`

MICROPILES FOR REINFORCING STEEP SLOPES

A dissertation submitted in partial fulfillment of the requirement for the award of degree of

MASTER OF TECHNOLOGY

IN

GEOTECHNICAL ENGINEERING

BY

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CANDIDATE’S DECLARATION

I do hereby certify that the work presented is the report titled “**micropiles for reinforcing steep slopes**” in partial fulfillment of requirement for the award of the degree of “Master of Engineering” in geotechnical engineering submitted in the Department of Civil Engineering, Delhi Technological University, is an authentic record of my own work carried out from December 2015 to June 2016 under the supervision of Dr. Amit Kumar Shrivastava (Assistant Professor), Department of Civil Engineering.

I have not submitted the matter embodied in the report for the award of any other degree or diploma.

Devashish Raghav

Date:30th June 2016 (2K14/GTE/09)

CERTIFICATE

This is to certify that above statement made by the candidate is correct to best of my knowledge.

Dr. Amit Kumar Shrivastava

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ABSTRACT

A Micropile is a small diameter (< 300mm), which is drilled and grouted non displacement pile that is typically reinforced. Micropiles can be used as elements for foundation support which resist seismic and static loading conditions, and as in-situ reinforcements to provide stability to slopes and excavated sites. During our routine excavation activities, the slope of the excavation leads to extra land requirement and less economic construction practices. A solution to this problem is the use of micropiles to increase the slope of the embankment and aim for vertical excavations and provide economical methods of construction. This paper presents one protoype in which exacavation is done along side a road. Based on that protoype, numerical analysis, analytical analysis and physical model analysis were conducted for the embankment. In all the analyses of the embankment, it was subjected to maximum vehicular load as per IRC-6. The numerical analysis was done using software SLIDE. The analytical analysis, which includes design of micropiles, was conducted using the FHWA manual on design and construction of micropiles. And the physical modelling was done using a scale factor of 1/50 and loading was applied to the physical models and the deformation behavior of the model embankment was studied. Finally, comparison was made among the numerical, analytical and phyical models and results were compiled.

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CHAPTER 1
INTRODUCTION

Micropiles were first used in Italy in 195Os, because of the demand for innovative techniques tounderpin historic buildings and monuments whichsustained damage during World War II. A reliable method was essentially required to support structural loads with as minimum movement as possible and to install in access-restrictive environments with almost negligible disturbance to the existing structure. So, an Italian specialty contractor named Fondedile, along with Dr. FernandoLizzi developed the technique.

The uses of micropiles have grown significantly and have been used mainly as elements for foundation support to resist static and seismic loading conditions, and as in-situ reinforcements for slope and excavation stability.

A Micropile is a small diameter (< 300mm), drilled and grouted non displacement pile that is typically reinforced.

Micropiles are more advantageous as compared to the more conventional available support systems

The grouting and drilling equipment whichare used for micropile installation are relatively small and can be easily mobilized in restrictive areas which would prevent the entry of conventional pile installation equipment.

These can be installed in areas of variable, difficult,or unpredictable geologic conditions like ground with boulders and cobbles,fills with buried utilities and running sands, soft clays, and high groundwater which is not conducive to conventional drilled shaft systems that can cause cause minimal impacts to micropile installations.

Micropiles can be easily installed in contaminated and hazardous soils. Due to their small diameter, less spoil is caused by the drilling operation that would be caused by conventional drilling systems.And, the flush effluent can be kept in check easily at the ground surface bycontainerization or by the use of lined surface pits. All these factors greatly reduce the handling costs and potential for surface contamination.

Grout mixes are designed to withstand chemically aggressive soils and groundwater. To avoid and reduce deterioration from corrosive and acidic environments, special admixtures can be included in the grout mix design. To eliminate the need of increasing the foooting size, micropiles are added to an existing pile cap. By this approach, the additional tension, compression and moment resistance relatedto increased structural loads can be resisted

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Manytimes adjacent utilities and/or structures restrict the possibility of enlarging the existing pile caps, thus eliminating more traditional piling systems. By this approach, design analyses need to consider the relative stiffness of the existing piles and the micropiles to estimate individual loads.

1.1 CLASSIFICATION OF MICROPILES

1.1.1 *Design Application*

Case 1: Micropile elements, that are directly loaded& where the reinforcemets provided in the piles resist the majority of the load applied on them.

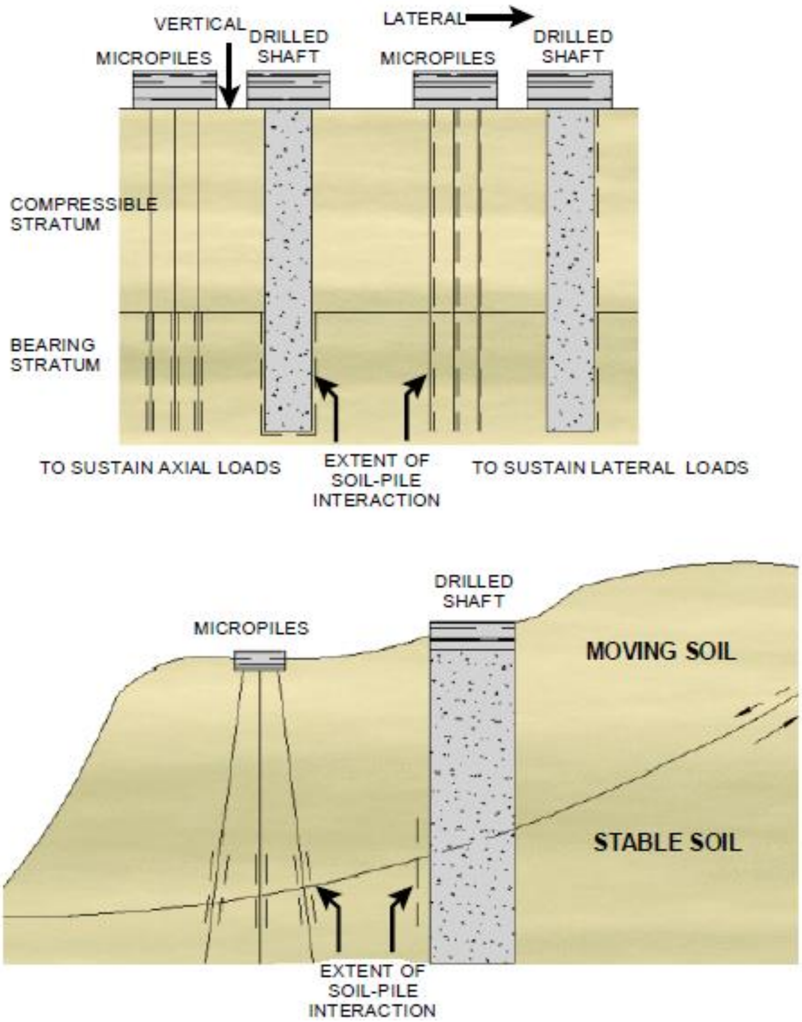

Figure 1.1 CASE 1 Micropiles (Directly Loaded)*(FHWA)*

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Applications for Case 1

a) New Foundations

b) Under pinning of existing structures

c) Seismic retrofitting of existing structures

d) Scour protection

e) Earth retention

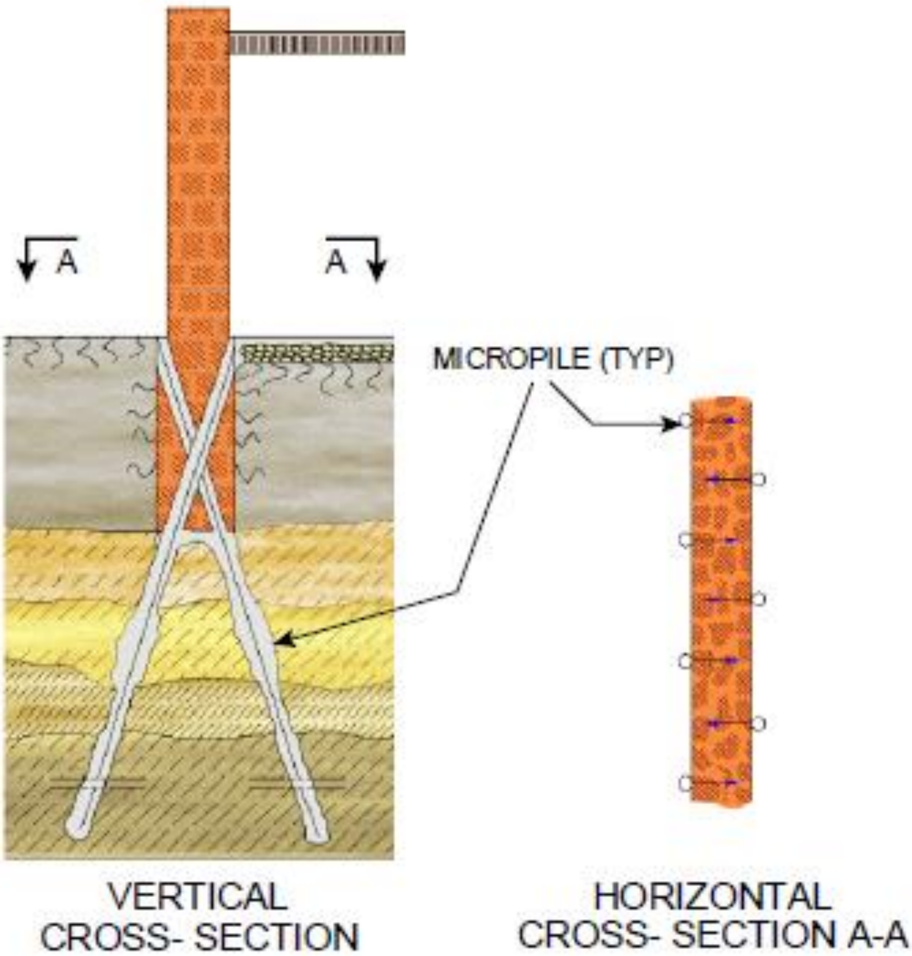

Figure 1.2. Classical Arrangement of Root Piles for Underpinning (*FHWA)*

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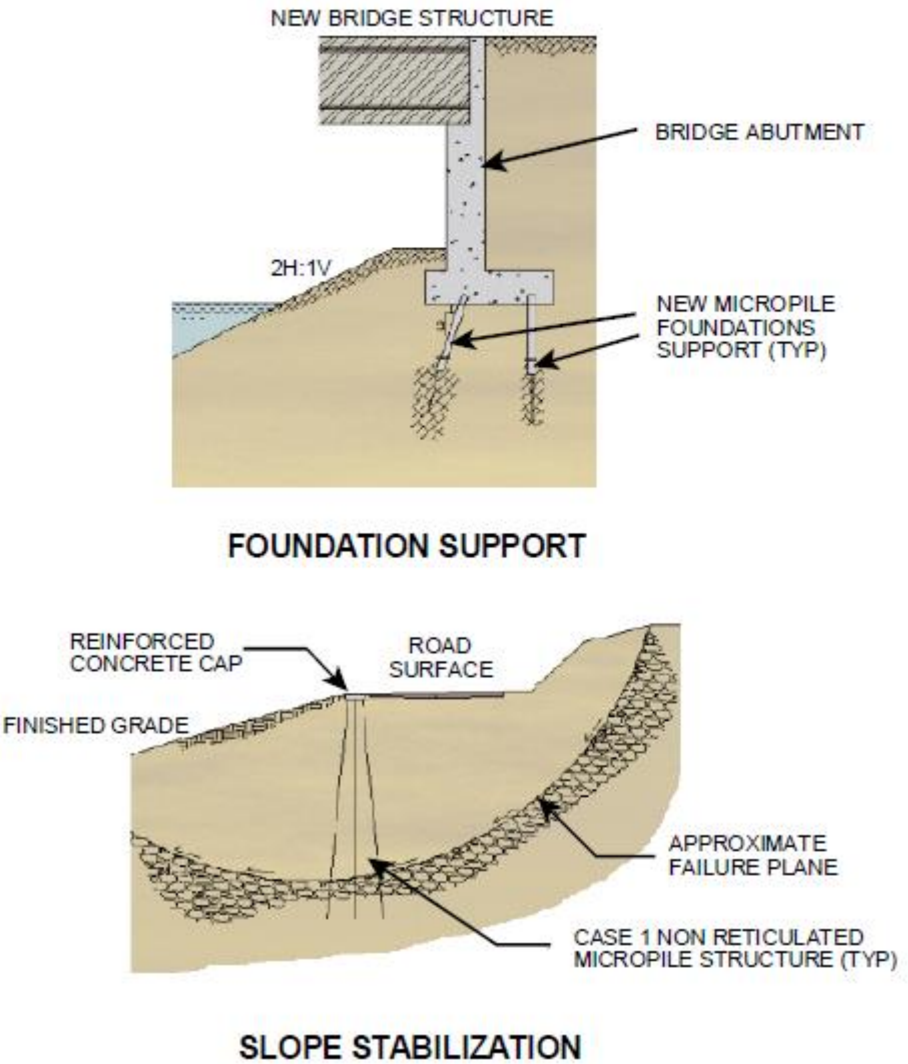

Figure 1.3Micropile Arrangements (*FHWA)*

Case 2: Micropile elements internally reinforce and circumscribe the soil to make a reinforced soil composite which resists the the load that is applied on it.

Applications for Case 2

a) Slope Stabilization

b) Earth retention

c) Ground strengthening and protection

d) Settlement reduction

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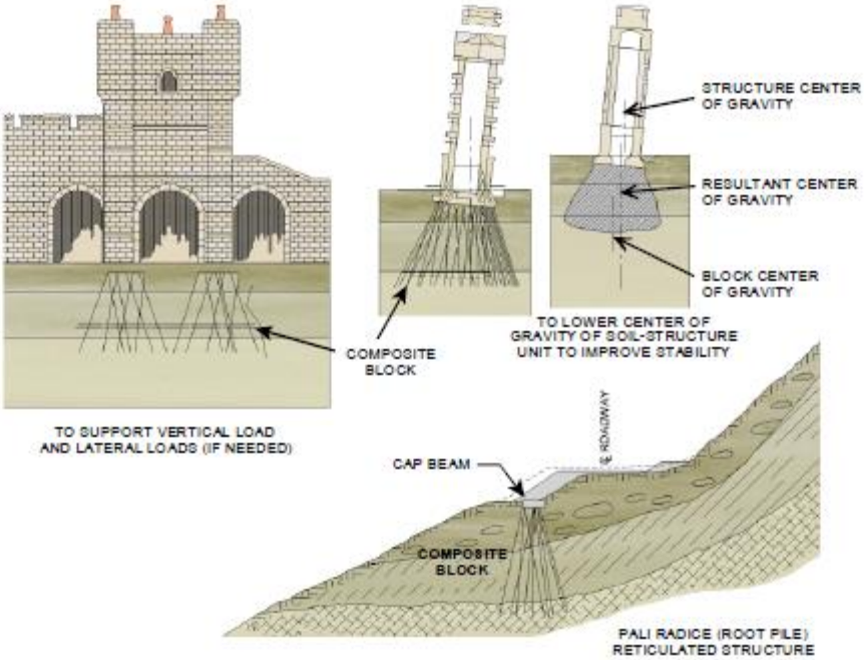

Figure 1.4 Micropiles-Reticulated Pile Network with Reinforced (*FHWA)*

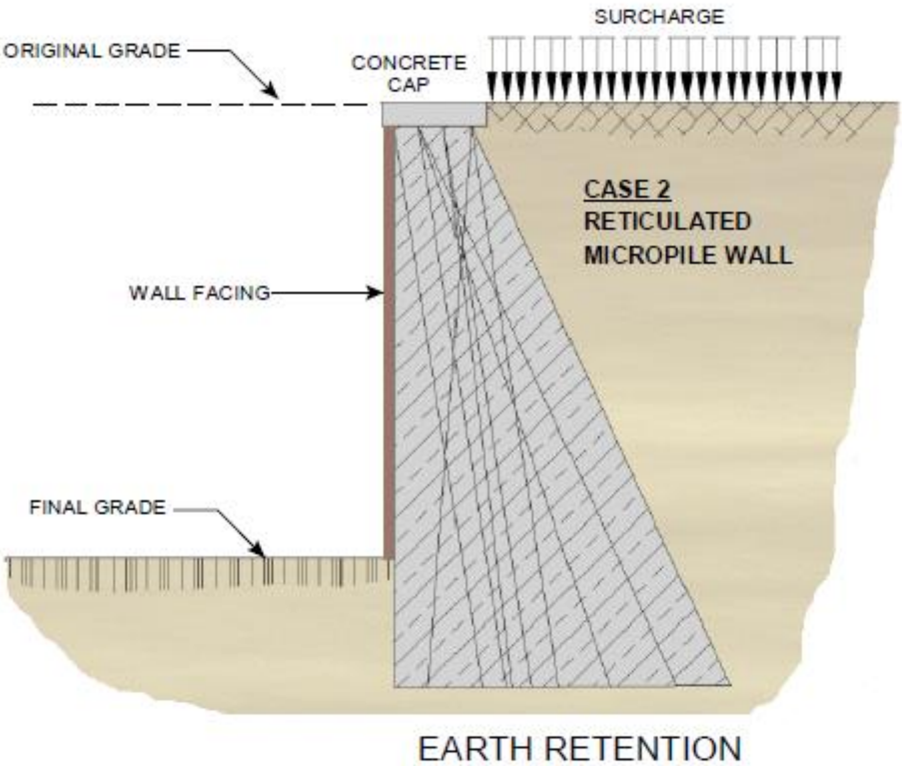

Figure 1.5Micropile Arrangements (*FHWA)*

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1.1.2 Construction type

The grouting method is generally the most sensitiveconstruction control over ground/grout bond capacity. The grouting method affects the Grout-to-grout capacity.

a)Type A: Gravity Grout

The grout isplaced under gravity using only neat cement or sand cement motors.

b) Type B: Pressure through Casing

In this casethe hole is filled with neat cement groutand the temporary steel casing is withdrawn simultaneously. Injection pressures can vary from 0.5 to 1.0 MPa. The pressure is generally limited to avoid fracturing of the ground surrounding the hole.

c) Type C: Single Global Post Grout

This is a two step process:

1) As of Type A

2) Prior to hardening of the primary grout, similar grout is injected one time via a sleeve grout pipe at pressure of at least 1.0Mpa.

d) Type D: Multiple Repeatable Post Grout

It is done in a two step process of grouting similar to Type C with some modifications to step 2 where the pressure is injected at a pressure of 2.0 to 8.0 Mpa.

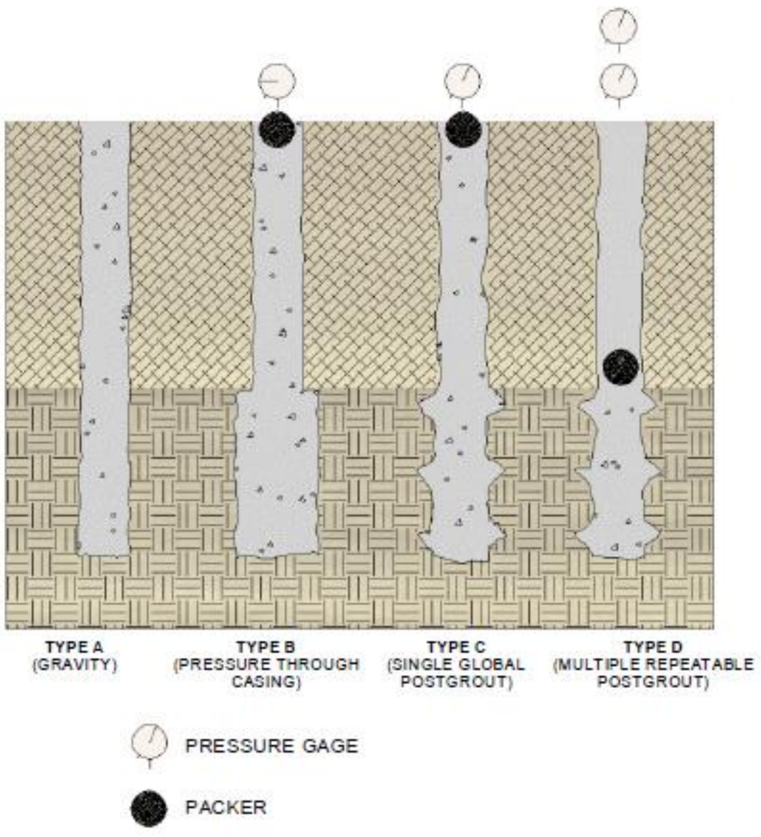

Figure 1.6Micropile Classification Based on Grouting (*FHWA)*

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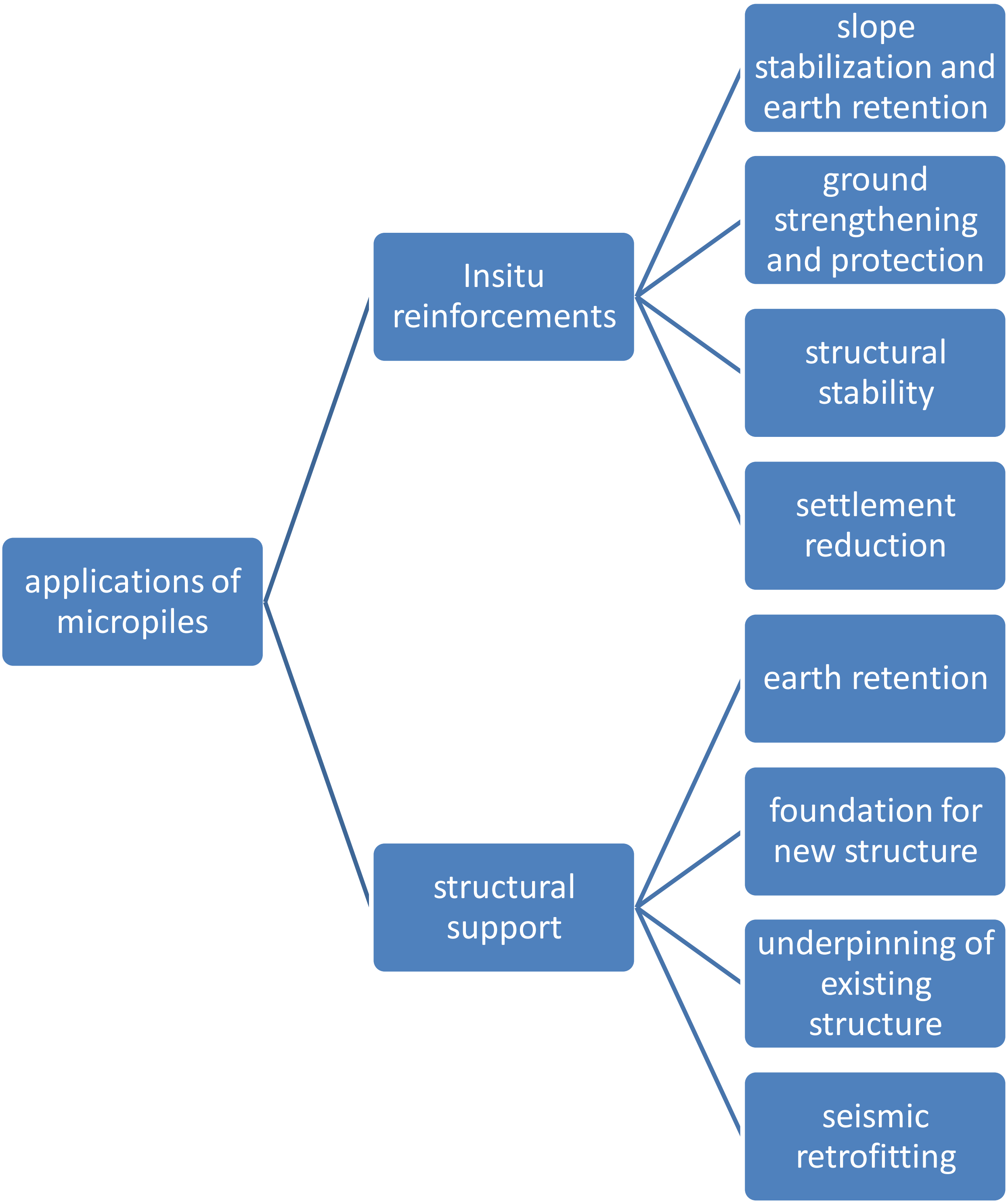

Figure 1.7Flow Chart of Applications of Micropiles

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1.2 ADVANTAGES OF MICROPILES

The advantages of the micropiles are as follows:

1. Micropiles are most often used for underpinning of the existing structure where minimum vibration or noise is required.
2. They can be easily laid where there is a constraint of low head room.
3. Micropiles can be readily installed at any angle below the horizontal using the same equipment used for grouting.
4. They offer a cost effective and practical solution to costly alternates such as pile systems and serve as a solution to job sites which have difficult access.
5. They do not require drilling platforms or large access roads.

1.3 MICROPILE LIMITATIONS

Vertical micropiles may be limited in cost effectiveness and lateral capacity. The ability of micropiles can be enhanced if they are installed on an incline. Because of the high slenderness ratio (length/diameter), micropiles might not be an acceptable solution for conventional seismic retrofitting in areas where liquefaction may occur.

The use of micropiles for slope stabilization, however, continues to increase. But, it is recommended that performance data should be collected on such projects because withdetailed design procedures and experiences, the use of micropiles will continue to evolve.

1.4 ECONOMIES OF MICROPILE

The cost involved in installation of micropiles generally exceeds the cost involved in conventional piling systems, especially in the case of driven piles.

Cost effectiveness of micropiles depends on a lot of factors. It is very important to calculate the cost of using micropiles based on the environmental, subsurface, and physical factors. For example, for an open site with soft, clean, uniform soils and unrestricted access, micropiles will likely not be a competitive solution. However, for the delicate underpinning of an existing bridge pier in a heavily trafficked old industrial or residential area, micropiles can provide the most cost-effective solution.

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1.5 OBJECTIVE

The main objectives of this project are

1. To obtain the maximum slope of an excavated cut and aim for vertical cut using micropiles.
2. To conductnumerical, analytical and physical analysis of the model embankment..
3. To compare the various results and to select the most suitable configuration of micropile.
4. And finally to show that the micropiles can beeffectively used to reinforce steep slopes and to act as an earth retention system too.

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CHAPTER 2
LITERATURE REVIEW

2.1 PREVIOUS RESEARCHES

Esmaeili et al. (2013)conducted an experimental and numerical study on micropiles. They made three experimental models of embankments of 10 m in height on a scale of 1=20 to set up a number of loading tests: one based on a non-reinforced embankment and two others based on reinforced embankments that are stabilized with two different arrangements of micropiles. During laboratory tests, the authors collected the data of displacements of embankment crest and bed surface, and axial strain of micropiles were measured using the instrumentation tools. Then three numerical models were developed by using the PLAXIS-3D code based on the FEM and a comparison was made between the experimental and numerical data to verify the outputs of the numerical analyses.

Only poorly graded sand with gravel (SP) and clayey sand (SC) were, respectively selected to model the bed and no other soil was considered and no modifications to the result were provided that can be used with other types of soil.

The models generated failed to considered the different soil profile that can exist on a field. The model considered only one type of soil at every depth.

Seo et al. (2013) conducted experiments on a fully instrumented field-scale load test on a 0.2-m-diameter micropile socketed 4.2m into limestone layers (2.7m into weathered limestone and 1.5m into hard limestone). The results showed that no base resistance was mobilized until the pile-head settlement reached approximately 7% of the diameter of the test micropile.

Base resistance was neglected because of the possible existence of loose debris or rock at the bottom of the shafts. But if the loose debris or rock is not present at the bottom of the shaft then base resistance can exist and the result might differ.

Misra et al. (2004) presented and evaluated analytical relationships for micropile pullout load-displacement behavior, which explicitly considered the micropile–soil interaction.

The authors assumed the micropile–soil interface to be elastoplastic. The author also assumed that the load transfer to the ground occurs through the soil–grout mixture in the immediate vicinity of the micropile.A homogeneous soil–grout shear boundary layer was assumed by the author which can vary and affect the result of the experiment conducted.

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Shields (2007) compared the pile buckling loads obtained from a semi empirical relationship to the allowable loads permitted by current and proposed codes and design guidelines. It was concluded in the paper that there are some designs permitted under the codes and design guidelines for which buckling may be a controlling factor.

2.2 SCALING LAWS

Scale models are used in geotechnical engineering as they offer the advantage of simulating complex systems under controlled conditions, and the opportunity to gain insight into the fundamental mechanisms operating in these systems. In many circumstances (e.g., a static lateral pile load test), the scale model may be a more economical option than the corresponding full-scale test. In addition to qualitative interpretation, scale model test results are often used as calibration benchmarks for analytical methods, or to make quantitative predictions of the prototype response.

Scale models can have three types of similarity to the prototype, namely, geometric, kinematic or dynamic similarity.

2.2.1. GEOMETRIC SIMILARITY

For geometric similarity to exist, the ratio of corresponding length dimension between model and prototype must be the same. The geometric parameters are length, width, height, area, volume, diameter.

$$\frac{Lm}{Lp} = \frac{Wm}{Wp} = \frac{Hm}{Hp} = \lambda$$

$$\frac{Am}{Ap} = (\lambda)^2, \frac{Vm}{Vp} = (\lambda)^3$$

2.2.2. KINEMATIC SIMILARITY

Kinematic parameters are: velocity, acceleration and discharge. At all corresponding points in the model and prototype the ratio of velocity as well as acceleration must be same (both in magnitude and direction). Such similarity can be attained if flownets for the models and proto type are geometrically similar. The kinematic parameters are velocity, acceleration and discharge.

2.2.3. DYNAMIC SIMILARITY

For dynamic similarity to exist between model and prototype, identical type of forces (viscous, pressure, elastic etc) must be parallel and must bear the same ratio at all corresponding set of points.

Dynamic parameters are force and power.

For kinematic similarity	Geometric similarity must exist
For dynamic similarity	Geometric similarity must exist Kinematic similarity must exist but is not the sufficient condition for dynamic similarity

In our project we will be using geometric and dynamic similarity to model the experiment.

Table 2.1. Froude's Scaling Table

Physical parameter	Unit	Multiplication factor
Length	[m]	Λ
Mass	[kg]	$\lambda^3 .\ \rho_F / \rho_M$
Force	[N]	$\lambda^3 .\ \rho_F / \rho_M$
Moment	[Nm]	$\lambda^4 .\ \rho_F / \rho_M$
Acceleration	[m/s^2]	$a_F = a_M$
Time	[s]	$\lambda^{0.5}$
pressure	[Pa=N/m^2]	$\lambda .\ \rho_F / \rho_M$

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2.3. BISHOP'S SIMPLIFIED METHOD OF ANALYSIS

Bishop's method of slices (1955) is useful if a slope consists of several types of soil with differentvalues of *c* and Φ and if the pore pressures *u* in the slope are known or can be estimated. The methodof analysis is as follows: consider a section of an earth dam having a sloping surface *AB*. *ADC* is an assumedtrial circular failure surface with its center at *O*. The soil mass above the failure surface is dividedinto a number of slices. The forces acting on each slice are evaluated from limit equilibrium of theslices. The equilibrium of the entire mass is determined by summation of the forces on each of theslices.Consider for analysis a single slice *abed*. The forces acting on this slice are:

W = weight of the slice

N = total normal force on the failure surface *cd*

U = pore water pressure = ul on the failure surface *cd*

F_r = shear resistance acting on the base of the slice

E_1 E_2 =normal forces on the vertical faces *be* and *ad*

T_1 T_2 = shear forces on the vertical faces *be* and *ad*

θ = the inclination of the failure surface *cd* to the horizontal

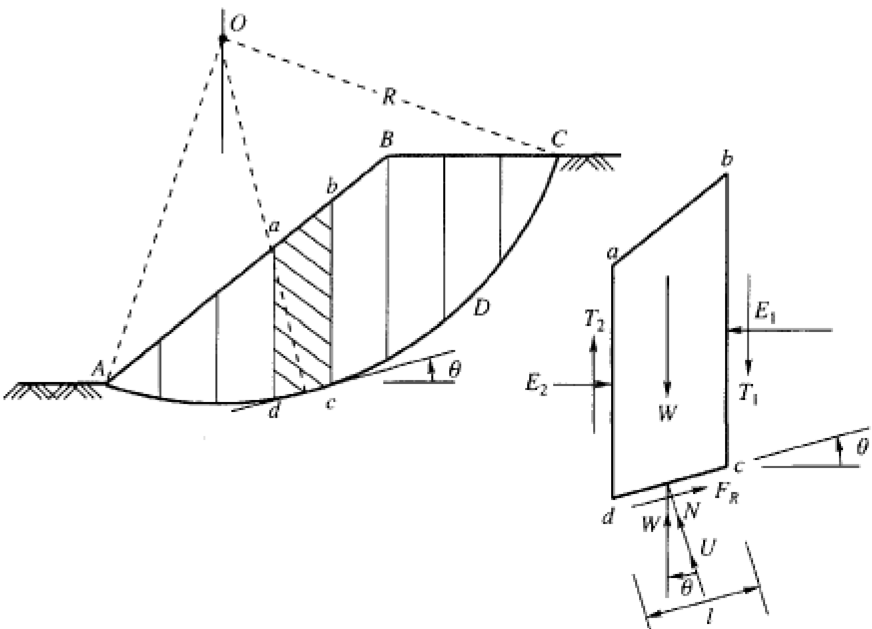

Figure 2.1 Bishop's Simplified Method (Murthy)

$$F_s = \frac{\Sigma\{c'l\cos\theta + [(W - U\cos\theta) + \Delta T]\tan\Phi'\}\frac{1}{m\theta}}{\Sigma W\sin\theta}$$

`

Where $m_\theta = \cos\theta + \frac{\tan\Phi' \sin\theta}{Fs}$

2.4. JANBU SIMPLIFIED METHOD OF ANALYSIS

It is very similar to the bishop's simplified method of slope stability analysis. In this method it is assumed that no shear exists between the slices. The horizontal forces are neglected in this method of analysis i.e. E in the figure 2.1 is neglected. It is good for circular sliding surfaces and the results are conservative if used for other cases. It should not be used if large horizontal forces are involved.

2.5.METHODOLOGY

A literature review was conducted on micropiles and all the major works done on micropiles were studied.Engineering properties of sand were found and recorded.A prototype was made in which if an excavation was to be made along side a road for some construction activity.The maximum load on road was obtained using IRC:6-2014 STANDARD SPECIFICATIONS AND CODE OF PRACTICE FOR ROAD LOADS AND STRESSES.The maximum load obtained was $12t/m^2$.

Next, the embankment was modeled on SLIDE software and slope stability analysis was performed and critical slope angle was found.

In the next step, micropile was modeled in the software on the embankment and appropriate resisting force was given to the micropile to make the embankment stable at the maximum slope.

After the numerical analysis, analytical analysis was performed in which micropiles were designed as per FHWA Micropile Design and Construction (2005 version).

After obtaining the best possible configuration of the micropiles, a physical model was constructed with the help of suitable scaling laws. The embankment was constructed and the micropiles were installed in the embankment and the loading was applied. The failure pattern was observed and recorded.All the results were obtained and compared and the validity of the numerical and analytical design was verified with the help of the physical model.

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CHAPTER 3
EXPERIMENTAL INVESTIGATIONS

3.1 SPECIFIC GRAVITY

The specific gravity of a soil sample is defined as the ratio of the mass of a given volume ofsoil solids to the mass of an equal volume of water. Specific gravity can be determined by:

$$G = \frac{(W2-W1)}{(W2 - W1)-(W3- W4)}$$

Where,

W1 = weight of pycnometer

W2 = weight of pycnometer + soil

W3 = weight of pycnometer + soil + water

W4 = weight of pycnometer + water

Table 3.1. Specific Gravity Calculation

	Sample 1	Sample 2	Sample 3
W1 (g)	695.32	696.10	696.95
W2 (g)	895.40	896.32	897.10
W3 (g)	1689.52	1689.13	1690.34
W4 (g)	1564.20	1564.90	1565.90
G	**2.67**	**2.63**	**2.64**

The average of the above three values was taken as the value of specific gravity of the soil sample i.e. **G = 2.65**

3.2 PARTICLE SIZE DISTRIBUTION

The dry sieve analysis of the soil was conducted. The sieves were arranged in a decreasing manner from top to bottom. The mass of soil retained on each sieve was noted down and percentage finer was calculated . The percentage finer was plotted against sieve size on a semi log graph sheet as shown in figure 3.1.

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Table 3.2. Sieve Analysis Of Sand

Sieve size (mm)	Mass retained (g)	Percent mass retained	Cumulative percent retained	Percent finer
4.75mm	16.50	1.65	1.65	98.35
2.36mm	33.71	3.37	5.02	94.98
1.18mm	57.53	5.75	10.77	89.23
0.600mm	2.479	2.48	13.25	86.75
0.300mm	661.14	66.10	79.35	20.67
0.150mm	181.80	18.18	97.53	2.47
0.075mm	4.94	0.49	98.02	1.98
pan	19.85	1.98	100	

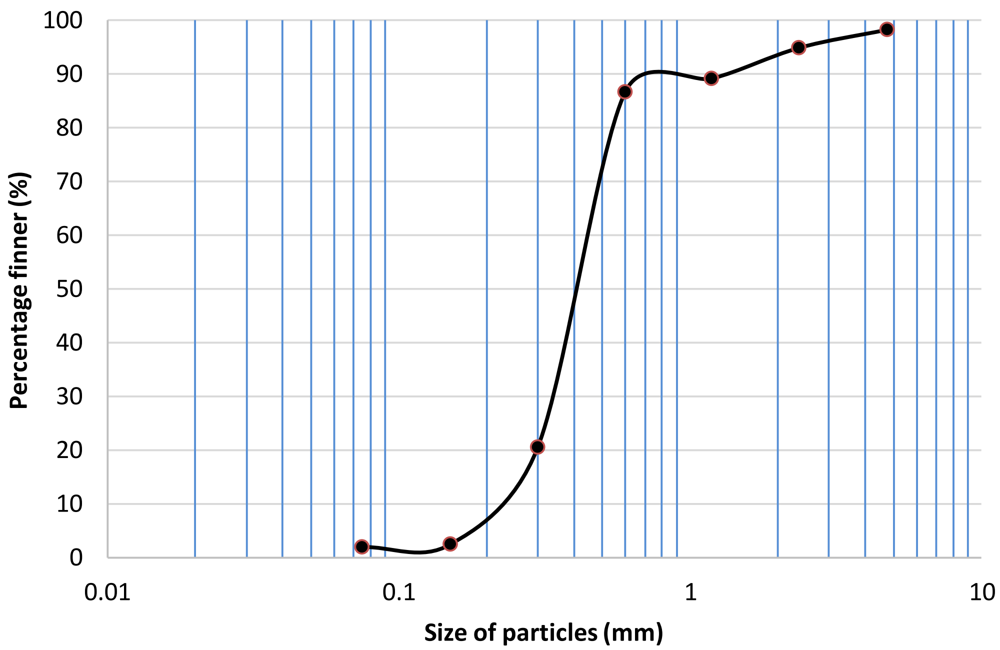

Figure 3.1. Sieve Analysis of Sand

The coefficient of uniformity was calculated to be, C_u = 1.45

The coefficient of curvature was calculated to be, C_c = 1.37

On the basis of C_u and C_c the soil is classified as **poorly graded sand SP**.

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3.3 DIRECT SHEAR TEST

The direct shear test was conducted on the sand sample to find out the cohesion (c) and angle of internal friction (Φ) of the soil. The soil sample was subjected to three different normal loading i.e. 0.5 kg/cm^2, 1 kg/cm^2 and 1.5 kg/cm^2. The maximum shear force corresponding to the normal stress applied is plotted and the equation of the trendline is obtained to get the required parameters i.e. cohesion and angle of internal friction as shown in figure 3.2.

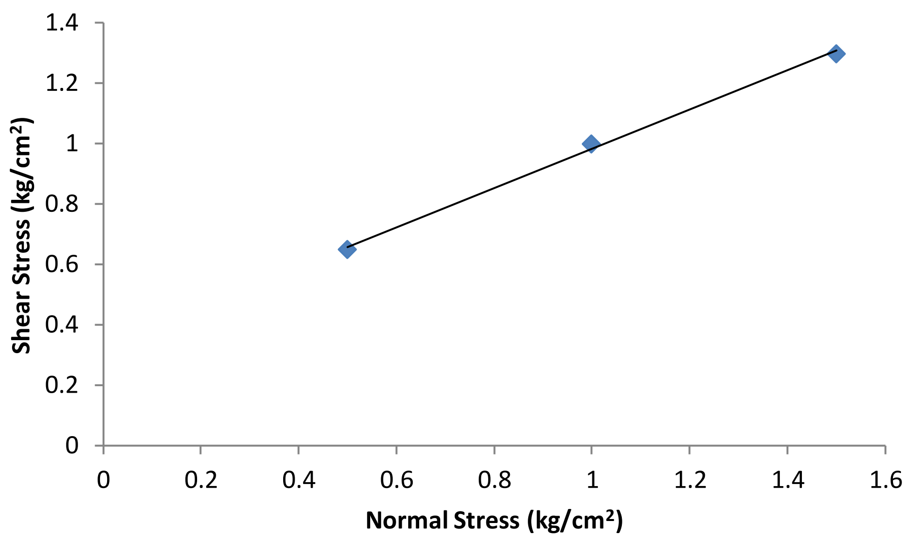

Figure 3.2. Direct Shear Test

The cohesion of the sand is found out by y intercept of the graph which is a very small numberas expected in the case of sand so it is taken as 0 for calcutions; the equation of the line is y = 0.65x + 0.33. The angle of internal friction is the tan^{-1} of the slope of the graph obtained which is 33^0.

3.4 MODIFIED PROCTOR COMPACTION TEST

This method is used to obtain the maximum dry density of a soil specimen. The soil is placed in a mould in five layers and each layer is given 56 blows of hammer. The test is repeated several times at different water content to obtain different densities. A graph is plotted between water content and density of the soil at the corresponding water content as shown in figure 3.3. The peak of the graph gives the maximum dry density of the soil sample and corresponding water content is the optimum moisture content of the soil sample.

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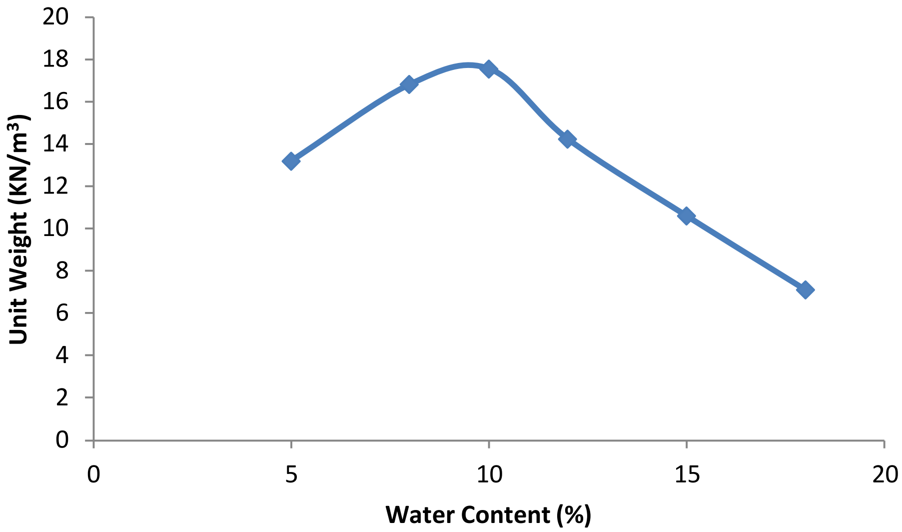

Figure3.3 Compaction Curve

All the properties of sand which were calculated are summarized in the table 3.3 below.

Table 3.3. Properties of Sand used

Parameters	Values
D_{10}	0.24 mm
D_{30}	0.33 mm
D_{60}	0.35 mm
C_u	1.45
C_c	1.37
Natural water content	3.77%
Maximum unit weight	17.95KN/m^3
Minimum unit weight	12.85 KN/m^3
Specific gravity	2.65
C	0.6 kg/cm^2
Φ	33^0

`

CHAPTER 4
DESIGN OF MICROPILES

4.1. NUMERICAL DESIGN

Numerical analysis is performed in civil engineering to analyse problems with complex geometries, loading conditions and material properties. In this project, the numerical analysis of the sand slope stabilisation was performed using the software SLIDE.The stability of the embankment, with and without micropiles, was analysed using the software.

4.1.1. ABOUT THE SOFTWARE

Slide is one of the most comprehensive slope stability analysis software available currently, which is complete with rapid drawdown, support design and sensitivity and probabilistic analysis, and finite element groundwater seepage analysis. All types of soil and slopes, retaining walls, earth dams and embankments can be analyzed. State of the art CAD capabilities allow you to create and edit complex models very easily.

It is the only slope stability software which has built-in finite element groundwater seepage analysis for transient conditions or steady-state conditions. Flows, pressures and gradients are calculated based on user defined hydraulic boundary conditions. Seepage analysis is fully integrated with the slope stability analysis or can be used as a standalone module.

Slide hasextensive probabilistic analysis capabilities and we may assign statistical distributions to any input parameters, including material properties, loads, support properties and location of water table. The probability of reliability/failure index is calculated, and it provides a measure of the failure risk associated with a slope design. Sensitivity analysis can allow us to determine the effect of various variables on the factor of safety of the slope.

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4.1.2. PARAMETERS INVOLVED

1. Units of measurement and failure direction.
2. Methods of analysis of slope failure (bishop's simplified and janbu simplified used in this case).
3. Material properties such as unit weight, saturated unit weight, cohesion and angle of internal friction.

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4. Support properties such as name of support, type of support (end anchored, geotextile, micropile or grouted tie back), force application (active or passive), out of plane spacing, pile shear strength and direction of force.

4.1.3. STEPS INVOLVED

1. First the embankment was constructed in the software and the sand properties were assigned to it.
2. Then the load was applied to the model embankment and then analysis was performed on it to determine the factor of safety of the global minimum failure plane.
3. Then the slope of the embankment was reduced till the embankment was just stable with a factor of safety of 1.1 on the maximum expected load.
4. The slope angle was found to be 29°.

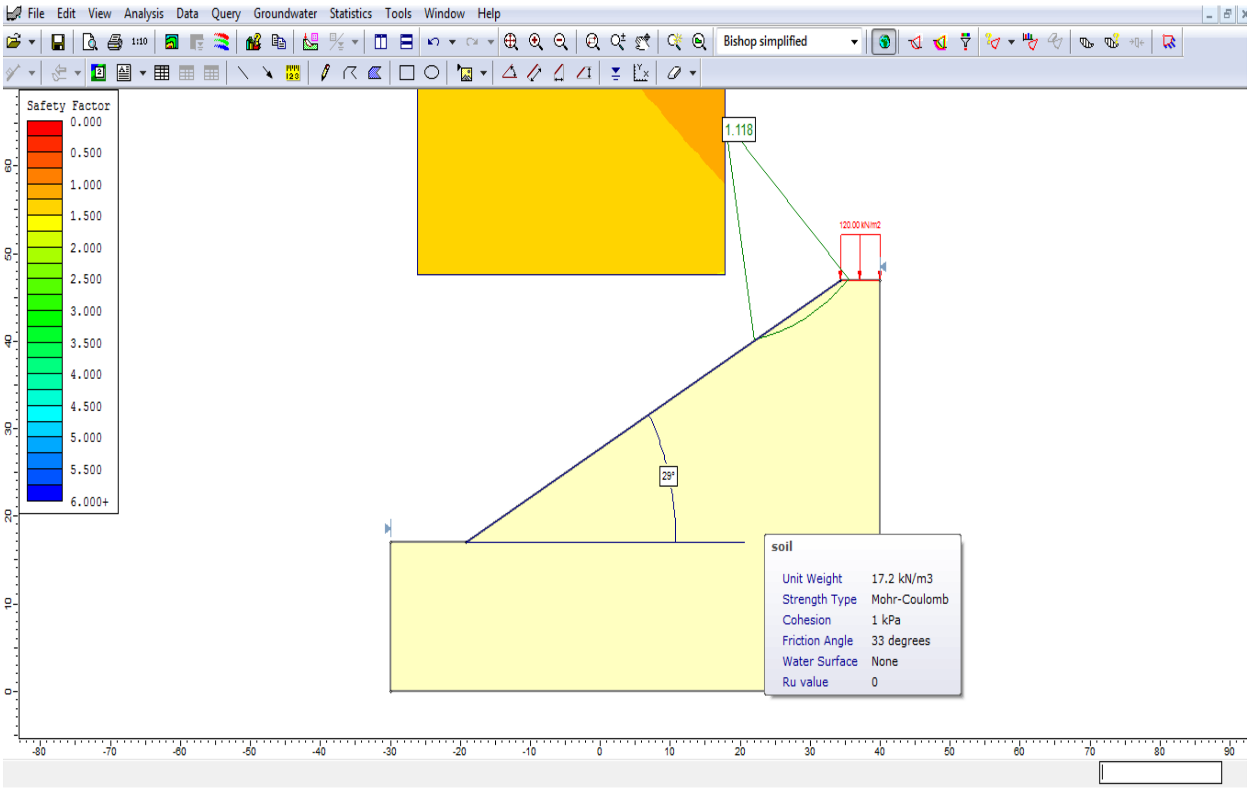

Figure 4.1. Minimum Slope of Embankment

5. A target factor of safety was then adopted for further analyses of theembankment, i.e. 1.3.

6. A micropile was introduced in the embankment. The resisting force provided by the micropile was entered manually and it was increased gradually till the embankment was stable with factor of safety of 1.3 or above.
7. The slope of the embankment was increased gradually and it was found that embankment was stable at a amximum slope of 87^{o}with the desired factor of safety against slope failure.
8. The resisting force and the factor of safety obtained of the micropile was noted down for further comparison.

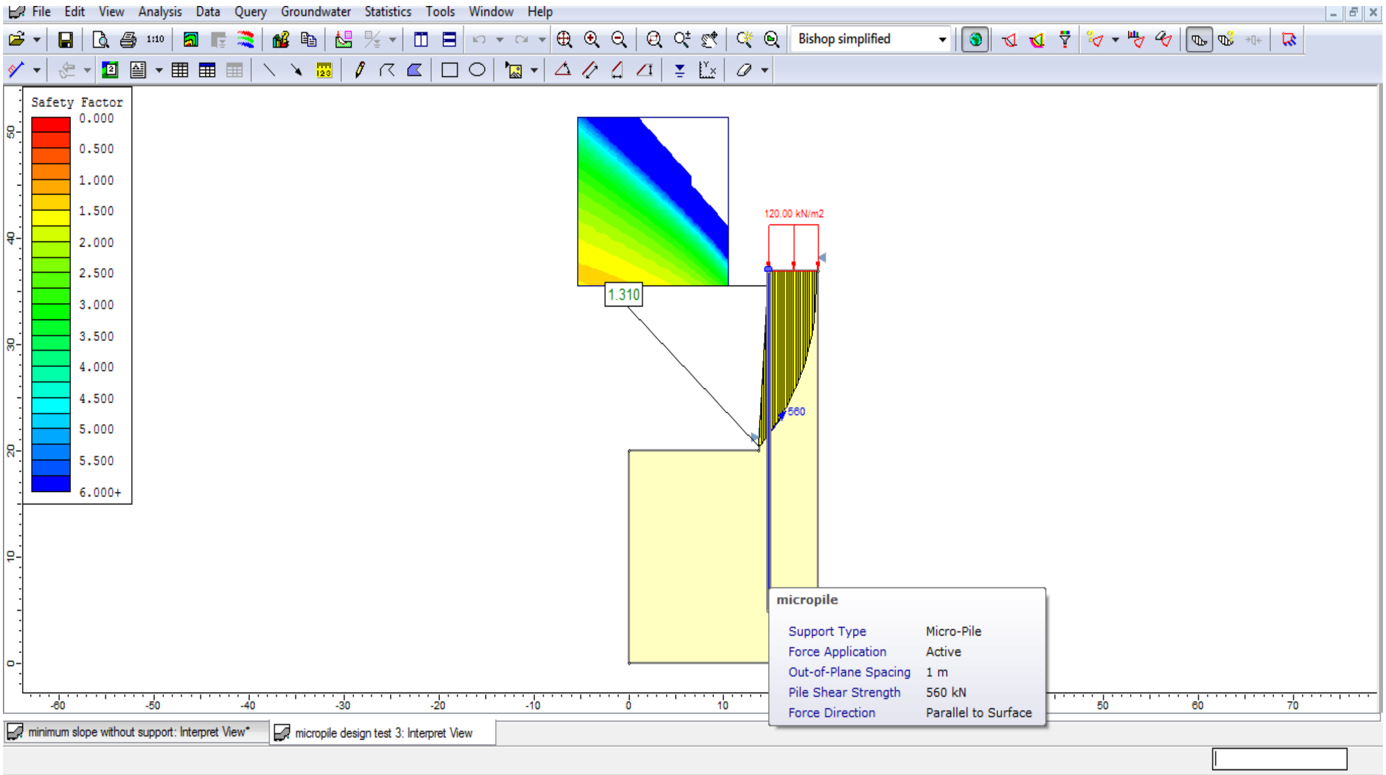

Figure 4.2. Embankment After Introduction of Micropile

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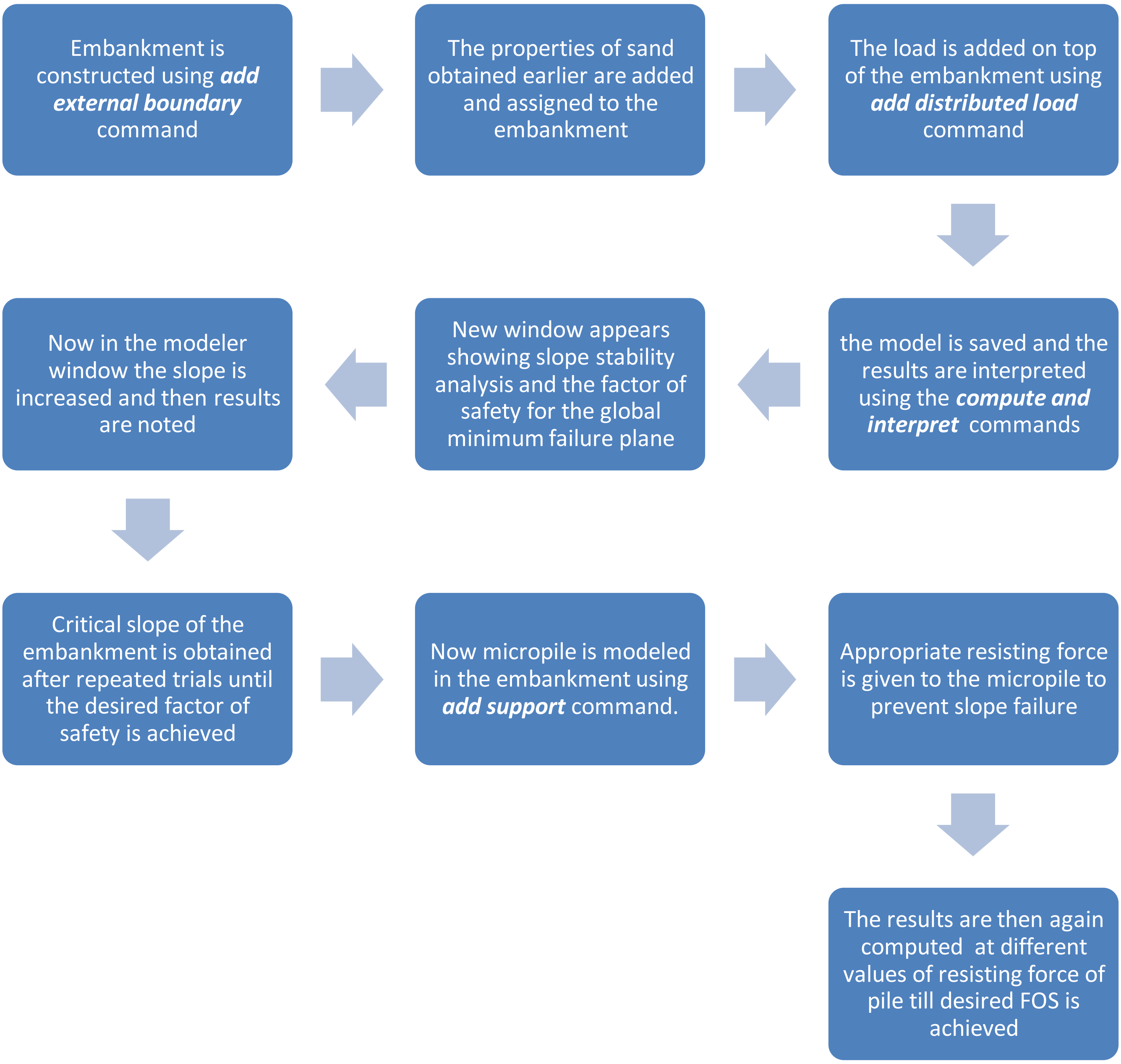

Figure 4.3. Flow Chat Showing Steps for Numerical Analysis

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4.2. ANALYTICAL DESIGN

1. A critical length of pile is found by using the L_{cr}/D ratio.
2. The ultimate pile load capacity is calculated, $Q_u = q_{pu}A_b + f_sA_s$

   Where,

   a. q_{pu} = unit point bearing resistance of the soil
   b. A_b= area of the base of the pile
   c. f_s= unit skin friction resistance
   d. A_s= surface area of the pile in contact with the soil

3. Then the load due to embankment is calculated.
4. A factor of safety of 1.5 is required and accordingly micropiles are designed.
5. In a similar way, micropiles were designed for diameter 200mm and 300mm and with varying depth i.e. 12m, 15m, 18m and 20m.
6. A minimum factor of safety of 1.5 was maintained in the design.
7. The resisting forces obtained at various depths of the micropiles of 200mm and 300mm were calculated and compared.
8. The spacing of the micropiles of varying diameter at different depths was also compared and the best configuration of micropiles was selected.

Example of design of micropile of diameter 300mm and depth of embedment 15m:

For concrete piles, $\delta = \frac{3}{4}\Phi$

Hence, $\delta = 24.75^o$

$L_{cr}/D = 15$, $L_{cr} = 4.5m$

Limiting effective stress at 4.5m = 17.2×4.5 = 77.4 KN/m^2

From 4.5m to 15m, effective stress = 77.4 KN/m^2

Skin friction over the length 4.5m:

Average effective stress = 38.7 KN/m^2

$f_{s(av)} = 38.7 \times 1 \times \tan 24.75^o = 17.84\ KN/m^2$

Skin friction resistance = $17.84 \times \pi \times 0.3 \times 4.5 = 75.66$ KN

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Skin friction over the remaining length

= (77.4×1×tan 24.75°)×π×0.3×10.5 = 353.089 KN

Total skin friction resistance,

Q_f = 353.089 + 75.66 = 428.743 KN

load from embankment

= (5.3×17.2) + (120×5.3) = 727.16 KN

Provide 2 micropiles in 1m.

Hence the resisting force = 1227.776 KN

Factor of safety = $\frac{1227.776}{727.16} = 1.6 > 1.5$

Spacing obtained = 800mm c/c

Hence provide 2 micropiles per metre length with a spacing of 700mm c/c

Now the external stability of the pile is checked and its factor of safety against overturning and sliding were computed using conventional earth pressure theories.

1. Factor of safety against overturning:

$$\frac{\Sigma M_R}{\Sigma M_O} \geq 2$$

2. Factor of safety against sliding:

$$\frac{(\ \Sigma\text{Vertical Loads}) \ + \ (\text{Shear capacity of piles})}{\Sigma\text{Horizontal Loads}} \geq 1.5$$

Now the external stability of the pile was checked and its factor of safety against overturning and sliding were computed using conventional earth pressure theories.

3. Factor of safety against overturning:

$$\frac{\left(0.5 \times 3.44 \times 17.2 \times 15^2 \times \frac{15}{3}\right) + (560 \times cos50° \times 16.520)}{\left(0.5 \times 0.29 \times 17.2 \times 32^2 \times \frac{32}{3}\right) + (0.29 \times 120 \times 32 \times \frac{32}{2})} = 2.13$$

Hence the pile is safe against overturning.

4. Factor of safety against sliding:

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$$\frac{(0.5 \times 3.44 \times 17.2 \times 15^2) + (560cos50°)}{(0.5 \times 0.29 \times 17.2 \times 32^2) + (0.29 \times 120 \times 32)} = 1.9$$

Hence the micropile structure is safe against sliding.

Similarly the safety of micropile structure against overturning and sliding were checked for all other cases and the factor of safety obtained for every case is shown:

4.3. PHYSICAL MODEL ANALYSIS

A model embankment wasconstructed with a suitable scaling factor i.e. 1/50 and the behavior of the model was observed. The failure pattern of the embankment and the deflection characteristics were observed and noted. The results obtained from analytical design were then verified with the help of physical model formed.

4.3.1. CONSTRUCTION OF EMBANKMENT

A poorly graded sand was taken for construction of embankment required for the physical model. The corresponding soil parameters were calculated. In order to obtain the best compacted model embankment, the embankment was constructed in 10-cm layers.

Figure 4.4. Model Embankment

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4.3.2. MICROPILE CONSTRUCTIONAND INSTALLATION

Micropile was made using a steel tube of diameter 18mm after taking into account the relevant scaling factors. To make the soil as a reinforced soil composite, the steel tube was perforated so that the grout can flow out into the soil and reinforce it. The grout was made using a water to cement ratio of 0.5.

For installation of each micropile, its location was first determined, and then the perforatedsteel casing was rammed and placed into the embankment. Then, the grout was injected into the steel casing.

After the installation of the micropiles, loading was applied with the help of mechanical weights on the embankment, without micropiles. Then the angle at which the embankment failed was noted.

After that micropiles were installed and loading was applied again to the embankment, and the slope was increased. As expected, we were able to increase the slope of the embankment with the help of micropiles and stabilise the slope.

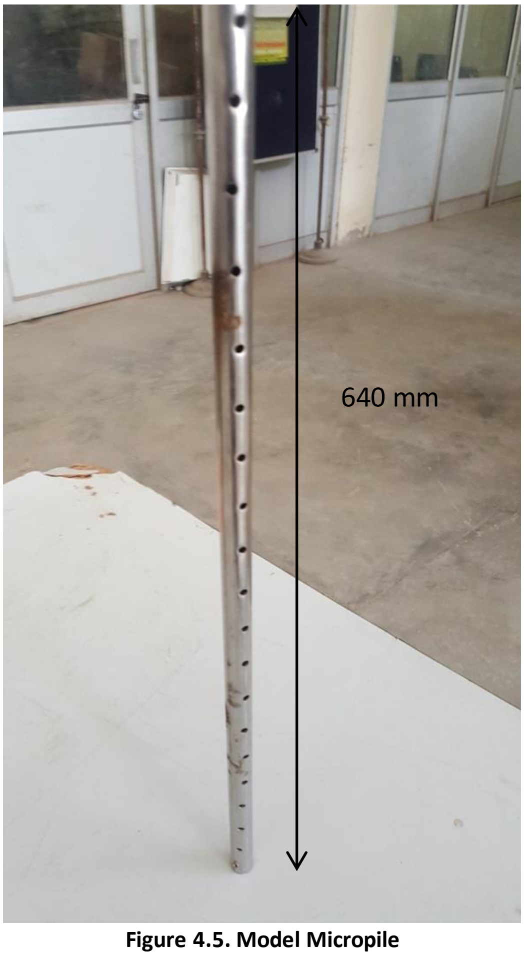

Figure 4.5. Model Micropile

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4.3.3. SCALING THE MICROPILE

Wood (2004) gave the following steps for the scaling the micropile properties.In order to scale down the micropile lengthfor a model, we are required to multiply the actual length of the micropile by the scaling factor of 1/50.

To scale down the properties of micropiles section, consider the micropile of prototype of equivalent diameter of a grouted section, d_p312.8mm, which was calculated based on the equivalentarea method.

We should consider scaling the soil stiffness while selecting the dimensions of micropile in the laboratorymodel. The diameter of the grouted section of the laboratory model can be given by the following equation:

$$\frac{(EI)m}{(EI)p} = (1/N)^{9/2}$$

Where N = scaling factor

After substituting theproperties of full-scale micropile in the above equation, the diameter of the equivalent grout section is calculated as18mm.

Table 4.1. Physical Model Scaling

Parameters	Prototype	Model	Scaling ratio(λ)
Top width	5.3m	.106m	1/50
Depth of cut	17m	0.34m	1/50
Length of micropile below the cut	15m	0.3m	1/50
Total length of micropile	32m	0.64m	1/50
Diameter of micropile	300mm	18mm	Woods formula

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4.3.4 PROCEDURE OF EXPERIMENT

1. The embankment was constructed in the container. The sand selected was a poorly graded sand with a unit weight of 17.2 kN/m^3 and angle of internal friction 33^0.
2. After proper construction of the embankment, the crest of the embankment was levelled and the slope was set at 43^0.
3. As calculated earlier the critical slope angle for embankment stability was calculated to be 29^0 so the slope was set higher than that to show the embankment failure at the applied load.
4. The loading was applied with the help of beams available in the lab. The beams were weighed and were arranged on the embankment so that uniform loading is applied on the embankment. It is shown in the figure 4.6.

Figure 4.6. Loading Applied on the Embankment

5. After application of loading, it was observed that the embankment started developing cracks on its slope which were small at first, but after sometime they propogated very and

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developed into big cracks as the initial slope of the embankment was fixed at 43^0. This is shown in the figure below 4.7a,b. The slope of the embankment was then gradually reduced to find the angle at which the embankment didn't develop any cracks at the applied loading. It was observed that the critical angle of slope failure was calculated to be 28^0, since the embankment did not develop any crack on its surface at this angle when the desired loading was applied.

Figure 4.7a. Cracks Developed due to Loading on Embankment Without Micropiles

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Figure 4.7b. Cracks Developed due to Loading on Embankment without Micropiles

6. After this, the loading was removed and the embankment was constructed again by the same procedure as adopted earlier.
7. Now, the positions of micropiles were marked and the installation of micropile was done by driving them through the sand using hands at first and then when the micropiles reached a significant depth slight blows of hammer were used to drive the piles into the sand.
8. The cross section of the micropile is shown in the figure 4.8.

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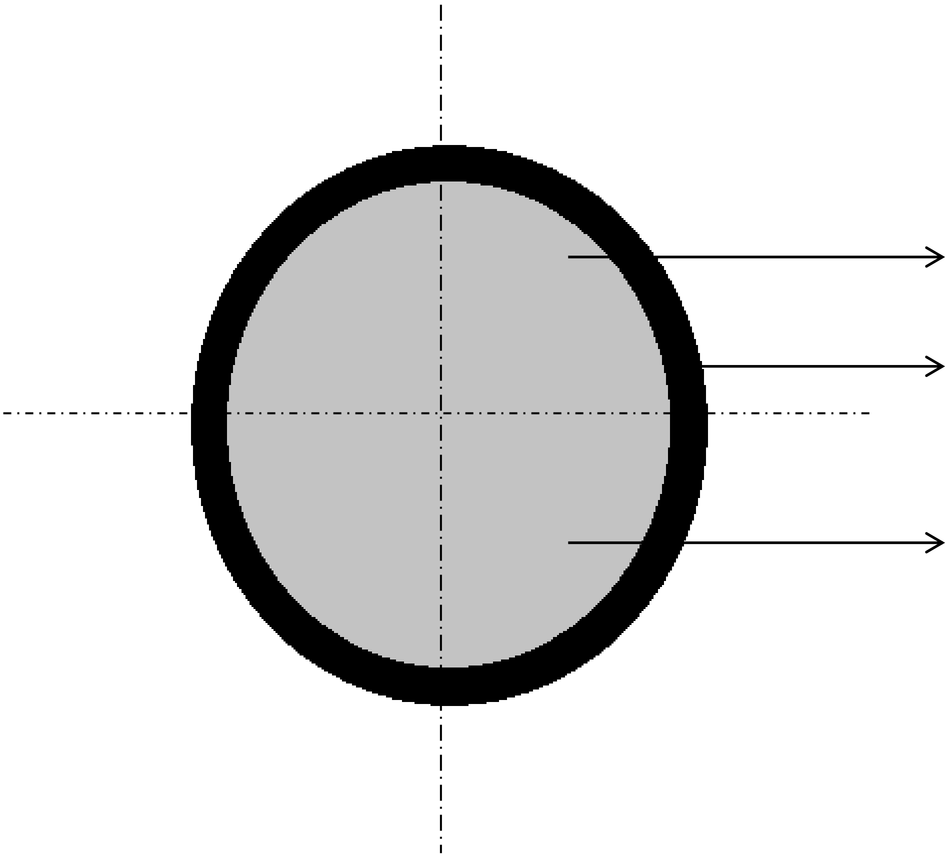

Figure 4.8. Micropile Cross Section

9. After the installation of micropiles, the grout was inserted into the micropiles under pressure so as to make the grout reach the surrounding soil to form a reinforced soil composite. It can be seen in the figure 4.9a,b.

Figure 4.9. Grout Veins Formed in the Sand

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10. After insertion of grout into the micropiles, the loading was applied on the embankment. The initial slope of the embankment was again kept at 43^0.
11. Then after checking the embankment for the presence of any cracks on its surface, the slope of embankment was reduced gradually by removing the soil in front of micropiles so as to increase the slope as shown in fig 4.10.
12. The maximum slope thus obtained at which the embankment was stable was noted down.
13. The slope of the embankment was gradually reduced and the maximum slope obtained was noted down.

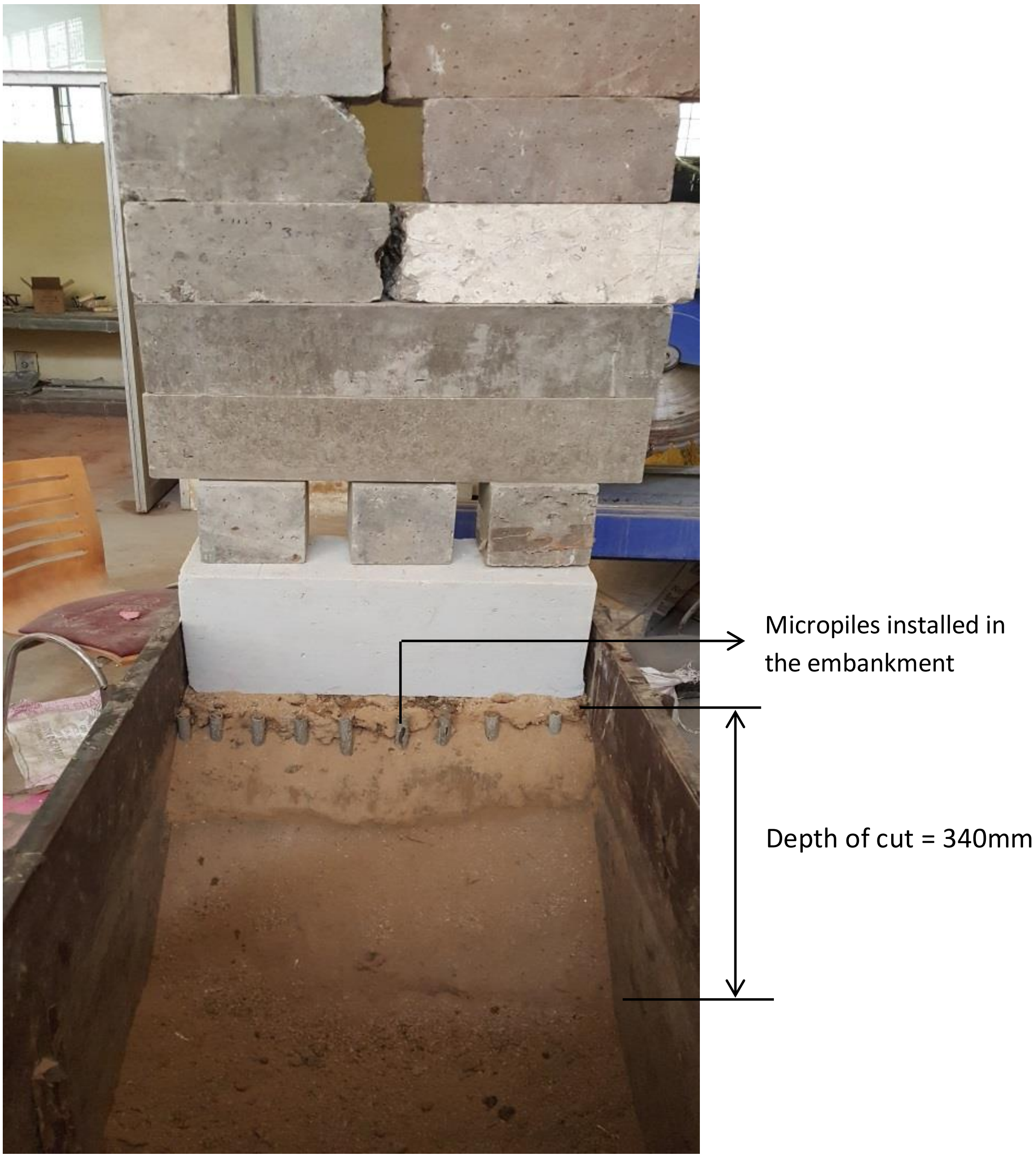

Figure 4.10. Loading Applied on Embankment after Introduction of Micropiles

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CHAPTER 5
RESULTS AND DISCUSSIONS

The critical slope angle of the embankment as calculated through numerical method was calculated to be 29^0.

The micropiles of varying diameter i.e. 200mm and 300mmand varying depth of embedment i.e. 12m, 15m, 18m and 20m were designed and their corresponding resisting force at a constant factor of safety were recorded and is shown in the figure 5.1.

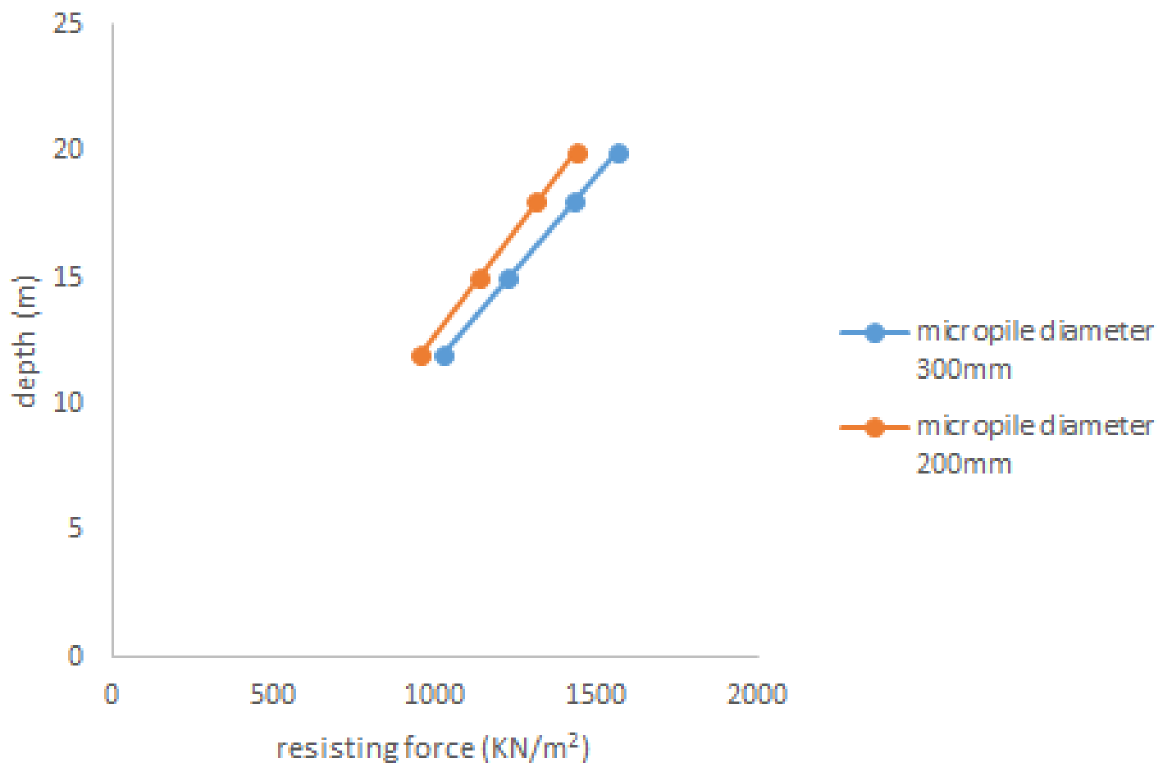

Figure 5.1. Comparison of Resisting Force of Micropiles of Different Diameter

The micropile design was carried out to compare the spacing of the 200mm and 300mm of the micropile at varying depths of embedment, i.e. 12m, 15, 18m and 20m. This is shown in the table 5.1 and 5.2.

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Table 5.1. Comparison of Spacing of Micropiles (300mm) at Different Depths

Diameter of micropile (mm)	Spacing (mm)	Depth (m)
300	580	12
300	700	15
300	820	18
300	1050	20

Table 5.2. Comparison of Spacing of Micropiles (200mm) at Different Depths

Diameter of micropile (mm)	Spacing (mm)	Depth (m)
200	320	12
200	390	15
200	450	18
200	493	20

After the design of the micropile and comparison of the pile load carrying capacity, the micropiles were checked for external stability against the applied load. The factor of safety against sliding and overturning against the applied load was checked and compared for the micropiles of 200mm and 300mm diameter. It is shown in the table 5.3 and 5.4.

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Table 5.3. External Stability Check of Micropile(300mm)

Diameter of micropile (mm)	Depth (m)	Factor of safety against overturning	Factor of safety against sliding
300	12	1.95	1.7
300	15	2.1	1.9
300	18	2.24	2.25
300	20	2.5	2.4

Table 5.4. External Stability Check of Micropiles (200mm)

Diameter of micropile (mm)	Depth (m)	Factor of safety against overturning	Factor of safety against sliding
200	12	1.76	1.35
200	15	1.9	1.48
200	18	2.03	1.62
200	20	2.15	1.74

The physical model was constructed and the loading was applied on it. The loading was applied on the embankment using beams that were not in use in the lab. The slope of the embankment at which no cracks developed on the embankment at the desired loading was noted down. It was found out to be 28^0.

Then the micropiles were installed on the embnakment and the loading was applied on it. The slope was then reduced gradually to find the maximum slope of the embankment at which it is stable. The maximum slope, after installation of micropiles, was calculated to be 83^0.

Thus a major increase in slope was obtained after the installation of micropiles.

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5.1. COMPARISON OF NUMERICAL AND ANALYTICAL METHODS

The factor of safety obtained by numerical and analytical analysis of micropiles of diameter 200mm and 300mm and at different depths of embedment were compared. It was found that the results were comparable and the results were close with very less difference. It is shown in the figure 5.2 and 5.3.As expected the factor of safety was greater when micropiles of larger diameter were used.

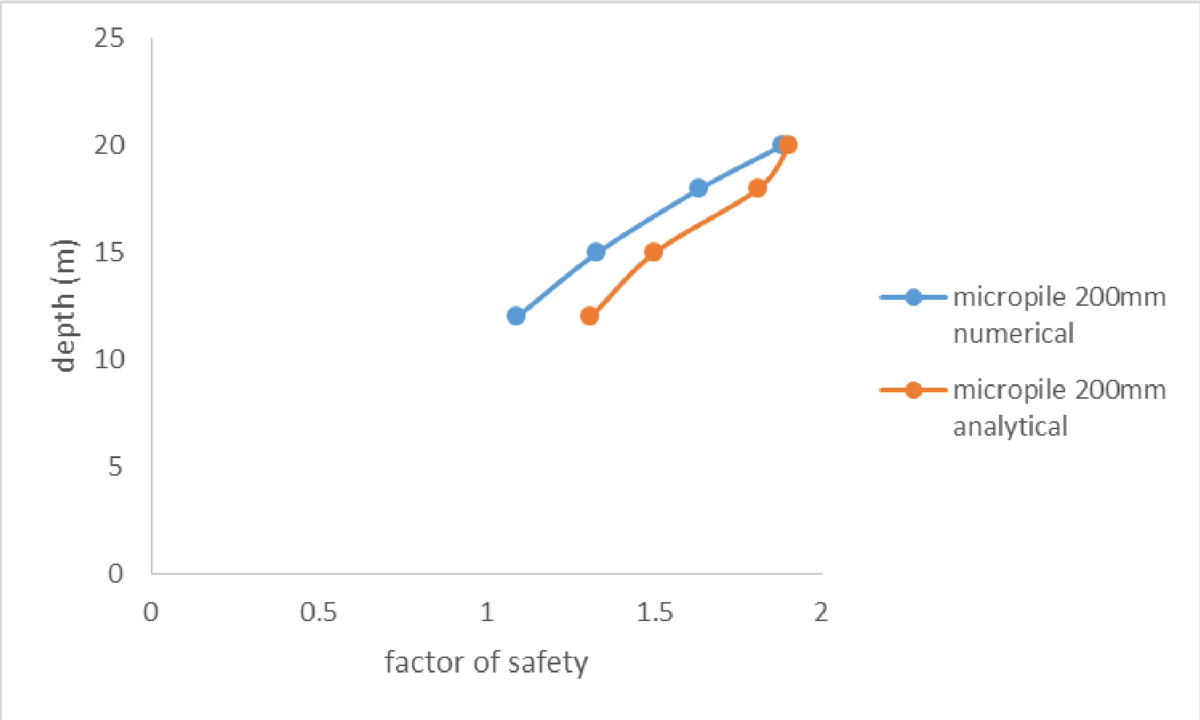

Figure 5.2. Comparison between Numerical and Analytical Analysis Of Micropile Of 200mm

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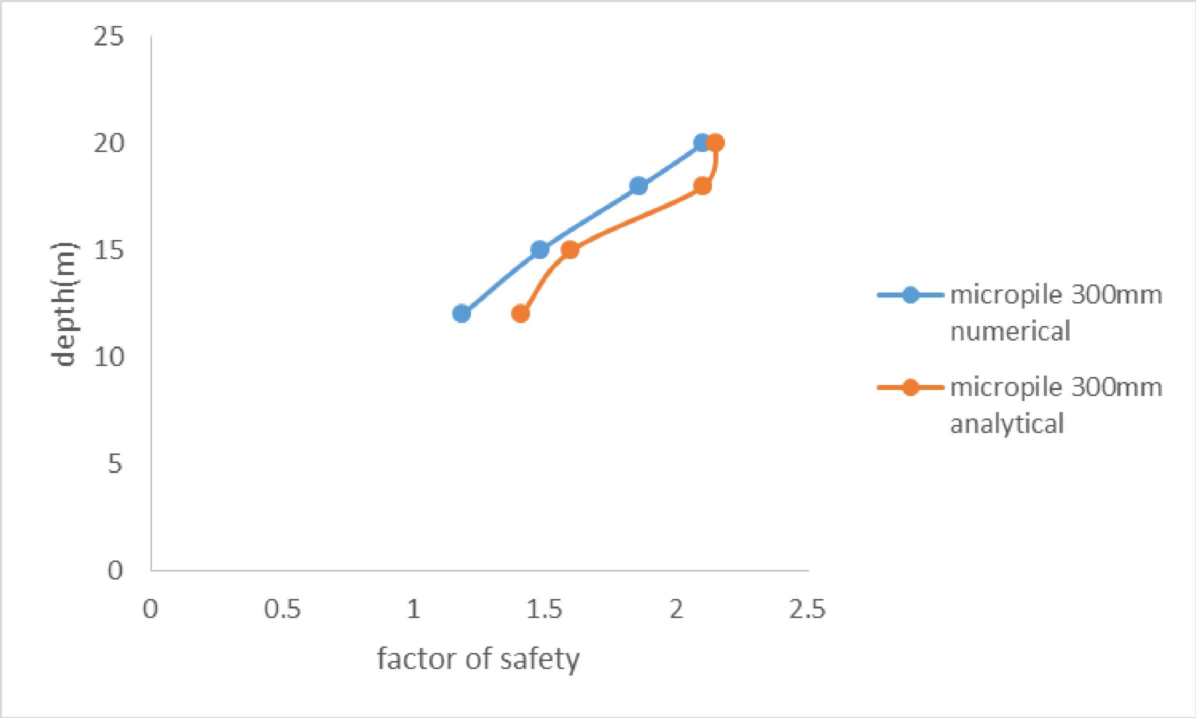

Figure 5.3. Comparison between Numerical and Analytical Analysis Of Micropile 300mm

After the comparison of all the cases, micropile with diameter 300mm and depth 15m is the best option with c/c spacing of 500mm so as to provide 2 micropiles per metre length of the embankment. This specification of micropile was seleted on the basis of providing the maximum resisting force at maximum spacing possible and minimum depth of embedment

A casing of wall thickness 10mm will be used and outer diameter 300mm. Pile casing steel area = 9110.6mm^2.Yield strength of casing = 240 MPa. Grout compressive strength = 30MPa

5.2. COMPARISON OF NUMERICAL & PHYSICALMODEL

As it was calculated earlier through numerical modelling the critical angle of slope failure of embankment was found to be 29^0. After modelling the same in the physical model it was found that the embankment was stable at 28^0. So the results as obtained by numerical analysis were very close to each and were verified using physical model. Hence, the numerical analysis was correct as its results were found in close comparison with the physical model with an error of 3.44%.

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The embankment was analysed for slope stability after introducing the micropiles in the embankment to increase the slope. It was found that the slope of the embankment was increased from 29^0 to 87^0.

The same was modelled in a physical model using appropriate scaling factors and it was found that after the introduction of the micropiles the slope of the embankment increased from 28^0 to 83^0. This is comparable to the results obtained by numerical analysis as the results are very close with an error of 4.6%.

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CHAPTER 6
CONCLUSIONS

The main conclusions drawn from this dissertation work are as follows:

1. The numerical analysis of the model embankment showed that the critical slope angle of the embankment was 29^0. At this angle the embankment can be stable without the application of micropiles.
2. In the numerical analysis of the embankment, micropile was introduced in the embankment and the resisting force of 560KN of the micropile gave the factor of safety above 1.3 which was required.The slope of the embankment increased from 29^0 to 87^0 after the introduction of micropiles.
3. After conducting the analytical analysis of micropiles, it was found that the pile load capacity of the micropiles increased with an increase in the diameter of the micropile at the same depth and at the same factor of safety.
4. Analytical analysis of the micropiles of constant diameter at different depths showed that the pile load capacity of the micropile increased with an increase in the depth of embedment of the micropiles on account of the increase in the passive resistance of the soil developed in front of the pile.
5. Comparing the spacing of the micropiles at constant diameter showed that the spacing of the micropiles increased after an increase in the depth of the micropile due to increase in the passive resistance of the micropile which inturn increases the pile load capacity.
6. After finishing the analytical analysis the best possible configuration of micropile was selected, with minimum diameter and maximum spacing and appropriate factor of safety, was selected which isa micropile with diameter 300mm and depth 15m is the best option with c/c spacing of 500mm so as to provide 2 micropiles per metre length of the embankment.
7. The prototype embankment was modelled into a physical model by scaling down the various parameters using a scale of factor of 1/50.The critical slope of the embankment was calculated to be 28^0 in the physical model.
8. After the introduction of micropile, the slope of the physical embankment at the desired loading increased from 28^0 to 83^0.
9. The results of the physical models were compared to the numerical and analytical analyses of the micropile and it was found that the results were very close to each other with very less error. This proved the validity of the numerical and analytical analyses.

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10. Thus, micropile can be successfully used to reinforce steep slopes and can be effectively used as an earth retention systemwhere the space available is less as it does not take up the construction space like other earth retaining structures.

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CHAPTER 7
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