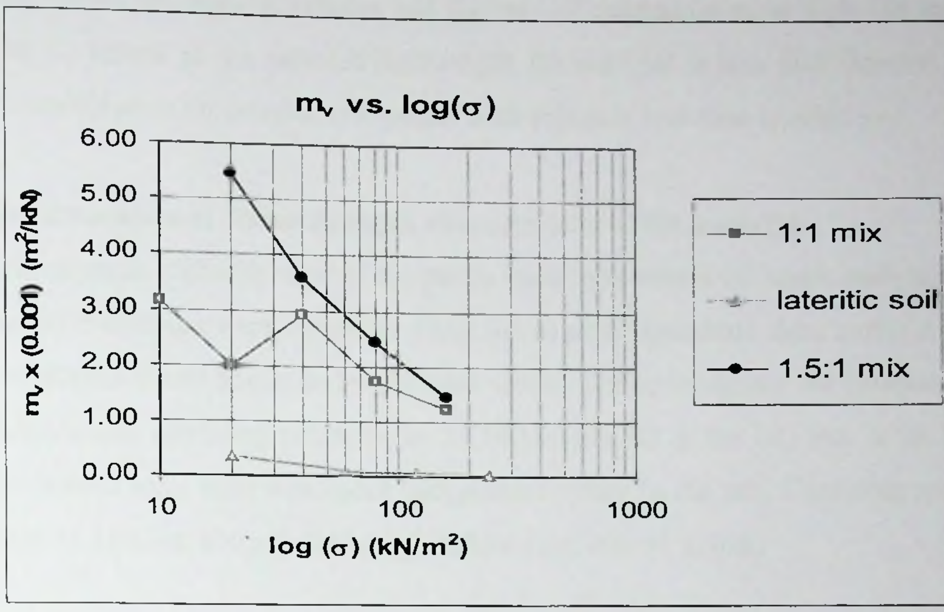


Figure: 3.66 Effect on paddy husk mixing on mv for lateritic soil as saturated

The results indicate that the developed mixes at saturated stage are more compressible than the lateritic soils. However, their compressibility is within acceptable limits to be used a fill material.

Coefficient of consolidation

Variation of C_v value against vertical stress for lateritic soil and different percentage of paddy husk mixture are shown in Figure3.67

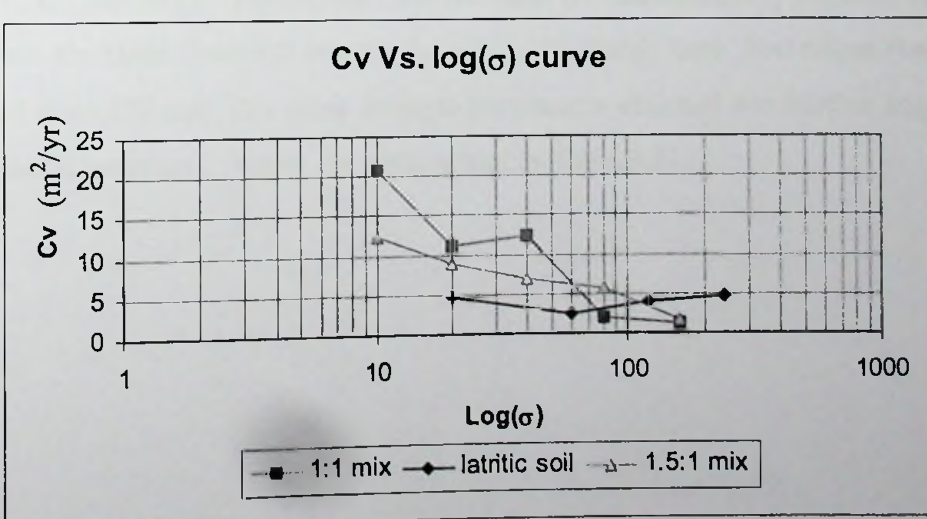


Figure-3.67: Effect on paddy husk mixing on C_v for lateritic soil as saturated

When C_v values are high, it implies that the rate of consolidation is high. At saturated condition C_v values of the selected lightweight fill material is less than lateritic soil so rate of consolidation for selected soil-paddy husk mixes is less than lateritic soil.

3.4.3 Determination of Shear strength characteristic of fill material

The shear strength characteristics of the paddy husk lightweight fill under both undrained and drained conditions were measured using the tri axial apparatus. Specimens that were 38mm in diameter and 85mm in height were tested. The specimens were compacted and were subjected to confining pressures of 25,50,100 kN/m2 in the UU test. With the CD tests the sample were saturated under backpressure prior to the test. Confining pressures used were 75,150,200 kN/m2 and backpressures used was 50 kN/m2

Figure 3.68, 3.70 show deviator stress versus axial strain data obtained from the 1:1, 1: 1.5 Paddy husk: Soil lightweight fill material in the UU test respectively. There is an increase in deviator stress as confining pressure increases for the soil and paddy husk mixture. But specimen did not show the ultimate shear failure. Therefore the deviator stress value relevant to 20% strain was used to obtain the Mohr circle for further calculations. Similar approach was followed for CD test. Figure 3.67 and Figure 3.69 shows deviator stress versus axial strain data obtained from the lightweight fill material in the CD test. Figure 3.59, Figure 3.71 show the Mohr Coulomb envelop for 1:1, 1: 1.5 Paddy husk: Soil mixes respectively obtained from UU test. Similarly Figure 3.73, Figure 3.75 show the Mohr Coulomb envelop for 1:1, 1: 1.5 Paddy husk: Soil mixes respectively obtained from CD test. The shear strength parameters obtained are friction angle ϕ and the cohesion intercept C values are summarized in Table.3.24.

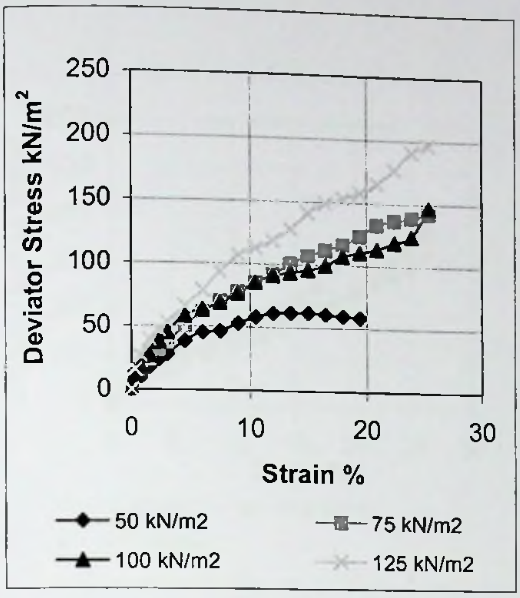


Figure 3.68 Deviator stresses Vs Strain for 1:1 AS compacted mix

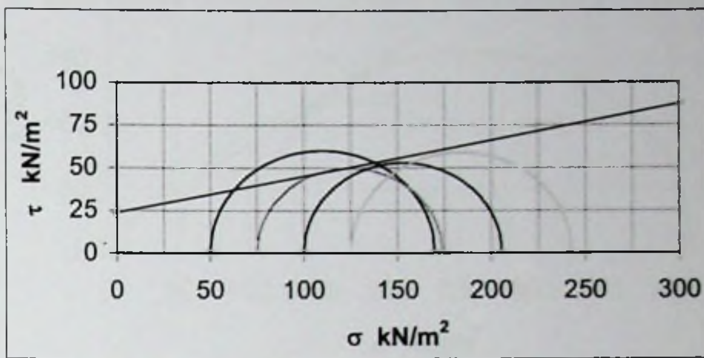


Figure 3.69 Mohr Circle plot for 1:1 compacted mix from UU test

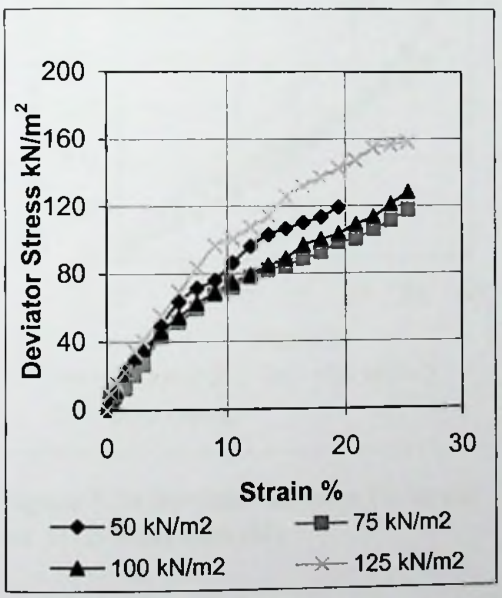


Figure 3.70 Deviator stresses Vs Strain for 1:1.5 AS compacted mix

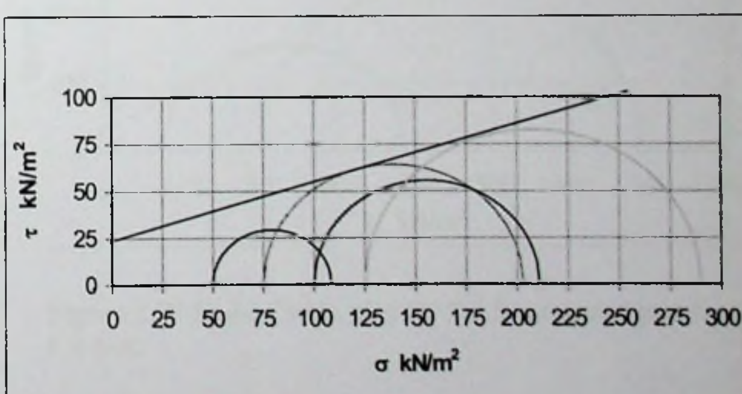
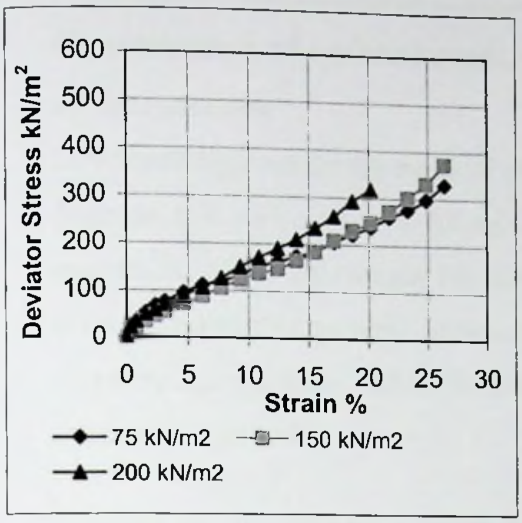


Figure 3.71 Mohr Circle plot for 1:1.5 compacted mix from UU test



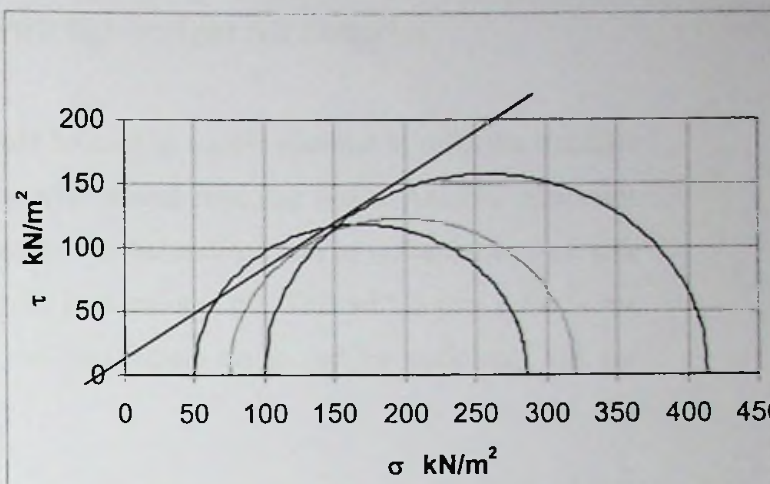
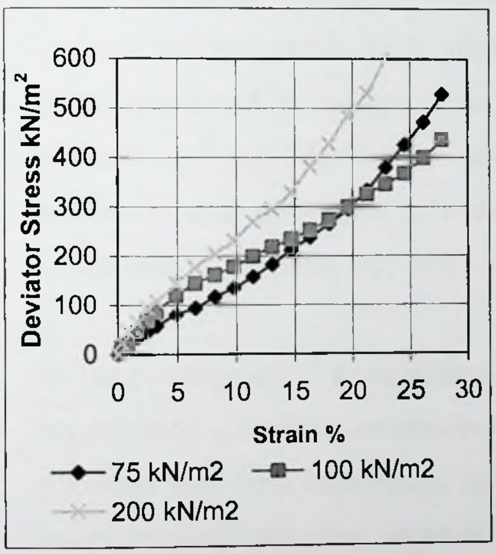


Figure 3.72 Deviator stresses Vs Strain for 1:1 saturated mix

Figure 3.73 Mohr Circle plot for 1:1 saturated mix from CD test.



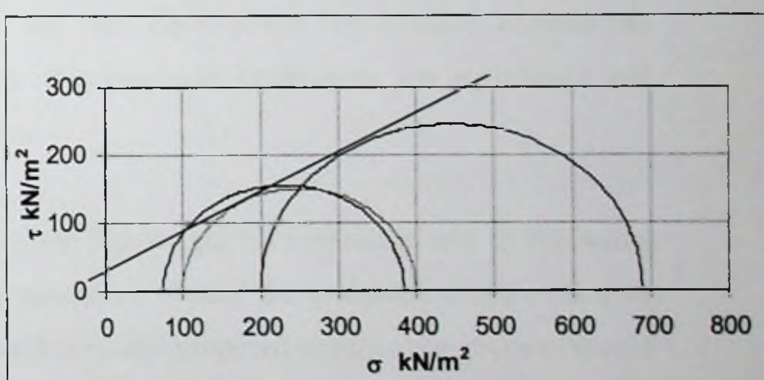


Figure 3.74 Deviator stresses Vs Strain for 1:1.5 saturated mix

Figure 3.75 Mohr Circle plot for 1:1.5 saturated mix from CD test.

Soil type	Cu (kN/m^2)	Φu	Cd (kN/m^2)	Φd
1:1 Paddy husk: soil	24	15^0	12.5	35^0
1: 1.5 Paddy husk: soil	24	16^0	30	30^0
Lateritic soil	108	0^0	40	33^0

Table 3.24 Shear strength parameters

Construction of embankment with lightweight fill material

4.1 Introduction

Developed lightweight fill material could be used as a refill material to refill the trenches done on soft soil, and as a filling material behind retaining walls. Another important application of the lightweight fill material is in the construction of embankments on soft clays completely or partially replacing the conventional fill. This will improve the factor of safety against shear failure in the soil also reduce the in service settlement and the construction period.

Lightweight fills impose a lower bearing pressure and generate less settlement than conventional fill material such as lateritic soil and rock fill. As well as reducing the loads on the ground, lightweight fill requires less internal strength to support their own weight. Vertical and horizontal loads, differential settlements and the amount of long-term settlements could be reduced. The aspects to be studied in construction of complete embankment with lightweight fill are the; development of suitable construction techniques and guidelines to ensure the long-term settlements are minimized and environmental problems also minimized.

As the construction of embankment by the lightweight fill material is new to SriLankan environment a suitable construction procedure should be evaluated which will give minimum long-term settlements. In addition, this proposed construction process should be economical than other alternate ground improvement solutions such as preloading, preloading with vertical drains and use of stone columns.

For the proposed construction process a typical cross section from the Colombo Katunayake expressway subgrade was selected. The subsoil consists of peaty clay underlain by a dense sand layer and the water table is very close to natural ground level.

The primary requirement in the design of the road embankment is that the in-service settlements should be within acceptable limits. To ensure this different ground improvement methods will have to be used depending on the prevailing site conditions such as the soft layer thickness and the height of the embankment. This project concentrates on the use of preloading as the method of improvement.

With the preloading process the embankment may have to be done in stages to ensure that there will be no catastrophic shear failures at any stage. In the staged construction process, the soft soil was allowed to consolidate under the weight of the safe fill height placed during the stage 1.Thereafter the next stage of fill is placed. In all stages a sufficient safety margin is maintained against possible shear failure.

4.2 Conventional process of preloading in road embankments

In the preloading process a load equivalent to or exceeding the design load on the soil is applied before the construction of the structure and left until the required consolidation of the soft clay is achieved. During this time soil will consolidate and will increase its shear strength. With the removal of preload soil will become an over consolidated soil and residuals settlements due to the structural load will be small. In a road embankment, the road pavement and traffic load is taken as the structural load.

In the case of a road embankment, the required height increase may be large and the thickness of the soft clay layer could also be high. The preload fill needed to compensate for the structural load of the pavement and the traffic load will have be applied above this. As a result, it may be necessary to do the construction in stages. In such cases, the embankment construction within the available time may not be possible with preloading alone. It may be necessary to use prefabricated vertical drains and stone columns in some cases, specially when the soft soil layer thickness is high.

However, in this study the construction of a 3-5m high embankment on a site underlain by 8,10m of soft peaty clay is considerd. The width of the embankment is 30m.The construction was done with preloading only, but the period of construction was somewhat long.

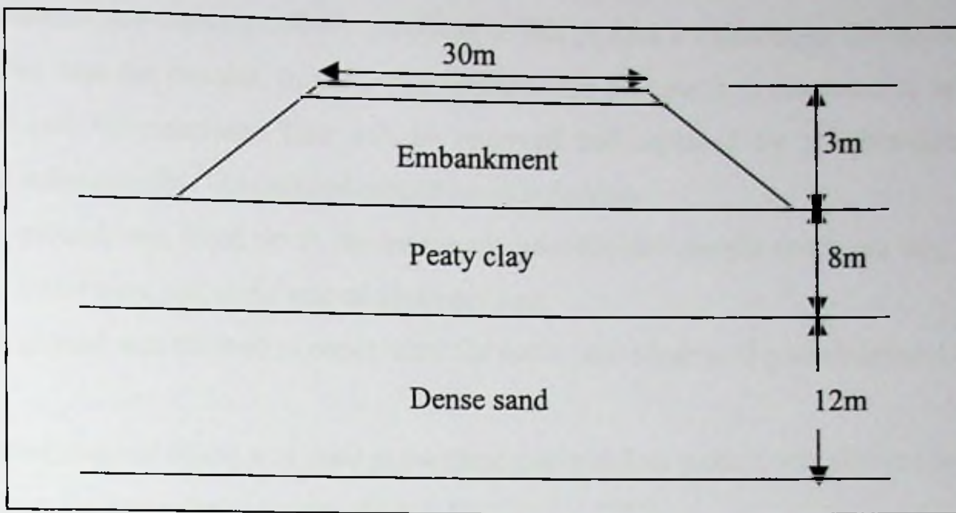


Figure 4.1. The soil profile and the embankment geometry

In this study two different proposals were studied for the construction of embankment over a soft soil. The construction sequence in the conventional construction sequences process is;

1. Embankment was constructed by placing soil in layers at a rate of 10cm/day. This process was continued till the maximum possible safe height of the embankment is achieved. It is that a drainage blanket (sandy soil) of thickness 300mm is placed above the peaty clay. It is modelled as same as the normal fill.
2. Soft ground was allowed to consolidate for some time (To an appropriate degree of consolidation) under the weight of the embankment fill placed.
3. If the required height of the embankment is greater than that placed already, second stage of filling was done to the maximum safe height at the same rate.
4. Soft ground was allowed to consolidate for some time (To an appropriate degree of consolidation)
5. If further filling is needed it is done in another stage and soft soil was allowed to consolidate further.
6. Then the surcharge equivalent to pavement and traffic load is removed.
7. Pavement is constructed gradually. (Within 90 days)
8. Vehicle load (a load of $10kN/m^2$) is applied.

The possible reductions that could be gained in the consolidation period with the use of the vertical drains are not considered in this study.

4.3 Proposed process using a fill material

In the construction sequence newly proposed in this project a lightweight fill material is introduced into the process. In the initial construction process it is proposed to use the conventional fill materials. This will be removed and replaced by a lightweight fill material subsequently. The detailed procedure is as follows.

1. The ground was filled up to the maximum possible safe height (with out any shear failure), by lateritic soil at the rate of 10cm per day.
2. Soft ground was allowed to consolidate for some time (degree of consolidation U= 70%).
3. Second stage of filling was done at the same rate and Soft ground was allowed to consolidate for some time (degree of consolidation U= 70%).
4. The lateritic fill above the water table was removed.
5. Refilling was done by the proposed lightweight fill material.
6. Pavement was constructed gradually in 90 days.
7. Traffic load was simulated by application of a static load of $10kN/m^2$.

The height of the lateritic fill placed and the degree of consolidation to be achieved before the next stage of filling was to be decided by doing number of trial analyses. The criterion to be satisfied in these analyses was that the in-service settlement has to be less than 50 mm. It is admitted that in practice sometimes tolerance limits as high as 150mm are adopted.

4.4 Need for accurate modelling

The selected sections for the proposed construction process consist of soft peaty clay layers up to 8-12m thicknesses. Peat typically has high natural water content, high void ratio and high compressibility. The hydraulic conductivity of peat is typically high at high void ratios but reduces significantly as the peat compresses. This behaviour of peat should be captured in any type of numerical modeling.

Peat available at the proposed site has undrain shear strength parameters $\phi_u=0^0$ and $c_u=10$ kN/m^2 effective shear strength parameters $\phi'=26^0-28^0$ and $c'=0$ (Kugan, 2003). The initial

4.3 Proposed process using a fill material

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3. Second stage of filling was done at the same rate and Soft ground was allowed to consolidate for some time (degree of consolidation U= 70%).
4. The lateritic fill above the water table was removed.
5. Refilling was done by the proposed lightweight fill material.
6. Pavement was constructed gradually in 90 days.
7. Traffic load was simulated by application of a static load of 10kN/m^2.

The height of the lateritic fill placed and the degree of consolidation to be achieved before the next stage of filling was to be decided by doing number of trial analyses. The criterion to be satisfied in these analyses was that the in-service settlement has to be less than 50 mm. It is admitted that in practice sometimes tolerance limits as high as 150mm are adopted.

4.4 Need for accurate modelling

The selected sections for the proposed construction process consist of soft peaty clay layers up to 8-12m thicknesses. Peat typically has high natural water content, high void ratio and high compressibility. The hydraulic conductivity of peat is typically high at high void ratios but reduces significantly as the peat compresses. This behaviour of peat should be captured in any type of numerical modeling.

Peat available at the proposed site has undrain shear strength parameters ϕ_u=0^0 and c_u =10 kN/m^2 effective shear strength parameters ϕ'=26^0-28^0 and c'=0 (Kugan, 2003). The initial effective stresses within the peat are initially very low. If loaded too quickly without significant pore pressure dissipation during loading, peat is likely to fail. For example Flaate and Rygg (1964), Ripley and Leonoff (1961) and Lupien et al. (1983) reported "shear failures" that, on the basis of published data, appear to involve failure in the peat and not in the soil underlying by peat. These failures do not usually involve the formation of a definite sliding surface. Rather, the collapse involves rapid and excessive shear deformation that give rise to large embankment settlements and lateral movements. So it should be emphasized that an embankment may be considered to have failed owing to excessive settlement (which is a serviceability assessment) even though collapse though ultimate failure, in the sense of unconfined plastic flow, has not occurred.

As the embankment on peat fails by excessive displacement due to localized plastic failure zone at the toe, the conventional limit equilibrium methods of analysis are not suitable. The limitation is the fact that these methods do not provide any information regarding deformation prior to collapse. In addition these available conventional equations developed under certain assumption could not capture the anistropic behaviour of the peat. As such, actual stress path of the peat should be followed to capture anistropic yielding behaviour of peat.

The proposed construction process consists of number of construction stages such as filling, removing, refilling, loading etc. For accurate modeling of this construction sequence, the best option available is to do a simulation using an appropriate finite element program.

4.5 Selection of a finite element program.

With the advance of computer technology over the recent years, the finite element method has become a very feasible and desirable method of analysis. Accurate stress strain characteristics of a material now can be accurately and readily simulated via the incremental technique with an appropriate soil model. This allows for the analysis of built up and excavated slopes where the stress history is important in the stability assessment. The most significant aspect of the finite element method is its ability to cater for the many physical and geomechnical phenomena within its flexible framework. It can be used to solve problems involving nonhomogeneous materials, complex pore pressure and boundary conditions and time dependent sequential loadings. In short, the method has made possible the numerical modeling of many realistic geotechnical problems, which cannot be analyzed adequately by conventional methods.

The application of the finite element method to the analysis of stresses and displacements in embankments has been well established and applied successfully by many workers. However, the use of the method in the evaluation of slope stability has not received wide attention. The main advantage of FE analysis over the limit equilibrium method is the information on stresses, deformations, and pore pressures are provided for the problem. However, FE analysis lacks the overall stability assessment, which its counter part can provide.

The proposed construction process of highway embankments on soft clay consists of number of stages. The soil layer placed need to be modeled by adding elements rather than applying loads or increasing gravity loads. The placed layers are not expected to undergo any shear failure. Suitable constitutive models are needed to capture behaviour of each soil layers and fill materials. To satisfy the above requirements there are some popular commercial finite element programs such as; ABAQUS, SAGE CRISP, PLAXIS and ZSOIL. Out of these packages SAGE CRISP was very famous and used for the embankment analysis successfully by researchers in the past. This program is available at the University of Moratuwa Geotechnical division. As Such, SAGE CRISP was selected for the simulation of the construction process.

4.6 Critical State Programme (CRISP)

The computer program known as CRISP is the result of continuing research work on Critical State Soil mechanics through years at Cambridge University. Started in 1975, with an original system design, (Zytynnski, 1976), this program had continued to receive considerable amount of enhancements and modifications. CRISTINA, CRISTINA (1980), CRISP (1984,1987,1990) are the end products of continued research work at each stage. The recent enhancements to the programs are mostly; optimizing computer storage, computing time, new element types, and incorporation of Hrovslev surface and to make the program portable. The windows version with increased user friendliness (SAGE CRISP) was developed in late 1990's.

The types of analyses that SAGE CRISP can handle are undrained, drained or fully coupled (Biot) consolidation analysis of two dimensional plane strain or axisymmetric (with axisymmetric loading) solid bodies. Anistropic elasticity, inhomogeneous elasticity, elastic perfectly plastic model (with Tresca, Mohr-Coulomb, Von Mises, Drucker-Prager yield criterion) can be used in addition to Critical State Models.

Linear strain triangle (LST), cubic strain triangle (CST), and Linear strain quadrilateral (LSQ) (with pore pressure as degree of freedom for consolidation analysis) can be used as element types.

For fixity specification, any boundary can be fixed its displacements, pore pressures. Pore pressure fixity can be introduced either by its absolute or excess values. The facility of removal and addition of elements can be used to simulate excavation and construction, often found in geotechnical work. The number of material zones, originally limited to 10 may be increased to a desired number by few modifications to the original program. The stop restart facility allows the analysis to be continued from previous run and this may be an important feature when running the program with large number of load steps.

CRISP uses tangent stiffness approach in analyzing non-linear problems. i.e., incremental or tangent approach. During incremental loading the stiffness appropriate to the current stress levels are used in calculations.

4.6.1 Finite Element Formulation

For the finite element formulation an explicit expression of constitutive law describing stress changes in terms of strain changes is required. Further more, all components of the stress and strain tensor must be related. This has been done using the basics relations implied by the normality law of associative plasticity.

The invariants of stresses in principal stress space are expressed as;

$$p = \frac{(\sigma_1 + \sigma_2 + \sigma_3)}{3}$$

And,

$$q = \left[\frac{1}{2}(\sigma_1 - \sigma_2)^2 + \frac{1}{2}(\sigma_1 - \sigma_2)^2 + \frac{1}{2}(\sigma_1 - \sigma_2)^2 \right]$$

The volumetric strain is defined in terms of void ratio by,

$$\varepsilon_v = (\varepsilon_1 + \varepsilon_2 + \varepsilon_3) = -\frac{(e_0 - e)}{(1 + e_0)}$$

Where e_0 is the void ratio at the start of loading.

When CRISP starts execution, initially it compares the current stress state with the yield locus specified by the user. If the current stress state is within the yield locus, it computes the elastic stiffness matrix (D_e) to calculate elastic strain and thus the stresses.

The constitutive matrix for elastic behaviour is,

$$\begin{pmatrix}
K + \frac{4}{3}G & K - \frac{2}{3}G & K - \frac{2}{3}G & 0 & 0 & 0 \\
K - \frac{2}{3}G & K + \frac{4}{3}G & K - \frac{2}{3}G & 0 & 0 & 0 \\
K - \frac{2}{3}G & K - \frac{2}{3}G & K - \frac{4}{3}G & 0 & 0 & 0 \\
0 & 0 & 0 & G & 0 & 0 \\
0 & 0 & 0 & 0 & G & 0 \\
0 & 0 & 0 & 0 & 0 & G
\end{pmatrix}$$

Where,

G=Shear Modulus

K=Bulk Modulus

If ν' is specified in the input, G will be calculated from equation

$$G = \frac{E}{2(1 + \nu')}$$

And bulk modulus is calculated from

$$K = \frac{3P}{\kappa} (1-\nu')(1 + e_0)$$

And,

$$K = \frac{dp}{d\varepsilon^e_v}$$

Elastic Behaviour

The elastic volumetric strain is expressed as,

$$d\varepsilon^e_v = \frac{\kappa}{1+e_0} \frac{dp}{p}$$

The above equation defines the tangent Young's modulus such that,

Writing k as,

$$\kappa = \frac{E}{3(1-2\nu')}$$

If a suitable value of ν' is substituted, the tangential Young's modulus E, can be calculated in terms of mean effective stress and k. Therefore, the constitutive matrix for elastic response is formed in terms of bulk modulus, K and ν' or G. The elements of this

$$E = \frac{3P}{\kappa} (1 - 2\nu')(1 + e_0)$$

D_e matrix are functions of e_{cs}, k, λ and either ν' or G.

It should be noted here that the assumption made for the Cam Clay models imply that zero elastic strains inside the yield locus. This means the shear modulus should be infinitely large. Since this causes difficulty in implementing the model in finite element program, the program is allowed to calculate some finite realistic elastic shear strain inside the yield locus. The user specified ν' or G value cater for this computations.

Plastic Behavior

In order to perform non-linear finite element analysis using elasto-plastic models of soil behaviour, it is necessary to compute the modulus matrix D_{ep}. where $(\sigma') = D_{ep}$

$$D_{ep} = \left[1 - \frac{D_E a a^T}{a^T D_E a - c^T H a} \right] D_E$$

(ε). Starting from the yield function f $(\sigma, h)=0$,there is a piece of standard manipulation to obtain a formula for D_{ep} (ref. Zienkiewicz, 1977).

Where σ is the total stress and h is the hardening parameter.

Where, $a = \partial g/\partial \sigma$, $c = \partial f/\partial h$ and H is a matrix relating changes in hardening parameters to changes in the incremental plastic strain $dH = Hd\varepsilon^P$.

Incorporation of Pore Water Pressure

CRISP always make the computations in effective stress terms. Therefore, when an undrained analysis performed, the program uses the input parameters "K_w " the bulk modulus of water, in the calculation procedure. The following steps show how the program uses the parameter K_w in undrained analysis.

The effective stress law can be written in matrix notation:

$\sigma = \sigma' + m u_w$

Here, u_w is the pore water pressure and m is a vector indicating which stress terms participate in the effective stress relation. For example, if a fully three dimensional stress condition is considered,

$$\sigma = [\sigma_x \, \sigma_y \, \sigma_z \, \tau_{xy} \ \tau_{yz} \ \tau_{zx}]^T$$

$$\sigma' = [\sigma'_x \, \sigma'_y \, \sigma'_z \, \tau_{xy} \ \tau_{yz} \ \tau_{zx}]^T$$

$$m = [1\ 1\ 1\ 0\ 0\ 0\ ]^T$$

Suppose an element of soil undergoes an incremental total stress change $\Delta\sigma$ which results in a change of pore pressure Δu and incremental strain $\Delta\varepsilon$. Suppose also that incremental effective stresses are related to the incremental strains by relationship,

$$\Delta\sigma' = D'\Delta\varepsilon$$

Where, D' may describe either an elastic or an elastic-plastic law. The assumption is now made that the volumetric strain experienced by the soil is due entirely to a change in volume of pore water. The volumetric strain experienced by the pore water is equal to $[(1+e)/e]\ m^T\Delta\varepsilon$ where, e is the current voids ratio. Then the change in pore water pressure is given by,

$$\Delta u_w = K_w[(1+e)/e]\ m^T\Delta\varepsilon$$

Combining this with the effective stress law and the incremental effective stress strain relation the following equation is obtained,

$$\Delta\sigma = D'\Delta\varepsilon + K_w\ [(1+e)/e]\ m^T\Delta\varepsilon$$

CRISP uses this equation in the following way:

1. The program expects in an undrained analysis that the material properties supplied relate to changes in effective stress.

2. When calculating the element stiffness matrices the program adds in the terms corresponding to the volumetric stiffness of the pore water.

3. Following the solution of the finite element equations the program calculates the changes in effective stresses and pore water pressure separately.

In a drained analysis the users sets $k_w = 0$ and no changes in pore pressures are calculated. For elastic material behaviour the above procedure for an undrained analysis is equivalent to using a Poisson's ratio close to 0.5.However the procedure has the advantage that the pore pressure changes are calculated explicitly and exactly the same technique is valid for an elasto plastic material law. It is well known that in the conventional linear elastic finite element analysis the use of value of 0.5 can lead to numerical ill conditioning of the finite element equations. The use of value of Poisson's ratio in the range suggested above is equivalent to the use of a value of Poisson's ratio in the range 0.49 to 0.499 and should provide reasonably accurate results.

In undrained analysis CRISP uses Kw the bulk modulus of water to compute the incremental excess pore pressure as described above. It is recommended to set the value of Kw between 50 to 500 times the effective bulk densities of the soil.

When calculating the element stiffness matrix the program adds in the terms corresponding to the volumetric stiffness of the pore water. A reasonable value of Kw may be selected by observing the predicted pore water pressures.

Solution Procedure.

Out of the number of techniques for analyzing non-linear problems using finite elements, CRISP uses incremental or tangent stiffness approach i.e. during the analysis the stiffness properties are updated depending on the stress levels after each incremental load. These current stiffness properties are directly used to form the constitutive matrix for subsequent computations.

If only few increments are used, the tangent stiffness method produces a solution, which tend to drift away from the true or exact solution. This means a stiffness response for a strain hardening model and the displacements are always under predicted. Therefore it is important to select size of increments for realistic and accurate predictions.

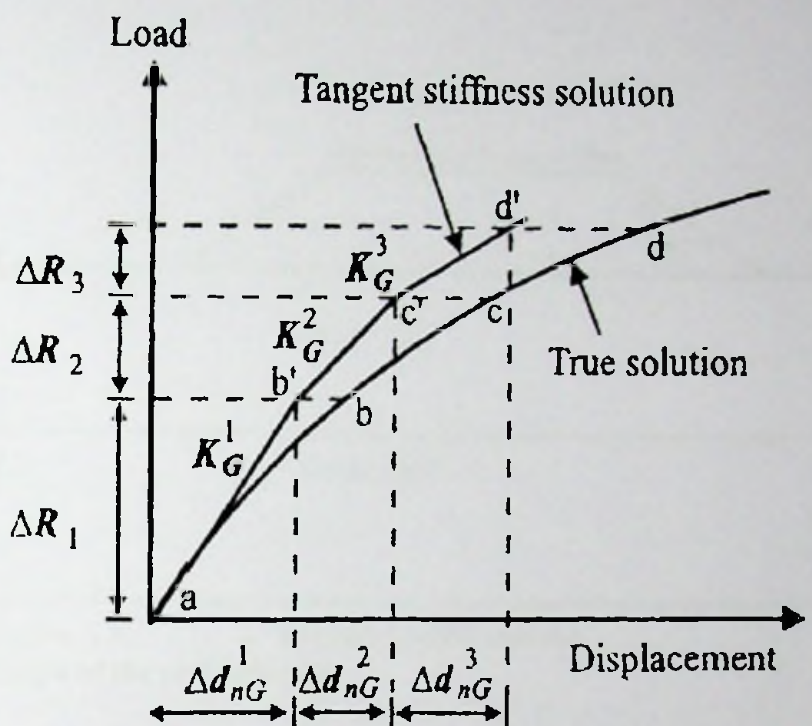


Figure 4.2 Tangent stiffness methods

4.7Modeling Process

The deformation behaviour, stability, and the pore pressure developments of an embankment are controlled by the pattern of the embankment loading and the load distribution in the foundation soil. Varying soil properties of embankment foundation is directly related with the resulting deformation pattern through the constitutive relation ship of the soil. As discussed earlier finite element is the only tool, which can closely model the loading pattern and the load distribution in the foundation soil and thus the behaviour of embankment. This method is effectively used in geometrical disintegration of the continuum, incorporating appropriate constitutive relationships and in defining complex boundary conditions.

The selected section in the Colombo Katunayake expressway is shown in Figure 4.1.as described earlier the proposed embankment consists of staged filling with the surcharge.

Each stage filling is followed by a consolidation stage. The replacement should be done by lightweight fill material and finally pavement and embankment loads were given as a U.D.L.

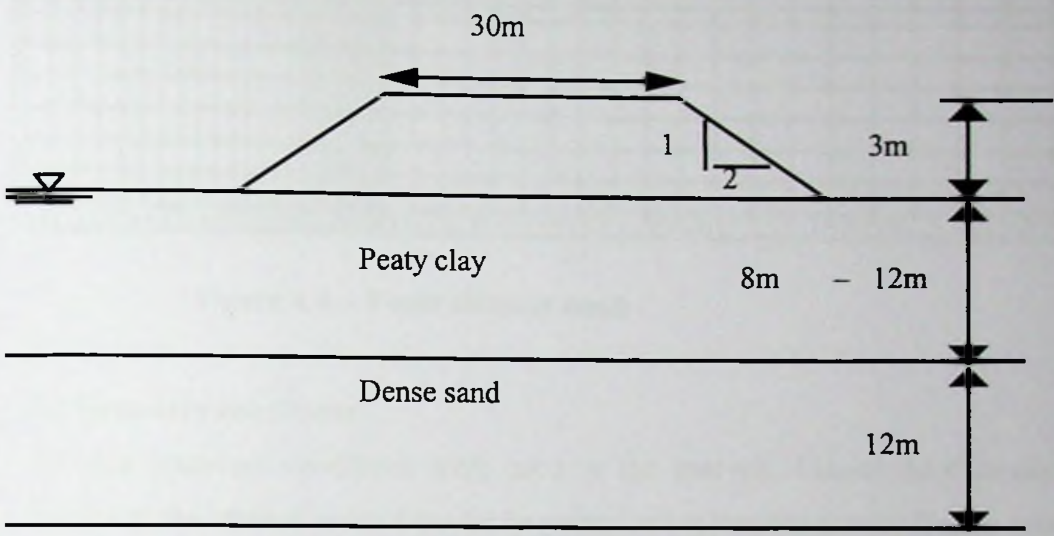


Figure 4.3: - The soil profile and the Shape of the embankment

4.7.1 Mesh Generation

Since the symmetrical conditions prevail in this embankment, consideration of half of the embankment geometry for the analysis purposes will reveal the performance of embankment. In order to satisfy compatibility condition finer elements should be made where rapid stress changes are occurring. In an embankment in which complex stress changes take place the size of the finite elements may cause fair amount of error in the analysis. Normally, below the toe of the embankment, where abrupt stress changes occur, finer elements are appropriate. Under the above concept a typical finite element discretisation of the proposed embankment is shown in Figure 4.4. Linear quadrilateral elements were used in other places except below the toe. Since there are complex stress change close to toe, this area was modeled by small triangular elements. In addition to this, width of the mesh was selected in a way that there were no stress changes close to the boundary.

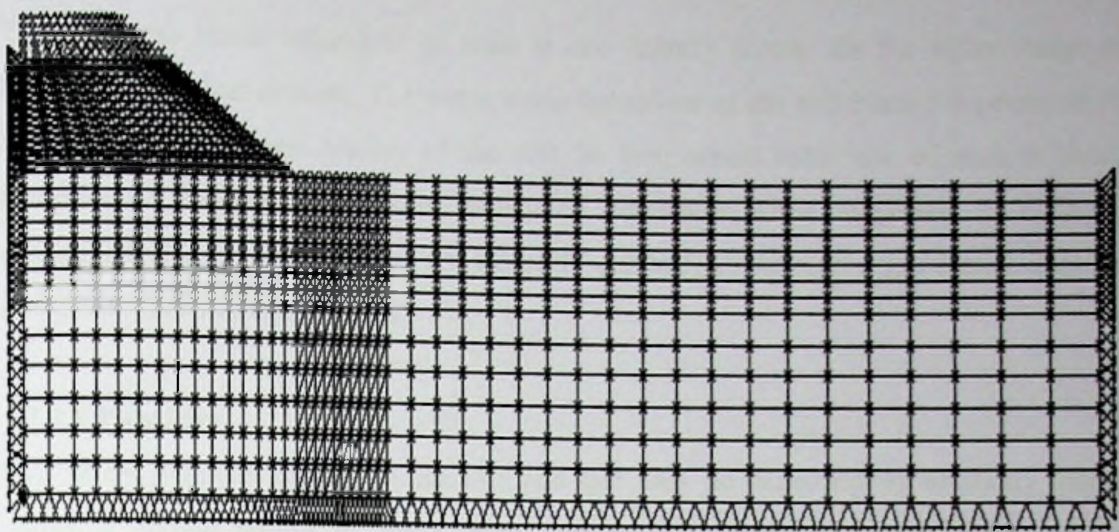


Figure 4.4: - Finite element mesh

4.7.2 Boundary conditions

Following boundary conditions were used in the analysis. Lateral displacements are restricted at the vertical ends of the far boundary and symmetrical axis. Due to symmetry the soil under the centre of embankment only will experience only vertical displacements. At the far boundary also the same conditions would apply. Lateral and vertical displacements are restricted along the bottom boundary to simulate the bedrock at the 20m depths. Free drainage boundary with pore water fixity was applied along the top surface and the bottom boundary of the soft peaty clay. Since there was dense sand at the bottom, two-way drainages were expected.

4.7.3 Selection of models

Embankment design calculation requires an appropriate choice of a soil model to represent the soil behaviour. Settlement calculations typically assume that the soil is linear elastic, and bearing capacity calculation typically assume that the soil is rigid and perfectly plastic. But construction under working loads will certainly have proceeded well beyond a linear elastic range and yet are unlikely to have attained conditions of perfect plasticity.

Typical stress strain behaviour of soils is not linearly elastic for the entire range of loading of practical interest. The stress strain behaviour of the soil mainly depends on its loading path and stress history of the soil. In fact, actual behaviour of soils is much complicated and shows a great variety of behaviour when subjected to different conditions. Drastic idealizations are therefore essential to develop a good mathematical model for practical applications.

4.7.3.1 Plasticity

Soil rarely behaves entirely elastically, and can only be described by elasticity theory within a certain region of stress space. Beyond this region of stress space, plastic deformation occurs. Hence, an understanding of plasticity theory is essential. It is thought that soil only behaves elastically for shear strains approximately less than 10^{-5} (Clayton et al., 1995).

The plastic behaviour of an ideal elastic-plastic material is specified by a yield surface, a flow rule, and a hardening law. The yield surface separates states of stress, which cause only elastic strain from states of stress, which cause both plastic and elastic strains. Strain increments are plotted on the same axes as associated stresses, and the normal to the plastic potential gives the plastic strain increment vector and the flow rule relates the direction of the plastic strain increment vector to the stress state. When the flow rule is associated the plastic strain increment vector is normal to the yield surface. If the plastic strain increment vector is not normal to the yield surface, then the flow is said to be non-associated. However, for any flow rule, a plastic potential can be drawn through a point in stress space. The plastic potential is drawn so as to be perpendicular to the plastic strain increment vector, as shown in Figure 4.5. Thus for associated flow, the yield surface and plastic potential coincide.

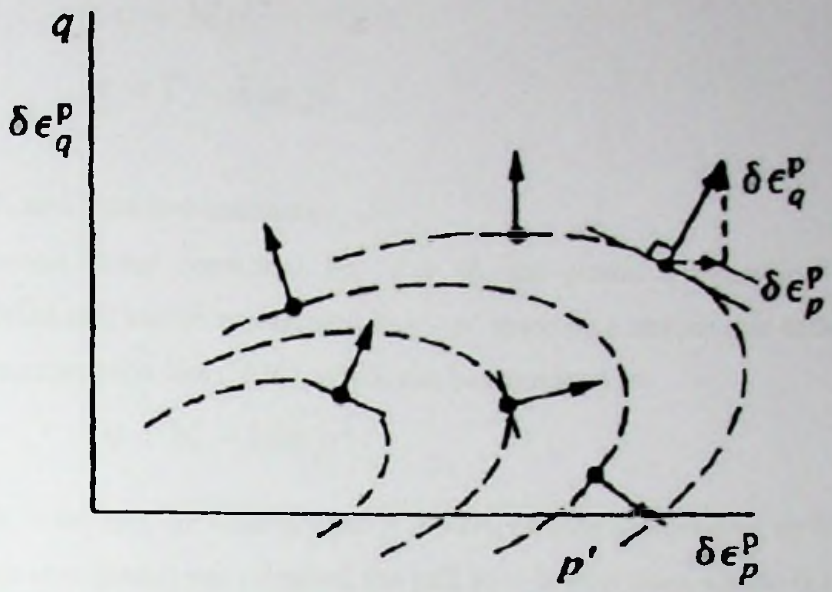


Figure 4.5. Plastic potentials and plastic increment strain vectors (Wood, 1990).

The hardening law relates the magnitude of a plastic strain increment to the magnitude of an increment of stress, as the state of stress causes plastic deformation and the material strain hardens. If the shape of the yield surface is assumed to be constant, and its size is assumed to be a function of plastic volumetric strain only (as is usually the case), then the model is said to be a 'volumetric hardening' model.

4.7.3.2 Critical State

The critical state concept is based on the consideration that, when a soil sample is sheared, it will eventually reach an ultimate or critical state at which plastic shearing can continue indefinitely without changes in volume or effective stresses. This condition can be expressed by:

$$\frac{\partial p'}{\partial \varepsilon_q} = \frac{\partial q}{\partial \varepsilon_q} = \frac{\partial v}{\partial \varepsilon_q} = 0$$

Where v is the specific volume.

When the critical state is reached, critical states for a given soil form a unique line in q-p'-v space referred to as the critical state line (CSL), which has the following equations in q-. p'.-v space:

$$q = Mp'$$

$$v = \Gamma - \lambda \ln p'$$

Where Γ, and λ are soil constants.

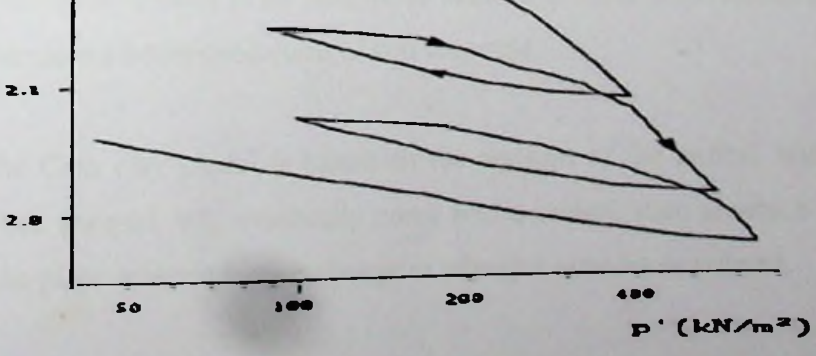
For isotropic stress conditions (i.e. q = 0), the plastic compression of a normally consolidated soil can be represented in $v - p'$ space by a unique line called the isotropic normal compression line (NCL), which can be expressed as:

$$v = N - \lambda \ln p'$$

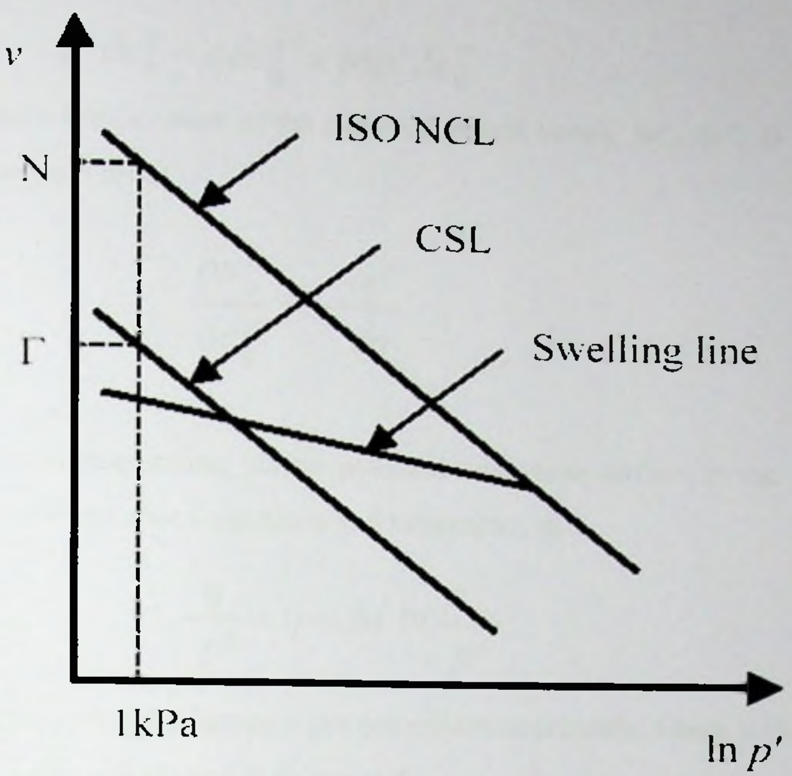
Where N is the specific volume when p' = 1kPa or 1MPa, depending on the chosen units. If the soil is unloaded and reloaded, the path in v. ln p' is quasi-elastic (i.e. hysteretic), as shown in Figure 4.6(a). However, the behaviour is often idealised as perfectly elastic, as shown in Figure 4.6(b), so that the equation of a typical unload-reload line is:

$$v = v_{\kappa} - \kappa \ln p'$$

Where v_{κ} and κ are soil constants. For this reason, unload-reload lines are known as 'κ-lines', as used in critical state soil models such as Cam clay.



(a)



(b)

Figure 4.6. (a) True unload-reload behaviour and (b) idealised unload-reload behaviour of speswhite kaolin in v . ln p' space (Al-Tabbaa, 1987).

4.7.3.3 Cam clay

In this section, an elastic-plastic model, Cam clay, developed by Roscoe, Schofield and Thurairajah (1963) at the University of Cambridge is briefly described. This model is the basis for several more advanced models. In recent times, this classical critical state model has been modified in various ways by many researchers to model different soil types and loading conditions in an attempt to achieve a better understanding of soil behaviour and therefore a better prediction of soil response.

The Cam clay model is based on the concept of the critical state which says that soil, when sheared, will eventually come into a critical state at which unlimited shear strains take place without further change in effective stresses or volume.

The Cam clay yield surface is derived from the work equation as follows:

$$p'\delta\varepsilon_p^p + q\delta\varepsilon_q^p = Mp'\delta\varepsilon_q^p$$

Since the direction of the strain increment vector, $\delta\varepsilon^p_p$, $\delta\varepsilon^p_q$ is assumed to be normal to the yield locus.

$$\frac{\delta\varepsilon_p^p}{\delta\varepsilon_q^p} = -\frac{\delta p'}{\delta q}$$

The corresponding plastic potential and yield surface in the *q-p'* space are given by combining above equations and integrating, as:

$$\frac{q}{p'} = \eta = M \ln \frac{p'_c}{p'}$$

Where p'_c is the isotropic pre-consolidation pressure, which is the value of p' when $\eta = 0$. The curve is plotted in Figure. 4.7.

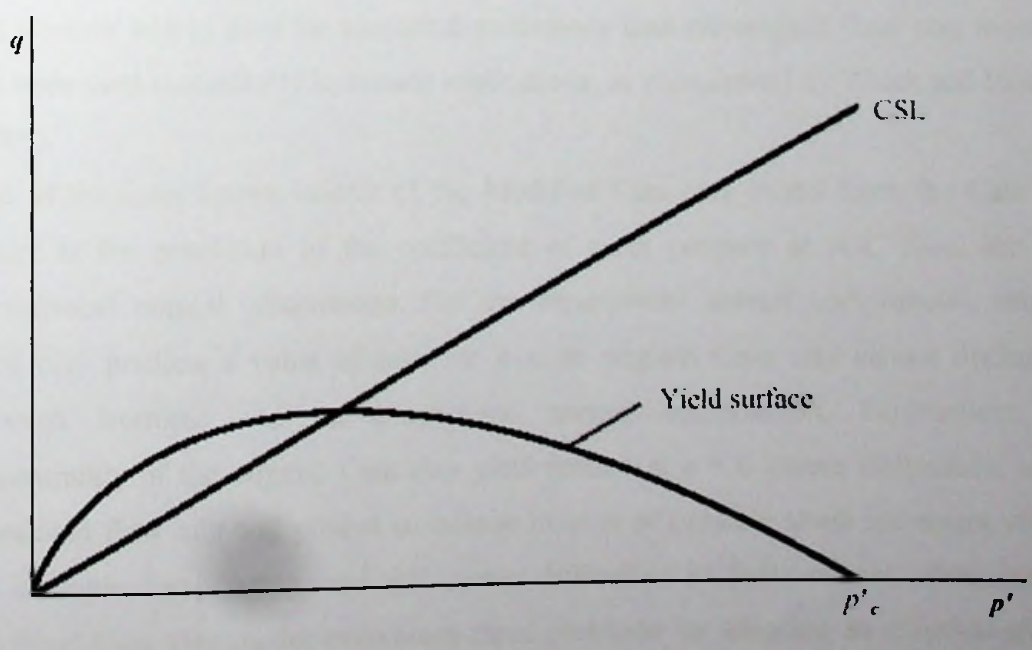


Figure 4.7. Cam clay yield surface.

In Cam clay, it is assumed that the plastic flow obeys the principle of normality or has an associated flow rule: that is, the plastic potential and the yield surface coincide. This is convenient when implementing the model in finite element calculations because the constitutive matrix, $[D_{ep}]$, is symmetric if the plastic potential, g, is equal to the yield surface, f.

The yield surface is assumed to expand at a constant shape, and the size of the yield surface is assumed to be related to changes in volume only, according to the equation:

$$\delta\varepsilon_p^p = \frac{\lambda - \kappa}{v}\frac{\delta p'_c}{p'_c}$$

This is known as volumetric hardening.

4.7.3.4 Modified Cam clay

Modified Cam clay was developed by Roscoe and Burland (1968) as a modification of the original Cam clay model developed by Roscoe, Schofield and Thurairajah (1963). This model successfully reproduces the major deformation characteristics of soft clay, and is more widely used for numerical predictions than the original Cam clay model. It has been used successfully in several applications, as summarised by Wroth and Houlsby (1985).

One of the main improvements of the Modified Cam clay model from the Cam clay model is the prediction of the coefficient of earth pressure at rest, $K_{o,nc}$, for one-dimensional normal compression. For one-dimensional normal compression, original Cam clay predicts a value of zero for $\eta_{o,nc}$ so original Cam clay cannot distinguish between isotropic and one-dimensional normal compression. Furthermore, the discontinuity of the original Cam clay yield surface at q = 0 causes difficulties, as the associated flow rule will predict an infinite number of possible strain increment vectors for isotropic compression, and this causes difficulties in finite element formulations. Modified Cam clay model overcomes these problems by adopting an elliptical-shaped yield surface, as shown in Figure 4.8, and which has the following expression,

$$q^2 = \mathrm{M}^2\left(p'p'_c - p'^2\right)$$

Or

$$\frac{p'}{p'_c} = \frac{\mathrm{M}^2}{\mathrm{M}^2 + \eta^2}$$

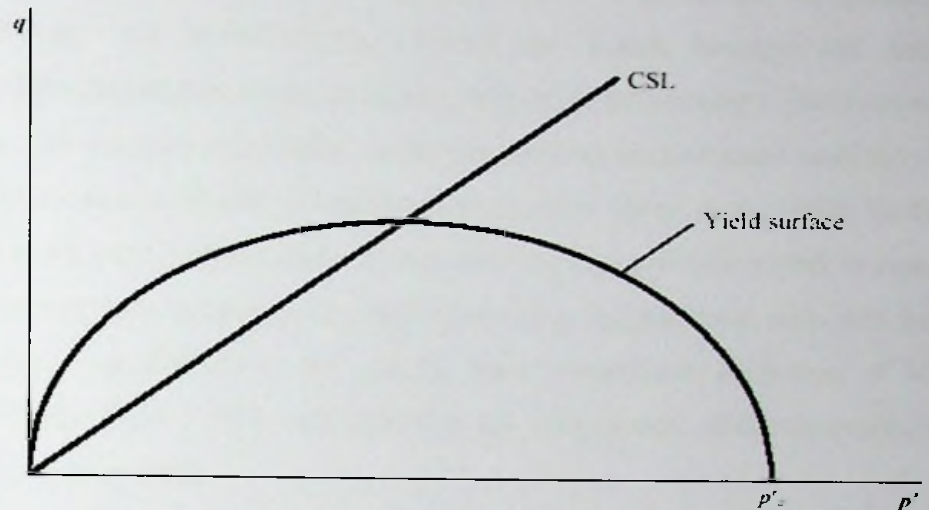


Figure 4.8 Modified Cam clay yield surface.

When the stress states are within the current yield surface, the elastic properties of Modified Cam clay are the same as those in the Cam clay model as described in section 4.7.3.3

Since it is assumed that the soil obeys the normality condition, the plastic potential, *g*, is the same as the yield surface, *f*:

$$g = f = q^2 - \mathrm{M}^2\left[p'\left(p'_c - p'\right)\right] = 0$$

Where *g* and *f* are the plastic potential and yield surface functions respectively. The flow rule for Modified Cam clay is then calculated by application of the normality condition:

$$\frac{\delta\varepsilon_p^p}{\delta\varepsilon_q^p} = \frac{\mathrm{M}^2 - \eta^2}{2\eta}$$

The yield surface is assumed to expand at a constant shape, and the size is controlled by the isotropic pre-consolidation pressure, p'_c. The hardening relationship for Modified Cam clay is:

$$\delta\varepsilon_p^p = \frac{(\lambda - \kappa)}{v}\frac{\delta p_c'}{p_c'}$$

4.7.4 Soil constitutive models for the proposed Embankment

Constitutive models included in the CRISP programme are "Cam Clay" (both original and the modified version), "Elastic Perfectly Plastic" (options for von Mises, Tresca, Drucker-Prager and Mohr-Coulomb models) and "Elastic Isotropic and Anistropic" models. This programme allows undrained, drained, or fully coupled (Biot) consolidation analysis. The coupled consolidation model incorporates the undrained modified cam clay deformations and the drained consolidation settlements (Britto et al., 1987). As described in section 4.6.3 the modified cam clay model is the best available model to represent an anistropic soft soil like peat, which undergoes large deformations. Also this model had been applied successfully in the past by many researchers. Prediction of MIT trial embankments (Worth, 1977) and Malaysian test embankment (Balasubramaniam, 1988) are some cases reported.

In this embankment analysis Peaty clay was successfully modeled by the modified cam clay model. Due to lack of data to find parameters for cam clay models dense sand was modeled by the Mohr-Coulomb elastic perfectly plastic model. In this analysis, the deformation in the dense sand is negligible and the Mohr-Coulomb elastic perfectly plastic model is more than adequate to represent the behaviour of the sand. In this study attempts were made to initially model the fill materials such as conventional lateritic soil fill and the light weight fill by modified Cam clay, but the modified Cam clay model require the stress history detail such as preconsolidation pressure. The simulation of embankment filling and removing in the CRISP is done by adding and removing layers. For these types of operations CRISP does not allow the fill material to be modelled by a stress history models such as Critical State Soil Models. Due to the limitation in the available parameters for hyperbolic model, it was finally decided to use original "Mohr coulomb elastic perfectly plastic model" to model the fill materials. "The Elastic perfectly plastic Model" represented the behavior these layers adequately.

4.7.5 Evaluation of Modified Cam Clay Model parameters for peaty clay

The values of all parameters required for cam clay model can be determined from the results of tests commonly performed in soil mechanics practice. The parameters required for modified Cam clay are; Slope of Normal Consolidation line (λ), Slope of swelling line (k), Slope of Critical State line (M), Void ratio (e_{cs}), Poison ratio (ν), Permeability (k_x, ky), and Unit weight (γ).

Strain parameters

According to the modified cam clay model " λ line " could be obtained from a (υ, ln P') plot (Crisp technical reference manual, 1996) using isotropic consolidation test. But many researchers found the values of the slopes of the two normal consolidation lines, one from the one dimensional consolidation and other from the isotropic consolidation are very close for all practical purposes. As such, both can be assumed to be equal to λ. Similarly the slope of lines can both to be taken as equal to k. (Simith, 1990). In this study standard consolidation tests were done on the peat and from the υ (specific volume) Vs ln p' plot, the values of λ, k were determined (Figure 4.9).

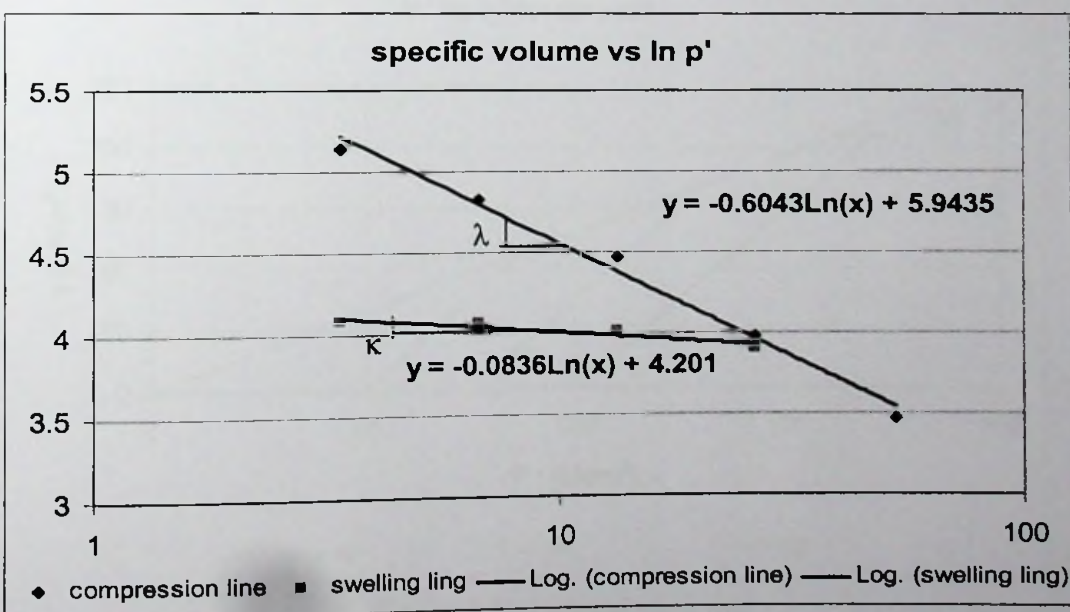


Figure 4.9 Determination of modified Cam Clay parameters.

Frictional constant M

Triaxial tests (drained or undrained with pore pressure measurements) on isotropically consolidated samples can be used to obtain the frictional constant M (i.e. the slope of the Critical state line). With different consolidation pressures, a series of tests are needed to compute the constant M. The best fit drawn, through p', q values at failure gives the value of M.For critical state model, this line is assumed to pass through the origin. Alternatively, if values of ϕ' have already been determined for the soil, then

$$M = \frac{6 \sin \phi'}{3 - \sin \phi'}$$

corresponding M value can be calculated from:

As discussed earlier the consolidated drained triaxial tests were done on 38mm diameter 85mm high samples. The test samples were consolidated at known cell pressures 50kN/m^2, 75kN/m^2, and 100kN/m^2 at the start of the tests. The tests were continued to large strain to ensure the samples are close to the critical state. The end points of the triaxial test results in the (p', q) plot give the critical state line. Gradient of the critical state line give the value of M shown in Figure 4.10.

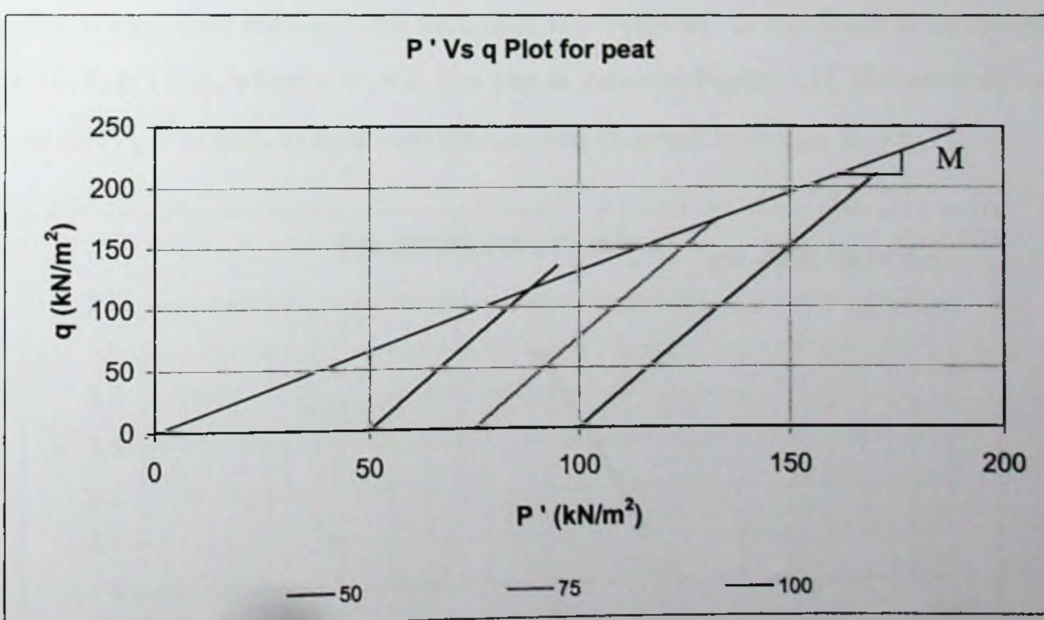


Figure 4.10 Critical state line

Location of CSL in e- ln (p') plot; e_{cs}

The parameter describing the location of the CSL, in e vs ln p' space is defined as the void ratio on the critical state line corresponding to unit pressure. This parameter can be computed using either the water content measurements or the λ and κ values already obtained and one water content value at any point on the stable state boundary surface. In the latter case the following relationship can be used to compute e_{cs}.

$$V_\lambda = \Gamma + (\lambda - \kappa)\left[\ln(2) - \ln\left[1 + \left(\frac{\eta}{M}\right)\right]\right]$$

The term V_λ refers to the specific volume on the virgin compression curve at unit pressure and e_{cs} is equal to κ-1.From the above equations, it is clear that the value of e_{cs} depends on the units of p used in V_λ calculation.

In this study the moisture contents were determined at the end of the each consolidated drained triaxial tests. Using the moisture content w and specific gravity G_s void ratio e was calculated according to the following relation ships.

$$e = w.G_s$$

Critical state void ratio e_{cs} is the void ratio at p'=1kN/m^2. It was found from the intercept of (v, ln p') plot, where v = 1+ e. The plot is shown in Figure 4.11.The value of intercept give the V_λ. The critical state void ratio, e_{cs} can be found from $e_{cs} = V_\lambda - 1$

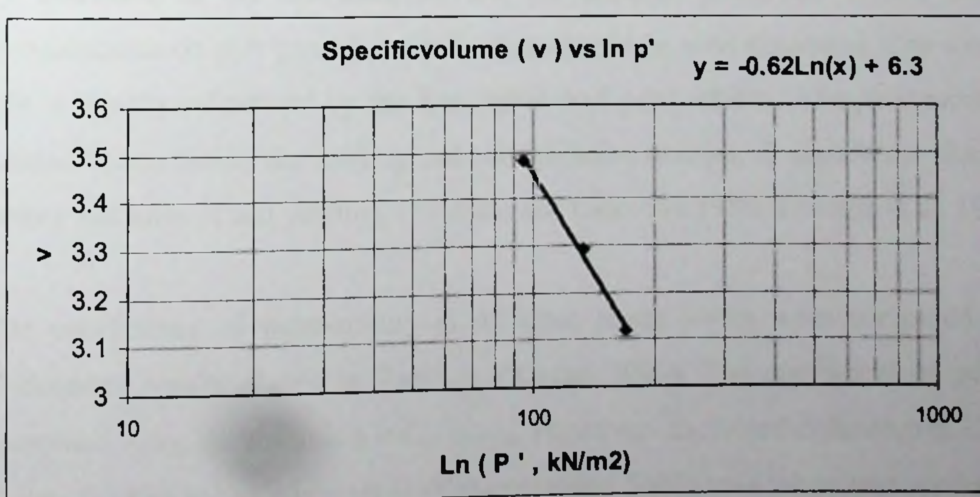


Figure 4.11: -Critical state void ratio

Poisson's ratio (ν') or Shear Modulus (G)

In Cam-Clay model, the elastic strains are assumed to be zero. In finite element formulation, this assumption creates difficulty, as it requires infinite value of shear modulus (G) to make elastic strains to be zero. CRISP avoids this difficulty by allowing some realistic shear strains inside the yield locus to be calculated. It is user's choice to define either ν' or G to account for such shear strains. For the actual soil behaviour, ν'and G values varies with the stress levels. CRISP allows the user to specify either constant value of ν' or G. If ν' is specified, then G values are also allowed to vary accordingly as G is calculated from K the bulk modulus after each load increment (tangent stiffness method). Therefore, it is usually convenient to specify ν' which means that G varies with stress level in the same way as K.

In this study ν' = 0.3 was used and it was calculated from the K_0 values ($k_0 = 1 - \sin \phi'$);

Where

$$\nu' = \left(\frac{K_0}{1+K_0}\right)$$

Permeability (k_x, k_y)

As discussed in the Literature Review for accurate prediction of the behaviour of embankments on soft ground, consolidation should be well simulated. The consolidation rate is mainly influenced by the foundation soil permeability. The permeability of soft ground varies during the loading and consolidation process. A significant change occurs before and after of soil yielding. (Tavenas and Leeroueil 1980; Tavenas et al. 1983).

The coefficients of permeability at different stress levels were computed using the Oedometer results shown in Table 4.1(Kugan 2003). The coefficient of permeability computed using the formula $k = C_v . m_v . \gamma_w$. However the noted difference in C_v field and C_v lab for SriLankan peaty clays (Karunawardna 2002) was taken into account and the Oedometer results were multiplied by a factor of 2.5 This multiplication factor was further confirmed by the back analysis of large scale consolidation performed in University of Moratuwa (Kugan 2003).

Large-scale consolidation test

The large-scale model was set up with remoulded peaty clay deposited in an open barrel of diameter 560mm and height 860mm. Arrangements to read the settlement and pore pressure were made as shown in Figure 4.12. In the large-scale model test, a load of 30kN/m^2 (in step of 5kN/m^2) was applied and the settlement was monitored by maintaining the load for a long period (about 75 days). Load incrementing was done at two days time interval in step of 5kN/m^2.

Simulation of large-scale consolidation model

Figure 4.13 shows the finite element mesh used to represent the large-scale model in CRISP. The above mesh has axisymmetric triangular elements with fixed supports at the bottom to simulate the drum bottom and roller supports at edges to simulate the drum side surface. The drainage was assumed to be only in the top surface. The same loading processes were simulated by applying surface loads and settlement vs. time curves were obtained by varying permeability values. With the CRISP it was possible to vary it only for two times. For the stress level 0-10 kN/m^2 a permeability value of 4.06E-08 ms^{-1} was used, for stress level 10-30 kN/m^2 a permeability value of 9.08E-09 ms^{-1} was used.

Figure 4.12 Large scale setup

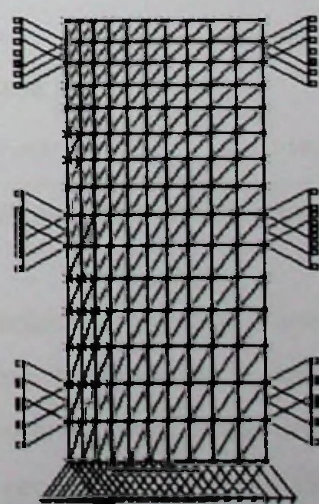


Figure 4.13 Finite element mesh of large scale

Results

The results obtained through the finite element modelling of the Kurgan's consolidation test, were compared with the observation made by Kugan (2003). The comparison of observed settlements in the large-scale model and Finite element prediction is given in Figure 4.14.

Stress range (kN/m^2)	0-20	20-40	40-80	80-160
Cv (m^2/yr)	5.02	2.6	3.51	4.11
Permeability (m/s)*E-09	16.24	3.63	5.12	2.66

Table 4.1: -Permeability with stress range

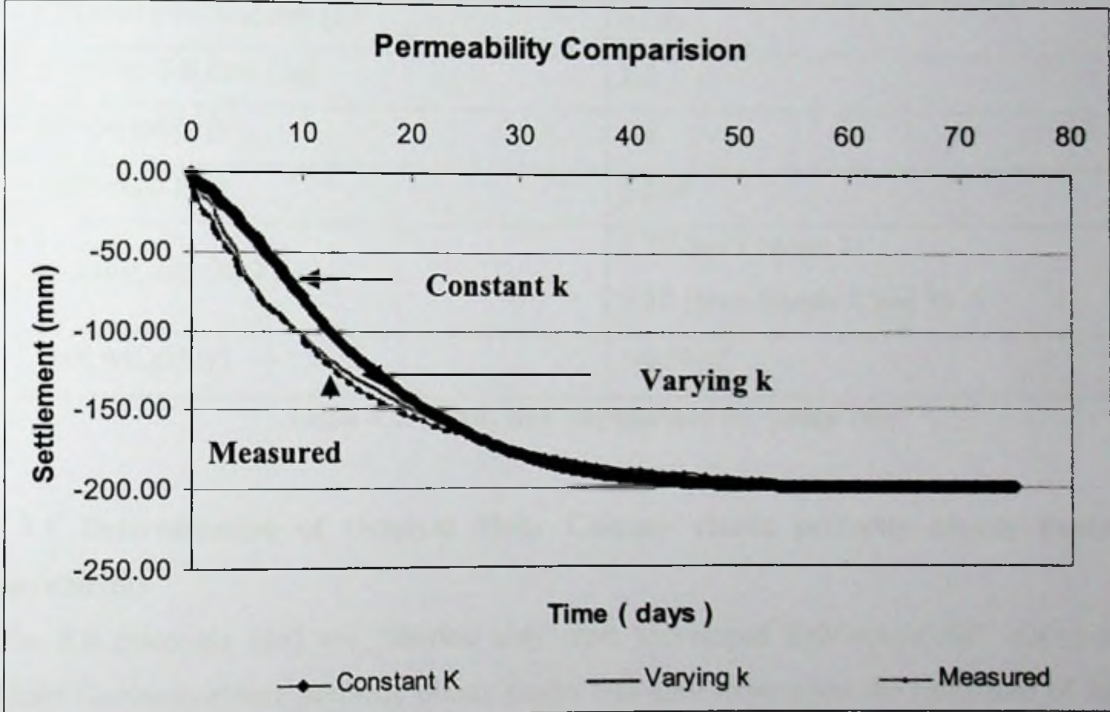


Figure 4.14 Comparison of observed settlement and settlements computed with CRISP.

It is clear from the Figure 4.14 when average constant permeability is used measured settlement is higher at the initial stages. But when permeability is varied both the measured and the observed values were in close agreement. Thus, it was decided to vary the permeability. But there is no facility to vary the permeability in the CRISP. Thus, the only way it could be achieved was by varying material zone. But one material zone can be changed only once. So in the analysis it could change the permeability once and the permeability values were selected as discussed below.

For the embankment construction stage 1 the permeability value was selected to be 2.5 times the lab permeability corresponding to the stress range 0 kN/m^2-40kN/m^2. For the subsequent stages permeability value was selected to be 2.5 times the lab permeability corresponding to the stress range 40 kN/m^2-160 kN/m^2 as given in Table 4.1.The parameters for the Modified Cam clay model were obtained on typical peaty clay from Colombo Katunayake expressway are listed in Table 4.2.

Slope of N.C line (λ)	0.603
Slope of swelling line (k)	0.083
Slope of C.S line (M)	1.2
Poison ratio (ν)	0.3
Void ratio (e_{cs})	5.264
Permeability (k_x=k_y)	1.2E-8m/s (stage 1) 6.5E-9m/s (stages 2 and 3)
Unit weight (γ)	14kN/m^3

Table 4.2: -Cam clay parameters for peaty clay

4.7.6 Determination of Original Mohr Column elastic perfectly plastic model parameters

The fill materials used are "lateritic clay" and "developed lightweight fill". Original Mohr Coulomb elastic perfectly plastic model was used to simulate the behaviour of fill materials. The parameters required for Original Mohr Column elastic perfectly plastic model are; young's modulus (E'), Poison ratio (ν'), Cohesion (C'), Friction angle (ϕ'), Permeability (k) and bulk density (γ).

In this study, model parameters for the dense sand were derived based on SPT data. For the developed lightweight fill material types and lateritic fill, properties were determined from the respective laboratory tests results, which were presented in chapter 3. Young's modulus was obtained as shown in Figure 4.15. Cohesion and Friction angle were obtained from the respective Mohr circles. The poison's ratios for all the materials were assumed empirically. Permeability was obtained from and the following relationship,

$$k = C_v * m_v * \gamma_w$$

Bulk densities were obtained from the respective Proctor curves, and the model parameters for the Original Mohr Column elastic perfectly plastic model are shown in Table 4.2

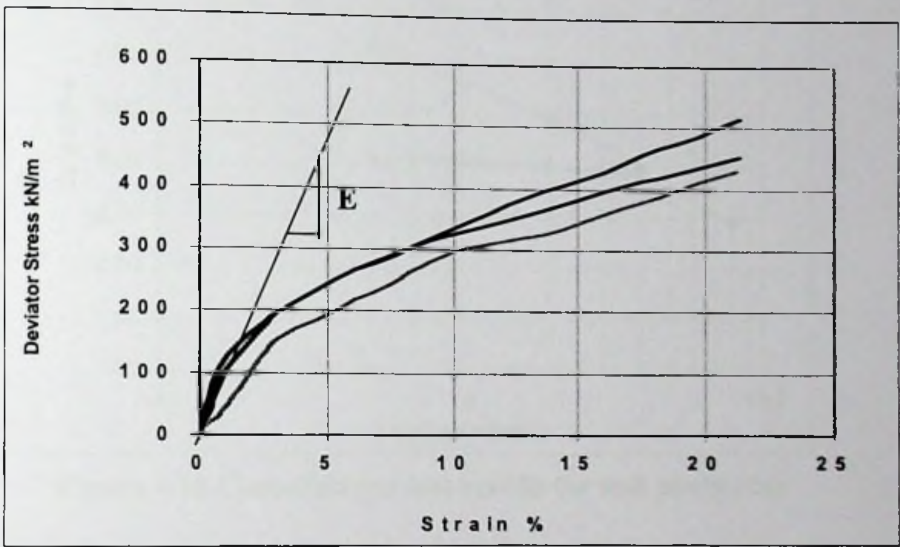


Figure 4.15 Detail of determination of young's modulus

Material	Dense sand	Lateritic soil	Tyre soil mix	Saw dust soil mix	Paddy husk soil mix
E' (kN/m^2)	70, 000	10,000	10 000	7500	5000
C' (kN/m^2)	0	40	10	10	12.5
ϕ'	30	33	37	35	35
ν	0.3	0.3	0.3	0.3	0.3
K_x (ms^{-1})	-	2.75E-10	1E-8	1E-8	1E-8
K_y(ms^{-1})	-	2.75E-10	1E-8	1E-8	1E-8
γ (kN/m^3)	20	20.5	14.0	8.5	10.5

Table 4.3: -Elastic perfectly plastic model parameters for fill materials and dense sand.

4.7.7 Insitu stresses

When using the critical state model constitutive relation, the maximum consolidation stress and initial stresses have a very significant importance. The maximum consolidation pressure was obtained using the standard consolidation test data.

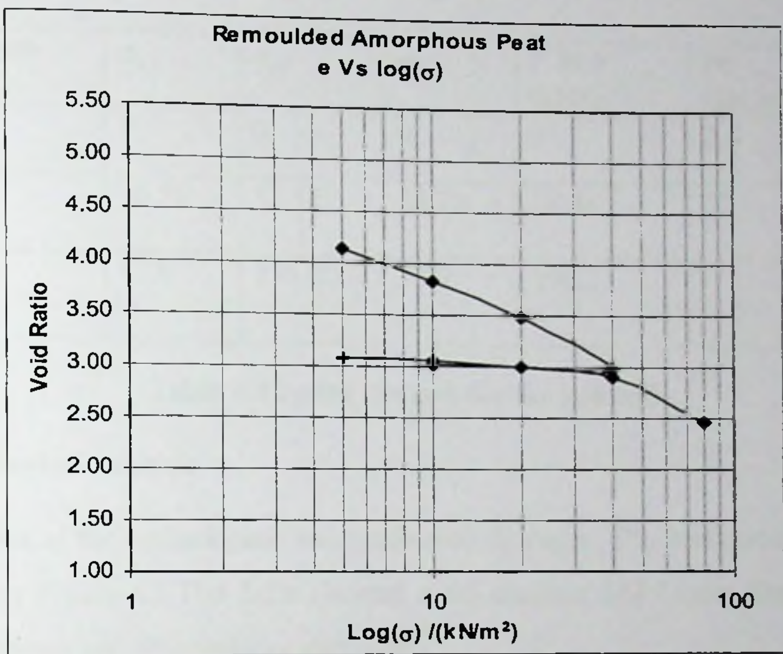


Figure 4.16 Consolidation test results for soft peaty clay

The e vs log σ plot indicated that the soil is normally consolidated thus the maximum preconsolidation pressure was taken to be the same as the effective vertical overburden stress at the point. Although vertical stress is zero at the top a non-zero small value of p'c was given at the top to avoid numerical difficulties.

$$P'c = \left(\frac{\left[\frac{q}{M} \right]^2}{p'_{max}} \right) + p'_{max}$$

The Insitu vertical stresses and horizontal stresses are given by ;

$$\sigma_{yy} = (\gamma - \gamma_w)^* h$$

$$\sigma_{xx} = k_0 * \sigma_{yy}$$

Since the soil is normally consolidated k_0 is found using Worth's method. Which is a simplified version of Jaky's (1994) relation

$$k_0 = K_{nc} = 1 - \sin \phi'$$

All the parameters obtained for the Insitu stresses are given in Table 4.4

Depth (m)	σ_{xx}	σ_{yy}	σ_{zz}	P.W.P (kN/m^2)	Pc (kN/m^2)
0	0	0	0	0	2.5
8	16.76	33.52	16.76	78.48	32
20	77.9	155.80	77.9	196.2	-

Table 4.4 Insitu stresses for the sub soil

4.7.8 Method of analysis

The analysis of the embankment was performed in stages. The mesh used in the analysis is shown in Figure 4.3.The finite element mesh consists 144 Linear Strain Quadilateral (LSQ) elements and 50 triangular elements.

The coupled consolidation analysis was performed during loading and consolidation stage. Coupled consolidation analysis calculates total settlement during loading and consolidation stage. It was evident in many embankment analyses the coupled consolidation analysis gives better result than the undrained analysis during loading. (A.S.Balasubramaniam 1992). The CRISP formulation of consolidation is based on Biot's three dimensional consolidation theories. Physical non-linearity is handled by dividing the applied load and time into a number of increments and solving the system of equations using a tangent stiffness approach.

For coupled consolidation analysis, the drainage conditions were imposed at top and bottom surface of the soft soil. The construction of embankment was simulated by applying the embankment elements layer by layer. Construction sequences were closely simulated such that the filling rate was selected to be 0.1m/day.

Embankment stability

In general stability problems embankments are analysed using limit equilibrium methods. The stability problems are rarely analysed using FEM. The major draw back in the finite element approach is that it does not directly provide a factor of safety. However, the subsoil-yielding pattern upon loading, approaching critical state and increasing the rate of settlements, can be used to predict the stability of an embankment during a finite element analysis.

Hunter and Fell (2003) has shown that the vertical displacement at toe and particularly beyond the toe of the embankment is a good indicator of an impending failure condition. A measurable change in rate or direction of vertical displacement is observable in most of the case studies beyond the threshold height and within the range 70 – 95 % of the eventual failure height. For points beyond the toe, negligible vertical deformation were usually observed during the initial period of embankment construction, and failure condition was identifiable by heave movements or large increases in the rate of heave movement with increasing embankment height.

Using the above concepts suitable construction sequences were evaluated for the staged loading analysis. The procedures are discussed in detail in the proceeding sections..

Construction sequences

The peat layer under the embankment is extremely soft with low strength parameter and Cu values in the range at or below 10 kN/m^2. Deciding the initial stage safe height of fill is very important. As Such, several types of analysis were done to determine the possible safe fill thickness. Figure 4.17 presents the settlement behaviour at a point near the toe with the increase of fill height. Figure 4.18 presents the same for a point 5m beyond the toe.

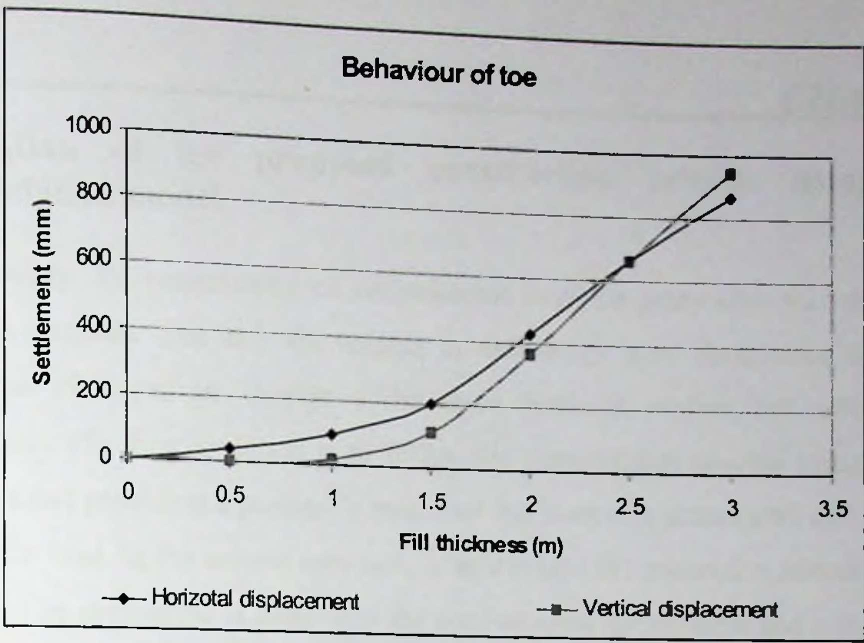


Figure 4.17 Behaviour of toe

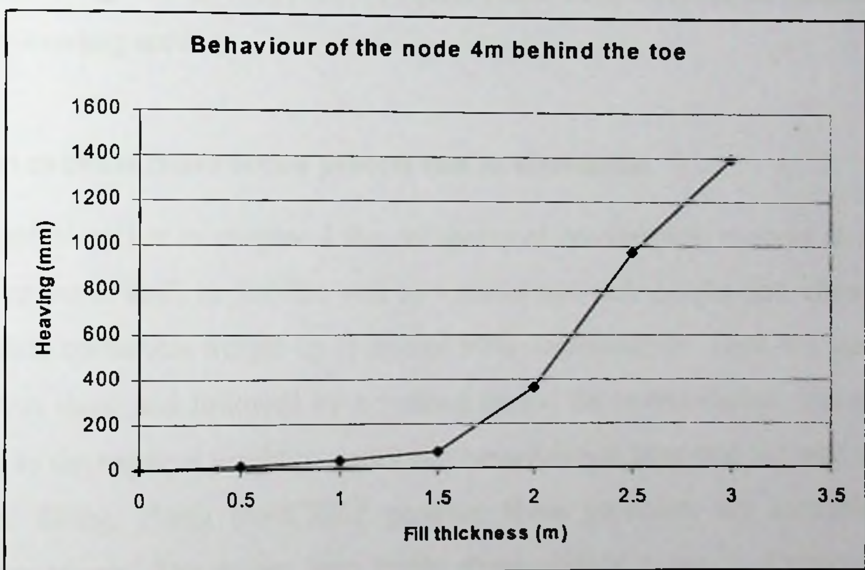


Figure 4.18 Behaviour of the point 5m beyond the toe

It is clear from all two plots that when the fill height increases beyond 1.5m the displacements (settlements) begins to increase at a rapid rate. This implies that the constructed embankment is approaching failure. As such, the safe fill height would be less than 1.5m.It is taken to be 1.0m in this analysis.

Simulation of the proposed construction process using CRISP consolidation model

In this study, the construction of embankment over the peaty clay was done in three different methods. The safe fill heights in each stage were determined based on the procedure discussed in chapter 4.The three methods studies fall into two basic approaches. The first approach is to follow the conventional process using lateritic fill materials and providing a preload to minimise the in-service settlements due to pavement and traffic load. In the second approach, a lightweight fill material is introduced into the process. The preloading is done with the conventional fill material and a part of the fill was removed and replaced with lightweight fill subsequently. A further variation in the use of lightweight fill gave the third method. These three methods are described in detail in the proceeding sections.

5.1 Conventional construction process and its simulation

As described earlier in chapter 4 the conventional construction process is to a fill with normal material such as lateritic soil to a maximum safe height and allow the peat to consolidate under that weight up to around 90% consolidation. Then the second stage of filling was done and followed by a waiting period for consolidation. Construction was done up to the required height in stages and consolidation time was allowed after the each stage of filling. Using the CRISP program these processes are simulated to be in following stages. The stages are; Insitu stage, Filling stage 1, Consolidation stage 1,Filling stage 2, consolidation stage 2,Filling stage 3, consolidation stage 3,filling stage 4, consolidation stage 4,Filling stage 5, Consolidation stage 5, removing surcharge, pavement construction, application of vehicle load and further consolidation

5.1.1 Insitu stage

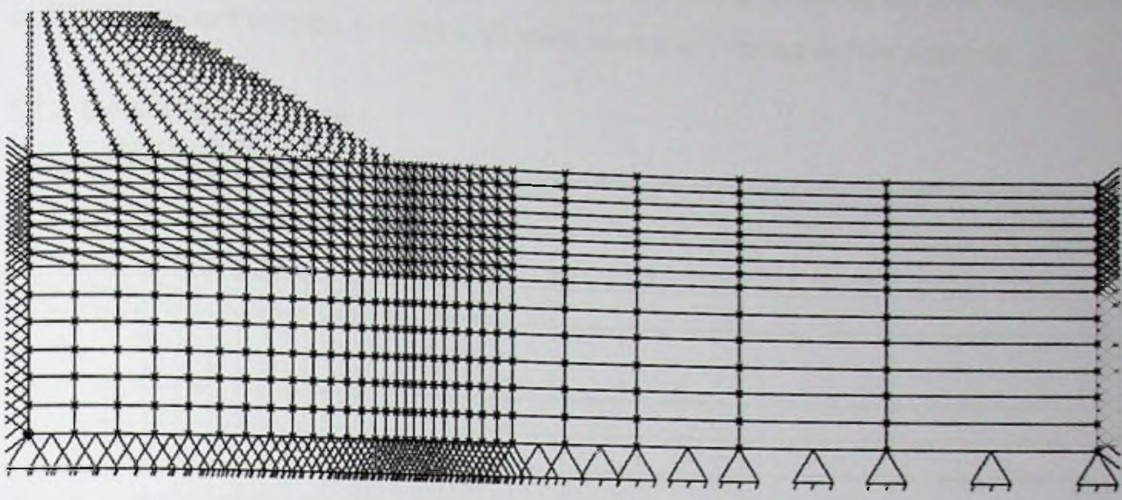


Figure 5.1 Insitu stage mesh.

Figure 5.1 shows the finite element mesh used to represent the 8m-thick peaty clay layer. The above mesh has L.S.Quadrilateral consolidating elements and small enough triangular consolidating elements with fixed supports at the bottom edges to simulate the bedrock and roller supports at side edges to simulate the adjacent soil movement as discussed in chapter 4. Insitu stress parameters were given same as discussed in section 4.4.

5.1.2 Filling Stage 1

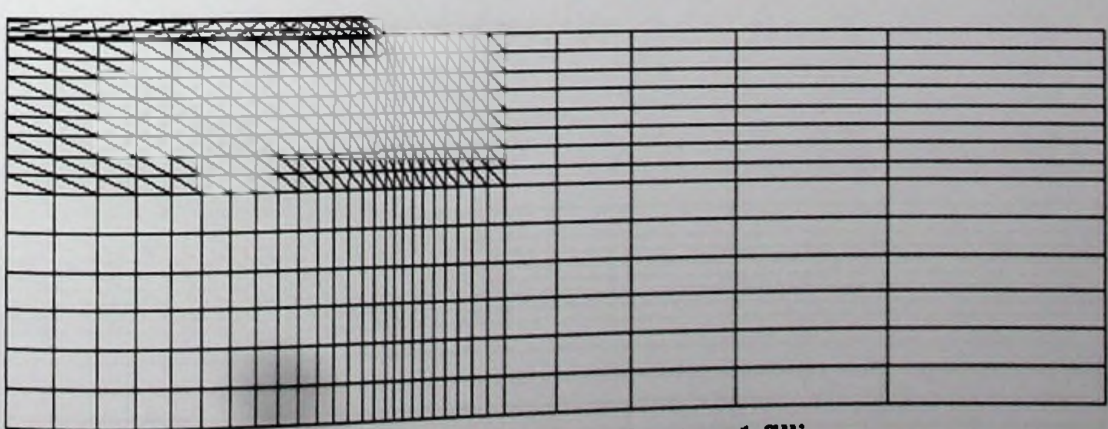


Figure 5.2 Deformed Mesh after the stage 1 filling.

Figure 5.2 shows the outline of the mesh and deformed mesh pattern after first stage of 1m filling by lateritic soil. The filling was simulated by adding 1m thick meshes to the existing insitu mesh. The deformations are corresponding to the immediate settlement and small amount of consolidation settlement taken place during the construction.

5.1.3 Consolidation stage 1

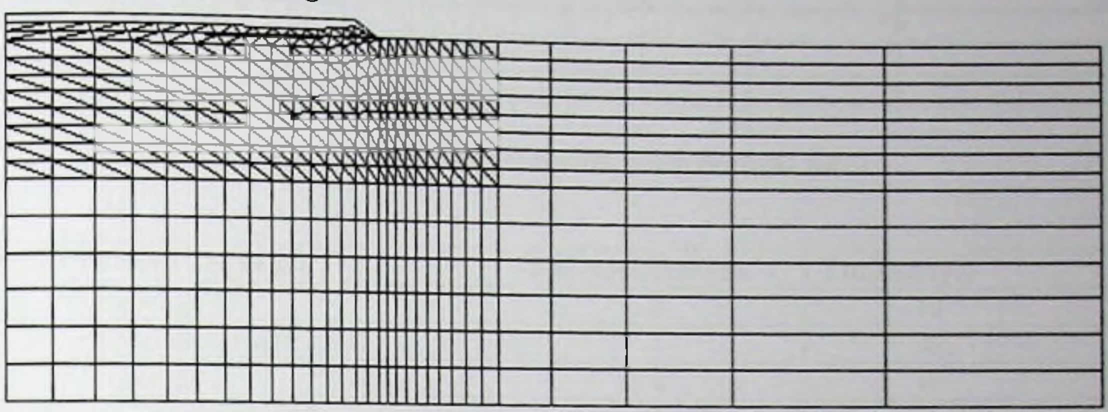


Figure 5.3 Deformed mesh after the stage 1 filling & consolidation

Figure 5.3 shows the deformed mesh pattern after first stage of 1m filling was left for one-year for consolidation. There is no heaving at the toe or beyond the toe. Embankment settlements at this stage do not cause any functional difficulties.

Similarly all the filling stages and consolidation stages are carried out for all the stages. Figure 5.4 and Figure 5.5 shows the final constructed and deformed meshes. Table 5.1 shows the detailed construction stages for the construction of 3m high conventional embankments over a 8m thick layer of peaty clay.

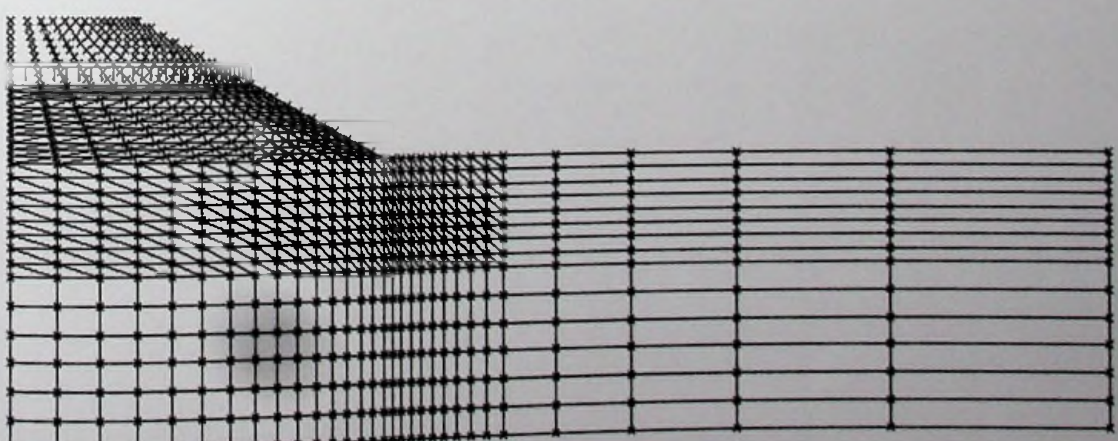


Figure 5.4 Final finite element mesh

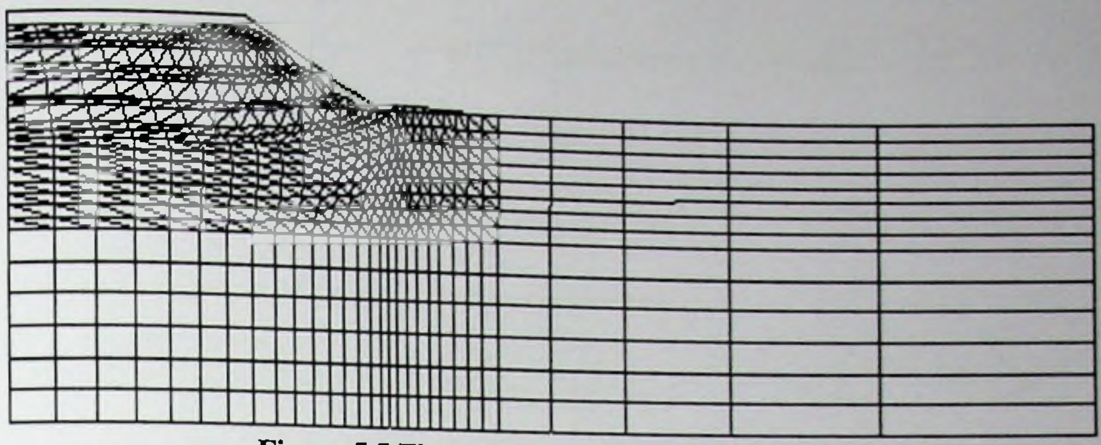


Figure 5.5 Final deformed finite element mesh

Construction Sequence	Fill thickness (m)	Time (days)
(1) Filling stage 1	1.0	10
(2) Consolidation stage 1	-	365
(3) Filling stage 2	1.0	10
(4) Consolidation stage 2	-	365
(5) Filling stage 3	1.5	15
(6) Consolidation stage 3	-	365
(7) Filling stage 4	2.0	20
(8) Consolidation stage 4	-	180
(9) Filling stage 5	1.0	10
(10) Consolidation stage 5	-	180
(11) Removing surcharge	1.5	5
(12) Pavement load	-	90
(13) Traffic load	-	-
(14) After 5 years	-	1825

Table 5.1: - Details of conventional construction techniques for 3m high embankments

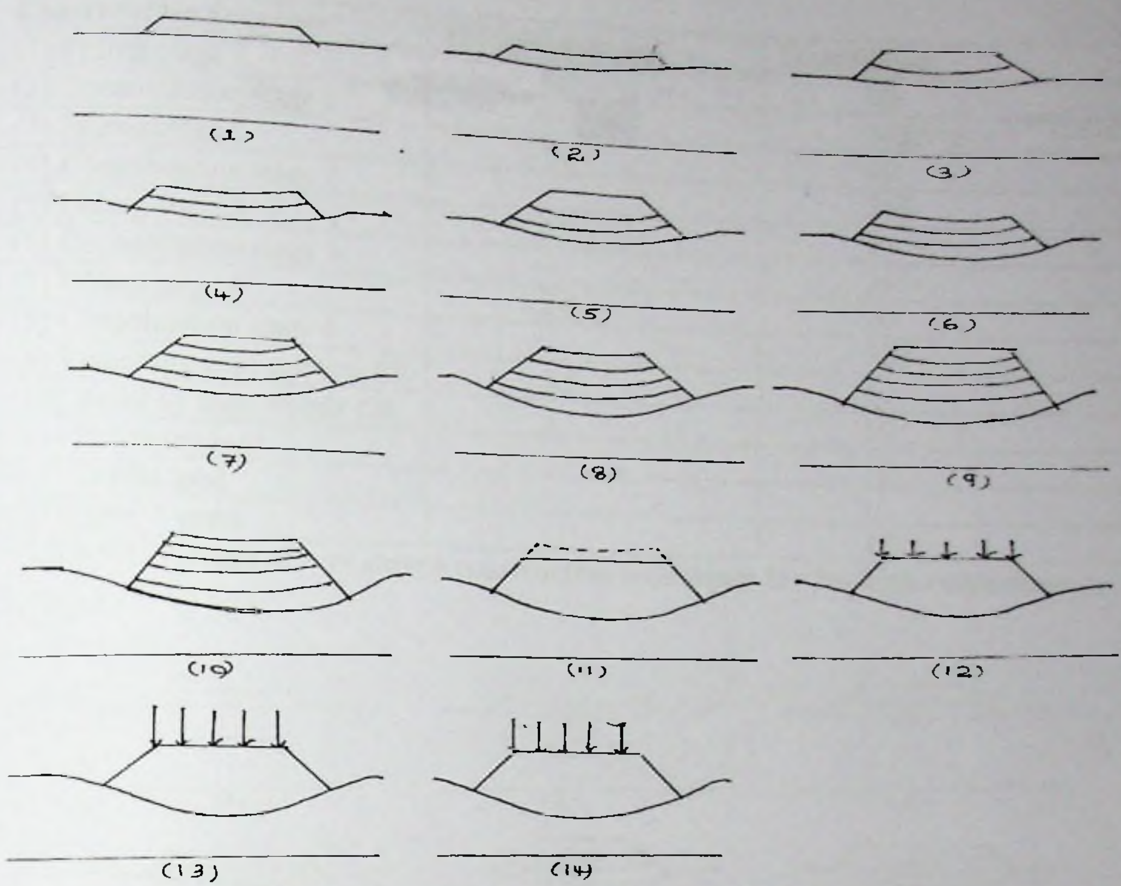


Figure 5.6 Graphical representation of construction of 3m embankment by lateritic soil

5.2 Proposed Construction Process 1, Using Lightweight fills materials.

5.2.1 Using tyre chips: soil mixes

In this proposed construction process, the maximum fill thickness of 5.5m conventional lateritic soils was to be placed in 4 lifts. The 3m lateritic soils above the water table level was removed and the ground was refilled with lightweight fill material to achieve a final embankment height of 3m above the original ground. Numbers of trial analyses were done varying the thickness of the fill placed at different stages. The criterion to satisfy was that the in service settlement has to be less than 50mm.The above mentioned fill layer thicknesses were finally selected since they satisfy the set criterion. The construction details are shown in Table 5.2.

Construction Sequence	Fill thickness (m)	Time (days)
(1) Filling stage 1	1.0	10
(2) Consolidation stage 1	-	365
(3) Filling stage 2	1.0	10
(4) Consolidation stage 2	-	365
(5) Filling stage 3	1.5	15
(6) Consolidation stage 3	-	365
(7) Filling stage 4	2.3	20
(8) Consolidation stage 4	-	360
(9) Removing conventional fill	3.5	10
(10) Refill by light weight fill	3	10
(11) Pavement load	-	90
(12) Traffic load	-	-
(13) After 5 years	-	1825

Table 5.2: - Detail of tyre shreds construction techniques for 3m high embankments

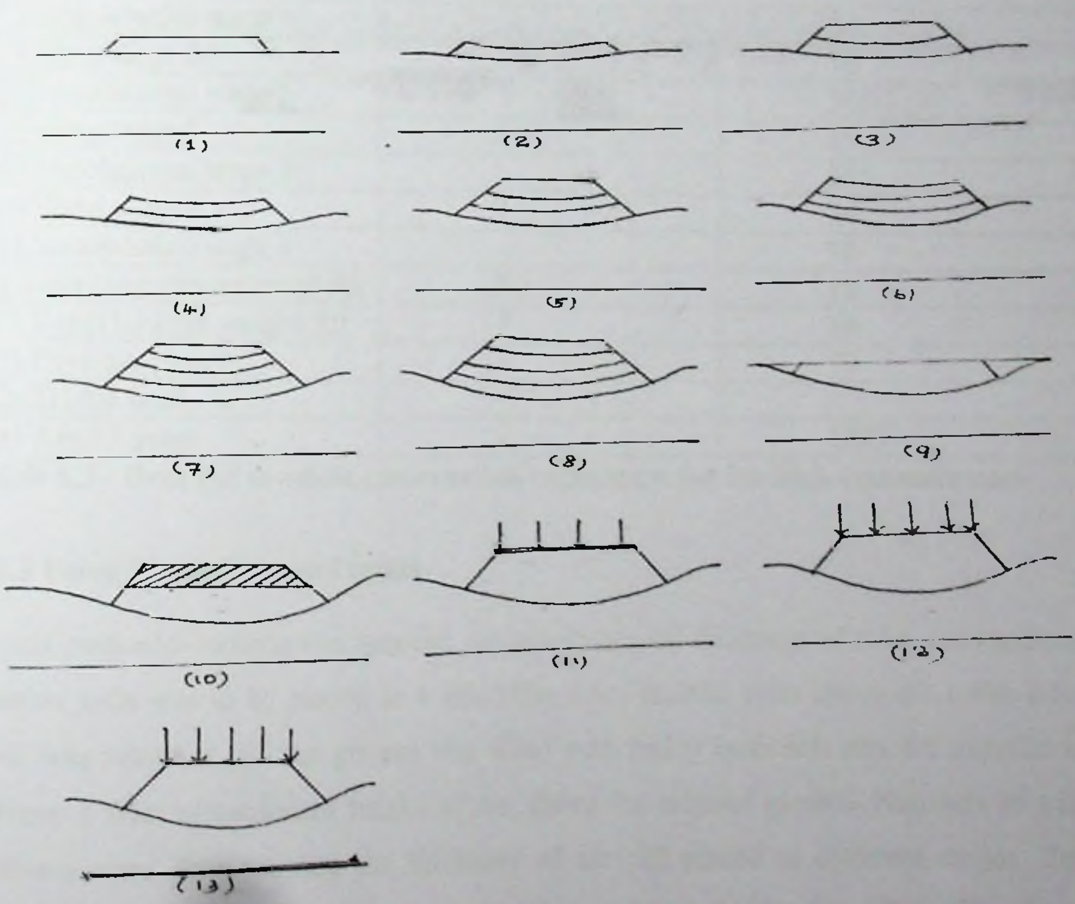


Figure 5.7 Graphical representation of construction of 3m embankment by tyre chips: soil mix-process 1

5.2.2 Using sawdust soil mixes

In this proposed construction process, the maximum fill thickness of 5.5m conventional lateritic soils was placed in 4 lifts. The 3.5m lateritic soils above the water table level was removed and the ground was refilled with 3m sawdust soil mix fill material to achieve a final embankment height of 3m above the original ground. Numbers of trial analyses were done varying the thickness of the fill placed at different stages. The criterion to satisfy was that the in service settlement has to be less than 50mm.The above mentioned fill layer thicknesses were finally selected since they satisfy the set criterion. The construction details are shown in Table 5.3.

Construction Sequence	Fill thickness (m)	Time (days)
(1) Filling stage 1	1.0	10
(2) Consolidation stage 1	-	365
(3) Filling stage 2	1.0	10
(4) Consolidation stage 2	-	365
(5) Filling stage 3	1.5	15
(6) Consolidation stage 3	-	365
(7) Filling stage 4	1.5	20
(8) Consolidation stage 4	-	90
(9) Removing conventional fill	3.0	10
(10) Refill by light weight fill	3	10
(11) Pavement load	-	90
(12) Traffic load	-	-
(13) After 5 years	-	1825

Table 5.3 - Detail of sawdust construction techniques for 3m high embankments

5.2.3 Using Paddy husk soil mixes

In this proposed construction process, the maximum fill thickness of 5.5m conventional lateritic soils was to be placed in 4 lifts. The 3.5m lateritic soils above the water table level was removed and the ground was filled with paddy husk soil mix fill material to achieve a final embankment height of 3m above the original ground. Numbers of trial analyses were done varying the thickness of the fill placed at different stages. The criterion to satisfy was that the in service settlement has to be less than 50mm.The above mentioned fill layer thicknesses were finally selected since they satisfy the set criterion. The construction details are shown in Table5.4

Construction Sequence	Fill thickness (m)	Time (days)
(1) Filling stage 1	1.0	10
(2) Consolidation stage 1	-	365
(3) Filling stage 2	1.0	10
(4) Consolidation stage 2	-	365
(5) Filling stage 3	1.5	15
(6) Consolidation stage 3	-	365
(7) Filling stage 4	2.1	20
(8) Consolidation stage 4	-	90
(9) Removing conventional fill	3.5	10
(10) Refill by light weight fill	3	10
(11) Pavement load	-	90
(12) Traffic load	-	-
(13) After 5 years	-	1825

Table 5.4: - Detail of paddy husk construction techniques of 3m high embankments

5.3 Proposed Construction Process 2

The peaty clay considered in the analysis was a normally consolidated soil. As discussed earlier, in initial stages only 1m thick lateritic fill could be placed without any shear failure. As such to construct 3-5m high embankment 4-5 years long construction periods are required. This is fairly long. In order to reduce the construction period another type of construction process selected. In this construction process instead of initial lateritic fill, proposed lightweight fill is used in all the stages. Lateritic fill is used only for surcharging. In this analysis, saturated densities were considered for the fills, which go under the water table.

5.3.1 Using tyre soil mixes

In this proposed construction process, the maximum fill thickness of 5m-tyre soil lightweight mix was to be placed in 3 lifts and one-year consolidation period was to be followed by each lift. Another 1m lateritic soil was placed as a surcharge to minimize the further service settlement due to pavement and traffic loads. Embankment height of 3m was achieved by this construction process in 4.0 years time.

Construction Sequence	Fill thickness (m)	Time (days)
(1) Filling stage 1	1.5	10
(2) Consolidation stage 1	-	365
(3) Filling stage 2	1.5	10
(4) Consolidation stage 2	-	365
(5) Filling stage 3	2.0	15
(6) Consolidation stage 3	-	365
(7) Filling stage 4 (By lateritic soil)	1.0	10
(8)Consolidation stage 4	-	240
(9) Removing conventional fill	1.0	10
(10) Pavement load	-	90
(11) Traffic load	-	-
(12) After 5 years	-	1825

Table 5.5: - Detail of tyre soil mix construction techniques for 3m high embankments-Construction process 2

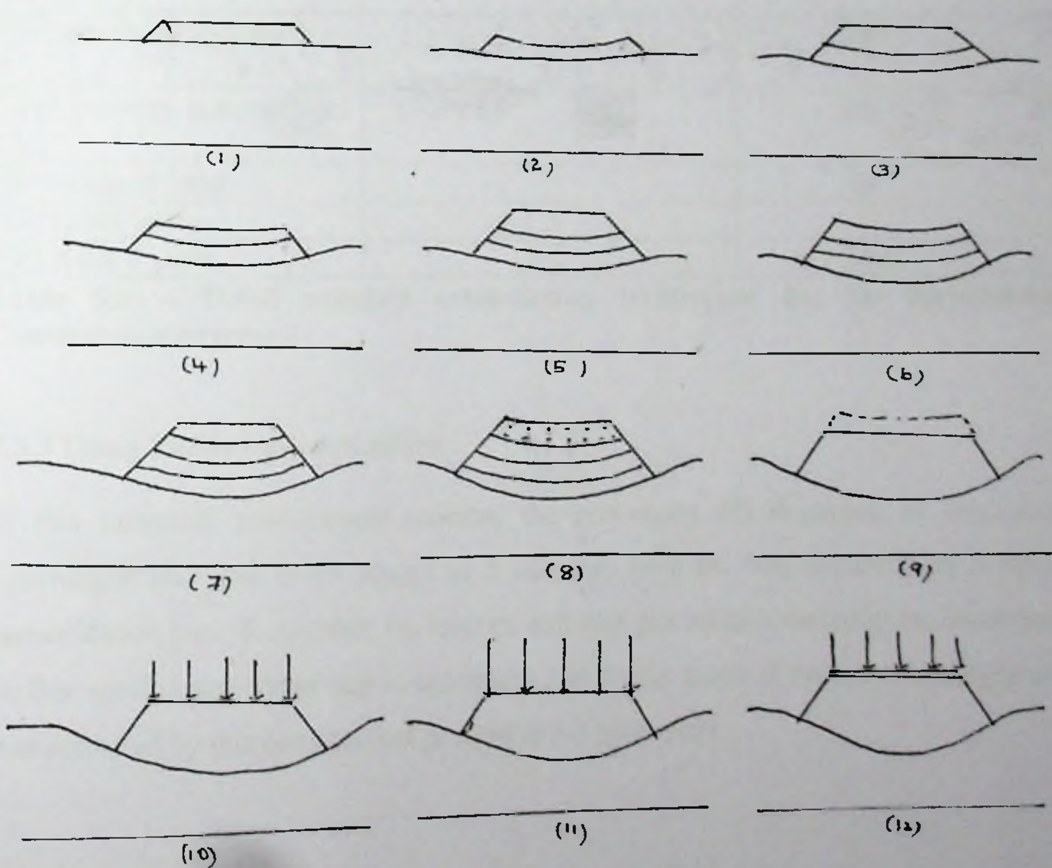


Figure 5.8 Graphical representation of construction of 3m embankment by tyre chips: soil mix-process 2

5.3.2 Using Saw dust soil mixes

In this proposed construction process, the maximum fill thickness of 5m-tyre soil lightweight mix was to be placed in 3 lifts and each lift was followed by a one-year consolidation period. Another 1m lateritic soil was placed as a surcharge to minimize the further service settlement due to pavement and traffic loads. Embankment height of 3m was achieved by this construction process in 3.0 years time.

Construction Sequence	Fill thickness (m)	Time (days)
(1) Filling stage 1	2.0	10
(2) Consolidation stage 1	-	365
(3) Filling stage 2	2.0	10
(4) Consolidation stage 2	-	365
(5) Filling stage 3 By lateritic soil	0.8 1.0	15
(6) Consolidation stage 3	-	240
(7) Removing conventional fill	1.0	10
(8) Pavement load	-	90
(9) Traffic load	-	-
(10) After 5 years	-	1825

Table 5.6: - Detail sawdust construction techniques for 3m embankments-Construction process 2

5.3.3 Using Paddy husk soil mixes

In this proposed construction process, the maximum fill thickness of 5m-tyre soil lightweight mix was to be placed in 3 lifts and each lift was followed by a one-year consolidation period. Another 1m lateritic soil was placed as a surcharge to minimize the further service settlement due to pavement and traffic loads. Embankment height of 3m was achieved by this construction process in 3.3 years time.

Construction Sequence	Fill thickness (m)	Time (days)
(1) Filling stage 1	1.5	10
(2) Consolidation stage 1	-	365
(3) Filling stage 2	2.0	10
(4) Consolidation stage 2	-	365
(5) Filling stage 3	1.4	15
(6) Consolidation stage 3	-	180
(7) Filling stage 4 By lateritic soil	1.0	10
(8) Consolidation stage 4	-	180
(9) Removing conventional fill	1.0	10
(10) Pavement load	-	90
(11) Traffic load	-	-
(12) After 5 years	-	1825

Table 5.7: - Detail of paddy husk construction techniques of 3m high embankments- Construction process 2

ANALYSIS AND RESULTS

6.1 Construction process 1

The effectiveness of the proposed construction process 1 could be illustrated by comparing with the settlement vs. time behaviour in the case of using conventional lateritic fill. Figure 6.1 compares the; settlement behaviour, fill thickness and construction period for the proposed process and the conventional process. Analyses were done with different types of lightweight fill material to show their effectiveness and the results obtained are presented in Table 6.1, Table 6.2 and Table 6.3.

As expected, Figure 6.1 illustrates that the use of lightweight fill materials reduces the total settlement and the construction periods. As such, the amount of soil needed for filling will be reduced and the project could be completed within shorter period. It is very clear from the graphs that as the density of the material is reduced the process will be more effective.

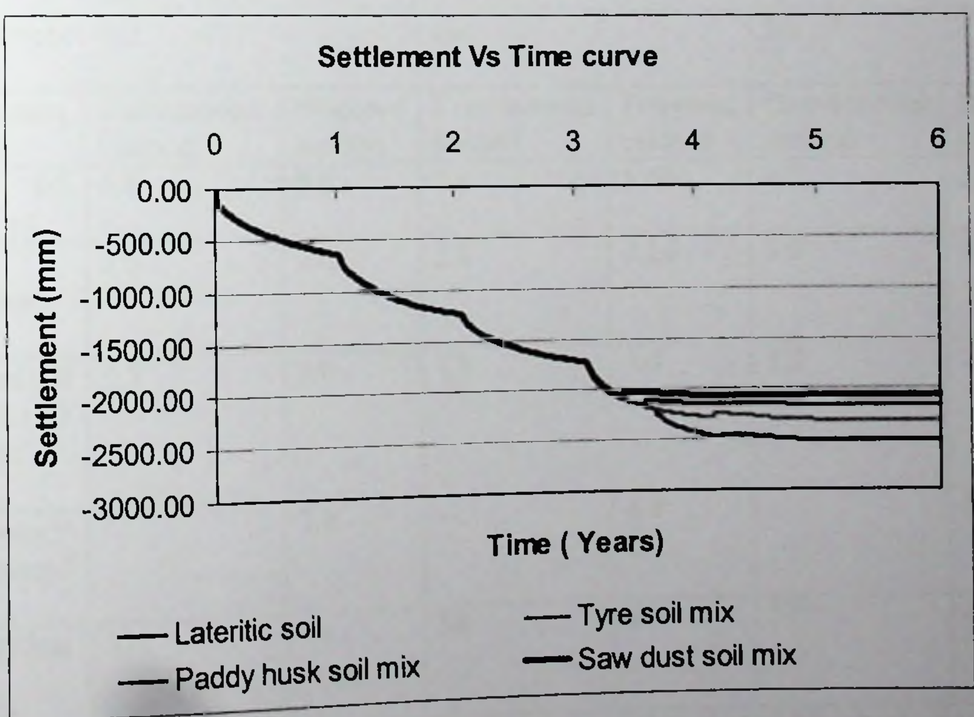


Figure 6.1 Construction results of 3m embankments over 8m depth peaty clay- Construction process 1.

Embankment height	3m		4m		5m	
Construction methods	Conventional method	Proposed method	Conventional method	Proposed method	Conventional method	Proposed method
Total fill placed (m)	6.5	**5.8**	7.8	**6.5**	9.0	**7.5**
Total settlement (m)	2.5	**2.3**	2.8	**2.5**	3.0	**2.5**
Height of fill removed (m) (Lateritic fill)	1.5	**3.5**	1.0	**4.0**	1.0	**5.0**
Light weight fill placed (m)	-	**3.0**	-	**4.0**	-	**5.0**
Construction period (months)	53	**53**	58	**53**	58	**56**

Table 6.1: -Construction of the embankment with Soil: Tyre chip mixes-Construction process 1

Embankment height	3m		4m		5m	
Construction methods	Conventional method	Proposed method	Conventional method	Proposed method	Conventional method	Proposed method
Total fill placed (m)	6.5	**5.0**	7.8	**5.70**	9.0	**5.75**
Total settlement (m)	2.5	**2.0**	2.8	**2.20**	3.0	**2.25**
Height of fill removed (m) (Lateritic fill)	1.5	**3.0**	1.0	**3.5**	1.0	**3.5**
Light weight fill placed (m)	-	**3.0**	-	**4.0**	-	**5.0**
Construction period (months)	53	**44**	58	**47**	58	**48**

Table 6.2: - Construction of the embankment with Soil: Sawdust mixes-Construction process 1

Embankment height	3m		4m		5m	
Construction methods	Conventional method	Proposed method	Conventional method	Proposed method	Conventional method	Proposed method
Total fill placed (m)	6.5	**5.6**	7.8	**5.9**	9.0	**6.0**
Total settlement (m)	2.5	**2.1**	2.8	**2.4**	3.0	**2.5**
Height of fill removed (m) (Lateritic fill)	1.5	**3.5**	1.0	**3.5**	1.0	**3.5**
Light weight fill placed (m)	-	**3.0**	-	**4.0**	-	**5.0**
Construction period (months)	53	**46**	58	**48**	58	**53**

Table 6.3: - Construction of the embankment with Soil: Paddy husk mixes-Construction process 1

6.2 Construction process 2

Although construction process 1 indicates that the use of lightweight fill materials reduces the total settlement and the construction periods, it is evident from the graphs that total construction period is little bit longer. In the construction process 1 the care was taken to ensure that the proposed lightweight fill materials do not get mixed with the water (submerged) after the construction of the embankment. But in the literature number of studies were reported by many researchers to show that the tyre shreds are not harmless when it is mixed with water. Therefore, another construction process was proposed in chapter 5 to ensure that the use of lightweight fill reduces the construction period significantly.

The effectiveness of the proposed construction process 2 is illustrated by plotting the settlement vs. time behaviour for the refilling with the proposed lightweight fill material and the process with the conventional lateritic fill. Figure 6.2 compares the settlement behaviour; fill thickness and construction period for the proposed process and the conventional process. The settlement behaviour of polystyrene blocks are used for the construction is also shown in the same figure. Analyses were done with different types of lightweight fill material to show their effectiveness and the results obtained are presented in Table 6.4, Table 6.5 and Table 6.6

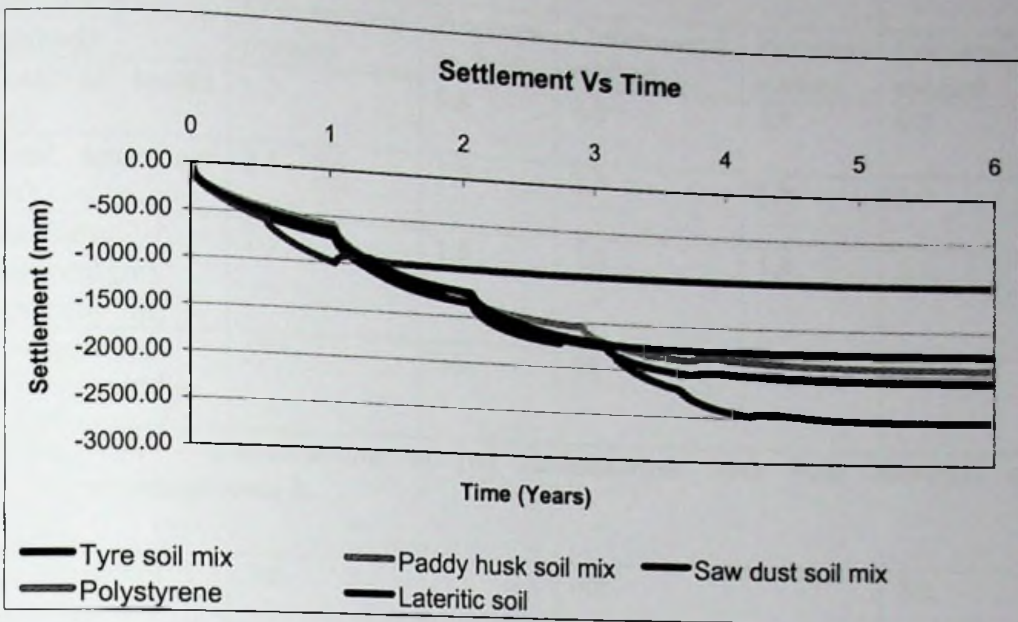


Figure 6.2 Construction results of 3m embankments over 8m depth peaty clay.

As expected, Figure 6.2 illustrates that the use of lightweight fill materials reduces the total settlement and the construction periods significantly. As such, the amount of soil needed for filling will be reduced and the project could be completed within shorter period. It's very clear from the graphs that if the density of the material is reduced the process will be more effective than the construction process 1.

Embankment height	3m		4m		5m	
Construction methods	Conventional method	Proposed method	Conventional method	Proposed method	Conventional method	Proposed method
Total fill height (m)	6.5	**6.0**	7.8	**7.4**	9.0	**8.5**
Total settlement (m)	2.5	**2.0**	2.8	**2.4**	3.0	**2.5**
Surcharge removed (m) (Lateritic fill)	1.5	**1.0**	1.0	**1.0**	1.0	**1.0**
Construction period (months)	53	**48**	58	**50**	58	**55**

Table 6.4: - Construction of the embankment with Soil: Tyre chip mixes- Construction process 2.

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Embankment height	3m		4m		5m	
Construction methods	Conventional method	Proposed method	Conventional method	Proposed method	Conventional method	Proposed method
Total fill height (m)	6.5	**5.8**	7.8	**6.9**	9.0	**8.0**
Total settlement (m)	2.5	**1.8**	2.8	**1.9**	3.0	**2.0**
Surcharge removed (m) (Lateritic fill)	1.5	**1.0**	1.0	**1.0**	1.0	**1.0**
Construction period (months)	53	**36**	58	**39**	58	**39**

Table 6.5: - Construction of the embankment with Soil: Sawdust mixes-Construction process 2.

Embankment height	3m		4m		5m	
Construction methods	Conventional method	Proposed method	Conventional method	Proposed method	Conventional method	Proposed method
Total fill height (m)	6.5	**5.9**	7.8	**7.1**	9.0	**8.5**
Total settlement (m)	2.5	**1.9**	2.8	**2.1**	3.0	**2.5**
Surcharge removed (m) (Lateritic fill)	1.5	**1.0**	1.0	**1.0**	1.0	**1.0**
Construction period (months)	53	**40**	58	**42**	58	**55**

Table 6.6: - Construction of the embankment with Soil: Paddy husk mixes-Construction process 2.

6 .3 Parametric study performed

A, parametric study was done to assess further the effectiveness of the proposed construction process with different types of lightweight fill material developed. Parameters varied are;

- Soft soil thickness and
- Embankment height

The main concerns are the settlement and shear failure of the embankment. As such, the vertical displacement of the center node in the Finite Element mesh was taken as the main index to measure the settlement.

6.3.1 Influence of the embankment height

Depending on the project, the required rise of the embankment could vary. When the embankment height increases the amount of lightweight fill material needed for refilling also increases. Thus the effectiveness of the construction process in view of the reduction of fill thickness and construction period is more important. Table 6.7 compares the result for; lateritic fills (A) Light weight fills tyre soil mix (B), paddy husk soil mix (C) and saw dust soil mix (D) for embankment constructed over 8m deep soft soil layers if the construction process 1 is followed during the construction.

Embankment height	3m				4m				5m			
	A	B	C	D	A	B	C	D	A	B	C	D
Total Fill thickness (m)	6.5	5.8	5.6	5.0	7.8	6.5	5.9	5.75	9.0	7.6	6.0	5.75
Total settlement (m)	2.5	2.3	2.1	2.0	2.8	2.5	2.4	2.25	3.0	2.6	2.5	2.25
Construction Period (months)	53	53	46	44	58	52	48	47	58	58	53	47

Table 6.7: - Effectiveness of the process 1 with the variation of embankment thickness over 8m thickness soft layer.

Similarly Table 6.8 compares the result for; lateritic fills (A) Light weight fills tyre soil mix (B), paddy husk soil mix (C) and saw dust soil mix (D) for embankment constructed over 8m deep soft soil layers for the construction process 2.

Embankment height	3m				4m				5m			
	A	B	C	D	A	B	C	D	A	B	C	D
Total Fill thickness (m)	6.5	6.0	5.9	5.8	7.8	7.4	7.1	6.9	9.0	8.5	8.25	8.0
Total settlement (m)	2.5	2.0	1.9	1.8	2.8	2.4	2.1	1.9	3	2.5	2.25	2.0
Construction Period (months)	53	48	40	36	58	50	42	39	58	55	51	39

Table 6.8: - Effectiveness of the process 2 with the variation of embankment thickness over 8m thickness soft layer.

6.3.2 Influence of the thickness of peaty clay layer

The effectiveness of the proposed construction process 1 could be further illustrated by increasing the thickness of the peaty clay. Table 6.9, Table 6.10 and Table 6.11 compare the effectiveness of the lightweight fill for the 3m, 4m and 5m embankments over soft peaty clay layers of thicknesses 8m and 10m.

Thickness of peaty clay (m)	Lateritic soil		Tyre mix		Paddy husk mix		Saw dust mix	
	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth
Total Fill thickness (m)	6.5	7.0	5.8	6.7	5.6	6.1	5.0	5.5
Total settlement (m)	2.5	3.0	2.3	2.8	2.1	2.6	2.0	2.5
Construction Period (month)	53	67	53	63	46	57	44	57

Table 6.9: - Effectiveness of the process with variation of the thickness of the peaty clay layer for 3m embankments - Construction process 1.

Thickness of peaty clay (m)	Lateritic soil		Tyre mix		Paddy husk mix		Saw dust mix	
	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth
Total Fill thickness (m)	7.8	8.5	6.5	7.5	5.9	6.8	5.75	6.1
Total settlement (m)	2.8	3.5	2.5	3.0	2.4	2.8	2.25	2.6
Construction Period (month)	58	76	52	74	48	66	47	58

Table 6.10: - Effectiveness of the process with variation of the thickness of the peaty clay layer for 4m embankment - Construction process 1.

Thickness of peaty clay (m)	Lateritic soil		Tyre mix		Paddy husk mix		Saw dust mix	
	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth
Total Fill thickness (m)	9.0	9.7	7.6	8.2	6.0	7.0	5.75	6.3
Total settlement (m)	3.0	3.7	2.6	3.2	2.5	3.0	2.25	2.8
Construction Period (month)	58	79	58	78	53	66	47	62

Table 6.11: - Effectiveness of the process with variation of the thickness of the peaty clay layer for 5m embankment - Construction process 1.

Similarly, the effectiveness of the proposed construction process 2 could be further illustrated by increasing the thickness of the peaty clay. Table 6.12, Table 6.13 and Table 6.14compare the effectiveness of the lightweight fill for the 3m 4m and 5m embankments over soft peaty clay layers of thicknesses 8m and 10m.

Thickness of peaty clay (m)	Lateritic soil		Tyre mix		Paddy husk mix		Saw dust mix	
	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth
Total Fill thickness (m)	6.5	7.0	6.0	6.9	5.9	6.6	5.8	6.4
Total settlement (m)	2.5	3.0	2.0	2.9	1.9	2.6	1.8	2.4
Construction Period (month)	53	67	48	61	40	52	36	50

Table 6.12: - Effectiveness of the process with the variation of the thickness of the peaty clay layer for 3m embankments - Construction process 2.

Thickness of peaty clay (m)	Lateritic soil		Tyre mix		Paddy husk mix		Saw dust mix	
	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth
Total Fill thickness (m)	7.8	8.5	7.4	8.25	7.1	7.8	6.9	7.5
Total settlement (m)	2.8	3.5	2.4	3.25	2.1	2.8	1.9	2.5
Construction Period (month)	58	76	50	68	42	57	39	52

Table 6.13: - Effectiveness of the process with variation of the thickness of the peaty clay layer for 4m embankment - Construction process 2.

Thickness of peaty clay (m)	Lateritic soil		Tyre mix		Paddy husk mix		Saw dust mix	
	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth	8m Depth	10m Depth
Total Fill thickness (m)	9.0	9.7	8.5	9.4	8.25	9.0	8.0	8.6
Total settlement (m)	3.0	3.7	2.5	3.4	2.25	3.0	2.0	2.6
Construction Period (month)	58	79	55	72	51	63	39	53

Table 6.14: - Effectiveness of the process with variation of the thickness of the peaty clay layer for 5m embankment - Construction process 2.

6.4 Analysis with assignment a preconsolidation pressure.

Both the proposed construction processes analysed above showed that a few years time is required to complete the project. The soft soil considered in the analysis presented in the preceding sections were taken to be normally consolidated soil. The preconsolidation pressure at any point is taken to be as same as the effective overburden pressure. However at some sites the results of the consolidation tests conducted on undisturbed samples indicated that a small Preconsolidation pressure exists in the peaty clay. (Or the peaty clays are slightly over consolidated). As such, another set of analyses were done with the construction process 1 assuming the preconsolidation pressure at the ground surface is $10kN/m^2$.

The effectiveness of the proposed construction process 1 for this case is illustrated by plotting together the settlement vs. time behaviour for the refilling with the proposed lightweight fill material and the process with the conventional lateritic fill in Figure 6.3.This should be compared with Figure 6.1.The construction period corresponding to all different construction approaches are significantly reduced as compared to results presented in Figure 6.2.

Results are presented for the use of different type of lightweight fill materials. The advantages of the use of lightweight fill material in the reduction of construction time and overall settlement is clearly evident. The use of lightweight fill materials reduces the total settlement and the construction periods. As such, the amount of soil needed for filling will be reduced and the project could be completed within shorter period. It is very clear from the graphs that if the density of the material is reduced the process will be more effective. Analyses were done with different types of lightweight fill material varying the embankment height and the peat layer thickness to highlight the effectiveness of the proposed method. The results obtained are presented in Tables 6.15, Table 6.16 and Table 6.17.

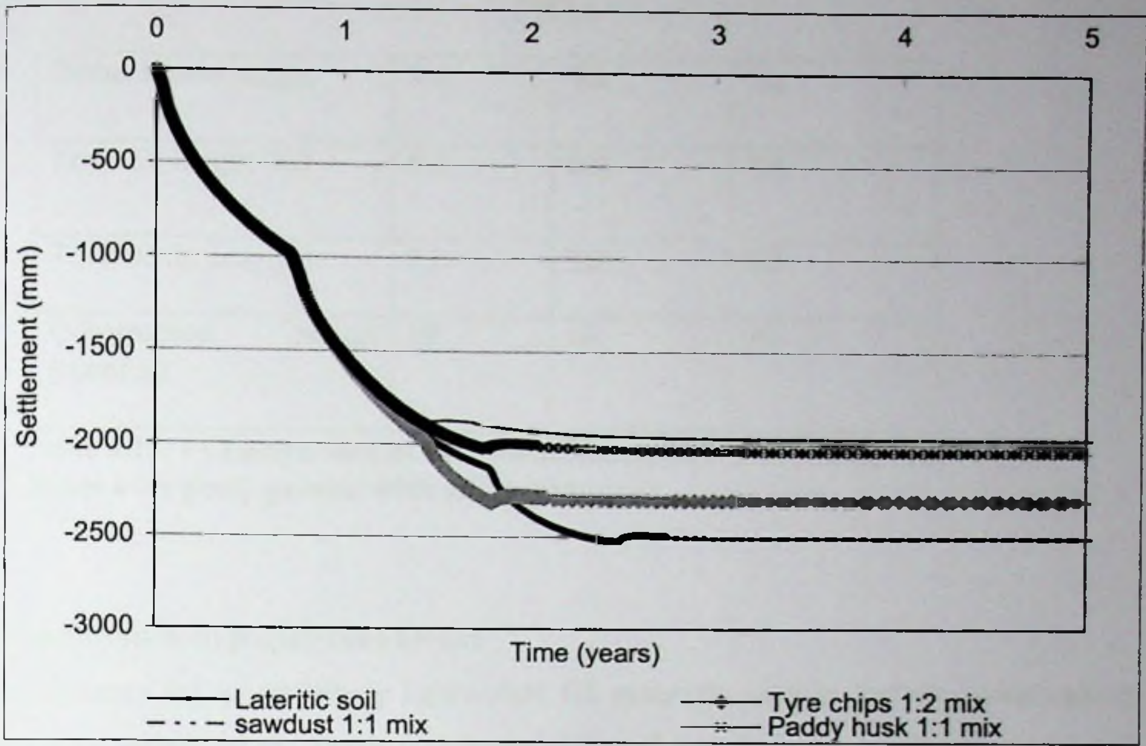

Figure 6.3: -Construction result of 3m embankment over 8m depth peaty clay.

Embankment height	3m	4m	5m
Total fill height (m)	5.8	6.4	7.6
Total settlement (m)	2.3	2.4	2.6
Construction period (months)	28	20	20

Table 6.15: - Construction of the embankment with Soil: Tyre chip mixes over peaty ground with stiff crust.

Embankment height	3m	4m	5m
Total fill height (m)	5	6.2	7.25
Total settlement (m)	2	2.2	2.25
Construction period (months)	16	20	20

Table 6.16: - Construction of the embankment with Soil: Sawdust mixes over peaty ground with stiff crust.

Embankment height	3m	4m	5m
Total fill height (m)	5.1	6.2	7.5
Total settlement (m)	2.1	2.2	2.5
Construction period (months)	19	20	20

Table 6.17: - Construction of the embankment with Soil: Paddy husk mixes over peaty ground with stiff crust.

6.5 Analysis with polystyrene blocks

Polystyrenes are an extremely lightweight fill materials used in highway construction. The unit weight of the block is 15kg/m^3-25kg/m^3 and the average elastic modulus is 2500kg/m^3. The polystyrenes blocks were taken as an elastic material and used in the proposed construction process over 8m-deep soft peaty clay layer. The settlement vs. time graphs shown in construction process 2 in Figure 6.2 illustrates that the polystyrene blocks reduces the construction period from 5 years to one year. The effectiveness of the use of polystyrene blocks are highlighted in Table 6.18.This table should be compared with Table 6.4, Table 6.5 and Table 6.6 to clearly understand the advantages of using this extremely lightweight material.

Embankment height	3m	4m	5m
Total fill height (m)	4	5.1	6.2
Total settlement (m)	1	1.1	1.2
Construction period (months)	12	12	12

Table 6.18: - Construction of the embankment with polystyrene blocks

6.6 Concluding comments

In this study it is proposed to incorporate lightweight fill materials with the preloading techniques to reduce the in-service settlement and construction period of the road embankment. Weights of the proposed lightweight fill materials are much lower than the conventional lateritic fill and they impose less load on the soft soil. Since proposed lightweight fill materials impose lesser loads, settlement of soft soil is less and the factor of safety against shear failure would be increased. As such, a greater thickness of lightweight fill could be placed in every construction stage as compared to the conventional fill. Ultimately the required embankment height could be achieved in shorter time period with lesser settlement.

Another advantage of lightweight fill material is obtained from refilling. When initial fill was done by lateritic soil and allowed to consolidate, later that fill could be removed and refilling could be done by lightweight fill. If the reduction in the fill load can be made equal to the pavement and traffic load the in-service settlement in the soft soil can be reduced to negligible values.

By considering the above advantages of lightweight fills two different construction procedures were proposed incorporating the lightweight fill material in the preloading process with the objective of reducing the in-service settlements and construction periods of the road embankments done on soft soil. In the construction procedure 1, staged fillings were done by conventional fill and soft soil was allowed to achieve an appropriate degree of consolidation. When an embankment height equal or grater than the finished level is achieved, the conventional fill above the ground level was removed and refilling was done by a light weight fill material. If a lightweight fill is not incorporated in the analysis the required degree of consolidation is greater and resulting settlements, fill thicknesses and construction period would be greater.

Further advantages were gained by another procedure named procedure 2, where the initial filling was done by the lightweight fill and soft soil was allowed to consolidate. The surcharge equivalent to the pavement and traffic load was done by the conventional lateritic fill. Once the desired level of consolidation is achieved the surcharge was removed and pavement was constructed. Although the placed thickness is high in this process the consumed volume of fill is less, as the surcharge will be removed later.

Soft peaty clays have not usually been subjected to much erosion and therefore have not experienced greater vertical stresses in the past. In the above analyses, the soft peaty clay was taken, as normally consolidated clay the preconsolidation pressure at any point is same as the effective vertical stresses at that point. However, in most locations surface layer of the peaty clay has been subjected to wetting and drying due to fluctuation in the ground water level. This process results in the peaty clay having a surface crust of a higher strength (Mair, 1992). This could result in a very small over consolidation effect. This had been observed at some SriLankan sites. The surface crust extents down to a depth of approximately 2m below the ground level (Mair, 1992).

As such, another set of analysis was done with the construction process 1 assuming the preconsolidation pressure of $10kN/m^2$ at the ground surface. The results indicated that if the presence of such a small over consolidation effect is accounted for, the required construction periods and fill thicknesses are significantly reduced.

CONCLUSIONS

Presence of peat in the sub soil creates immerse construction problems due to their very high water contents and void ratios. This gives rise to high initial settlements during the construction and settlements continue for a long time during the service period of the road as well. Also, the shear strength is very low and the risk of catastrophic failures during the construction is high.

Heavy structures done under these conditions are invariably being supported on piled foundation. However for moderately loaded structures, service lines and for roads it would not be an economical option. Improvement of the engineering properties of soft peat clay would be more economical in such cases.

Another option specially in the construction embankments is to use a lightweight fill material. Lightweight fill materials have low densities so they impose reduced loads. Hence it can be used in embankment construction to reduce settlement and shear failure. Many different types of lightweight material such as expanded polystyrene blocks, expanded clay, pulverised fuel ash are used in developed countries. Such materials are expensive and not available in SriLanka. We need to find economically competitive solutions with the use of local materials.

Therefore attempts were made to develop a lightweight fill material by mixing conventional lateritic fill materials with; tyre chips, paddy husk, and sawdust. Different mix proportions were tried out and the mixes with proportions of 1:3 (tyre chips: soil) 1:2 (tyre chips: soil), 1:1 (saw dust: soil), 2:3 (saw dust: soil), 1:1 (paddy husk: soil) and 1:1.5 (paddy husk: soil) produced lightweight fill materials of acceptable workability and adequate engineering properties. Lateritic soils are the most widely used and commonly available fill material. It is relatively inexpensive compared to the granular fills. Tyre chips, Sawdust and Paddy husk are waste products that are available relatively in large quantities. As such, cost of the production of the lightweight fill would not be much greater than the cost of the conventional lateritic fills.

The primary requirement in the design of the road embankment is that the in-service settlements should be within acceptable limits. To ensure this, different ground improvement methods were used depending on the prevailing site conditions. The common method is the preloading method. In this research the proposed lightweight fill materials are incorporated with the preloading technique.

Different construction procedures were proposed combining preloading method and lightweight fill material to reduce the service settlement and the construction period of the embankments done on soft soil. Proposed construction processes were numerically simulated by the finite element program CRISP. The effectiveness of the proposed methods were thoroughly examined for different thicknesses of the peaty clay layer and for different embankment heights.

In the construction procedure 1, stage fillings were done by conventional fill and under the fill an appropriate degree of consolidation of the soft soil was achieved. When an embankment surface level greater than the required level by the appropriate margin is achieved, the conventional fill above the ground level was removed and some refilling was done by the lightweight fill material. Thereafter road embankment was constructed and traffic load is applied. The process was numerically simulated by the program CRISP. The appropriate initial surface levels, consolidation periods etc. were decided by several trial analyses. The criterion to be satisfied is that in-service settlement is less than 50mm.

Further reductions in required fill thicknesses and construction periods were achieved by procedure 2. In construction procedure 2, initial staged filling was done by proposed lightweight fill and soft soil was allowed to consolidate. The surcharge equivalent to the pavement and traffic load was done by the conventional lateritic fill. Thereafter the surcharge was removed and pavement was constructed.

The proposed construction process is found to be more effective with the decrease of the density of fill. This is very clear when the embankment done by polystyrene blocks was simulated as it only took one year for the constructions. This emphasizes the need to do research to develop such extremely low-density materials at a comparable low cost locally. If the presence of a small overconsolidation effect is accounted for (as seen in some sites), the required construction period and the fill thickness can be significantly reduced

In this study the proposed raw materials used for lightweight fills sawdust and paddy husk are biodegradable. As such, further studies should be done to assess their degradation within the construction period and to find way of minimizing the deterioration of engineering characteristics.

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