Stability Analysis of Retaining Wall using GEO5 in Kuranchery

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ABSTRACT

In recent times the natural hazards in India & all round the world are increasing day by day. One such landslide was in Kuranchery village in Thekkumkara Panchayath, Thissur. Analysis of the soundness of earth retaining structures in such slopes and embankments could be a tough geotechnical task. The software package GEO5 permits geotechnical engineers to hold out stability analysis of the retaining wall designed. it's a simple to use suite that consists of individual programs with a unified and easy interface. retaining walls are structures that are accustomed retain earth (or the other material) in a position wherever the ground level changes suddenly. The 'cantilever wall' is that the most common variety of retaining wall and is economical up to regarding 8 m. Counter fort walls ar appropriate for holding wall heights 8.0m to 10.0m. The lateral force because of earth pressure is that the main force that acts on the retaining wall which has the tendency to bend, slide and overturn it. This paper shows application of GEO5 slope stability software package to evaluate stability of the retaining wall designed to stop more landslides within the space. in order to enhance stability of the backfill, crusher dust is employed that has fine particles like soft sand, crusher dust is used as an economical filling and packing material.

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ABBREVIATIONS

CBR	California Bearing Ratio
MDD	Maximum Dry Density
UCS	Unconfined Compressive Strength
OMC	Optimum Moisture Content
FOS	Factor of Safety
FLAC	Fast Lagrangian Analysis of Continua
FEM	Finite Element Method
MSL	Mean Sea Level
CEC	Cation Exchange Capacity
ECEC	Effective Cation Exchange Capacity

PDF	Portable Document Format
MS word	Microsoft word
IS	Indian Standard
ASTM	American Society for Testing and Materials
PI	Plasticity index
LL	Liquid Limit
PL	Plastic Limit
BM	Bending Moment
CI	Clay- Intermediate
NP	Not Possible
SW	Well graded sand

NOTATIONS

K_v	Vertical Seismic co-efficient
K_h	Horizontal Seismic co-efficient
Km	Kilometre
K_{zy}	Kozhukully series
$\underline{f}$	Gravelly Clay loam texture
F	Percentage of slope
St	Stone cover
R	Rock cover
pH	power of hydrogen
H^+	Hydrogen ion
Al^{3+}	Aluminium ion
Na^+	Sodium ion
K^+	Potassium ion
Ca^{2+}	Calcium ion
Mg^{2+}	Magnesium ion
w	Water content
G	Specific gravity
e	Axial strain
A	Area of cross section
q_u	Axial stress

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A_o	Initial area of cross section
L_o	Initial length of sample
%	Percentage
mm	Millimetre
m	Metre
W_s	Surcharge pressure on backfill
kN/m^2	Kilo newton per metre square
β	Slope of the backfill
γ	Unit weight of the soil
ϕ	Angle of internal friction
H	Overall Height
K_a	Active pressure co-efficient
K_p	Passive pressure co-efficient
l	Center to center distance between the counterforts
h	Height of the backfill
b	Base width
q_o	Safe bearing capacity
e	Eccentricity
Y_{min}	Depth of foundation
μ	Co-efficient of friction
A_{st}	Area of steel
P_t	Percentage of steel
M_u	Ultimate bending moment
d	Depth
θ	Angle
ϕ	Diameter
f_{ck}	Characteristic compressive strength
f_y	Characteristic yield strength
τ	Shear strength
δ	Soil wall interface friction
kPa	Kilopascal
>	Greater than
<	Less than
ρ	Reinforcement ratio

CHAPTER 1
INTRODUCTION

1.1 GENERAL

The Kuranchery landslide made news on August 16, 2018 when a portion of the hill on the Machad forest range came down and washed away four houses killing 19 people. Although landslide is a natural, geological phenomenon involving land movement it can be truly devastating when it occurs on someone's property. It can occur in offshore, coastal and onshore environments when there is a specific sub-surface condition. However usually there has to be a trigger either a natural or human cause such as soil erosion, earthquakes, melting glaciers, deforestation, cultivation, construction or vibrations from traffic. Landslides can affect a limited area or can be true natural disasters. Whatever the situation, finding a solution is essential in preventing further damages such as building damages or roadblocks. An unprecedented rainfall and an underlying sloping stratum of clay sediment can be blamed for much of the destruction. The thick clay layer acted like a stopper, essentially trapping the rainwater percolating through the soil above it and groundwater levels rose below. The wetted earth material then built up over days into a weighty, muddy mass that slid off the surface of the clay table, sending all the earth and homes above it toppling down below.

1.2 RETAINING WALLS

Retaining walls are structures designed to restrain the soil (or the other material) in a position wherever the bottom level changes suddenly. They are usually employed in areas with steep slopes or wherever the landscape has to be shaped severely for construction or engineering projects. However, retaining walls are found to be a awfully economical answer against landslides. There are numerous ways in which of constructing a retaining wall, the foremost common varieties being:

a. Gravity walls: they manage to resist pressure from behind due to their own mass
b. Piling walls: made of steel they are usually used in tight spaces with soft soil having 2/3 of the wall beneath the ground
c. Cantilever walls: they have a large structural footing and convert horizontal pressure from behind the wall into vertical pressure on the ground below
d. Counter fort walls: they are suitable and economical for retaining wall heights 8.0m to 10.0m.
e. Anchored walls: they use cables or other stays anchored in the rock or soil behind to increase resistance

The type of wall that will be used depends on the circumstances of every case. Soil type, slope angle, groundwater characteristics and other specifics will be considered before deciding on the proper solution.

The lateral force due to earth pressure is the main force that acts on the retaining wall which has the tendency to bend, slide and overturn it. The present thesis focuses on the stability analysis and designing the counter fort type of wall. The main considerations are the external stability of the section and the adherence to the recommendations of IS 456:2000. Satisfying the external stability criteria is primarily based on the section giving the required factor of safety. The ratio of resisting forces to the disturbing forces is the factor of safety, and this factor of safety should always be greater than unity for the structure to be safe against failure with respect to that particular criteria. Different modes of failure have different factors of safety.

1.3 STABILITY ANALYSIS

In older times, the stability analysis is done by using graphs or hands. The conventional methods used for the analysis are Limit Equilibrium methods. The method is mainly three types. Swedish circle method, Friction circle method and Bishop's method. Nowadays all analysis can be done through software. GEO5 is such advanced software suitable for solving geotechnical problems based on traditional analytical method and Finite Element Method. Basic geotechnical approaches implemented in the GEO5 programs are applicable all over the world. GEO5 offers a unique way of applying standards, which significantly simplifies the work of a designer and at the same time, allows for complying with all required approaches. It is an accurate and easy to use tool in all geotechnical problems. The output of the GEO5 analysis is factor of safety, defined as the ratio of the shear strength to the shear stress required for equilibrium. The factor of safety is determined for heights of wall with varying depths of soil and crusher dust as backfill material. If the value of factor of safety is less than 1.5, the wall is unstable. For the safe standing of retaining wall, it is necessary to maintain the factor of safety.

1.4 RESEARCH OBJECTIVES

The main objective of this thesis is the proposal for the construction of Retaining wall in Kuranchery and stability analysis using GEO5 software**.**

The specific objectives are:

a. To Determination of basic properties of foundation soil and fill material.
b. To analyze the stability of a retaining wall using GEO5 software with crusher dust as backfill at various depths
c. Stability of the counterfort retaining walls are to be analyzed at different heights.

1.5 SCOPE OF THE WORK

The thesis mainly aims at creating an earth resisting structure and to retain the soil slope. It is intended to be a preventive measure to resist the harsh environmental hazards and resisting settlement. Through the use of crusher dust as backfill, an efficient utilization of waste materials is also made. Adding crusher dust as backfill material along with the soil of the area improves the properties of the soil as thereby increases the strength of the fill.

1.6 THESIS OUTLINE

The thesis consists of 10 chapters. Chapter 1 consists of introduction part of the research work, objectives and scope of the study. Chapter 2 describes literature reviews related to the study. Chapters 3, 4, 5 and 6 describes the study area, various input data collected for study, software and materials used and also the step by step procedure adopted for the analysis. Chapter 7 describes about the various experiments conducted on soil and crusher dust. Chapter 8 gives the brief description of the steps involved in design of retaining walls. The results of the tests and stability analysis are evinced in the chapter 9. Chapter 10 provides the summary and conclusions based on an overall review of the results obtained from the current study. Further scope for improvement is also included in the chapter.

CHAPTER 2
LITERATURE REVIEW

2.1 GENERAL

Design and analysis of retaining wall requires the determination of soil parameters and appropriate techniques for the analysis of stability. There are different techniques adopted in the following literatures to assess the soil properties, design the retaining wall and analyse its stability. To achieve a greater level of accuracy, the developer needs to study characteristics of different methods and also determine the appropriate method for the situation before its usage in real application. The choice of the method is one of the important elements that have an influence the accuracy of analysis.

2.2 REVIEW ON STUDIES

C.N.V. Satyanarayana Reddy.etal (2015) conducted a study on unstable style of the bolstered soil retentive walls with sand and device dirt as fill materials. The study indicated higher stability of rock flour wall over bolstered sand wall. In additionally showed that the soundness of retentive walls will increase with increase in friction angle of fill material with reinforcing material.

P.V.V. Satyanarayana (2013) conducted a study on the performance of crusher dust as a fill material rather than red soil and sand. Through the study it had been found that, device mud particles are the same as sand particles. It offers additional shear strength at wider variation of wet contents and additionally maintains high dry densities. It will with stand high strengths in terms of CBR and angle of shearing resistance. Crusher dust will so be used as a decent fill material for subgrade.

Yash Chaliawala and Gunvant Solanki (2015) created a comparative study of cantilever and counter fort wall. Priced against every optimum style of wall for explicit height was calculated by exploitation quantity of concrete and therefore the amount of steel. It absolutely was found that Cantilever retaining walls are unit economically appropriate for all heights up to six meter and Counter fort walls are unit appropriate for retaining wall of height about eight meter to ten meter for the traditional conditions assumed.

Naman Agarwal (2015) studied the result of stone dust on some geotechnical properties of soil. He found that adding fifty percentage of stone dust is effective in decreasing optimum wet content of soils that is advantageous in decreasing amount of water needed throughout compaction. The study reveals the actual fact that with increase within the proportion of stone dirt, MDD of soil will increase. The compounding of soils with stone dust is additionally found to enhance its cosmic radiation. There is a good result on the relative density of the soil on the compounding of stone dust with them. Adding thirty percentage of stone dirt is found to be optimum just in case of relative density.

A.Sridharan (2005) conducted the shear strength studies on soil-quarry dirt mixtures. It had been all over that, the majority utilization of crusher dust waste matter was attainable through geotechnical applications. The paper presents the shear strength behavior of quarry dust and soil-quarry dust mixes.

C.H. Juang (1998) created the steadiness analysis of existing slopes considering uncertainity. It absolutely was a way of addressing soil parameter uncertainity in stability analysis of slopes. The soil parameter uncertainity within the stability analysis of an existing slope was taken into consideration within the study.

Anissa Maria (2015) studied the form of slide surface of gravity retaining walls constructed on sand by the tiny scale curved dynamic load tests. It had been ascertained that soils and structures receive the static load of the building made each within and on the surface and also dynamic loads. Laboratory modeling experiments were conducted to check the movement of soil grains and numerous dynamic hundreds were analyzed.

Sabat (2012) conducted series of tests and it was found that over that addition of quarry dust decreases Liquid limit, Plastic limit, physical property index, Optimum wet content, cohesion and will increase shrinkage limit, most dry density, Angle of internal friction of expansive soil.

Ali and Koranne (2011) conferred the results of an experimental programme undertaken to analyze the impact of the stone dust and the ash compounding in numerous percentages on expansive soil. They discovered that at optimum percentages, i.e., twenty to half-hour of admixture, the swelling of expansive clay is sort of controlled and there's a marked improvement in alternative properties of the soil likewise. It's terminated by them that the mix of equal proportion of stone dust and ash is more practical than the addition of stone dust/fly ash alone to the expansive soil in dominant the swelling nature.

Bshara et al. (2014) reported the impact of stone mud on geotechnical properties of poor soil and terminated that the CBR and MDD of poor soils is improved by compounding stone dust. They additionally indicated that the liquid limit, plastic limit, physical property index and optimum wet content decrease by adding stone mud that successively will increase quality of soil as road sub-grade material.

Soosan et al. (2001) known that crusher dust exhibits high shear strength and is useful as a geotechnical material. Stone dust could be a material that possesses pozzolanic additionally as coarser contents in it whereas different materials like ash possesses solely pozzolanic property and no coarser soil particles. Important improvement within the properties of soils is rumored by totally different researchers by admixture it with stone dirt. During this study stone dirt by dry weight of soil was taken as 100%, 20%, 30%, 40% and 50% taken and mixed with the soil so as to examine the impact of blending on OMC, MDD and CBR properties of soil.

Slaman et.al (2011) studied the planet pressure distribution behind holding walls subjected to line load. The planet pressure distribution generated behind a twenty meter high wall was calculable by the finite part technique and compare thereupon obtained from classical earth pressure theories. From the analyses, author had found that the most pressure is within the wall base. The worth of the lateral earth pressure at the wall base is concerning (10 to 20%) but that obtained by Coulomb equation.

A.Hossain, M. A. A. Sadman , M. M. Rashid , and M. Ashikuzzaman (2019) studied the seismic Stability of Slopes in Cohesive Soils exploitation GEO5 software package. LEM module of GEO5 software package has been accustomed to analyze a uniform slope model with clay sort soil. From the study it had been summarized that: a) For all ratios of Kv/Kh, issue of safety decreases with increase of horizontal seismic constant Kh, considering all ways of research. b) For all ratios of Kv/Kh, issue of safety will increase with the rise of cohesion worth considering all ways of research.

Ali Akbar Firoozi, Ali Asghar Firoozi and Mojtaba Shojaei Baghini (2016) created a review on the clayey soils. The geotechnical properties of soil like its grain size distribution, shear strength, softness, plastic limit, liquid limit was outlined by correct laboratory testing. Moreover, the in place determination of strength and deformation properties of soil was created, as a result of this technique avoids perturbing samples throughout field examination. Two main processes could involve slight physical and chemical alteration or decomposition and recrystallization. Moreover, the clay minerals and soil organic matter area unit colloids. And also the most significant property of colloids is their tiny size and huge area. It absolutely was found that, the clay particles play a really necessary role within the chemical process that take play in soil and influence the movement and retention of contaminants, metals, and nutrients within the soil.

Mr.Utkarsh Mathur, Mr.Nitin Kumar, Mr.Trimurti Narayan Pandey and Mr.Amit Choudhary (2017) studied the index properties of soil. Easy check was needed for index properties, called classification check. The check needed for the determination of engineering properties was found to be elaborate and time overwhelming. The index properties are given some information concerning engineering properties like permeability, compressibility and shear strength. It absolutely was tacitly assumed that soils with like index properties have identical engineering properties.

Chugh (2005) conducted finite part analysis for a model of cantilever wall and model counterfort wall victimization FLAC second and 3D. The discretization of the wall into finite-difference grid affected the natural frequency of free vibrations; the grid size effects were a lot of pronounced for moving response within the transverse direction than within the axial direction. The numerical results of natural frequency were found to be in agreement with those of the legendary analytical solutions.

Salman etal. (2011) calculable the world pressure distribution behind a 20m high wall using the two-dimensional finite component code, CRISP. The results showed oscillations within the values of earth pressure as a result of the appliance of line hundreds. These oscillations within the higher 1/2 the wall was found to extend with the increasing load and reduce with the decreasing section of the load. Within the lower 1/2 the wall, the lateral earth pressure was near the linear distribution with the utmost price at the bottom.

Clough and Duncan (1971) computed the response of six meter high gravity wall placed on six meter deep sand foundation experimented earlier by Terzaghi (1934). The analysis was performed victimization one-dimensional parts to simulate the interface between the wall and also the backfill. The minimum active and most passive pressures were found to be in sensible agreement with the results of the classical earth pressure theory, whereas, the quantity of movement needed for reaching the total active and full passive conditions was found to be in sensible agreement with the results of Terzaghi (1934).

2.3 SUMMARY

The review indicates that the employment of crusher dust as a fill material together with the soil helps in up the soil properties and ensures higher stability for the structure. Each classical and analytical ways are used for the soundness analysis of retaining walls. In the present study, analysis using GEO5 software system could be adopted because it is a combination of both the analytical technique and Finite part technique (F.E.M.). Analytical verification methods offer effective and rapid structure design and verification. It's doable to transfer analytical model into a F.E.M. program wherever the structure is verified by finite part technique. Comparison of independent solutions contributes to increasing the safety and protection. The aim of the study is to analyse the soundness of wall designed, using GEO5.

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CHAPTER 3

STUDY AREA

3.1 GENERAL

Kuranchery is a small Village/hamlet in Wadakkanchery Block in Thrissur District of Kerala State, India. It comes under Thekkumkara Panchayath. It belongs to Central Kerala Division. It is located 14KM towards North from District headquarters Thrissur, 4KM from Wadakkanchery and 293KM from State capital Thiruvananthapuram.

3.2 SATELITE VIEW

The satellite view of Kuranchery is shown in fig 3.1

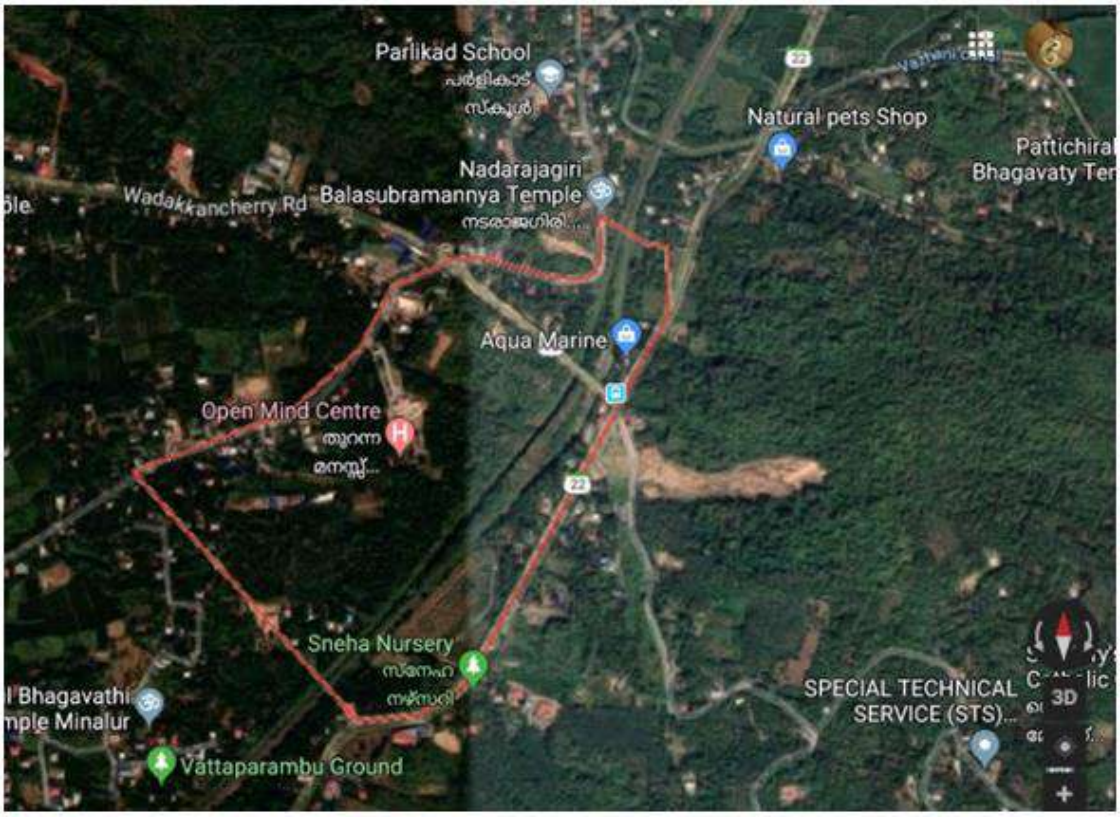

Fig 3.1: Location of Kuranchery

CHAPTER 4
DATA COLLECTION

4.1 GENERAL

The data relating to the soil details of Kuranchery area (Thekkumkara panchayath) was collected from the Office of the Assistant Director, Soil Survey and Conservation Department, Thrissur

4.2 SOIL DETAILS OF KURANCHERY

Kzyf_F2 St2R1 Ives 4st: Kozhukully series, gravelly clay loam 15 – 25% slope, 0.1 to 0.3 % stone cover, and 2 to 10 % rock cover

Kozhukully series is deep, brownish, strongly acidic, well drained, moderately eroded, with gravelly clay loam surface texture occurring on moderately steep to steep lands. Stone cover 0.1 to 3 % and rock cover 2 to 10 % of the surface. These are fairly cultivable lands which are marginal for sustained use under irrigation because of very severe limitations. Rubber, teak and pulp wood trees are raised in these units. The surface samples collected from these units show low to medium availability of nitrogen, low availability of phosphorous and medium to high availability of potassium. Among secondary and micronutrients, sulphur, copper and manganese are adequate, while manganese, zinc, iron and boron are deficient.

Addition of fertilizers as per soil test data coupled with organic manure incorporation is recommended. Cover cropping, bench tracing, construction of contour bunds, digging of silt pits and trenches etc. are some measures suggested to conserve the water and soil topsoil otherwise lost during heavy down pour and to help in percolation of water, which improve the water table of the area.

4.3 LEGEND

Kzy	Kozhukully series
f	Gravelly clay loam texture
F	15 – 25% slope
2	Moderate erosion
St2	0.1 -3% stone cover
R1	2 -10% rocks cover

4.4 KOZHUKULLY SERIES (Kzy)

Kozhukully series are deep soils, well drained, moderately fine textured, brownish to red and acidic. These soils developed from gneissic parent material and occur on moderately steep side slopes of low hills (5 -25%). Presence of weathered gneissic stones within the profile and on the surface is a characteristic feature of these soils. The general elevation is 20 to 200 m above MSL. The climate is humid tropical.

Taxonomic class: Clayey mixed isohyperthermic family of Typic Dystrustepts

Typifying pedon: Kozhukully gravelly clay – cultivated

4.4.1 Range in characteristics

The thickness of solum ranges from 100 to 150 cm. The thickness of Ap horizon is 15–20cm. The colour ranges from dark yellowish brown to dark brown in hues of 7.5YR and 10YR, value 3 and 4 and chroma 2 to 4. Texture varies from gravelly sandy clay loam to gravelly clay Strong acidity is noted. The B horizon is 85 to 105 cm thick. Texture of the B horizon ranges from gravelly clay loam to gravelly clay and colour ranges from yellowish red to dark reddish brown in hue 5YR, value 3 to 5 and chroma 3 to 6. Strong acidity is noticed.

Horizon	**Depth**	**Description**
Ap	0 - 17	Brown (7.5YR 4/2M) gravelly clay: moderate medium subangular blocky; firm, sticky and plastic; several fine pores; teeming medium coarse roots; moderately fast permeability; clear swish boundary; pH scale 5.01
Bw1	17 - 44	Dark reddish brown (5YR 3/3M) gravelly clay; moderate medium sub angular blocky; firm, sticky and plastic; several fine pores; common medium roots; moderately fast permeability; clear wavy boundary; pH scale 5.11
Bw2	44 - 67	Yellowish red (5YR 4/6M) gravelly clay: moderate medium subangular blocky; firm, sticky and plastic; few fine pores; few quartz; gneissic gravels; common medium roots; moderately fast permeability; clear wavy boundary; pH scale 5.26
BC	67 - 107	Yellowish red (5YR 5/6M) gravelly clay: moderate medium subangular blocky; firm, sticky and plastic; moderate permeability; pH scale 5.53
C	107+	Gneissic rock as parent material

4.4.2 Associated Series

The associated series is Koottala series. Soils are well drained to somewhat excessively drained, with moderately rapid permeability.

4.4.3 Use and vegetation

Soils are put under coconut, rubber and trees.

4.4.4 Type location

Sy. No. 147/3 Kozhukully village of Thrissur taluk, Thrissur district.

4.4.5 General Interpretation

Land capability class – IIIe, IVe

Land irrigability class – 3t, 4t

4.4.6 Fertility status

Nitrogen : medium

Phosphorous : medium

Potassium : medium

4.4.7 General recommendations

The depth of the soil ranges from 100 - 150 cm. presence of weathered gneissic stones within the profile and on the surface is a characteristic feature of these soils. The nutrient status is low to medium generally hence recommended dose of fertilizers for each crop will have to be applied in split doses.

4.5 ANALYTICAL RESULT OF KOZHUKULLY SERIES

Depth	Gravel Content		Particle size Distribution							
	% Wt	% Vol	Very Coarse	Coarse	Medium Coarse	Fine	Very fine	Total sand	Silt	Clay
0 – 17	41	50	2.60	13.00	17.70	10.2	0.85	44.35	6.00	49.65
17- 44	33	30	2.00	3.00	7.00	11.0	7.00	30.00	14.00	56.00
44 – 67	43	33	1.50	2.20	7.90	16.3	4.40	32.30	18.00	49.70
67– 107	33	37	1.80	3.00	12.80	8.50	10.60	36.70	7.30	56.00

Table 4.1: Particle size Distribution

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Depth	pH	EC (dS/m)	Exchangeable acidity			CEC (cmol/kg)
	1:2.5	1:2.5	H^+	$A1^{3+}$	Total Acid	
0 – 17	5.01	0.35	0.2	0.8	1.0	7.5
17- 44	5.11	0.29	0.2	0.4	0.6	7.3
44 – 67	5.26	0.30	0.4	0.8	1.2	5.9
67 - 107	5.53	0.63	0.2	0.8	1.0	5.8

Table 4.2: Acidity

Depth (cm)	Organic Carbon %	Exchangeable Bases				ECEC (cmol/kg)	Base Saturation %
		Na^+	K^+	Ca^{2+}	Mg^{2+}		
0 – 17	1.79	0.34	0.81	0.96	0.70	8.3	37.47
17- 44	1.09	0.34	0.28	1.00	0.56	7.7	39.86
44 – 67	0.53	0.47	0.25	0.96	0.50	6.7	36.95
67 - 107	0.57	0.43	0.28	0.88	0.70	6.6	39.48

Table 4.3: Exchangeable Bases

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CHAPTER 5
SOFTWARE USED

5.1 GENERAL

The software used for the study is GEO5. It works on the combined principles of both analytical methods and Finite Element Method. GEO5 adopts a unique system of implementing standards and partial safety factors which are separate from structural input.

5.2 APPLICATIONS OF GEO5

GEO5 is a geotechnical software package that is used to solve various geotechnical problems. Besides the common geotechnical engineering task (slope stability, foundations, retaining walls), it also includes the applications for the analysis of tunnels, building damage due to tunneling or rock slope stability. The powerful programs in GEO5 suite is based on both analytical method as well as finite element method. The analytical method of computation (e.g. slope stability, sheeting design) allow users to design and also to check structures quickly and efficiently. The designed structure is transferred into the FEM where the finite element method is used for the overall general analysis of the structure. It saves designers time as well as compares two independent solutions, thus increasing the design safety. It is a powerful and easy to use package which consists of individual programs having a consistent graphical interface. Each program analyses a different geotechnical task but all modules can communicate with each other and form an integrated package.

GEO5 software package that could be used for:

a. Analysis of stability
b. Design of excavation
c. Design of retaining wall
d. Design of foundation
e. Analysis of soil settlement
f. Model of digital terrain
g. Analysis of advanced finite element (F.E.)

5.3 FEATURES OF GEO5

The main features of GEO5 software are as follows:

i. An Intuitive tool:

GEO5 computer code may be a terribly intuitive and straightforward to use tool. The users principally don't require any intensive tutorial before victimization programs – they can work with confidence with it at intervals of some minutes. However you'll be able to use type of coaching and documentation resources whenever required.

ii. Maintain Standards:

The basic approaches that are enforced within the GEO5 programs are applicable everywhere the planet. Even so, most countries adopt their own standards and conventions. GEO5 offers a novel manner of applying common place that considerably simplifies the work of a designer and at a similar time permits for obliging with all needed approaches.

iii. Availability of Localizations:

Fine has been unceasingly maintaining the localizations of the package to deliver final comfort to GEO5 users. Presently, GEO5 is offered in fourteen language versions.

iv. A Low -cost modular system

The GEO5 programs area unit is cheap and it is possible to shop for the whole suite. Several users begin with one program that's required at the time and additional purchase is made for more modules in step with their budget.

v. Simple and controlled data input

In most applications you'll be able to style and design check a structure among an hour with no special coaching. After you come back to work with a GEO5 program once, you instinctively acumen to input file and use the program. Any modification of input file is straight away displayed on screen, providing you with absolute management of the method.

vi. Comprehensive Outputs

GEO5 programs generate clear text and graphical outputs that may be simply altered in keeping with wants of the user like addition of the company emblem, insertion of images and so on.

Created pictures are continually up to date to now, in keeping with latest information. Outputs is written directly from the program, saved as PDF or exported to external text editor (MS word).

vii. Technical support

Basic technical support is on the market freed from charge to any GEO5 user. Fine offers programme of widened support – fine maintenance. The service is on the market to Associate in nursing anyone for an annual fee, and enclosed square measure hotline phone support, skilled engineering help and unlimited access to computer code upgrades.

5.4 RETAINING WALL DESIGN PROGRAMS

GEO5 contains multiple programs for design of retaining walls and supporting structures – mainly due to conserving simplicity clarity of input. Each program enables analysis of the structure according to geotechnical aspects, but also verification of wall material. Analysis of the stability of natural manmade slopes and embankments is a difficult geotechnical task. The slope stability analysis is carried out to minimize the circumstances of failing slopes and landslides. Through proper measurement of slope stability the slope failing can be determined. The basic program for stability analysis is slope stability. The consideration of interslice force and the complete equilibrium of the sliding mass is the main difference between limit equilibrium analysis methods. The appropriate analysis method results the effectiveness of all slope failure remediation method.

 Cantilever Wall

 MSE Walls

 Abutment

 Earth pressure

 Prefab Wall

 Redi –Rock Wall

Nailed slope

Gravity wall

Gabion

Masonry wall

5.5 SUMMARY

The software GEO5 allows geotechnical engineers to carry out limit equilibrium slope stability analysis of existing natural slopes, unreinforced man – made slopes or slopes with soil reinforcement. It cooperates with all programs for analysis of retaining wall designs. Overall stability analysis of all retaining wall types can be performed directly with the slope stability program. Foundations can be analysed using the spread footing or pile programs

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CHAPTER 6

MATERIALS AND METHODOLOGY

6.1 GENERAL

The planning and constructing a soil retaining structure, it's necessary to aim to anticipate the relevant changes in properties and conditions that will have an effect on them throughout the design, guaranteeing that the stability isn't compromised by any predictable modification. The soil explorations were meted out in two stages, preliminary and elaborated. The preliminary explorations include the geologic study of the site the location, the positioning and site reconnaissance mission. Numerous tests were conducted as a district of the elaborated investigation program.

6.2 SOIL FROM KURANCHERY

Kuranchery soils occur on moderately sloping to moderately steep side slopes of low hills (5–25%). The general elevation is 20 to 200 m above the MSL.

Fig 6.1: Slope of Kuranchery (Collected during site investigation).

The climate is humid tropical. During the site investigations, soil was collected from Kuranchery. The soil samples were collected in polythene gunny bags and then air-dried. Fig 6.1 shows the overview of Kuranchery slope after the landslide of August 2018.

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6.3 CRUSHER DUST

Crusher dust/stone dust is a solid material that is generated from the stone crushing business that is copiously offered in India. It known that crusher dust exhibits high shear strength and is useful as a geotechnical material. It is a fabric that possesses pozzolanic property similarly as has coarser contents in it whereas alternative materials like ash possess solely pozzolanic property and no coarser soil particles. Recycled device mud has several sensible applications round the home and in construction. It are often used as an economical filling and wadding around water tanks blended with natural sands to boost concrete shrinkage and water demand and as a fabric to back-fill trenches with, as construction material in strengthened earth retentive walls, strengthened soil beds and strengthened versatile pavements as a fill material because of its stability, free debilitating nature and sensible resistance characteristics. It also can be used as a concrete mixture so as to produce distinctive textures and as a substitute for concrete once making pathways and driveways.

Natural soils containing plastic fines like silt and clay particles cause immense quantity of deformation beneath serious masses at saturated conditions and their settlement ends up in many failures. Areas like sub-grades, embankments and low lying areas need sensible quality of fabric for his or her effective functioning with regard to strength and voidance. The crusher dust used with such soils has sensible geotechnical applications like:

a. The MDD of soil was found to increase from with the increase in percentage of Crusher Dust.
b. OMC of soil decreases with the increase in percentage of Crusher Dust
c. The specific gravity of soil first increases to with the increase in percentage of stone dust and subsequently it decreases to on further increasing the stone dust content to 50%
d. High CBR and shearing resistance values can enhance their potential use as sub-base material in flexible pavements and also as an embankment material

Fig 6.2: Crusher dust (Collected from quarry)

The production costs of crusher dust are relatively low compared to other building materials. Crusher dusts use less water than other alternatives and have excellent load bearing capabilities and durability. It is fire and heat resistant, non-plastic, and alkaline when exposed to moisture, making it an ideal material to use in construction. It also has applications in horticulture as a natural fertilizer. It contain minerals that are insoluble to water, which makes it an ideal material to stop mineral leaching in soils to reduce water logging and to raise the pH levels of the soil. The crusher dust for the study was collected from Alppara, Peechi Kerala. The sample collected is shown in the fig 6.2.

6.4 METHODOLOGY

The various steps involved in the stability analysis of retaining wall in Kuranchery are as follows:

Step 1: Site reconnaissance

Site reconnaissance mission would facilitate to decide future programme of field investigations, that is, to assess the necessity for preliminary or elaborated investigations. This might additionally facilitate in deciding scope of labor, strategies of exploration to be adopted, field tests to be meted out and administrative arrangements needed for the investigation.

Step 2: Material collection

The soil and crusher dust are the main material used for this project. The soil samples was collected in polythene covers and then air-dried. The crusher dust was collected from a quarry at Poovanchira, Thrissur Kerala.

Step 3: Determination of properties of soil and crusher dust

Soil characteristics were determined using Specific Gravity, Particle size distribution, free swell, Atterberg limits, Light compaction, Unconfined Compression Tests, California Bearing Ratio tests etc. Same tests are conducted on the crusher dust samples, and the properties are determined.

Step 4: Designing of Retaining wall

Technically, while designing, all necessary parameters and requirements are considered and all the possible solutions are generated. The design of retaining wall includes the following steps:

a. Fixation of the base width and the other dimensions of the retaining wall
b. Performing stability checks and computation of maximum and minimum bearing pressure.
c. Design of various parts like stem, toe slab, heel slab, counterfort wall.

Step 5: Stability analysis using GEO5 software

The analysis using GEO5 software consists of three different cases:

- Selection of suitable height of retaining wall.
- With the selected height as constant, selection of suitable backfill mix
- Finally, stability analysis for various water table depths.

CHAPTER 7
EXPERIMENTAL STUDY

7.1 GENERAL

Basic properties of the collected sample were determined using laboratory tests. Laboratory tests includes

- Moisture content determination test
- Specific gravity test
- Atterbergs limits
- Hydrometer test
- Light Compaction test
- Unconfined compression test
- CBR test
- pH

7.2 WATER CONTENT DETERMINATION-OVEN DRY METHOD

Natural water content is set by oven drying methodology as per IS: 2720 (Part II) – 1973. This technique covers the determination of water content of soils expressed as a proportion of the oven-dry weight. The soil specimen taken shall the representative of the soil mass.

Clean the container with lid, dry and weigh (W1). Take the desired amount of the soil specimen within the container fragmented and placed loosely, and weigh with lid (W2). Then keep it in an oven with the lid removed, and maintain the temperature of the oven at 110 ± 5°C. Dry the specimen within the oven for 24 hour. Each time the container is taken out for weighing. Replace the lid on the container and cool the container in a desiccator. Record the ultimate mass (W3) of the container with lid with dried soil sample. The % of water content (W) shall be calculated as follows:

$$W = \frac{W_2 - W_3}{W_3 - W_1} \text{ X } 100$$

Where,

W = water content percentage

W_1 = Mass of container with lid in gram

W_2 = Mass of container with lid with wet soil in gram

W_3 = Mass of container with lid with dry soil in gram

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7.3 SPECIFIC GRAVITY

The specific gravity of solid particles (soil sample) is determined in a laboratory using density bottle fitted with a stopper having a hole. The density bottle fitted with a stopper having a hole. The density bottle (fig 7.1) of 50 millilitre unit} capacity is employed. [IS: 2720 (part2) 1980]. The mass of the bottle, together with that of the stopper is taken. about 5-10 g of oven dry sample is taken within the bottle and weighed. water is then additional to hide the sample. Water is additional till the bottle is [*fr1] full. additional water is added to the bottle to make it full. The stopper is inserted within the bottle and mass is taken.

The bottle is empty, washed and so refilled with distilled water. The bottle should be crammed to the same mark as within the previous case. The mass of the bottle crammed with the water is taken.

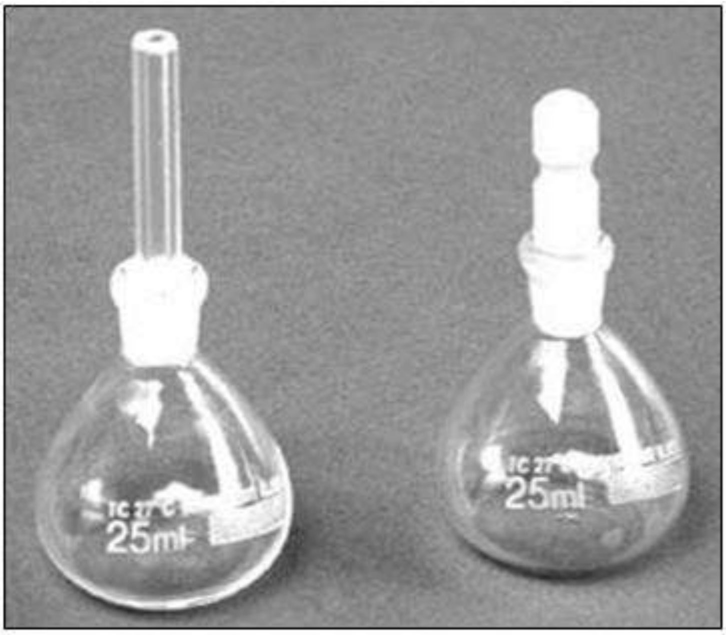

Fig 7.1: Density bottle

The specific gravity of crusher dust particles can be determined in a laboratory using pycnometer bottle by IS-2720-part-3-1980 (shown in fig 7.2).

Fig.7.2 pycnometer bottle

When receiving the sample it's dried in oven at a temperature of 105 to 1150C for a period of 16 to 24 hours. After that, dry the pycnometer and weigh it with its cap and take regarding two hundred g to three hundred g of oven dried sample passing through 4.75mm sieve into the pycnometer and weigh once more. Add water to cover the sample and screw on the cap. Shake the pycnometer well to get rid of entrapped air for regarding ten to twenty minutes. when the air has been removed, fill the pycnometer with water and weigh it. Clean the pycnometer by washing totally and fill the cleansed pycnometer fully with water up to its top with cap screw on. Weigh the pycnometer when drying it on the surface completely. the specific gravity of device dirt is decided mistreatment the relation:

$$G = \frac{W_2 - W_1}{(W_2 - W_1) - (W_3 - W_4)}$$

Where,

W_1 = Weight of dry pycnometer

W_2 = Weight of pycnometer and dry sample

W_3 = Weight of pycnometer, soil sample and water

W_4 = Weight of pycnometer and water

7.4 CONSISTENCY LIMITS OR ATTERBERG LIMITS

The Atterbergs Limits are} a basic measure of the character of a fine-grained soil. looking on the water content of the soil, it's going to seem in four states: solid, semi-solid, plastic and liquid. In every state the consistency and behavior of a soil is completely different and therefore so are its engineering properties.

Thus, the boundary between every state will be defined based on a modification within the soil's behaviour. The water content at that soil changes from one state to a different is understood as consistency limits.These tests are principally used on clayey or silty soils since these are the soils that expand and shrink thanks to moisture content. Clays and silts with chemicals react with the water and therefore change sizes and have variable shear strengths. therefore these tests square measure used wide within the preliminary stages of building any structure to insure that the soil can have the proper quantity of shear strength and not an excessive amount of change in volume as it expands and shrinks.

7.4.1 Liquid Limit

The liquid limit was applied as per IS:2720, part 5-1985. The Liquid Limit (LL) is the water content comparable to the arbitrary limit between liquid and plastic state of consistency of the soil. it's outlined as the minimum water content at which the soil continues to be in a liquid state, however features a little shearing against flowing which may be measured by normal means that. Flow curve is plot with variety of blows on x axis and water content on y axis. The water akin to twenty five blows is that the liquid limit. The original liquid limit test of Atterberg concerned with intermixture a pat of clay in a very little spherical bell-bottom ceramic ware bowl of 10-12cm diameter. A groove was cut across the pat of clay with a spatula, and the bowl was then stricken persistently against the palm of 1 hand. Casagrande later standardized the equipment and also the procedures to form the measurement more repeatable. Soil is placed into the metal cup portion of the device and a groove is created down its center with a homogenous tool of 13.5mm width. The cup is repeatedly dropped 10mm onto a tough rubber base throughout which the groove closes up step by step as a result of the impact. the number of blows for the groove to shut is recorded. The wet content at that it takes 25 drops of the cup to cause the groove to shut over a distance of 13.5mm is defined as the liquid limit. The check is generally run at many moisture contents, and also the wet content which requires twenty five blows to close the groove is interpolated from the check results. The Liquid Limit test is outlined by ASTM standard test technique D 4318. The test methodology additionally permits running the test at one moisture content wherever twenty to thirty blows are needed to close the groove; then a correction factor is applied to get the liquid limit from the moisture content. The Casagrande equipment is shown in fig 7.3.

Fig 7.3: Casagrande Apparatus

7.4.2 Plastic Limit

The plastic limit was applied as per IS:2720, part 5-1985. The plastic limit (PL) is the water content where soil transitions between brittle and plastic behaviour. A thread of soil is at its plastic limit once it begins to crumble once rolled to a diameter of 3mm (fig 7.4). to enhance test result consistency, a 3mm diameter rod is usually used to gauge the thickness of the thread once conducting the test. At this water content, the soil loses its plasticity and passes to the semi-solid state. The shear strength at the plastic limit, is regarding a hundred times that at the liquid limit.

The plasticity index (PI) could be a measure of the plasticity of a soil. The plasticity index is that the size of the range of water contents wherever the soil exhibits plastic properties. The PI is the distinction between the liquid limit and also the plastic limit.

Fig 7.4: Rolling of threads

The PI is given by the equation

$$PI = LL - PL$$

Where,

PI = Plasticity Index

LL = Liquid Limit

PL = Plastic Limit

Soils with a high PI tend to be clay, those with a lower PI tend to be silt, and those with a PI of (non-plastic) tend to own very little or no silt or clay. The importance of the plasticity index is in the incontrovertible fact that the malleability index may be a description of what proportion a soil expands and shrinks. once a structure is made on a soil with a high plasticity index the structures foundation is much more likely to crack and fail. thus it's very vital to understand what the plasticity index and in turn the liquid limit and plastic limit are used to find the plastic index. it is also used for classification of soil.

7.4.3 Shrinkage limit

The shrinkage limit was meted out as per IS:2720, part 6-1972. Shrinkage Limit is the maximum water content at which a discount in water content doesn't cause a considerable reduction in volume of the soil mass. At shrinkage limit, on any reduction in water, air enters into the voids of soils and thus keeps the volume constant. The equipment (fig 7.5) will be used to determine shrinkage limit and to calculate different shrinkage factors like shrinkage magnitude relation, shrinkage index and volumetric shrinkage. it's the water content at that the soil changes from semi-solid state.

Fig 7.5: Shrinkage limit

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7.5 HYDROMETER TEST

Hydrometer will be used for particle size analysis. A special kind of measuring system with a protracted stem (neck) will be used. The stem is marked from prime to bottom, usually in the range of 0.995 to 1.030. hydrometer is first tag (fig 7.6). Suspension ready is added to 1000ml of jar and water is added to that to bring the level to 1000ml mark. The suspension is mixed thoroughly by inserting a bung on the open end of the jar and turning it the other way up and back many times. The jar is then placed on a table and a stop watch is started. The measuring system is inserted in suspension and also the presentation is taken once ½ minute of the commencement of the geological phenomenon. any readings area unit taken once one minute, 2 minutes, and 4 minutes of the commencement of geological phenomenon. The hydrometer is then removed from the jar and rinsed with distilled water and floated during a comparison cylinder containing distilled water with the dispersing agent added to a similar concentration as in the soil specimen. further readings are taken once 8, 15 and 30 minutes and 1, 2, 4, 8 and 24 hours reckoned from the start of the sedimentation. for each of those readings, the hydrometer is inserted regarding 20 seconds before the reading. The hydrometer is taken out after the reading and floated in the comparison cylinder.

Fig 7.6: Hydrometer test Apparatus

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7.6 LIGHT COMPACTION TEST

Compaction is the concentration of soil by reduction of air voids. the light compaction technique was administered by as per IS:2720, part 8-1983. the aim of a laboratory compaction test is to determine, the amount of water to be added for field compaction of soil and resultant density expected. Compactive effort depends on the amount of water the soil contains during soil compaction. The apparatus is shown in fig 7.7.

Fig 7.7: Compaction mould and hammer

The soil is sometimes compacted into the mould to a particular quantity of equal layers, each receiving a number blows from a standard weighted hammer at a specific height. This method is then repeated for varied moisture contents and also the dry densities are determined for each. The graphical relationship of the dry density to moisture content is then plotted to determine the compaction curve. the maximum dry density is finally obtained from the peak point of the compaction curve and its corresponding moisture content, also referred to as the optimal moisture content. Compaction of clay was meted out using standard proctor test with 3 layers on every twenty five blows. The values of optimum moisture content and maximum dry density are obtained in a plot of dry density versus moisture content.

7.7 UNCONFINED COMPRESSION TEST

Unconfined compression test was followed by as per IS: 2720, part 10-1991. This check is conducted on undisturbed or remoulded cohesive soils that are ordinarily saturated. This check may be thought-about as a special case of triaxial compression check once the confining pressure is zero and the axial compressive stress solely is applied to the cylindrical specimen. The stress could also be applied and the deformation and the load readings square measure noted till the specimen fails. the world of cross section of specimen for varied strains is also corrected assuming that the volume of the specimen remains constant and it remains cylindrical. The subsequent equations were used.

$$\text{Axial strain (e)} = \frac{L}{L_o}; \text{Corrected area of cross section (A)} = A_o(1-e)$$

$$\text{Axial stress (qu)} = P/A\ (kg/cm^2)$$

Where,

A_0= Initial area of cross section of the sample(cm^2)

L_0= Initial length of sample (cm)

P= Axial stress (kg)

Graphs are plotted between axial strain (e) Vs axial stress (qu). the maximum price is the unconfined compression strength of clay sample. Soil sample without water hyacinth fibre were tested to find out the optimum moisture content based on compressive stress. Samples for conducting unconfined compression check were prepared at optimum moisture content using moulds. In this study the stress is applied and the deformation and loading readings are noted until the specimen fails. The maximum axial strain is noted. Sample after test is shown in fig 7.8.

Fig 7.8: Specimen after UCC test

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7.8 CALIFORNIA BEARING RATIO TEST (CBR)

California bearing ratio abbreviated as CBR could be a check used to determine the bearing capacity of soil. CBR is that the ratio of force per unit area needed to penetrate a soil mass with standard circular piston at the speed of one.5 mm per minute to that required for corresponding penetration of a regular material. Load which has been obtained from the check on crushed stone (standard materials) is that the standard load. the standard material is claimed to possess CBR 100 percent.

Soil sample passing through 20mm IS sieve is compacted dynamically at maximum dry density using heavy compaction procedure. Remove the collar and trim the soil carefully and weigh the specimen along with the mould and base plate. Surcharge weight up to the calculable weight of the pavement with 2.5kg, but not less than 5kg is to be placed on top of the soil within the mould. Load is applied when putting the mould on the loading machine by using penetration rate of 1.25 mm/min; use a seating load of 4 kilogram that isn't thought-about for the ultimate calculations. Record the load readings for numerous penetration values and also the chart is plotted. CBR values of two.5mm and 5mm penetration is calculated from their individual load values using CBR equation. The experimental set up of CBR is shown in fig 7.9.

Fig 7.9: California Bearing Ratio Tester

7.9 DIRECT SHEAR

Direct shear test or Box shear check is used to see the shear strength of the crusher dust sample. The direct shear check will be determined in laboratory by IS: 2720(Part 13)-1986. The check is dole out on a sample confined in an exceedingly metal box of sq. cross-section that is split horizontally at mid-height. alittle clearance is maintained between the 2 halves of the box. The sample is sheared on a preset plane by

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moving the top half of the box relative to the bottom half. The box is typically square in plan of size 60 millimetre x 60 millimetre (fig 7.10)

The Shear strains are calculated by dividing horizontal displacements with the specimen length, and shear stresses are obtained by dividing horizontal shear forces with the shear area. The shear stress versus horizontal displacement is plotted. the maximum value of shear stress is read if failure has occurred, otherwise scan the shear stress at 200th shear strain. the most shear stress versus the corresponding normal stress is premeditated for every check, the cohesion and also the angle of cut resistance of the crusher dust is determined from the graph. The test apparatus is shown in fig 7.11.

Fig 7.10: Shear box, Porous Stones, Grid Plates, Loading pad

Fig 7.11: Direct Shear Test Apparatus

CHAPTER 8

DESIGN PROCEDURES OF RETAINING WALL

8.1 GENERAL

This chapter covers the design procedures adopted for cantilever and counterfort retaining walls

8.2 DESIGN OF CANTILEVER RETAINING WALL

In the design, earth pressure co-efficients are calculated based on Rankine's theory and coulomb's theory. It is assumed that a triangular pressure distribution is developed on the back of the wall due to backfill earth. All earth pressure forces are considered to act on a vertical plane, which pass through the rear end of the base slab. The wall inclination between wall and backfill are assumed zero for the plane while calculating the earth pressure co-efficients.

The design parameters are:

- Height of the earth to be retained, h (m)- design is economical for less than 10 m height of the earth. It is the difference between the level of earth on either side of the wall.
- Surcharge pressure on backfill, W_s (kN/m^2) – Gravity loads acting on backfill due to the construction of buildings or the movement of vehicles near the top of the retaining wall. Since these loads are not found in the considered area, here it is taken as zero.
- Slope of backfill, β (^{0}) with the horizontal.- is the angle made by backfill soil with the horizontal.
- Unit weight of the soil (γ) –From the data collected, it is found to be 145 kN/m^2
- Soil wall interface friction –the wall friction has been neglected in the analysis, but it is required for calculation of sliding factor of safety for the wall. The typical value of wall friction is $\frac{2}{3}\emptyset$ to $\frac{3}{4}\emptyset$ between soil and concrete wall.
- Concrete density varies between 23-25kN/m^2
- Depth of foundation-is the depth from surface of soil in front of the wall to the bottom of the base
- Bearing capacity of soil corresponding to the given depth of foundation
- Base thickness in mm –the thickness of base slab is taken as 8-12% of total height (H) of wall.

Length of heel slab in m –is the length from face of the wall to the rear end of the base slab, which includes the thickness of the stem. For preliminary design considerations, the length of heel slab is taken as

$H\sqrt{\frac{K_a}{3}}$ (According to Pillai and Menon, Reinforced Concrete Design, 2011). The length of toe slab is given in m.

8.3 DESIGN OF COUNTERFORT RETAINING WALL

When H exceeds about 8m, counterfort retaining walls are more economical than cantilever retaining walls. The stem and heel thickness is more in this case. More bending and more steel are required. Cantilever-T type walls are uneconomical and hence counterforts-Trapezoidal section is provided. 1.5m -3m c/c is provided between the counterforts.

8.3.1 Parts of Counterfort Retaining wall:

The parts are same as that of cantilever retaining wall plus counterfort.

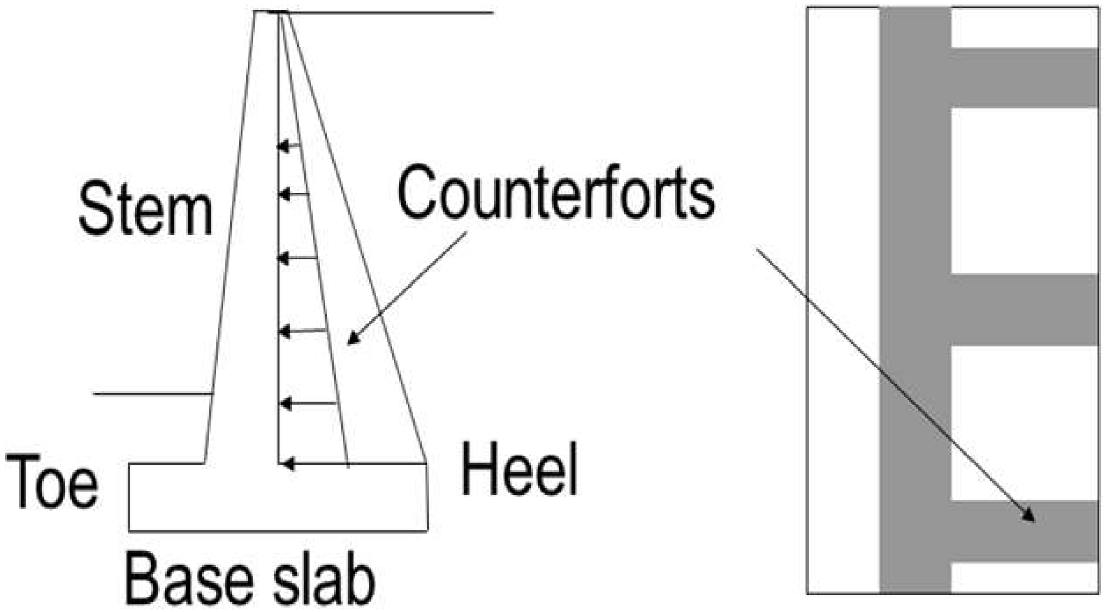

Fig 8.1: Cross section and plan

8.3.2 Design of Stem

The stem acts as a continuous slab (fig 8.2). Soil pressure acts as the load on the slab. The earth pressure varies linearly over the height. The slab deflects away from the earth face between the counterforts. The bending moment in the stem, is maximum at the base and reduces towards top. But the thickness of the wall is kept constant and only the area of steel is reduced.

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Maximum Bending moments for stem are given as follows (fig 8.3):

- Maximum positive B.M= $pl^2/16$ (occurring mid-way between counterforts)
- Maximum -ve B.M= $pl^2/12$ (occurring at inner face of counterforts)

Where 'l' is the clear distance between the counterforts and 'p' is the intensity of soil pressure

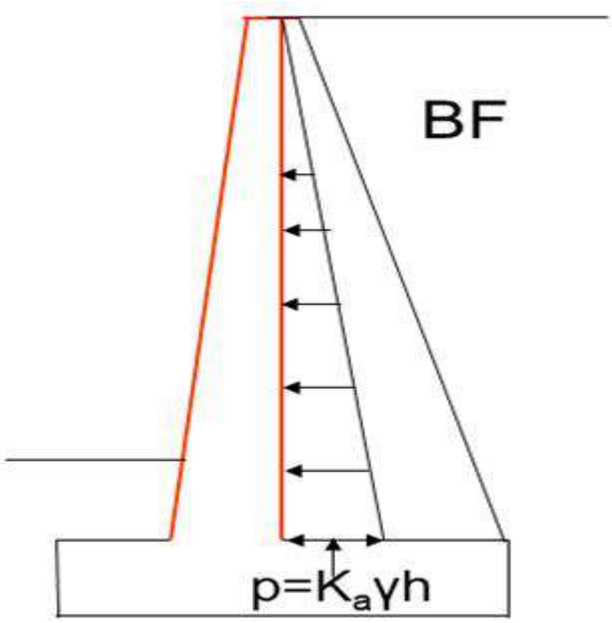

Fig 8.2 Pressure on stem

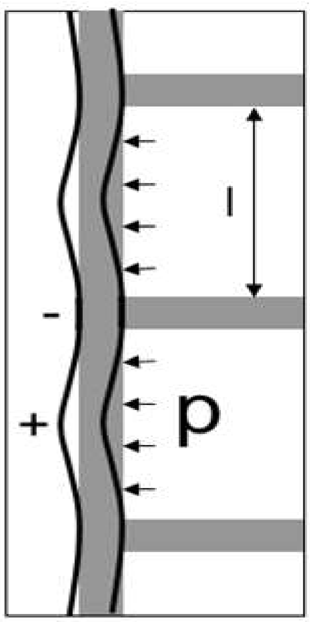

Fig 8.3 Bending moment on stem

8.3.3 Design of Toe slab

- The base width = b = 0.6 H to 0.7 H
- The projection=1/3 to 1/4 of base width.

The toe slab is subjected to an upward soil reaction and is designed as a cantilever slab fixed at the front face of the stem. Reinforcement is provided on earth face along the length of the toe slab. In case the toe slab projection is large i.e. > b/3, front counterforts are provided above the toe slab and the slab is designed as a continuous horizontal slab spanning between the front counterforts.

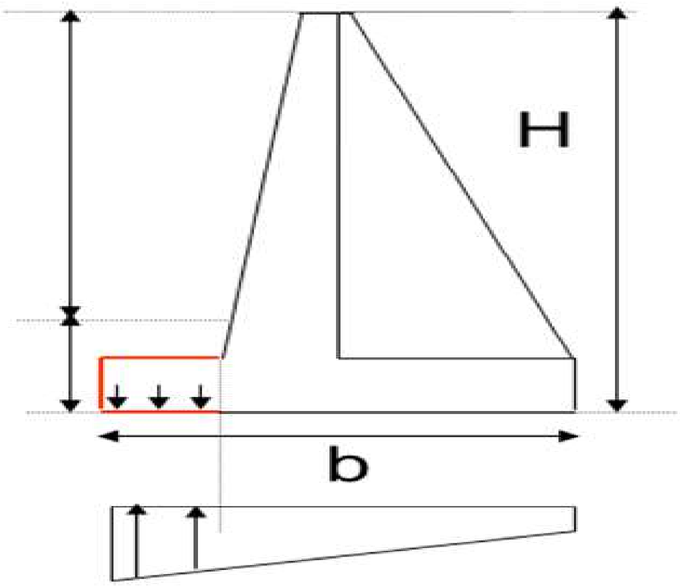

Fig 8.4: Toe slab reaction

8.3.4 Design of Heel slab

The heel slab is designed as a continuous slab spanning over the counterforts and is subjected to downward forces due to weight of soil plus self-weight of slab and an upward force due to soil reaction.

- Maximum positive B.M= $pl^2/16$ (mid-way between counterforts)
- Maximum negative B.M= $pl^2/12$ (occurring at counterforts)

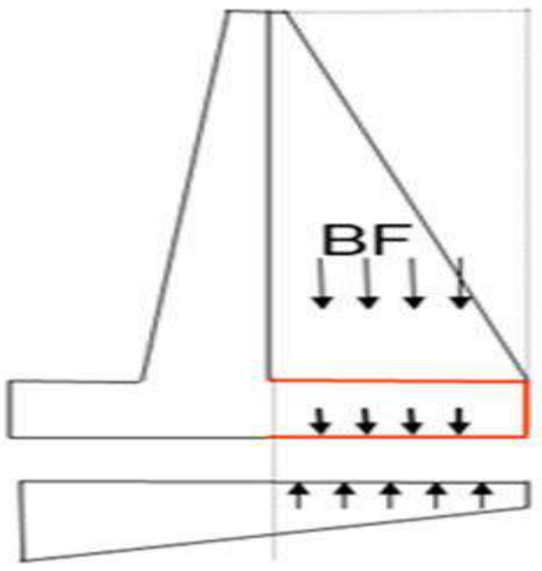

Fig 8.5: Heel slab reaction

8.3.5 Design of Counterforts

The counterforts are subjected to outward reaction from the stem. This produces tension along the outer sloping face of the counterforts. The inner face supporting the stem is in compression. Thus counterforts are designed as a T-beam of varying depth. The main steel provided along the sloping face shall be anchored properly at both ends. The depth of the counterfort is measured perpendicular to the sloping side.

8.3.6 Behaviour of counterfort retaining wall

Important points involved in the behaviour of counterfort retaining wall are:

- Loads on Wall
- Deflected shape
- Nature of BMs
- Position of steel
- Counterfort details

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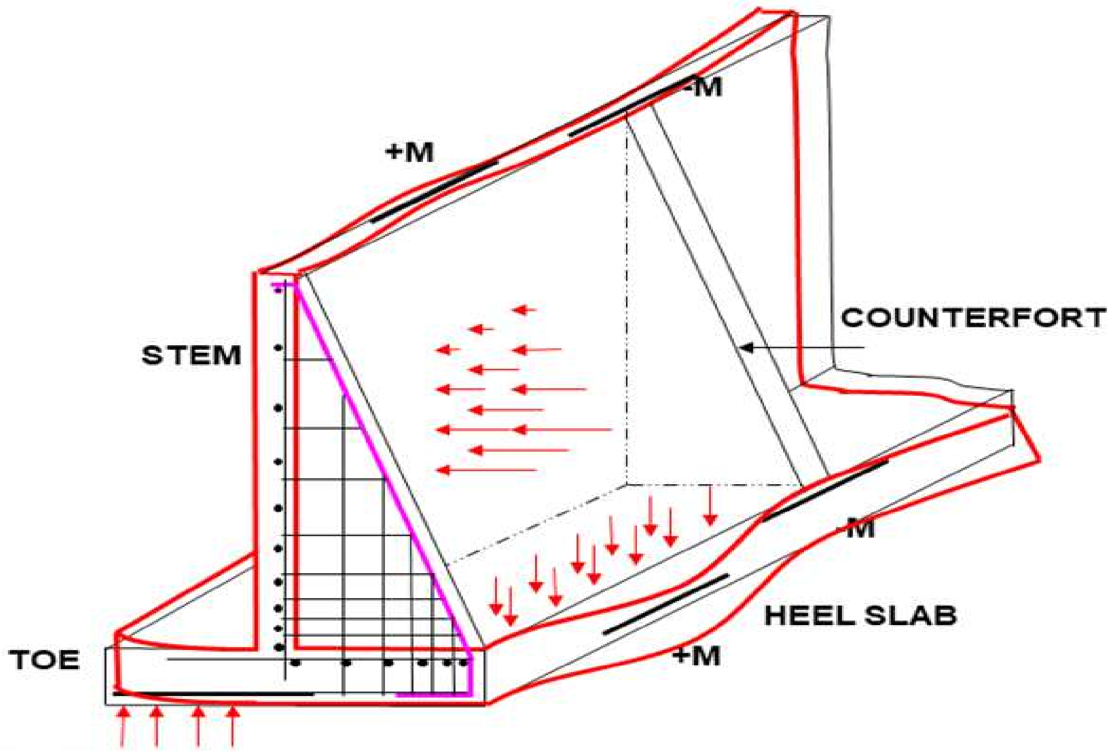

Fig 8.6 Behaviour of counterfort retaining wall

CHAPTER 9

RESULTS AND DISCUSSIONS

9.1 GENERAL

The chapter covers the results of the testing programs and the design of the retaining wall and soil nailing. The results that presented include soil properties and the various testing results of the crusher dust.

9.2 PROPERTIES OF SOIL SAMPLE

Soil characteristics were determined using Specific Gravity test, Hydrometer test, Free swell test, Atterberg's limits, Light compaction test, Unconfined Compression Tests, California Bearing Ratio test etc. The test results are shown in Table 9.1.

SL NO	PROPERTIES	VALUES
1	Natural water content (%)	13.51
2	Particle size distribution	
	Percentage of sand (%)	36.70
	Percentage of clay (%)	56.00
	Percentage of silt (%)	07.30
3	Liquid limit (%)	46.00
4	Plastic limit (%)	20.00
5.	Specific gravity	2.50
6	Maximum Dry Density (g/cc)	1.79
7.	Optimum Moisture content (%)	13.80
8	Unconfined Compression Strength (kN/m^2)	2.61
9.	California Bearing Ratio	13.00
10	Cohesion	0.13
11.	Ph	5.53
12	IS Classification	CI

Table 9.1: Properties of soil

9.2.1 Flow Curve

Flow Curve is the graph plot between number of blows on X-axis and water content on Y-axis. The liquid limit is obtained as water content corresponding to 25 Number of blows. From the flow curve, the liquid limit was obtained as 46%. Figure 9.1 shows the flow curve.

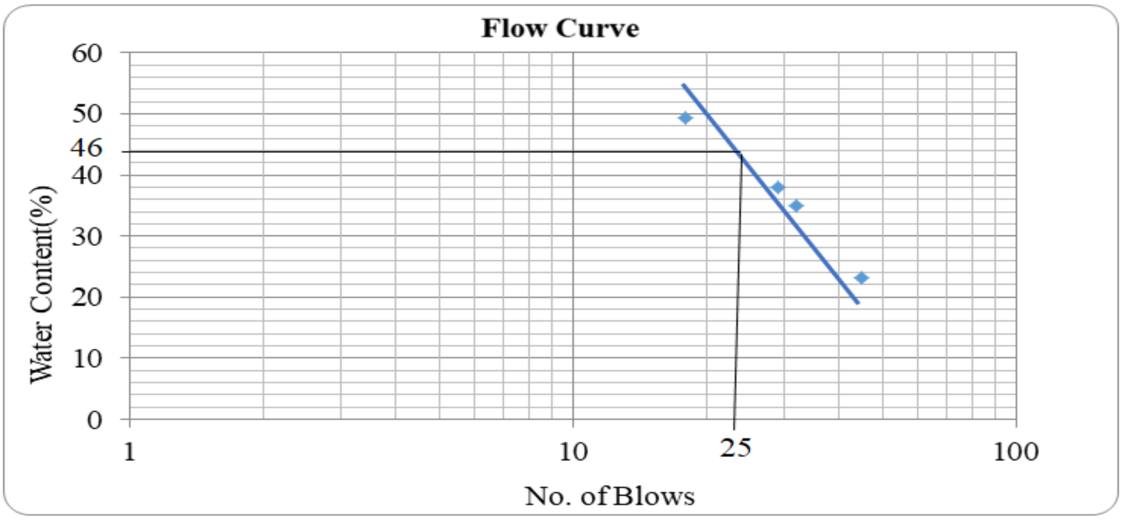

Fig. 9.1 Flow curve

9.2.2 Compaction Curve

Compaction curve is the graph plot between water content on X-axis and dry density on Y-axis. Dry density increases with increase in water content up to optimum moisture content and then decreases. The water content corresponds to maximum dry density is taken as optimum moisture content. Here Maximum dry density is obtained as 1.79 g/cc and Optimum moisture content as 13.8%. Figure 9.2 shows the compaction curve.

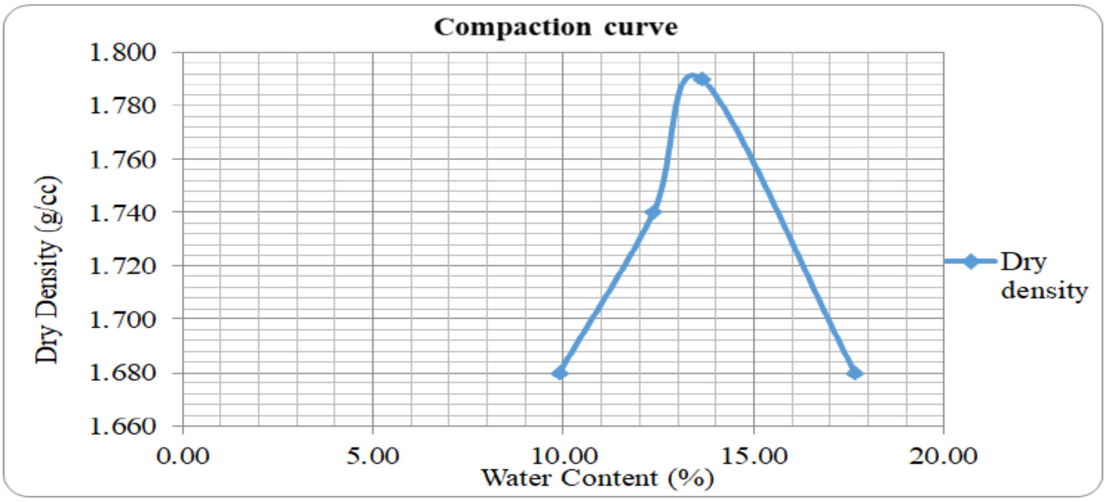

Fig. 9.2 Compaction Curve

9.2.3 Unconfined Compression Strength Curve

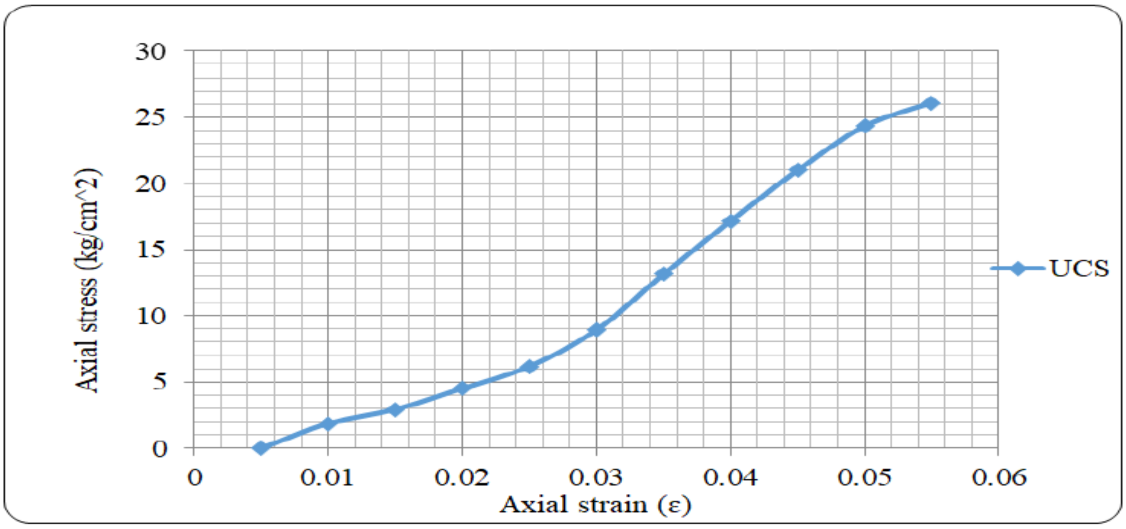

Fig. 9.3 UCS Curve

Unconfined Compression Strength (UCS) curve is the graph plot between axial strain on X-axis and axial stress on Y-axis. The maximum stress from the curve gives the value of unconfined compressive strength. Here compressive strength was obtained as 2.61 kN/m^2. Figure 9.3 shows the UCS curve.

9.2.4 Load Penetration Curve

The Load penetration curve is the graph plot between penetration on X-axis and load on Y-axis is given below. California Bearing Ratio corresponding to 2.5 mm penetration is 10.9 % and that corresponding to 5.0 mm penetration is 13.0 %. Fig 9.4 shows the load penetration curve

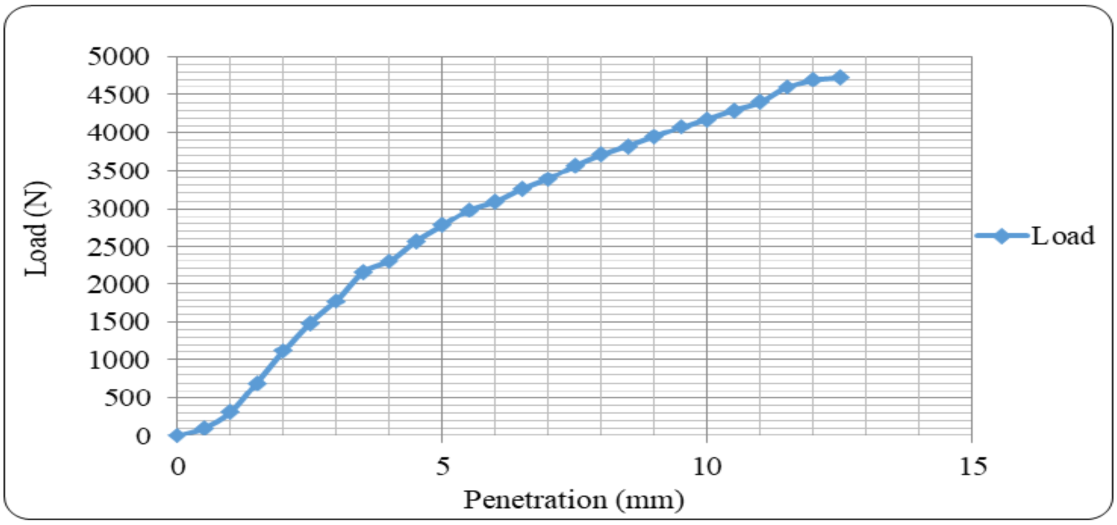

Fig. 9.4 Load Penetration Curve

9.3 PROPERTIES OF CRUSHER DUST

The crusher dust characteristics were determined using Specific Gravity test, Light compaction test, Unconfined Compression Tests, California Bearing Ratio test etc. The test results are shown in Table 9.2.

SL. NO.	PROPERTIES	VALUES
1	Particle Size distribution	
	Percentage of sand (%)	90.7
	Percentage of gravel (%)	0.20
	Percentage of fines (%)	9.10
2	Liquid limit (%)	N.P.
3	Plastic limit (%)	N.P.
4	IS classification	SW
5	Specific Gravity	2.533
6	Maximum Dry Density (g/cc)	1.9
7	Optimum Moisture Content (%)	13
8	Angle of internal friction (deg)	44.71
9	Co-efficient of Uniformity	7.86
10	Co-efficient of curvature	1.003

Table 9.2: Properties of crusher dust

9.3.1 Particle Size Distribution Curve

Particle size distribution curve is the graph plot between sieve size on X-axis and percentage finer on Y-axis. The sample was found to contain 90.70 % of sand particles, 0.20 % of gravel and 9.10 % fines. The curve is shown in the figure 9.5.

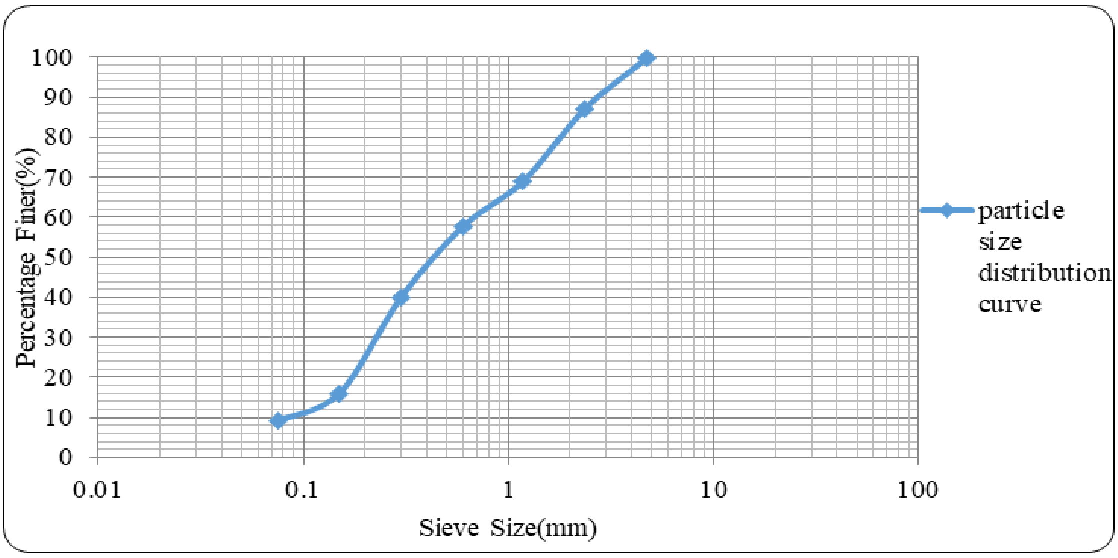

Fig 9.5: Particle size distribution curve

9.3.2 Compaction Curve

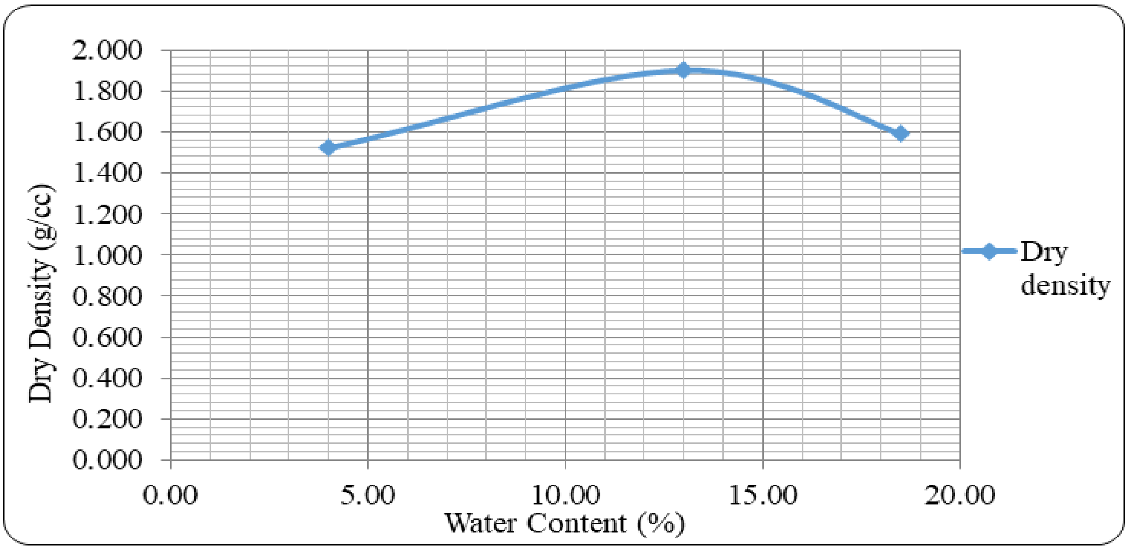

Fig.9.6 Compaction curve

The graph is plot between water content on X-axis and dry density on Y-axis. Dry density increases with increase in water content up to optimum moisture content and then decreases. The water content corresponds to maximum dry density is taken as optimum moisture content.

Here Maximum dry density is obtained as 1.9 g/cc and Optimum moisture content as 13 %. Figure 9.6 shows the compaction curve.

9.3.3 Direct Shear

The graph obtained is plot between shear strength on X-axis and dry density on Y-axis. The angle of internal friction obtained is 44.71^0. Graph is shown in fig 9.7

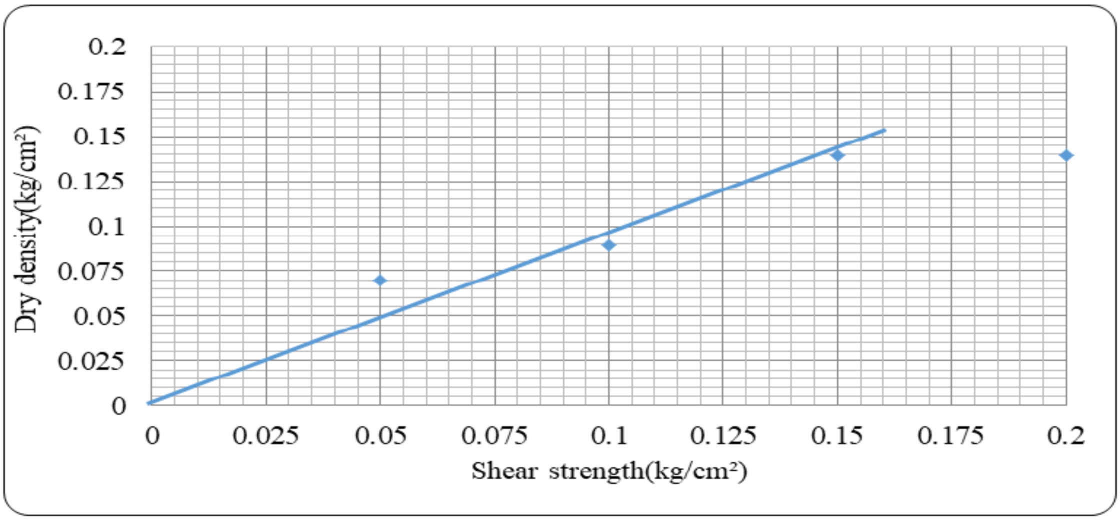

Fig 9.7 Direct shear

The various parameters required for the design of the retaining wall has been obtained from the soil test results as well as the data collected from the soil survey department. The thesis presents the stability analysis of the retaining wall constructed at Kuranchery covering about 20m height with a single layer of walls with a height of about 10m. Cantilever retaining wall has been designed as a first trial.

9.4 DESIGN OF CANTILEVER RETAINING WAL

Cantilever design was done for various embankment heights are shown in table 9.3:

Design Parameters:						
1	Height of earth to be retained, h (m)	4.50	5.00	5.50	6.00	6.50
2	Surcharge pressure on the backfill, (kN/m^2)	0	0	0	0	0
3	Angle of internal friction of soil, Ø (°)	20	20	20	20	20
4	Slope of backfill, β(°) with the horizontal	0	0	0	0	0
5	Unit weight of the soil, γ (kN/m^2)	16	16	16	16	16
6	Soil-wall interface friction, δ(°)	20	20	20	20	20
7	Concrete density (kN/m^2)	25	25	25	25	25
Wall Foundation Design						
1	Depth of foundation	2.3	2.3	2.3	2.3	2.3

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	$\frac{q_o}{\gamma}\left(\frac{1-\sin\emptyset}{1+\sin\emptyset}\right)^2$					
2	Overall height of the wall, H (m)	6.8	7.3	7.8	8.3	8.8
3	Bearing capacity of soil, q_o (kN/m^2)	145	145	145	145	145
Wall dimensions						
1	Base thickness (mm) $\left(\frac{H}{12}\right)$	560	610	650	690	735
2	Length of heel slab (m) $\left(H\sqrt{\frac{K_a}{3}}\right)$	3.00	3.50	3.75	4.00	4.00
3	Length of the toe slab (m)	1.50	1.75	1.88	2.00	2.00
4	Total length of base slab (m)	4.50	5.25	5.63	6.00	6.00
5	Thickness of the stem at base (mm)	560	610	650	690	735
6	Thickness of stem at top (mm)	300	300	300	300	400
Earth pressure analysis						
1	Stability against overturning ($FS_o > 1.4$)	2.37	2.79	2.81	2.81	2.51
2	Eccentricity of vertical reaction from C.G. of the footing, e (m)	0.51	0.41	0.43	0.46	0.6
3	Maximum pressure on soil at the base, P_{max} (should be > q_o)	144.35	134.8	142.73	151.9	161
4	Check for sliding stability, FS_s (Should be < 1.4. If not safe, provide shear key)	0.7	0.76	0.76	0.76	0.89
Adjustments for safe FS_o						
1	Length of heel slab (m)	-	-	-	4.20	5.00
2	Length of toe slab (m)	-	-	-	2.10	2.50
3	Total length of base slab (m)	-	-	-	6.30	7.50
4	Safe P_{max}	-	-	-	139.3	123
Adjustments for safe FS_s (Provision of shear key)						
1	Width of shear key (mm) (Should be > thickness of base slab)	565	650	700	700	770
2	Distance from toe (m) (Should be > length of toe slab)	3.50	3.75	4.00	4.30	4.00
3	Depth of shear key (mm)	300	300	350	350	400

Table 9.3: Cantilever design data

As the height increases, cantilever wall is observed to be unstable. Thus for a design requiring height about 10 m, counterfort retaining wall design is tried.

9.5 DESIGN OF COUNTER FORT RETAINING WALL

The detailed design for counterfort retaining wall having an embankment height of 8.5 m is given below. Safe bearing capacity of the soil (q_o) is 145 kN/m^2. The unit weight of the soil is found to be 18 kN/m^2. The angle of internal friction is 25^0. M25 grade concrete and Fe415 steel is adopted for design. The design is as per the guidelines of IS 456: 2000

Step 1: Depth of foundation (Y_{min})

$Y_{min} = \frac{q_o}{\gamma}\left(\frac{1-\sin Ø}{1+\sin Ø}\right)^2 = \frac{145}{18}\left(\frac{1-\sin 25^0}{1+\sin 25^0}\right)^2 = 1.30$ m

Co-efficient of active pressure, $K_a = \frac{1-\sin Ø}{1+\sin Ø} = = \frac{1-\sin 25^0}{1+\sin 25^0} = 0.40$

Co-efficient of passive pressure, $K_p = \frac{1}{K_a} = \frac{1}{0.4} = 2.50$

Step 2: Dimensions of various parts

Overall height, H = 8.50 + 1.30 = 9.80 m

Normally, base width is taken in the range 0.6H to 0.7H. Adopt base, b as 6.4 m. Maximum toe width for counter fort retaining wall is taken in the range $\frac{b}{3}$ to $\frac{b}{4}$. Adopt toe width as 2 m. Thickness of the base slab is in the range of $\frac{H}{15}$ to $\frac{H}{20}$. t = $\frac{9800}{15}$ to $\frac{9800}{20}$. Adopt thickness of the base slab as 570 mm. Assume thickness of the vertical wall as 350 mm.

Spacing of the counter forts, l $= 3.5\left(\frac{H}{\gamma}\right)^{0.25} = 3.5\left(\frac{9.80}{18}\right)^{0.25} = 3$ m (c/c)

Pressure at the bottom of the stem, P = $K_a\gamma h = 0.40 \times 18 \times (9.80 - 0.57) = 66.96$ KN/m^2

Preliminary dimensions of the wall are shown in figure:

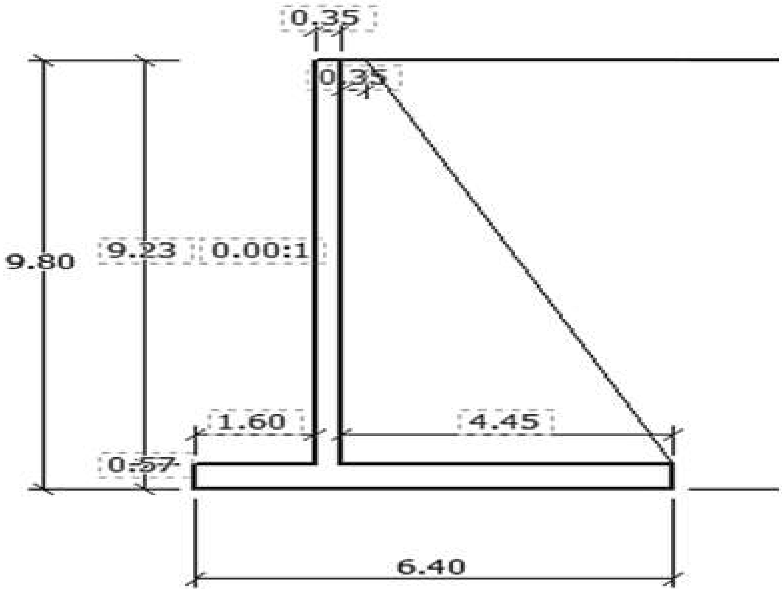

Fig 9.8: Dimensions of counterfort retaining wall

Step 3: Check for stability

Description of loads	Vertical load (KN)	Lever arm from A (m)	Moment (KNm)
Weight of stem, W_1	$(0.35 \times 9.23 \times 1) \times 25$ $= 80.76$	$1.6 + \frac{0.35}{2}$ $= 1.75$	143.349
Weight of base slab, W_2	$(0.57 \times 6.4 \times 1) \times 25$ $= 91.20$	$\frac{6.4}{2}$ $= 3.20$	291.84
Weight of earth over heel slab, W_3	$(4.45 \times 9.23 \times 1) \times 18$ $= 739.323$	$1.6 + 0.35 + \frac{4.45}{2}$ $= 4.175$	3086.67
	$\Sigma W = 911.283$		$\Sigma M_r = 3521.859$

Table 9.4: Stability check (a)

Horizontal earth pressure on full height of the wall,

$P_h = \gamma h^2 K_a/2 = 18 \times 9.23^2 \times 0.4/2 = 306.69$ kN

Overturning moment, $M_o = P_h \times \frac{h}{3} = 306.69 \times \frac{9.8}{3} = 1001.85$ kNm

Check for Overturning:

$(F.S)_o = \frac{M_r}{M_o} = \frac{3521.859}{1001.85} = 3.51 > 1.55$

Hence safe against Overturning

Check for Sliding:

Resisting force = $\Sigma\mu W = 0.58 \times 911.283 = 528.54$ KN (→)

Neglecting passive pressure of soil at toe,

$(F.S)_s = \frac{\Sigma\mu W}{Ph} = \frac{528.54}{306.69} = 1.72 > 1.55$

Hence, structure is safe against Sliding.

$\Sigma W\bar{x}$ is the net moment at A (toe) = M_r - M_o

i.e., $\bar{x} = \frac{Mr - Mo}{\Sigma W} = \frac{3521.859 - 1001.85}{911.283} = 2.76$ m

Eccentricity, e = = $\frac{b}{2} - \bar{x} = \frac{6.4}{2} - 2.76 = 0.44$ m < $\frac{b}{6}$ (~1.06)

Maximum Pressure at A (toe),

$P_A = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right) = \frac{911.283}{6.4}\left(1 + \frac{6 \times 0.44}{6.4}\right) = 201.123\ kN/m^2 > 145\ kN/m^2$

Hence it is unsafe. In order to make it safe, increase length of toe slab to 2.4 m, so that total width b = 7.2 m. Revised computations are:

Description of loads	Vertical load (KN)	Lever arm from A (m)	Moment (KNm)
Weight of stem, W_1	$(0.35 \times 9.23 \times 1) \times 25$ = 80.76	$2.4 + \frac{0.35}{2}$ = 2.575	207.957
Weight of base slab, W_2	$(0.57 \times 7.2 \times 1) \times 25$ = 102.60	$\frac{7.2}{2}$ = 3.40	369.36
Weight of earth over heel slab, W_3	$(4.45 \times 9.23 \times 1) \times 18$ = 739.323	$2.4 + 0.35 + \frac{4.45}{2}$ = 4.975	3678.13
	$\Sigma W = 922.683$		$\Sigma M_r = 4255.447$

Table 9.5: Stability check (b)

Check for Overturning:

$(F.S)_o = \frac{M_r}{M_o} = \frac{4255.447}{1001.85} = 4.24 > 1.55$

Hence safe against Overturning

Check for Sliding:

Resisting force = $\Sigma\mu W = 0.58 \times 922.683 = 535.16$ KN (→)

Neglecting passive pressure of soil at toe,

$(F.S)_s = \frac{\Sigma\mu W}{Ph} = \frac{535.16}{306.69} = 1.74 > 1.55$

Hence, structure is safe against Sliding.

$\Sigma W\bar{x}$ is the net moment at A (toe) = M_r - M_o

i.e., $\bar{x} = \frac{Mr - Mo}{\Sigma W} = \frac{4255.447 - 1001.85}{922.683}$ =3.5 m

Eccentricity, e = = $\frac{b}{2} - \bar{x} = \frac{7.2}{2} - 3.15 = 0.1$ m $< \frac{b}{6}$ (~1.1)

Maximum Pressure at A (toe),

$P_A = \frac{\Sigma W}{b}\left(1 + \frac{6e}{b}\right) = \frac{922.683}{7.2}\left(1 + \frac{6 \times 0.10}{7.2}\right) = 137.97\ kN/m^2 > 145\ kN/m^2$

Hence it is safe.

The minimum pressure at heel,

$P_D = \frac{\Sigma W}{b}\left(1 - \frac{6e}{b}\right) = \frac{922.683}{7.2}\left(1 - \frac{6 \times 0.10}{7.2}\right) = 117.47\ kN/m^2$

The distribution of stress beneath the base is shown in figure.

The intensity of pressure at junction of stem with toe, i.e. under B,

PB = 117.47+ (137.97-117.47) × (4.8)/7.2= 131.14 kN/m2

The intensity of pressure at junction of stem with heel, i.e. under C,

PC = 117.47+ (137.97-117.47) × (4.45)/7.2 = 130.14 kN/m2

Step 4: design of Toe slab

Since the projection of toe is small, it's designed as a cantilever mounted at the stem

Neglecting the load of the soil on top of the toe slab, the forces performing on the toe block are:

a. Downward force due to toe slab

b. Upward soil pressure on length AB

Total downward pressure, P = self-weight of toe slab

i.e. P = thickness of the base slab × density of concrete = 0.57 × 25 = 14.25 KN/m^2

The intensity of pressure at B = 131.14 kN/m^2

Bending Moment at critical section,

$M_u = 1.5\times (131.14\times \frac{2.4^2}{2} + (137.97 - 131.14) \times \frac{2}{3} \times 2.4 \times 2.4 - (25\times 2.4\times 0.5\times \frac{2.4}{2}))$

M_u = 544.31 KNm

$\frac{Mu}{bd^2} = \frac{544.31 \times 10^6}{1000 \times 520^2} = 2.01$

From SP 16, table 3, by interpolation,

Percentage of steel, $P_t = 0.618 + \left(\frac{0.635-0.618}{2.05-2.00} \times (2.01 - 2.00)\right)$ =0.0034 %

0.15% of steel is required. Thus A_{st}= 0.15% of (1000×520) = 780 mm^2

Using 16 mm bars, spacing = $\frac{1000 \times \frac{\pi}{4} \times 16^2}{780}$ = 257.77 mm

However, the spacing is limited to 110 mm c/c from shear considerations. Thus provide 16 mm bars at 110 c/c, area provided = 1827 mm^2, P_t= 0.47 %

The bars shall be extended beyond the front face of the wall for a distance equal to the development length of 750 mm (47 × 16)

Distribution steel= 0.12 ×1000 × 570/100 = 684 mm^2.

Provide 12 mm bars at 165 mm c/c

Check for shear:

Since the soil induces compression in the walls, the critical section for shear is taken at a distance d from the face of the stem. Intensity of pressure at distance d (= 390mm) from the face of the toe

$P_E = 117.47 + (137.97 - 117.47) \times \frac{4.8+0.39}{7.2}$ = 132.25 kN/m^2

Step 5: Design of Heel Slab

The heel slab is designed as a continuous slab supported on counterforts. The downward force will be maximum at the edge if the slab where the intensity of soil pressure is minimum.

Consider 1 m strip near the outer edge D. the forces acting near the edge are:

a) Downward weight of soil of height 9.23 m = 18 × 9.23 × 1 =166.14kN/m

b) Downward weight of heel slab = 25 × 0.57 × 1= 14.25kN/m

c) Upward soil pressure of intensity 117.47kN/m^2 = 117.47 × 1 =117.47kN/m

Net downward force at D= p= 166.14 + 14.25 – 117.47 = 62.92 kN/m

Net downward force at C = 166.14 + 14.25 – 130.14 = 50.25 kN/m

Width of the counterfort is 400 mm. Clear spacing between counterforts is 2.6 m. Maximum negative ultimate moment in heel slab at counterfort

M_u =1.5 × $\frac{wl^2}{12}$ == $1.5 \times \frac{62.92 \times 2.6^2}{12}$ = 53.17 KNm

$\frac{M_u}{bd^2} = \frac{53.17 \times 10^6}{1000 \times 390^2} = 0.35$

Provide 0.12% steel. A_{st}= (0.12/100) × 1000 × 570 = 684 mm^2. Provide 12 mm bars at 165 mm c/c

Step 6: Design of Stem (Vertical Slab)

The stem acts as a continuous slab spanning between the counterforts. It is subjected to linearly varying earth pressure having maximum intensity at bottom.

Consider 1 m wide strip at bottom of stem at C.

The intensity of earth pressure, $P_h = K_a\gamma h = 0.4 \times 18 \times 9.23 = 66.456$kN/m^2

Area of steel on earth side near counterforts: Maximum negative ultimate moment,

$M_u = 1.5 \times \frac{wl^2}{12} == 1.5 \times \frac{66.456 \times 2.6^2}{12} = 56.16$ KNm

Required d= $\sqrt{\frac{56.16 \times 10^6}{1000 \times 0.57}}$ = 313.9 mm. Provide total depth of 320 mm. Assuming effective cover as 50 mm, d= 320-50 = 270 mm

Provide 12 mm bars at 110 mm c/c. As the earth pressure decreases towards the top, the spacing of the bars is increased with decrease in height.

Provide 0.12% cross section area as distribution steel. Provide 8 mm bars at 300 mm on each face in vertical direction

Step 7: Design of Counterforts

Width of the counterfort = 400 mm. The counterforts are provided at 3 m c/c.

They are subjected to earth pressure and downward reaction from the heel slab.

At any section at any depth, h, below the top E the total horizontal earth pressure acting on the counterfort = $\frac{1}{2}\gamma h^2 k \times$ c/c distance b/w counterforts $=18\times h^2\times3\times0.5\times0.4=10.8h^2$

Bending moment at any depth h = $10.8h^2\times h/3 = 3.6\ h^3$

Bending moment at the base at C = $3.6 \times 9.23^3 = 2830.79$ kNm

Ultimate moment, $M_u = 1.5\times 2830.79 = 4246.185$ kNm

Net downward pressure on heel slab at D = wt. due to earth pressure + wt. of heel slab

= 18 × 9.23 + 25 ×0.57 – 117.47 = 62.92kN/m^2

Net downward pressure on heel slab at C

= 18 × 9.23 + 25 ×0.57 – 130.14 = 50.25 kN/m^2

Total downward force at D = 62.92 × c/c distance = 62.92 × 3 = 188.76 kNm

Total downward force at C = 50.25 × c/c distance = 50.25 × 3 = 150.75 kNm

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As mentioned earlier, counterfort acts as T- beam. It can be seen that the depth available is much more than that required from bending moment considerations

Even assuming rectangular section,

The available depth is obtained as: d= $\sqrt{\frac{4246.185 \times 10^6}{2.76 \times 400}}$ = 1961.24 mm.

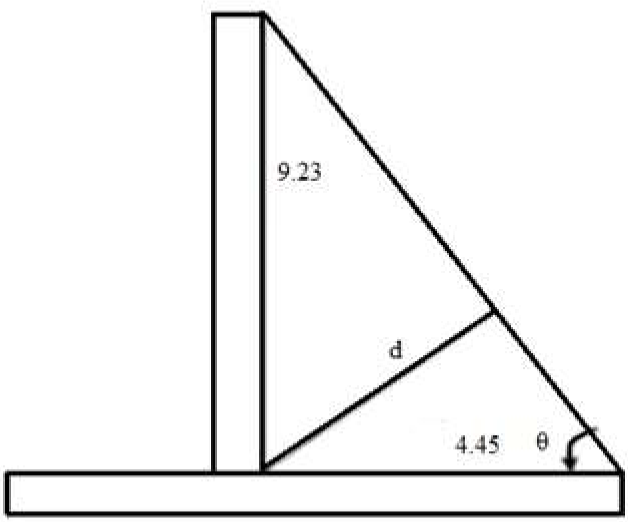

Fig 9.9: Available depth

The effective depth is taken at right angles to the reinforcement

tan θ = 9.23/4.45 = 2.07, θ = 64.26^0

Thus d= 4450sin θ – effective cover = 3958.45mm >>1961.24 mm

$\frac{M_u}{bd^2} = \frac{4246.185 \times 10^6}{400 \times 3958.45^2}$ = 0.677

Provide 1.2% steel. A_{st}= 1900 mm^2

A_{st-min} = 0.85bd/f_y = 0.85 × 400 × 3958.45 / 415 = 3243.07 mm^2

Provide 5 – 22mm + 5 -22mm bars. Area provided = 3800 mm^2

P_t = 100 × 3800/(400 × 3958.45) = 0.24 %

The height h where half of the reinforcement can be curtailed is approximately equal to $\sqrt{H} = \sqrt{9.23}$ = 3.04m

Curtail 5 bars at 3.04 – L_d from top i.e. 3.04 – 1.03 = 2.01 m from top

Step 8: Detailing

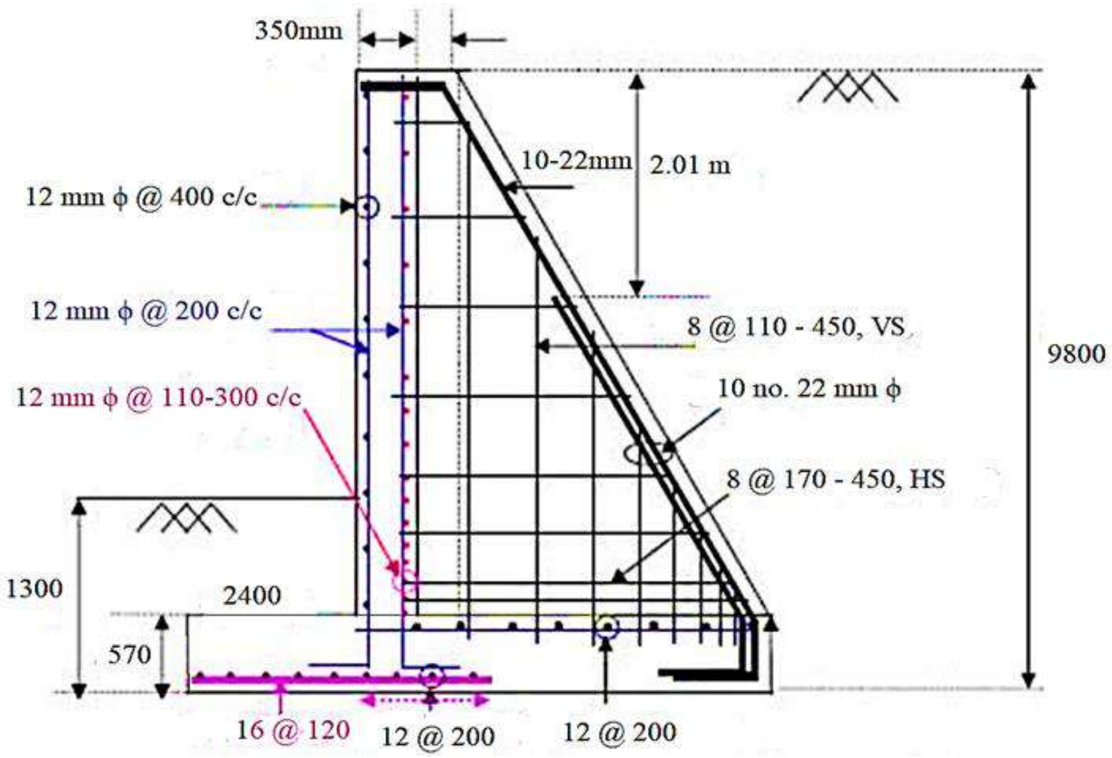

Fig 9.10: Cross sectional details of counterfort wall

The manually designed counterfort retaining wall of overall height 9.8 m has been analysed using the geotechnical software GEO5.

9.6 STABILITY ANALYSIS OF COUNTERFORT RETAINING WALL USING GEO5

The stability analysis of the counterfort retaining wall has been performed as three separate cases as follows:

Case I : Analysis for stability of selected height

Case II : Trial for various backfill depths of crusher dust and backfill material

Case III : Check of effect of water table at various depths

9.6.1 Case I: Analysis for stability of selected height

Analysis for stability of designed overall height of 9.8 m, considering the backfill to be of soil (gravelly clay loam) and the water table is not considered.

I. Analysis for overall height of 9.8 m

A. Open the software GEO5, cantilever wall design program. Click on the “project” option.

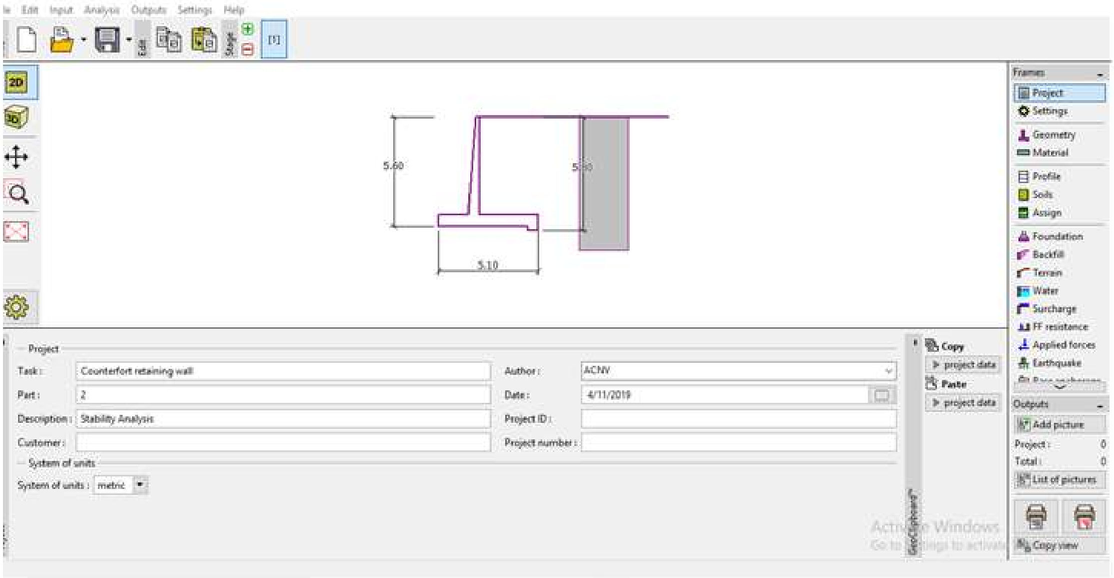

Fig 9.11: Project Frame

- Insert Task, description, Author name, Part, Date, and Customer name (if any)
- Insert the unit in metrics
- Click on "Analysis methods" and select proper codes and methods.

B. Click on the "settings" option

- Verification methodology – Classical way
- Choose analysis standard – IS 456
- Analysis theory – Mononobe Okabe

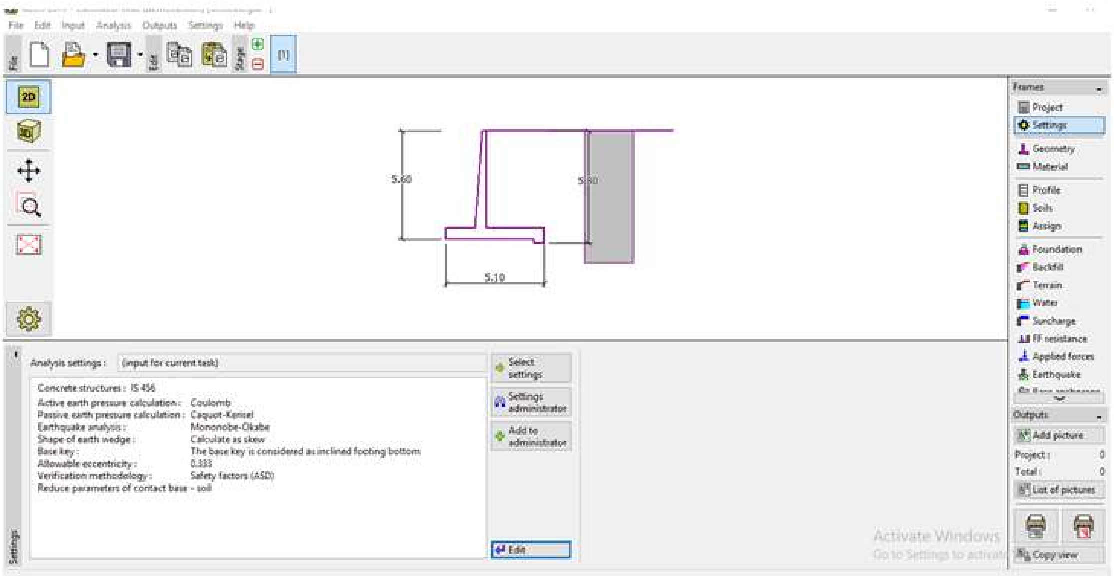

Fig 9.12: Settings frame

C. In the frame "Geometry", choose the wall shape and enter its dimensions.

- Select shape and select type as counterfort.
- Insert dimensions as in chart of geometry

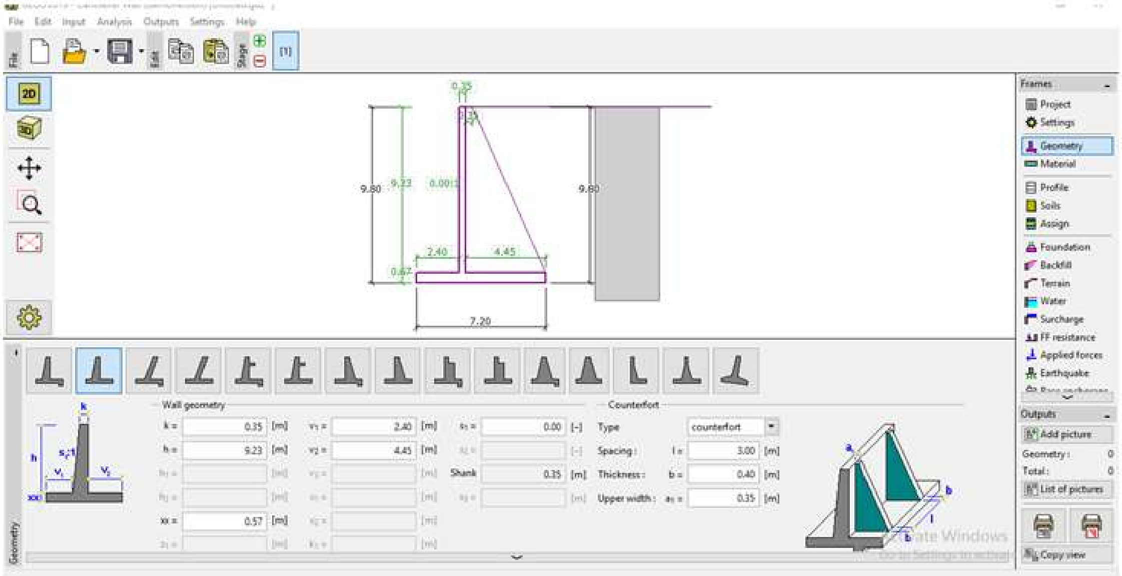

Fig 9.13: Geometry frame

D. In the frame "material", enter the material of the wall

- Enter unit weight- 25 kN /m^3
- Click on catalogue & select characteristic strength of concrete f_{ck}=25 MPa
- Click on catalogue and select longitudinal reinforcement-f_y=415 MPa

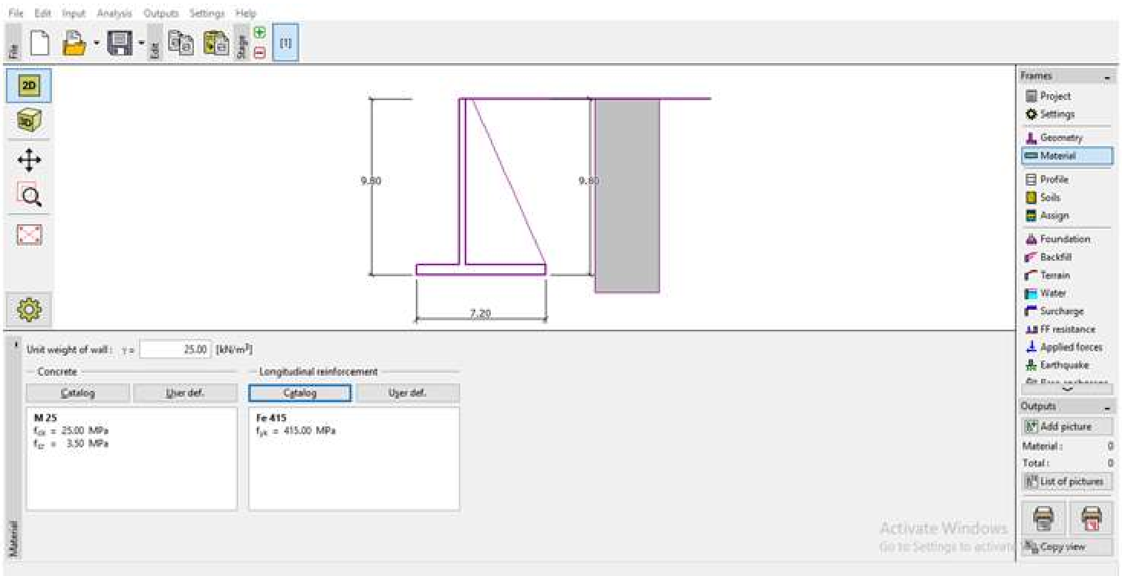

Fig 9.14: Material frame

E. In the frame “profile”, enter the depth of soil layers

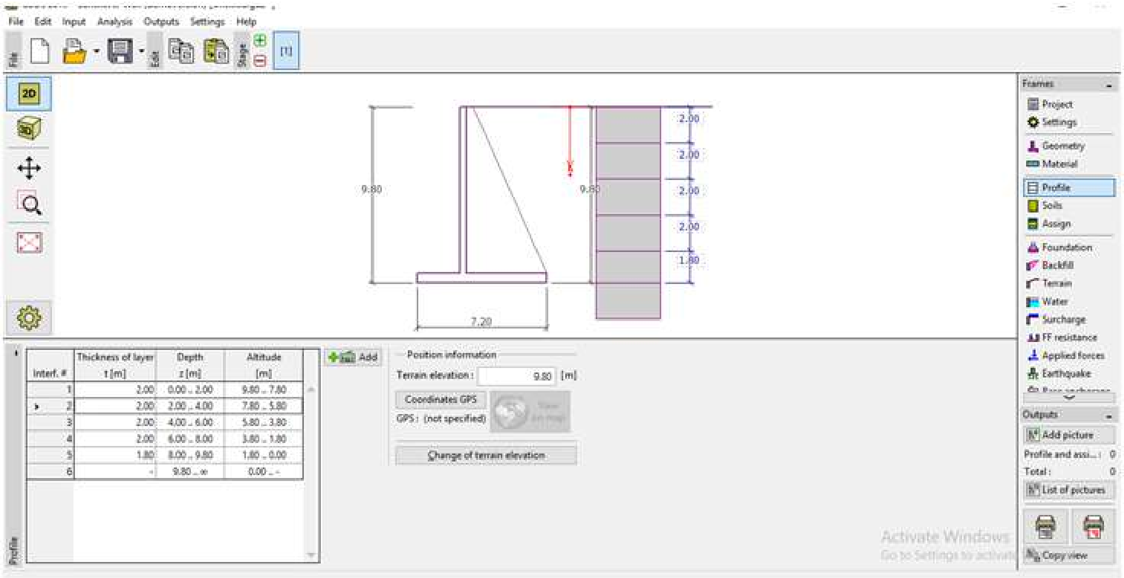

Fig 9.15: Profile frame

- “add” depth of top soil layer from top

F. In the frame “Soils”, define the parameters of soil

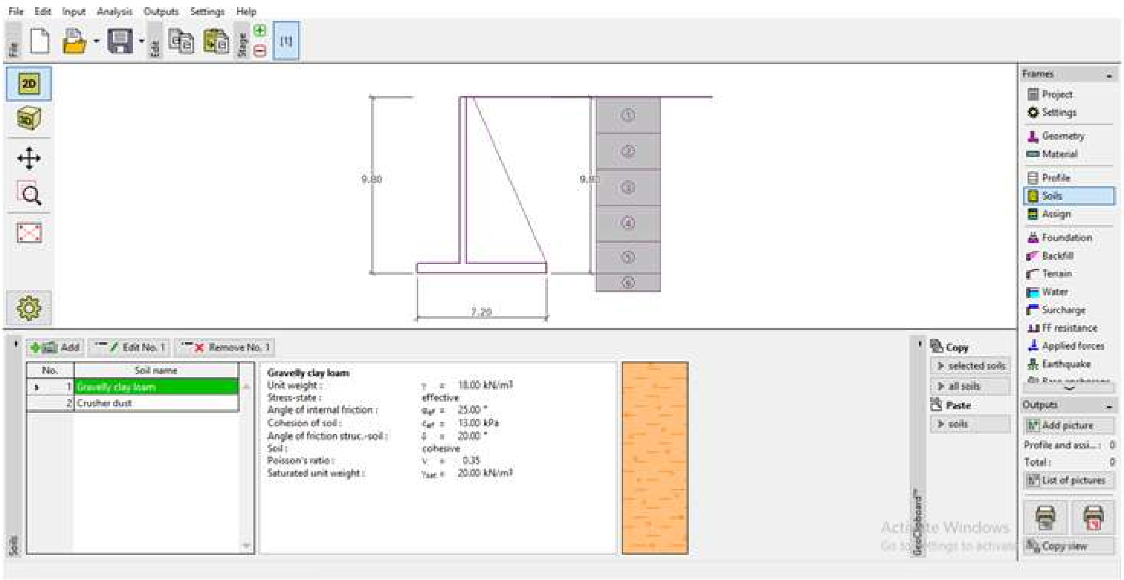

Fig 9.16: Soil frame

- click on “add”
- Enter the properties γ, c, ϕ and δ
- Wall stem is normally analyzed for pressure at rest. For pressure at rest analysis, select “cohesion less” soil. Since our soil is cohesive, select “cohesive” soil
- Enter γ_{sat}= 20 kN /m^3

- Enter name and choose Pattern of soil

Otherwise from option "Classify" select type of soil as shown below and enter properties. The magnitude of active pressure depends also on the friction between the structure and soil.

G. Assign the properties by clicking on the option "Assign"

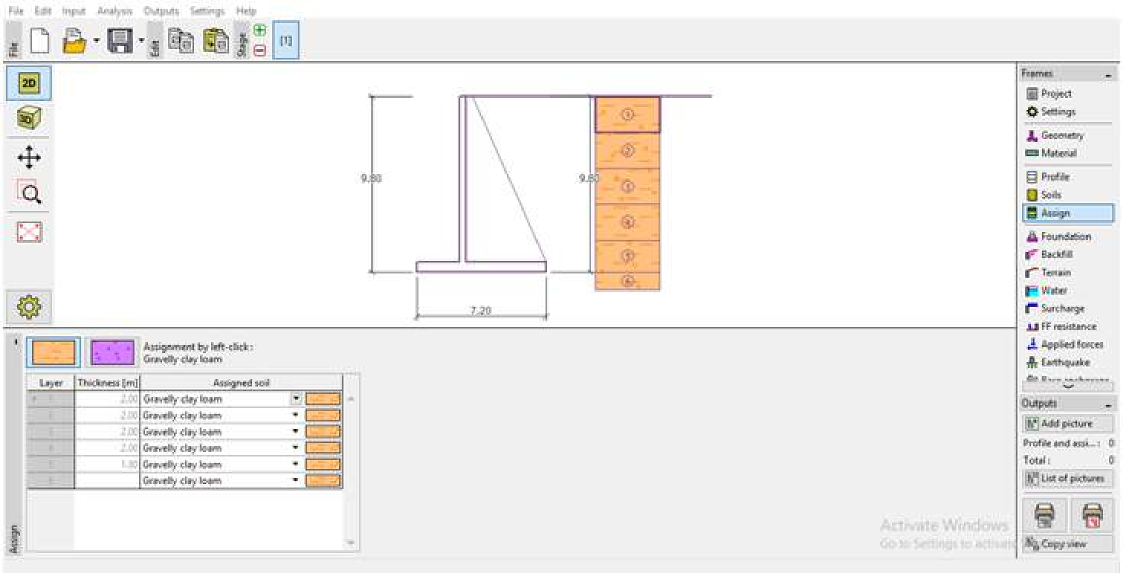

Fig 9.17: Assign frame

H. In the frame "Terrain" choose the horizontal terrain shape

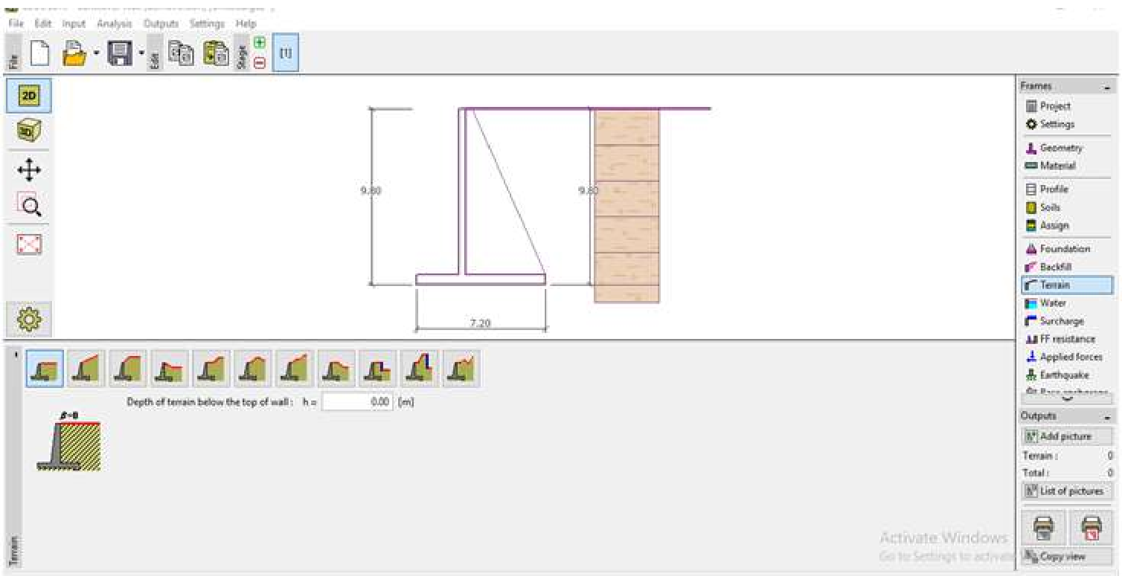

Fig 9.18: Terrain frame

I. In the frame “Water”, select the type of water close to the structure and its parameters

- Select position of Water table

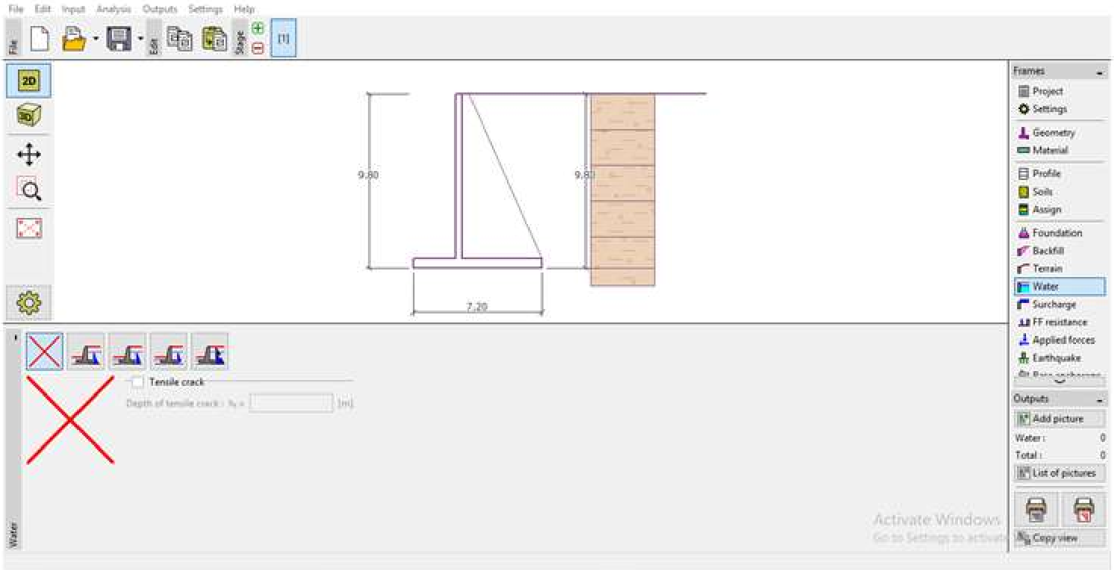

Fig 9.19: Water frame

- In the first case, water table is not considered, that is, the water table is assumed to be at infinite depth

J. Open up the frame “Verification” and analyze the results of overturning and slip of the cantilever wall

K.

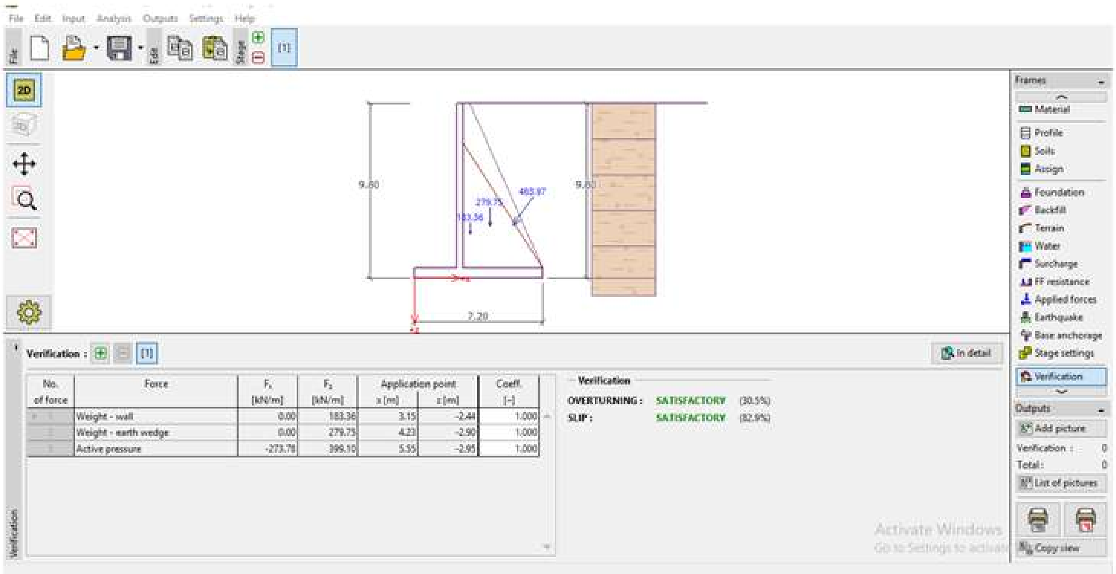

Fig 9.20: Verification frame

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Note: The button "In detail" in the right section of the screen opens a dialog window with detailed information about the analysis results. The overturning and slip of the wall are both satisfactory.

If the verification of the slip is not satisfactory we have several possibilities how to improve the design. For example, we can use better soil behind the wall, anchor the base, increase the friction by bowing the footing bottom or anchor the stem. These changes would be economically and technologically complicated, so choose the easiest alternative. The most efficient way is to change the shape of the wall and introduce a wall jump. Change of the design refers to the change of the geometry of the wall.

L. In the frame "Bearing capacity", perform an analysis for design bearing capacity of the foundation soil 145 kPa

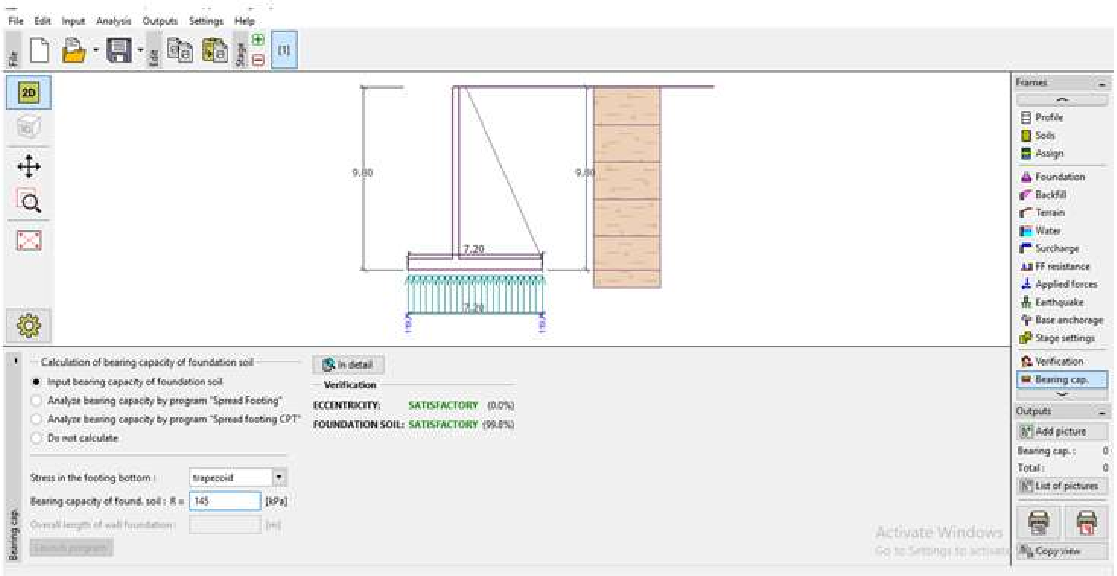

Fig 9.21: Bearing capacity frame

Note: In this case, we analyze the bearing capacity of the foundation soil on an input value, which we can get from geological survey, resp. from some standards. These values are normally conservative, so it is generally better to analyze the bearing capacity of the foundation soil in the program Spread footing that takes into account other influences like inclination of load, depth of foundation etc.

M. Open up the frame "Stability", and analyze the overall stability of the wall

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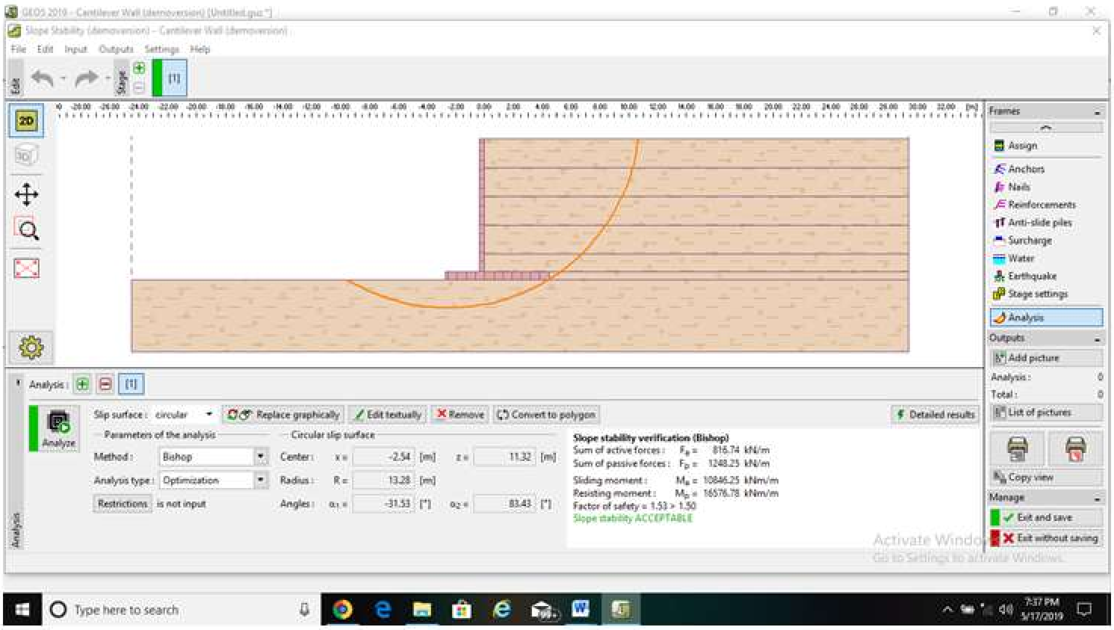

Fig 9.22: Stability frame

Results or pictures will be shown in the report of analysis in the program Counterfort wall

N. Results of analysis

Overturning	:30.50 % ; SATISFACTORY
Slip	:82.90 % ; SATISFACTORY
Eccentricity	:0.00 % ; SATISFACTORY
Foundation soil	:99.80 % ; SATISFACTORY
Factor of Safety	:1.53 > 1.5 ; SATISFACTORY
Overall stability	: This counterfort wall is overall SATISFACTORY

9.6.2 Case II: Trial for various backfill depths of crusher dust and backfill material

In this trial, all steps are same as that of 9.6.1 except for the frame “assign”, were different layers have to be assigned with the required type of fill. The slope stability results of various cases obtained are as follows:

Case 1: Backfill completely with gravelly clay loam

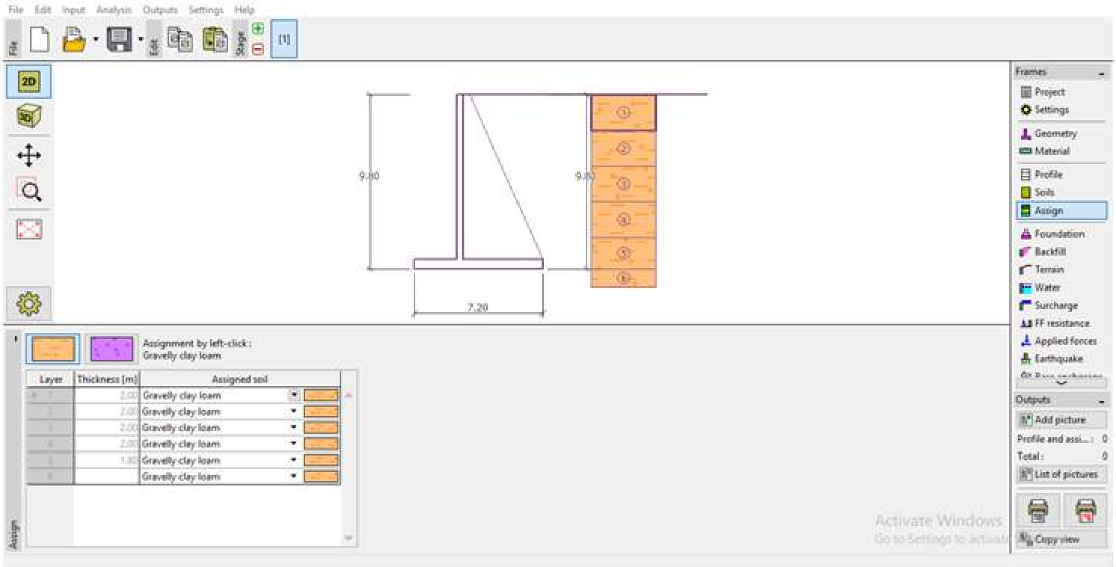

Fig 9.23: Assign frame

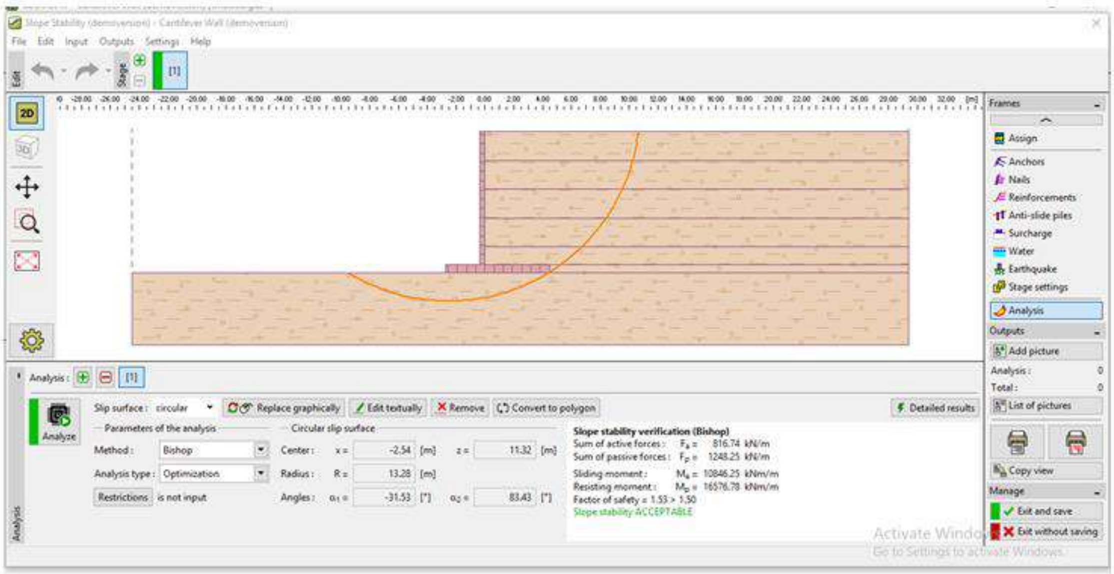

Fig 9.24: Stability frame

Factor of Safety = 1.53 > 1.50

Case 2: Backfill of 2m crusher dust from top and remaining with gravelly clay loam

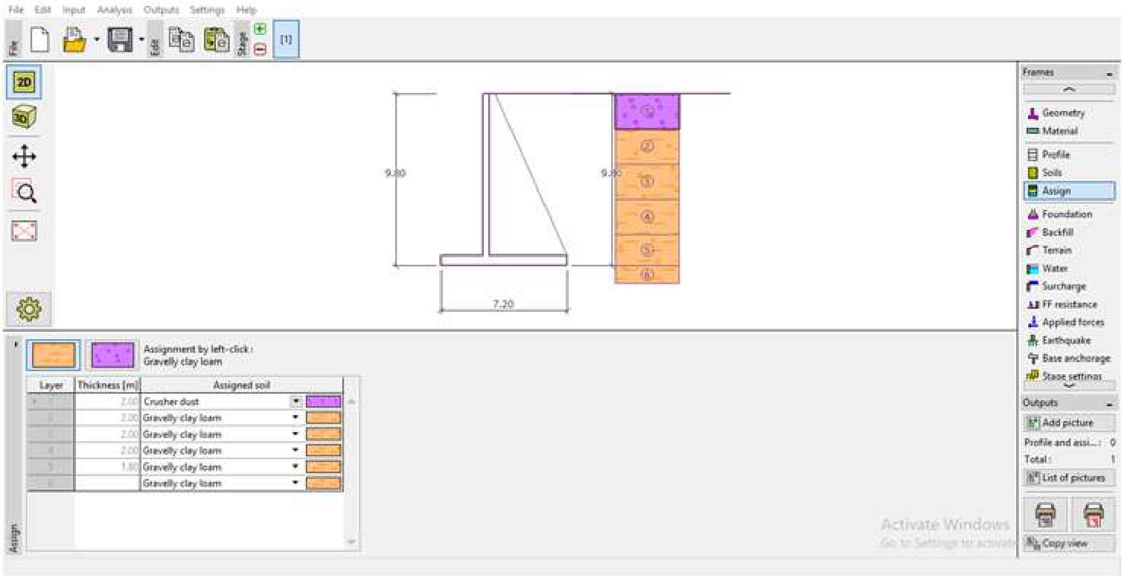

Fig 9.25 Assign frame

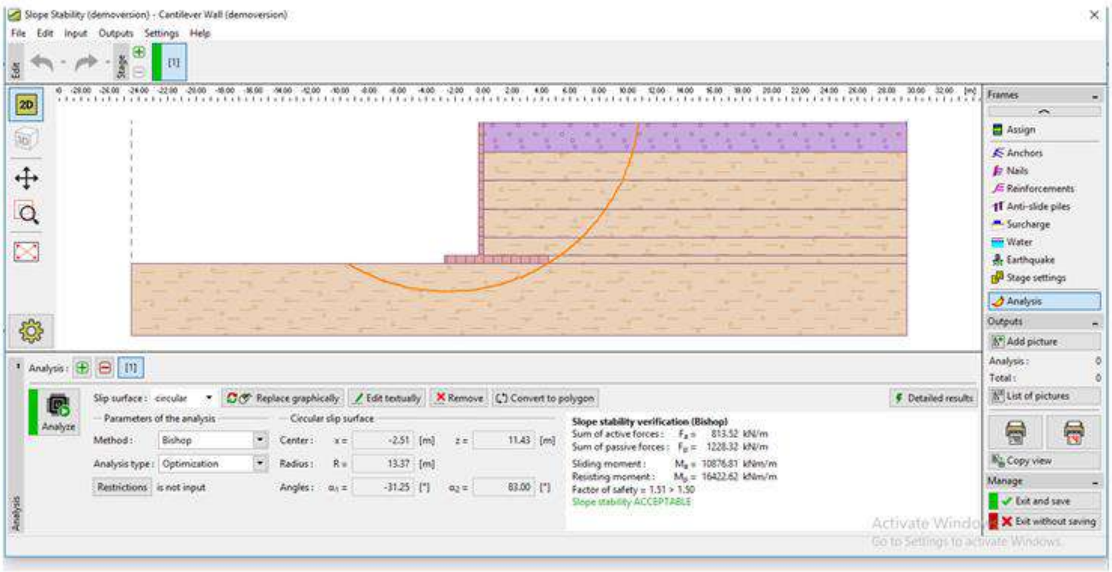

Fig 9.26: Stability frame

Factor of Safety = 1.51 > 1.50

Case 3: Backfill of 6m crusher dust from top and remaining with gravelly clay loam

Fig 9.27: Assign frame

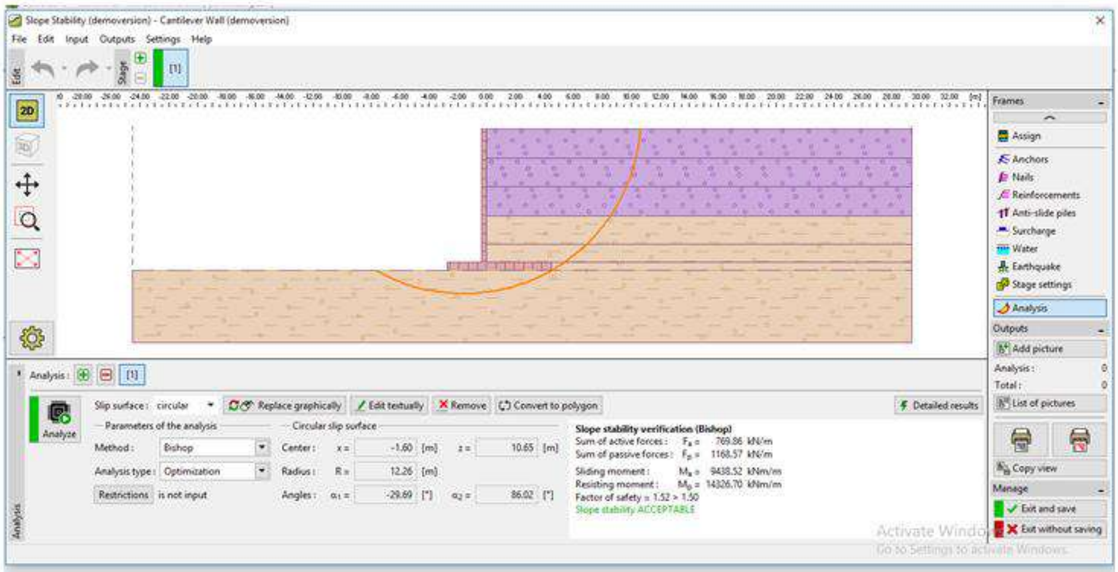

Fig 9.28: Stability frame

Factor of Safety = 1.52 > 1.50

Case 4: Backfill of 8 m crusher dust from top and remaining with gravelly clay loam

Fig 9.29: Assign frame

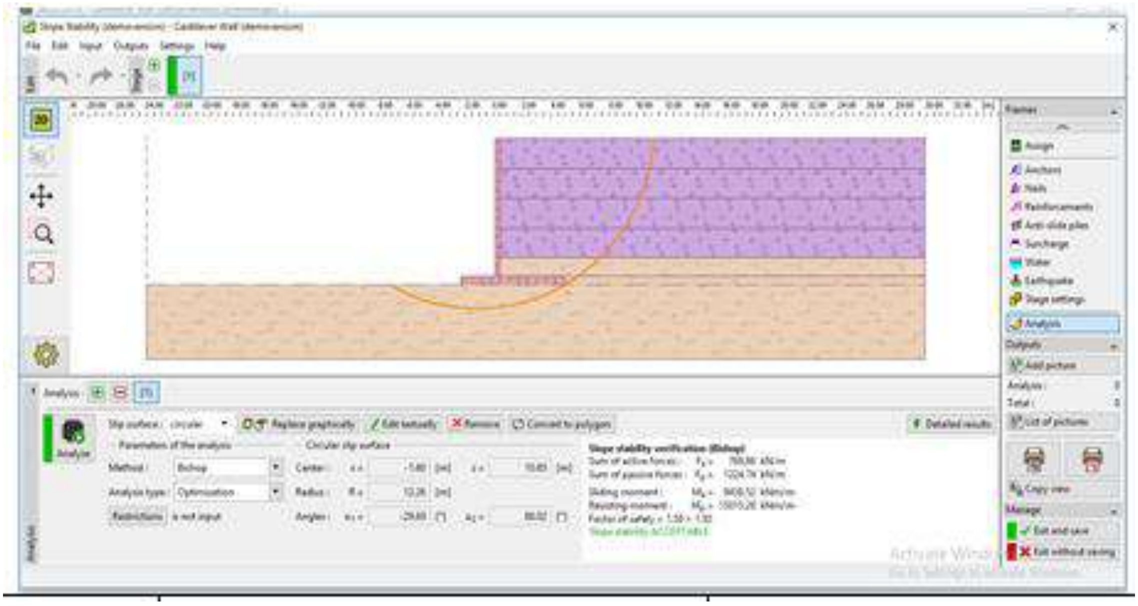

Fig 9.30: Stability frame

Factor of Safety = 1.59 > 1.50

Case	Backfill criteria	Factor of Safety (should be > 1.5)
1	0m crusher dust + 9.8m soil	1.53
2	2m crusher dust + 7.8m soil	1.51
3	6m crusher dust + 3.8m soil	1.52
4	8m crusher dust + 1.8m soil	1.59

Table 9.6: Variations in Factor of safety

From the results, it is observed that the backfill with crusher dust fill upto 8m gives the maximum factor of safety and hence, this criterion is adopted for further analysis.

9.6.3 Case III: Check for the effect of water table for various heights.

The effect of water table for various depths below the ground level has been analysed. The stability results of different cases are as follows:

Case 1: Water table is at considered to be at the surface

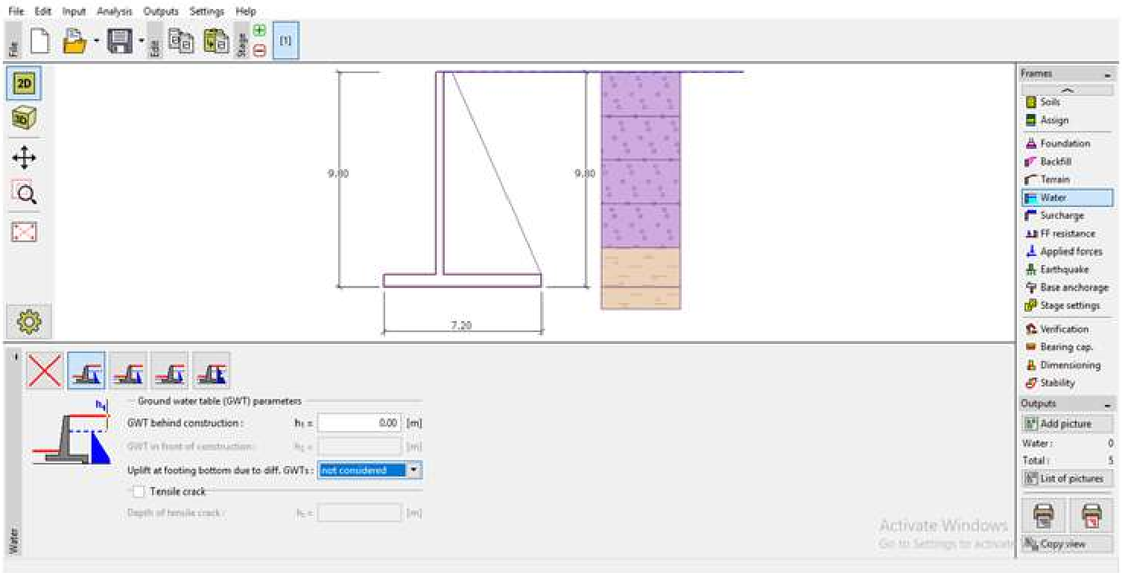

Fig 9.31: Water frame

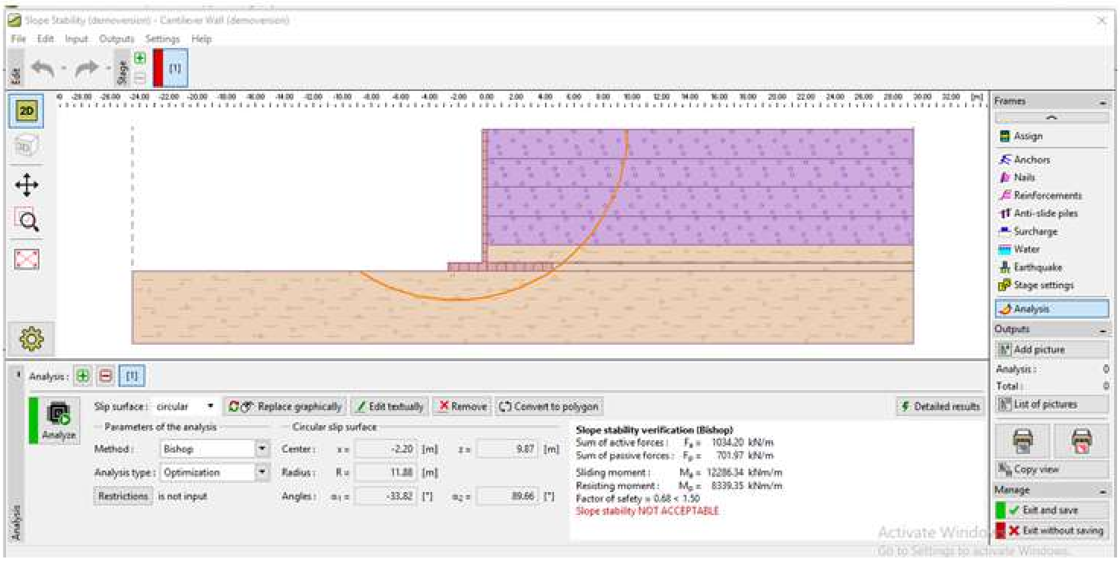

Fig 9.32: Stability frame

Factor of Safety = 0.68 < 1.50; NOT SATISFACTORY

Case 2: Water table is considered at a depth of 10m

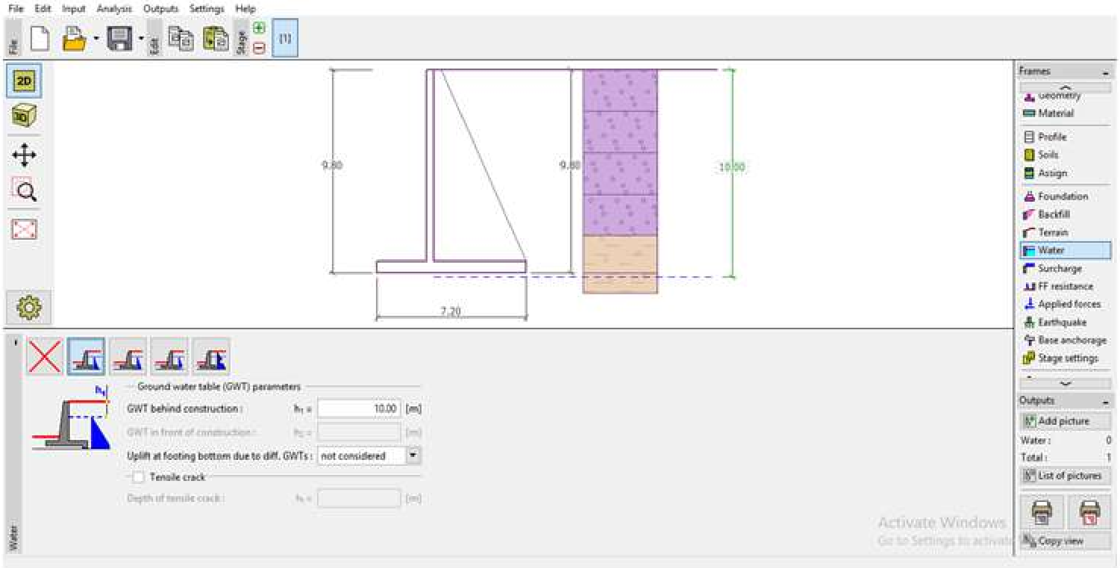

Fig 9.33 Water frame

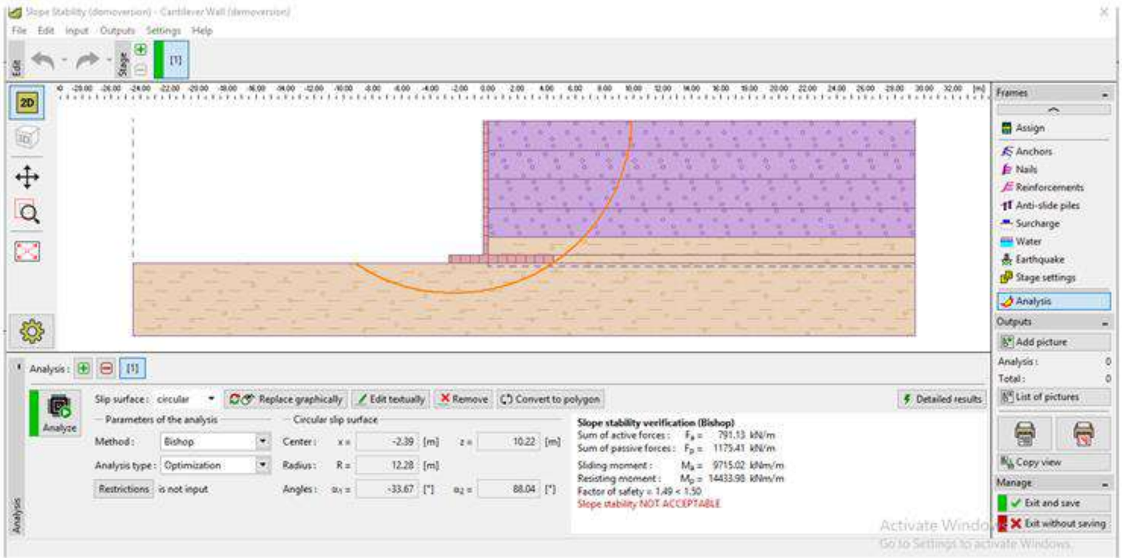

Fig 9.34: Stability frame

Factor of Safety = 1.49 < 1.50; NOT SATISFACTORY

Case 3: Water table is considered at a depth of 20m

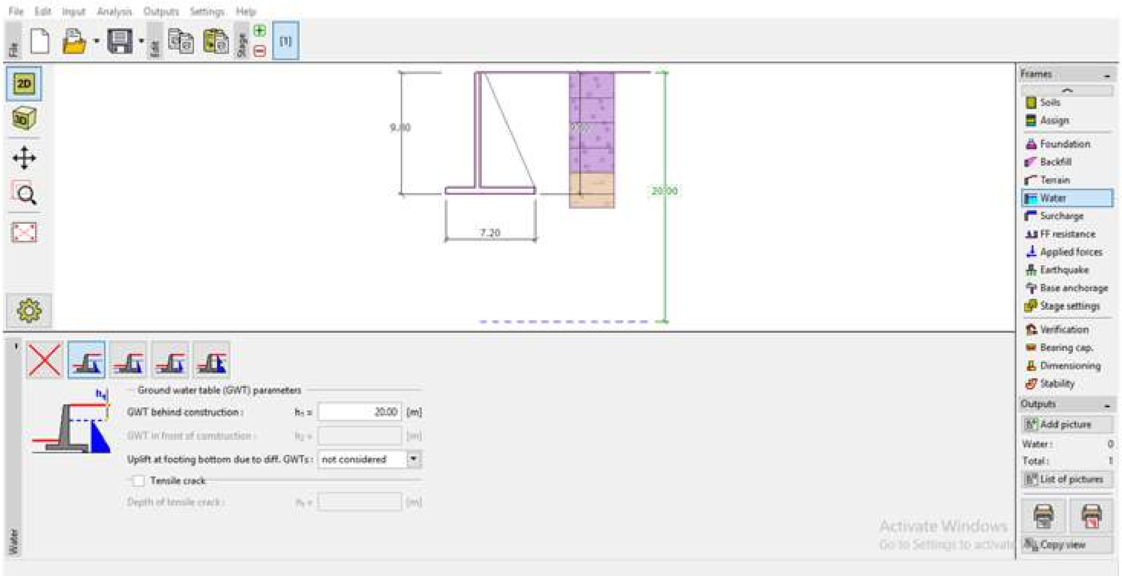

Fig 9.35: Water frame

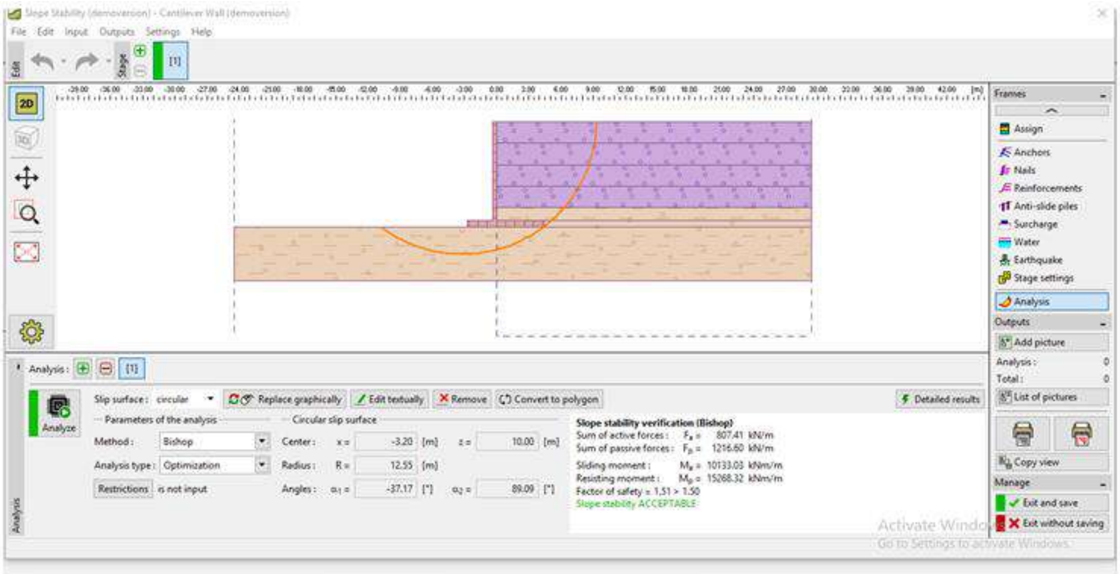

Fig 9.36 Stability frame

Factor of Safety = 1.51 > 1.50; SATISFACTORY

Case 4: Water table is considered at a depth of 40m

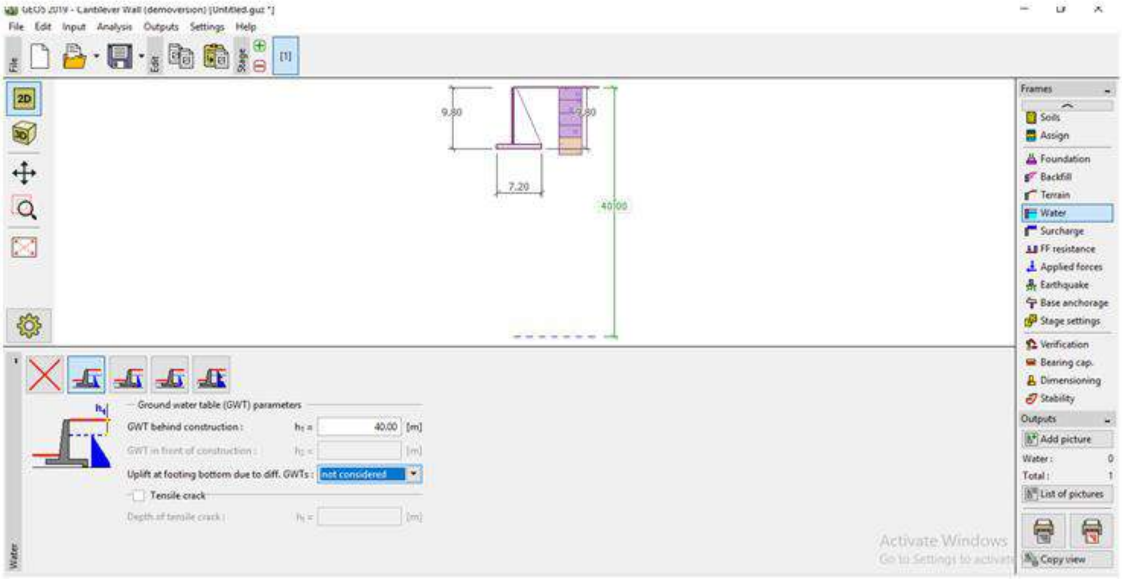

Fig 9.37 Water frame

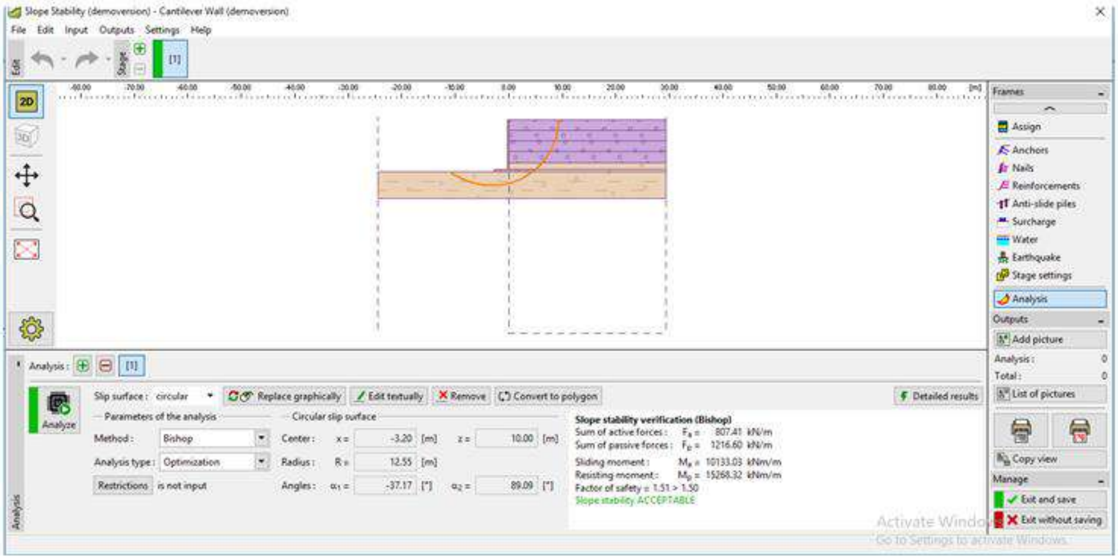

Fig 9.38: Stability frame

Factor of Safety = 1.51 > 1.50; SATISFACTORY

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Case 5: Water table is considered at a depth of 60m

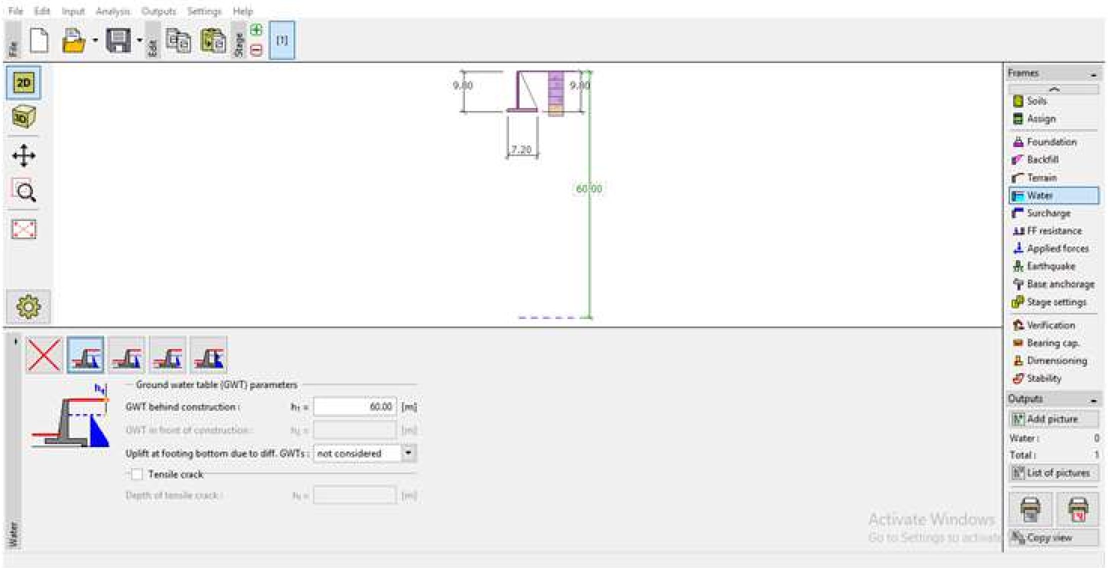

Fig 9.39: Water frame

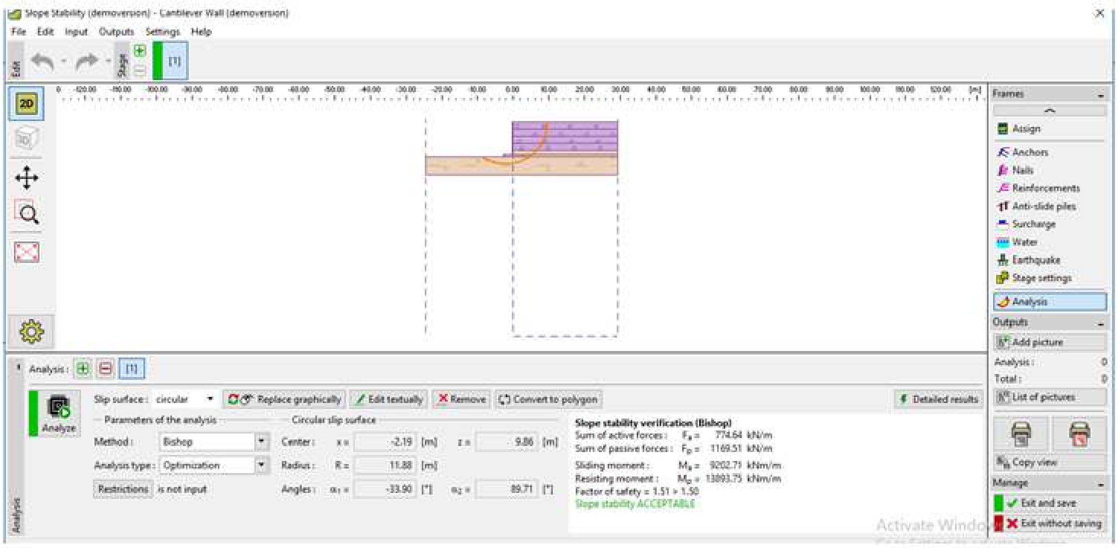

Fig 9.40: Stability frame

Factor of Safety = 1.51 > 1.50; SATISFACTORY

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Case 6: Water table is considered at a depth of 120m

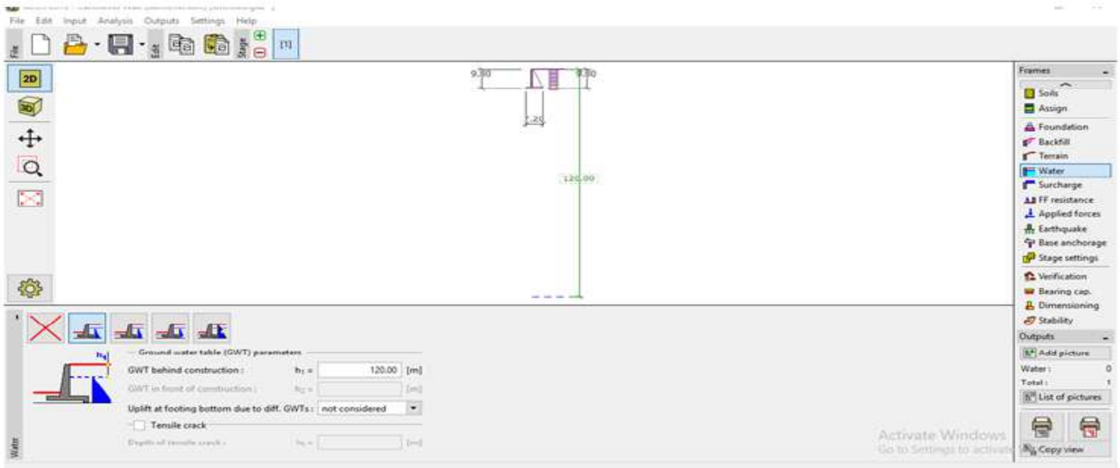

Fig 9.41 Water frame

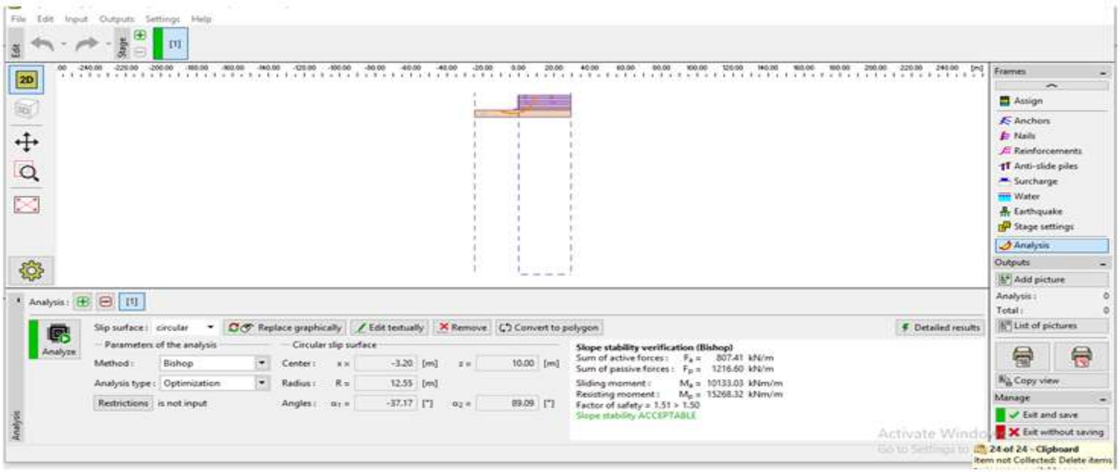

Fig 9.42: Stability frame

Factor of Safety = 1.51 < 1.50; SATISFACTORY

Case	Depth of water table	Factor of safety (should be > 1.5)
1	At the surface	0.68
2	10 m below the surface	1.49
3	20 m below the surface	1.51
4	40 m below the surface	1.51
5	60 m below the surface	1.51
6	120 m below the surface	1.51

Table 9.7: Variations in Factor of Safety

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From the results, it is observed that, as the depth of the water table increases, there is no visible effect on the structure. From the data collected, is is found that the water table level of Kuranchery ranges between 20m and 120m. The designed stucture is thus observed to have the same effect in this depth.

9.7 CHECKS ON THE FINAL DESIGN OF COUNTERFORT RETAINING WALL – USING GEO5

The counterfort retaining wall of overall depth of 9.8m and having a backfill consisting of 8m crusher dust from the top and the remaining portion of soil, is finalized as the design for the thesis. Various checks have been performed using GEO5 are the results obtained are as follows:

9.7.1 Check for overturning and slip

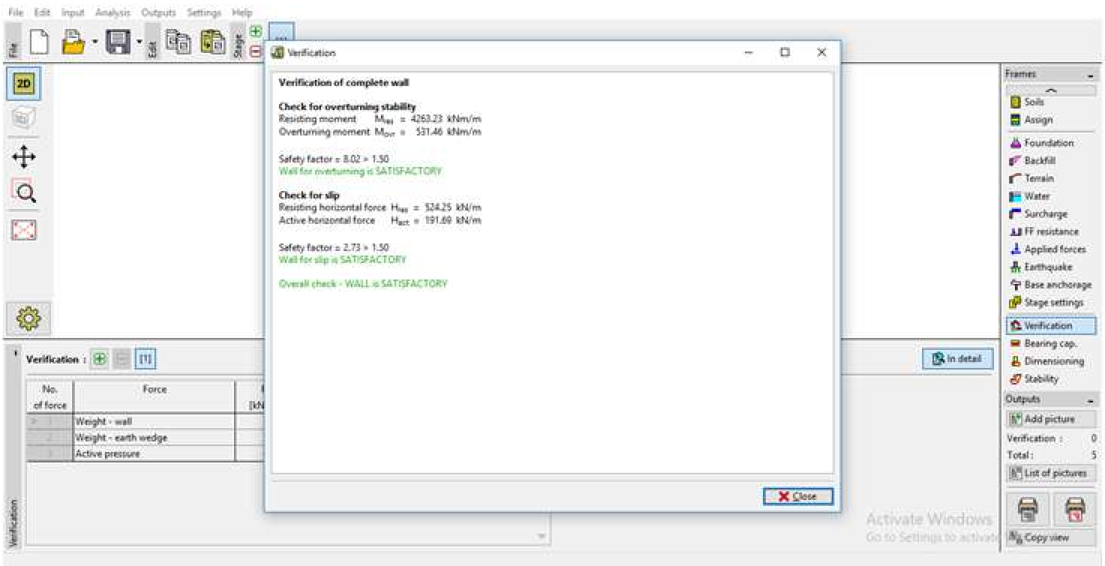

Fig 9.43: Verification frame

Overturning and slip are found to be satisfactory as the factor of safety obtained is greater than 1.5. The diagrammatic representation is shown in fig 9.41.

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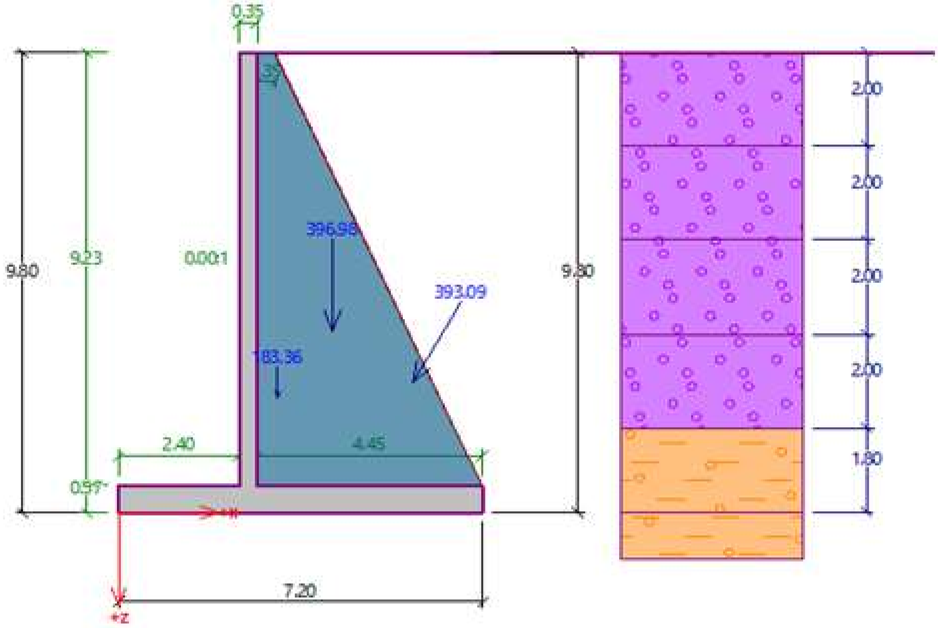

Fig 9.44 Diagrammatic representation of verification

9.7.2 Check for eccentricity and foundation soil

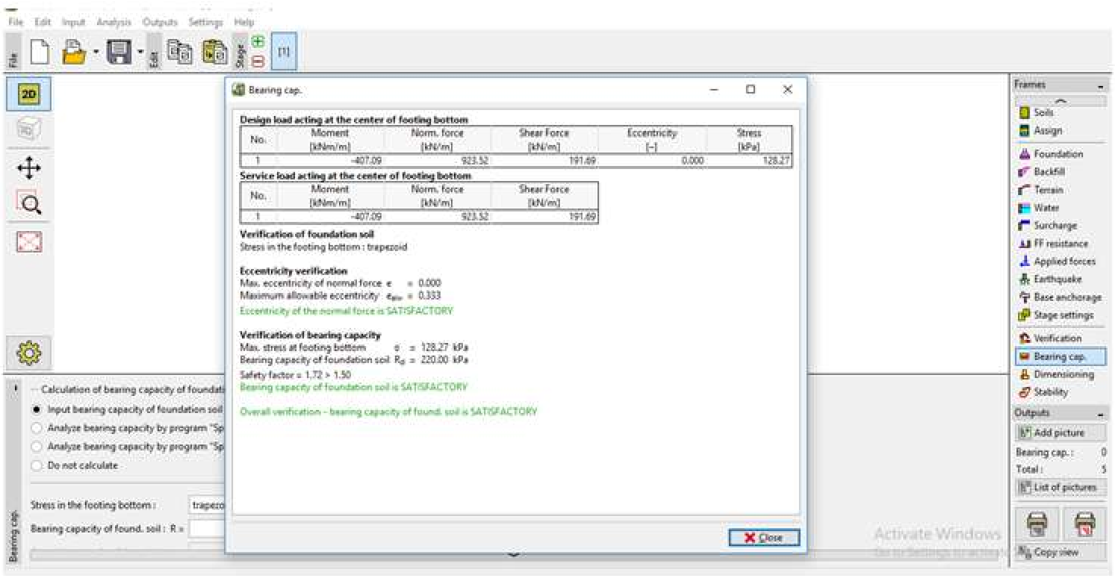

Fig 9.45: Bearing capacity frame

Eccentricity and foundation soil are found to be satisfactory as the factor of safety obtained is greater than 1.5. The diagrammatic representation is shown in fig

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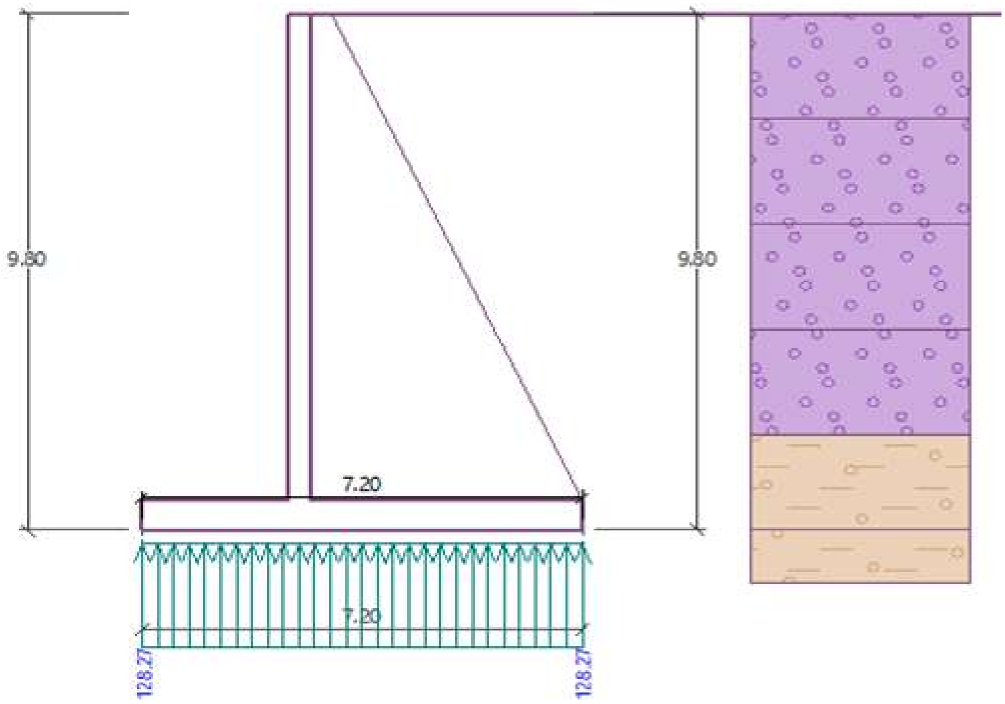

Fig 9.46: Diagrammatic representation of bearing capacity

9.7.3 Wall stem check

Provisions for the satisfactory wall jump check are as follows:

✓ *Wall stem check - front vertical reinforcement – Mu*

Wall check at the construction joint 4.62 m from the wall crest

σ_{Hi} = 94.98 kPa

Mu = 0.03 $\times\sigma_{Hi}\times H_1\times l$ / 4×b= 0.03× 94.98× 9.23× 3.00 / 4× 1.00 = 19.73 kNm Reinforcement & dimensions of the cross-section= 6 no. 20 mm ϕ bars, cover 30.0 mm

Cross-section width = 1.00m and cross-section height = 0.35m

Reinforcement ratio, $\rho = 0.61\% > 0.20\% = \rho_{min}$

Position of neutral axis, $x = 0.05m < 0.15m = x_{max}$

Ultimate moment, $M_{rd} = 192.16$ kNm > 19.73 kNm = M_u

Cross-section is SATISFACTORY. (Fig 9.44)

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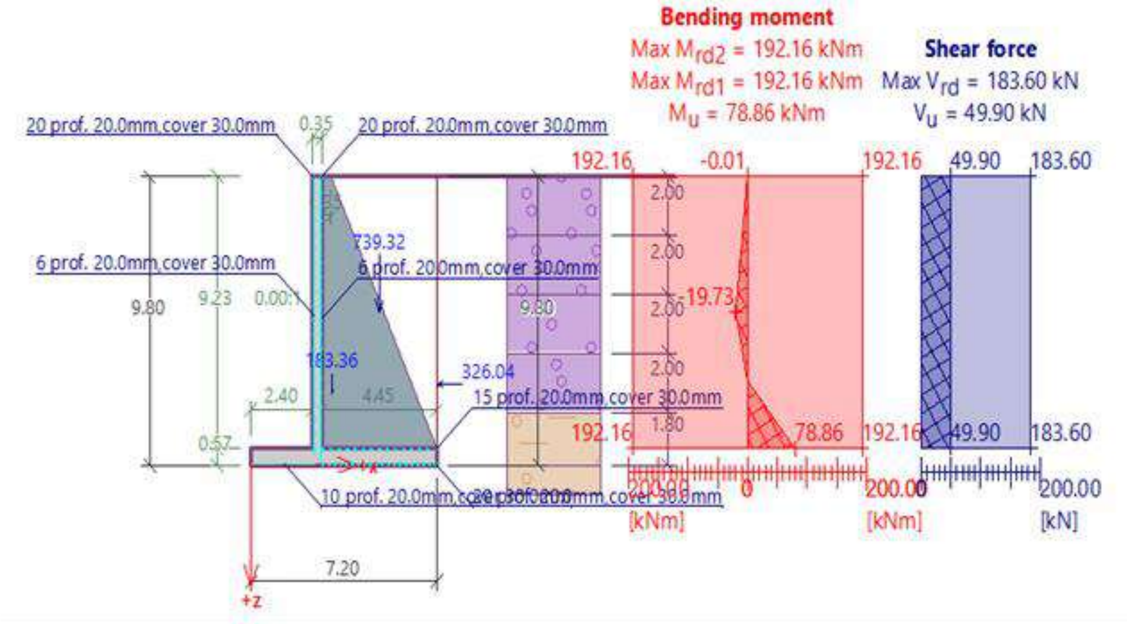

Fig 9.47: Dimensioning- wall stem check (a)

✓ *Wall -stem check - front vertical reinforcement – V_u*

Wall check at the construction joint 9.23 m from the wall crest

Reinforcement & dimensions of the cross-section= 6 no. 20mm ϕ, cover 30mm

Cross-section width = 1.00 m and cross-section height= 0.35 m

Ultimate shear force V_{rd} = 183.60 kN > 49.90 kN = V_u

Cross-section is SATISFACTORY. (Fig 9.44)

- *Wall stem check - back vertical reinforcement*

Wall check at the construction joint 9.23 m from the wall crest

σ_{Hi} = 94.98 kPa

$M_u = 0.03 \times \sigma_{Hi} \times H_1 \times l \times b = 0.03 \times 94.98 \times 9.23 \times 3.00 \times 1.00 = 78.86$ kNm

Reinforcement & dimensions of the cross-section = 6 prof. 20mm, cover 30mm

Cross-section width = 1.00 m and cross-section height= 0.35 m

Reinforcement ratio, $\rho = 0.61\% > 0.20\% = \rho_{min}$

Position of neutral axis, $x = 0.05m < 0.15m = x_{max}$

Ultimate shear force, Vrd = 183.60 kN > 49.90 kN = V_u

Ultimate moment, M_{rd} = 192.16 kNm > 78.86 kNm = M_u

Cross-section is SATISFACTORY. The diagrammatic representation is shown in fig 9.45:

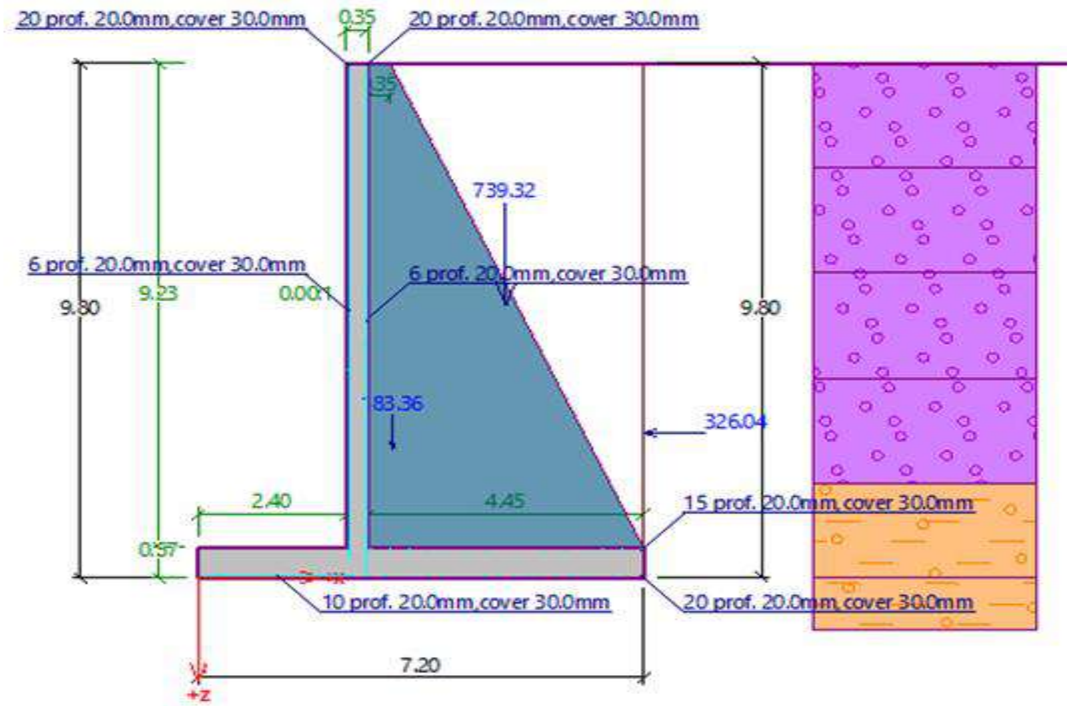

Fig 9.48 Dimensioning- wall stem check (b)

- *Wall stem check - front horizontal reinforcement*

σ_{pi}= 33.27 kPa

$M_u = 1 / 20 \times \sigma_{pi} \times l^2 = 1 / 20 \times 33.27 \times 3.00^2 = 138.19$ kNm

Reinforcement & dimensions of the cross-section= 20 no. 20mm, cover 30mm

Cross-section width = 9.23 m and cross-section height= 0.35 m

Reinforcement ratio, $\rho = 0.22\% > 0.20\% = \rho_{min}$

Position of neutral axis, $x = 0.03m < 0.15m = x_{max}$

Ultimate shear force, V_{rd} = 1094.22 kN > 460.62 kN = V_u

Ultimate moment, M_{rd} = 676.71 kNm > 138.19 kNm = M_u

Cross-section is SATISFACTORY. The diagrammatic representation is shown in fig 9.46:

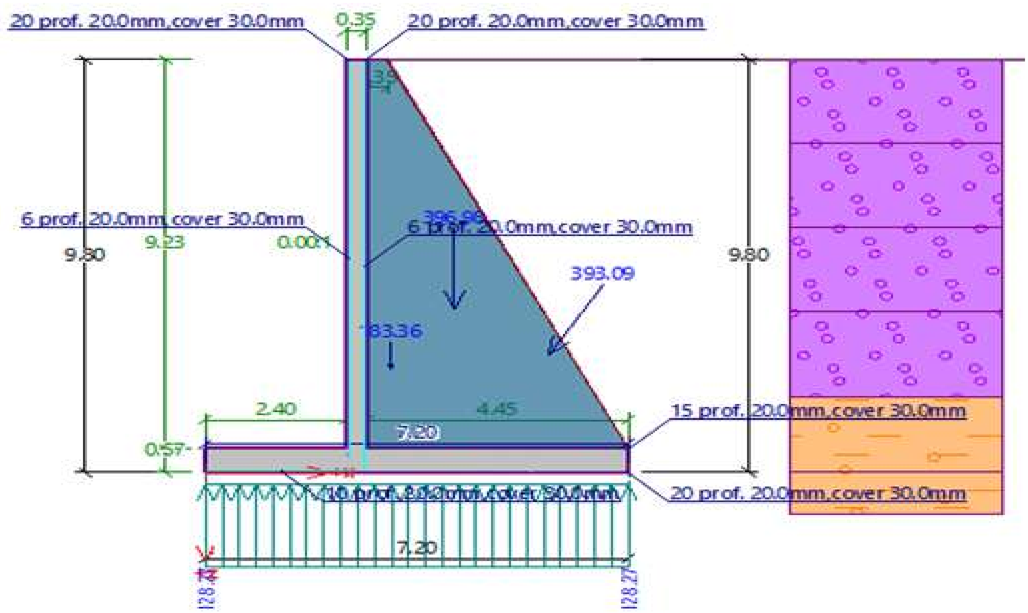

Fig 9.49 Dimensioning- wall stem check (c)

9.7.4 Wall jump check

Provisions for the satisfactory wall jump check are as follows:

- *Wall jump check - bottom reinforcement*

Reinforcement & dimensions of the cross-section: 10 no. 20mm, cover 30mm

Cross-section width = 1.00 m and cross-section height= 0.57 m

Reinforcement ratio, $\rho = 0.57\% > 0.20\% = \rho_{min}$

Position of neutral axis, $x = 0.13m < 0.25m = x_{max}$

Ultimate shear force, $V_{rd} = 296.22$ kN > 273.64 kN = V_u

Ultimate moment, $M_{rd} = 540.4$ kNm > 328.3 kNm = M_u

Cross-section is SATISFACTORY. The diagrammatic representation is shown in fig 9.46.

9.7.5 Wall Heel check

Provisions for the satisfactory wall heel check are as follows:

- *Wall heel check - bottom reinforcement*

$\sigma_j = 74.11$ kPa

$M_u = 1 / 12 \times \sigma_j \times l^2 = 1 / 12 \times 74.11 \times 3.00^2 = 247.35$ kNm

Reinforcement & dimensions of the cross-section = 20 no. 20.mm, cover 30 mm

Cross-section width = 4.45 m and cross-section height= 0.57 m

Reinforcement ratio, $\rho = 0.27\% > 0.20\% = \rho_{min}$

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Position of neutral axis, $x = 0.06m < 0.25m = x_{max}$

Ultimate shear force, $V_{rd} = 934.78$ kN > 494.70 kN = V_u

Ultimate moment, $M_{rd} = 1147.41$ kNm > 247.35 kNm = M_u

Cross-section is SATISFACTORY. The diagrammatic representation is shown in fig 9.46.

- *Wall heel check - bottom reinforcement*

$\sigma_j = 74.11$ kPa

$M_u = 1 / 20 \times \sigma_j \times l^2 = 1 / 20 \times 74.11 \times 3.00^2 = 148.41$ kNm

Reinforcement & dimensions of the cross-section = 18 no. 20.mm, cover 30 mm

Cross-section width = 4.45 m and cross-section height= 0.57 m

Reinforcement ratio, $\rho = 0.24\% > 0.20\% = \rho_{min}$

Position of neutral axis, $x = 0.05m < 0.25m = x_{max}$

Ultimate shear force, $V_{rd} = 894.97$ kN > 494.70 kN = V_u

Ultimate moment, $M_{rd} = 1037.56$ kNm > 148.41 kNm = M_u

Cross-section is SATISFACTORY. The diagrammatic representation is shown in fig 9.46.

9.7.6 Counterfort check

Wall check at the construction joint 9.23 m from the wall crests. Reinforcement & dimensions of the cross-section:

- 15 prof. 16.0 mm, cover 30.0 mm
- 9 prof. 22.0 mm, cover 30.0 mm
- 8 prof. 22.0 mm, cover 30.0 mm

Cross-section width = 4.40 m and cross-section height= 4.80 m

Reinforcement ratio, $\rho = 0.50\% > 0.20\% = \rho_{min}$

Position of neutral axis, $x = 0.96m < 2.28m = x_{max}$

Ultimate shear force, $V_{rd} = 938.56$ kN > 929.45 kN = V_u

Ultimate moment, $M_{rd} = 14903.95$ kNm > 2471.02 kNm = M_u

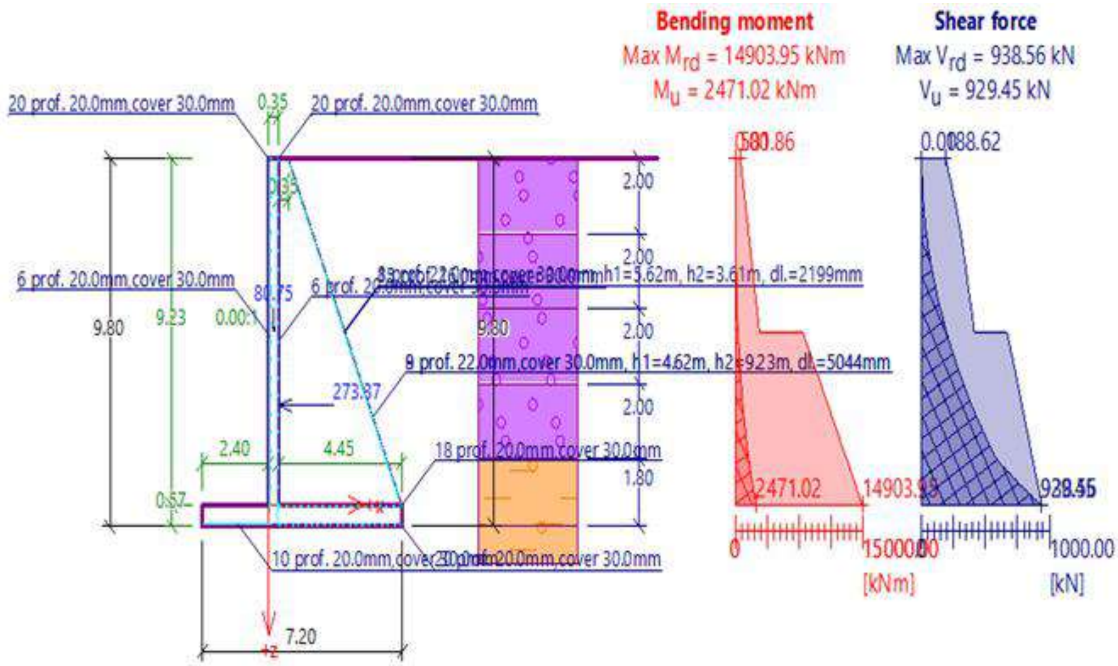

Fig 9.50: Dimensioning- counterfort check

Cross-section is SATISFACTORY. The diagrammatic representation is shown in fig.9.47.

The three dimensional view of the finalized design are shown in fig 9.48 and fig 9.49.

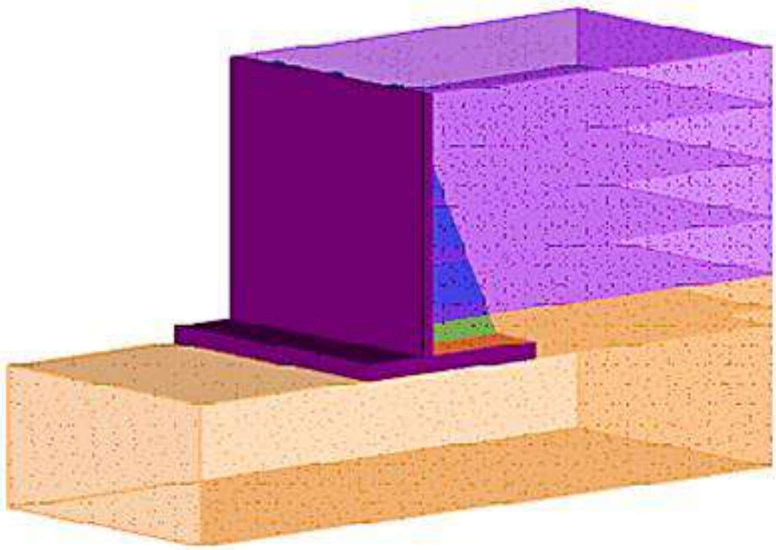

Fig 9.51: 3-D view (a)

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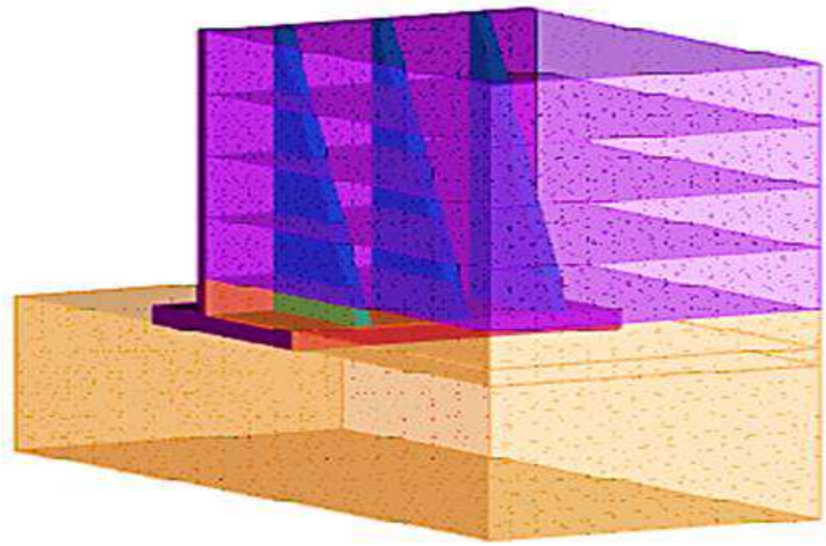

Fig 9.52: 3-D view (b)

Fig 9.50 shows the alignment of the retaining wall along the slope with the aid of cut and fill.

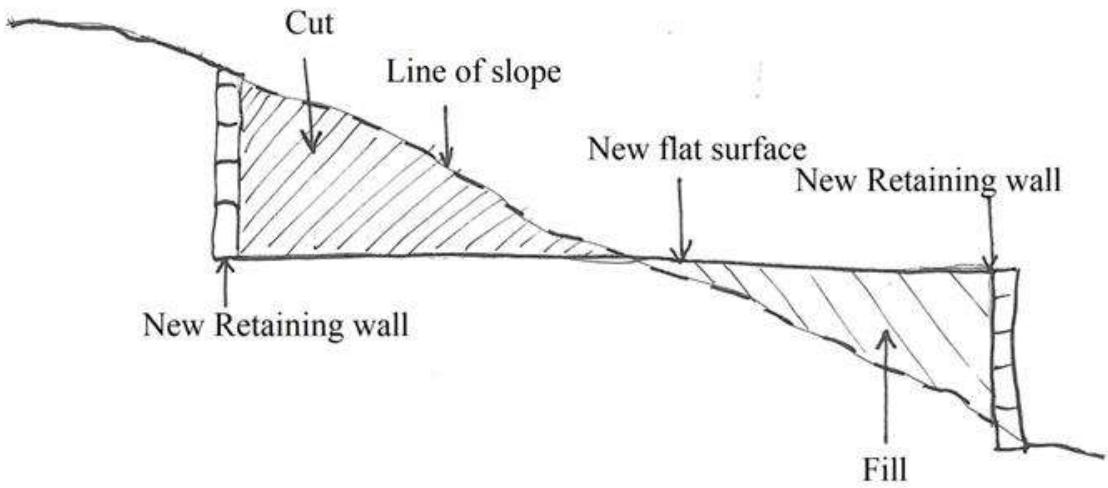

Fig 9.53: Retaining wall alignment -cut and fill

The fill or excavation depth is less than 2m from the natural ground level at any point. The toe of the fill or the top of the excavation is not less than 1.5m from a side or rear allotment boundary. Fill material placed over the service does not impose any additional surcharge loading on the service. Where excavation is carried out, a minimum cover of 600mm is maintained around all utility infra-structure (to top, sides and base of services). Compaction with a vibrating roller is not carried out within 600mm of any utility infrastructure.

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CHAPTER 10

SUMMARY AND CONCLUSIONS

10.1 SUMMARY

The stability analysis of Retaining wall at Kuranchery is carried out using GEO5 software. Evaluation of the stability of the whole structure under the service loads, including overturning, sliding and bearing failure modes, have been performed successfully. The soil parameters required for the design of the retaining wall was obtained from the geotechnical test conducted on the soil sample collected from Kuranchery. The test results showed that the soil is of gravelly clay loam texture and has a safe bearing capacity of $145 kN/m^2$, with a unit weight of $18 kN/m^3$. Primarily, cantilever retaining wall was designed for various heights. But from the results it is observed that the stability is altered as the height of the wall retaining wall increases and thus the structure becomes more uneconomical. Thus it can be concluded that above heights of 8m, counterfort retaining wall is economical and hence a wall of overall height 9.8m has been designed and analysed for stability as per the recommendations of IS 456: 2000.

10.2 CONCLUSIONS

The stability of the backfill is improved by the addition of the crusher dust/quarry dust which is a waste product obtained from the quarry. Angle of internal friction for crusher dust (44.71°) is more than gravelly clay loam from Kuranchery (25°). The addition of quarry dust decreases the OMC and increases the MDD of the soil, decreases the cohesion and increases the angle of internal friction. As angle of internal friction increases, the factor of safety increases for a fill material and the design become stable. From the GEO5 analysis, it is observed that the factor of safety increases with the increase in depth of crusher dust as fill material. The stability analysis to check the effect of water table level on the structure was made and it is observed that as the depth of water table increases, it has least effect on the structure. It can be thus generalized in this condition that, as the depth of the water table increases beyond twice the length of the base wall of the retaining wall, there is least effect of it on the structure. The thesis mainly focuses on the height of about 20m by providing counterfort walls each of 9.8 m height (layout shown in fig 9.50).

10.3 RECOMMENDATIONS

- It is advisable to provide s sub soil drainage system behind all retaining walls so as not to worsen drainage problems or cause surface water to be a nuisance to neighbouring properties.
- Due to the time limitations, only the stability analysis of retaining wall has been made in the study. But it is recommended to design and analyze soil nailing for heights above so as to assure further stability.

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