STUDY ON SLOPE STABILIZATION BY SOIL NAILING

Project Report submitted in partial fulfillment for the degree of Bachelor of Technology in Civil Engineering

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Certificate

This is to certify that project report entitled “ **Study on slope stabilization by soil nailing and stability analysis using geo5 software**”,submitted by Sachin Goyal and Tushar Soni in partial fulfillment for the award of degree of Bachelor of Technology in Civil Engineering to Jaypee University of Information Technology, Waknaghat, Solan has been carried out under my supervision.This work has not been submitted partially or fully to any other University or Institute for the award of this or any other degree or diploma.

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Date:

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ABSTRACT

Since its development in Europe in the early 1970s, soil nailing has become a widely accepted method of providing temporary and permanent earth support, underpinning and slope stabilization on many civil projects in the United States. In the early years, soil nailing was typically performed only on projects where specialty geotechnical contractors offered it as an alternate to other, conventional systems. More recently, soil nailing has been specified as the system of choice due to its overall acceptance and effectiveness. However, although the theoretical engineering aspects of soil nailing may be well understood, there is a far lesser degree of understanding, even within the geotechnical community, as to the site conditions – where ,when and why – under which soil nailing should, and should not, be used. The purpose of this paper, therefore, is to offer experienced-based guidelines to owners, engineers, designers and general contractors trying to decide if soil nailing is the right system for their project. Typical soil nail details, procedures, design, monitoring and testing considerations, and case studies are presented as a tool to aid in making those decisions

A landslide stabilization system using tiered soil nail walls and a mechanically stabilized earth (MSE) wall was instrumented and monitored to evaluate overall unstable slopes. Site conditions, design aspects, and construction of the soil nail and MSE walls are described. Performance based on field observations of ground movements and load transfer in soil nails is described and discussed. Recommendations for applying soil nail walls to slope stabilization are presented performance and facilitate comparisons between design assumptions and field observations. This project demonstrates the feasibility of utilizing soil nail walls for stabilization of active landslides, extending the application of soil nailing beyond its traditional scope of stabilization of cut slopes or for potentially

Key words: Soil nailing,

Soil Stabilization,

Retention wall,

Landslide,

Mechanically Stabilized earth

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CHAPTER 1

INTRODUCTION

1.1 Nails and soil nailing

Soil nailing is a technique in which soil slopes, excavations or retaining walls are passively reinforced by the insertion of relatively slender elements - normally steel reinforcing bars. Such structural element which provides load transfer to the ground in excavation reinforcement application is called nail (Fig. 1.1). Soil nails are usually installed at an inclination of 10 to 20 degrees with horizontal and are primarily subjected to tensile stress. Tensile stress is applied passively to the nails in response to the deformation of the retained materials during subsequent excavation process. Soil nailing is typically used to stabilize existing slopes or excavations where top-to-bottom construction is advantageous compared to the other retaining wall systems. As construction proceeds from the top to bottom, shotcrete or concrete is also applied on the excavation face to provide continuity. Fig. 1.2 depicts cross section of a grouted nailed wall along with some field photographs of the same in Fig. 1.3. In the present era, soil nailing is being carried out at large in railway construction work for the stabilization of side lopes in existing track-road or laying of new tracks adjoining to an existing one (Fig. 1.4).

FIGURE 1. **Soil nail with centralizers**

(www.williamsform.com/Ground_Anchors/Soil_Nails_Soil_Nailing/soil_nail_soil_nailing.html)

Various types of soil nailing

Various types of soil nailing methods are employed in the field:

1. **Grouted nail-** After excavation, first holes are drilled in the wall/slope face and then the nails are placed in the pre-drilled holes. Finally, the drill hole is then filled with cement grout.
2. **Driven nail-** In this type, nails are mechanically driven to the wall during excavation. Installation of this type of soil nailing is very fast; however, it does not provide a good corrosion protection. This is generally used as temporary nailing.
3. **Self-drilling soil nail-** Hollow bars are driven and grout is injected through the hollow bar simultaneously during the drilling. This method is faster than the grouted nailing and it exhibits more corrosion protection than driven nail.
4. **Jet-grouted soil nail-** Jet grouting is used to erode the ground and for creating the hole to install the steel bars. The grout provides corrosion protection for the nail.
5. **Launched soil nail-** Bars are "launched" into the soil with very high speed using firing mechanism involving compressed air. This method of installation is very fast; however, it is difficult to control the length of the bar penetrating the ground.

Elements of nailed structure

Various components of a grouted soil nail are discussed in this section. The cross-section of a nailed wall is presented in Fig. 1.5 along with field photographs of various components in Fig. 1.6

1. **Steel reinforcing bars** – The solid or hollow steel reinforcing bars (with minimum strength of 415 kPa) are the main component of the soil nailing system. These elements are placed in pre-drilled drill holes and grouted in place.
2. **Centralizers**- PVC material, which is fixed to the soil nail to ensure that the soil nail is centered in the drill hole.
3. **Grout** – Grout is injected in the pre-drilled borehole after the nail is placed to fill up the annular space between the nail bar and the surrounding ground. Generally, neat cement grout is used to avoid caving in drill-hole; however, sand-cement grout is also applied for open-hole drilling. Grout transfers stress from the ground to the nail and also acts as corrosion protection to the soil nail. Grout pipe is used to inject the grout.
4. **Nail head** – The nail head is the threaded end of the soil nail that protrudes from the wall facing. It is a square shape concrete structure which includes the steel plate, steel nuts, and soil nail head reinforcement. This part of structure provides the soil nail bearing strength, and transfers bearing loads from the soil mass to soil nail.
5. **Hex nut, washer, and bearing plate** – These are attached to the nail head and are used for connecting the soil nail to the facing. Bearing plate distributes the force at nail end to temporary shortcrete facing.
6. **Temporary and permanent facing** – Nails are connected to the excavation or slope surface by facing elements. Temporary facing is placed on the unsupported excavation prior to advancement of the excavation grades. It provides support to the exposed soil, helps in corrosion protection and acts as bearing surface for the bearing plate. Permanent facing is placed over the temporary facing after the soil nails are installed.
7. **Drainage system** – Vertical geocomposite strip drains are used as drainage system media. These are placed prior to application of the temporary facing for collection and transmission of seepage water which may migrate to the temporary facing.
8. **Corrosion protection -** Protective layers of corrugated synthetic material [HDPE (High Density Polyethylene) or PVC tube] surrounding the nail bar is usually used to provide additional corrosion protection.

Advantage and Disadvantages of soil nailing

Some advantage and disadvantage of soil nailing procedure are addressed in other literatures (Yeung, 2008, FHWA-SA-96-069R, FHWA0-IF-03-017) and presented in this section.

1.4.1 Advantage of soil nailing

Soil nailing has several advantages over other ground anchoring and top to down construction techniques. Some of the advantages are described below:

- Less disruptive to traffic and causes less environmental impact than other construction techniques.
- Installation of soil nail walls is relatively faster and uses typically less construction materials. It is advantageous even at sites with remote access because smaller equipment is generally needed.
- Easy adjustments of nail inclination and location can be made when obstructions (e.g., cobbles or boulders, piles or underground utilities) are encountered. Hence, the field adjustments are less expensive.
- Compared to ground anchors, soil nails require smaller right of way than ground anchors as soil nails are typically shorter. Unlike ground anchor walls, soldier beams are not used in soil nailing, and hence overhead construction requirements are small.
- Because significantly more soil nails are used than ground anchors, adjustments to the design layout of the soil nails are more easily accomplished in the field without compromising the level of safety
- It provides a less congested bottom of excavation, particularly when compared to braced excavations
- Soil nail walls are relatively flexible and can accommodate relatively large total and differential settlements. Measured total deflections of soil nail walls are usually within tolerable limits. Soil nail walls have performed well during seismic events owing to overall system flexibility
- Soil nail walls are more economical than conventional concrete gravity walls when conventional soil nailing construction procedures are used. It is typically equivalent in cost or more cost-effective than ground anchor walls. According to Cornforth (2005) soil nailing can result in a cost saving of 10 to 30 percent when compared to tieback walls. Shotcrete facing is typically less costly than the structural facing required for other wall systems.

Disadvantage of soil nailing

Some of the potential disadvantages of soil nail walls are listed below:

- In case of soil nailing, the system requires some soil deformation to mobilize resistance. Hence soil nailing is not recommended for applications where very strict deformation control is required. Post tensioning of soil nails can overcome this shortcoming, but this step in turn increases the project cost.
- Soil nail walls are not well-suited for grounds with high groundwater table for difficulty in drilling and excavation due to seepage of ground water into the excavation, corrosion of steel bars and change in grout water ratio.
- Soil nails are not suitable in cohesionless soils, because during drilling of hole, the un-grouted hole may collapse. However, in such a case drilling can be conducted by providing casing during the drilling process.
- Soil nails are drilled inside the slope wherein they might contain utilities such as buried water pipes, underground cables and drainage systems. Therefore, they should be placed at a safe distance, if possible, by changing its inclination or length or spacing to achieve this distance.
- Construction of soil nail walls requires specialized and experienced contractors.

1.5 Various issues affecting soil nailed slope

There are several factors that affect the feasibility and stability of soil nailing in slopes or excavations. As mentioned earlier, construction of soil nailing is subjected to favorable ground conditions. There are also various internal and global stability factors for soil nailed slopes.

Favorable ground condition- Soil nailing is well suited for Stiff to hard fine-grained soils which includes stiff to hard clays, clayey silts, silty clays, sandy clays, sandy silts, and combinations of theses. It is also applicable for dense to very dense granular soils with some apparent cohesion (some fine contents with percentage of fines not more than 10-15%). Nailing is not suitable for dry, poorly graded cohesionless soils, soils with cobbles and boulder (difficult to drill and increases construction cost), highly corrosive soil (involves expensive corrosion protection), soft to very soft fine-grained soils, and organic soil (very low bond stress or soil nail interaction force leading to excess nail length). Soil nailing is also not recommended for soils with high ground water table.

External stability- The external or global stability of nailed slope includes stability of nailed slope, overturning and sliding of soil-nail system, bearing capacity failure against basal heave due to excavation. Sometimes long-term stability problem also come into picture, e.g., seasonal raining. In such cases, though ground water table may be low, the seeping water may affect the stability of nailed slope without facing or proper drainage system.

Internal stability- It comprises of various failure modes of nailed structure e.g. nail soil pull-out failure, nail tensile failure, and facing flexural or punching shear failure.

Such issues may be overcome by

- Conducting adequate ground investigation and geotechnical testing for identification of soil parameters and ground characterization.
- Performing in-situ test for soil nail interaction and nail strength. Effective design of nailed slope system.
- Stability analysis is a major part in design of nailed slope structure. It involves proper evaluation of nail-soil interaction forces (bond stress) and nail strength which further requires interpretation from respective in-situ tests (nail pull-out capacity, nail tensile capacity test etc).

1.6 Construction procedure of nailed structure

Soil nailed structures are generally constructed in stages and it involves following steps:

- Excavation till the depth where nails will be installed at a particular level
- Drilling nail holes
- Nail installation and grouting
- Construction of temporary shotcrete facing

Subsequent levels are then constructed and finally permanent facing is placed over the wall. The details of the construction methodology and equipments are described in chapter 6. Some of the field photographs of soil nail construction procedure are presented in Fig

1.7 Testing and inspection

Soil nailing for slope or excavation involves various tests and monitoring at different stage of construction.

- **Before construction-** As mentioned earlier, ground exploration and geotechnical testing is conducted before commencement of excavation. It includes boring, sampling, field testing (SPT, CPT and ground water level determination), and lab experiments (grain size distribution, Atterberge limits, moisture content, consolidation, unconfined compression and triaxial tests). Test nails (5% of total nails required in construction) are used for nail pull-out test or ultimate test prior to the installation of nails for estimation of bond strength. Apart from ultimate test, some verification tests are also carried out on test nails.
- **During construction-** A minute inspection should be performed for quality control of the construction materials (storage and handling of nail tendons, reinforcements, cement, drainage material and checking of their required specification). Construction works do also need to be monitored properly at various stages (excavation, soil nail hole drilling, tendon installation, grouting, structural wall facing and drainage).
- **Performance monitoring-** It is important to monitor the performance of nailed slopes for improvement in future construction and design of such structures. Hence, some of the nailed slopes are instrumented for their performance monitoring. The parameters monitored are

  Horizontal and vertical movement of wall face, surface and overall structure

  Performance of any structure supported by the reinforced ground Deterioration of facing and other soil nailing elements Nail loads and change of distribution with time Drainage behavior of ground Slope inclinometer, electronic distance measuring equipments are installed at various survey positions on the nailed structure, and load cells are installed at nail head for such monitoring purpose

Chapter 2

LITERATURE REVIEW

Different reported techniques are discussed below:

- Smith et al. [1] proposed a method to use of Launched soil nails to stabilize Shallow Slope Failure on Urban Access Road.
- Ansari et al. [2] discussed the Soil Nailing Earth Shoring System.
- Gurpersuad et al. [3] proposed a method to Pull-out capacity of soil nails in unsaturated soils.
- Joy et al. [4] proposed a method to investigate on the Dynamic Behaviour of Soil Nail Walls.
- Turner et al. [5] discussed Landslide Stabilization Using Soil Nail and Mechancially Stabilized Earth Walls.
- Menkiti et al. [6] discussed Performance of soil nails in Dublin glacial till.
- Lum et al. [7] proposed a method to Static Pull-out Behaviour of Soil Nails in Residual Soil.

#load and settlement curve using plate load test (from reference)

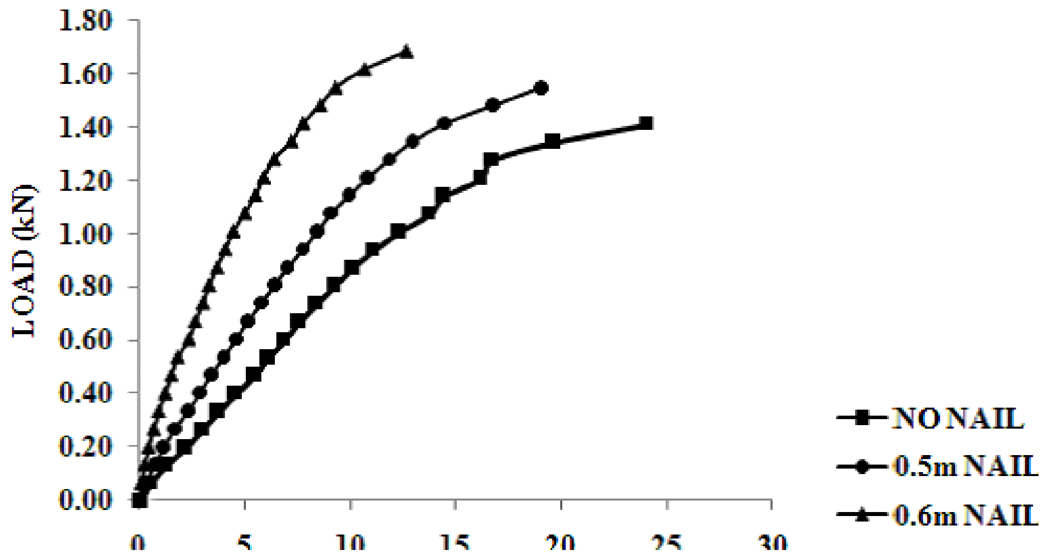


Fig 2

AFTER MAKING SETTELEMENT VS LOAD GRAPH (DOING PLATE LOAD TEST)

- THE FAILURE OF SOIL WALL MODEL OCCUR AT 1.344KN AND SETTLEMENT=19.59 mm
- THE FAILURE OF SOIL WALL MODEL OCCURS AT 1.56KN AND SETTLEMET =12.95mm(after inserting 0.5mm (nails))
- THE FAILURE OF SOIL WALL MODEL OCCURS AT 1.6 KN AND SETTEMENT =7.22mm (after inserting 0.6 mm(nails)

2.1) Analytical study

2.1.1) Slope stability

Slope stability is the potential of soil covered slopes to withstand and undergo movement. Slope stability, or the lack thereof, rests upon the ability of a slope to resist stress excess to what is normally acceptable for the material property of the soil or rock inherent to the construction slope. Slope movements, such as translational or rotational slope failures occur when sheer stress exceeds sheer strength of the materials forming the slope . Factors contributing to high sheer stress include: lack of lateral support, excessive surcharges, lateral pressures and removal of underlying support. On the other hand, low sheer strength, due to inherently weak materials, soil weathering (swelling, shirking and cracking) and low inter-granular force due to seepage pressure, also contributes to slope instability.

The field of slope stability encompasses static and dynamic stability of slopes of earth and rock-fill dams, slopes of other types of embankments, excavated slopes, and natural slopes in soil and soft rock.Slope stability investigation, analysis (including modeling), and design mitigation is typically completed by geologists, engineering geologists, or geotechnical engineers. Geologists and engineering geologists can also use their knowledge of earth process and their ability to interpret surface geomorphology to determine relative slope stability based simply on site observations.

Factors of safety are generally used in evaluating slope stability. The factor of safety can be defined as the ratio of the total force available to resist sliding to the total force tending to induce sliding. A stable slope is considered to be in a condition where the resisting forces are greater than the disturbing forces. Conversely, an unstable slope failure situation exists when resisting and disturbing forces are equal and the factor or safety equals F=1. A Limit Equilibrium condition exists when the forces tending to induce sliding are exactly balanced by those resisting sliding

2.1.2)SLOPE STABILITY ANALYSIS

Slope stability analysis is performed to assess the safe design of human-made or natural slopes (e.g. embankments, road cuts, open-pit mining, excavations, landfills etc.) and the equilibrium conditions. Slope stability is the resistance of inclined surface to failure by sliding or collapsing. The main objectives of slope stability analysis are finding endangered areas, investigation of potential failure mechanisms, determination of the slope sensitivity to different triggering mechanisms, designing of optimal slopes with regard to safety, reliability and economics, designing possible remedial measures, e.g. barriers and stabilization.

Successful design of the slope requires geological information and site characteristics, e.g. properties of soil/rock mass, slope geometry, groundwater conditions, alternation of materials by faulting, joint or discontinuity systems, movements and tension in joints, earthquake activity etc. The presence of water has a detrimental effect on slope stability. Water pressure acting in the pore spaces, fractures or other discontinuities in the materials that make up the pit slope will reduce the strength of those materials. Choice of correct analysis technique depends on both site conditions and the potential mode of failure, with careful consideration being given to the varying strengths, weaknesses and limitations inherent in each methodology.

2.1.3) Analysis according to theory of limit states (LSD)

The verification methodology based on the theory of "Limit states" proves the safety by comparing a resisting variable (resisting force, strength, bearing capacity) and a variable causing failure (sliding force, stress).

$$X_{pas} > X_{act}$$

where: X_{pas}- A variable resisting the failure (resisting force, strength, capacity)

X_{act}- A variable causing the failure (sliding force, stress)

X_{act} is in general determined from the design parameters of soils and loading:

- soil parameters are reduced by corresponding coefficients
- load (its action) is increased by corresponding coefficients

X_{pas} is determined based on the following assumptions:

- soil parameters are reduced by corresponding coefficients
- the calculated structure resistance is reduced by a corresponding coenfficient

In general, it can be stated that the verification based on "Limit states" is more modern and apt approach in comparing to the "Safety factor". However, it is less lucid..

The value of utilization Vu is calculated and then compared with the value of 100%. The value of utilization is given by:

$$V_u = \frac{M_a}{M_p} 100 < 100\%$$

where: M_a- sliding moment

M_p- resisting moment

The resisting moment M_p is determined considering the reduction with the help of overall stability of construction γ_s.

2.1.4)Analysis according to safety factor (ASD)

The verification methodology based on the "Safety factor" is historically the oldest and most widely used approach for structure safety verification. The principal advantage is its simplicity and lucidity.

In general, the safety is proved using the safety factor:

$$FS = \frac{X_{pas}}{X_{act}} > FS_{req}$$

where: FS-Computed safety factor

X_{pas}- A variable resisting the failure (resisting force, strength, capacity)

X_{act}- A variable the causing failure (sliding force, stress)

FS_{req}- Required factor of safety

When performing the analysis using the "Safety factor", neither the load nor the soil parameters are reduced by any of the design coefficients.

Verification according to the factor of safety:

$$\frac{M_p}{M_a} > SF_s$$

where: M_a- sliding moment

M_p- resisting moment

SF_s- factor of safety

2.1.5) Short Term and Long Term Stability

In carrying our slope stability analyses for design purposes it is wise to check both short term and long term conditions. For the short term conditions an effective stress analysis could be used, but this will require an estimate of the pore pressures that will be developed. Alternatively a total stress analysis could be used, but this would only be applicable in cases where the pore pressure changes are entirely dependent upon stress changes. For long term conditions an effective stress analysis is normally carried out, since the pore pressures are usually independent of stress changes. For this analysis estimates of the pore pressures, for example, by means of flownets, are required. For natural slopes and slopes in residual soils, they should be analyzed with the effective stress method, considering the ma ximum water level under severe rainstorms.

2.1.6)Effective/total stress in soil

Vertical normal stress σz is defined as:

$$\sigma_z = \gamma_{ef}.z + \gamma_w.z$$

where: σ_z-vertical normal total stress

γ_{ef}- submerged unit weight of soil

z- depth bellow the ground surface

γ_w- unit weight of water

This expression in its generalized form describes so called concept of effective stress:

$$\sigma_z = \sigma_{ef} + u$$

where: σ- total stress (overall)

σ_{ef}- effective stress (active)

u- neutral stress (pore water pressure)

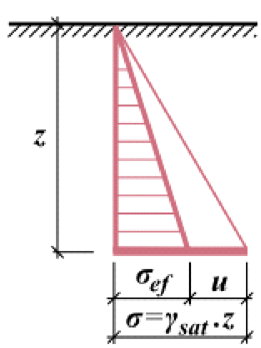


FIGURE3. (Total, effective and neutral stress in the soil)

Effective stress concept is valid only for the normal stress σ, since the shear stress τ is not transferred by the water so that it is effective. The total stress is determined using the basic tools of theoretical mechanics, the effective stress is then determined as a difference between the total stress and neutral (pore) pressure (i.e. always by calculation, it can never be measured). Pore pressures are determined using laboratory or in-situ testing or by calculation. To decide whether to use the total or effective stresses is no simple. The following table may provide some general recommendations valid for majority of cases. We should realize that the total stress depends on the way the soil is loaded by its self-weight and external effects. As for the pore pressure we assume that for flowing pore water the pore equals to hydrodynamic pressure and to hydrostatic pressure otherwise. For partial saturated soils with higher degree of it is necessary to account for the fact that the pore pressure evolves both in water and air bubbles.

Assume conditions	**Drained layer**	**Undrained layer**
short – term	effective stress	total stress
long – term	effective stress	effective stress

In layered subsoil with different unit weight of soils in individual horizontal layers the vertical total stress is determined as a sum of weight of all layers above the investigated point and the pore pressure:

$$\sigma_z = \int_0^z \gamma \, dz + \gamma_w (z - d)$$

where: σ_z- vertical normal total stress

γ- unit weight of soil

- unit weight of soil in natural state for soils above the GWT and dry layers

- unit weight of soil below water in other cases

d- depth of the ground water table below the ground surface

z- depth bellow the ground surface

γ_w- unit weight of water

2.1.7)Definition of the factor of safety (FOS)

The factor of safety for slope stability analysis is usually defined as the ratio of the ultimate shear strength divided by the mobilized shear stress at incipient failure. There are several ways in formulating the factor of safety F. The most common formulation for F assumes the factor of safety to be constant along the slip surface, and it is defined with respect to the force or moment equilibrium:

Moment equilibrium: generally used for the analysis of rotational land- slides. Considering a slip surface, the factor of safety F_m defined with respect to moment is given by:

$F_m=M_r/M_d$

Where M_r is the sum of the resisting moments and M_d is the sum of the driving moment. For a circular failure surface, the center of the circle is usually taken as the moment point for convenience. For a non-circular failure surface, an arbitrary point for the moment consideration may be taken in the analysis. It should be noted that for methods which do not satisfy horizontal force equilibrium (e.g. Bishop Method), the factor of safety will depend on the choice of the moment point as 'true' moment equilibrium requires force equilibrium. Actually, the use of the moment equilibrium equation without enforcing the force equilibrium cannot guarantee 'true' moment equilibrium.

Force equilibrium: generally applied to translational or rotational failures composed of planar or polygonal slip surfaces. The factor of safety F_f defined with respect to force is given by:

$F_f=F_r/F_d$

Where F_r is the sum of the resisting forces and F_d is the sum of the driving forces.

An acceptable factor of safety shall fulfill the basic requirement from the soil mechanics principle as well as the long-term performance of the slope. The geotechnical engineers should consider the current slope conditions as well as the future changes, such as the possibility of cuts at the slope toe, deforestation, surcharges and excessive infiltration. For very important slopes, there may be a need to monitor the pore pressure and suction by tensiometer and piezometer, and the displacement can be monitored by the inclinometers, GPS or microwave reflection. Use of strain gauges or optical fibers in soil nails to monitor the strain and the nail loads may also be considered if necessary. For larger-scale projects, the use of the classical monitoring method is expensive and time-consuming, and the use of the GPS has become popular in recent years.

2.1.8) Critical Slope Surfaces

A critical slope surface exists when a combination of soil and slope factors create a high potential for slope face failure and subsequent erosion. Over-steepened freshly graded or disturbed slopes are considered critical when resistance to surface erosion is low and sheer and strength resistance tolerances are exceeded. The potential for slope face failure of the slope can compound with inadequate slope face compaction under super saturated conditions. In such cases, soil movements are influenced by numerous parameters including, but not limited to, angle of repose, soil structure, slope length and erodibility.

2.1.9) Circular slip surface

All methods of limit equilibrium assume that the soil body above the slip surface is subdivided into blocks (dividing planes between blocks are always vertical). Forces acting on individual blocks are displayed in figure.

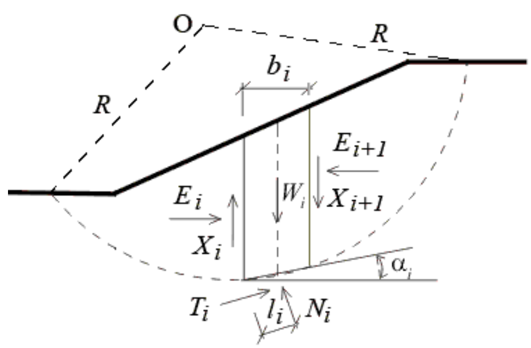


FIGURE4. (Circular slip surface)

Here, Xi and Ei are the shear and normal forces acting between individual blocks, Ti and Ni are the shear and normal forces on individual segments of the slip surface, Wi are weights of individual blocks.

Individual methods of slices differ in their assumptions of satisfying the force equations of equilibrium and the moment equation of equilibrium with respect to the center O.

Various methods that can be adopted are:

- Fellenius / Petterson
- Bishop
- Spencer
- Janbu
- Morgenstern-Price

- Shahunyants
- ITF Method

Ground water specified within the slope body (using one of the five options) influences the analysis in two different ways. First when computing the weight of a soil block and second when determining the shear forces. Note that the effective soil parameters are used to relate the normal and shear forces.

Optimization of circular slip surface

The goal of the optimization process is to locate a slip surface with the smallest factor of slope stability SF. The circular slip surface is specified in terms of 3 points: two points on the ground surface and one inside the soil body. Each point on the surface has one degree of freedom while the internal point has two degrees of freedom. The slip surface is defined in terms of four independent parameters. Searching for such a set of parameters that yields the most critical results requires sensitivity analysis resulting in a matrix of changes of parameters that allows for fast and reliable optimization procedure. The slip surface that gives the smallest factor of slope stability is taken as the critical one. Parameters of individual slip surfaces and results from optimization runs can be displayed in output document.

This approach usually succeeds in finding the critical slip surface without encountering the problem of falling into a local minimum during iteration. It therefore appears as a suitable starting point when optimizing general slip surfaces such as the polygonal slip surface.

The optimization process can be restricted by various constraints. This becomes advantageous especially if we wish the searched slip surface to pass through a certain region or to bypass this region. Optimization restrictions are specified as a set of segments in a soil body. The optimized slip surface is then forced to bypass these segments during optimization.

2.1.9) Polygonal slip surface

Solution of the slope stability problem adopting the polygonal slip surface is based on the determination of the limit state of forces acting on the soil body above the slip surface. To introduce these forces the slip surface above is subdivided into blocks by dividing planes. Typically, these planes are assumed vertical, but this is not a required condition, e.g. the Sarma method considers generally inclined planes.

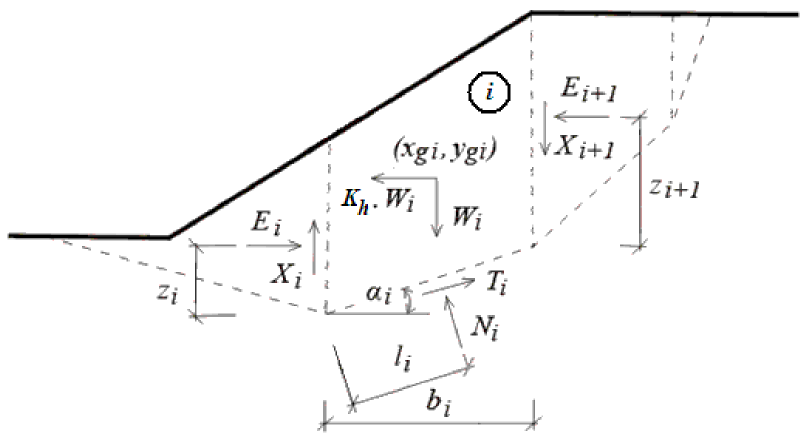


FIGURE5. (Polygonal Slip surface)

The figure shows forces acting on individual blocks of soil. If the region above the slip surface is divided in blocks, then for the evaluation of unknowns we have: n normal forces Ni acting on individual segments and corresponding n shear forces Ti; n-1 normal forces between blocks Ei and corresponding n-1 shear forces Xi ; n-1 values of zi representing the points of application of forces Ei, n values of li representing the points of application of forces Ni and one value of the factor of safety SF. Forces Xi can be in some methods replaced by the values of inclination of forces Ei.

To following set of equations is available to solve the problem of equilibrium: n horizontal and n vertical equations of equilibrium written for individual blocks, n moment equations of equilibrium for individual blocks and n relations between Ni and Ti forces developed on blocks according to the Mohr-Coulomb theory. In total there are 4n equations for 6n-2 unknowns. This suggests that 2n-2 unknowns must be chosen a prior. Individual methods differ from each other in the way these values are selected.

Most often points of application of individual forces acting between blocks or their inclinations are selected. Solving the problem of equilibrium it proceeds in an iterative manner, where the selected values must allow for satisfying both the equilibrium and kinematical admissibility of the obtained solution.

Various methods that can be adopted are:

- Sarma
- Spencer
- Janbu
- Morgenstern-Price
- Shahunyants

- ITF Method

Optimization of polygonal slip surface

The slip surface optimization proceeds such that the program changes subsequently locations of individual points of this surface and checks, which change of location of a given point results in the maximal reduction of the factor of slope stability SF. The end points of the optimized slip surface are moved on the ground surface, internal points are moved in the vertical and horizontal directions. The step size is initially selected as one tenth of the smallest distance of neighboring points along the slip surface. With every new run the step size is reduced by one half. Location of points of slip surface is optimized subsequently from the left to the right and it is completed when there was no point moved in the last run.

When optimizing the polygonal slip surface the iteration process may suffer from falling into the local minimum (with respect to gradual evolution of locations of nodal points) so not always the process is terminated by locating the critical slip surface. Especially in case of complex slope profile it is therefore advantageous to introduce several locations of the initial slip surface. Combination with the approach used for circular slip surfaces is also recommended. Therefore, the critical slip surface assuming circular shape is found first and the result is then used to define the initial polygonal slip surface.

The optimization process can be restricted by various constraints. This becomes advantageous especially if we wish the searched slip to pass through a certain region or to bypass this region. The restriction on the optimization process can be performed in two different ways:

Optimization restrictions are specified as a set of segments in a soil body. The optimized slip surface is then forced to bypass these segments during optimization.

Another way of restricting the optimization process is to fix location of selected points along the optimized slip surface or allow for moving these points only in one of two directions, either vertically or horizontally

chapter 3
METHODOLOGY

3.1)experimental work

3.1.1) Sieve Analysis Test

Testing objectives:

The Standard grain size analysis test determines the relative proportions of different grain sizes as they are distributed among certain size ranges.

Need and Scope:

The grain size analysis is widely used in classification of soils. The data obtained from grain size distribution curves is used in the design of filters for earth dams and to determine suitability of soil for road construction, air field etc. Information obtained from grain size analysis can be used to predict soil water movement although permeability tests are more generally used.

Apparatus Required:

Stack of Sieves including pan and cover,Balance (with accuracy to 0.01 g),Rubber pestle and Mortar (for crushing the soil if lumped or conglomerated),Mechanical sieve shaker,Oven

Notice: The balance to be used should be sensitive to the extent of 0.1% of total weight of sample taken.

Test Procedure:

- take a representative oven dried sample of soil that weighs about **500** g. (this is normally used for soil samples the greatest particle size of which is 4.75 mm)

- If soil particles are lumped or conglomerated crush the lumped and not the particles using the pestle and mortar.
- Determine the mass of sample accurately. W_t (g)
- Prepare a stack of sieves. sieves having larger opening sizes (i.e lower numbers) are placed above the ones having smaller opening sizes (i.e higher numbers). The very last sieve is #200 and a pan is placed under it to collect the portion of soil passing #200 sieve. Here is a full set of sieves. (#s 4 and 200 should always be included)

Sieve Number	Opening Size (mm)
4	4.750
6	3.350
8	2.360
12	1.680
16	1.180
20	0.850
30	0.600
40	0.425
50	0.300
60	0.250
80	0.180
100	0.150
140	0.106
200	0.075
270	0.053

TABLE 1. (SIEVE SIZE AND OPENING SIZE)

- Make sure sieves are clean, if many soil particles are stuck in the openings try to poke them out using brush.
- Weigh all sieves and the pan separately.
- Pour the soil from step 3 into the stack of sieves from the top and place the cover, put the stack in the sieve shaker and fix the clamps, adjust the time on 10 to 15 minutes and get the shaker going.

3.1.2) OPTIMUM MOISTURE CONTENT AND DRY DENSITY TEST

This test is done to determine the maximum dry density and the optimum moisture content of soil using heavy compaction as per IS: 2720 (Part 8) – 1983.The apparatus used is

Cylindrical metal mould – it should be either of 100mm dia. and 1000cc volume or 150mm dia. and 2250cc volume and should conform to IS: 10074 – 1982.
ii) Balances – one of 10kg capacity, sensitive to 1g and the other of 200g capacity, sensitive to 0.01g
iii) Oven – thermostatically controlled with an interior of non corroding material to maintain temperature between 105 and 110^{o}C
iv) Steel straightedge – 30cm long
v) IS Sieves of sizes – 4.75mm, 19mm and 37.5mm

PREPARATION OF SAMPLE

A representative portion of air-dried soil material, large enough to provide about 6kg of material passing through a 19mm IS Sieve (for soils not susceptible to crushing during compaction) or about 15kg of material passing through a 19mm IS Sieve (for soils susceptible to crushing during compaction), should be taken. This portion should be sieved through a 19mm IS Sieve and the coarse fraction rejected after its proportion of the total sample has been recorded. Aggregations of particles should be broken down so that if the sample was sieved through a 4.75mm IS Sieve, only separated individual particles would be retained.

Procedure To Determine The Maximum Dry Density And The Optimum Moisture Content Of Soil

A) Soil not susceptible to crushing during compaction –

1) A 5kg sample of air-dried soil passing through the 19mm IS Sieve should be taken. The sample should be mixed thoroughly with a suitable amount of water depending on the soil type

(for sandy and gravelly soil – 3 to 5% and for cohesive soil – 12 to 16% below the plastic limit). The soil sample should be stored in a sealed container for a minimum period of 16hrs.

2) The mould of 1000cc capacity with base plate attached, should be weighed to the nearest 1g (W_1). The mould should be placed on a solid base, such as a concrete floor or plinth and the moist soil should be compacted into the mould, with the extension attached, in five layers of approximately equal mass, each layer being given 25 blows from the 4.9kg rammer dropped from a height of 450mm above the soil. The blows should be distributed uniformly over the surface of each layer. The amount of soil used should be sufficient to fill the mould, leaving not more than about 6mm to be struck off when the extension is removed. The extension should be removed and the compacted soil should be levelled off carefully to the top of the mould by means of the straight edge. The mould and soil should then be weighed to the nearest gram (W_2).

3) The compacted soil specimen should be removed from the mould and placed onto the mixing tray. The water content (w) of a representative sample of the specimen should be determined.

4) The remaining soil specimen should be broken up, rubbed through 19mm IS Sieve and then mixed with the remaining original sample. Suitable increments of water should be added successively and mixed into the sample, and the above operations i.e. ii) to iv) should be repeated for each increment of water added. The total number of determinations made should be at least five and the moisture contents should be such that the optimum moisture content at which the maximum dry density occurs, lies within that range.

B) Soil susceptible to crushing during compaction –

Five or more 2.5kg samples of air-dried soil passing through the 19mm IS Sieve, should be taken. The samples should each be mixed thoroughly with different amounts of water and stored in a sealed container as mentioned in Part A)

C) Compaction in large size mould –

For compacting soil containing coarse material upto 37.5mm size, the 2250cc mould should be used. A sample weighing about 30kg and passing through the 37.5mm IS Sieve is used for the test. Soil is compacted in five layers, each layer being given 55 blows of the 4.9kg rammer. The rest of the procedure is same as above.

REPORTING OF RESULTS

Bulk density γ in g/cc of each compacted specimen should be calculated from the equation,

$$\gamma = (W_2 - W_1) / V$$

where, V = volume in cc of the mould.

The dry density Yd in g/cc

$$\gamma_d = 100Y/(100+w)$$

The dry densities, Yd obtained in a series of determinations should be plotted against the corresponding moisture contents,w. A smooth curve should be drawn through the resulting points and the position of the maximum on the curve should be determined. A sample graph is shown below:

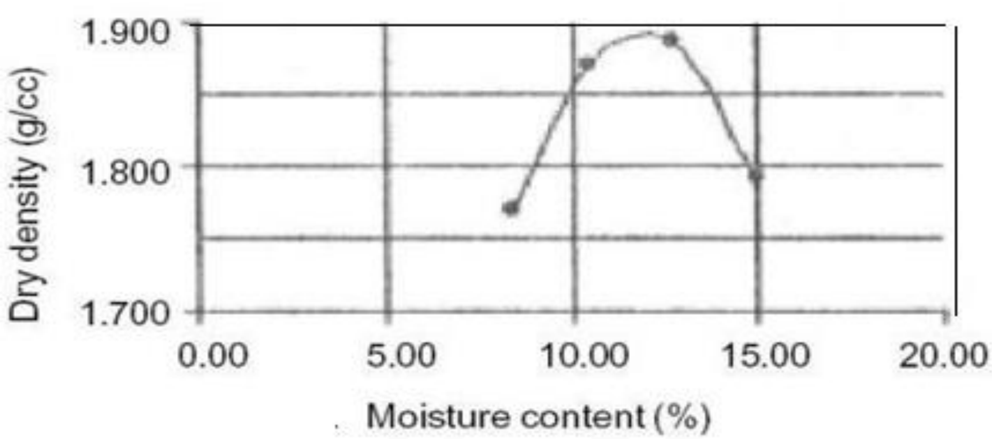


FIGURE 6

The dry density in g/cc corresponding to the maximum point on the moisture content/dry density curve should be reported as the maximum dry density to the nearest 0.01. The percentage moisture content corresponding to the maximum dry density on the moisture content/dry density curve should be reported as the optimum moisture content and quoted to the nearest 0.2 for values below 5 percent, to the nearest 0.5 for values from 5 to 10 percent and to the nearest whole number for values exceeding 10 percent

3.1.3) DIRECT SHEAR TEST

Objective

To determine the shearing strength of the soil using the direct shear apparatus.

NEED AND SCOPE

In many engineering problems such as design of foundation, retaining walls, slab bridges, pipes, sheet piling, the value of the angle of internal friction and cohesion of the soil involved are required for the design. Direct shear test is used to predict these parameters quickly. The laboratory report cover the laboratory procedures for determining these values for cohesionless soils.

PLANNING AND ORGANIZATION

Apparatus

1. Direct shear box apparatus
2. Loading frame (motor attached).
3. Dial gauge.
4. Proving ring.
5. Tamper.
6. Straight edge.
7. Balance to weigh upto 200 mg.

8. Aluminum container.

9. Spatula.

KNOWLEDGE OF EQUIPMENT:

Strain controlled direct shear machine consists of shear box, soil container, loading unit, proving ring, dial gauge to measure shear deformation and volume changes. A two piece square shear box is one type of soil container used.

A proving ring is used to indicate the shear load taken by the soil initiated in the shearing plane

PROCEDURE

1. Check the inner dimension of the soil container.

2. Put the parts of the soil container together.

3. Calculate the volume of the container. Weigh the container.

4. Place the soil in smooth layers (approximately 10 mm thick). If a dense sample is desired tamp the soil.

5. Weigh the soil container, the difference of these two is the weight of the soil. Calculate the density of the soil.

6. Make the surface of the soil plane.

7. Put the upper grating on stone and loading block on top of soil.

8. Measure the thickness of soil specimen.

9. Apply the desired normal load.

10.Remove the shear pin.

11. Attach the dial gauge which measures the change of volume.

12. Record the initial reading of the dial gauge and calibration values.

13. Before proceeding to test check all adjustments to see that there is no connection between two parts except sand/soil.

14. Start the motor. Take the reading of the shear force and record the reading.

15.Take volume change readings till failure.

16. Add 5 kg normal stress 0.5 kg/cm^2 and continue the experiment till failure

17. Record carefully all the readings. Set the dial gauges zero, before starting the experiment

#) GENERAL REMARKS

1. In the shear box test, the specimen is not failing along its weakest plane but along a predetermined or induced failure plane i.e. horizontal plane separating the two halves of the shear box. This is the main draw back of this test. Moreover, during loading, the state of stress cannot be evaluated. It can be evaluated only at failure condition i.e Mohrs circle can be drawn at the failure condition only. Also failure is progressive.
2. Direct shear test is simple and faster to operate. As thinner specimens are used in shear box, they facilitate drainage of pore water from a saturated sample in less time. This test is also useful to study friction between two materials and one material in lower half of box and another material in the upper half of box.
3. The angle of shearing resistance of sands depends on state of compaction, coarseness of grains, particle shape and roughness of grain surface and grading. It varies between 28^o(uniformly graded sands with round grains in very loose state) to 46^o(well graded sand with angular grains in dense state).
4. The volume change in sandy soil is a complex phenomenon depending on gradation, particle shape, state and type of packing, orientation of principal planes, principal stress ratio, stress history, magnitude of minor principal stress, type of apparatus, test procedure, method of preparing specimen etc. In general loose sands expand and dense sands contract in volume on shearing. There is a void ratio at which either expansion contraction in volume takes place. This void ratio is called critical void ratio. Expansion or contraction can be inferred from the movement of vertical dial gauge during shearing.
5. The friction between sand particle is due to sliding and rolling friction and interlocking action.

The ultimate values of shear parameter for both loose sand and dense sand approximately attain the same value so, if angle of friction value is calculated at ultimate stage, slight disturbance in density during sampling and preparation of test specimens will not have much effect.

3.1.4) UNCONFINED COMPRESSION TEST

OBJECTIVE

Determine shear parameters of cohesive soil

NEED AND SCOPE OF THE EXPERIMENT

It is not always possible to conduct the bearing capacity test in the field. Some times it is cheaper to take the undisturbed soil sample and test its strength in the laboratory. Also to choose the best material for the embankment, one has to conduct strength tests on the samples selected. Under these conditions it is easy to perform the unconfined compression test on undisturbed and remoulded soil sample. Now we will investigate experimentally the strength of a given soil sample.

PLANNING AND ORGANIZATION

We have to find out the diameter and length of the specimen.

EQUIPMENT

Loading frame of capacity of 2 t with constant rate of movement and proving ring of 0.01 kg sensitivity for soft soils ; Soil trimmer ; Frictionless end plates of 75 mm diameter (Perspex plate with silicon grease coating) ; Evaporating dish (Aluminum container).

Soil sample of 75 mm length., dial gauge (0.01 mm accuracy) ; Balance of capacity 200 g and sensitivity to weigh 0.01 g. Oven, thermostatically controlled with interior of non-corroding material to maintain the temperature at the desired level ; Sample extractor and split sampler. Dial gauge (sensitivity 0.01mm) ; Vernier calipers.

EXPERIMENTAL PROCEDURE (SPECIMEN)

In this test, a cylinder of soil without lateral support is tested to failure in simple compression, at a constant rate of strain. The compressive load per unit area required to fail the specimen as called Unconfined compressive strength of the soil.

Preparation of specimen for testing

A. Undisturbed specimen

Note down the sample number, bore hole number and the depth at which the sample was taken. Remove the protective cover (paraffin wax) from the sampling tube. Place the sampling tube extractor and push the plunger till a small length of sample moves out. Trim the projected sample using a wire saw. Again push the plunger of the extractor till a 75 mm long sample comes out. Cutout this sample carefully and hold it on the split sampler so that it does not fall. Take about 10 to 15 g of soil from the tube for water content determination. Note the container number and take the net weight of the sample and the container. Measure the diameter at the top, middle, and the bottom of the sample and find the average and record the same. Measure the length of the sample and record. Find the weight of the sample and record.

B. **Moulded sample**

For the desired water content and the dry density, calculate the weight of the dry soil Ws required for preparing a specimen of 3.8 cm diameter and 7.5 cm long.

Add required quantity of water W_w to this soil.

$$W_w = W_S \quad W/100 \text{ gm}$$

Mix the soil thoroughly with water.

Place the wet soil in a tight thick polythene bag in a humidity chamber and place the soil in a constant volume mould, having an internal height of 7.5 cm and internal diameter of 3.8 cm .After 24 hours take the soil from the humidity chamber and place the soil in a constant volume mould, having an internal height of 7.5 cm and internal diameter of 3.8 cm. Place the lubricated moulded with plungers in position in the load frame. Apply the compressive load till the specimen is compacted to a height of 7.5 cm. Eject the specimen from the constant volume mould. Record the correct height, weight and diameter of the specimen.

Test procedure

Take two frictionless bearing plates of 75 mm diameter. Place the specimen on the base plate of the load frame (sandwiched between the end plates).Place a hardened steel ball on the bearing plate. Adjust the center line of the specimen such that the proving ring and the steel ball are in the same line. Fix a dial gauge to measure the vertical compression of the specimen. Adjust the gear position on the load frame to give suitable vertical displacement. Start applying the load and record the readings of the proving ring dial and compression dial for every 5 mm compression. Continue loading till failure is complete. Draw the sketch of the failure pattern in the specimen.

3.1.5)CONSOLIDATION TEST

Purpose:

This test is performed to determine the magnitude and rate of volume decrease that a laterally confined soil specimen undergoes when subjected to different vertical pressures. From the measured data, the consolidation curve (pressure-void ratio relationship) can be plotted. This data is useful in determining the compression index, the recompression index and the preconsolidation pressure (or maximum past pressure) of the soil. In addition, the data obtained can also be used to determine the coefficient of consolidation and the coefficient of secondary compression of the soil.

Standard Reference:

ASTM D 2435 - Standard Test Method for One-Dimensional

Consolidation Properties of Soils.

Significance:

The consolidation properties determined from the consolidation test are used to estimate the magnitude and the rate of both primary and secondary consolidation settlement of a structure or an earth fill. Estimates of this type are of key importance in the design of engineered structures and the evaluation of their performance.

Equipment:

Consolidation device (including ring, porous stones, water reservoir, and loadplate), Dial gauge (0.0001 inch = 1.0 on dial), Sample trimming device, glassplate, Metal straight edge, Clock, Moisture can, Filter paper.

Test Procedure:

1. Weigh the empty consolidation ring together with glass plate.
2. Measure the height (h) of the ring and its inside diameter (d)
3. .Extrude the soil sample from the sampler, generally thin-walled Shelby tube. Determine the initial moisture content and the specific gravity of the soil as per Experiments 1 and 4, respectively (Use the data sheets from these experiments to record all of the data).
4. Cut approximately a three-inch long sample. Place the sample on the consolidation ring and cut the sides of the sample to be approximately the same as the outside diameter of the ring. Rotate the ring and pare off the excess soil by means of the cutting tool so that the sample is reduced to the same inside diameter of the ring. It is important to keep the cutting tool in the correct horizontal position during this process.
5. As the trimming progresses, press the sample gently into the ring and continue until the sample protrudes a short distance through the bottom of the ring. Be careful throughout the trimming process to insure that there is no void space between the sample and the ring.
6. Turn the ring over carefully and remove the portion of the soil protruding above the ring. Using the metal straight edge, cut the soil surface flush with the surface of the ring. Remove the final portion with extreme care.
7. Place the previously weighed Saran-covered glass plate on the freshly cut surface, turn the ring over again, and carefully cut the other end in a similar manner.
8. Weigh the specimen plus ring plus glass plate.
9. Carefully remove the ring with specimen from the Saran-covered glass plate and peel the Saran from the specimen surface. Center the porous stones that have been soaking, on the top and bottom surfaces of the test specimen. Place the filter papers between porous stones and soil specimen. Press very lightly to make sure that the stones adhere to the sample. Lower the assembly carefully into the base of the water reservoir. Fill the water reservoir with water until the specimen is completely covered and saturated.
10. Being careful to prevent movement of the ring and porous stones, place the load plate centrally on the upper porous stone and adjust the loading device.
11. Adjust the dial gauge to a zero reading. With the toggle switch in the down (closed)

position, set the pressure gauge dial (based on calibration curve) to result in an applied pressure of 0.5tsf (tons per square foot).

12. Simultaneously, open the valve (by quickly lifting the toggle switch to the up (open) position) and start the timing clock.
13. Record the consolidation dial readings at the elapsed times given on the data sheet.
14. Repeat Steps for different preselected pressures (generally includes loading pressures of 1.0, 2.0, 4.0, 8.0, and 16.0 tsf and unloading pressures of 8.0, 4.0, 2.0, 1.0 and 0.5 tsf)
15. At the last elapsed time reading, record the final consolidation dial reading and time, release the load, and quickly disassemble the consolidation device and remove the specimen. Quickly but carefully blot the surfaces dry with paper toweling. (The specimen will tend to absorb water after the load is released.)
16. Place the specimen and ring on the Saran-covered glass plate
17. Weigh an empty large moisture can and lid.
18. Carefully remove the specimen from the consolidation ring, being sure not to lose too much soil, and place the specimen in the previously weighed moisture can. Place the moisture can containing the specimen in the oven and let it dry for 12 to 18 hours.
19. Weigh the dry specimen in the moisture can.

3.1.7) SOIL CLASSIFICATION

ON THE BASIS OF RESULTS OBTAINED FROM ABOVE EXPERIMENTS,SOIL WAS CLASSIFIED AS FOLLOWS:-

- Plastic index
- Plasticity index(I_p) = liquid limit(w_L)- plastic limit(w_p)

$$\boxed{I_p=41.9-21.85=20.05}$$

- on plotting the plasticity index and liquid limit on the plasticity chart it is found that the intersecting point lies b\w hatched zone.
- Thus the soil is classified as clayey sand.

3.2)Modeling and simulation

Slope Stability Analysis Methods (Analytical)

3.2.1)Bishop Method

The Modified (or Simplified) Bishop's Method proposed by Alan W. Bishop of Imperial College is a method for calculating the stability of slopes. The simplified Bishop Method uses the method of slices to discretize the soil mass and determine the FS (Factor of Safety).

With this method, the analysis is carried out in terms of stresses instead of forces which were used with the Ordinary Method of Slices. The stresses and forces which act on a typical slice and which are taken into account in the analysis are shown in Fig. 1.

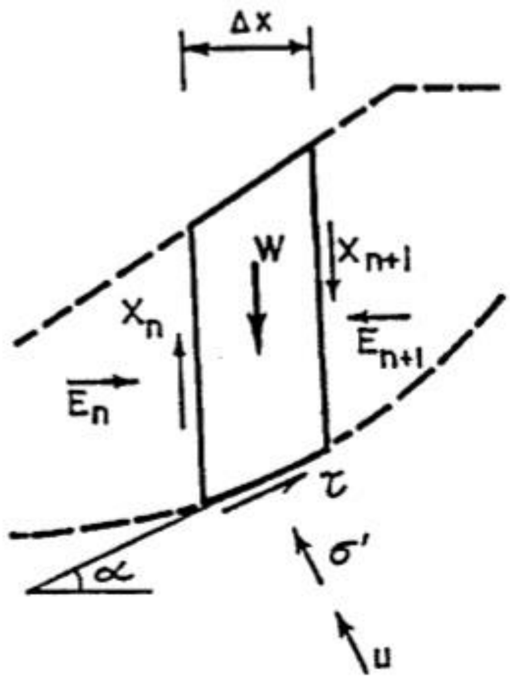


FIGURE7. (Stresses and Forces Acting on a Typical Slice)

This method satisfies vertical force equilibrium for each slice and overall moment equilibrium about the center of the circular trial surface. Since horizontal forces are not considered at each slice, the simplified Bishop method also assumes zero interslice shear forces………………..

3.2.2)Fellenius / Petterson

Developed by Wolmar Fellenius as a result of slope failures in sensitive clays in Sweden, it reduces the force resolution of the slope to a statically determinate structure. The simplest method of slices assumes only the overall moment equation of equilibrium written with respect to the center of the slip surface i.e. side forces (shear and compressional) are not significant. The shear and normal forces between blocks X_i and E_i are neglected. The factor of safety FS follows directly from the following expression:

$$FS = \frac{1}{\sum_i W_i . \sin \alpha_i} . \sum_i \left[c_i . l_i + \left(N_i - u_i . l_i \right) . \tan \varphi_i \right]$$

where:

u_i- pore pressure within block

c_i,φ_i- effective values of soil parameters

W_i- block weight

N_i- normal force on the segment of the slip surface

α_i- inclination of the segment of the slip surface

l_i- length of the segment of the slip surface

3.2.3)Spencer

The Spencer method is a general method of slices developed on the basis of limit equilibrium. It requires satisfying equilibrium of forces and moments acting on individual blocks. The blocks are created by dividing the soil above the slip surface by dividing planes. Forces acting on individual blocks are displayed in the following figure.

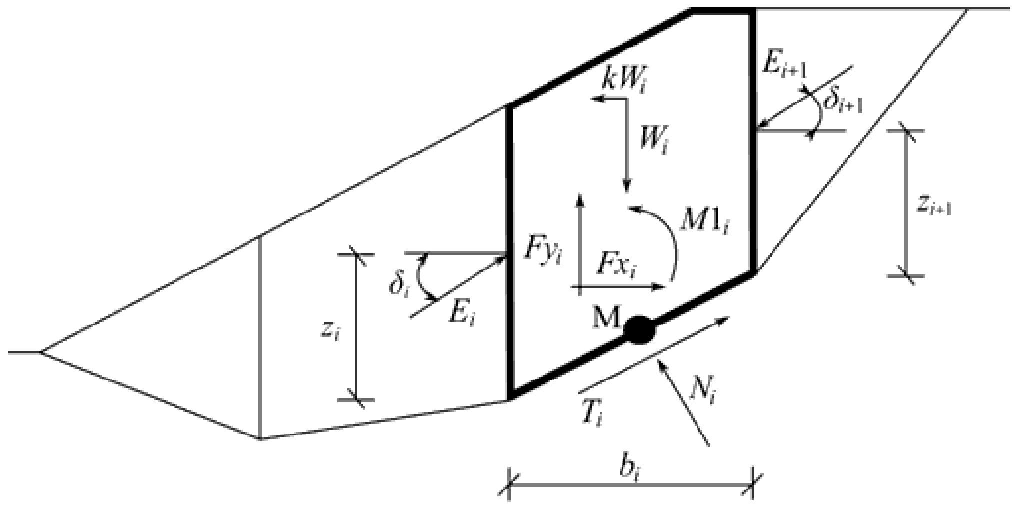


FIGURE 8. (Static scheme – Spencer method)

Each block is assumed to contribute due to the following forces:

Wi- block weight, including material surcharge having the character of weight including the influence of the coefficient of vertical earthquake K_v K_h*W_i- horizontal inertia force representing the effect of earthquake, Kh is the factor of horizontal acceleration during earthquake

N_i- normal force on the slip surface

T_i- shear force on the slip surface

E_i ,E_{i+1}- forces exerted by neighboring blocks, they are inclined from horizontal plane by angle δ

F_{xi},F_{yi}- other horizontal and vertical forces acting on block

M_{1i}- moment of forces F_{xi}, F_{yi} rotating about point M, which is the center of the i^{th} segment of slip surface

U_i- pore pressure resultant on the i^{th} segment of slip surface

The following assumptions are introduced in the Spencer method to calculate the limit equilibrium of forces and moment on individual blocks:

- dividing planes between blocks are always vertical
- the line of action of weight of block Wi passes through the center of the i^{th} segment of slip surface represented by point M
- the normal force Ni is acting in the center of the i^{th} segment of slip surface, at point M
- inclination of forces E_i acting between blocks is constant for all blocks and equals to δ

3.2.4)Janbu

Janbu is a general method of slices developed on the basis of limit equilibrium. It requires satisfying equilibrium of forces and moments acting on individual blocks (only moment equilibrium at last uppermost block is not satisfied). The blocks are created by dividing the soil above the slip surface by dividing planes. Forces acting on individual blocks are displayed in the following figure:

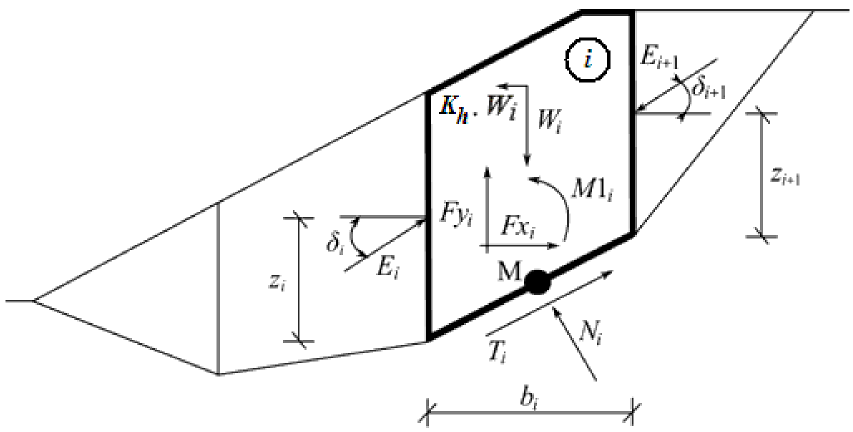


FIGURE9. (Static scheme – Janbu method)

Each block is assumed to contribute due to the following forces:

W_i- block weight, including material surcharge having the character of weight including the influence of the coefficient of vertical earthquakeK_v

K_h*W_i- horizontal inertia force representing the effect of earthquake, K_h is the factor of horizontal acceleration during earthquake

N_i-normal force on the slip surface

Ti-shear force on the slip surface

E_i ,E_{i+1}- forces exerted by neighboring blocks, they are inclined from horizontal plane by angle δ_i resp. δ_{i+1} and lie at the height z_i resp. zi_{+1} above slip surface

F_{xi},F_{yi}- other horizontal and vertical forces acting on block

M_{1i}- moment from forces F_{xi}, F_{yi} rotating about point M, which is the center of the i^{th} segment of slip surface

U_i- pore pressure resultant on the i^{th} segment of slip surface

The following assumptions are introduced in the Janbu method to calculate the limit equilibrium of forces and moment on individual blocks:

- dividing planes between blocks are always vertical
- the line of action of weight of block Wi passes through the center of the i^{th} segment of slip surface represented by point M
- the normal force N_i is acting in the center of the i^{th} segment of slip surface, at point M
- position z_i of forces E_i acting between blocks is assumed, at slip surface end points is z = 0

3.2.5) Morgenstern-Price

Morgenstern-Price is a general method of slices developed on the basis of limit equilibrium. It requires satisfying equilibrium of forces and moments acting on individual blocks. The blocks are created by dividing the soil above the slip surface by dividing planes. Forces acting on individual blocks are displayed in the following figure:

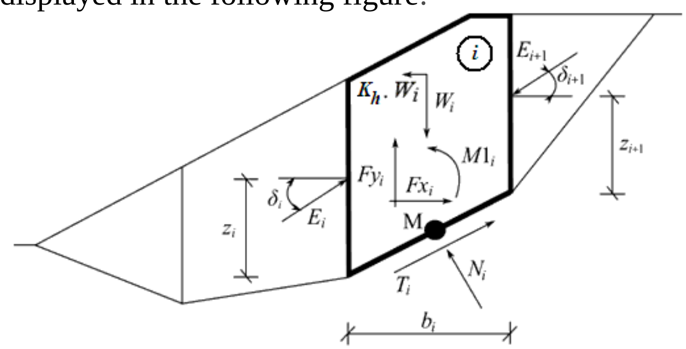


FIGURE10. (Static scheme – Morgenstern-Price method)

Each block is assumed to contribute due to the same forces as in Spencer method.
The following assumptions are introduced in the Morgenstern-Price method to calculate the limit equilibrium of forces and moment on individual blocks:

- dividing planes between blocks are always vertical
- the line of action of weight of block Wi passes through the center of the ith segment of slip surface represented by point M
- the normal force Ni is acting in the center of the ith segment of slip surface, at point M
- inclination of forces Ei acting between blocks is different on each block (δi) at slip surface end points is δ = 0

CHAPTER 4

4.1)Experimental Work

Results from different test performed

Sieve analysis, liquid limit, plastic limit test were the tests performed for classification of soil.

4.1.1) Grain size analysis(sieve analysis)

The following results were obtained on performing grain sieve analysis on the soil under consideration:

I.S sieve No. or size	Wt. of empty sieve(g)	Wt. of soil +sieve(g)	Wt. retained on each sieve(g)	Cumulative Mass Retained(g)	Cumulative % retained on each sieve	% finer
10mm	503.5	503.5	0	0	0	100
4.75mm	418.5	435	16.5	16.5	1.65	98.35
2mm	402	478	76	92.5	9.25	90.75
1mm	374.3	521	146.7	239.2	23.91	76.09
600µ	362.8	440.9	78.1	317.3	31.72	68.28
425 µ	351	399.2	48.2	365.5	36.54	63.46
300 µ	354.6	378.2	23.6	389.11	38.90	61.10
212 µ	336.9	371	34.1	423.2	42.32	57.69
150 µ	357.9	375	17.1	440.3	44.02	55.98
75 µ	329.8	655.3	325.3	765.8	76.56	23.44
PAN	255.9	490.4	234.5	1000.3	100	0

TABLE 2 (SIEVE ANALYSIS)

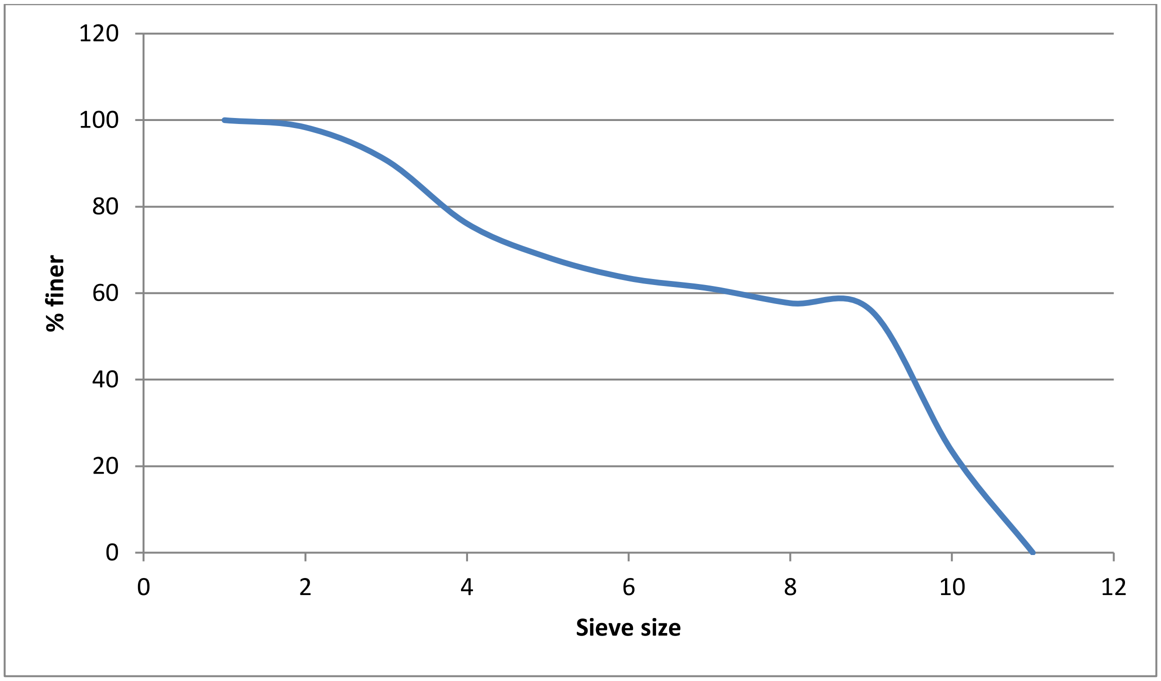


Fig 11

Discussion

The various soil parameters calculated frm the results obtained from the graph between grain size (mm) and % finer

- Effective size, D_{10} of soil =0.019mm
- Uniformity cofficient,C_U = 14.21
- Cofficient of curvature,c_c =1.949
- % of gravel =1.649%
- % of coarse sand=7.59%
- % of medium sand =27.29%
- % of fine sand = 40.017%
- % of silt and clay =23.442

4.1.2) plastic limit test

The following results were obtained on performing plastic limit test on the concerned soil:-

SI No.	Wt. of container(g)	Wt. of container +wet soil(g)	Wt. of container +dry soil(g)	Moisture Content (%)
1	18.8	27.8	26.2	21.6%
2	17.0	28.5	26.4	22.10%

(table 3)

- Plastic limit of soil was taken as average of the two moisture contents.
- Plastic limit of soil was found to be 21.85%

4.1.3) Liquid Limit Test

The following results were obtained on performing liquid limit test on the concerned soil:-

No. of blows	Weight of container(g)	Weight of container+soil(g)	Wt. of container+dry soil(g)	Moisture content (%)
52	20.4	28.6	26.5	34.426
27	19.5 moisture content	29.1	26.3	41.176

(table 4)

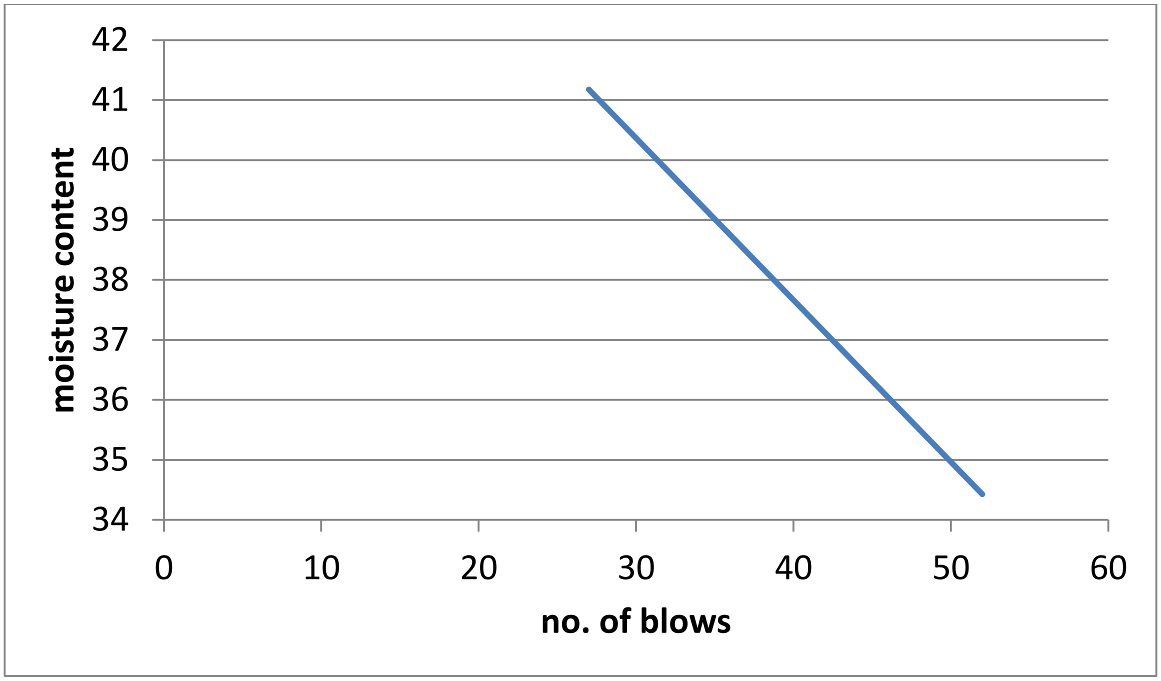


Fig 12

4.1.4) SOIL CLASSIFICATION

ON THE BASIS OF RESULTS OBTAINED FROM ABOVE EXPERIMENTS,SOIL WAS CLASSIFIED AS FOLLOWS:-

- Plastic index
- Plasticity index(I_p) = liquid limit(w_L)- plastic limit(w_p)

  I_p=41.9-21.85=20.05
- ``on plotting the plasticity index and liquid limit on the plasticity chart it is found that the intersecting point lies b\w hatched zone.
- Thus the soil is classified as clayey sand.

4.2)Modelling and Simulation

4.2.1) Analysis of Man-Made Slope

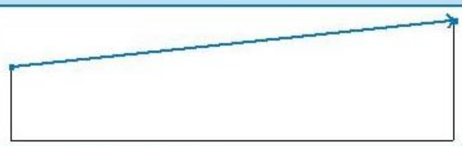
A man made slope was surveyed and analysis was carried out using Slope Stability suite of GEO5 Software.

4.2.2)Inputs

The following inputs were added:

Approx values of slope of parmar bhawan (JUIT).

Interface

No.	Interface location	Coordinates of interface points [m] x	z	x	z	x	z
1		0.00	0.00	30.00	3.15		

Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
1	Ramp		28.00	10.00	15.00

Uniform Distributed Load was added as follows:

Details of f(x)). Result of a sensitivity analysis has shown that the constant function yields FoS values almost the same as the half-sine function.

Surcharge

No.	Surcharge new	Surcharge change	Type	Type of action	Location z [m]	Origin x [m]	Length l [m]	Width b [m]	Slope α [°]	Magnitude q, q_1, f, F	Magnitude q_2	unit
1	Yes		strip	permanent	on terrain	x = 0.00	l = 30.00		6.00	1000.00		kN/m^2

Earthquake
Horizontal Seismic Coefficient, $K_h = 0.1$
Vertical Seismic Coefficient, $K_v = 0.05$

4.2.3)Analysis

Circular Slip Surface method (Fellenius/Petterson Method)

Analysis 1 (stage 2)

Circular slip surface

Slip surface parameters							
Center :	x =	-54.38	[m]	Angles :	α_1 =	5.79	[°]
	z =	627.85	[m]		α_2 =	6.20	[°]
Radius :	R =	630.10	[m]				
The slip surface after optimization.							

Slope stability verification (Fellenius / Petterson)

Sum of active forces : F_a = 880.47 kN/m

Sum of passive forces : F_p = 1757.60 kN/m

Sliding moment : M_a = 554786.07 kNm/m

Resisting moment : M_p = 1107465.69 kNm/m

Factor of safety = 2.00 > 1.50

Slope stability ACCEPTABLE

Polygonal Slip Surface method (Sarma Method)

Analysis 2 (stage 2)

Polygonal slip surface

Coordinates of slip surface points [m]									
x	z	x	z	x	z	x	z	x	z
5.80	0.61	6.03	0.62	7.10	0.72	15.49	1.51	27.88	2.93
The slip surface after optimization.									

Slope stability verification (Sarma)

Factor of safety = 1.65 > 1.50

Slope stability ACCEPTABLE

Methods	**Type of slip surface**	**Factor of safety**
Bishop	**Circular slip**	**2.10**
Fellenius **Sarma** **Janbu** **Spencer** **Morgensten-price**	**Polygonal slip**	**2** **1.65** **1.66** **1.65** **1.67**

Chapter 5

Conclusion of soil nailing

The approach of designing soil nails to support the earth pressure generated by the liquefied loose fill has led to the use of steeply inclined nails. While the nail force can be mobilised by the unbalanced earth pressure acting on the grillage facing, the steep nail inclination results in significant slope movement especially when sliding failure prevails, for instance, due to liquefaction confined to a thin layer of loose fill at depth. The steeply inclined nail arrangement needs to be used in conjunction with vertical nails or other form of fixity (e.g. embedded concrete footing) at the slope toe and even with such provisions the slope movements that could develop in the event of an interface liquefaction failure could be considerable. By constructing a hybrid nail arrangement (i.e. sub-horizontal nails at the upper part and steeply inclined at the low part) slope movement could be reduced even when toe fixity is absent. Where the hybrid nail arrangement is adopted, vertical nails are not required at the slope toe. It is recommended that the hybrid nail arrangement be adopted as far as possible to enhance the robustness of the ground-nail-facing system

SOIL PARAMETERS	VALUE
Effective size, D_{10} of soil	0.019mm
Uniformity cofficient,C_U	14.21
Cofficient of curvature,c_c	1.949
% of gravel	1.649%
% of coarse sand	7.59%
% of medium sand	27.29%
% of fine sand	27.29%
% of silt and clay	23.442%
Plastic limit	21.85%
liquid limit	41.9

Conclusion of GEO5

GEO5 software is an easy -to -use suite designed to solve various geotechnical problems. Slope Stability Analysis can be done easily and efficiently using Geo5.
In the Stability Analysis suite we can easily draw the interface and add soil accordingly. Embankments, earth cuts, rigid body, surcharge and water table can also be introduced accordingly. Earthquake settings can also be done if the slope analyzed is present in an earthquake zone.
Construction can be done in different stages so that it's easy to edit later. Analysis can be done using either circular slip surface or polygon slip surface and the software can optimize the slip surface.
If the slip surface fails i.e. factor of safety is less than 1.5, the software gives the option of adding either anchors or soil reinforcements. Also we can design pile wall using earth cut and embankments. Then again analysis can be performed to see if the slope is stable now or not.
Slope Stability Analysis can also be performed using FEM suite of GEO5. In FEM module generates mesh and analyses the slope by reducing soil parameters "c" and "φ".
After optimization of circular slip surface, by seeing the results, we can say that Fellenius/Petterson method is safer as it gives least factor of safety. While in case of polygonal slip surface Janbu method is safest.
If our soil profile allows the use of FEM module of GEO5 then it is better than the analytical methods which are used in Slope Stability, FEM is better because it does not make any assumptions like the analytical methods.
In all Geo5 is easy to learn and use. Both Slope Stability and FEM program of it are efficient and also provides some counter measures to make the slope stable.

Future Scope

Stability of slopes, natural and manmade, is particularly important for any hill road. Disturbance to slope can occur due to erosion caused by rain-fall and run-off and consequent slides.
Geo5 Software is a vast software and only two of its program are used in this project i.e. Slope Stability and Foundation Design that would be used in next semester. The understanding of the full software and using it in Design and Analytical problems is wide scope of research.
Using software's in solving geotechnical problems is itself a wide scope of research .

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LIST OF SYMBOLS

Symbols	Description
α	Inclination of slice base
β	Inclination of slice top
θ	Angle subtended by the slip circle at centre
α_1	Inclination of the back slope
β_1	Slope face angle with respect to the vertical
θ_1	Inclination of failure plane
φ	Soil effective angle of internal friction
C	Soil effective cohesion
L	Length of failure plane
W	Weight of the sliding mass
Q	Surcharge load
N_F	Normal force on failure surface
S_F	Shear force on failure surface
R	Radius of circular slip surface
S_u	Undrained shear strength
$\bar{x}$	Horizontal distance between circle centre and the centre of the sliding mass
R_c	Perpendicular distance from the circle centre to shear force
L_{arc}, L_{chord}	Lengths of the circular arc and chord defining the failure surface
δ	Angle of line of action of surcharge with vertical
b	Width of slice\