“ANALYSIS OF VARIOUS IMPROVEMENTS IN CANTILEVER RETAINING WALL USING GEO 5 SOFTWARE”

A PROJECT

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Under the supervision of

Mr. Chandra Pal Gautam

Assistant Professor

by

ShubhamShakya

(131613)

Kshitij Mehta

(131630)

to

JAYPEE UNIVERSITY OF INFORMATION TECHNOLOGY

WAKNAGHAT, SOLAN – 173 234

HIMACHAL PRADESH, INDIA

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ABSTRACT

In hilly regions it is very important to build a retaining structures which with stand the lateral earth pressure exerted by water pressure, surcharge loads and self-weight of the wall. Retaining walls are built to withstand almost vertical (steeper than 70degrees) or vertical slopes of earth masses. In the past years, there has been rapid development of highways and unstability of retaining walls to cause embankment landslides has become common. We have discussed certain different materials which help in reducing the effective stress on the retaining wall such geofoam and soil mixture of shredded tires and controlled low strength materials. We have also studied what structural modifications can be made into the design of retaining walls so as to increase the stability of retaining walls. We have modified the basic retaining wall by providing a relief shelve and a shear key at various levels. In this project we have also studied the behavior of retaining walls under seismic conditions. This report summarizes various methods which we can use to increase the strength of a retaining wall. Analysis of different retaining walls is done on GEO 5 software.

CHAPTER – 1

INTRODUCTION

1.1General

Retaining walls are rigid structures which are designed to restrain soil to undesirable slopes. These structures are used to bind the soils between different gradients often in regions of terrain having unnatural slopes or in places where the landscape needs to be shaped severely.

The retaining wall is constructed to resist the lateral pressure of soil, when there is a desired difference in ground elevation that exceeds the repose angle of the soil.

A basement wall can be considered as a retaining wall. But the term usually refers to a cantilever retaining wall, which is a detached structure without having any lateral support at its top. These are cantilevered from a footing and rise above the grade on one side to retain a higher level grade on the opposite side. The walls must be strong enough to accommodate the lateral pressures generated by loose soils or, in some cases, water pressures.

Each retaining wall bolsters a “wedge” of soil. The wedge is characterized as the soil which extends beyond the failure plane of the soil type present at the wall site, and can be calculated once the soil friction angle is known. As the setback of the wall increases, the size of the sliding wedge is reduced. This reduction lowers the pressure on the retaining wall.

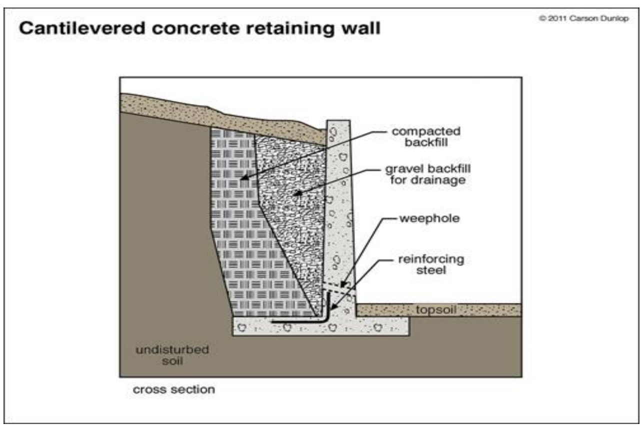
The most essential thought in proper design and installation of retaining walls is to recognize and counteract the tendency of the retained material to move down the because of gravity. Due to this a lateral earth pressure is being created back of the wall which depends on the angle of internal friction (Φ), the cohesive strength of the material which is retained, as well as the direction and magnitude of movement the retaining structure undergoes.

1.2Types of Retaining Walls

1.2.1 Cantilever Retaining Wall

Cantilevered retaining walls are made either from steel reinforcements, cast in situ concrete or masonry.

These walls distribute loads in such a way so as to convert horizontal pressures into vertical pressures on the surface below.



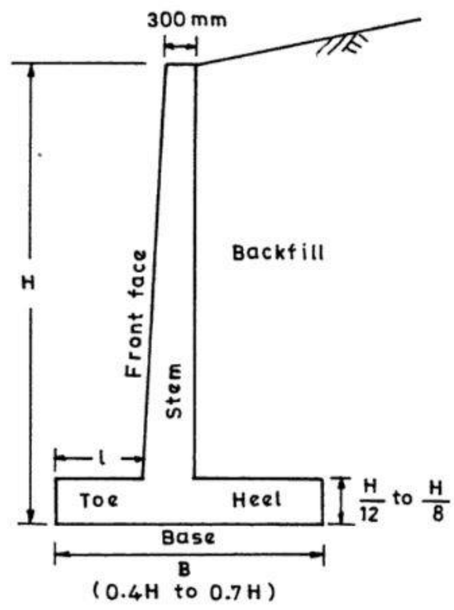


Fig.1.1 Cantilever concrete retaining wall **Fig.1.2 Typical Proportions**

1.2.2Gravity Retaining Wall

Gravity walls resist pressure from behind by virtue of their mass or weight alone.

They can be constructed using concrete, stone or even brick masonry. Usually they possess a thicker cross section. Also their geometry helps them to sustain their stability.

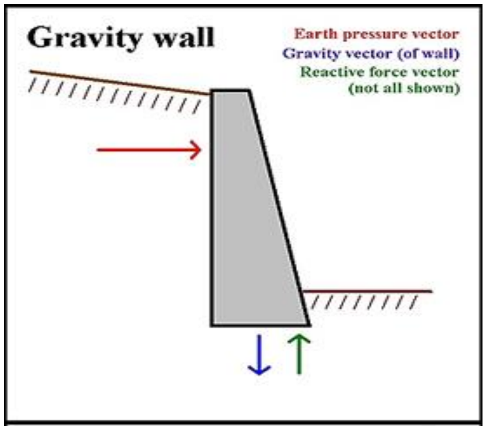


Fig.1.3 Gravity Wall

1.2.3 Bored Pile Retaining Wall

Bored piles retaining walls are made by combining a sequence of bored piles, accompanied by excavating the excess soil.

The requirement of type of bored pile retaining wall depends upon the type of project.Types of bored pile retaining wall may include a series of earth anchors, reinforcing beams, soil improvement operations and shotcrete reinforcement layer.

Fig.1.4 Bored pile retaining wall

1.2.4 Sheet Piling Retaining Wall

They are commonly used in soils having soft texture and tight spaces. Steel, vinyl or wood planks are mainly used for making sheet pile retaining walls which are driven into the ground. The material is usually driven 1/3 above ground, 2/3 below ground, but this may be altered depending on the environment.

Steel sheet piling is usually preferred over other options due to the following reasons:
1. High resistance is provided to driving stresses

2. Weight of the material is light
3. Reusable on several projects
4. Service life is quite strong above or below water only with modest protection
5. Easy to adapt the pile length by either welding or bolting

Fig.1.5 Sheet pile retaining wall

1.2.5 Anchored Retaining Wall

An anchored retaining wall can be constructed in any of the ways mentioned above, additionally, its strength can be enhanced using cables.

This method is usually very complex but it finds it usefulness where high loads are expected.

Fig.1.6 Anchored retaining wall

CHAPTER – 2

LITERATURE REVIEW

2.1 General

2.1.1Forces taken up by Retaining Walls

- **Lateral forces**: Pressure created due to backfill and surcharge.
- **Vertical forces**: -

Forces acting downwards:

1. Self-weight of the retaining wall;
2. Weight of soil above heel slab.

Forces acting upwards: Soil pressure underneath the base slab creates a force in upward direction.

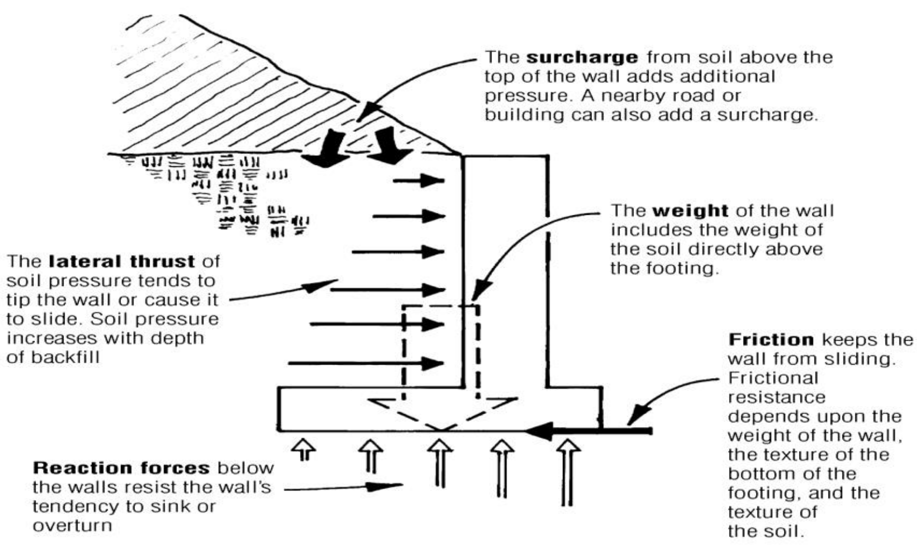


Fig.2.1 Forces acting on a retaining wall

2.2Lateral Earth Pressure

Lateral earth pressure is the pressure that soil exerts in the horizontal direction. The lateral earth pressure is important because it affects the consolidation behavior and strength of the soil and because it is considered in the design of geotechnical engineering structures such as retaining walls, basements, tunnels, deep foundations and braced excavations.

The coefficient of lateral earth pressure, K, is defined as the ratio of the horizontal effective stress, σ'_h, to the vertical effective stress, σ'_v.

The minimum stable value of K is called the active earth pressure coefficient, K_a; the active earth pressure is obtained, for example, when a retaining wall moves away from the soil.

The maximum stable value of K is called the passive earth pressure coefficient, K_p; the passive earth pressure would develop, for example against a vertical plow that is pushing soil horizontally.

2.2.1 Effect of Wall Movement on Earth Pressure

Fig.2.2 presents the results of test conducted by Terzaghi(1929) on large scale model retaining walls. When the wall is rigid and unyielding, the soil mass is in state of rest and there are no deformations and displacements. The earth pressure corresponding to this state is called *earth pressure at rest* (represented by point A in the Fig.2.2). If the wall rotates about the toe, thus moving away from the backfill, the soil mass expand and results in decrease of the earth pressure.

As the wall starts moving away from the backfill, the backfill has a tendency to move away from the soil mass and tries to move downwards and outwards with respect to the wall.

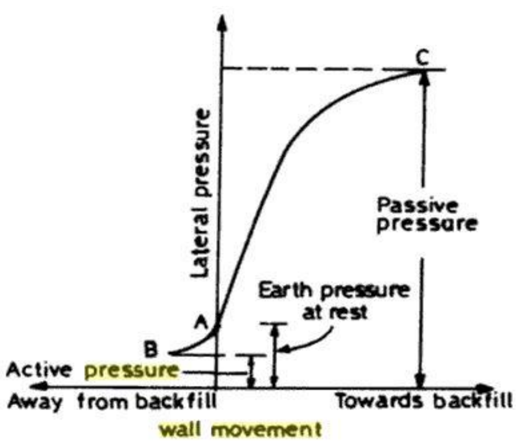


Fig.2.2 Relationship between wall movement and earth pressure

2.2.2At Rest Pressure

At rest lateral earth pressure, represented as K_0, is the in situ lateral pressure. It can be measured directly by a dilatometer test (DMT) or a borehole pressure meter test (PMT).

Two of the more commonly used empirical relations are presented below.

Jaky (1948) for normally consolidated soils:

$$K_{0(NC)} = 1 - \sin \phi'$$

Mayne &Kulhawy (1982) for over-consolidated soils:

$$K_{0(OC)} = K_{0(NC)} * OCR^{(\sin \phi')}$$

The latter requires the OCR profile with depth to be determined. OCR is the over-consolidation ratio and is the effective stress friction angle.

2.2.3Soil Lateral Active Pressure and Passive Resistance

The active state occurs when a retained soil mass is allowed to relax or deform laterally and outward (away from the soil mass) to the point of mobilizing its available full shear resistance (or engaged its shear strength) in trying to resist lateral deformation. That is, the soil is at the point of incipient failure by shearing due to unloading in the lateral direction. It is the minimum theoretical lateral pressure that a given soil mass will exert on a retaining that will move or rotate away from the soil until the soil active state is reached. In order to resist further lateral deformation passive state occurs when a soil mass is externally forced laterally and inward (towards the soil mass) to the point of mobilizing its available full shear resistance.

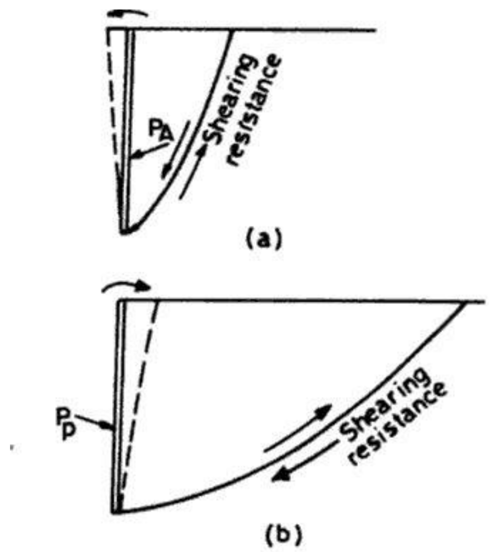


Fig.2.3 Development of active and passive earth pressure

It is the maximum lateral resistance that a given soil mass can offer to a retaining wall that is being pushed towards the soil mass. That is, the soil is at the point of incipient failure by shearing, but this time due to loading in the lateral direction. The minimum lateral pressure and the maximum lateral resistance possible from a given mass of soil is the active pressure and the passive resistance which is being generated respectively.

2.2.4Rankine Theory

Rankine's theory is a stress field solution that predicts active and passive earth pressure. The assumptions made by this theory are that the soil is cohesion-less, frictionless wall, vertical soil-wall interface, the failure surface is planar, and the resultant force is angled parallel to the backfill surface. The equations for active and passive lateral earth pressure coefficients are given below. Note that φ' is the angle of shearing resistance of the soil and the backfill is inclined at angle β to the horizontal.

$$K_a = \cos\beta \frac{\cos\beta - \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}{\cos\beta + \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}$$

$$K_p = \cos\beta \frac{\cos\beta + \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}{\cos\beta - \left(\cos^2\beta - \cos^2\phi\right)^{1/2}}$$

For the case where β is 0, the above equations simplify to

$$K_a = \tan^2\left(45 - \frac{\phi}{2}\right) = \frac{1 - \sin(\phi)}{1 + \sin(\phi)}$$

$$K_p = \tan^2\left(45 + \frac{\phi}{2}\right) = \frac{1 + \sin(\phi)}{1 - \sin(\phi)}$$

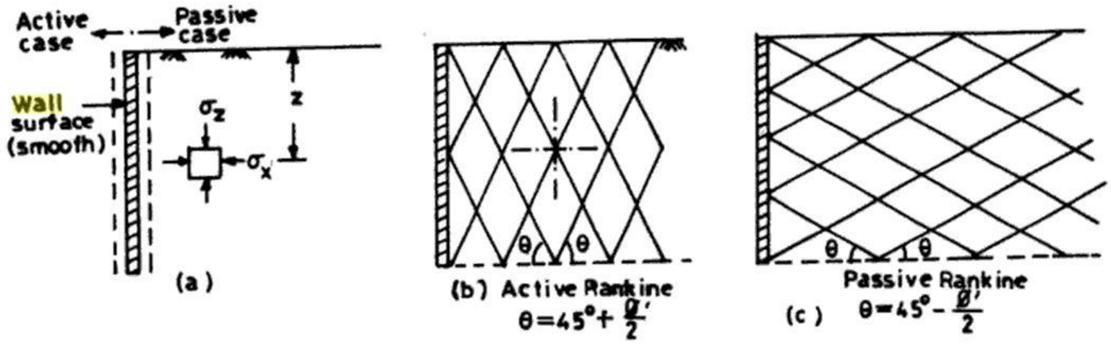


Fig.2.4 Active and passive Rankine states

2.2.5 Coulomb's Theory

Coulomb used limit equilibrium theory, which takes into account the failing soil block as a free body in order to determine the limiting horizontal earth pressure.

The limiting horizontal pressures at failure in extension or compression are used to determine the K_a and K_p respectively. As the problem is indeterminate, a lot of potential failure surfaces must be analyzed to identify the critical failure surface (i.e. the surface that produces the maximum or minimum thrust on the wall). Assumptions that were made by Coulomb are:

1. The backfill is dry, cohesion less, homogeneous, isotropic and ideally plastic material, elastically undeformable but breakable.

2. The slip surface is a plane surface which passes through the heel of the wall.

3. The wall surface is rough. The resultant earth pressure on the wall is inclined at an angle δ to the normal to the wall, where δ is the angle of the friction between the wall and backfill.

4. The sliding wedge itself acts as a rigid body & the value of the earth pressure is obtained by considering the limiting equilibrium of the sliding wedge as a whole.

5. The position and direction of the resultant earth pressure are known. The resultant pressure acts on the back of the wall at one third height of the wall from the base and is inclined at an angle δ to the normal to the back. This angle is called the angle of wall friction.

6. The back of the wall is rough & relative movement of the wall and the soil on the back takes place which develops frictional forces that influence the direction of the resultant pressure.

Angle of wall friction = δ

Angle between soil/wall interface = θ

Angle of internal friction = ϕ

$$K_a = \frac{\cos^2(\phi - \theta)}{\cos^2\theta \cos(\delta + \theta)\left(1 + \sqrt{\frac{\sin(\delta+\phi)\sin(\phi-\beta)}{\cos(\delta+\theta)\cos(\beta-\theta)}}\right)^2}$$

$$K_p = \frac{\cos^2(\phi + \theta)}{\cos^2\theta \cos(\delta - \theta)\left(1 - \sqrt{\frac{\sin(\delta+\phi)\sin(\phi+\beta)}{\cos(\delta-\theta)\cos(\beta-\theta)}}\right)^2}$$

2.3 Paper Description

Dr. D.N.Shinde (2015), "Optimum Static Analysis of Retaining Wall with & without shelf /Shelve at different level"*International Journal of Engineering Research and General Science.*"Retaining wall with pressure relief shelves is one of the special types of retaining wall. High reinforced concrete retaining walls may be used economically by providing relief shelves on the back fill side of wall. Such walls may be termed as the retaining wall with relief shelf. lateral earth pressure on wall and increasing overall stability of the structure. This results in an economical design because less material goes into the wall as compared to massive structure of cantilever or even counterfort retaining walls without the shelves."

Conclusion

The retaining wall having relief shelve came out to be advantageous over the cantilever and counterfort retaining wall.

1. The best location for the single shelf is observed to be in between 0.4 h to 0.5 h for the maximum reduction in earth pressure, less bending moments and less deflection where h is the height of the stem.

2. The deflection of the stem is reduced by about 41.50% by providing shelf at 0.5 h than the deflection given without shelf.

3. The deflection of the stem depends mainly on the shelf location and it increases for the shelf located from 0.2 h to 0.8 h.

M.D. Bolton (1996), "Geotechnical design of retaining wall" ***Institution of Structural Engineers.***"The safe and economic design of a retaining wall depends upon the appropriate mobilization of strength in the adjacent soil. Dense soil tends to be brittle, so that it loses its strength even under strains compatible with the expected displacement of the walls. Loose soil tends to be so compliant that it fails to fully develop its available strength**.** The concept of mobilisable soil strength offers a logical and scientific basis for the design of all geotechnical structures."

Y.S. Au-Yeung (1994), "Gravity retaining wall subjected to seismic loading"

Retaining walls in Hong Kong are not routinely designed against earthquake loading. However, abutment walls of highways and railway bridges are generally designed for a horizontal ground acceleration of 5% g applied at centre of gravity of the structure, with a partial load factor of 1.4. This is equivalent to adopting a design horizontal acceleration of 7% g for ultimate limit state design of such structures.

Suresh Kumar, Dinesh Kumarand& Nancy Mittal, "Use of Flyash in making Controlled Low Strength Material (CLSM) For Use as Self-Compacted Structure Backfill"

"CLSM is a self-compacting, flowable, low-strength cementitious material used primarily as backfill, void fill and utility bedding as an alternative to compacted fill. Several terms are currently used to describe this material, including flowable fill, unshrinkable fill, controlled density fill, flowable mortar, plastic soil cement, soil-cement slurry, K-Krete and other various names.Controlled low-strength materials are defined by "Cement and Concrete Terminology (ACI 116R)" as materials that result in a compressive strength of 8.3Mpa or less. However, most current CLSM applications require unconfined compressive strengths of 2.1Mpa or less. This lower-strength requirement is necessary to allow for future excavation of CLSM. The term CLSM can be used to describe a family of mixtures for a variety of applications."

S. Bali Reddy & A. MuraliKrishna (2015), "Recycled Tire Chips Mixed with Sand as Lightweight Backfill Material in Retaining Wall Applications: An Experimental Investigation"

“Performances of retaining walls under static and seismic loading conditions depend upon the type of backfill soil. Generally, clean granular cohesionless backfill materials are preferred. However, new lightweight fills materials like shredded tire chips, geofoam, fly ash, plastic bottles etc.are being explored as alternative backfill materials now-a-days. These lightweight materials are beneficial in reducing earth pressures and lateral displacements of the retaining walls. Scrap tires are undesired urban waste and are increasing every year. In future, volume of waste tires is going to increase in significant amount in many developing countries. Re-use of scrap tires in civil engineering applications is essential step in creating a sustainable future. In some situations, use of tire derived materials may provide greater economy than those materials traditionally used. Several researchers have evaluated the engineering properties of the scrap tire chips and sand tire chip mixtures by conducting permeability, compressibility, large direct shear tests, and triaxial tests on the samples.”

2.4Problems in retaining wall

Failure of a retaining wall does not necessarily mean total collapse, but rather signs of impending instability and likelihood of a collapse. Total collapses are relatively rare. In a total collapse the wall overturns, slides, topples, or otherwise causes a massive letting loose of the retained earth with resulting damage above and below the wall.

Fortunately, retaining walls are quite forgiving, nearly always displaying telltale signs of trouble and alerting an observer to call for professional help before a collapse. After an evaluation, and determination of the causes, most walls can be saved.

Problems-

1. Reinforcing not in the right position

If the stem shows sign of trouble (excessive deflection and/or cracking) the size, depth, and spacing of reinforcing should be verified. Testing laboratories have the devices (usually a magnetic field measuring Pachometer) which can locate reinforcing and depth with reasonable accuracy, up to about 4 inches’ depth. For exact verification you can first locate the reinforcing then chip out to determine its exact depth and bar size.

2.Saturated backfill

Since retaining walls are generally designed assuming a well-drained granular backfill, if surface drainage is allowed to penetrate and accumulate in the backfill, the pressure against the wall can be doubled. Ponding of water behind the wall not only indicates poor grading, but clayey soil impeding the downward seepage of water.

The surface of the backfill should be graded to direct water away from the wall, or by the use of drainage channels adjacent to the wall to intercept surface water and divert it to disposal. Often surface water problems are attributable to a misdirected or poorly timed irrigation system. Poor backfill material, such as containing clay, can swell and increase wall pressure.

3. Weep holes that don't weep

They become clogged when there is no filtering, such as a line of gravel or crushed rock placed along the base to provide a channel for water to find weep holes, or to be conduced by an embedded perforated pipe.

CHAPTER – 3

MATERIALS AND METHODOLOGY

3.1 Test Materials

After studying many the research papers on how we can decrease the effective stresses on the retaining wall we finalized two structural methods and three backfill replacement methods.

The structural method includes providing a shear key at the base and providing a relief shelve on the stem of the retaining wall and providing soil reinforcement

The backfill replacement materials that we will use are –

Replacing backfill with gravel

Replacing backfill with CLSM

Mixture of soil and shredded tires

3.2Tests Conducted on soil sample

Soil sample was taken from the backfill of retaining wall near Shastri Bhawan.

Test-1: Moisture Content

Sieve used: 2mm

Soil Sample	Weight of Moist Soil (g) (1)	Mass of Container (g) (2)	Mass of Container + Soil(wet) (g) (3)	Mass of Container + Soil (dry)(g) (4)	Moisture Content (%)
1	101.3	20.4	121.7	118.6	2.54
2	102.4	20.4	122.8	119.5	2.68
3	102.6	20.4	123	119.6	2.76

Table 3.1: Moisture Content Readings

Calculation: [{(3)-(4)}/(3)]*100

Result (2.54+2.68+2.76)/3 = 2.66 %

Test-2: Direct Shear Test

Rate of Srain:0.625mm/min

Proving Ring	Horizontal dis. (mm)	Shear strain	Corrected area(cm^2)	Shear stress(N/cm^2)	Normal Stress (N/cm^2)
11	0.1	0.001666667	35.94009983	0.690574042	3.4335
15	0.2	0.003333333	35.88039867	0.94325875	
16	0.3	0.005	35.82089552	1.007814	
17	0.4	0.006666667	35.7615894	1.072578167	
17	0.5	0.008333333	35.70247934	1.074353958	
19	0.6	0.01	35.64356436	1.20273325	
20	0.7	0.011666667	35.58484349	1.268124167	
20	0.8	0.013333333	35.52631579	1.270213333	

Table 3.2 : DST Readings

Proving ring	Horizontal Dis (mm)	Shear strain	Corrected area (cm^2)	Shear stress(N/cm^2)	Normal Stress(N/cm^2)
6	0.1	0.001666667	35.94009983	0.37667675	5.3955
8	0.2	0.003333333	35.88039867	0.503071333	
11	0.3	0.005	35.82089552	0.692872125	
13	0.4	0.006666667	35.7615894	0.820206833	
15	0.5	0.008333333	35.70247934	0.947959375	
17	0.6	0.01	35.64356436	1.07612975	
18	0.7	0.011666667	35.58484349	1.14131175	

19	0.8	0.013333333	35.52631579	1.206702667
20	0.9	0.015	35.4679803	1.2723025
21	1	0.016666667	35.40983607	1.33811125
22	1.1	0.018333333	35.35188216	1.404128917
23	1.2	0.02	35.29411765	1.4703555
24	1.3	0.021666667	35.2365416	1.536791
25	1.4	0.023333333	35.17915309	1.603435417
25	1.5	0.025	35.12195122	1.606046875
26	1.6	0.026666667	35.06493506	1.673004667
27	1.7	0.028333333	35.00810373	1.740171375
27	1.8	0.03	34.95145631	1.74299175
28	1.9	0.031666667	34.89499192	1.810471833
28	2	0.033333333	34.83870968	1.813396667
29	2.1	0.035	34.7826087	1.881190125
29	2.2	0.036666667	34.7266881	1.884219417
29	2.3	0.038333333	34.67094703	1.887248708
29	2.4	0.04	34.61538462	1.890278

Table 3.3: DST Reading

Proving ring	Horizontal Dis (mm)	Shear strain	Corrected area (cm^2)	Shear stress(N/cm^2)	Normal Stress(N/cm^2)
5	0.1	0.001666667	35.94009983	0.313897292	7.3575
7	0.2	0.003333333	35.88039867	0.440187417	
8	0.3	0.005	35.82089552	0.503907	

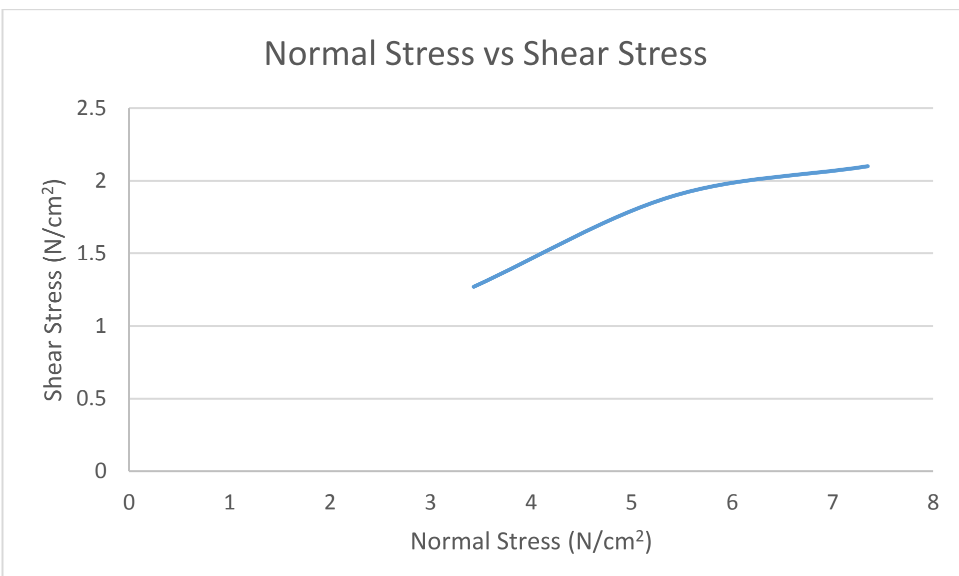
10	0.4	0.006666667	35.7615894	0.630928333
13	0.5	0.008333333	35.70247934	0.821564792
14	0.6	0.01	35.64356436	0.8862245
15	0.7	0.011666667	35.58484349	0.951093125
17	0.8	0.013333333	35.52631579	1.079681333
18	0.9	0.015	35.4679803	1.14507225
18	1	0.016666667	35.40983607	1.1469525
19	1.1	0.018333333	35.35188216	1.212656792
20	1.2	0.02	35.29411765	1.27857
20	1.3	0.021666667	35.2365416	1.280659167
20	1.4	0.023333333	35.17915309	1.282748333
21	1.5	0.025	35.12195122	1.349079375
21	1.6	0.026666667	35.06493506	1.351273
22	1.7	0.028333333	35.00810373	1.417917417
24	1.8	0.03	34.95145631	1.549326
25	1.9	0.031666667	34.89499192	1.616492708
26	2	0.033333333	34.83870968	1.683868333
27	2.1	0.035	34.7826087	1.751452875
29	2.2	0.036666667	34.7266881	1.884219417
29	2.3	0.038333333	34.67094703	1.887248708
30	2.4	0.04	34.61538462	1.95546
30	2.5	0.041666667	34.56	1.95859375
31	2.6	0.043333333	34.50479233	2.027118417
31	2.7	0.045	34.44976077	2.030356625

32	2.8	0.046666667	34.39490446	2.099194667
32	2.9	0.048333333	34.34022258	2.102537333
32	3	0.05	34.28571429	2.10588

Table 3.4: DST Readings

Shear Stress(N/cm^2)	Normal Stress(N/cm^2)	Intercept	Slope	C-value	Φ-value(in degrees)
1.27	3.43	0.612083333	0.211734694	0.612083333	12
1.89	5.39				
2.1	7.35				

Table 3.5:C and Φ Values



Graph3.1: Normal Stress v/s Shear Stress

The graph is being plotted between shear stress and normal stress and the values of C and Φ are calculated.

Test-3: Calculation of Specific Gravity using Density Bottle

Volume of Density Bottle: 50ml

Weight of Empty Density Bottle (w1): 28.3g

Sample Soil	Weight of Empty Density Bottle (w1) (g)	Weight of Density Bottle + Water (w2)(g)	Weight of Density Bottle + Dry Soil (w3) (g)	Weight of Density Bottle + Soil + Water (w4) (g)	Specific Gravity
1	28.3	79	55.2	92.7	2.037
2	28.3	79	56	92.9	2.007
3	28.3	79	55.5	92.8	2.029

Table 3.6: Density Bottle Readings

Calculation: (w3-w1)/[(w2-w1) -(w4-w3)]

Result:(2.037+2.007+2.029)/3 = 2.024

Test-4: Particle Size Distribution

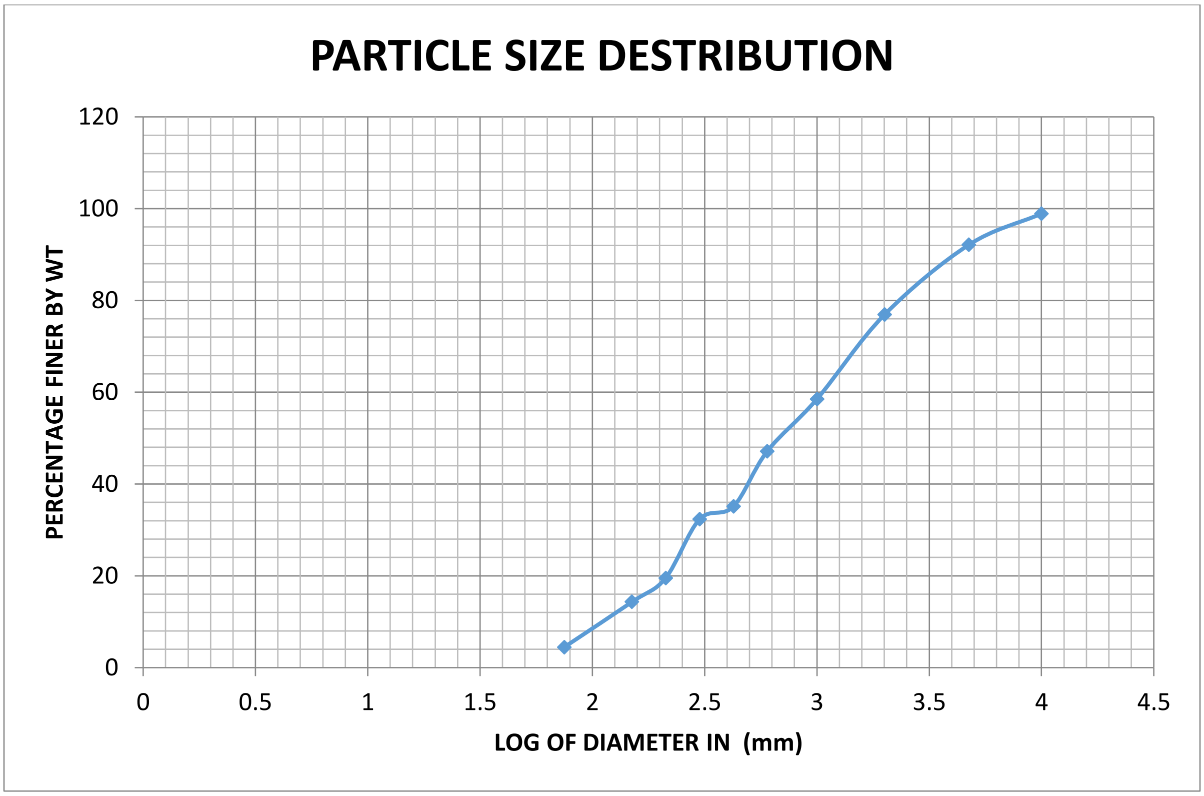
Weight of Sample – 500g

Fig.3.1 Particle Size Distribution

Sieve Size (Microns)	Weight Retained (gm)	Percentage Retained	Cumulative% Retained	Percentage Finer Than	Log of Sieve Size (mm)
10000	5.5	1.1	1.1	98.9	4
4750	34	6.8	7.9	92.1	3.676
2000	76	15.2	23.1	76.9	3.301
1000	92	18.4	41.5	58.5	3
600	57	11.4	52.9	47.1	2.778
425	60	12	64.9	35.1	2.628
300	14	2.8	67.7	32.3	2.477
212	64	12.8	80.5	19.5	2.326
150	26	5.2	85.7	14.3	2.176
75	48.5	9.7	95.54	4.46	1.875

PAN	20	4	99.4	0.6	0
TOTAL	497	99.4			
ERROR	3	0.6			

Table 3.7 Particle Size Distribution Readings



Graph 3.2 Particle Size Distribution Curve

3.3Tests Conducted On CLSM

Cement used was **Jaypee cement** provided by the university. The cement was **PPC (fly ash based)** conforming to Part 1 of IS 1489:1991.

Test-1: Flow Table

Weight of Tray: 1.52kg

Weight of Cement: 500g

Weight of Sand: 6kg

Weight of Water: 2litres

Flow table dimensions

Readings: -

Notation	Dia. (cm)
D1	12
D2	11
D3	11
D4	11.2
D5	10
D6	12.5
D7	12

Table 3.8: Flow Table Readings

Fig.3.2 Flow Table test

Result**:** Since the flow table reading is less than 15cm we can conclude that the CLSM is of low flowability.

Test-2: Slump Cone Test

Dimensions of slump cone

Weight of Cement: 700g

Weight of Sand: 8kg

Weight of Water: 2.4L

Initial Height: 25.5cm

Final Height: 19.5 cm

Result -The slump cone results are within the range specified for CLSM.

Test-3: pH Test

Weight of Cement: 20g

Weight of Sand: 250g

Water: 100ml

Result:pH came out to be 11.5

3.4 Test conducted on soil mixed with shredded rubber tyres

Test-1 Direct Shear Test

Sample 1 – 2.27% of soil is replaced by shredded rubber tyres.

Strain Rate- 1.25mm/min

Proving Ring	Horizontal Dis.(mm)	Shear Strain	Corrected Area(cm^2)	Shear Stress(N/cm^2)	Normal Stress(N/cm^2)
7.5	0.1	0.001666667	35.94009983	0.470845938	5.3955
10	0.2	0.003333333	35.88039867	0.628839167	
11	0.3	0.005	35.82089552	0.692872125	
12.5	0.4	0.006666667	35.7615894	0.788660417	
15	0.5	0.008333333	35.70247934	0.947959375	
16	0.6	0.01	35.64356436	1.012828	
16	0.7	0.011666667	35.58484349	1.014499333	
17.5	0.8	0.013333333	35.52631579	1.111436667	
17.5	0.9	0.015	35.4679803	1.113264688	
20	1	0.016666667	35.40983607	1.274391667	
21	1.1	0.018333333	35.35188216	1.340304875	
22.5	1.2	0.02	35.29411765	1.43839125	
24	1.3	0.021666667	35.2365416	1.536791	
24	1.4	0.023333333	35.17915309	1.539298	
25	1.5	0.025	35.12195122	1.606046875	
25	1.6	0.026666667	35.06493506	1.608658333	

25	1.7	0.028333333	35.00810373	1.611269792
25.5	1.8	0.03	34.95145631	1.646158875
27.5	1.9	0.031666667	34.89499192	1.778141979
30	2	0.033333333	34.83870968	1.942925
30	2.1	0.035	34.7826087	1.94605875

Table 3.9 DST Readings

Proving Ring	Horizontal Dis.(mm)	Shear Strain	Corrected Area(cm^2)	Shear Stress (N/cm^2)	Normal Stress (N/cm^2)
12.5	0.1	0.001666667	35.94009983	0.784743229	9.3195
20	0.2	0.003333333	35.88039867	1.257678333	
25	0.3	0.005	35.82089552	1.574709375	
30	0.4	0.006666667	35.7615894	1.892785	
31	0.5	0.008333333	35.70247934	1.959116042	
34	0.6	0.01	35.64356436	2.1522595	
35	0.7	0.011666667	35.58484349	2.219217292	
36	0.8	0.013333333	35.52631579	2.286384	
37.5	0.9	0.015	35.4679803	2.385567188	
40	1	0.016666667	35.40983607	2.548783333	
41	1.1	0.018333333	35.35188216	2.616785708	
41	1.2	0.02	35.29411765	2.6210685	

42.5	1.3	0.021666667	35.2365416	2.721400729
45	1.4	0.023333333	35.17915309	2.88618375
47.5	1.5	0.025	35.12195122	3.051489063
47.5	1.6	0.026666667	35.06493506	3.056450833
47.5	1.7	0.028333333	35.00810373	3.061412604
50	1.8	0.03	34.95145631	3.2277625
50	1.9	0.031666667	34.89499192	3.232985417

Table 3.10 DST Readings

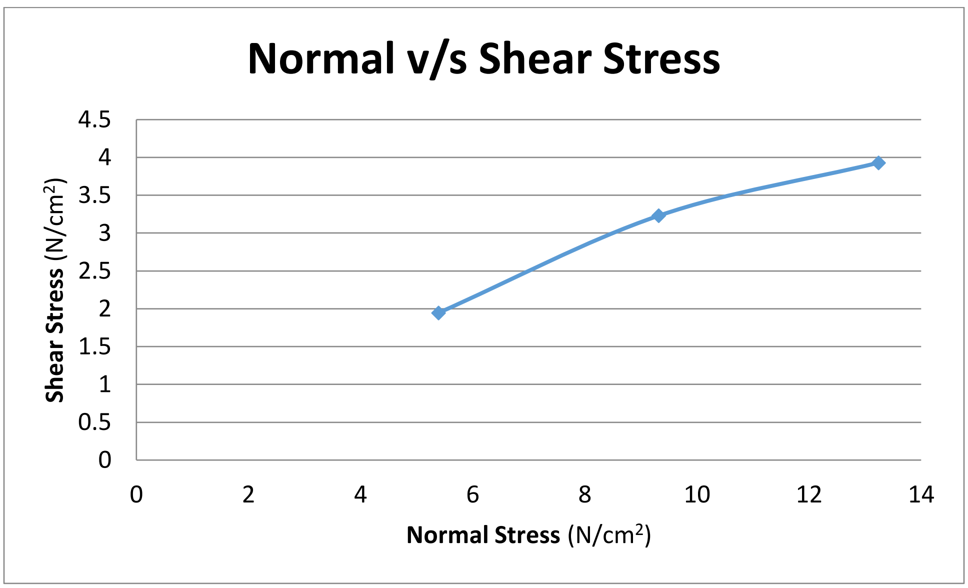
Proving ring	Horizontal dis.(mm)	Shear Strain	Corrected Area(cm^2)	Shear Stress (N/cm^2)	Normal Stress (N/cm^2)
2.5	0.1	0.001666667	35.94009983	0.156948646	13.2435
10	0.2	0.003333333	35.88039867	0.628839167	
17.5	0.3	0.005	35.82089552	1.102296563	
25	0.4	0.006666667	35.7615894	1.577320833	
35	0.5	0.008333333	35.70247934	2.211905208	
39.5	0.6	0.01	35.64356436	2.500419125	
42.5	0.7	0.011666667	35.58484349	2.694763854	
46	0.8	0.013333333	35.52631579	2.921490667	
50	0.9	0.015	35.4679803	3.18075625	

50	1	0.016666667	35.40983607	3.185979167
52.5	1.1	0.018333333	35.35188216	3.350762188
54.5	1.2	0.02	35.29411765	3.48410325
57.5	1.3	0.021666667	35.2365416	3.681895104
57.5	1.4	0.023333333	35.17915309	3.687901458
60	1.5	0.025	35.12195122	3.8545125
61	1.6	0.026666667	35.06493506	3.925126333
61	1.7	0.028333333	35.00810373	3.931498292

Table 3.11 DST Readings

Shear Stress(N/cm^2)	Normal Stress(N/cm^2)	Intercept	Slope	C-value	Φ-value(in degrees)
1.946	5.39	0.68042796	0.252767174	0.68042796	14.18
3.23	9.3195				
3.93	13.24				

Table 3.12 C and Φ values



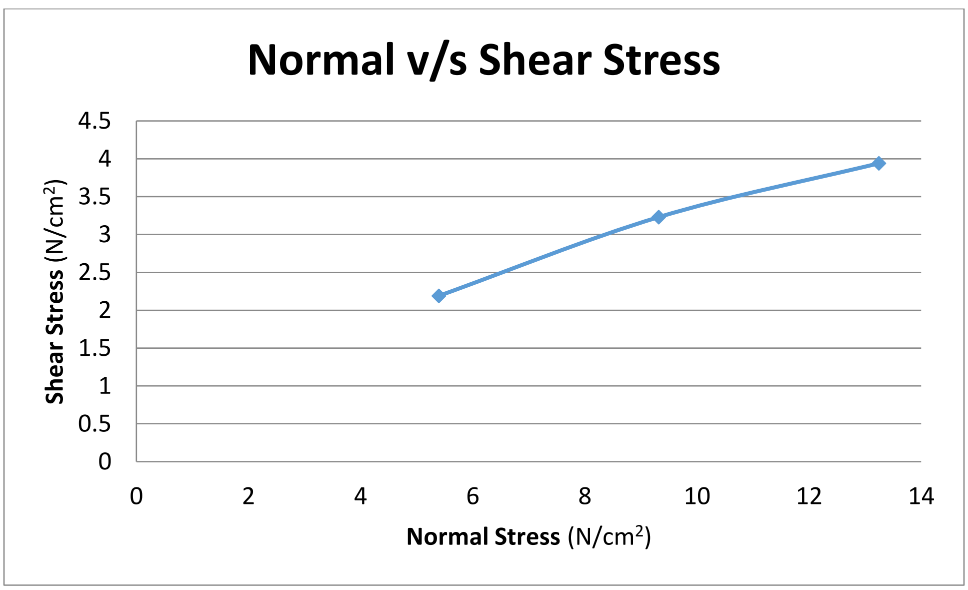
Graph 3.3: Normal Stress v/s Shear Stress

Sample 2 –4.65% of soil is replaced by shredded rubber tyres.

Strain Rate- 1.25mm/min

Shear Stress(N/cm^2)	Normal Stress(N/cm^2)	Intercept	Slope	C-value	Φ-value(in degrees)
2.19	5.3955	1.041875	0.222986748	1.041875	12.57
3.23	9.3195				
3.94	13.2435				

Table 3.13 C and Φ values



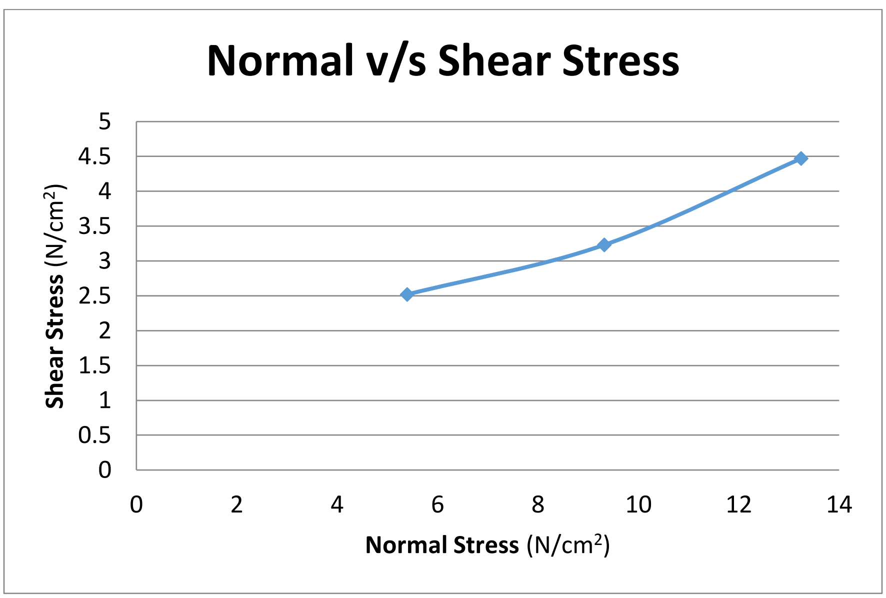
Graph 3.4: Normal Stress v/s Shear Stress

Sample 3–6.77% of soil is replaced by shredded rubber tyres.

Strain Rate- 1.25mm/min

Shear Stress(N/cm^2)	Normal Stress(N/cm^2)	Intercept	Slope	C-value	Φ-value(in degrees)
2.52	5.3955	1.091041667	0.248470948	1.091041667	13.95
3.23	9.3195				
4.47	13.2435				

Table 3.14 C and Φ values



Graph 3.5: Normal Stress v/s Shear Stress

CHAPTER - 4

DESIGNING

4.1 Structural Design Solutions

4.1.1 Drainage Solutions

1.When the backfill of the retaining wall has a soil which has very low permeability (clay) then the water which would be collected in the backfill would not be able to drain and it would create excess surcharge which would finally results in failure of the retaining wall.

To allow the drainage through the backfill we can replace some amount of backfill by gravel. The gravel will help the standing water to drain and reduce the effective stress.

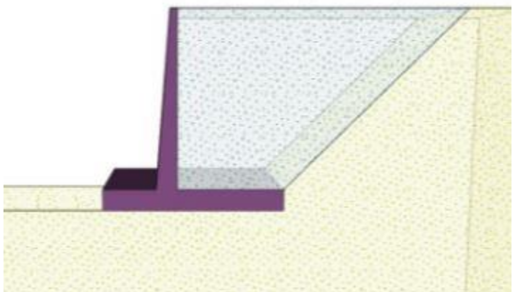


Fig.4.1 Gravel in the backfill at an angle of 45 degree

4.1.2 Structural Solutions

- **Providing a relief shelve**

Retaining wall with pressure relief shelves is one of the special types of retaining wall. High reinforced concrete retaining walls may be used economically by providing relief shelves on the back fill side of wall. They reduce the lateral earth pressure on wall and increasing overall stability of the structure. Due to this the design of the wall becomes very economical as the material used in the wall is very less as compared to huge structures of cantilever or even counterfort retaining walls without the shelves.

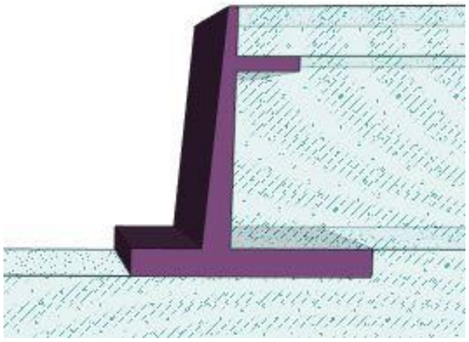


Fig.4.2 Retaining wall with shelve

- **Providing a shear key**

The reason for installing shear keys is to increase the extra passive resistance developed by the height of shear keys. However, active pressure developed by shear keys also increases simultaneously. There is a net improvement of sliding resistance because of the fact that the increase of passive pressure exceeds the increase in active pressure.

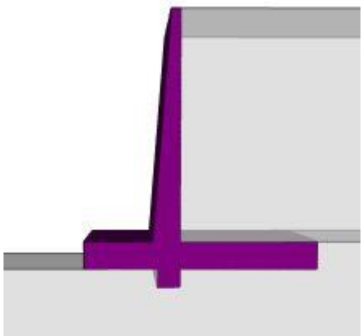


Fig.4.3 Retaining wall with shear key

4.2 Design

All the designing is done by using Geotechnical software GEO 5

4.2.1 Simple Cantilever

Analysis of Cantilever Wall

Data Input

Project

Date-5-12-2016

Settings

India- Standard

Material and standards

Concrete structures-IS:456

Wall analysis

Earth Pressure (active) calculation : Coulomb

Earth Pressure (passive)calculation :Caquot-Kerisel

Earthquake Analysis :Mononobe-Okabe

Shape of Earth wedge : calculated as skew

Base Key : the base key is considered as inclined footing bottom

Allowable eccentricity : 0.333

Factor of safety for overturning - 1.5

Factor of safety for sliding resistance - 1.5

Factor of safety for bearing capacity - 1.5

Material of structure

Unit Weight γ = 25 KN/m^3

Analysis of concrete structures are carried out according to the standard IS 456

Concrete – M20

Compressive Strength f_{ck} = 20 MPa

Tensile Bending Strength σ = 6.67 MPa

Longitudnal Steel f_{yk} =415 MPa

No.	Coordinate X (m)	Depth Z (m)
1	0.00	0.00
2	0.00	6.20
3	2.00	6.20
4	2.00	7.00
5	-1.90	7.00
6	-1.90	6.20
7	-.90	6.20
8	-.40	0.00

Table 4.1: Geometry of structure

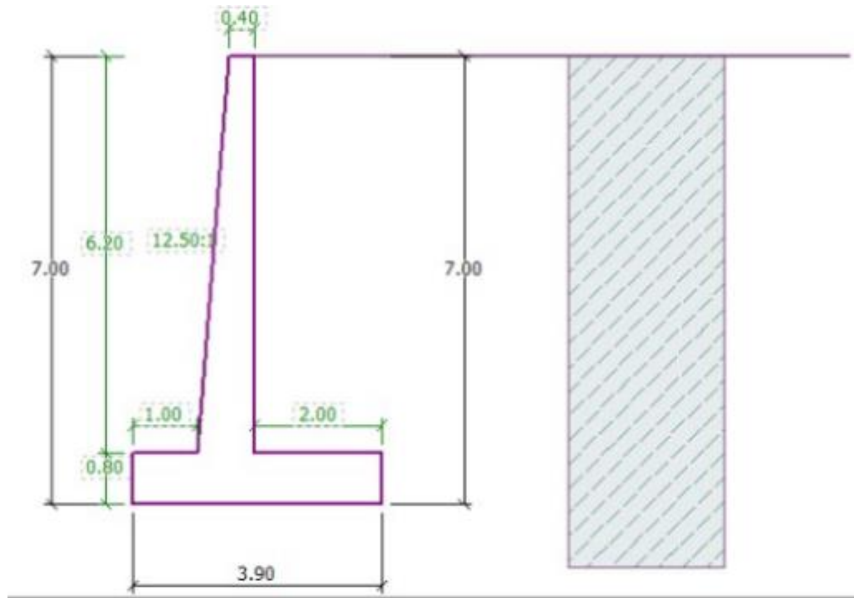


Fig.4.4 Dimensions(all in meters)

No.	Name	Pattern	Φ_{ef} [°]	C_{ef} [KPa]	γ [kN/m3]	γ_{su} [kN/m3]	δ [°]
1	Soil No.1		30.00	0.00	20.00	10.00	0.00

Table 4.2: Basic Soil Parameters

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Soil No. 1

Unit weight: γ= 20.00 KN/m^3
Stress-state: effective
Angle of internal friction: φ_{ef}= 30.00°
Cohesion of soil: c_{ef} = 0.00
Angle of friction struc.-soil δ = 0.00

Soil: cohesionless

Saturated unit weight γ_{sat}= 20.00 kN/m^3

Backfill

Soil on the front face of the structure- Soil No.1

Foundation

Foundation Type: soil from geological profile

Terrain profile

Terrain behind the structure is flat.

Water influence

Ground water table is located below the structure.

No.	Surcharge	Action	Mag.1 [KN/m^2]	Mag.2 [KN/m^2]	Ord x [m]	Length l [m]	Depth z [m]
1	No						On terrain

Table 4.3: Input surface surcharges

Resistance on front face of the structure

Resistance on front face of the structure is not considered.

Settings of the stage of construction

Design situation: permanent
The wall is free to move. Active earth pressure is therefore assumed.

Verification No. 1

Name	F_{hor}[KN/m]	App.Pt. Z [m]	F_{vert} [KN/m]	App. Pt. x [m]	Design coefficient
Weight-wall	0.00	-2.15	178.36	1.73	1.00
Weight-earth wedge	0.00	-1.95	69.28	2.56	1.00
Active Pressure	163.33	-2.33	178.71	3.03	1.00

Table 4.4: Forces acting on construction

Verification of complete wall

Overturning stability check

Resisting moment M_{res} = 1026.27 kNm/m

Overturning moment M_{ovr}= 381.10 kNm/m

Factor of safety = 2.69 > 1.50

Wall for overturning is SATISFACTORY

Slip Check

Resistive horizontal force Hres = 246.15 kN/m

Active horizontal force Hact =163.33 kN/m

Factor of safety = 1.51 > 1.50

Wall for slip is SATISFACTORY

Overall check - WALL is SATISFACTORY

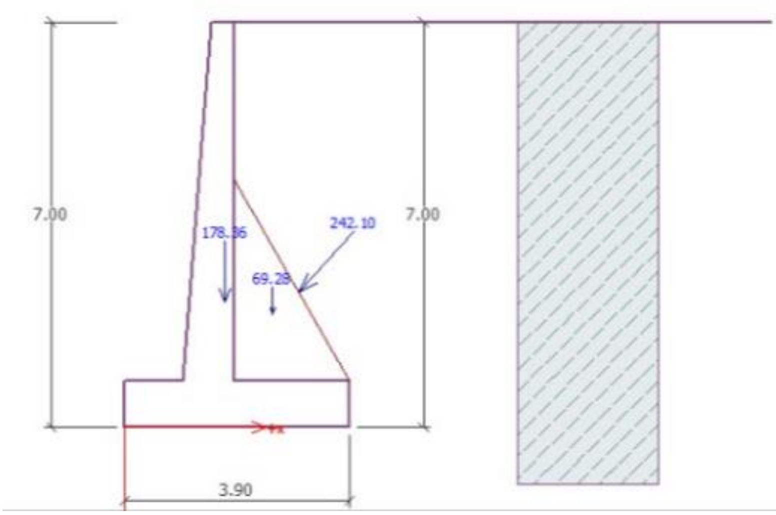


Fig.4.5 Forces acting on retaining wall

Bearing capacity of foundation soil

No.	Moment [kNm/m]	Norm. force [kN/m]	Shear Force [kN/m]	Eccentricity [–]	Stress [kPa]
1	185.36	426.35	163.33	.112	140

Table 4.5: Design load acting at the center of footing bottom

No.	Moment [kNm/m]	Norm. force [kN/m]	Shear Force [kN/m]
1	185.36	426.35	163.33

Table 4.6: Service load acting at the center of footing bottom

Foundation soil verification

Eccentricity verification

Max. eccentricity of normal force e = .112

Maximum allowable eccentricity e_{alw} = .333

Eccentricity of the normal force is SATISFACTORY

Verification of bearing capacity

Max. stress at footing bottom σ = 140.87 kPa

Bearing capacity of foundation soil R_d = 200.00 kPa

Factor of safety = 1.42 > 1.50

Bearing capacity of foundation soil is NOT SATISFACTORY

Overall verification - bearing capacity of found. soil is NOT SATISFACTORY

Dimensioning No. 1

Name	F_{hor} [kN/m]	App.Pt. z [m]	F_{vert} [kN/m]	App.Pt. x [m]	Design coefficient
Weight – wall	0.00	-2.15	178.36	1.73	1.00
Weight – earth wedge	0.00	-1.95	69.28	2.56	1.00
Active Pressure	163.33	-2.33	178.71	3.03	1.00

Table 4.7: Forces acting on construction

Front Wall jump check

Reinforcement and dimensions of the cross-section

Bar diameter = 20.0 mm
Number of bars = 5
Reinforcement cover = 30.0 mm
Cross-section width = 1.00 m
Cross-section depth = 0.80 m

Reinforcement ratio = 0.21 % > 0.20 %

Position of neutral axis = 0.08 m <   0.36 m
Ultimate shear force   = 260.28   > 146.90
Ultimate moment   = 411.92   > 75.058 kNm

Cross-section is SATISFACTORY

Dimensioning No. 2

Name	F_{hor} [kN/m]	App.Pt. z [m]	F_{vert} [kN/m]	App.Pt. x [m]	Design coefficient
Weight – wall	0.00	-2.70	100.41	.56	1.00
Pressure at rest	192.12	-2.07	.00	.90	1.00

Table 4.8: Forces acting on construction

Wall stem check

Reinforcement and dimensions of the cross-section

Bar diameter   = 20.0 mm
Number of bars   = 5
Reinforcement cover = 30.0 mm
Cross-section width   = 1.00 m
Cross-section depth   = 0.90 m

Reinforcement ratio   = 0.18 % <   0.20 %

Ultimate shear force   = 274.88 kN> 192.12kN

Cross-section is NOT SATISFACTORY, increase reinforcement area

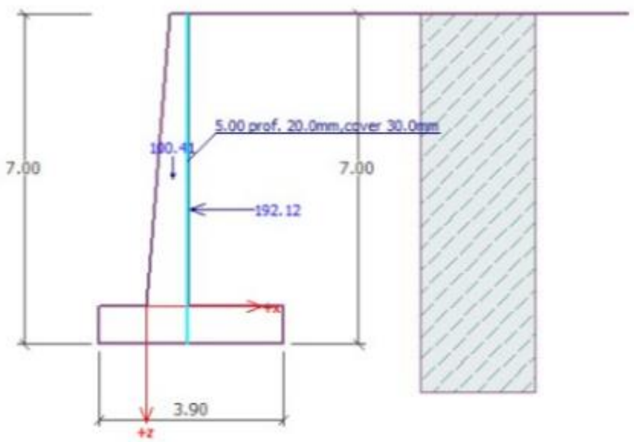


Fig.4.6 Wall stem check

Dimensioning No. 3

Name	F_{hor} [kN/m]	App.Pt. z [m]	F_{vert} [kN/m]	App.Pt. x [m]	Design coefficient
Weight – wall	0.00	-.40	40.00	2.90	1.00
Weight – earth wedge	0.00	-1.95	69.28	2.56	1.00
Active Pressure	163.33	-2.33	178.71	3.03	1.00
Contact stress	0.00	0.00	-147.55	2.73	1.00

Table 4.9: Forces acting on construction

Back Wall jump check

Reinforcement and dimensions of the cross-section

Bar diameter = 20.0 mm
Number of bars = 5
Reinforcement cover = 30.0 mm
Cross-section width = 1.00 m
Cross-section depth = 0.80 m

Reinforcement ratio = 0.21 % > 0.20 %
Position of neutral axis = 0.08 m < 0.36 m
Ultimate shear force = 260.28 kN> 140.40 kN

Ultimate moment = 411.92 kN/m > 165.51 kNm

Cross-section is SATISFACTORY.

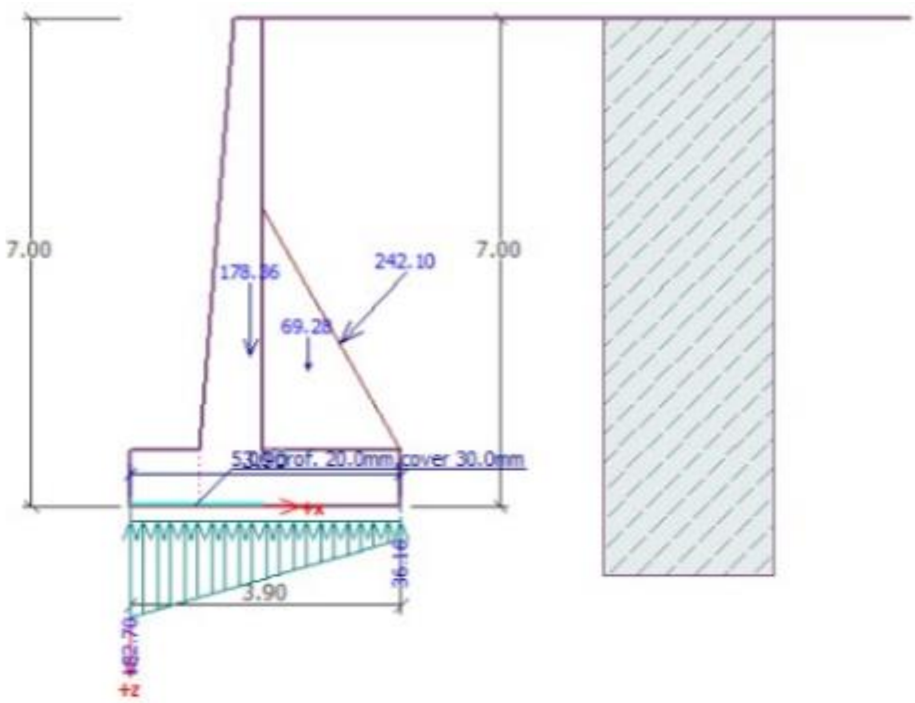


Fig.4.7Wall jump check

Slope stability analysis

Input data

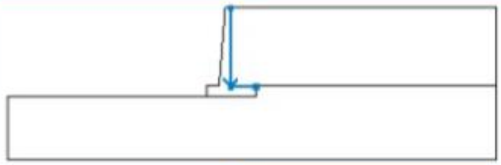
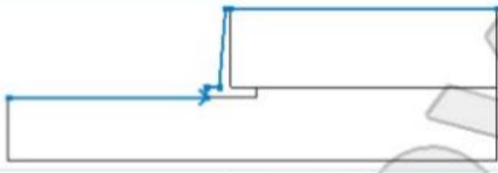
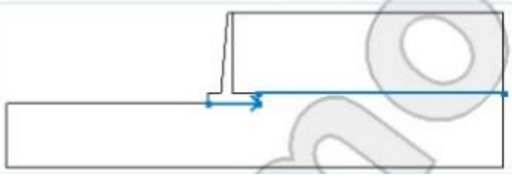
Project

Settings

India – Standard

Stability analysis

Earthquake analysis: Standard
Verification methodology:Factor of safetys (ASD)

No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		0.00	0.00	0.00	-6.20	2.00	-6.20
2		-17.50	-7.00	-1.90	-7.00	-1.90	-6.20
		-0.90	-6.20	-0.40	0.00	0.00	0.00
		21.00	0.00				
3		-1.90	-7.00	2.00	-7.00	2.00	-6.20
		21.00	-6.20				

Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
1	Soil No.1		30.00	0.00	20.00

Soil parameters - uplift

No.	Name	Pattern	γ_{sat} [kN/m^3]	γ_s [kN/m^3]	n [–]
1	Soil No.1		20.00		

Water

Water type: No water

Tensile crack

Tensile crack not inputted.

Earthquake

Earthquake not included.

Settings of the stage of construction

Design situation: permanent

Results

Slip surface analysis

Slip surface parameters							
Center :	x =	-2.23	[m]	Angles :	α_1 =	-29.40	[°]
	z =	0.44	[m]		α_2 =	87.05	[°]
Radius :	R =	8.54	[m]				
The slip surface after optimization.							

Slope stability verification (Morgenstern-Price)

Factor of Safety = 1.29 < 1.5

4.2.2 Simple Cantilever with Gravel as backfill

Input Data

Project

Date-5-12-2016

Settings

India- Standard

Material and standards

Concrete structures-IS:456

Wall analysis

Earth Pressure (Active) calculation : Coulomb

Earth Pressure (Passive)calculation :Caquot-Kerisel

Earthquake Analysis :Mononobe-Okabe

Shape of Earth wedge : calculated as skew

Base Key : the base key is considered as inclined footing bottom

Allowable eccentricity : 0.333

Factor of safety for overturning - 1.5

Factor of safety for sliding resistance - 1.5

Factor of safety for bearing capacity - 1.5

Material of structure

Unit Weight γ = 25 KN/m^3

Analysis of concrete structures are carried out according to the standard IS 456

Concrete – M20

Compressive Strength f_{ck} = 20 MPa

Tensile Bending Strength σ = 3.13 MPa

Longitudinal Steel f_{yk} =415 MPa

No.	Coordinate X (m)	Depth Z (m)
1	0.00	0.00
2	0.00	5.00
3	3.00	5.00
4	3.00	5.60
5	-2.20	5.60
6	-2.20	5.00
7	-0.70	5.00
8	-0.30	0.00

Table 4.10: Geometry of structure

The origin [0,0] is located at the most upper right point of the wall.

Wall section area = 5.62 m^2

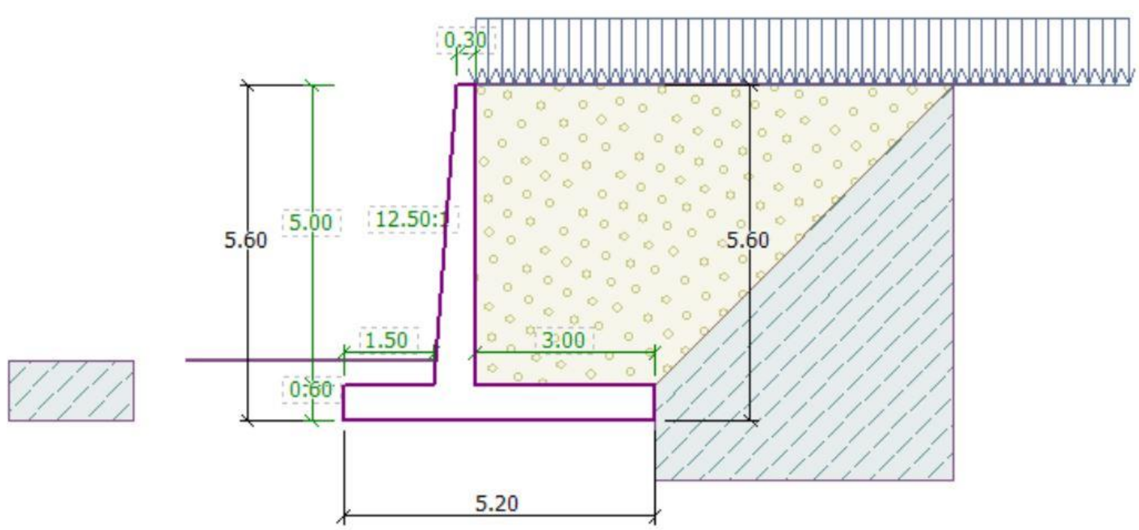


Fig.4.8 Retaining wall dimensions (in meters)

No.	Name	Pattern	Φ_{ef} [°]	C_{ef} [KPa]	γ [kN/m3]	γ_{su} [kN/m3]	δ [°]
1	Soil No.1		12.00	6.10	19.62	10.00	0.00
2	Well graded gravel (GW), medium dense		38.50	0.00	21.00	11.00	0.00

Table 4.11: Basic Soil Parameters

All soils are considered as cohesionless for at rest pressure analysis.

Soil parameters

Soil No. 1

Unit weight: γ= 19.62 KN/m^3
Stress-state: effective
Angle of internal friction: φ_{ef}= 12.00°
Cohesion of soil: c_{ef} = 6.10 kPa
Angle of friction struc.-soil δ = 0.00

Soil: cohesionless
Saturated unit weight γ_{sat}= 20.00 kN/m^3

Well graded gravel (GW), medium dense

Unit weight: γ= 21.00 KN/m^3
Stress-state: effective
Angle of internal friction: φ_{ef}= 38.50°
Cohesion of soil: c_{ef} = 0.00 kPa
Angle of friction struc.-soil δ = 0.00

Soil: cohesionless
Saturated unit weight γ_{sat}= 21.00 kN/m^3

Backfill

Soil on the front face of the structure- Well graded gravel (GW), medium dense

Foundation

Type of foundation: soil from geological profile
Terrain profile

Terrain behind the structure is flat.

Water influence

Ground water table is located below the structure.

No.	Surcharge	Action	Mag.1 [KN/m^2]	Mag.2 [KN/m^2]	Ord x [m]	Length l [m]	Depth z [m]
1	Yes	permanent	10.00				On terrain

Table 4.12: Input surface surcharges

Resistance on front face of the structure

Resistance on front face of the structure: not considered.

Soil on front face of the Structure-Soil No.1

Soil thickness in front of the structure h = 1.00 m

Settings of the stage of construction

Design situation: permanent
The wall is free to move. Active earth pressure is therefore assumed.

Verification No. 1

Layer	Thickness[m]	A	φ_d	c_d	γ	δ_d	K_a

No.1							
1	1.00	25.75	38.5	0.00	21.00	38.50	0.535
2	4.00	25.75	38.5	0.00	21.00	38.50	0.535
3	0.60	0.00	38.50	0.00	21.00	0.00	0.233

Table 4.13: Active pressure behind the structure – partial results

Layer No.	Start[m] End[m]	σ_z[kPa]	σ_w[kPa]	Pressure [kPa]	Hor. Comp. [kPa]	Vert. comp.[kPa]
1	0.00	0.00	0.00	0.00	0.00	0.00
	1.00	21.00	0.00	11.24	4.89	10.13
2	1.00	21.00	0.00	11.24	4.89	10.13
	5.00	105.00	0.00	56.22	24.43	50.64
3	5.00	105.00	0.00	24.43	24.43	0.00
	5.60	117.60	0.00	27.36	27.36	0.00

Table 4.14: Active pressure distribution behind the structure (without surcharge)

Point No.	Depth [m]	Hor. Comp. [kPa]	Vert. comp. [kPa]
1	0.00	2.33	4.82
2	1.00	2.33	4.82
3	5.00	2.33	4.82
4	5.00	2.33	0.00

5	5.60	2.33	0.00

Table 4.15: Pressure profile due to surcharge

Name	F_{hor}[KN/m]	App.Pt. Z [m]	F_{vert} [KN/m]	App. Pt. x [m]	Design coefficient
Weight-wall	0.00	-1.40	140.50	2.30	1.00
Weight-earth wedge	0.00	-2.54	188.39	3.23	1.00
Active Pressure	76.60	-1.87	126.60	4.40	1.00
Surcharge	13.03	-2.80	24.11	3.99	1.00
Surcharge	0.00	-5.60	5.88	2.49	1.00

Table 4.16: Forces acting on construction

Verification of complete wall

Check for overturning stability

Resisting moment M_{res} = 1026.27 kNm/m

Overturning moment M_{ovr}= 179.47 kNm/m

Factor of safety = 8.92 > 1.50

Wall for overturning is SATISFACTORY

Check for slip

Resisting horizontal force Hres = 134.91 kN/m

Active horizontal force Hact =89.63 kN/m

Factor of safety = 1.51 > 1.50

Wall for slip is SATISFACTORY

Overall check - WALL is SATISFACTORY

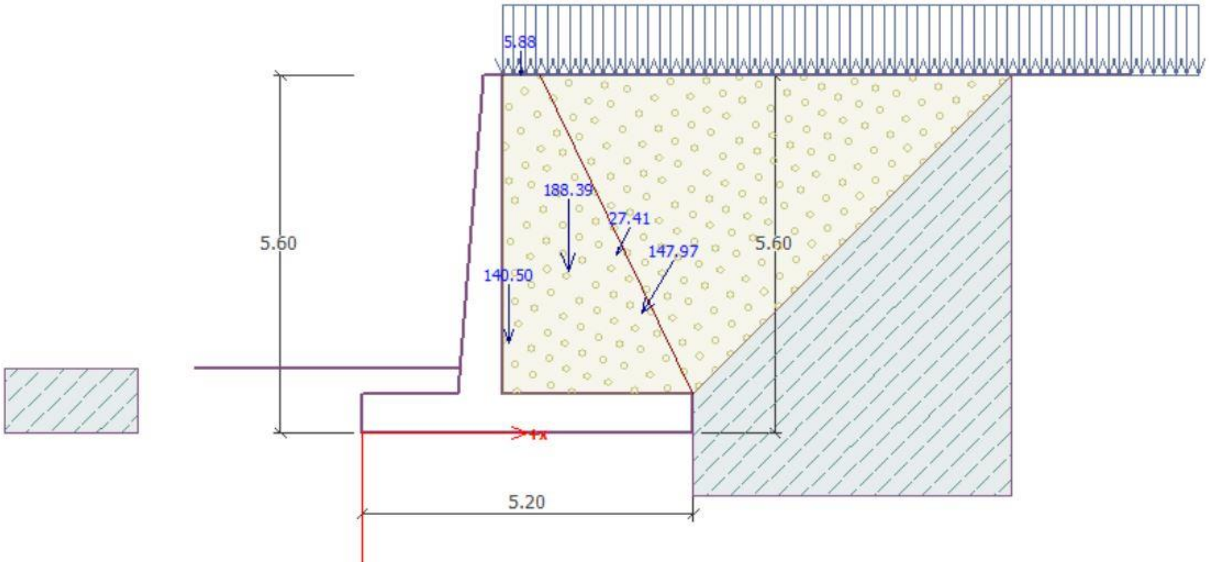


Fig.4.9 Verification

Bearing capacity of foundation soil

No.	M0oment [kNm/m]	Norm. force [kN/m]	Shear Force [kN/m]	Eccentricity [–]	Stress [kPa]
1	-158.55	485.49	89.63	0.00	93.36

Table 4.17: Design load acting at the center of footing bottom

No.	Moment [kNm/m]	Norm. force [kN/m]	Shear Force [kN/m]
1	-158.55	485.49	89.63

Table 4.18: Service load acting at the center of footing bottom

Verification of foundation soil

Eccentricity verification

Max. eccentricity of normal force e = 0.00

Maximum allowable eccentricity e_{alw} = .333

Eccentricity of the normal force is SATISFACTORY

Verification of bearing capacity

Max. stress at footing bottom σ = 93.36 kPa

Bearing capacity of foundation soil R_d = 200.00 kPa

Factor of safety = 2.14 > 1.50

Bearing capacity of foundation soil isSATISFACTORY

Overall verification - bearing capacity of found. soil is SATISFACTORY

Dimensioning No. 1

Layer No.	Thickness[m]	α [°]	φ_d[°]	c_d [kPa]	γ [kN/m^3]	K_r
1	1.00	0.00	38.50	0.00	21.00	0.377
2	4.0	0.00	38.50	0.00	21.00	0.377

Table 4.19: Pressure at rest behind the structure- partial results

Layer No.	Start[m]	σ_z[kPa]	σ_w[kPa]	Pressure	Hor. Comp.	Vert.

	End[m]			[kPa]	[kPa]	comp.[kPa]
1	0.00	0.00	0.00	0.00	0.00	0.00
	1.00	21.00	0.00	7.93	7.93	0.00
2	1.00	21.00	0.00	7.93	7.93	0.00
	5.00	104.97	0.00	39.63	39.63	0.00

Table 4.20: Pressure at rest distribution behind the structure (without surcharge)

Point No.	Depth [m]	Horz. Comp.[kPa]	Vert. Comp. [kPa]
1	0.00	3.77	0.00
2	1.00	3.77	0.00
3	5.00	3.77	0.00

Table 4.21: Pressure profile due to surcharge

Name	F_{hor} [kN/m]	App.Pt. z [m]	F_{vert} [kN/m]	App.Pt. x [m]	Design coefficient
Weight - wall	0.00	-2.17	62.48	0.44	1.00
Pressure at rest	99.04	-1.67	0.00	0.70	1.00
Surcharge	18.87	-2.50	0.00	0.70	1.00

Table 4.22: Forces acting on construction

Wall stem check

Reinforcement and dimensions of the cross-section

Bar diameter = 20.0 mm
Number of bars = 6
Reinforcement cover = 30.0 mm
Cross-section width = 1.00 m
Cross-section depth = 0.70 m

Reinforcement ratio = 0.29 % > 0.20 %

Position of neutral axis = 0.10 m <   0.32 m
Ultimate shear force   = 263.94 kN> 117.91 kN
Ultimate moment   = 421.69 kNm> 206.78 kNm

Cross-section is SATISFACTORY

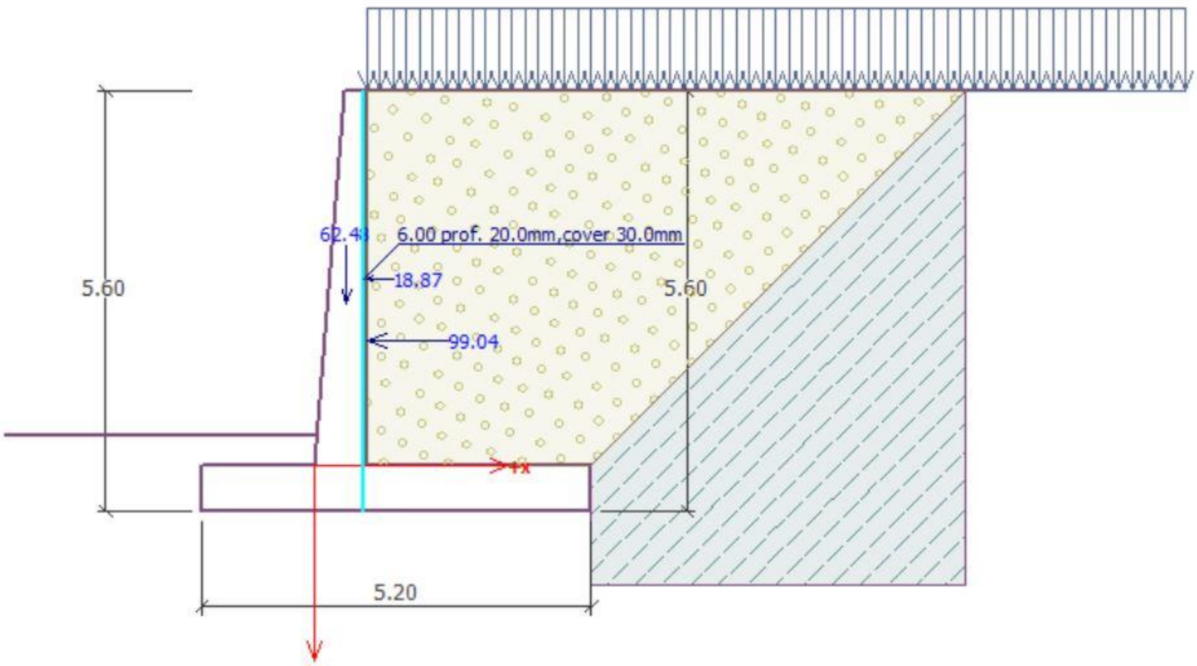


Fig.4.10 Wall Stem Check

Slope stability analysis

Input data

Project

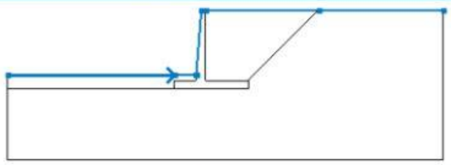
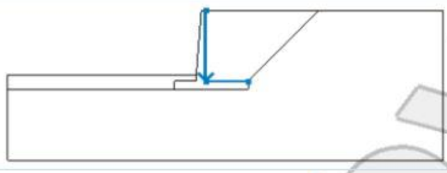
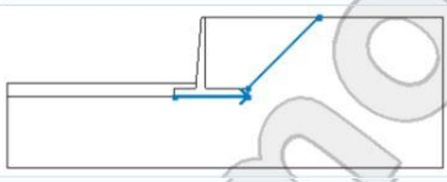
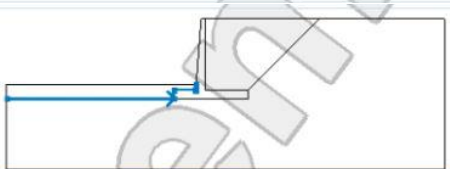
Settings

India – Standard

Stability analysis

Earthquake analysis: Standard
Verification methodology: Factor of safetys (ASD)

Interface

No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		-14.00	-4.60	-2.20	-4.60	-0.67	-4.60
		-0.30	0.00	0.00	0.00	8.00	0.00
		16.80	0.00				
2		0.00	0.00	0.00	-5.00	3.00	-5.00
3		-2.20	-5.60	3.00	-5.60	3.00	-5.00
		8.00	0.00				
4		-14.00	-5.60	-2.20	-5.60	-2.20	-5.00
		-0.70	-5.00	-0.67	-4.60		

Soil parameters- effective stress

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m³]
1	Soil No.1		12.00	6.10	19.62

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m³]
2	Well graded gravel (GW), medium dense		38.50	0.00	21.00

Soil parameters - uplift

No.	Name	Pattern	γ_{sat} [kN/m³]	γ_s [kN/m³]	n [–]
1	Soil No.1		20.00		
2	Well graded gravel (GW), medium dense		21.00		

Soil parameters

Soil No. 1

Unit weight: γ = 19.62 KN/m^3
Stress-state: effective
Angle of internal friction: φ_{ef}= 12.00^o
Cohesion of soil: c_{ef} = 6.10 kPa
Angle of friction struc.-soil δ = 0.00

Soil: cohesionless
Saturated unit weight γ_{sat}= 20.00 kN/m^3

Well graded gravel (GW), medium dense

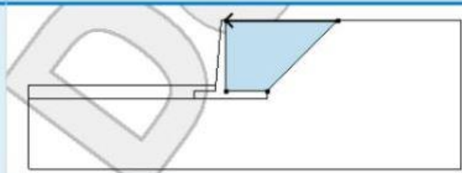
Unit weight: γ = 21.00 KN/m^3
Stress-state: effective
Angle of internal friction: φ_{ef}= 38.50^o
Cohesion of soil: c_{ef} = 0.00 kPa
Angle of friction struc.-soil δ = 0.00

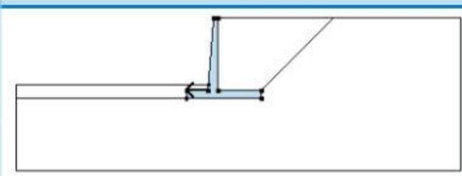
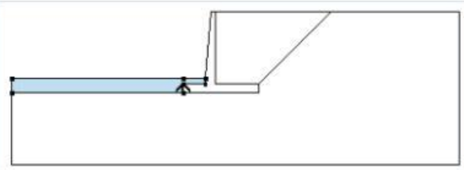
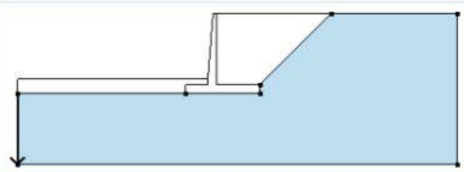
Soil: cohesionless
Saturated unit weight γ_{sat}= 21.00 kN/m^3

Rigid Bodies

No.	Name	Sample	γ [kN/m³]
1	Wall material		25.00

Assigning and surfaces

No.	Surface position	Coordinates of surface points [m] x	z	x	z	Assigned soil
1		8.00	0.00	0.00	0.00	Well graded gravel (GW), medium dense
		0.00	-5.00	3.00	-5.00	

No.	Surface position	Coordinates of surface points [m] x	z	x	z	Assigned soil
2		-0.70	-5.00	-2.20	-5.00	Wall material
		-2.20	-5.60	3.00	-5.60	
		3.00	-5.00	0.00	-5.00	
		0.00	0.00	-0.30	0.00	
		-0.67	-4.60			
3		-2.20	-5.60	-2.20	-5.00	Soil No.1
		-0.70	-5.00	-0.67	-4.60	
		-2.20	-4.60	-14.00	-4.60	
		-14.00	-5.60			
4		-14.00	-5.60	-14.00	-10.60	Soil No.1
		16.80	-10.60	16.80	0.00	
		8.00	0.00	3.00	-5.00	
		3.00	-5.60	-2.20	-5.60	

Water

Water type: No water

Tensile crack

Tensile crack not inputted.

Earthquake

Earthquake not included.

Settings of the stage of construction

Design situation: permanent

Results (Stage of construction-1)

Analysis 1

Circular slip surface

Slip surface parameters

Center $x = -1.41$ m $z = 1.21$ m
Radius $R = 8.10$ m
Angles $\alpha_1 = -44.17^o$
$\alpha_2 = 81.41^o$

Slope stability verification (Bishop)

Sum of active forces $F_a = 341.14$ kN/m

Sum of passive forces $F_p = 339.95$ kN/m

Sliding moment $M_a = 2763.25$ kNm/m

Resisting Moment $M_p = 2753.61$ kNm/m

Factor of Safety = 1.00< 1.5

Slope stability not acceptable

CHAPTER - 5

RESULTS AND DISCUSSION

5.1 Comparison of different cases of retaining walls.

5.1.1 Simple cantilever

Overturning FOS = 2.34 > 1.5

Slip FOS = 0.38 < 1.5

Slope stability FOS = 0.72 < 1.5

5.1.2Simple cantilever with gravel as backfill

Overturning FOS = 8.92> 1.5

Slip FOS = 1.51 < 1.5

Slope stability FOS = 1.00 < 1.5

5.1.3 Simple cantilever with soil mix with shredded rubber tyres

Overturning FOS = 5.02> 1.5

Slip FOS = 0.96 < 1.5

Slope stability FOS = 1.12 < 1.5

As we can see from the above comparison between the different backfill options that we have got we can deduce that:

a) In the overturning of the structure case, the structure will be safest when we provide well graded gravel as backfill as it can provide proper drainage and there will be no extra water pressure generated.
b) For slip condition also the best suited backfill will be of well graded gravel.

5.1.4 Cantilever with shear key

If we want to increase the sliding resistance without widening the base of the retaining wall, then shear key must be provided.
X= distance from the heel

Case-1 Depth of shear key = 0.9

X [m]	Overturning	Slip	Slope stability	Bearing capacity
0	3.92	0.64	1.06	2.05
0.5	3.91	0.55	1.04	2.16
1	3.91	0.55	1.02	2.16
1.5	3.9	0.55	1.02	2.15
2	3.89	0.55	1.02	2.15

Table 5.1

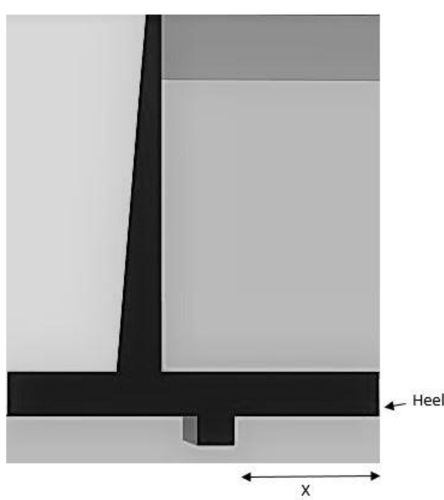


Fig.5.1 Cantilever with shear key

Case-2 Depth of shear key = 1.2

X [m]	Overturning	Slip	Slope stability	Bearing capacity
0	4.09	0.67	1.11	1.96
0.5	4.07	0.51	1.06	2.18
1	4.06	0.51	1.04	2.17
1.5	4.05	0.51	1.03	2.16
2	4.04	0.51	1.02	2.15

Table 5.2

5.1.5 Cantilever with shear key having gravel backfill

Case-1 Depth of shear key = 0.9

Width of shear key = 0.6

X [m]	Overturning	Slip	Slope stability	Bearing capacity
0	9.11	1.95	1.03	2.11
0.5	9.1	1.38	1	2.12
1	9.08	1.38	0.99	2.12
1.5	9.08	1.38	1	2.12
2	9.08	1.38	1	2.12

Table 5.3

Case-2 Depth of shear key = 1.2

Width of shear key = 0.6

X [m]	Overturning	Slip	Slope stability	Bearing capacity
0	9.46	2.73	1.07	2.08

0.5	9.44	1.26	1.03	2.1
1	9.41	1.26	1	2.1
1.5	9.39	1.26	0.99	2.1
2	9.36	1.26	0.99	2.1

Table 5.4

5.1.6 Cantilever with shear key having soil mix with shredded rubber tyres as backfill

X= distance from the heel

Case-1 Depth of shear key = 0.9

X [m]	Overturning	Slip	Slope stability	Bearing capacity
0	5.86	1.11	1.18	2.47
0.5	5.85	0.94	1.14	2.52
1	5.85	0.94	1.14	2.52
1.5	5.84	0.94	1.14	2.52
2	5.83	0.94	1.14	2.52

Table 5.5

Case-2 Depth of shear key = 1.2

X [m]	Overturning	Slip	Slope stability	Bearing capacity
0	6.12	1.19	1.21	2.39
0.5	6.11	0.86	1.18	2.5
1	6.1	0.86	1.15	2.5
1.5	6.08	0.86	1.14	2.5
2	6.07	0.86	1.14	2.5

Table 5.6

From the comparison of cantilever wall having shear key at different positions and in different backfill conditions, we can deduce the following points:

a) The factor of safety for overturning, slip and slope stability is maximum at the point when the shear key is provided right below the heel and as we move towards the stem the factor of safety starts decreasing because the increased sliding resistance is provided from the difference of passive and active forces at the sides of the key.
b) We can also see that as we have increased the depth of the shear key there is an increase in the factor of safety in each case irrespective of the backfill material.
c) It is also seen that the best backfill material came out to be gravel in this case.

5.1.7 Cantilever with shelve

Y = distance from top of the stem to the shelve

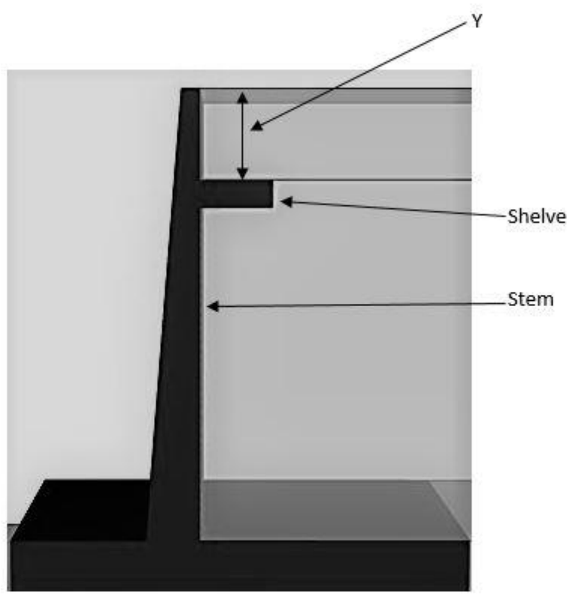


Fig.5.2 Cantilever with shelve

Case-1 Depth of shelve = 0.5

Width of shelve = 0.8

Y [m]	Overturning	Slip	Slope stability	Bearing capacity
1	4.07	0.63	1.02	2.21
1.5	3.92	0.62	1.02	2.17
2	3.83	0.61	1.02	2.14

2.5	3.84	0.61	1.02	2.15

Table 5.7

Case-2 Depth of shelve = 0.5

Width of shelve =1.2

Y [m]	Overturning	Slip	Slope stability	Bearing capacity
1	4.39	0.65	1.02	2.13
1.5	4.11	0.64	1.02	2.18
2	3.93	0.62	1.02	2.15
2.5	3.83	0.61	1.02	2.14

Table 5.8

5.1.8 Cantilever with shelve having soil mix with shredded rubber tyres as backfill

Y = distance from top of the stem to the shelve

Case-1 Depth of shelve = 0.5m

Width of shelve = 0.8m

Y [m]	Overturning	Slip	Slope stability	Bearing capacity
1	6.32	1.07	1.14	2.46
1.5	5.97	1.06	1.14	2.5
2	5.75	1.04	1.14	2.53
2.5	5.75	1.04	1.14	2.53

Table 5.9

Case-1 Depth of shelve = 0.5m

Width of shelve = 1.2m

Y [m]	Overturning	Slip	Slope stability	Bearing capacity

1	6.72	1.09	1.14	2.39
1.5	6.38	1.08	1.14	2.42
2	5.95	1.06	1.14	2.49
2.5	5.73	1.04	1.14	2.52

Table 5.10

When we provide a relief shelve towards the backfill side it reduces the earth pressure on the wall, which results in reducing the thickness of the wall and leads to an economic design. From the comparison of cantilever wall having relief shelve at different levels and in different backfill conditions, we can deduce the following points:

a) Firstly, if we provide the relief shelve when we have gravel as backfill then it gives a constant value of factor of safety at different positions but still the constant value is greater than the one which we get in all other backfill cases.
b) As we move the shelve downward there is a decrease in factor of safety.

5.1.9 Cantilever with geogrid

Geogrid Properties

Tensile Strength R_t = 50 kN/m

Coefficient of interaction = 0.8

Slope stability FOS = 0.72 < 1.5 (Y = 1m)

= 0.72< 1.5 (Y = 2m)

5.2 Seismic Analysis

We study the structures in seismic zones by imparting an extra acceleration due to earthquake forces in the horizontal and vertical direction. In vertical direction the acceleration can be upward or downward depending upon the earthquake.

Horizontal acceleration due to earthquake K_h = 0.1
Horizontal acceleration due to earthquake K_v = 0.15 (upward/downward)

5.2.1 Simple Cantilever

	Earthquake Vertical acc.(Upward)	Earthquake Vertical acc.(downward)
Overturning	3.14	3.71
Slip	0.51	0.6
Slope	0.78	0.74
Bearing Capacity	2.19	1.92

Table 5.11

5.2.2 Simple Cantilever with gravel as backfill

	Earthquake Vertical acc.(Upward)	Earthquake Vertical acc.(downward)
Overturning	5.66	6.82
Slip	1.01	1.19
Slope	0.75	0.71
Bearing Capacity	2.34	1.92

Table 5.12

5.2.3 Simple Cantilever with gravel as backfill (having shear key)

	Earthquake Vertical acc.(Upward)	Earthquake Vertical acc.(downward)
Overturning	5.05	6.05
Slip	1.41	1.72
Slope	0.98	0.92
Bearing	2.22	1.84

Capacity		

Table 5.13

5.2.4 Simple Cantilever with gravel as backfill (having shelve)

	Earthquake Vertical acc.(Upward)	Earthquake Vertical acc.(downward)
Overturning	5.61	6.77
Slip	1.24	1.45
Slope	0.96	0.89
Bearing Capacity	2.36	1.92

Table 5.14

From the above tables we can conclude that:

If we have gravel as backfill then for the overturning stability case the introduction of both relief shelve and shear key reduces the factor of safety.

5.2.5 Simple Cantilever with soil mix with shredded rubber tyres as backfill

	Earthquake Vertical acc.(Upward)	Earthquake Vertical acc.(downward)
Overturning	3.39	4.02
Slip	0.56	0.66
Slope	0.78	0.74
Bearing Capacity	2.32	2.01

Table 5.15

5.2.6 Simple Cantilever with soil mix with shredded rubber tyres as backfill (having shear key)

	Earthquake Vertical acc.(Upward)	Earthquake Vertical acc.(downward)
Overturning	2.98	3.52
Slip	0.67	0.8
Slope	1.02	0.94
Bearing Capacity	1.98	1.77

Table 5.16

5.2.7 Simple Cantilever with soil mix with shredded rubber tyres as backfill (having shelve)

	Earthquake Vertical acc.(Upward)	Earthquake Vertical acc.(downward)
Overturning	3.98	4.56
Slip	0.74	0.86
Slope	1.01	0.93
Bearing Capacity	2.42	2.04

Table 5.17

5.2.8 Comparison of retaining walls having different general dimensions

Case-1

Soil = Sample soil

Width = 0.4m

Depth = varying

Depth varying (acc. Upwards)			

Depth (m)	0.6	0.8	1
Slip	0.99	0.96	0.94
Overturning	5.44	5.13	4.85
Slope Stability	0.75	0.73	0.73

Table 5.18

Depth varying (acc. Downwards)			
Depth(m)	0.6	0.8	1
Slip	1.17	1.14	1.11
Overturning	6.54	6.19	5.87
Slope Stability	0.7	0.7	0.7

Table 5.19

Depth varying (no earthquake)			
Depth(m)	0.6	0.8	1
Slip	1.72	1.68	1.63
Overturning	10.89	10.24	9.65
Slope Stability	0.85	0.83	0.83

Table 5.20

Case-2

Soil = Sample soil

Width = varying

Depth = 0.6m

Width varying (acc.upward)			
Width (m)	0.5	0.7	0.9
Slip	1.01	1.03	1.06
Overturning	5.59	5.88	6.18
Slope Stability	0.75	0.75	0.76

Table 5.21

Width varying (acc. downward)			
Width (m)	0.5	0.7	0.9
Slip	1.19	1.22	1.26
Overturning	6.73	7.12	7.51
Slope Stability	0.71	0.72	0.73

Table 5.22

Width varying (no earthquake)			
Width (m)	0.5	0.7	0.9
Slip	1.72	1.87	1.96
Overturning	10.89	12.36	13.39
Slope Stability	0.85	0.86	0.88

Table 5.23

Case-3

Soil = Gravel as backfill

Width = 0.4m

Depth = varying

Depth varying (acc. Upward)			
Depth (m)	0.6	0.8	1
Slip	0.99	0.96	0.94
Overturning	5.44	5.13	4.85
Slope Stability	0.75	0.73	0.73

Table 5.24

Depth varying (acc. Downward)			
Depth	0.6	0.8	1

Slip	1.17	1.14	1.11
Overturning	6.54	6.19	5.87
Slope Stability	0.7	0.7	0.7

Table 5.25

Depth varying (no earthquake)			
Depth	0.6	0.8	1
Slip	1.72	1.68	1.63
Overturning	10.89	10.24	9.65
Slope Stability	0.85	0.83	0.83

Table 5.26

Case-4

Soil = Gravel as backfill

Width = Varying

Depth = 0.6m

Width varying (acc. Upwards)			
Width (m)	0.5	0.7	0.9
Slip	1.01	1.03	1.06

Overturning	5.59	5.88	6.18
Slope Stability	0.75	0.75	0.76

Table 5.27

Width varying (acc. Downwards)			
Width (m)	0.5	0.7	0.9
Slip	1.19	1.22	1.26
Overturning	6.73	7.12	7.51
Slope Stability	0.71	0.72	0.73

Table 5.28

Width varying (no earthquake)			
Width (m)	0.5	0.7	0.9
Slip	1.77	1.87	1.96
Overturning	11.37	12.36	13.39
Slope Stability	0.86	0.86	0.88

Table 5.29

Case-5

Soil = Soil mix with shredded rubber tyres as backfill

Width = 0.4m

Depth = varying

Depth varying(acc Upwards)			
Depth (m)	0.6	0.8	1
Overturning	3.17	3.61	3.83
Slip	0.66	0.72	0.75
Slope Stability	1	1.01	1.02

Table 5.30

Depth varying(acc Downwards)			
Depth (m)	0.6	0.8	1
Overturning	4.02	4.83	4.59
Slip	0.8	0.84	0.87
Slope Stability	0.93	0.94	0.95

Table 5.31

Depth varying(no earthquake)			
Depth (m)	0.6	0.8	1

Overturning	5.73	6.23	6.76
Slip	1.04	1.09	1.15
Slope Stability	1.14	1.15	1.17

Table 5.32

Case-6

Soil = Soil mix with shredded rubber tyres as backfill

Width = varying

Depth = 0.6m

Width Varying (acc Upwards)			
Width (m)	0.5	0.7	0.9
Overturning	3.51	3.28	3.07
Slip	0.71	0.68	0.65
Slope Stability	1.02	1	0.98

Table 5.33

Width Varying (acc Downwards)			
Width (m)	0.5	0.7	0.9
Overturning	4.15	3.9	3.67
Slip	0.82	0.75	0.76

Slope Stability	0.94	0.92	0.91

Table 5.34

Width Varying (no Earthquake)			
Width (m)	0.5	0.7	0.9
Overturning	5.95	5.54	5.18
Slip	1.05	1.02	0.98
Slope Stability	1.15	1.13	1.1

Table 5.35

5.2.9 Varying magnitude of earthquake

Case-1

Soil = Sample Soil

Width = 0.4m

Depth = 0.6m

Horizontal acceleration due to earthquake K_h = 0.1
Horizontal acceleration due to earthquake K_v = varying

Vertical acceleration (g)(upward)	0.05	0.1	0.15
Slip	0.53	0.51	0.5
Overturning	3.17	3.08	2.99

Slope Stability	0.76	0.76	0.77

Table 5.36

Vertical acceleration (g)(downward)	0.05	0.1	0.15
Slip	0.55	0.57	0.58
Overturning	3.35	3.44	3.53
Slope Stability	0.74	0.73	0.73

Table 5.37

Case-2

Soil = Gravel as backfill

Width = 0.4m

Depth = 0.6m

Horizontal acceleration due to earthquake K_h = 0.1
Horizontal acceleration due to earthquake K_v = varying

Vertical acceleration (g) (upward)	0.05	0.1	0.15
Slip	1.05	1.02	0.99
Overturning	5.81	5.62	5.44
Slope Stability	0.73	0.74	0.75

Table 5.38

Vertical acceleration (g) (downward)	0.05	0.1	0.15
Slip	1.11	1.14	1.17
Overturning	6.17	6.35	6.54
Slope Stability	0.72	0.71	0.7

Table 5.39

Case-3

Soil = Soil mix with shredded tyre as backfill

Width = 0.4m

Depth = 0.6m

Horizontal acceleration due to earthquake K_h = 0.1
Horizontal acceleration due to earthquake K_v = varying

Vertical Acceleration (g) (upwards)	0.05	0.1	0.15	0.2
Overturning	3.37	3.27	3.17	3.07
Slip	0.7	0.68	0.66	0.65
Slope Stability	0.97	0.98	1	1.02

Table 5.40

Vertical Acceleration (g)(downwards)	0.05	0.1	0.15	0.2
Overturning	3.55	3.65	3.74	3.85
Slip	0.73	0.75	0.77	0.79
Slope Stability	0.94	0.93	0.92	0.9

Table 5.41

Conclusion

From our analysis in GEO 5 software and through experimentation on soil sample and the soil mixed with shredded rubber tyres we came to the conclusion;

- Effective stress is significantly reduced when we change the backfill from our normal soil sample to backfill containing gravel or backfill having soil mixed with shredded rubber tyres as backfill.
- We designed cantilever wall having relief shelve and shear key in GEO 5 software and analyzed the structure in various cases in which the dimensions of the structure were altered in every backfill condition.
- Also, we observed that if we add gravel along with providing shear key below the heel we get better resistance against effective stress.
- It was found out that providing shelve near the top of the retaining wall provides us with the best possible results i.e resistance against stresses.
- Lastly we performed seismic analysis in which various dimensions along with the aforementioned backfill conditions were analysed and we came to the conclusion that in majority of the cases gravel used as backfill provides the best possible conditions.

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