# “STABILITY OF SLOPE USING

# SOIL NAILING TECHNIQUE”

A

PROJECT

***Submitted in partial fulfillment of the requirements for the award of the degree of***

**BACHELOR OF TECHNOLOGY**

**IN**

**CIVIL ENGINEERING**

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# CERTIFICATE

This is to certify that the work which is being presented in the project title **"STABILITY OF SLOPE USING SOIL NAILING TECHNIQUE"** in partial fulfillment of the requirements for the award of the degree of Bachelor of technology and submitted in Department of Civil Engineering, Jaypee University of Information Technology, Waknaghat is an authentic record of work carried out by **Aweshesh Jindal (121608) and Shubham Sharma (121654**) during a period from July 2015 to June 2016 under the supervision of **Mr. Saurabh Rawat,** Assistant Professor, Deptt. of Civil Engineering, Jaypee University of Information Technology, Waknaghat.

The above statement made is correct to the best of my knowledge.

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Aweshesh Jindal (121608)

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# CONTENTS

# ABSTRACT

Soil nailing has emerged as an effective technique of ground reinforcement for improving the engineering properties of the soil. It has emerged as a cheaper technique for stabilizing the steep slopes. Such techniques like soil nailing and dowelling, have received tremendous development over the last 25 years.

The project report presents a study on the stability of slopes by using soil nails. In this project report, an attempt has been made to study the soil nailing technique and carrying out the technique in the laboratory by preparing slope models with slope angles of 45$^o$ and 60$^o$. Unreinforced and reinforced slope models have been prepared with the sand and tested under UTM for increasing surcharge. Load vs settlement behavior, failure patterns and nail forces along the nails have been studied under the surcharge loading. It is observed that reinforced slopes have nails have higher load carrying capacity than unreinforced slopes. Moreover, a deep surface failure is obserbed for the slopes. The lower nails are found to bear more forces.

*Keywords: Soil nailing, unreinforced slope, reinforced slope, slope angles, reinforcement forces.*

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# LIST OF ABBREVIATIONS AND SYMBOLS

| | |
|---|---|
| PSD | Particle Size Distribution |
| LEM | Limit Equilibrium Method |
| FEM | Finite Element Method |
| FOS | Factor of Safety |
| UTM | Universal Testing Machine |
| $\phi$ | Angle of internal friction |
| $\gamma$ | Unit weight |
| $\varepsilon$ | Axial strain |
| $\mu$ | Microns |
| $\sigma$ | Normal Stress |
| $\tau$ | Shear stress |

# CHAPTER 1

# INTRODUCTION

## 1.1 General

Ground (Soil) reinforcement is defined as a technique to improve the engineering characteristics of soil. Reinforced earth consists of a compacted soil mass within which reinforcing elements or membranes , usually in the form of horizontal strips of metal (such as galvanised steel, stainless steel or aluminium alloys), rods of metals, wire grids, fibre glass strips/rods, bamboos or geo textiles, are embedded. The essential feature of the reinforced earth is that friction develops between compacted layers of the earth and the reinforcing elements. The soil transfers the forces built up in the earth mass to the reinforcement by means of friction. Thus, tension is developed in reinforcement.

The technology of ground reinforcement has been familiar to mankind throughout the civilization. Ingenious techniques have been known to be applied to ancient structures as far back as 2100 B.C. which involve layering of materials bearing tensile strength inter bedded with compressive materials like soil and gravel to form a reinforced composite. Since then many other types of ground improvement and reinforcing techniques have arose, including that of soil nailing. Ground reinforcement techniques may be classified broadly into two main categories (Schlosser and Juran, 1979):

1. In-situ soil reinforcement

2. Remoulded soil reinforcement

The reinforced earth technique mentioned above follows the remoulded soil reinforcement method where the soil is built up together with the reinforcement, which may comprise of geo-grids, geo-textiles or steel strips. However, since many geotechnical applications require reinforcement that needs to be placed in-situ, such as slopes or excavated walls, rather than built up structures, such as embankments. The former (placed in-situ) category has been developed in recent times to be an important aspect of ground reinforcement. Such techniques like soil nailing and dowelling, have received tremendous development over the last 25 years.

## 1.2 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).

Fig. 1.1 Soil nail with centralizers

(Ref: www.williamsform.com )

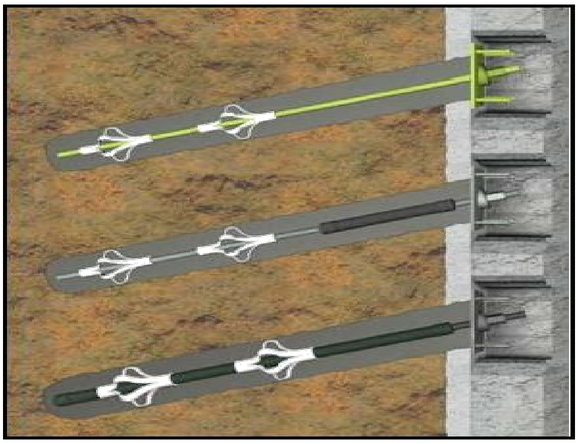


Fig. 1.2 Cross-section of a grouted soil nailed wall

(Ref: www.williamsform.com )

Fig. 1.3 Application of soil nailed wall in highways.
(Ref: www.classes.ce.ttu.edu )

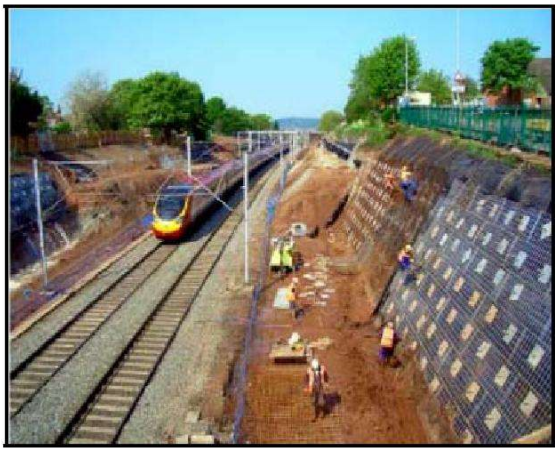


Fig. 1.4 Application of soil nailed wall in railways.
( Ref: www.geofabrics.com )

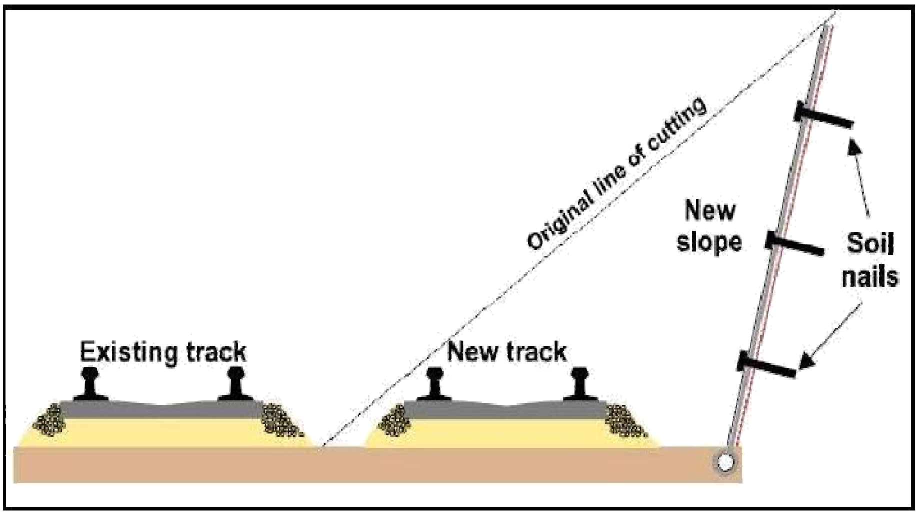



Fig. 1.5 Soil nailing in railway construction for laying of new tracks adjoining to existing ones.
(Ref: www.geofabrics.com )

## 1.3 Description of Soil Nailing Technique

Soil nailing is a method of slope stabilization or ground improvement that involves the use of passive inclusions, usually steel bars (known as soil nails), to reinforce in-situ retained ground. Its installation is progressive and is carried out simultaneously with soil excavation in front of the retained wall. This takes place in a series of successive phases as shown in Fig 1.5 (US Federal Highway Administration, 1999).

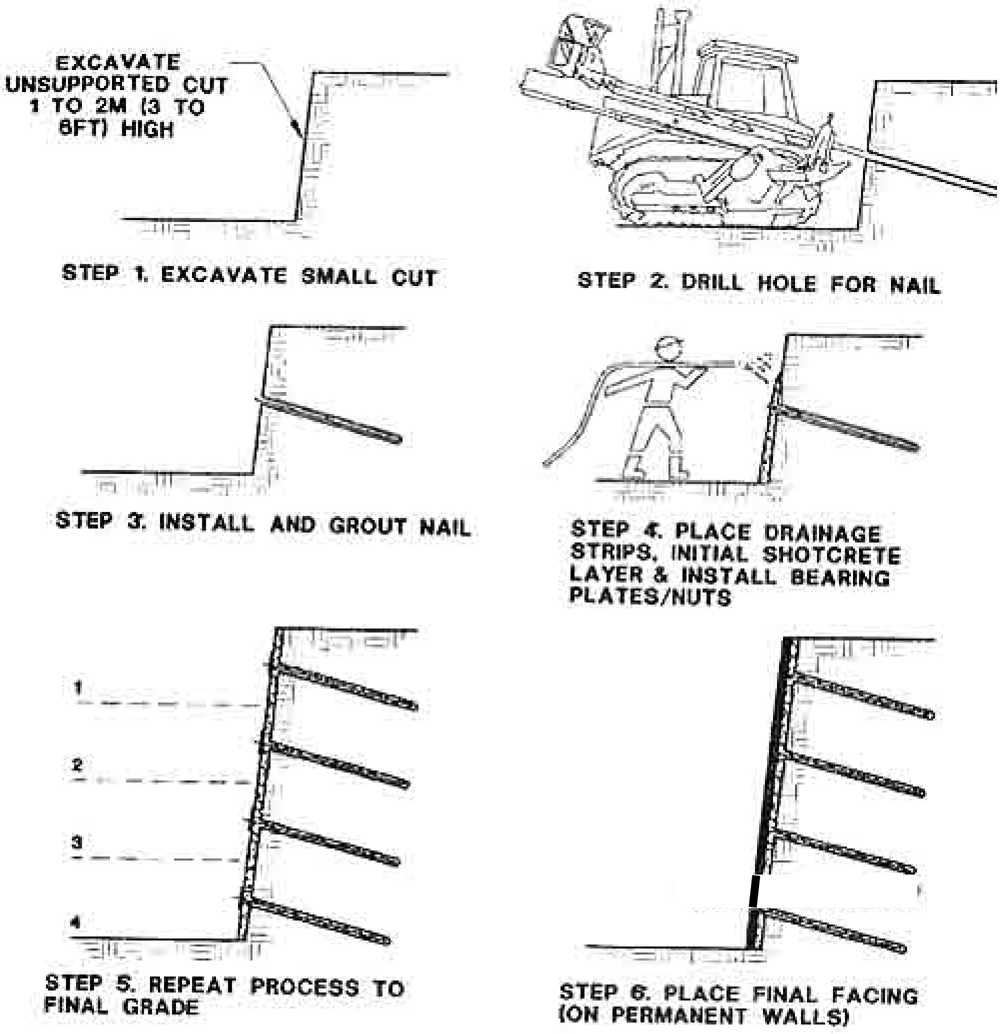



Fig.1.6 Steps involved in construction of a soil nailed slope
(Ref: US Federal Highway Administration, 1999)

The following are the typical sequence to construct a soil nail wall using the drill and grout method of nail installation:

- Excavate Initial Cut to a depth slightly below the first row of nails, typically about 1 to 2m depending upon the ability of the soil to stand unsupported for a minimum of 24 to 48 hours.
- Drill Hole for Nail at predetermined locations to a specified length and inclination using a drilling method appropriate for the ground.
- Install and Grout Nails. The nails which are commonly 19 to 35mm bars are inserted into the hole and the drill hole is filled with cement grout to bond the nail bar to the surrounding soil.

- Place Drainage System. A 400mm wide prefabricated synthetic drainage net placed in vertical strips between the nail heads on a horizontal spacing equal to that of the nails, is usually installed against the excavation face before shotcreting to provide drainage.
- Place Construction Facing which consist of mesh reinforced wet-mix shotcrete layer and Bearing Plates which consist of a steel bearing plate and securing nut at each nail head.
- Repeat the Process to Final Grade. Repeat the sequence of excavate, install nail and drainage system and placing of construction facing until the final grade is achieved.
- Place Final Facing. For architectural and long term structural durability reasons, the final facing (commonly the CIP concrete facing) is placed.

The reinforcement principle of the soil nailing method may seem to resemble that of the reinforced earth method. However due to the method of installation, the soil nailing method produces a very different behaviour from that of reinforced earth which is generally marked by the point of maximum displacement. Soil nailing produces greater displacements at the top of the excavation while reinforced earth show larger displacements near the bottom. This shows that the method of installation has a great impact on the mobilization of forces within the system and should be properly understood with the properties and geometry of the materials involved to gain an understanding of the overall behaviour of the system.

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 exc avation. 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.

## 1.4 Mechanism of Soil Nailing behaviour in reinforcement of Soil Structure

The purest form of soil nailing, without the use of any pretension or preloading and connected with a weak facing, acts in response to the deformation of the system. This is because the nails are placed as passive inclusions and offers no support to the system when initially installed. However, with excavation of the soil in front of the retained soil, the soil moves in active response to the unloading and undergoes deformation. The deformation of the soil transfers the loading to the nails. Two possible types of interaction are developed.

1. The primary action is the interaction of shear stress along the nail-soil interface, which is subsequently transferred into the nail as tensile forces.

2. The secondary action is the action of shear and bending, which is developed as a result of passive pressure of the earth along the nail. This is observable when shear zones in the soil develop to form active and passive zones.

When loading of the system takes place, the soil nailed wall may approach failure mainly by either breakage due to insufficient structural capacity of the nail, pullout of nail due to lack of adherence at the nail-soil interface, or global instability of the retained slope or structure (external failure). There may be other forms of failure locally due to excessive excavation depth prior to installation of subsequent nail or piping of soil. In general, they may be summarized into four forms:

1. Instability during excavation phases
2. Overall sliding of the reinforced mass
3. Lack of Friction between soil and nails
4. Breakage of the nails

Based on these failure modes, design may be made using limit equilibrium methods to find out safety against different modes of failure. However, the behaviour of soil nails is also subjected to the many variations in design specifications of geometry and layout, coupled with the variation of site and materials used make for a very complicated design process.

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.

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, 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.

## 1.5 Chapter outline

The Project Report has been presented in five chapters. Brief details about each chapter are as follows:

**Chapter 1**: The first chapter of project report provides a brief introduction to the concept of soil nailing and the use of finite element method in stability analysis of nailed slopes.

**Chapter 2**: The second chapter deals with the review of available literature on analytical and experimental studies. The objective and scope of the present study are also mentioned in this chapter.

**Chapter 3**: The third chapter discusses about materials, setup and methodology. In the chapter, preliminary tests carried out on the soil are described. This chapter also discusses the step-by-step preparation of the model slopes with and without nails. Test procedure on slopes and strain measurement on nails is also described in this chapter.

**Chapter 4:** The fourth chapter deals with the results and discussions. Various experiments on soil are carried out and their results are compiled and discussed in this chapter. Also, results of slope failure, load vs. settlement curves for different slope angles and nail force variation with time is also described in this chapter.

**Chapter 5:** The fifth chapter is last chapter and deals with the conclusions derived based on the experimental results.

# CHAPTER 2

# LITERATURE REVIEW

## 2.1 General

This chapter presents the review of developments of the theoretical concepts, analysis methods and model analysis done by various researchers leading to the present design methodology of analysis of unreinforced slope.

## 2.2 Analytical study

Soil – nail interaction is a complex aspect of soil mechanics analysis. A solution for the interaction between stiff reinforcement and soil, which is important for soil – nailing design must be understood. In order to analyze this interaction researchers adopted a number of methods.

A kinematic limit analysis for the design of nailed soil structures is developed by *Juran et al. (1990)*. Design of soil-nailed systems has been traditionally done using slope-stability analysis methods. These methods have been developed to incorporate the effect of the available tension and shear resistance of the passive reinforcements on the slope stability. However, they provide only a global safety factor. In this design methodology, a kinematical limit analysis design approach that provides a rational estimate of maximum tension and shear forces mobilized in each reinforcement is presented. This design method enables one to estimate nail forces and to evaluate local stability at the level of each nail. The design approach is also used to analyze the various failure mechanisms observed on model walls and predicted critical model heights are compared with experimental results. This method is used to evaluate the effect of the main design parameters such as inclination and bending stiffness of the nails, embankment slope, facing inclination, soil strength characteristics on the magnitude and location of the maximum nail forces and on the structure stability. The design methodology was evaluated by both full-scale experiments and reduced-scale model tests. The proposed method provides a rational approach to predict the progressive pullout failure, which is generally induced by the sliding of the upper nails.

*Juran et al. (1990)* made comparative studies of various available design methods (that is, French method, Davis method and kinematic design method) to analyze nailed retaining structures. French and Davis methods ensure global stability of the structure where as the kinematic method is based upon the local stability, which can be more critical than the global stability. The Davis method yields a more conservative design scheme as compared to the French method. The kinematic and French methods include the effect of ground water and nail bending stiffness on the stability of the structures.

The importance of some variables like the shape of the assumed failure surface, wall height, inclination and length of nails and global stability of nailed soil walls is reported by *Long et al. (1990)*. He made a comparison of results obtained with circular, bilinear and three-part wedge failure surfaces to show that the three-part wedge is the least constrained failure surface. The estimated factor of safety increases with the inter slice force inclination. For low values of nail capacity, all the three failure surfaces predict similar factor of safety. The influence of the soil nails becomes more important when the nail capacity factor exceeds 1.25 and the failure surface becomes more curved. The higher curvature increases the difference between the stability values computed by using circular and non-circular failure surfaces. It is found that the number of rows of nails can significantly influence the shape of the failure surface, its location and the factor of safety. They concluded that the factor of safety increases with the increase in the number of rows of nails.

It is observed that for a standard Coulomb material, the logarithmic spiral slip line is the only kinematically admissible solution and in the reinforced soil system, the vertical forces transferred to the foundation soil were relatively small as compared to the lateral earth pressure *Juran et al. (1992)*. The close agreement between the kinematical limit analysis and the test data from instrumented full-scale structures appeared to strongly justify the engineering assumptions used in their kinematical analysis. The kinematical limit equilibrium analysis and the model test results illustrated that the mobilization of the bending stiffness did not improve the structural stability. It resulted in a significant decrease of the structural stability and should therefore be considered in the design. The bending stiffness effect depends upon a number of parameters including nail stiffness, nail inclination, soil stiffness and boundary conditions. Therefore, it is necessary to develop an appropriate analysis procedure to predict the bending stiffness for different design schemes and experimentally verify the same. The contribution of bending stiffness in comparison to the bending stiffness due to the axial force in nail is very small towards the improvement in soil shearing resistance. For simplification of design analysis by the effect of bending stiffness should be ignored. This is suggested by *Jewell and Pedley (1992)* on the basis of which they designed equation both for grouted and non grouted nails.

*Liang et al. (1998)* developed a displacement based approach for the stability analysis of anchor-reinforced slope. The effects of reinforcement on the slope stability due to different parameters, such as soil properties, anchor characteristics and anchor-soil interface properties are discussed. Interaction between the anchor and the soil at their interface is due to relative movements between them. From the results of progressively mobilized anchor forces, the allowable soil movement for the given anchor parameters could be determined. Also, for a given anchor reinforced slope, the anchor working limit is generated along a vertical section of the slope, which controls the anchor resistance to the slope and limits the potential soil movement. They suggested that the anchor dimension and inclination should be properly designed to achieve the maximum efficiency.

*Griffiths and Lane (1999)* stated that Finite Element method in conjunction with an elastic-perfectly plastic (Mohr-Coulomb) stress-strain method has been found to be a reliable and robust method for assessing the factor of safety of slopes. One of the main advantages of the FE approach is that the factor of safety emerges naturally from the analysis without the user having to commit to any particular form of the mechanism a priori. The FE approach for determining the factor of safety of slopes has satisfied the criteria for effective computer-aided analysis. The widespread use of this method should be seriously considered by geotechnical practitioners as a more powerful alternative to traditional limit equilibrium methods.

*Nadher Hassan Al-Baghdadi* in his research work, Stabilization of Earth Slopes using Soil Nailing presented a parametric study using commercial computer program "Slide 6", which utilize different methods for solving slope stability problem. Bishop method was used to analyze un nailed and nailed slopes with granular soil, different slope heights and angles. . Some of nails parameters were studied such as, positions of nail, length of nail, angle of nail inclination, and nail spacing. He reported that the optimum length of nails, with spacing larger than to cause block effect, related in obvious manner with height and angle of slope. In the case of spacing which can cause block effect, the increment in length of nails causes increment in F.S. because the failure surface cannot pass through the nails group so it pushed behind the block (nails group), which cause increasing in failure surface length. The best angle of nail inclination is ranged between (5 to 25 degrees) below the horizon. Also, a relationship could be obtained between the nail angle and slope angle and it could be useful in design procedure.  The spacing of nail was found to be (1 m) to give the best improvement of F.S.

## 2.3 Experimental Study

Documented failures of soil – nailed systems for the most part are non – existent in the literature, and a few full – scale and model testing research programs has been conducted. (*Kitamura et al., 1988)* studied the effect of steel bar reinforcement in vertically loaded reinforced sand slopes. They concluded that the maximum and minimum effects of the reinforcement are obtained for the horizontal and the inclined upward placement of the nails, respectively. The inclined downward reinforcement was less effective compared to the horizontal one.  They found that the bending and the shearing resistance of steel bars did not contribute significantly to the reinforcement effect. The largest increase of axial stress at each loading step is located between the slip surfaces of reinforced and unreinforced slopes.

A number of small-scale model tests of reinforced slopes are conducted by *Gutierrez and Tatsuoka (1988)* to measure the tensile reinforcement forces and strain fields. They analyzed the results by limit equilibrium method modifying the ordinary method of slices by incorporating the inclined reinforcement forces and the inclined footing pressures. They concluded that their method was simple and accurate. Similarly a series of model tests are performed by *Hayashi et al. (1990)* to investigate the failure mechanism of steel bar reinforced cut slopes. A modified Bishop's design method is developed by considering the tensile force and the shear resistance mobilized in individual reinforcement.

*Zodinpuii (2011)* did model tests on soil nailed slopes by applying a gradually increasing load. Hollow aluminium pipes are used as nails for reinforcing the slopes. The increase in the load carrying capacity of the reinforced slopes as compared to that of the unreinforced slopes is found to increase substantially. The effect of the inclination of the nails is also studied by varying the nail inclinations (i= 0º, 15º and 30º) for each slope angle (β = 30º, 45º and 60º) respectively. The nail with 0º inclination to the horizontal gave the maximum improvement in the load carrying capacity of the slopes followed closely by the 15º nail inclination. The maximum nail forces are found in the topmost row of the soil nails. The distribution of the forces along the nail is also found to be varying with the failure surface.

An Experimental Study on Horizontal and Inclined Nails in Sand carried out by *Javia, Vaibhav et al. (2013)* reported that that horizontally driven nails have more FOS than inclined nails in sand. It is concluded that the ultimate load increases with horizontal nailing as compared to inclined nailing. The experiment showed that the value of FOS is higher for θ= 0° i.e. using horizontal nails in excavations having vertical face in sand for driven nails. Earlier investigators (Juran and Elias, 1990: Sabahit et al., 1996: Patra, 1998, 2001: Swami Saran et al., 2005[6]) have also obtained similar results.

## 2.4 Summary of the Literature Review

Soil nailing has been emerged as an effective technique in recent years, widely used to stabilize the steep slopes. Its design is often controlled by the allowable deformation level especially when buildings and/or other underground facilities exit near the excavation. The majority of slope stability analyses performed in practice still use traditional limit equilibrium approaches involving methods of slices that have remained essentially unchanged for decades. The finite element method represents a powerful alternative approach for slope stability analysis which is accurate, versatile and requires fewer assumptions regarding the failure mechanism. Slope failure in the finite element model occurs `naturally' through the zones in which the shear strength of the soil is insufficient to resist the shear stresses (Griffiths and Lane, 1999). Therefore a three-dimensional finite element model should be developed for the deformation analysis of nailed soil structures. In this model, the soil nonlinearity, the soil–nail interaction and the staged construction should be considered. The comparison between the kinematical limit equilibrium and the model test has been carried out which resulted in study of parameters like nail stiffness, nail inclinations, soil stiffness and boundary conditions (Juran et al., 1992). Hence a comparable study between the finite element and model testing can also be done in order to study the same parameters which are of great importance with designing schemes. It is also suggested from the literature that many researchers have found that the nail inclination, nail dimension, angle of soil friction and failure surfaces of the reinforced slopes are important factors for analysis and design [(Liang et al., 1998), (Patra, 2005), (Biswas et al., 2006)].

Experimental study of the soil nails has been well defined. The maximum and minimum effects of reinforcement were obtained for horizontal and inclined upwards placement of nails respectively. The bending and shearing resistance of steel bars did not contribute significantly to the effect of the reinforcement (Kitamura et al., 1988). The bending stiffness, tensile resistance of nail and applied surcharge loading were also investigated (Drabkin et al., 1995). The loading capacity of a slope decreases with an increase in the slope angles ($\beta$). The maximum load capacity is found in the slopes with 0º nail inclination for each respective slope angles. The forces on the nails are determined at the centre of the nails and the distribution of forces along the length of the nail is determined. It is observed that the maximum capacity is observed at the top rows of nails (Zodinpuii, 2011).

## 2.5 Objectives of present study

In the present study, based on the literature review, the following objectives were determined:

- Experimental setup of unreinforce sand soil slope with slope angles of $45^o$ And $60^o$.
- Study of failure surface of unreinforced soil slope under increasing surcharge load.
- Study of load versus displacement behaviour of unreinforced slopes.
- Study of failure surface of reinforced slope ($45^o$ and $60^o$) with smooth hollow soil nails.
- Study of load versus settlement behavior of reinforced slopes.
- Study of nail forces at the one-third length of the nail with increase in surcharge loading for reinforced slopes.

## 2.6 Future Scope of the work

In the present study stability of slopes using smooth surface hollow soil nails has been demonstrated. For more reseach work following work can be carried out:

- Study of failure mechnism, load-displacememt behaviour of soil slopes with different soil-nail inclination under increase in surcharge loading.
- Use of helical nails instead of smooth nails in soil slopes.
- Soil nails under short term and long term loading.
- Dynamic analysis of soil nailed slopes.

CHAPTER 3

# MATERIALS, SETUP AND METHODOLOGY

## 3.1 Materials

### 3.1.1 Soil

Soil used in the project work was collected from Nalagarh, District Solan (Himachal Pradesh). Soil was collected by method of disturbed sampling after removing the top soil at 150mm depth and transported in sacks to the laboratrory. Soil was air dried and sieved with IS sieve no. 4.75 as required for the laboratory preliminary tests and the modelling of the slope.

**Fig. 3.1** Soil sample used in modelling of slope

### 3.1.2 Perspex Sheet

Perspex sheet is a poly fibre transparent sheet. The sheet used for the project work is 4mm thick manufactured by Asia poly industry. The sheet was bought from Bhagra steel sales,Taradevi near Shimla (H.P.). Cost of the sheet was Rs. 197 per square feet.

Sheet was used for the purpose of constructing a rectangular box for the model in which slope was to be laid.

**Fig. 3.2** Perspex sheet

## 3.2 Methodology

### 3.2.1 Preliminary testing on Soil

Some tests on the natural soil were carried out in accordance with the procedures outlined in IS 2720. The following tests were carried out on the Soil:

**1. Particle size distribution analysis (Sieve Analysis)**

A sieve analysis is a practice or procedure used to assess the particle size distribution of a granular material. A sieve analysis can be performed on any type of non-organic or organic granular materials including sands, crushed rock, clays, granite, feldspars, coal, soil, a wide range of manufactured powders, grain and seeds, down to a minimum size depending on the exact method.

Sieves are wire screens having square openings. Different standards such as Indian, British and US, designate the sieves differently. According to Indian standard Code IS: 460-1962(Revised), the sieve number is the mesh width expressed in mm for large sizes and in microns for small sizes; that is, a sieve with a mesh opening of 4.75mm is designated as a 4.75mm sieve and a 500 microns sieve refers to a sieve with a mesh opening of 0.500mm. Sieves vary in size from 80mm to 75 microns. The representative soil sample is separated into two fractions by sieving through the 4.75mm IS sieve. The fraction retained on the sieve (+ 4.75mm) is called the gravel fraction which is subjected to coarse sieve analysis. A set of sieves of size 80mm, 20mm, 10mm, 4.75mm is required for further fractioning of gravel fraction. The material passing 4.75mm sieve (- 4.75mm) is further subjected to fine sieve analysis if it is sand or to a combined sieve and sedimentation analysis if silt and clay sizes are also present. The set of IS sieves for fine sieve analysis consist of 2mm, 1mm, 600μ, 425μ, 212μ, 150μ and 75μ sieves. For a particular soil, all the sieves may not be required. Only such sieves are selected as would give a good grain size distribution curve.

In the dry sieve analysis, a suitable quantity of pulverized dry soil of known weight (about 500g) is taken and is sieved through a selected set of sieves arranged according to their sizes, with the largest aperture sieve at the top and the smallest aperture sieve at the bottom. A receiver is kept at the bottom (called 'pan') and a lid is placed on the topmost sieve of the stack. The amount of shaking depends upon the shape and number of particles. However, ten minutes of shaking by a mechanical shaker is usually sufficient. The amount of soil retained on each sieve is weighed to the nearest 0.1g. On the basis of the total weight of sample taken and the weight of soil retained on each sieve, the percentage of the total weight of the soil passing through each sieve (also termed as per cent finer than) can be calculated as below:

$$\text{\% retained on a particular sieve} = \frac{weight\ of\ soil\ retained\ on\ tha\ \ sieve}{total\ weight\ of\ soil\ taken} \times 100$$

Cumulative % retained = sum of % retained on all sieves of larger sizes and the % retained on that particular sieve

Percentage finer than the sieve under reference = 100% - Cumulative % retained

Fine sieving is performed on soil sample in laboratory and corresponding graph between % finer and sieve diameter (on semi log scale) was plotted.

A soil sample may be either well graded or poorly graded . A soil is said to be well graded when it has good repersentation of particle of all sizes. On the other hand, a soil is said to be poorly graded if it has an excess of certain particles and deficiency of others, or if it has most of particles of about the same sizes ; in latter case it is known as a uniformly graded soil.A curve with a flat portion repersent a soil in which some intermediate size particles are missing. Such a soil is known as gap graded or skip graded.

For coarse grained soil, certain particles sizes such as D10 , $D_{30}$, $D_{60}$ are important

The $D_{10}$ repersents a size, in mm such that 10% of particles are finer than this size. Similarly, the soil particles finer than $D_{60}$ size are 60% of total mass of the sample.The size $D_{10}$ is sometimes called the effective size or effective diameter. The unifomity cofficient $C_U$ (or cofficient of uniformity) is a measur of particle-size range and is given by the ratio of $D_{60}$ and $D_{10}$ sizes :

$$C_u = \frac{D_{60}}{D_{10}}$$

Similarly, the shape of the particle size curve is represented by the co-efficient of the curvature $C_c$, given by,

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$$

For a uniformly graded soil, $C_u$ is nearly unity. For the well graded soil, $C_c$ must be between 1 to 3 and in addition $C_u$ must be greater than 4 for gravels and 6 for sands.

Sieve analysis (fine sieving) was carried out on the soil sample to study the particle size characteristics. The readings obtained and then corresponding graph plotted between **percent finer** and **sieve diameter** (on log scale) as shown in fig.3..

Weight of the soil taken initially = 500 g.

**Table 3.1** Sieve analysis readings

| *Sieve No.* | *Weight retained in each sieve (g)* | *% retained on each sieve* | *Cumulative % retained on each sieve* | *% Finer (100% - cum%)* |
|---|---|---|---|---|
| 10 mm | 0 | 0 | 0 | 100 |
| 4.75 mm | 0 | 0 | 0 | 100 |
| 2 mm | 3.9 | 0.78 | 0.78 | 99.22 |
| 1 mm | 21.55 | 4.31 | 5.09 | 94.91 |
| 600 μ | 132.75 | 26.55 | 31.64 | 68.36 |
| 425 μ | 128.45 | 25.69 | 57.33 | 42.67 |
| 300 μ | 98.55 | 19.71 | 77.04 | 22.96 |
| 212 μ | 46.3 | 9.26 | 86.30 | 13.70 |
| 150 μ | 32.0 | 6.4 | 92.70 | 7.30 |
| 75 μ | 36.5 | 7.3 | 100 | 0 |

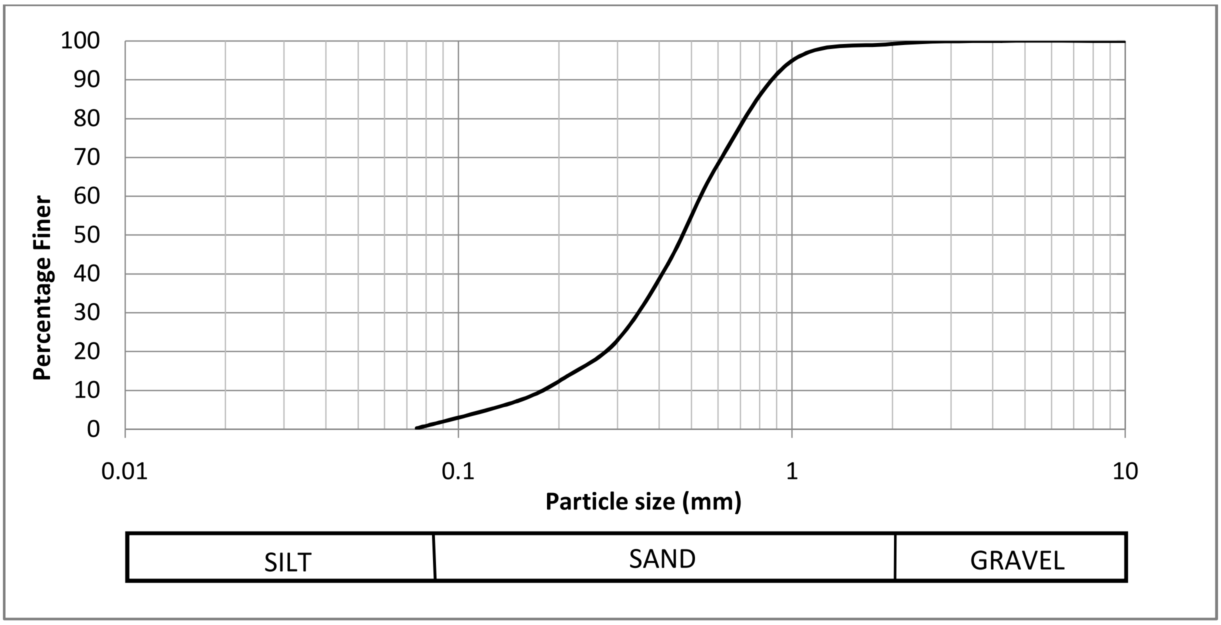



**Fig. 3.3** Grain size distribution curve

Now, the values of $D_{60}$, $D_{30}$, $D_{10}$ calculated from graph are:

$$D_{60} = 0.47 \text{ mm}$$

$$D_{30} = 0.35 \text{ mm}$$

$$D_{10} = 0.18 \text{ mm}$$

The unifomity cofficient $C_U$ (or cofficient of uniformity) is a measure of particle-size range and is given by the ratio of $D_{60}$ and $D_{10}$ sizes as,

$$C_u = \frac{D_{60}}{D_{10}}$$

$$C_u = \frac{0.47}{0.18}$$

Therefore, $C_u = 2.62$

Similarly, the shape of the particle size curve is represented by the co-efficient of the curvature $C_c$, given by

$$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$$

$$C_c = \frac{(0.35)^2}{0.18 \times 0.47}$$

Therefore, $C_c = 1.45$

**2. Specific gravity of soil (using pycnometer)**

Specific gravity, 'G' is defined as the ratio of the weight of a given volume of soil solids at a given temperature to the weight of an equal volume of distilled water at that temperature, both weights being taken in air. In other words, it is the ratio of the unit weight of soil solids to that of water:

$$G = \frac{\gamma_s}{\gamma_w}$$

where, $\gamma_s =$ unit weight of soil solids,

$\gamma_w$ = unit weight of water.

The Indian standard specifies 27°C as the standard temperature for reporting the specific gravity.Some qualifying words like: true, absolute, apparent, bulk or mass, etc. are sometimes added to the term 'specific gravity'. These qualifying words modify the sense of specific gravity as to whether it refers to soil particles or to soil mass. The soil solids have permeable and impermeable voids inside them, the permeable voids being capable of getting filled with water. If all the internal voids of soil particles (permeable and impermeable) are excluded for determining the true volume of solids, the specific gravity obtained is called absolute or true specific gravity. The apparent or mass or bulk specific gravity $G_m$ denotes the specific gravity of soil mass and given by

$$G_m = \frac{\gamma}{\gamma_w}$$

where, $\gamma$ = Bulk unit weight.

The value of G for a majority of soils lies between 2.65 and 2.80. Lower values are for coarse-grained soils. The presence of organic matter leads to very low values. Soils high in iron or mica exhibit high values. Table 3.1 gives typical values of G for different soils.

**Table 3.2** Typical values of G
(Ref: Ranjan, Gopal; Rao, A.S.R, "Basic and Applied Soil Mechanics")

| ***Soil type*** | ***Specific Gravity (G)*** |
|---|---|
| Clean sands and gravel | 2.65-2.68 |
| Silt and silty sands | 2.66-2.70 |
| Inorganic clays | 2.70-2.80 |
| Soils high in mica, iron | 2.75-2.85 |
| Organic soils | Quite variable : may fall below 2.0 |

Specific gravity of soil solids is determined using **pycnometer**. The procedure involves weighing first an empty, dried pycnometer bottle, say of weight $W_1$. Next, about 200-300g of soil dried in oven and cooled in a desiccator is placed in the pycnometer which is weighed again ($W_2$). The soil in the pycnometer is covered with water and stirred with glass rod. The pycnometer is gradually filled with water carefully removing the entrapped air. Vacuum pump is also sometimes used to expel the entrapped air. The pycnometer with the soil is filled up to the top with water and weighed, ($W_3$). Finally, the pycnometer is emptied completely, cleaned, dried and weighed after filling it with water up to the top, $W_4$.

Weight of dry soil, $W_s = W_2 - W_1$

Weight of water in observation (iii) = $W_3 - W_2$

Weight of water in observation (iv) = $W_4 - W_1$

Weight of water having the same volume as that of solids = $(W_4 - W_1) - (W_3 - W_2)$

$= (W_2 - W_1) - (W_3 - W_4)$

$$\text{Specific gravity of solids, } G = \frac{W2 - W1}{(W2 - W1) - (W3 - W4)} = \frac{Ws}{Ws - W3 + W4}$$

Specific gravity values are usually reported at 27$^o$C. If T$^o$ C is the test temperature, the specific gravity at 27$^o$C is given by:

$$G_{(27^oC)} = G_{(T^oC)} \times \frac{Specific\ gravity\ of\ water\ at\ T^oC}{Specific\ gravity\ of\ water\ at\ 27^oC}$$

Specific gravity of the soil sample used in the modeling of the slope was determined using pycnometer method. Calculations and results obtained as given below.

Weight of empty pycnometer ($W_1$) = 446.7 g

Weight of pycnometer filled with dry soil ($W_2$) = 652.9 g

Weight of pycnometer filled with soil and water ($W_3$) = 1342.0 g

Weight of pycnometer filled with only water ($W_4$) = 1212.8 g

Now, Weight of dry soil, $W_s = W_2 - W_1 = 652.9 - 446.7 = 206.2$ g

Weight of water having the same volume as that of solids = $(W_4 - W_1) - (W_3 - W_2)$

$= (W_2 - W_1) - (W_3 - W_4)$

$= W_s - W_3 + W_4$

$= 206.2 - 1342.0 + 1212.8$

$= 77$ g

Therefore, Specific Gravity of soil is given as,

$$G = \frac{W2 - W1}{(W2 - W1) - (W3 - W4)} = \frac{Ws}{Ws - W3 + W4} = \frac{206.2}{77}$$

$$G = 2.67$$

**3. Direct Shear test (for shear parameters)**

The concept of direct shear is simple and mostly recommended for granular soils, sometimes on soils containing some cohesive soil content. The cohesive soils have issues regarding controlling the strain rates to drained or undrained loading. In granular soils, loading can always assumed to be drained. A schematic diagram of shear box shows that soil sample is placed in a square box which is split into upper and lower halves. Lower section is fixed and upper section is pushed or pulled horizontally relative to other section; thus forcing the soil sample to shear/fail along the horizontal plane separating two halves. Under a specific Normal force, the Shear force is increased from zero until the sample is fully sheared. The relationship of Normal stress and Shear stress at failure gives the failure envelope of the soil and provide the shear strength parameters (cohesion and internal friction angle).

Direct Shear test was carried on the soil sample which was found out to be sand by grain size distribution to find out the shear parameters of the soil, i.e. Cohesion ( c ) and friction angle ( $\phi$). The test was carried out for three normal stresses of 0.5 kg/cm$^2$, 0.75 kg/cm$^2$ and 1 kg/cm$^2$. A graph between shear stress and normal stress was plotted in which y intercept of best fit line gave the cohesion value and slope of line gave the value of friction angle. Observations and calculations are as follows.

A) Normal Stress = 0.5 kg/cm$^2$

**Table 3.3** Direct shear test readings and calculations 1

| Dial gauge | Proving ring | Shear displacement (mm) | Shear Force (kg) | Strain | Shear stress (kg/cm$^2$) | Area (mm$^2$) |
|---|---|---|---|---|---|---|
| 0 | 0 | 0 | 0 | 0 | 0 | 36 |
| 20 | 1.2 | 0.2 | 3 | 0.003 | 0.083 | 36.12 |
| 40 | 1.5 | 0.4 | 3.75 | 0.006 | 0.103 | 36.241 |
| 60 | 1.6 | 0.6 | 4 | 0.01 | 0.11 | 36.363 |
| 80 | 1.7 | 0.8 | 4.25 | 0.013 | 0.116 | 36.486 |
| 100 | 1.7 | 1 | 4.25 | 0.0166 | 0.116 | 36.61 |
| 120 | 1.8 | 1.2 | 4.5 | 0.02 | 0.1225 | 36.734 |
| 140 | 1.9 | 1.4 | 4.75 | 0.023 | 0.128 | 36.86 |
| 160 | 1.9 | 1.6 | 4.75 | 0.026 | 0.128 | 36.986 |
| 180 | 1.9 | 1.8 | 4.75 | 0.03 | 0.127 | 37.113 |
| 200 | 2 | 2 | 5 | 0.033 | 0.134 | 37.241 |
| 220 | 2.2 | 2.2 | 5.5 | 0.036 | 0.147 | 37.37 |
| 240 | 2.2 | 2.4 | 5.5 | 0.04 | 0.146 | 37.5 |
| 260 | 2.3 | 2.6 | 5.75 | 0.043 | 0.152 | 37.63 |
| 280 | 2.3 | 2.8 | 5.75 | 0.046 | 0.152 | 37.762 |
| 300 | 2.4 | 3 | 6 | 0.05 | 0.158 | 37.894 |
| 320 | 2.5 | 3.2 | 6.25 | 0.053 | 0.164 | 38.028 |
| 340 | 2.6 | 3.4 | 6.5 | 0.056 | 0.17 | 38.162 |
| 360 | 2.7 | 3.6 | 6.75 | 0.06 | 0.176 | 38.297 |
| 380 | 2.7 | 3.8 | 6.75 | 0.063 | 0.175 | 38.434 |
| 400 | 2.7 | 4 | 6.75 | 0.066 | 0.175 | 38.571 |
| 420 | 2.8 | 4.2 | 7 | 0.07 | 0.18 | 38.709 |
| 440 | 2.8 | 4.4 | 7 | 0.073 | 0.18 | 38.848 |
| 460 | 2.7 | 4.6 | 6.75 | 0.076 | 0.173 | 38.989 |
| 480 | 2.7 | 4.8 | 6.75 | 0.08 | 0.172 | 39.13 |

B) Normal Stress = 0.75 kg/cm$^2$

**Table 3.4** Direct shear test readings and calculations 2

| Dial Gauge | Proving Ring | Shear Displacement (mm) | Shear Force (kg) | Area (mm$^2$) | Strain | Shear Stress (kg/cm$^2$) |
|---|---|---|---|---|---|---|
| 0 | 0 | 0 | 0 | 36 | 0 | 0 |
| 20 | 0.4 | 0.2 | 1 | 36.12 | 0.003 | 0.027 |
| 40 | 0.6 | 0.4 | 1.5 | 36.241 | 0.006 | 0.041 |
| 60 | 0.7 | 0.6 | 1.75 | 36.363 | 0.01 | 0.048 |
| 80 | 0.9 | 0.8 | 2.25 | 36.486 | 0.013 | 0.061 |
| 100 | 1.2 | 1 | 3 | 36.61 | 0.016 | 0.081 |
| 120 | 1.5 | 1.2 | 3.75 | 36.734 | 0.02 | 0.102 |
| 140 | 1.6 | 1.4 | 4 | 36.86 | 0.023 | 0.108 |
| 160 | 1.7 | 1.6 | 4.25 | 36.986 | 0.026 | 0.114 |
| 180 | 1.9 | 1.8 | 4.75 | 37.113 | 0.03 | 0.127 |
| 200 | 2.1 | 2 | 5.25 | 37.241 | 0.033 | 0.14 |
| 220 | 2.3 | 2.2 | 5.75 | 37.37 | 0.036 | 0.153 |
| 240 | 2.5 | 2.4 | 6.25 | 37.5 | 0.04 | 0.165 |
| 260 | 2.5 | 2.6 | 6.25 | 37.63 | 0.043 | 0.166 |
| 280 | 2.6 | 2.8 | 6.5 | 37.762 | 0.046 | 0.172 |
| 300 | 2.7 | 3 | 6.75 | 37.894 | 0.05 | 0.178 |
| 320 | 2.9 | 3.2 | 7.25 | 38.028 | 0.053 | 0.19 |
| 340 | 2.9 | 3.4 | 7.25 | 38.162 | 0.056 | 0.189 |
| 360 | 3 | 3.6 | 7.5 | 38.297 | 0.06 | 0.195 |
| 380 | 3 | 3.8 | 7.5 | 38.434 | 0.063 | 0.195 |
| 400 | 3 | 4 | 7.5 | 38.571 | 0.066 | 0.194 |
| 420 | 3.1 | 4.2 | 7.75 | 38.709 | 0.07 | 0.2 |
| 440 | 3.1 | 4.4 | 7.75 | 38.848 | 0.073 | 0.199 |
| 460 | 3.2 | 4.6 | 8 | 38.989 | 0.076 | 0.205 |
| 480 | 3.4 | 4.8 | 8.5 | 39.13 | 0.08 | 0.217 |
| 500 | 3.4 | 5 | 8.5 | 39.272 | 0.083 | 0.216 |
| 520 | 3.5 | 5.2 | 8.75 | 39.416 | 0.086 | 0.221 |
| 540 | 3.6 | 5.4 | 9 | 39.56 | 0.09 | 0.245 |
| 560 | 3.6 | 5.6 | 9 | 39.705 | 0.093 | 0.226 |
| 580 | 3.7 | 5.8 | 9.25 | 39.852 | 0.096 | 0.232 |
| 600 | 3.8 | 6 | 9.5 | 40 | 0.1 | 0.235 |
| 620 | 3.8 | 6.2 | 9.5 | 40.148 | 0.103 | 0.236 |
| 640 | 3.8 | 6.4 | 9.5 | 40.298 | 0.106 | 0.235 |

C) Normal Stress = 1.0 $kg/cm^2$

**Table 3.5** Direct shear test readings and calculations 3

| Dial Gauge | Proving Ring | Displacement (mm) | Shear Force (kg) | Area ($mm^2$) | Strain | Shear Stress ($kg/cm^2$) |
|---|---|---|---|---|---|---|
| 0 | 0 | 0 | 0 | 36 | 0 | 0 |
| 20 | 0.3 | 0.2 | 0.75 | 36.12 | 0.003 | 0.02 |
| 40 | 0.6 | 0.4 | 1.5 | 36.241 | 0.006 | 0.04 |
| 60 | 0.8 | 0.6 | 2 | 36.363 | 0.01 | 0.05 |
| 80 | 1.1 | 0.8 | 2.75 | 36.486 | 0.013 | 0.07 |
| 100 | 1.4 | 1 | 3.5 | 36.61 | 0.016 | 0.09 |
| 120 | 1.5 | 1.2 | 3.75 | 36.734 | 0.02 | 0.1 |
| 140 | 1.7 | 1.4 | 4.25 | 36.86 | 0.023 | 0.11 |
| 160 | 1.9 | 1.6 | 4.75 | 36.986 | 0.026 | 0.12 |
| 180 | 2 | 1.8 | 5 | 37.113 | 0.03 | 0.13 |
| 200 | 2.1 | 2 | 5.25 | 37.241 | 0.033 | 0.14 |
| 220 | 2.3 | 2.2 | 5.75 | 37.37 | 0.036 | 0.15 |
| 240 | 2.4 | 2.4 | 6 | 37.5 | 0.04 | 0.16 |
| 260 | 2.7 | 2.6 | 6.75 | 37.63 | 0.043 | 0.17 |
| 280 | 2.7 | 2.8 | 6.75 | 37.762 | 0.046 | 0.17 |
| 300 | 2.8 | 3 | 7 | 37.894 | 0.05 | 0.18 |
| 320 | 2.9 | 3.2 | 7.25 | 38.028 | 0.053 | 0.19 |
| 340 | 3.1 | 3.4 | 7.75 | 38.162 | 0.056 | 0.2 |
| 360 | 3.2 | 3.6 | 8 | 38.297 | 0.06 | 0.2 |
| 380 | 3.3 | 3.8 | 8.25 | 38.43 | 0.063 | 0.214 |
| 400 | 3.5 | 4 | 8.75 | 38.57 | 0.06 | 0.22 |
| 420 | 3.9 | 4.2 | 9.75 | 38.7 | 0.07 | 0.25 |
| 440 | 4.1 | 4.4 | 10.25 | 38.84 | 0.073 | 0.263 |
| 460 | 4.3 | 4.6 | 10.75 | 38.98 | 0.076 | 0.275 |
| 480 | 4.4 | 4.8 | 11 | 39.13 | 0.08 | 0.28 |
| 500 | 4.5 | 5 | 11.25 | 39.27 | 0.083 | 0.286 |
| 520 | 4.5 | 5.2 | 11.25 | 39.41 | 0.086 | 0.285 |
| 540 | 4.7 | 5.4 | 11.75 | 39.56 | 0.09 | 0.297 |
| 560 | 4.8 | 5.6 | 12 | 39.7 | 0.093 | 0.302 |
| 580 | 4.9 | 5.8 | 12.25 | 39.852 | 0.096 | 0.307 |
| 600 | 4.9 | 6 | 12.25 | 40 | 0.1 | 0.306 |
| 620 | 5 | 6.2 | 12.5 | 40.148 | 0.103 | 0.311 |
| 640 | 5.1 | 6.4 | 12.75 | 40.298 | 0.106 | 0.316 |
| 660 | 5.1 | 6.6 | 12.75 | 40.449 | 0.11 | 0.315 |
| 680 | 5.2 | 6.8 | 13 | 40.601 | 0.113 | 0.32 |
| 700 | 5.4 | 7 | 13.5 | 40.754 | 0.11 | 0.331 |
| 720 | 5.5 | 7.2 | 13.75 | 40.909 | 0.12 | 0.336 |
| 740 | 5.5 | 7.4 | 13.75 | 41.064 | 0.12 | 0.334 |
| 760 | 5.6 | 7.6 | 14 | 41.221 | 0.12 | 0.339 |
| 780 | 5.6 | 7.8 | 14 | 41.379 | 0.13 | 0.338 |
| 800 | 5.6 | 8 | 14 | 41.538 | 0.13 | 0.337 |

### 3.2.2 Modeling of the slope

After the preliminary tests on soil were done, the modeling of the slope in the box made by perspex sheet was carried out. Following are the steps involved in the modeling of slope from very beginning:

**1. Fabrication of the model box**

The 0.4cm thick Perspex sheet's dimensions were 240×120cm. A rectangular box like structure was needed, so perspex sheet was cut into pieces of required dimensions. The fabrication of the box was done at a welding store at Waknaghat, Distt. Solan (H.P.). The sides of sheet were joined using steel angles and fastening them with rivets.

Thus, a box with dimensions **50cm×22cm×35cm** was prepared in which slope was to be laid. The image in the figure 3.3 given below shows the model box.

**2. Laying of the slopes in layers in rectangular model box**

After the rectangular box was prepared, next step was to lay down the soil for the modelling of the slope.

Fisrt of all, the whole soil sample was sieved through 4.75mm IS sieve and the soil particles passing through that sieve was used in the modeling. Modeling of the slope was done in layers such that a **slope angles of 45$^o$ and 60$^o$** could be obtained. And this was done by constructing layers in form of steps and then putting more soil to form the slope.

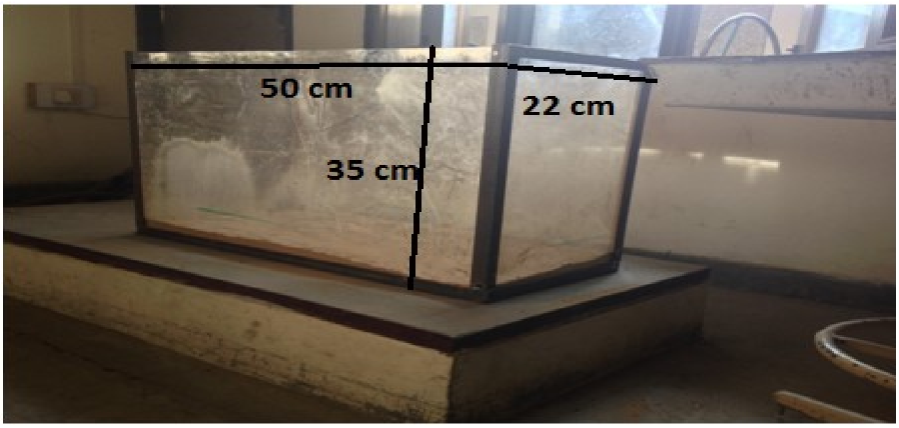



**Fig.3.4** Model box prepared by perspex sheet.

A total of 4 steps were formed by subsequent light compaction which will be shown in following images. Also, a very thin layer of tracer (a colour is used here) is also placed between every step of layer. Formation of each layer is described below in steps with images.

**Step 1. Laying down the first layer of soil.**

A layer 10 cm thick was made as the **base layer** completely along the length of model. Also, to maintain adequate water content in the soil to obtain the natural conditions, about 0.5 percent water of the total weight of soil was added in the soil and mixed thoroughly. The layer was formed by putting the soil in box and lightly compacting it after every 2cm height. In this similar manner , first layer or base layer was formed. After the layer of soil was ready, a very thin layer of tracer (Gulal) of 3mm was put above the top surface of base layer. Images for the above procedure is shown in the fig 3.5 and fig3.6.

**Fig.3.5** Base layer being laid in box.

**Fig.3.6** Top view of base layer.

**Step2. Laying down the second layer of soil.**

Now, the second layer that was constructed in the same manner as the first layer. This layer was smaller in length and was in accordance with the length required for making slopes of soil with slope angles as $45^{o}$ and $60^{o}$. Again, a 3mm layer of tracer was placed at the top of the second layer. Images showing formation of second layer are shown in fig 3.7 and fig 3.8.

**Fig.3.7** Second layer being laid.

**Fig.3.8** Front view of second layer

**Step3. Laying down the third layer of soil.**

The third layer was laid next in the same manner and the length was smaller than second layer and was according to the required length for the slopes of $45^{o}$ and $60^{o}$ each. A 3mm thick layer of tracer was put at the top of third layer. It is shown in fig 3.9.

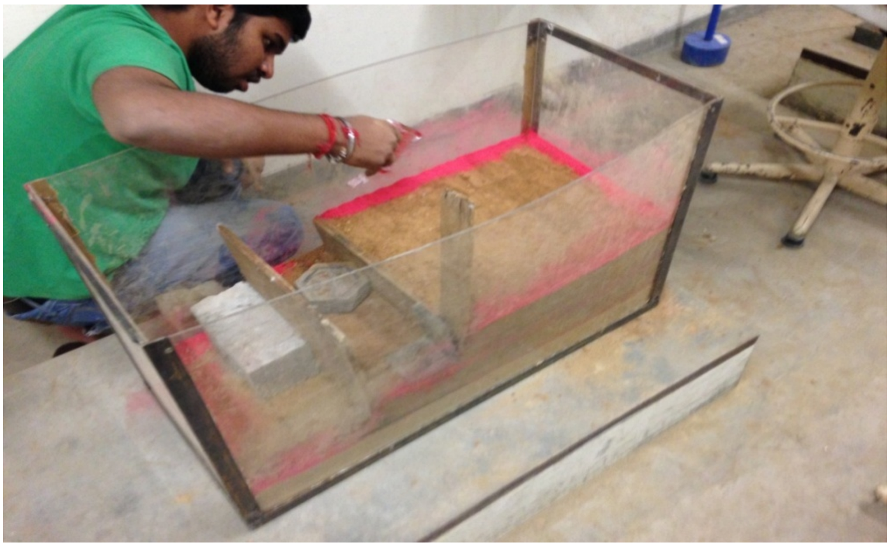


**Fig.3.9** Laying down of tracer over third layer.

**Step4. Laying down of fourth layer of soil (crest) of the slope model.**

The fourth layer or the crest of the slope was then laid of length 20cm was in accordance with the slope to be constructed. It was also prepared in the same way by compacting after every 2 cm and then getting upto the required height which is shown in the fig 3.10 and fig 3.11.

**Fig.3.10** Side view of all the layers with crest as top layer

**Fig.3.11** Top view of all the layers with crest as top layer

**Step5. Laying down the slopes (45$^o$ and 60$^o$).**

After the four layers of soil well compacted with layers of tracer between them were formed and a step or stair like structure was prepared, the next step was to lay down the slope. For laying the slope, soil was put into the model box over compacted layers starting from the third layer as the topmost layer is the crest of the slope model and then going down to the layers below upto base layer. Subsequent compaction was done while laying the slope and thus the slopes were prepared which is as shown in fig 3.12 and fig 3.13.

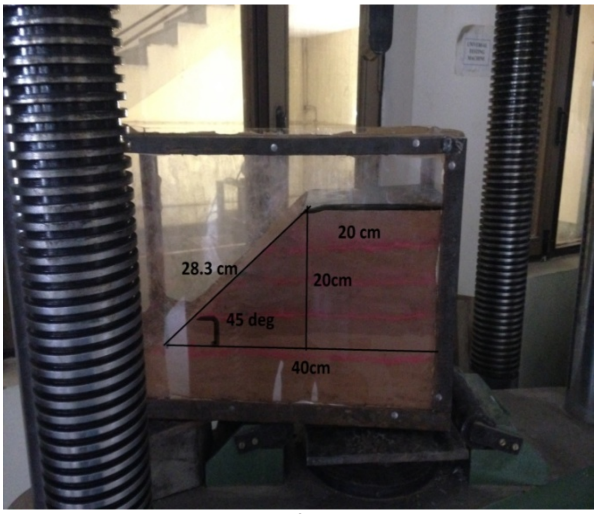



**Fig.3.12** Slope (45$^{o}$) in the model box.

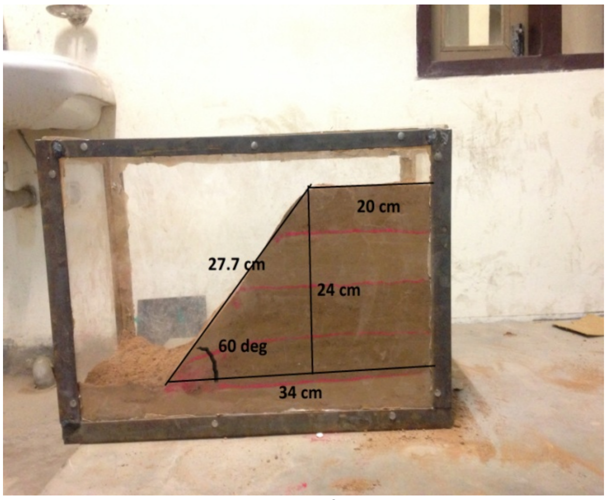



**Fig.3.13** Slope (60$^{o}$) in the model box

### 3.2.3 Modeling of the reinforced slope and installation of nails

The modeling of unreinforced slopes in the model box, load tests were carried out on the slopes to study the failures of slopes. Now, modeling of reinforced slopes (45$^{o}$ and 60$^{o}$ ) was done. Hollow aluminium nails shown in fig 3.15 were installed into the slopes on which load tests were carried out.

**1. Modeling of reinforced slopes**

For the modeling of reinforced slopes, first of all facing walls of appropriate dimensions required for construction of slope angles of 45$^{o}$ and 60$^{o}$ were formed from thin plywood which are shown in fig 3.14. The holes of required diameter for installation of nails were also carved out in facing walls.

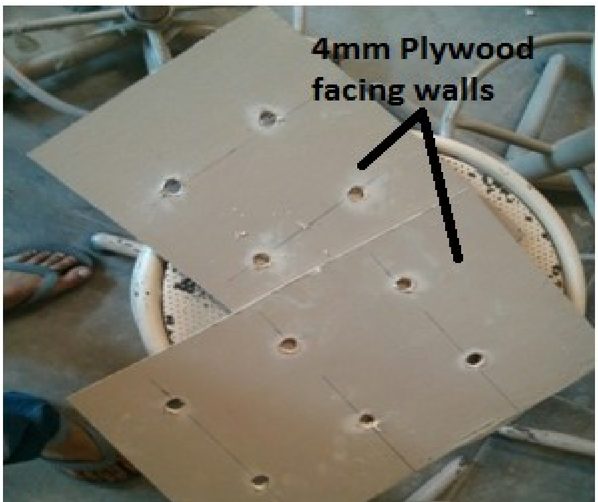



**Fig 3.14** Facing walls for slopes

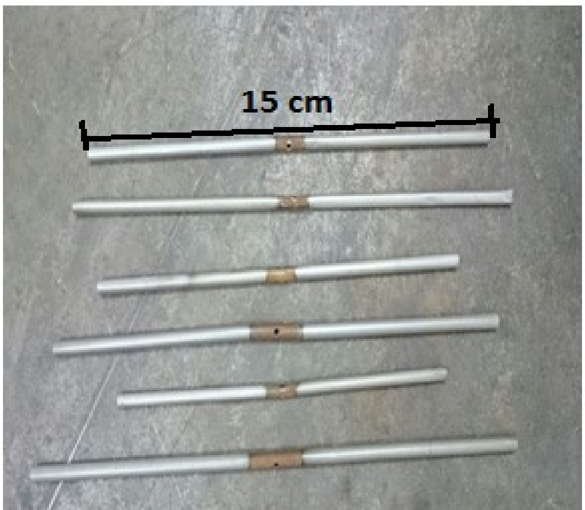



**Fig 3.15** Aluminium Nails

Facing walls were adjusted in the model box conforming to the dimensions and angle of the slopes to be constructed. After setting up of facing walls slopes ($45^o$ and $60^o$) were constructed in layers in the manner as constructed in the unreinforced slope models. Tracer after each layer was laid to note the deformations.

**2. Installation of Nails**

After the slopes were formed, hollow aluminium nails as shown in fig 3.15 were to be installed in the slopes. Six hollow nails each of length 15cm in 3 rows are reinforced into the each slope at a constant vertical and horizontal spacing. Thus, the ratio of length of the nail to slope height is maintained at 0.6 for all the cases. The nails are installed at right angles to the facing wall. The reinforcement (hollow aluminium nails) properties are given in Table 3.2.

Strain gages are glued to each nail to measure the tensile forces developed on the nails during loading. A strain gage of type BKCT-3 (resistance 119.2± 0.2 ohms, gage factor: 1.92± 2% and gage length 3 mm) as shown in fig 3.16 is glued to each nail to record local strain. Nails glued with strain gages is shown in fig 3.17.

**Table 3.2** Properties of Reinforcement

| Reinforcement Properties | Value |
|---|---|
| Elastic modulus of aluminium (nail), E | 69 GPa |
| Cross-setion area, A | 78.5 $mm^2$ |

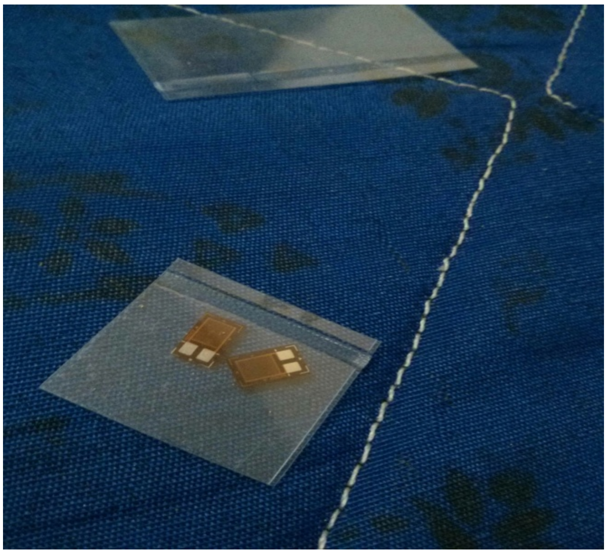


**Fig 3.16** Strain Gages

**Fig 3.17** Strain gages glued to nails

**3.Slope models with reinforced nails**

Strain gages were glued to the nails and connections are made with connecting wires which was done by soldering process.

These nails were then reinforced into the slopes. Fig 3.18 shows the reinforced slope of angle 45$^o$.

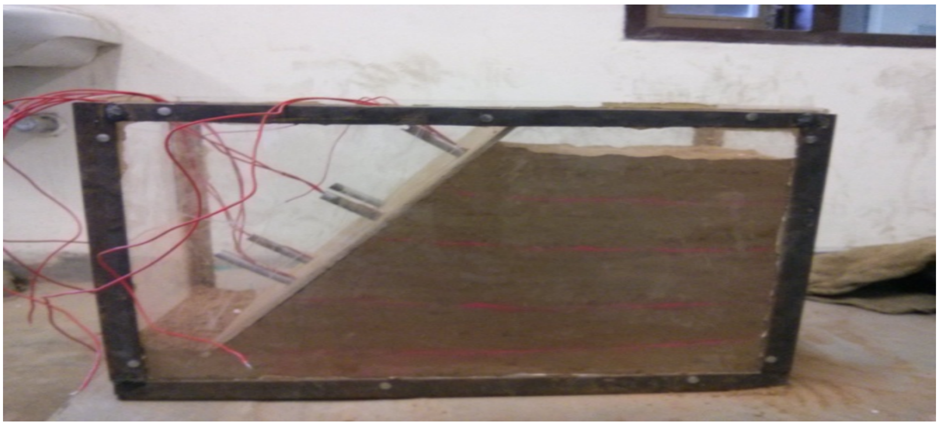


**Fig 3.18** 45$^o$ reinforced slope

### 3.2.4 Observation of strain on strain gage and calculation of nail forces

The slopes reinforced with hollow aluminium nails with strain gages glued to them when subjected to gradually increasing surcharge, induced tensile forces in nails and there is change in strains which was detected by strain gages. For the calculation of nail forces, strains on nails need to be observed.

For observing strains in nails on loading, multimeters were used which measured the resistances on starin gages glued to nails in unstrained and strained positions. After measuring resistances in strain gages, following formula was used to calculate strain:

$$\text{GF} = \frac{\frac{\Delta R_g}{R_g}}{\epsilon}$$

where, GF = gage factor,

$R_g$= resistance of strain gage unstrained,

$\Delta R_g$= change in resistance from unstrained to strained condition, and

$\epsilon$ = strain.

After calculating strain, stess- starin graphs were plotted for nails to determine maximum nail force after some interval of time and then corresponding nail force vs. time graph was plotted for top, middle and bottom nails for both 45$^o$and 60$^o$ reinforced slopes.

# CHAPTER 4

# RESULTS AND DISCUSSIONS

## 4.1 General

As per the described methodologies, the preliminary tests and experiments were carried out on the soil sample to study the various parameters and characteristics of the sample that was to be used for the modelling of slope. The results obtained in different tests have been discussed below.

Unreinforced and reinforced slopes prepared were subjected to gradually increasing surcharge using UTM. Their corresponding failure patterns were observed and studied. Load versus settlement curves were formed for both unreinforced and reinforced slope models of $45^{o}$ and $60^{o}$ and nail forces were calculated through measured values of strain on different nails using strain gages which is given below.

## 4.2 Particle size distribution analysis (Sieve Analysis)

Particle size distribution curve plotted between % finer and particle size (mm) is shown in fig 4.1.

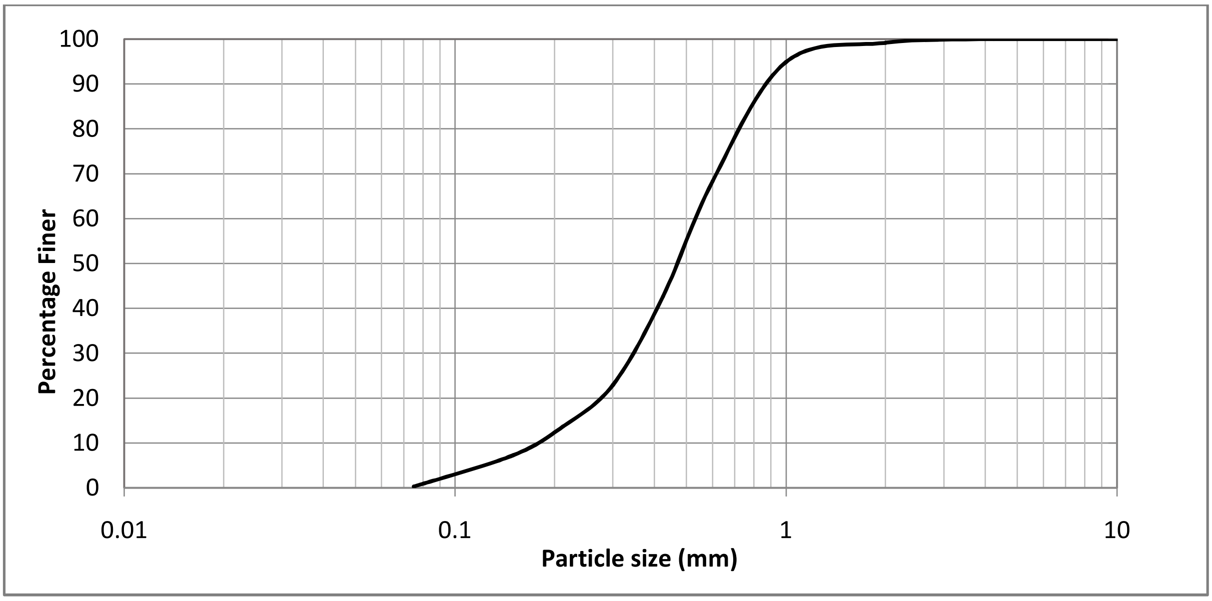



**Fig. 4.1** Grain size distribution curve

From the grain size distribution curve it is seen that soil is **sand.** Since the coefficient of curvature lies between 1 and 3, soil is classified as **well graded sand.**

## 4.3 Specific gravity of soil (Pycnometer test)

Specific gravity of soil sample was found using pycnometer test in the laboratory.

$$G = 2.67$$

Now, the value of specific gravity of soil sample to be used in modeling of slope came out to be **2.67**. This value of specific gravity on matching with table 3.1 shows that soil sample is **sand.**

## 4.4 Direct Shear Test (for shear parameters)

Direct Shear test was carried on the soil sample which was found out to be sand by grain size distribution to find out the shear parameters of the soil, i.e. Cohesion ( c ) and friction angle ( $\phi$). The test was carried out for three normal stresses of 0.5 kg/cm$^2$, 0.75 kg/cm$^2$ and 1 kg/cm$^2$. A graph between shear stress and normal stress was plotted in which y intercept of best fit line gave the cohesion value and slope of line gave the value of friction angle which is shown in fig. 4.2.

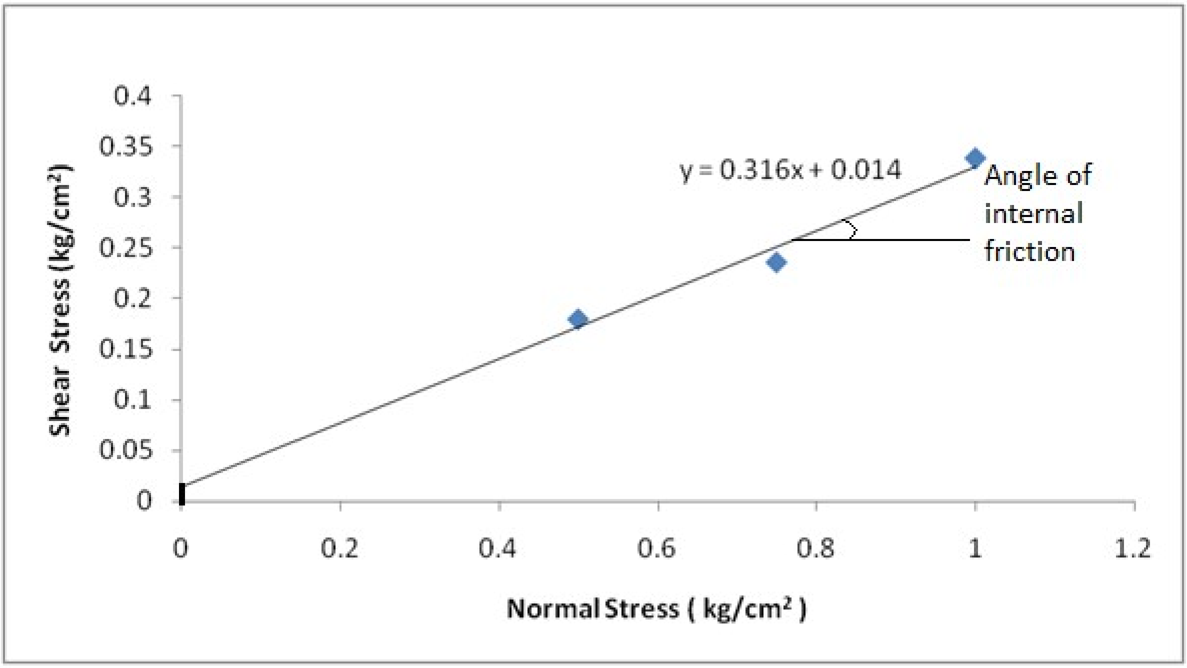



**Fig 4.2** Graph between Shear stress and normal stress

From the above graph, maximum shear stress values are plotted corresponding to normal stesses and a best fit is drawn which intercepts y-axis to give the value of cohesion (c) and slope of the line gives angle of internal friction ($\phi$).

Equation of line is: $y = 0.316x + 0.014$

So, from the equation, Cohesion, $c = 0.014$ kg/cm$^2$

$$c = 1.37 \text{ kPa} \quad ,$$

Angle of internal friction, $\phi = \tan^{-1}(0.316)$

$$\phi = 17.53^o$$

So, the cohesion value is found to be **1.37 kPa** and angle of internal friction as **17.53$^o$.**

From the above values of cohesion and angle of internal friction, it is seen that the angle of internal friction shows that soil is loose sand. Also, sand has a apparent cohesion value which is due to the addition of water to the sand while preparing the slopes.

## 4.4 Failure pattern of Slopes

Unreinforced and reinforced slopes were subjected to load by UTM and following are the images showing slopes before failure and after failure which shows their failure pattern.

**A) Unreinforced Slopes**

**Fig 4.3** 45$^o$ slope before failure

**Fig 4.4** 45$^o$ slope after failure

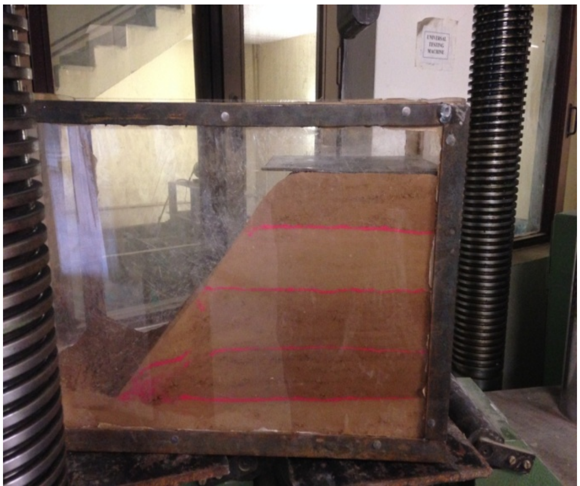


**Fig 4.5** 60$^O$ slope before failure

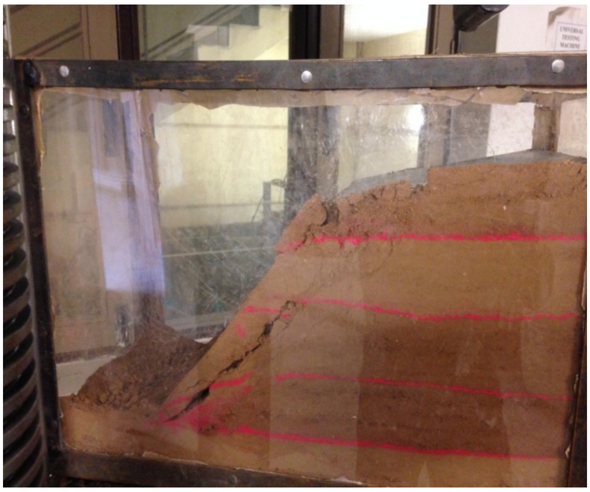


**Fig 4.6** 60$^o$ slope after failure

Above images shows the failure patterns of 45$^o$ and 60$^o$ slopes. It is seen than for 45$^o$ unreinforced slope, the failure observed is slip-surface failure. While for 60$^o$ slope, the failure observed is toe failure.

**B) Reinforced slopes**

**Fig 4.7** 45$^{O}$ reinforced slope before failure.

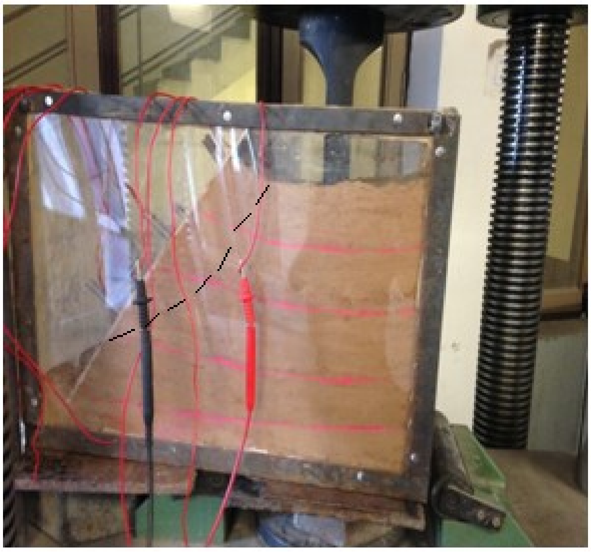


**Fig 4.8** 45$^{o}$ reinforced slope after failure.

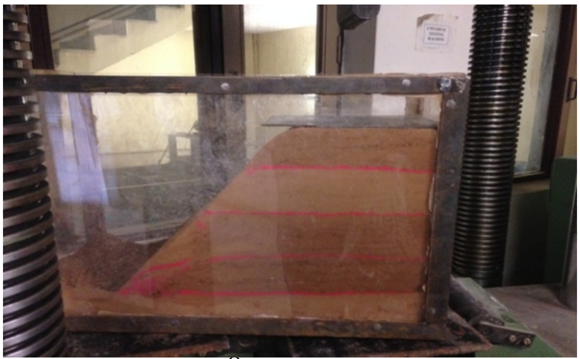


**Fig 4.9** 60$^{O}$ reinforced slope before failure.

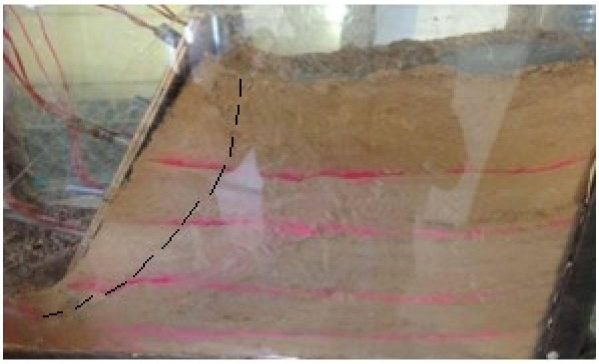
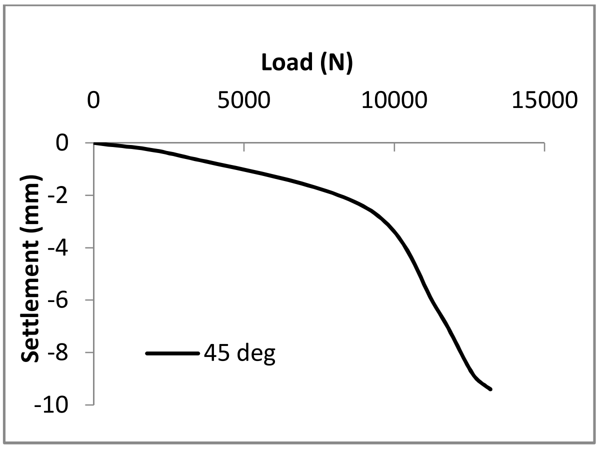


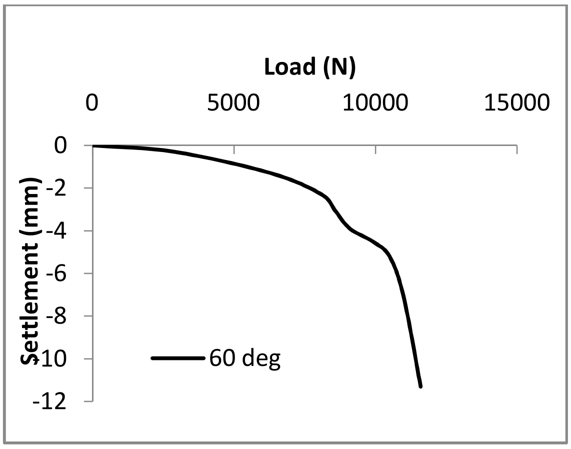
**Fig 4.10** 60$^{o}$ reinforced slope after failure.

Similarly, failure was observed for reinforced slopes. In this case, again the failure pattern for 45$^{o}$ slope is slip-surface failure and toe failure for 60$^{o}$ slope as seen from the settlement of tracer layers.

## 4.5 Load versus Settlement curves for slopes

After applying load on slopes using Universal Testing Machine (UTM), load and deflection values were observed and corresponding load vs. settlement curves were obtained for both the slopes, i.e. 45$^{o}$ and 60$^{o}$ which are shown in fig 4.11 and fig 4.12.



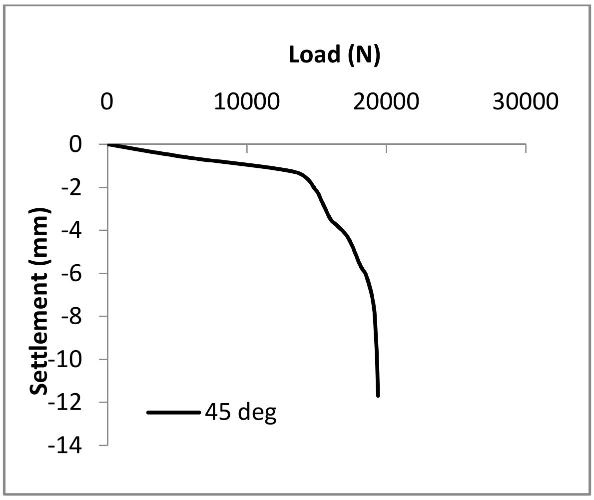



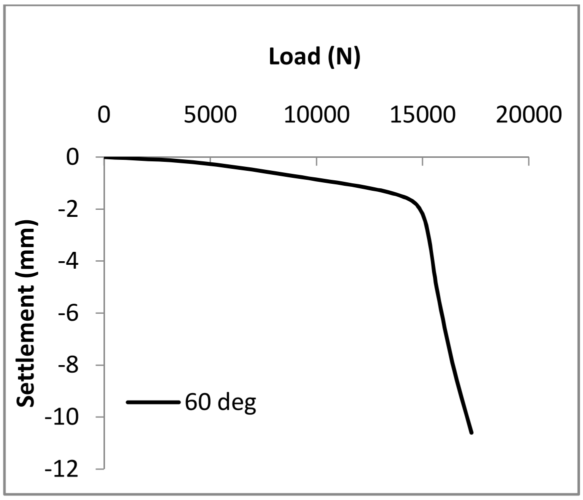

(a) $45^o$ slope (b) $60^o$ slope

**Fig 4.11** Load vs. Settlement curves for unreinforced slopes

From the load vs. settlement curves of unreinforced slopes, the values of load carrying capacity of $45^o$ and $60^o$ slopes are **13200 N** and **11600 N.**

Settlements in the slopes were found out to be **9.4 mm** and **11.3 mm** for $45^o$ and $60^o$ unreinforced slopes.




(a) $45^o$ slope (b) $60^o$ slope

**Fig 4.12** Load vs. Settlement curves for reinforced slopes

From the load vs. settlement curves of reinforced slopes, the values of load carrying capacity of $45^o$ and $60^o$ slopes are **19400 N** and **17300 N.**

Settlements in the slopes were found out to be **11.7 mm** and **10.6 mm** for $45^o$ and $60^o$ unreinforced slopes.

It is seen that reinforced slopes have more load carrying capacities than unreinforced slopes. Also, load carrying capacity decreases with increase in steepness of slopes.

## 4.7 Comparative study of load vs . settlement for unreinforced reinforced slopes

For comparison among unreinforced and reinforced slopes of $45^o$ and $60^o$ slope angles, combined load vs. settlement curves were formed for each slope angle. Fig 4.11 and Fig 4.12 shows the images for the graphs.

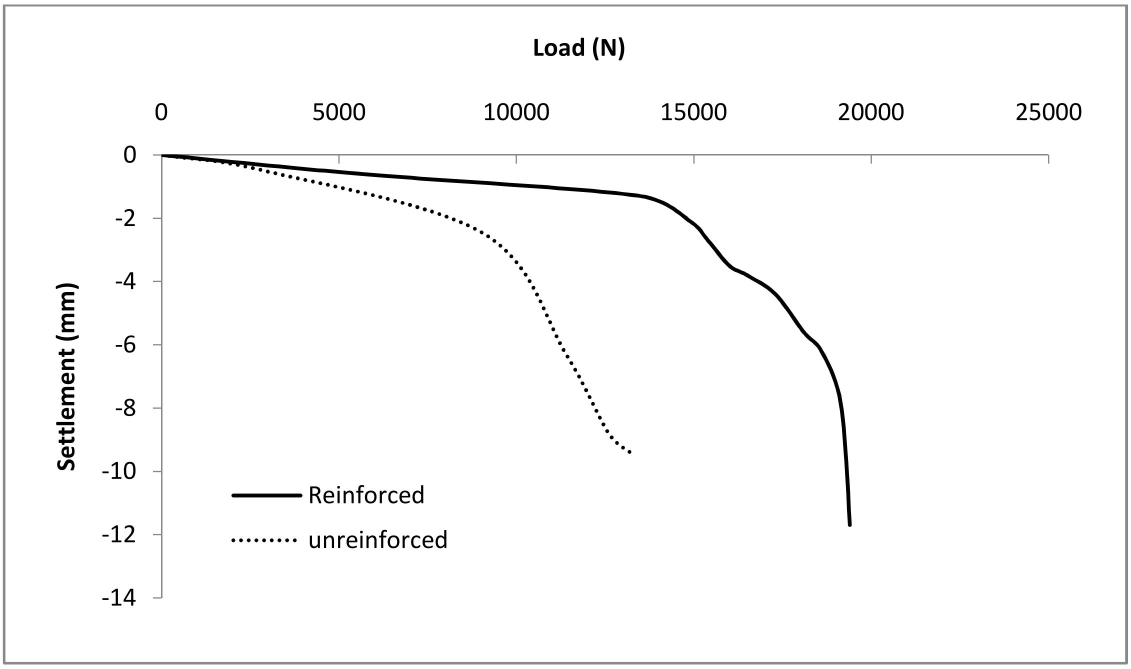



**Fig 4.13** Combined load vs. settlement curve for $45^o$ unreinforced and reinforced slope.

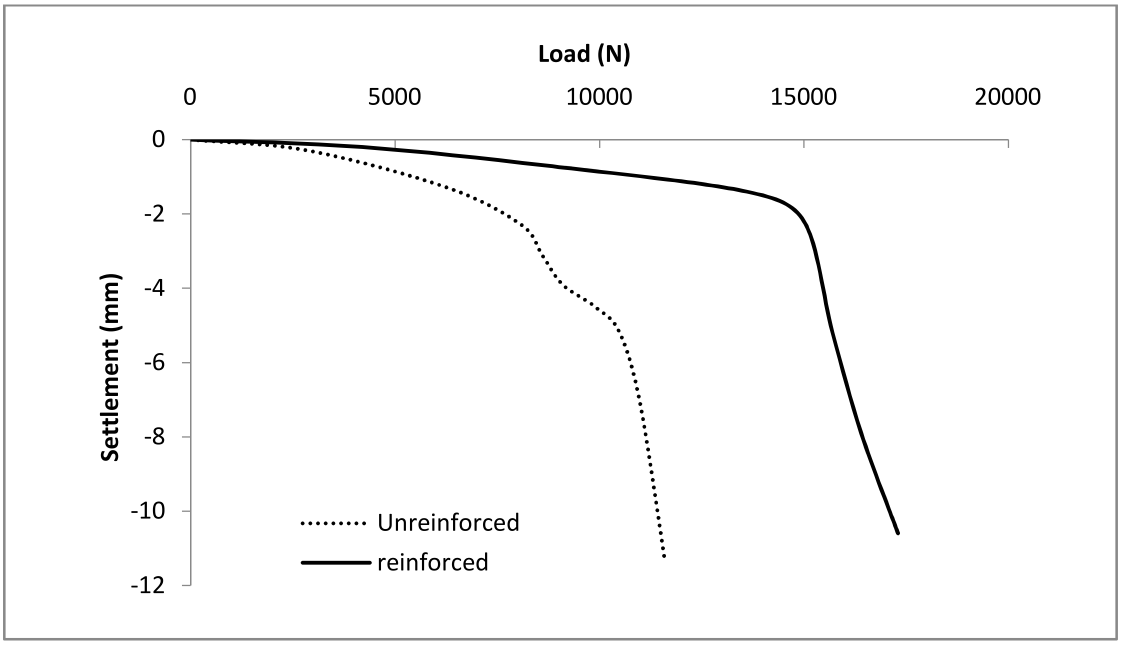



**Fig 4.14** Combined load vs. settlement curve for $60^o$ unreinforced and reinforced slope.

Table 4.1 shows the load carrying capacities and settlements of unreinforced and reinforced slopes.

**Table 4.1** Load carrying capacities and settlement in slopes

| **Slope Angle** | **Unreinforced** | | **Reinforced** | |
|---|---|---|---|---|
| | **Load (N)** | **Settlement (mm)** | **Load (N)** | **Settlement (mm)** |
| **45°** | 13200 | 9.4 | 19400 | 11.7 |
| **60°** | 11600 | 11.3 | 17300 | 10.6 |

From the above table, it is seen that there is 46.9% change in load carrying capacity for 45° slope and 49.1% in 60° slope when reinforced.

## 4.8 Nail forces variation with increase in surcharge loading

When the slopes of angle 45° and 60° were subjected to loading under UTM, then the strains generated in hollow aluminium nails were calculated by recording resistance changes in strain gages using multimeters and formulae as described in section 3.2.4.

Local strains corresponding to load value at different time intervals of each nail was measured with strain gages. The nail (reinforcement) forces were calculated from calculated strain using load versus strain relationship of aluminium. The elastic modulus of aluminium is 69 GPa (which is taken as elastic modulus of aluminium nail too). The variation of nail force as calculated from strain gage reading is graphically plotted against increasing surcharge loading for both slopes of 45° and 60° as shown in fig 4.12 and fig 4.13 respectively.

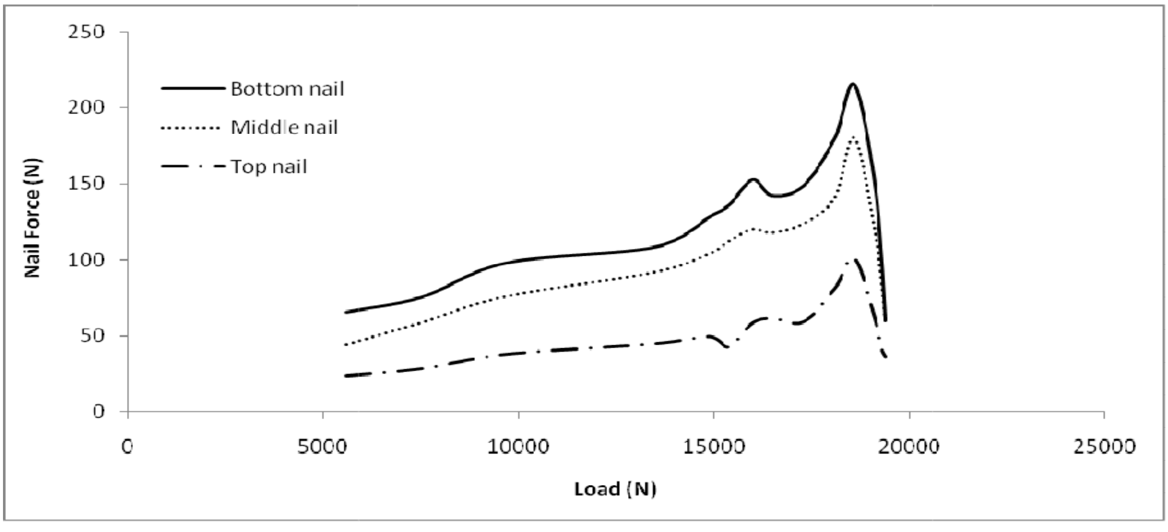



**Fig 4.15** Nail forces in 45° model slope while loading.

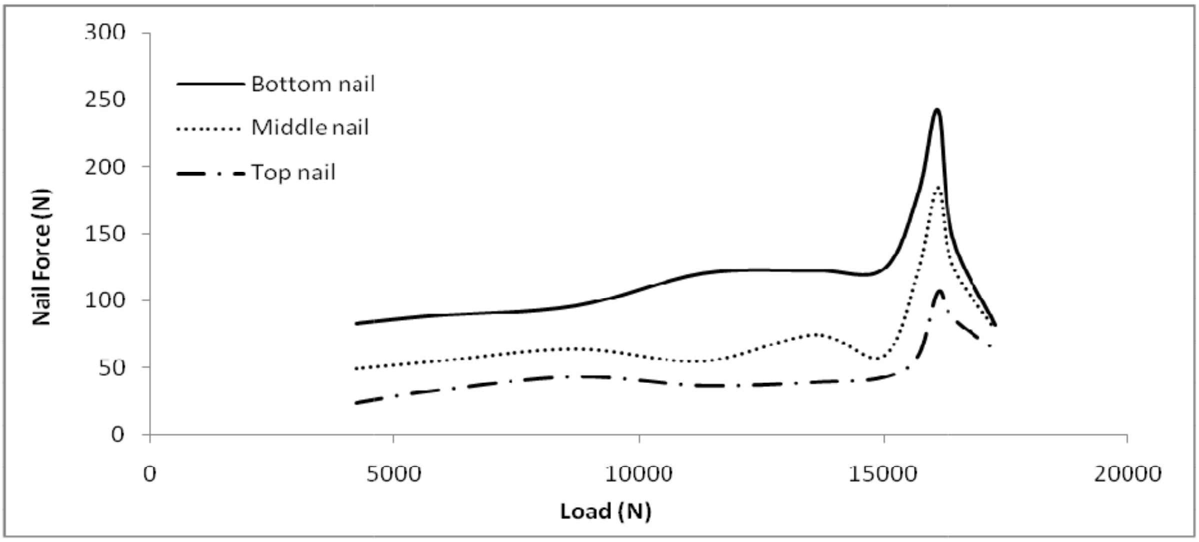



**Fig 4.15** Nail forces in $60^o$ model slope while loading.

From the above graphs, it is seen that nail forces are maximum at 18600 N and 16100 N for $45^o$ and $60^o$ slopes. It is also seen that bottom nails bear the maximum load followed by middle nails and lowest forces are generated in top nails.

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## CHAPTER 5

# CONCLUSIONS

## 5.1 General

In this major project report an attempt has been made to assess the soil nailing process in stability of slopes through comprehensive experimental and lab studies. This chapter presents the overview and the salient conclusions drawn from the work carried out under this project. Also, future scope of the project is discussed in this chapter.

## 5.2 Conclusions and remarks

Following conclusions were drawn from the work carried out:

- From Particle Size Distribution (PSD) curve, it was observed that the soil was **sand.** Further the value of coefficient of curvature ($C_c$) showed that soil **was well graded** sand which was used for modeling of unreinforced and reinforced slopes with slope angles 45$^o$ and 60$^o$. The specific gravity of the soil sample used was found out to be **2.67** by carrying out pycnometer test for specific gravity and soil was classified as **sand**. Also. the direct shear test results gave values of **shear parameters,** c and $\phi$ as 1.37 and 17.53$^o$ showing that it was loose sand and had small cohesion value due to addition of water.

- The **failure patterns** observed in the slopes were found to be only slip-surface failure for the 45$^o$ unreinforced slope and that of 60$^o$ slope model was observed as toe failure. For reinforced slopes of angles 45$^o$ and 60$^o$, the **failure patterns** observed were slip-surface failure and toe failure respectively.

- The load carrying capacity of reinforced slopes with hollow aluminium nails as reinforcement is greater than as compared to unreinforced slopes which was observed from load versus settlement curves. In **45$^o$** reinforced slope, an increase of **46.9%** in load carrying capacity is observed than unreinforced slope. For **60$^o$** reinforced slope, increase in ultimate load at failure was found to be **49.1%** than unreinforced slope. So, the ultimate load increases after reinforcement with nails. Also, the ultimate load decreases with increasing steepness.

- **Nail forces** which were measured were found to be **maximum** for **bottom nails** at any point of surcharge loading for both the slopes tested and the **minimum forces** were generated in **top** row of nails for both the cases as observed from nail force vs. time graph plotted. Earlier investigators as mentioned in chapter 2 have received similar results.

- Here, modeling of slope is done in a model box of Perspex sheet, i.e. slope is supported by Perspex sheet from four sides. So, it is observed that due to change in the boundary conditions for preparing and analyzing the unreinforced and reinforced slopes, results may slightly differ from results at actual site conditions where soil will be surrounded by soil or ground.

On the whole, the project report has attempted to provide an insight into the advantage of soil nailing in stability of slopes as an economical, easy and quick method through experimental study. Change in load carrying capacity of slopes, failure patterns and distribution of nail forces has provided a better understanding of the soil nailing technique.

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