ANALYSIS OF RETAINING WALL FOR STATIC AND DYNAMIC CONDITION BY USING GEO-5 SOFTWARE

A REPORT SUBMITTED FOR **MAJOR PROJECT-II**
SUBMITTED IN PARTIAL FULFILLMENT OF THE REQUIREMENTS FOR

THE AWARD OF THE DEGREE

OF

MASTER OF TECNOLOGY
IN
GEOTECHNICAL ENGINEERING

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AUGUST, 2020

CONTENTS

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CANDIDATES DECLARATION

I, **SUDHANSHU RANJAN, Roll No. 2K18/GTE/21** student of M. Tech (Geotechnical Engineering), hereby declare that the report titled **“Analysis of retaining wall for static and dynamic condition by using geo-5 software”** which is submitted by me to the Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is original and not copied from any source without proper citation. This work has not previously formed the basis for the award of any Degree, Diploma Associate ship, Fellowship or other similar title ofrecognition.

Place :- Delhi   Sudhanshu Ranjan

Date :- 31 August 2020

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CERTIFICATE

I hereby certify that the report titled **"ANALYSIS OF RETAINING WALL FOR STATIC AND DYNAMIC CONDITON BY USING GEO-5 SOFTWARE"** which is submitted by **SUDHANSHU RANJAN, 2K18/GTE/21**, Department of Civil Engineering, Delhi Technological University, Delhi in partial fulfillment of the requirement for the award of the degree of Master of Technology, is a record of the project work carried out by the student under my supervision. To the best of my knowledge this work has not been submitted in part or full for any Degree or Diploma to this University orelsewhere.

Place :- Delhi

Date :- 31 August 2020

Prof. Kongan Aryan

Supervisor

ACKNOWLEDGEMENT

I thank my parents, friends and God who all helped me a lot for completion of this project report. I wish to express my profound gratitude and indebtedness to Kongan Aryan, Professor, Department of Civil Engineering, Delhi Technological University, New Delhi, for introducing to this present topic and for his good inspiring guidance and valuable feedback throughout the project work. I like to thanks the developer of the geotechnical software "fine civil engineering", for giving me the permission to use the demo version of the GEO5 software. Using this trial, we achieved the study for cantilever retaining wall which is present in this research paper. By using this demo version software, it allowed to extend the problem of convergence of the structural models in the field of geotechnical.

I also express my deep gratitude to all Faculty member of Department of Civil Engineering and lab assistant and some PhD students of Delhi technological University, who help me lot in process of lab experiment through the manuscript and giving useful suggestions.
I would also express my gratitude to all the professors of the Department of Civil Engineering, Delhi Technological University, New Delhi, for their guidance and the support they have provided me.

Last but not the least, my sincere thanks to all my friends, seniors, google scholar who have patiently extended all sorts of help and guidance for accomplishing this undertaking work in time.

Place :- Delhi   Sudhanshu Ranajan

Date :- 31 August 2020

LIST OF TABLES

LIST OF FIGURES

List of equation

S.N0	EQUATION	REF
Eqn (2.1)	$K_{0(NC)} = 1 - \sin\varphi'$	[3]
Eqn(2.2)	$K_{0(oC)} = K_{0(NC)} * OCR^{(\sin\varphi)}$	[3]
Eqn(2.3)	$P_a = \gamma z \frac{1 - \sin\varphi}{1 + \sin\varphi}$	[4]
Eqn(2.4)	$P_a = \gamma z \frac{1 + \sin\varphi}{1 - \sin\varphi}$	[4]
Eqn(2.5)	$FOS = \frac{M_{RES}}{M_{OVR}}$	[3]
Eqn(2.6)	$P_{Max} = \frac{RV}{b}\left(1 + \frac{6e}{b}\right)$	[4]
Eqn(3.1)	$\frac{P_A}{\sin(\beta - \varphi')} = \frac{W}{\sin(90 + \theta + \delta - \beta + \varphi')}$	[4]
Eqn(3.2)	$W = \frac{\gamma H^2}{2} \cdot \frac{\cos(\theta - \alpha)\cos(\theta - \beta)}{(\cos\theta)^2 . \sin(\beta - \alpha)}$	[3]
Eqn(3.3)	$P_A = \frac{\gamma H^2}{2} \cdot \frac{\cos(\theta - \alpha)\cos(\theta - \beta)\sin(\beta - \varphi')}{(\cos\theta)^2 . \sin(\beta - \alpha) . \sin(90 + \theta + \delta - \beta + \varphi')}$	[3]
Eqn(3.4)	$P_A = \sigma_a \frac{\gamma H^2}{2}$	[3]
Eqn(3.5)	$K_a = \frac{\cos^2(\varphi' - \theta)}{\cos^2(\theta)\cos(\theta + \delta)\left[1 + \sqrt{\frac{\sin(\varphi' + \delta)\sin(\varphi' - \alpha)}{\cos(\delta + \theta)\cos(\theta - \alpha)}}\right]} 2$	[3]
Eqn(3.6)	$\sigma_a = \sigma_z K_a - 2C_{ef} . K_{ac}$	[3]
Eqn(3.7)	$K_a = \frac{\cos^2(\varphi - \theta - \beta)}{\cos^2(\alpha)\cos(\alpha + \delta)\left[1 + \sqrt{\frac{\sin(\varphi + \delta)\sin(\varphi - \beta)}{\cos(\delta + \alpha)\cos(\alpha - \beta)}}\right]} 2$	[3]
Eqn(3.8)	$K_{ac} = \frac{K_{ahc}}{\cos(\alpha + \delta)}$	[3]
Eqn(3.9)	$K_{ahc} = \frac{\cos\varphi\cos\beta\cos(\delta - \alpha)(1 + tg(-\alpha)tg\beta)}{1 + \sin(\varphi + \delta - \alpha - \beta)}$	[3]
Eqn(3.10)	$K_{ac} = \sqrt{K_a}$	[3]
Eqn(3.11)	$\sigma_{ax} = \sigma_a . \cos(\alpha + \delta)$	[3]
Eqn(3.12)	$\sigma_{az} = \sigma_a . \sin(\alpha + \delta)$	[3]
Eqn(3.13)	$\frac{P_P}{\sin(\beta + \varphi')} = \frac{W}{\sin(90 + \theta - \delta - \beta - \varphi')}$	[4]
Eqn(3.14)	$P_P = \frac{\gamma H^2}{2} \cdot \frac{\cos(\theta - \alpha)\cos(\theta - \beta)\sin(\beta + \varphi')}{(\cos\theta)^2 . \sin(\beta - \alpha) . \sin(90 + \theta - \delta - \beta - \varphi')}$	[3]
Eqn(3.15)	$P_p = K_p \frac{\gamma H^2}{2}$	[3][4]

Eqn(3.16)	$\mathrm{Kp} = \dfrac{\cos^2(\varphi' + \theta)}{\cos^2(\theta)\cos(\delta-\theta)\left[1 + \sqrt{\frac{\sin(\varphi'+\delta)\sin(\varphi'+\alpha)}{\cos(\delta-\theta)\cos(\theta-\alpha)}}\right]}2$	[3]
Eqn(3.17)	$\sigma_P = K_P\sigma_z \cos(\alpha+\delta)$	[13]
Eqn(3,18)	$\sigma_{px} = \sigma_z\ cos(\alpha+\delta)$	[13]
Eqn(3.19)	$\sigma_{pz} = \sigma_z \sin(\alpha+\delta)$	[13]
Eqn(3.20)	$F = \dfrac{\sum[C' + \left(\left(\frac{w}{b}\right) - u\right)\tan\varphi']}{\sum[\left(\frac{w}{b}\right)\sin\alpha]}$	[4]
Eqn(3.21)	$\varphi = \cos\alpha + \dfrac{\sin\alpha\tan\varphi}{F}$	[4]
Eqn(3.22)	$PAE = 0.5\gamma H^2 (1-Kv) * K_{AE}$	[4]
Eqn(3.23)	$K_{AE} = \dfrac{\cos^2(\varphi-\theta-\beta)}{\cos\theta\cos^2\beta\cos(\delta+\beta+\theta)\times\left[1+\sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]}2$	[3]
Eqn(3.24)	$P_{AEH} = P_{AE}\cos(\beta+\delta)$	[3]
Eqn(3.25)	$P_{AE} = \frac{1}{2}\gamma H^2(1-\mathrm{Kv})K_{AE}\cos(\delta+\beta)$	[3]
Eqn(3.26)	$P_{AE} = \frac{1}{2}\gamma H^2(1-\mathrm{Kv})K_{AE}\cos(\delta)$	[3]
Eqn(3.27)	$P_{AE} = 0.5\,\gamma H^2(1-\mathrm{Kv})K_{PE}$	[3]
Eqn(3.28)	$K_{PE} = \dfrac{\cos^2(\varphi+\theta-\beta)}{\cos\theta\cos^2\beta\cos(\delta-\beta+\theta)\times\left[1+\sqrt{\frac{\sin(\varphi-\delta)\sin(\varphi-\theta+i)}{\cos(\delta-\beta+\theta)\cos(i-\beta)}}\right]}2$	[3]
Eqn(3.29)	$\sigma_Z = \sum_{i=1}^{n}\gamma_i\,.h_i.[kpa]$	[11]
Eqn(3.30)	$k_0 = \dfrac{\mu}{1-\mu}$	[4]

ABBREVIATION

BS :- Below the surface

M_{res}:- Resisting moment

M_{ovr}:- Overturning moment

H_{res} :- Resisting horizontal force

H_{act} :- Active horizontal force

Satis :- Satisfaction

F_a :- Sum of active forces

F_p:-Sum of passive forces

M_a :-Sliding moment

M_p:- Resisting moment

FOS :- Factor of safety

e :- Eccentricity

e_{alw} :- Allowable eccentricity

FF Res :- Face resistance

ABSTRACT

A wall which is formed for opposing the horizontal physical force of earth becomes famous in the form of wall. This force is exerted because the soil repose angle exceeds due to the required modifications in ground height. Ahead of constructing this wall, means in its designing phase, it becomes essential to identify and balance the tendency of the retained material to lower inclination because of gravity. This force is entirely depends upon the angle of internal friction (ø) and the bonding power of the retained material. In addition it also depends, how much and in which direction retaining structure moves. Objective of this research is to analysis of gravity wall and cantilever retaining wall for varying terrains, soil condition and water table. Comparative analysis between classical and numerical approach (FEM). And also find out the most suitable retaining wall for different soil condition .Effect of dynamic loading resulted from earthquake of different soil condition, terrains and water table condition to be determined for gravity and cantilever walls. In thesis work two methodologies are used which is classical approach and numerical approach, Coulomb's, method are used for active and passive condition &Mononobe-Okobe method is used for earthquake (dynamic) condition and Bishop's method is used for slope stability, all methods are predicted by using of GEO 5 software. On the basis of present study following conclusions may be drawn that the factor of safety is decreasing with increase in slope angle. Force and moment are increasing with increasing in slope angle.The comparative study of finite element method and classical method as indicated that finite element approach is more near to the realistic condition and it evaluate the other parameter like stress intensity, shear strain deformation and FEM is also better due to in classical approach more assumption will be required, but in finite element method is that no assumption needs to be made in advance about the shape or location of the failure surface, slice side forces and their directions.

Keywords : Retaining wall, FEM, GEO-5 software, coulomb's method, Mononobe – Okobe method, Bishop's method

Chapter 1

INTRODUCTION

1.1 Overview

A wall which is formed for opposing the horizontal physical force of earth becomes famous in the form of wall. This force is exerted because the soil repose angle exceeds due to the required modifications in ground height. Such type of walls is required for protecting earth from abnormal inclination. Basic intention behind their usage is to bound earth in the middle of two heights usually in those regions which contains inclinations which are not desirable. These walls are also used in those regions where it becomes essential to form the picture of landscape in advance in order to fulfill some particular purposes like hill side farming or roadway overpasses. The overall structure of retaining wall consists retaining wall in addition to backfill soil. Such type of walls are commonly used in those infrastructures which have basements, in the construction of roads, and bridge in order to hold earthwork in a standing position. Retaining walls are commonly supported by soil (or rock) underlying the base slab, or supported on piles; as in case of bridge abutments and where water may erode or undercut the base soil as in water front. It is a wall which is formed for opposing the horizontal physical force of earth becomes famous in the form of wall. This force is exerted because the soil repose angle exceeds due to the required modifications in ground height. A wall which is formed for opposing the horizontal physical force of earth becomes famous in the form of wall. This force is exerted because the soil repose angle exceeds due to the required modifications in ground height. Ahead of constructing this wall, means in its designing phase, it becomes essential to identify and balance the tendency of the retained material to lower inclination because of gravity. As a result of this, lateral earth pressure behind the wall creates. This force is entirely depends upon the angle of internal friction (ø) and the bonding power of the retained material. In addition to this it also, it also depends, how much and in which direction retaining structuremoves.

Such types of walls are constructed in order to withstand with earth force. In addition to his, its construction becomes possible in the area where landscaping occurs in an artistic manner. Previously, the word retaining wall has been used for those walls whose construction is done in order to hold the force of earth inclination or other materials. In most of the cases these walls are constructed in a very effective way and become very useful. But, in some cases, its look like strong but actually not and in some other worst cases it turns in to a show piece. Whenever the level of earth elevation modified these walls make soil strong. at a change ground elevation. Such type of wall is made up of stabilizing stonework, therefore its structure is inflexible.

1.2Working of retaining Wall

The physical force which is exerted by the earth in the direction of structure is very significant. Therefore, the construction work of this wall is very complex. Ahead of its construction different parameters are considered, out of which most are technical. In the absence of this wall earth would not remain in inclination position. Therefore, it becomes essential that this wall must be very powerful.

This earth hold directly following the wall is what would give in to gravity and the inclination collapse in the absence of this wall. Remaining part usually stay in its original position, in the form of normal inclination. But, in situations where an earthquake came, this wall bear more pressure.

Normally, such walls are constructed with a small hidden bend. They are constructed in such a way so that earth force does not turn in to noticeable bend. Their construction should be done in such a way that it can avoid the water support behind them. If this support remains un avoided, then some addition force will developed. This force damages the wall. In some designs highly refined water discharge system are included in comparison to other design where a standard water discharge system is used. But, the most important point is the availability of water discharge system which is must.

From the construction point of view, the knowledge of level and circulation of sideways force in the middle of earth, its mass and an neighboring earth structure is very important. A significant contribution is provided by the values of energetic earths force in designing standards. These values are totally depend upon earth variables. In short we can say that a retaining walls exis in the form of wall which holds different kind of forces in different conditions and in some specified conditions it holds earthquake loads in accordance with the general principles specified in this section.

Basic intention behind their usage is to bound earth in the middle of two heights usually in those region which contains inclinations which are not desirable. These walls are also used in those regions where it becomes essential to form the picture of landscape in advance in order to fulfill some particular purposes like hill side farming or roadway overpasses. Structures widely used in the field of construction, e.g., bridge abutments, highway cuts, stream channels, waterfront areas, basement walls, material storage. It used for maintaining earth or water or other materials such as coal, ore, etc.; in which conditions do not permit the mass to assume its natural slope.

1.3 Design Parameters

- The design of this wall is completely depend upon the location on which it is constructed. The height of wall, type of water discharge system it required, and quality of material all are decided only after the examination of site.

- Design plan of mechanically stabilized earth walls are usually made by the manufacturer.

- A wall which is constructed on the basis of strategy and stipulation which is dually signed by a licensed Engineer becomes famous in the form of engineered wall.
- A wall which is constructed on the basis of strategy and stipulation of an engineer but in the absence of his signature becomes famous in the form of non-engineered walls. These type of walls are not constructed in places where traffic is expected near the top of the wall.

Types of retaining wall

1. Concrete: It is type of engineered wall. The basic intention behind its construction is to provide stability to an inclination in order to hold the earth behind it.
2. Masonry: Its design is almost identical to above described wall but in the construction of this wall, blocks of particular design are used in order to make it attracted. In the construction of this wall a stock of those blocks are used whose fabrication is done in advance. Blocks which are used in the construction of this wall either take soil support or not. In situations where the support of earth is taken this wall is considered in the form of MSE wall.
3. Rock retaining wall: It exists in the form of a wall which is very important and constructed with the help of rock materials. It provides support to earth mass in a very elegant way. Its construction takes place when the required height of wall is up to ten feet.
4. Railroad tie retaining wall: It is a type of wall which is constructed on the basis of of railroad connection. This wall is stable to the foundation rock .

5. Gabions: Single- or multi-celled rectangular wire mesh baskets that are filled with rock and wired together to form a retaining structure. It becomes possible to use this in the form of retaining walls in order to stabilize steep inclination in a mechanical way. It becomes very helpful where seepage is planned. Refer to the Standard Specifications for property requirements for gabions. Erosion control geo material is often placed behind gabion baskets to prevent the fine material of the retained soil from entering the basket. ¬
6. Geo synthetic: It is a bendable type of wall which is constructed with geotextile It is built when layers of fill material are placed in a row. The load of subsequently placed layer holds the folded geo synthetic from the earlier placed layer
7. MSE retaining wall: Such type of walls are constructed in crowded places. It becomes possible for this wall to maintain large disparity arrangement

.

1.4 ProblemStatement

At this point of time, for the designing of retaining walls large number of methods are available. Classical method is the most popular approach for the analysis of retaining wall. Various Methods have been evolved in the past based on classical approach, such as Coulomb's method, Rankine method, Muller- Breslau method, Caqout- Kerisel method, Absi method etc. These methods provide the safety parameters regarding overturning Slip as well as safe in bearing ability of soil based on certain assumptions.

The hypothesis of this research is the analysis of retaining walls not only by the classical approach (i.e. by the Coulomb's methods) but also through finite element method under static as well as Dynamic conditions with different soil conditions, terrain condition and ground water conditions. When these two methods are compared, then it has been find out that the former has some extra benefits in comparison to primary method the analysis of retaining walls problems over traditional classical methods is that no assumption needs to be made in advance about the shape or location of the failure surface, slice side forces and their directions. It becomes possible to use this method in the company of complicated slope structure and ground deposition in two or three proportion for the designing of machines in an effective way.

1.5 Objective:-

1. Objective of this research is to analysis of gravity wall and cantilever retaining wall for varying terrains, soil condition and watertable.
2. Comparative analysis between classical and numerical approach (FEM). And also find out the most suitable retaining wall for different soilcondition.

3. Effect of dynamic loading resulted from earthquake of different soil condition, terrains and water table condition to be determined for gravity and cantileverwalls.

CHAPTER 2

LITERATURE REVIEW

2.1 General

A wall which is formed for opposing the horizontal physical force of earth becomes famous in the form of wall. This force is exerted because the soil repose angle exceeds due to the required modifications in ground height. Such type of wall is required for protecting earth from abnormal inclination. Basic intention behind their usage is to bound earth in the middle of two heights usually in those region which contains inclinations which are not desirable. Retention walls are used to withstand a mass of earth or some other substance laterally to accommodate a transport facility. The walls are used in a number of applications, including right-of - way constraints, protection of “existing buildings, grade separations, new road bench building”, road widening, sloping stabilization, protection of vulnerable areas of environmental significance, stage and temporary support including excavations or underwater building supports. These walls are used in different applications. Various types of retention wall systems are required for earth preservation and satisfy particular project needs.

2.2Types of Retainingwall

Depending upon the basis of configuration and the methods which are used for the purpose offorce opposition, retaining walls are separated in to types which are described below.

1) Gravity wall
2) Cantilever retaining
3) Counter fort
4) Buttress
5) Crib
6) Gabion
7) Sheet Pile
8) Slurry

9) Secant Pile

10) In-Situ

Gravity wall: It a type ofretaining wall which depends completely on its own weight for thepurpose of standing. Earlier these walls are known in the form of retaining structures. “Gravity walls” are stabilized by their mass. They are built From the combination of solid concrete or rock rubble cement these walls are structured.

Crib retaining walls: These are gravity walls but they are filled with crushed stone and use a grid-like structure made with pre-cast concrete. Typically, these walls are good for planting, but not suitable for holding a tone of land.

Gabion retaining walls: These walls are filled with large stones and mesh. Used typically if erosion is a major concern. Retaining walls of Cantilever. Due to its design, these walls provide leverage with a base label sliding under the ground. Field walls, the walls of Cantilever are rather similar, but require an additional backrest.

They are constructed of heavy and durable materials such as concrete and steel. Some gravity walls use mortar, like dry stone walls, to hold their weight. It's made of mortar. They are economical for only small heights.This type of wall depends not on a foundation, but on the imperishable load mass of the wall for stability. The weight of the gravity walls (pierce, concrete or other heavy material) is dependent on to withstand pressure from behind and often has a slight reverse effect to enhance stability by retention of soil. The lateral forces of the backfill are resisted by the wall itself and develop little or no tension because of its massive nature. Consequently, they are usually not strengthened by steel. For heights of up to 3 m (10 feet) gravity walls are economical.

Cantilever Wall: - The most popular type of walls in use today involve walls that hold earth by walls cantilevering from a footing. These walls are known as "made" because of the lack of lateral control, they are free to rotate (about the foundation). Template retaining walls are typically made of stem or concrete, or both; however, as mentioned, other types may also be used. Cantilever's reinforced concrete walls typically consist of a horizontal foundation and a vertical stem wall. The earth's mass above the sky creates a solid wall.For heights up to 10 m (32 feet), the Cantilever walls are economical. This wall uses a cantilever operation to ensure the

retaining of the reinforced concrete. Most lift walls appear in the cross section as "L"s or" T"s reverted. They are built on solid to ensure stability.
Support foundations connected with reinforcing rods to the vertical portion of the wall. The base is then filled back to offset the vertical component of the wall.

Counter fort retaining wall: - Counter-fortified walls of retention contain wings that projected from the heel to the stem upward. The stalk thickness is thinner (as for cantilevered walls) between counter forts, and extends as a beam horizontally, between the
counter-forts (fling) walls. The counterforces function as fuel-efficient components, since the counterforces are confined to a larger base on the heel at times higher. The counter-forces are structurally efficiently. The high costs of constructing counter-fortifications and instilling stem walls typically render these walls less than 16 meters high difficult to use for walls.For a height greater than about ten meters, the vertical bracing method referred to as counter forts is economically feasible. See figure 2.3, Both the base plate and the wall side of the rear forts are horizontal in this situation.

Buttress wall: - These are similar to castle walls, except from the outside of the wall the wings build. In situations where the constraints on the land retention side do not allow room for a wide heel of a conventional retaining wall, such walls are normally used. Buttress walls provide additional support to a wall by transferring horizontal pressure onto the base below. The walls of the buttress may be hidden beneath the earth or built to be a decorative element of the house. The buttress-walls allow the contractor to make higher and thinner walls, with less concrete and the same pressure. These walls hold loads (like a beam), which transform horizontal
pressure from behind the walls into upside down vertical pressure. In some situations, heavy-duty walls are protected on the front or have a controller on the rear, which increases their resistance to heavy loads. “Buttresses” are small wings in the right angle to the wall 's principal theme. The stiff foundation for these walls is lower than the seasonal frost level. This kind of wall uses much less material than a typical wall of gravity.

Crib Wall: - Different materials like wood , concrete and even plastic are used for making crib walls. Walls of crib consist of boxes made of wood or pre-cast cement that are interlocked. The

boxes are then packed with broken stone or other gross grain materials
Establish a system for free drainage. Book and reinforced pre-cast concrete are two primary types of crib walls. Cribs are typically very wide and with the surrounding countryside they can be out of proportion and character. Heavy construction equipment is also essential to map out the courses, affecting potentially vulnerable areas. It can be used at moderate altitudes between 4 and 6 m.

Gabion Walls: - Gabions are multi-cell, welded wire or rectangular mesh boxes used for the construction of erosion control systems and the stabilization of steep slopes. They are rocks filled and used for support. Gabions are the commonly used in civil engineering to stabilize coastlines, river banks or erosion slopes. The walls, bridge ports, Wing walls, Culvert headwalls, Shore and beach barrier barriers, Temporary check dams etc. are included for use. The frequency of a flood water flow can be directly regulated around a fragile structure.
In small streams, gabions are also used as fish barriers. Owing to their modularity and the ability to stack in different ways, gabion baskets have many benefits over unloose riprap; they are also resistant to water removal. Gabions have benefits compared to more rigid structures because they can comply with the movement of the earth, dispose energy from running water and drain freely. In some cases their strength and efficiency can develop as sludge and vegetation fill the interstitial gaps and strengthen the structure. Often they are used to prevent stones that fall from the cliff of a thoroughfare or disturb the traffic. The longevity of the gifts depends on how long they last.The string, not the basket stuff. If the wire fails , the system will collapse. The most widely used galvanized steel wire is PVC-coated and stainless wire.

Sheet Pile Walls: - The holding of sheets of piles in deep soils and narrow spaces are commonly used. Board battery walls are commonly used to secure terrestrial paths up to ten meters deep as financially and technically productive temporary retention structures and deep excavation support systems. Used for the construction of continuous walls for buildings on the banks and for temporary building walls of > 6 m with anchors. They can be made from stainless steel, plastic and wood. Sheet piles are used in situations where the following is appropriate apart from slope protection:
The excavation security is in the vicinity of the excavation against groundwater inflow and/or

settlement-prone installations. The use of pre-stressed floor anchors and/or framework retention padding systems allows the surroundings to be secured from damage against settlement and existing buildings.

Slurry Walls: - A gum wall is used in soft earth environments, near open water or with a high base water table (by Gutberle in 1994) for the construction of reinforced cement walls. This method is commonly used for building diaphragm walls around tunnels and open cuts and setting foundations. The technique is used to trap water. The slurry walls are lower level walls, limiting soil-bentonites or concrete-structure flow (cut-over / barrier) or supporting scooping and structural exhaust systems. These walls are constructed at a depth of "100 foot and have a thickness" of 2 to 4 feet. These walls are designed to a depth of 100 feet and range from 2 feet to 4 feet in thickness. Usually, "the panels are 15 feet to 25 feet long" and are connected with each other by tongue-type groove (scales) which prevent the water from being drawn to the future ground. The walls have a depth of about 100 foot, which range from 2 to 3 ft.

Secant Pile walls: - Secant piles are designed such that one pile is intersected by another. The intersection of individual reinforced concrete piles shapes these walls. These "piles" are made by drilling and "boiling mud (bentonite)". The secant piles are about 3 inches overlapping. The tangent stack walls, where stacks do not overlap, are an alternative. "These piles" are placed flush together. The major benefit of secant and tangent walls is greater stability in alignment. The walls can also be more stable and the building process is less disruptive. One of the downside is the difficulty of waterproofing at the joints, their higher cost, and the difficulty in achieving vertical tolerances for the more deep stacks.

In situ walls:-The soil is not dependent on the mass of these walls. Instead, they rely on their bending powers to sustain the earth. Their incorporation into the ground or the anchoring structures assists them.

2.3 The forces on retaining wall:

The forces acting on “retaining wall” are categorized into:

1. “Resisting Forces include

i. Water Pressure(Pw)

ii. Earth Pressure in Passive State(Pp)

iii. Sliding Friction(F)

iv. Foundation Pressure(Pf)

2. Driving Forcesinclude

i. Water Pressure(Pw)

ii. Earth Pressure in Active State(Pa)

iii. Weight(W)

iv. Surcharge Load (q)”

EarthPressure

Earth pressure is called the weight or power of the soil on its boundaries. When the pressure of the ground acts on a retaining wall side (back or face), it is known as the pressure of the “lateral earth” The extent of the lateral earth pressures depends on the retaining wall's movement relative to the filling and on the soil's nature.

“The lateral earth pressure is usually computed using the classical theories proposed by Coulomb (1773) & Rankine (1857). The general wedge theory proposed by “Terzaghi (1943)” is more general and is an improvement over the earlier theories.”

Lateral Earth Pressure is a function of:-

- Type of amount of wallmovement
- Shear strength Parameter of soil
- Drainage conditions of backfill

Three conditions of Earth Pressure

At Rest EarthPressure:

When the retaining wall does not move at all, i.e. it is "resting" at the main horizontal and vertical stress ratio. The soil mass is resting while the wall is stable and incomplete and deformations and displacements do not occur. Earth pressure is called – rest earth pressure. Earth pressure is named at that state. Pressure on the side of the planet, depicted as K0. For normal condensed soil, K0 is measured.

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

Jaky (1948) ……………………(2.1)

Mayne & Kulhawy (1982) for over consolidated soils:

$$K_{0(oC)} = K_{0(NC)} * OCR^{(\sin\varphi)}$$

……………………….(2.2)

Where; OCR = over consolidation ratio and ϕ' is the effective stress frictionangle.

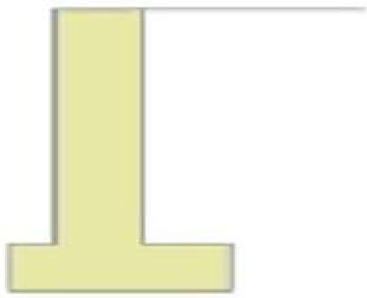

Fig 2.1 Earth Pressure at Rest[6]

Active EarthPressure:

When the wall rises from the rear wall, part of the back wall starts to break and begins to travel away from the rest of the earth mass and the backfill and slip surfaces can grow outwardly before failure occurs. At this point, the force acting on the wall is called active pressure on the earth. The minimum pressure on the lateral soil is possible.

$$P_a = \gamma z \frac{1 - \sin\varphi}{1 + \sin\varphi}$$

………………………….(2.3)

Where; γ = unit weight of soil, Z = depth

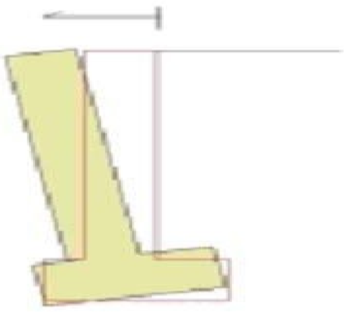

Fig 2.2 Active Earth Pressure[6]

Passive EarthPressure:

The ground is compressed and gives a resistance due to its shearing resistance as the wall is forced into rearfill, which results in a steady rise in earth strain. If this force reaches a value that the reverse surface does not resist, the failure will occur again and the surface glides. At this phase, the pressure is known as passive earth pressure. This is the highest pressure on the side of the soil that can be used.

$$P_a = \gamma z \frac{1 + \sin\varphi}{1 - \sin\varphi}$$

…………………………..(2.4)

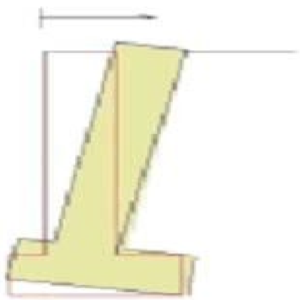

Fig 2.3 Passive Earth Pressure[6]

Failure of Retaining walls:-

There are five types of failure of retaining walls:

- Sliding failure
- Overturning failure
- Bearing failure
- Shallow Shear failure
- Deep Shear failure

Sliding failure: - Sliding insufficiency is nothing but the separation of the wall from the backfill when the base of the wall is shaving insufficiently.

Where μ = co-efficient of friction =tanδ

"R_V & R_h = Vertical & Horizontal component of resultant R of weight of wall & earth pressure"

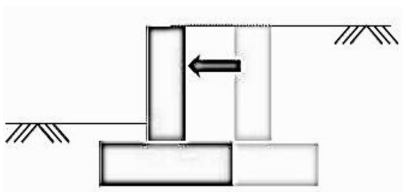

Fig 2.4Failure due to sliding[5]

Overturning failure: - The overturning fault is the movement of the wall around the toe triggered by overrunning forces to resist.

The F.O.S against overturning is given by

$$FOS = \frac{M_{RES}}{M_{OVR}}$$

……………………………….(2.5)

FOS < 1.5 – 2.0

Where;

“ΣM_R = Sum of resisting moment about toe”“ΣM_O = Sum of overturning moment about toe”

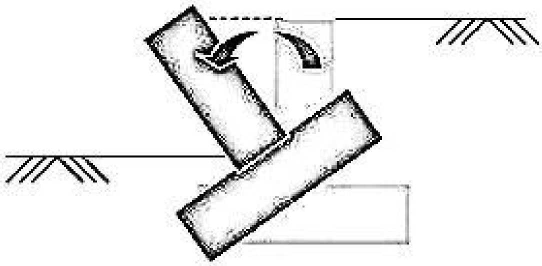

Fig 2.5 Failure due to overturning[5]

Bearing failure: - The tension of the resulting vertical force must not exceed the acceptable bearing capacity of the soil. The strain must not exceed. The distribution of the pressure is known to be linear.

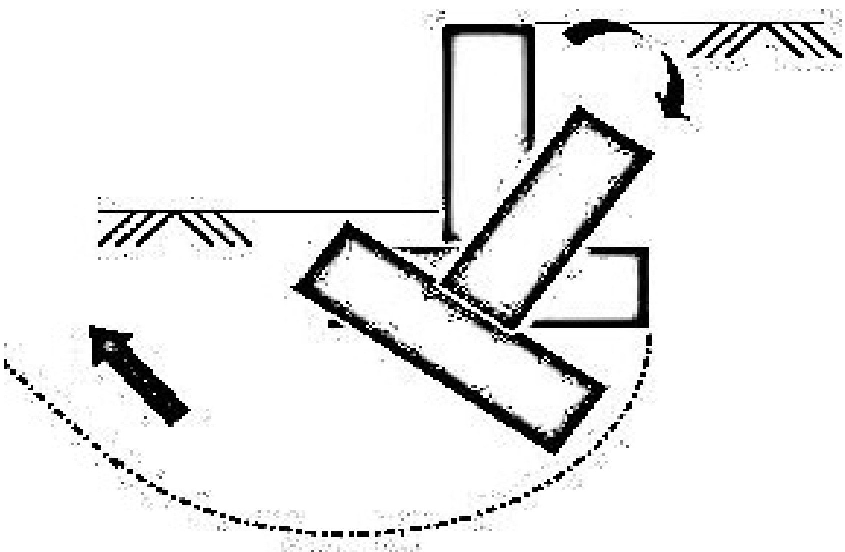

Fig 2.6 Failure due to Bearing[5]

$$P_{Max} = \frac{RV}{b}\left(1 + \frac{6e}{b}\right)$$

………………..(2.6)

Shallow Shear failure: - This kind of failure occurs on a surface. A failure occurs when the cylindrical surface of the soil is under extreme shear stress.FOS is lower than for shallow shear failure with horizontal sliding.There will be no shallow shear loss, but FOS vs. sliding will be greater than 1,5.

Deep Shear failure: - This kind of pitch failure happens slower on the ABC cylindrical surface. If the soil under the wall is poor at approximately 1.5 times the wall height.Trials and error techniques identifies the critical failure surface.The DSF critical failure surface passed through the edging of the heel slab, shows as shown in figure for the rear pitches of pit I is below 100.In this situation, it is also necessary to check the risk of over-settlement.

2.4 CaseStudies

(A) According to **B. M. Bakhtin in 2002** describes the determination of seismic earth pressure in three approaches. The first one assumes that a sliding wedge (bulging) is formed in the backfill behind the wall, whose pressure upon the wall is higher than in the static operation of the structure. This causes an additional load, namely, seismically dynamic or inactive earthpressure.
Second approach depends upon the theory that under the effect of earthquake wall involves a certain adjacent earth mass in vibrations and together with that earth mass forms a dynamic system with its own dynamic characteristics, which calls for the spectral dynamic theory to determine the loads on the wall. The third one considered the predominant role of earth mass in the work of the "wall – earth" system and assumes that the natural oscillations of the wall are not manifested in this case, and while the wave propagates inside the backfill mass and the foundation, dynamic loads are formed in the earth as a reaction to the restrictions of the earth displacement imposed by the more rigid retainingwall.

(B) According to **A. J. Khan and M. Sikder in 2004** one by one describes design methods in support of walls which are externally and internally stabilized. Different researchers propose these approaches. Typical design examples have been given for cost-comparison of such walls that were externally stabilized and internally stabilized, i.e. reinforced concreted cantilever retaining walls, reinforced metal strip walls, geotextile reinforced walls and anchored earth walls of various heights. The analyses show that the internal stability of the walls is much cheaper than in the externally stable wall in this analysis, and with the walls increasingly high this economic gain will increase. The construction techniques have been studied and planned for the various types of externally and internely reinforced walls such as RCCW, MSW, GTW and AEWs with a height of 2.1 m, 3.0 m, 4.2 m, 5.1 m and 6.0 m above the current level of the floor (EGL). In order to measure the cost per running meter of the walls, the walls are then measured. In comparison with the externally restored wall for the specified geometric and loading conditions considered in this analysis, internal stabilized walls are found to be significantly economical. The main difference in costs is due to the large quantities of concrete and stainless steel tanks that the RCCWs normally need compared to their counterparts. With wall heights the economic advantage obtained by internally balanced systems increases. The internally stabilized walls will save between 43 and 64 percent.

(C) According to **Wang Yuan-Zhan.et.al in 2004** describes thatBased on the principle of Coulombs, the “earth pressure against the retaining wall” is caused by a thrust from the back of the earth to the ground that passes through the lower edge of the wall and has the inclination of an angle of a line of a unit of the earth pressure, the resulting earth pressure and point of application of the resulting ear Theoretical answers theoretical answers to unit earth pressure, The formula, the formula for the mode of translation of the wall, the formulation for the coulomb and some experimental observations were all presented in this report. It is shown that the magnitudes of the earth pressure resulting from the wall rotation mode around the top are identical to the formula for the wall translation mode and “coulomb theory”. The mode of translation But there is a great variation in the distribution of atmosphere and surface pressure as well as the points of application of the resulting earth pressure. Finally, it concludes that for the mode of translation of wall movement and mode of movement in the wall around top, which is also equivalent to that defined by the coulomb principle, the value of the resulting earth pressure is similar. “The Earth Pressure Distribution on a retaining wall” is not linear and is different for moving the wall. The point at which the resulting earth pressure is applied in the mode of translation wall motion is about “(0.40-0.45) H” over the wall base. In the mode of rotation around the top of the wall the point of operation is around (0.46-0.57) H above the base of the walls and higher than in the mode of translation for the wall movement. The values of resulting Earth pressures are equal to the values defined by Coulomb's principle for many movement modes. Thus “Coulomb's theorem” can be used to measure the resulting earth pressure in the sliding stability study of the gravity retaining structures. In the application points of the resulting earth strain, there are major variations in different mode of movement. Therefore, the wall failure modes or mode of wall movement should be found in the rotating stability review of gravity retaining structures.

(D) According to **Deepankar Choudhury.et.al in 2006** describes the “Pseudo-static comparison and Pseudo- Dynamic methods for seismic earth pressure on retaining walls” require thorough knowledge of both “active and passive earth pressure.” The construction of this natural hazard needs special consideration in the event of an earthquake. But the available literatures are mainly pseudo-static under seismic conditions as an estimated solution to the actual dynamic existence of the complex problem, empirical earth pressure value. Current research is using a newly developed pseudo-dynamic approach to integrate the time-based behavior on the use of earthquake burdens and the influence on shear and primary waves to study the effects of parameter changes such as the friction angle of the earth, frozen wall angles, earthquake vibrations, earthquake shear and primary seismic wave velocity. For the equivalent static force, pseudo static accelerations, both horizontal and vertical accelerations are taken into account. The effect of changes in shear and primary waves in the time and period of the rigid earth's seismic retaining wall was demonstrated in the paper. It provides a more practical, non-linear, seismic, active distribution of the pressure of the Earth behind a retention wall compared with the Mononobe Okabe process. However, the traditional pseudo-static method does not only allow for a linear distribution of earth strain, regardless of the static or seismic conditions. The seismic passive Earth pressures are most sensitive to the wall friction angle, compared to the seismic active Earth pressure. The seismic active earth pressure is more seismic than traditional pseudo-static analysis using the pseudo-dynamic approach provided in this document and the seismic passive earth pressure is more seismic. In this way, the optimal seismic and passive pressure coefficients are given using pseudo-static methods as opposed to current values, as the construction of a holding wall against the destructive impact of the earthquake leads to a safe and secure approach.

(E) According to **Deepankar Choudhury and Shantiram Chatterjee in 2006** describes “Dynamic active earth pressure” on the structures of retention by means of traditional methods either uses pseudo-static analytical approaches for dynamic cases or a basic one-degree model of freedom for the retentive “wall-soil system.” The goal of this paper is to estimate the active Earth pressure behind retaining walls for wall mode translation under seismic conditions by a simplified two-degree dynamic model of liberation “mass-spring-dashpot (2-DOF)” The dynamic force on the wall is calculated to impact the horizontal portion.

For some typical cases, results in terms of displacement, speed and acceleration times are given which indicate the final wall motion in terms of the wall height that is required for the design. The non-dimensional diagram suggested in this study can be used in various movement conditions in the ground and backfill to measure the total dental ground force on the wall. Finally the findings were contrasted with the available “Scott model” and the validity of the present results were addressed. The results were compared. Complete dynamic earth pressure forces on the vertical retainer wall with a “semi-infinite”, homogenous and visco-elastic soil medium can be measured easily using the 2- DOF mass-spring-dashpot model, in contrast with traditional SDOF models. The key results of this study can also be summarized as follows: The current study also produces the critical influence zone distance for the complex earth impact. (2) For any input earthquake motion, the displacement, speed and acceleration time history can easily be acquired, and this in turn gives the amount of wall movement appropriate for design. (3) The “non-dimensional” geometry map proposed in the study can be used easily to measure the total dynamic earth power acting on the walls for various input soil motion and backfill damping characteristics. (4) Present findings well compare to the current Scott model (1973), but the substantial weakness in estimating earth force at a higher frequency in the traditional Scott (1973) model is corrected by the proposed system.

(F) According to **Deepakar Choudhury.et.al in 2006** describes In the design of the earthquake-prone “retaining wall”, “seismic active earth pressure behind” the rigid retaining wall is very important. Commonly used method “Mononobe-Okabe” takes “pseudo-static approach” into account, which provides an estimated linear distribution of seismic earth strain. In this paper we are using the “pseudo-dynamic approach” to quantify the variations in time and phase of the backfill distribution of seismic active earth pressure on a rigid holding wall that supports cohesiveness less backfill. The study takes account of planar rupture surface. Effects on the seismic, active earth pressure of the wide range of parameters such as “wall friction angle, soil friction angle, shear wave velocity and primary wave velocity were studied”. Tables and graphical non-dimensional results are given in order to compare the pseudo-static approach to illustrate the practical non-linearity of the distribution of seismic active earth pressure. The “seismic active earth” pressure distribution and the overall active thrust behind the containing wall are accomplished with the pseudo-dynamic approach by taking the time effect and phase shift of shear and primary waves spreading behind the rigid containing wall. Compared with the Mononobe-Okabe process, it offers an increasingly practical non-linear seismic active earth pressurization distribution behind the retaining wall. The non-linearity of the active distribution of earth pressure increases when seismicity results in a change in the point of application of the total active thrust required for the system. However, the classical pseudo-static method only provides a linear distribution of earth pressure regardless of the static or seismic situation, which lead to a major disadvantage in design criteria. The seismic active earth pressure measurement behind a rigid retaining wall is seen by taking into account “the effects of the horizontal and vertical seismic acceleration coefficients”, the angle of wall-friction, and the angle of soil-freezing. Pseudo-dynamic research found that both horizontal and vertical seismic accelerations are important in assessing seismic active earth pressure and, furthermore, their value is increased as the intensity of the earthquake increases. Horizontal as well as vertical seismic levels greatly alter the earth's active pressure. The active seismic thrust is very sensitive to the “soil's friction angle” is less sensible to the wall friction angle, and δ.

(G) According to **T.Mandal.et.al**in Oct 2011 describes thatRigid retaining wall maintaining conformity with lower soil cohesion under active and static earth pressure (pseudo-static and pseudo-dynamic). The weight of a unit during soil mass is believed to be constant. For the determination of active Earth pressure under different height of walls, Coulomb theory and Kötters (1903) equation have been used in the static analysis. The Kötter (1903) equation was used in the dynamic analysis. For different soil and wall properties the seismic active soil pressure coefficient is measured. The active seismic earth pressure at different wall levels is recorded in paper for the same cases. In his analysis, Coulomb 's theory and then his (1903) equation measure the static earth pressure in an active state. Comparisons are made between the earth's pressures that are the same and provide a linear pressure distribution with depth. The pseudo-static pressures on the earth differ by seismic acceleration (k_h, k_v), which demonstrate the linear pressure distribution along the soil depth. The pseudo-dynamic earth pressure varies according to the coefficient of seismic acceleration (k_h, kv) and primary waves that are replicated on the back of the rigid holding wall at the seismic earth pressure. In contrast to the pseudo-static method, this results in a more practical non-linear seismic active earth pressure distribution behind the retaining wall.

(H) According to **T. Mandal et.al in 2011** describes the Rigid retaining wall behavior filled with less soil cohesion under passive earth pressure (pseudo-static and pseudo-dynamic). Weight of the unit in the soil mass is expected to be consistent. For the determining of passive earth pressure at different wall heights the Coulomb principle and the Kötter equation have been adopted in the static analysis. These findings are demonstrated and are well matched. The Kötter equation was used in the dynamic analysis. For various soil and wall properties, the coefficient of passive earth pressure is measured. The seismic passive pressure on the earth with varying wall height is tracked and recorded in the paper in the same cases. The passive state static earth pressure is determined by the theory of Coulomb and contrasted with the equation of Kötter during his research. A comparison of these earth pressures is made that are similar and provide a linear distribution of pressure with depth. Pseudo-static earth pressure varies by seismic acceleration (k_h, k_v) which shows a linear pressure distribution along the ground depth. Pseudo-dynamic earth pressure varies according to the coefficient of seismic acceleration (k_h, k_v) and primary waves propagating on seismic ground pressure in the backside of the rigid retaining wall. In contrast with pseudo-static methods, it makes the retaining wall more concrete and not rigid seismic passive earth pressures.

(I) According to **MA Shao-jun et.al in 2012** describes a formula that was obtained by means of “pseudo-dynamic method” to calculate the “seismic active earth pressure” behind a wall. The actual dynamic effect, with time change and spread of shear and main wave speeds through the soil fill, was considered in this Formula. The impact of the tension crack was examined in the top part of the seismic backfill. Also studied have been the effects on the seismic active force from a “wall friction angle, soil friction angle and horizontal and vertical seismic coefficients.” The studies have shown that the overall seismic active force is increased as the coefficient horizontally increased, while the coefficient of internal friction and unit cohesion is decreased with the increase of the vertical seismic coefficient. In contrast with the previous theory, the seismic active force measured using the new approach is higher. The current research indicates a coherent early refill based on the pseudo-dynamic approach for the seismic active earth pressure at the retained wall. The depth of the seismically cracked region is also measured and the depth of the seismic stress crack is given a more practical value. The results of the present work show that the overall seismic activity increases, along with the increase of horizontal seismic coefficients, internal friction angle, unit cohesion and vertical seismic coefficient. The seismic active Earth Pressure is highly sensitive to seismic vertices as well as to internal friction angles, unit cohesion and seismic coefficients, but comparatively less sensitive. In the analysis, the active seismic earth strain is distributed. The seismic active force distribution behind the retention wall is more realistic than the pseudo Static form, which has a linear distribution of the earth active pressure. The current work provides higher values of seismic active power, compared to previous approaches.

(J) According to **Yong Wu et.al in 2012** describes The protection of the anchor system is established and the seismic energy "input-dissipation mechanism"The retention mechanism for an earthquake is studied to understand and efficient aseismic steps taken from the upper border theorem based on energy theory are proposed. In addition, a seismic architecture of a retaining wall is proposed according to a versatile retention theory by studying the wave characteristics of an earthquake that is destructive. Finally, an example was given and the result showed that the seismic behavior of an ordinary rigid retention walls poor and that the structure quickly fails under a strong seismic force in a particular direction. The new device can now distribute seismic energy well with an EPS damping pad. In this paper , an overview of the upper limit for "the failure mechanism of a wall in an earthquake" is adapted and the latest seismic architecture techniques studied. The energy is incorporated as a deciding factor in the study of stability by defining the stability factor as the ratio between internal and external work rates. Then an aseismic configuration of the holding wall is proposed in accordance with the flexible retaining theory. Finally, the following conclusions are drawn with the study of a field example:

(a)The retaining wall and slope are stable depending on the relative relationship between the seismic force executable external work rate, pitch gravity, retentive wall stability and the dissipation of internal energy on the sliding surface.

(b) The seismic force-induced permanent displacement of a single peak is minimal, but its accumulation contributes to a retaining wall failure.

(c) The typical retaining wall's seismic reaction is intense due to its high rigidity, resulting in many retaining walls which are even with a conservative design demolished during an earthquake. (d) A new energy dissipation within the wall, by deforming the EPS layer, will absorb the most seismic energy, which decreases the wall's seismic response and improves the stability of the main structure.

(K) According to **Xiaobo Ruan.et.al In 2013** describes The derivation of analytical speech against a retaining wall backfilled with solid soil to the active seismic pressure (ASPS). The surface of the failure is called smooth. For different variables, ASPS values are compared obtained via the “pseudo-static and pseudo-dynamic methods”. Parametric research has shown that the pseudo-static approach is more conservative than the “quasi-dynamic method”, which represents a nonlinear wall “distribution of ASPS”. This paper aims to describe the active seismic pressure of the soil against a battered retaining wall supporting a compact soil wedge-shaped backfill. For this reason, both the horizontal and vertical components of the seismic effect are taken into account and the pseudo-dynamic approach centered on Coulomb slider prisms.

(L) According to **Chetan Sharma and Vijay Baradiya in 2014** define Wall retention is a system that has a major goal of preventing side motion, preserving soil or water and can withstand vertical loads. The construction of retaining walls includes redesign, slipping and coating stability testing. A test section is assumed and the stability tests are determined. The geometry and the final section profile can be measured within a minimal time by means of these diagrams, for a given wall height, soil data and concrete strength. The diagrams are optimized for a broad variety of unit weight, instant, values. The key strength that is present on the wall that appears to bend, to slip and to overturn the lateral force due to the earth 's pressures. This study focuses on the creation of an alternating, sliding and bearing cantilever style wall. The key considerations are the section 's external stability using codal provision i.e. IS: 456:2000 In particular the section providing the appropriate safety factor meets the external stability criteria. In order to protect the system from failure with respect to these specific parameters, the relationship of resisting forces to disruption is a safety factor, and this safety factor must always be larger than unity. Multiple failure modes have various safety factors. In this paper the stability check for a cantilever wall is carried out with a computer programme which calculates different sections that meet the stability requirements based on the height and properties of the wall to be supported. The analysis and computation are carried out in many parts which satisfy the check conditions and show in these sections the size of the respective section and the stain requirement. Practical approaches to the construction of a retaining wall were taken into account and a rational concept was achieved by the coding. Following conclusions are taken on the basis of this report.

1. It was found that when the density of rear filling content is reduced, the measured safety factor increases. The resulting horizontal force affects the sliding protection factor and its permissible resistance.
2. In efficient construction of a retaining wall, the measured safety factor can be used.
3. The determined FOS is the basis for research against sliding.
4. As the material density is increased, the resulting turn-over time increases. It indicates that a triangular load distribution exists.

2.5Conclusion from LiteratureReview

During a literature review of previous years' nominal research paper, the comparative behavior of walls for homogeneous and non-homogeneous soil conditions and a comparative study of the classic and numerical method has not been examined. Classical and numerical methods to GEO5 software and the effects of an earthquake on various types of soil retention walls have been used for this work.

CHAPTER 3

METHODOLOGY

3.1 Overview

This chapter present the methodology developed for analysis of retaining wall. In thesis work two methodologies are used which is classical approach and numerical approach, by using of GEO 5 software. In classical approach Coulomb's, Rankine, Absi, Muller-Breslau, CaqoutKerisel are used for active and passive condition &Mononobe- Okobe method is usedfor earthquake (dynamic) condition and Bishop's method is used for slope stability. But only Coulombs, Bishop's method, MononobeOkobe methods are used in work. All methods are predicted by GEO 5 software. In numerical approach only Finite element method is used. And lastly comparative analysis between two approaches for retaining wall, which method is best suited in the examination of retainingwall.

3.2 Earth Pressure Method Coulomb'sTheory:-

The effect of earth sideways force on retaining wall was first noticed by Coulomb in the year of nineteen seventy six. For determining this force concept of limit equilibrium was implemented by him. According to this concept failing soil block are assumed in the form of open block. The limiting horizontal pressures at failure in extension or compression are used to determine the K_a and K_P respectively. In order to identify the surface by which maximum and minimum force is exerted on the wall it becomes necessary to examine lots of potential failure surfaces. The amount of friction available in the middle of earth and behind wall is also considered by Coulomb. This concept came in to existence through statics of a considered linear failuresurface.

Assumption: -

1. The earth is uniform andidentical
2. The cracked part of surface is flatsurface
3. The failed block is considered solid
4. The pressure surface is considered flatsurface
5. Wall friction is present on forced surface
6. Failure occurs from both sides

7. The soil is free from coherence.

Active Case:-

The energetic pressure came in to existence because of weight which tried to pull down the earth block and the soil and structure holds the movement of the block in the company of considered linear slip surface. The force direction is derived directly out of problem geometry . Connection in the middle of driving and the wall force becomes very useful in conditions where the structure is designed and it becomes possible to derive it out of the geometry alone via the sine rule.

$$\frac{P_A}{\sin(\beta - \varphi')} = \frac{W}{\sin(90 + \theta + \delta - \beta + \varphi')}$$

…………………………….(3.1)

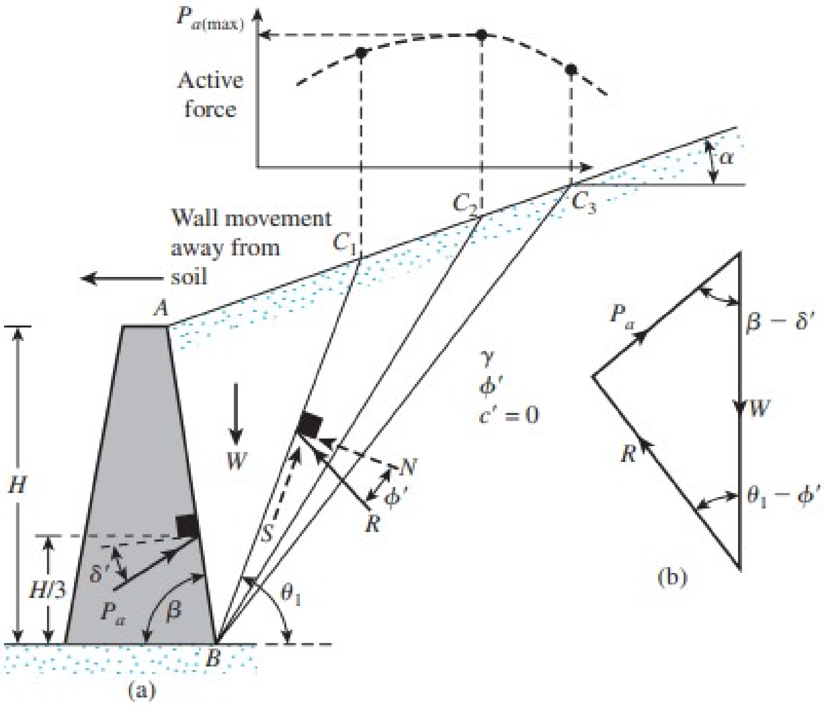

Fig 3.1 Active pressures on a retaining structure along with a force diagram[3]

$$W = \frac{\gamma H^2}{2} . \frac{\cos(\theta - \alpha)\cos(\theta - \beta)}{(\cos\theta)^2 . \sin(\beta - \alpha)}$$

…………………………………(3.2)

When the equations (3.2) and (3.1) are combined, a new equation derived in which horizontal active pressure is connected in the company of wall height. It considered failed surface inclination.

$$P_A = \frac{\gamma H^2}{2} . \frac{\cos(\theta - \alpha)\cos(\theta - \beta)\sin(\beta - \varphi')}{(\cos\theta)^2 . \sin(\beta - \alpha) . \sin(90 + \theta + \delta - \beta + \varphi')}$$

………………………………...(3.3)

Here;

γ : Represents backfill density

H : Represents structural height

In situation where we considered wall friction angle, δ, constant we obtain equation which has only one variable; the inclination of the failure surface, β. With the help of differentiation and back substitution, an equation in support of energetic force is derived out of most critical failed plane in the form

$$P_A = \sigma_a \frac{\gamma H^2}{2}$$

………………………(3.4)

Here the soil force coefficient becomes independent on β and is calculated by the equation given below:-

$$K_a = \frac{\cos^2(\varphi' - \theta)}{\cos^2(\theta)\cos(\theta + \delta)\left[1 + \sqrt{\frac{\sin(\varphi' + \delta)\sin(\varphi' - \alpha)}{\cos(\delta + \theta)\cos(\theta - \alpha)}}\right]^2}$$

………………………………(3.5)

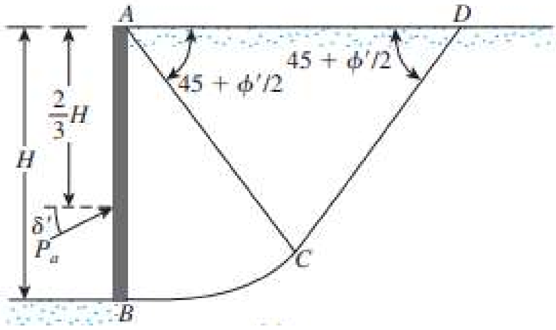

Fig 3.2 condition for failure under active conditions[3]

Active earth pressure is given by:-

$$\sigma_a = \sigma_z K_a - 2C_{ef}.K_{ac}$$

....(3.6)

Where;

σ_z : Represents upright staticstress

C_{ef} : Represents effective cohesion of soil

K_a :RepresentsEnergetic soil force coefficient

K_{ac} : Represents Energetic soil force coefficientdue tocohesion.

K_a can be given by the equation

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

………………………(3.7)

K_{ac} is calculated by the equation given below:

For, $\propto < \frac{\pi}{4}$

$$K_{ac} = \frac{K_{ahc}}{\cos(\alpha + \delta)}$$

…………………………........(3.8)

$$K_{ahc} = \frac{\cos\varphi \cos\beta \cos(\delta - \alpha)(1 + tg(-\alpha)tg\beta)}{1 + \sin(\varphi + \delta - \alpha - \beta)}$$

…………………………(3.9)

For, $\propto \geq \frac{\pi}{4}$

$$K_{ac} = \sqrt{K_a}$$

………………………(3.10)

Where;

δ : Represents frictional angle between wall and earth

α : Represents inclination the inclination of wall back face

β : Represents slope inclination behind the structure

φ : Represents earth's internal friction angle

Leveling and upright components of the,Active soil force turns in to

$$\sigma_{ax} = \sigma_a . \cos(\alpha + \delta)$$

…………………………(3.11)

$$\sigma_{az} = \sigma_a . \sin(\alpha + \delta)$$

…………………………(3.12)

Where;

σa : Represents Active soil force

δ : Represents frictional angle between wall and earth

α : Represents inclination the inclination of wall back face

Inactive Case:-

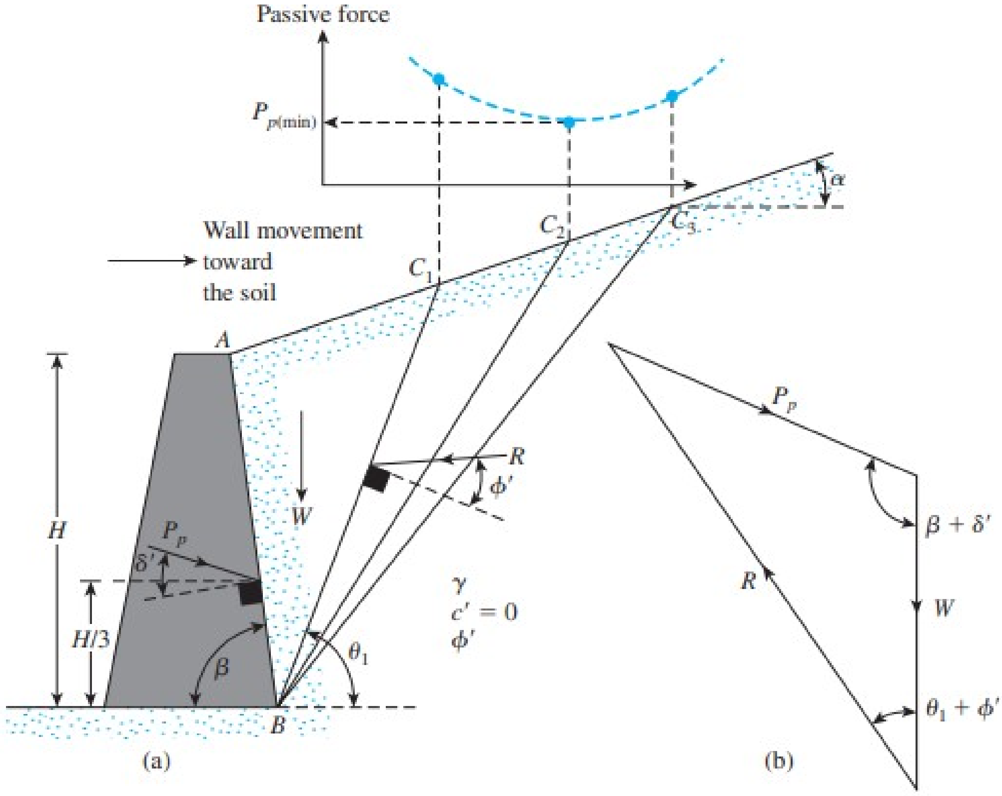

Fig 3.3 Passive pressures on a retaining structure along with a force diagram[3]

Wall sliding towards embankment. Soil wants to displaced upwards on the pressure surface AB but this movement is opposed by surface friction . Due to this, on this surface shearing pressure works in downward direction. The passive force of the soil becomes the resultant of standard force and the shearing stress. Shearing pressure starts rotating in upward direction in the company of angle of frictionof wall .

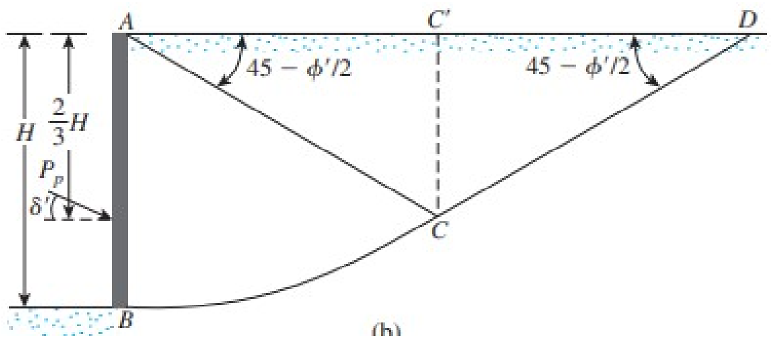

Fig.3.4 Failure condition in passive case[3]

Configuration of failed block in both situations does not altered, whereas direction of wall and frictional stress differ out of earlier one. Connection in the middle of driving and wall force can be derived with the help of sine rule in the form of

$$\frac{P_P}{\sin(\beta + \varphi')} = \frac{W}{\sin(90 + \theta - \delta - \beta - \varphi')}$$

…………………(3.13)

With the help of equation 3.9 and 3.13 it become possible to derive the equation for Coulombs inactive soil force in the form of:

Inactive pressure in case of highly critical plane can be derived when the methods which are specified in support of active case are applied. Passive pressure can be calculated by the equation give below

$$P_P = \frac{\gamma H^2}{2} \cdot \frac{\cos(\theta - \alpha)\cos(\theta - \beta)\sin(\beta + \varphi')}{(\cos\theta)^2 . \sin(\beta - \alpha) . \sin(90 + \theta - \delta - \beta - \varphi')}$$

……………………………(3.14)

Inactive pressure is calculated by the equation given below :

$$\mathrm{Pp} = \mathrm{Kp}\frac{\gamma \mathrm{H}^2}{2}$$

……………………………(3.15)

$$\mathrm{Kp} = \frac{\cos^2(\varphi' + \theta)}{\cos^2(\theta)\cos(\delta - \theta)\left[1 + \sqrt{\frac{\sin(\varphi' + \delta)\sin(\varphi' + \alpha)}{\cos(\delta - \theta)\cos(\theta - \alpha)}}\right]^2}$$

……………………………….(3.16)

From the formula ,passive earth pressure

$$\sigma_P = K_P \sigma_z \cos(\alpha + \delta)$$ ……………………………(3.17)

Where;

δ - Represents frictional angle between wall and earth

α - Represents inclination the inclination of wall back face

β - Represents slope inclination behind the structure

φ - Represents earth's internal friction angle

σpvand σphis calculated by the given equation

$$\sigma_{px} = \sigma_z \, cos(\alpha + \delta)$$

…………………………(3.18)

$$\sigma_{pz} = \sigma_z \sin(\alpha + \delta)$$

……………………….(3.19)

Where;

δ : Represents frictional angle between wall and earth
α : Represents inclination the inclination of wall back face

3.3 Slope StabilityMethod:-

In order to identify the durability of inclination which is considered in the form of security

parameter, retaining
wall use only Bishop’s method.

Bishop’s Method:- When Alan W. Bishop submitted its modified form Bishop's it becomes a method by which durability of inclination is identified. Exactly, this method is the expansion of Method of Slices.It has been noticed that this method generates value of security variable which is almost equal to the authentic value. It becomes possible to determine the security variable the equation given below is repeated over and over again

$$F = \frac{\Sigma[C' + \left(\left(\frac{w}{b}\right) - u\right) \tan \varphi']}{\Sigma[\left(\frac{w}{b}\right) \sin \alpha]}$$
…………………………….(3.20)

Where:-

$$\varphi = \cos \alpha + \frac{\sin \alpha \tan \varphi}{F}$$
……………………..(3.21)

C' : Represents useful coherence

Φ : Represents useful internal angle of internal friction

b : Represents width of one slice,

Here it is assumed that width of all slices are identical

“W” - Means weight of single slice

“U” - Means force of water available in the base of single slice

3.4 Earthquake Analysis Method:-

Mononobe -OkabeMethod
Up to this point of time, this is the primary method which is used by geotechnical engineers whenever an unnatural incidence of earthquake happed. The basic intention behind its usage is the estimation of sideways force of the soil. With the help of this method equilibrium equations

are solved. On the basis of this solution we get earthquake resistant sideways force of the soil. Such type of methods are completely depend upon hit and trial process. This method becomes the anti earthquake model of coulomb concept. It came in to existence on the basis of pseudo static earthquake loading in support of rough earth. In comparison to the various other types of complicated methods, such type of method becomes the initial choice for the evaluation process. Methods which are normally used for the purpose of estimation of active sideways force were invented by Okabe in the year of nineteen twenty six and by Mononobe in the year of nineteen twenty nine. This methods were invented in support of materials which are dry and free from cohesion.

These methods were invented on the basis of some **assumption which are described below**:

1. The output of structure is able to create minimum dynamic force
2. As soon as, minimum energetic force is attained, an earth block is at the point of incipient failure and the maximum shear strength becomes mobilized in the company of potential slidingsurface.
3. Earth's structure works in the form of rigid body. It maintains consistent acceleration across weight. Because of that it becomes possible to demonstrate the earthquake motion impact by the inertia forces

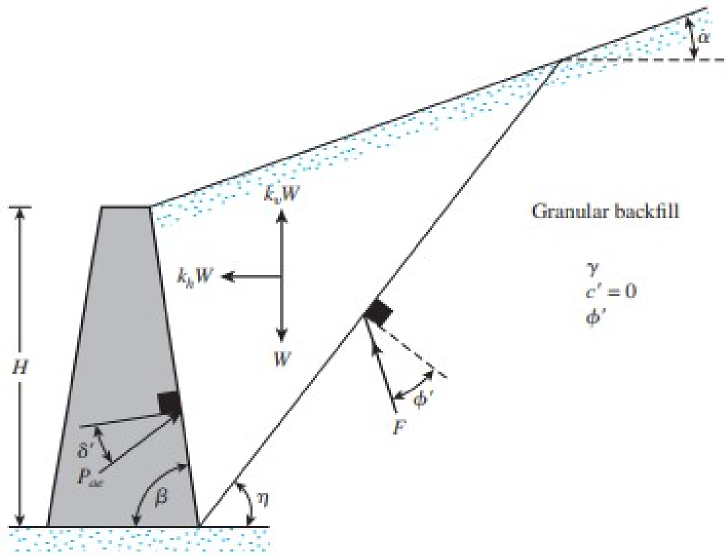

Fig 3.5 Forces considered in Mononobe-Okabe Analysis[3]

At the time of earthquake P_{AE} is calculated with the help of Coulomb concept. Extra forces which are represented in figure three point four are considered at the time of calculation. Value of PAE is calculated by the equation give below:

$$PAE = 0.5\gamma H^2 (1-Kv) * K_{AE}$$

…………………………………(3.22)

$$K_{AE} = \frac{\cos^2(\varphi - \theta - \beta)}{\cos\theta \cos^2\beta \cos(\delta + \beta + \theta) \times \left[1 + \sqrt{\frac{\sin(\varphi+\delta)\sin(\varphi-\theta-i)}{\cos(\delta+\beta+\theta)\cos(i-\beta)}}\right]} 2$$

……………………………….(3.23)

γ - Means earth's specific gravity

H- Represents structure height

Φ - Means Earth's friction angle

δ- Means structure friction angle

i- Represents ground surface inclination behind structure

β- Represents back wall inclination to the vertical

K_h- Represents plain earth acceleration per g

K_V -represent standing earth acceleration per g

Plain component of pressure P_{AE} may be represented in the form of $P_{AEH,}$

Here

$$P_{AEH} = P_{AE} \cos(\beta + \delta)$$

…………………………..(3.24)

$$P_{AE} = \frac{1}{2}\gamma H^2(1 - \mathrm{Kv})\mathrm{K}_{AE}\cos(\delta + \beta)$$

………………………………(3.25)

For the retaining wall with vertical back is $\beta = 0$

$$P_{AE} = \frac{1}{2}\gamma H^2(1 - \mathrm{Kv})\mathrm{K}_{AE}\cos(\delta)$$

……………………………….(3.26)

It was assumed by Mononobe and Okabe that the overall force which is calculated with the help of their scientific method would also put same pressure on the wall which was put by the primary active pressure.

Inactive holding force under earthquake situations may be expressed by the equations;

$$P_{AE} = 0.5\,\gamma H^2 (1 - \mathrm{Kv})\mathrm{K_{PE}} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.27)$$

Where

$$\mathrm{K_{PE}} = \frac{\cos^2(\varphi + \theta - \beta)}{\cos\theta \cos^2\beta \cos(\delta - \beta + \theta) \times \left[1 + \sqrt{\frac{\sin(\varphi-\delta)\sin(\varphi-\theta+i)}{\cos(\delta-\beta+\theta)\cos(i-\beta)}}\right]^2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots(3.28)$$

Value Validity and Weak points :-

Here those weak points are specified due to which this method is considered by the engineers at the time problems solving process.

a) This method is useful only when the soil is free from cohesion.
b) The table of water which is available behind structure, its impact was not taken in to consideration in theformula.
c) Such method has no answer in situations where $\Phi - \beta - \theta$ is less than equal to zero.
d) Traditional issues in civil engineering are not always related to structure having continuesbackfill.

→**Calculation of K_h and K_v**(Using IS 1893(part 1)): 2002[7]

K_h=$(ZISa)/(2Rg)$

Where;

Z- Represents Zone parameter

I - Represent importance parameter,

R - Response reduction factor, depending on the perceived seismic damage performance of the structure, characterized by ductile or brittle deformation.

S_a/g = Average response acceleration coefficient for rock or soil sites.

Zone factor, Z[7]

Seismic zone	II	III	IV	V
Intensity	Low	Medium	Severe	Very Severe
Z	0.10	**0.16**	0.24	0.36

Here soil site is medium so, S_a/g ={1+15T, $0.001 \leq T \leq 0.10$

{2.50, $0.10 \leq T \leq 0.55$

{1.36/T, $0.55 \leq T \leq 4.00$

I = 1.5 in support of importance service and community building and Dams.

R = 1 (assume)

$T = 0.09h/ d^{1/2}$

Calculation of K_h for 4m height and 5^0 slope angle;

$\tan 5^0 = 0.087 = h/b$……(1),

Here;

h= height of retaining wall and b represent is base dimension of the structure basic dimensions at ground level in meter in the company of sideways pressure assumed orientation .

0.087= 4/b

b= 45.7 m

Now, $T= 0.09h/b^{1/2}$

$= (0.09x\ 4)/ (45.7)^{1/2}$

T = 0.053, which is less than 0.10 then using formula

$S_a/g = 1+ 15T$

= 1.75

Now using equation (1) and calculate K_h = 0.084 And **K_v = 0.5x K_h**

= 0.042

Calculation of K_h, the zone factor is III.

Table No-3.1

Height(m)	Slope angle(^{0})	Kh	KV
4	5	0.084	0.042
	10	0.1016	0.0508
5	5	0.0912	0.0456
	10	0.1056	0.0528
6	5	0.0948	0.0478
	10	0.1144	0.0572
7	5	0.0984	0.0492
	10	0.12	0.06
8	5	0.102	0.051
	10	0.12	0.06

3.5 Numerical Approach Method (Finite ElementsAnalysis):-

Such type of methods becomes famous in the form of methods which can solve the numerical problems which are related to the values of boundary . It becomes possible that these problems include the irregular connection of materials. Such methods are already implemented in the form usual hardware for the purpose of designing and examination. This designing and examination work is done in support of geotechnical frameworks. It mainly predicts security parameter and examine contracts. This method is also used for the designing and examination work in support of enhanced earth framework. It becomes possible to achieve these qualities of FEM, but it becomes possible in conditions where material factors, fundamental formula and boundaries are specified in a proper manner.

Standard ideology related to FEM:-

The method of Finite element represents a pattern through the combination of finite elements. It is assumed, at nodes these elements are connected in the company of each other. With the help of this method consistency of structure is assessed in a numerical way. With the help of this method easy and complex geometric pattern of structure can be easily assessed.

In support of differential formula these method provides rough clarification of the problems which are related to the boundary values. In mathematical operational these clarification are provided in numerical way. For this purpose, variation methods are applied in order to decrease the error operations up to minimum level. It gives out consistent clarification solution.

In relation to soil mechanics these method becomes an important part in situations where the behavioral pattern is predicted through essential formula. Because of that, engineers get clarification of those technology based issues which are related geotechnical issues. Mostly, complex problems and the the issues which becomes impossible to solve by standard examination. For the identification of accurate behavioral pattern different type of earth soil designs and computed program are formed.

Steps in FEM

Steps followed in this method are:

1. Separate structure in to limited parts
2. Devlope qualities of allparts
3. Join all parts in order to achieve limited part design ofstructure
4. Apply recognized loads
5. Apply boundary specification
6. Estimate shifted directions
7. Estimate strain, and at last estimate force out ofstrain.

3.6 GEO 5 –In order to make a distinction in the middle of two fundamental situations FEM

Method are used :-

1. **A planar problem**: - It is a part of analysis which plays an important role in the solving of one dimensional structures (a tunnel, embankment, open cut, dam etc.). In order to achieve this purpose it refers, perpendicular dimension of the area beingsolved in terms of size is higher in comparison to the sideways dimensions .
2. **Axial symmetry:** It is a part of analysis which plays an important role in the solving of issues which are balanced in the direction of rotation. It is necessary that the geometrical

organization of wall and burden must be pleased these consideration. pumping groundwater from a circularborehole.

Type of analysis:-

1. **Stress :-** It serves to solve basic geotechnical problems in ground environment and a rock mass (e.g. for determining the vertical or horizontal geostatic stress, pore pressure, deformations, volumetric changes and sub-grade deformations, as well as analyzing internal forces along a diaphragm wall structure length (height) etc.).

2. **Unsteady flow**: It is a part of analysis which plays an important role in the determination of the evolution of pore pressures (the total head) and the current degree of saturation in the company of given time. In this case, the analysis methodology is similar to that of the stressanalysis.

3. **Steady flow**: It consider that the saturation degree does not change with time; each construction phase is entirely independent of each other (in contrast with the unsteadyflow).

4. **Slope stability**: At the time of assessment it decreases internal friction angle input values or the soil cohesion and seeks the onset of failure combined in the company evolution of critical area of localized plastic deformation. The consequences becomes security primary corresponding to classical slope stability analysismethods.

5. **Tunnels**: It is a part of analysis which plays an important role in the analysis of underground excavation (modeling of the 3D effect of excavation attributed to the New Austrian Tunneling Method) accounting for degradation of beams, temperature-induced loads acting on beams, swelling- induced loads acting within specified regions and monitoring ofresults.

1st stage construction method

Geostatic Stress: - Usual methods by which vertical geostatic stress is assessed works on the basis of equation given below:

$$\sigma_Z = \sum_{i=1}^{n} \gamma_i \,.\, h_i . [kpa]$$

Where

γ_i– soil density in i^{th} layer

h_i– heightof i^{th}layer

K_0 method: Situations in which specifications of other primary sideways force becomes necessary for user this method is employed. In comparison to usually combined earth, it becomes possible that the real sideways force of overly combined earth is high. K_0 exists in the form of soil variable. It is the coefficient of is side Ways force.

$$k_0 = \frac{\mu}{1-\mu}$$

Where , μ = poisons ratio

In situation where this variable is not defined it is calculated by the given equation

Earth input variables are selected on the basis of material used for the purpose of analysis. In the examination of stress not only Elastic modulus, but Poisson's ratio is also considered in the form of major input variables.

CHAPTER 4

RESULTS& DISSCUSION

InputData:-

In this work following data is used to analysis of retaining wall and these data has been taken from the Lab manual, (1990). “Manual on estimating soil properties for foundation design.[10]

Different type of soil condition, which properties are given here:-

1. **Clay[10]: -** Y= 19.0 KN/m^3, Y_{sat}= 20.17 KN/m^3, $Ø_{ef}$= 0^o, C_{ef}= 45kpa, Bearing Capacity of soil=250KPa, E_S=80 MPa, μ=0.3
2. **C-Phi[10]:-**Y= 18.00 KN/m^3, Ysat= 19.62 KN/m^3, $Ø_{ef}$= 19^0, C_{ef}= 26.00 kpa, Bearing Capacity of Soil= 350kpa, E_S=50 MPa, μ=0.3
3. **Sand[10]:-**Y= 18.00 KN/m^3, Y_{sat}= 21.00 KN/m^3, $Ø_{ef}$= 39.50^0, C_{ef}= 0 kPa, Bearing Capacity of soil= 450kPa, E_S=50 MPa, μ=0.3

Factor of safety for different condition;

1. For stability=1.5
2. For Slip condition=1.5
3. For overturning =2.0
4. Safety for Bearing capacity of soil =2.5

Dimension of different retaining wall

Here k_1=0.8m, K_2= 3.5m, K_3= 0.5m, S_1= 4.00, S_1 = S_2 =0

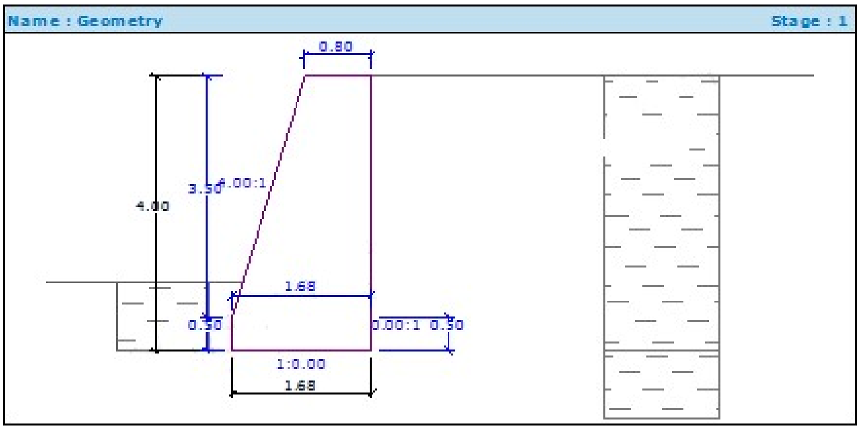

Figure 4.1 Dimension of 4m Gravity wall

5m, Gravity wall

Here ; k1=1m, K2= 4.5m, K3= 0.5m, S1= 4.00, S1 = S2 =0

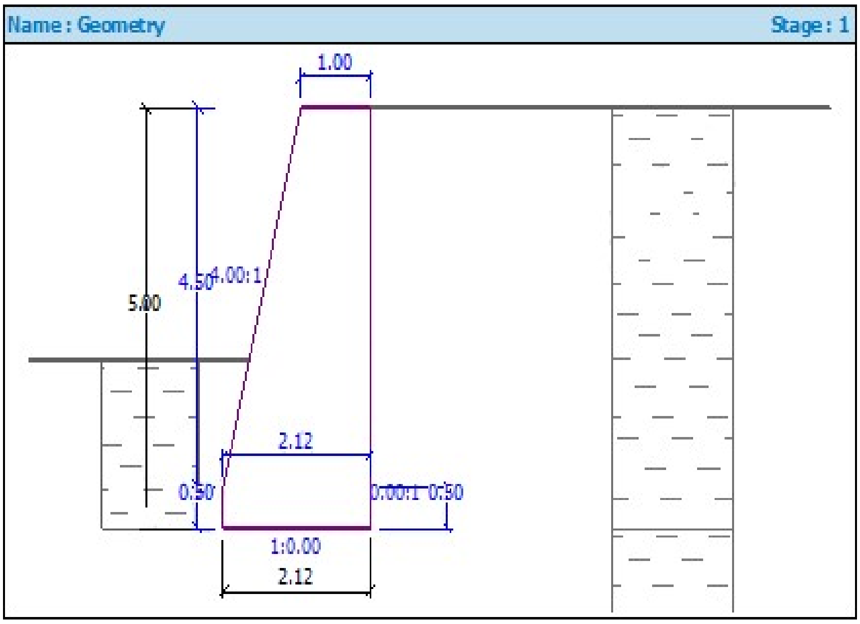

Figure 4.2 Dimension of 5m Gravity wall

6m, Gravity wall

Here ; k1=1m, K2= 5.5m, K3= 0.5m, S1= 4.00, S1 = S2 =0

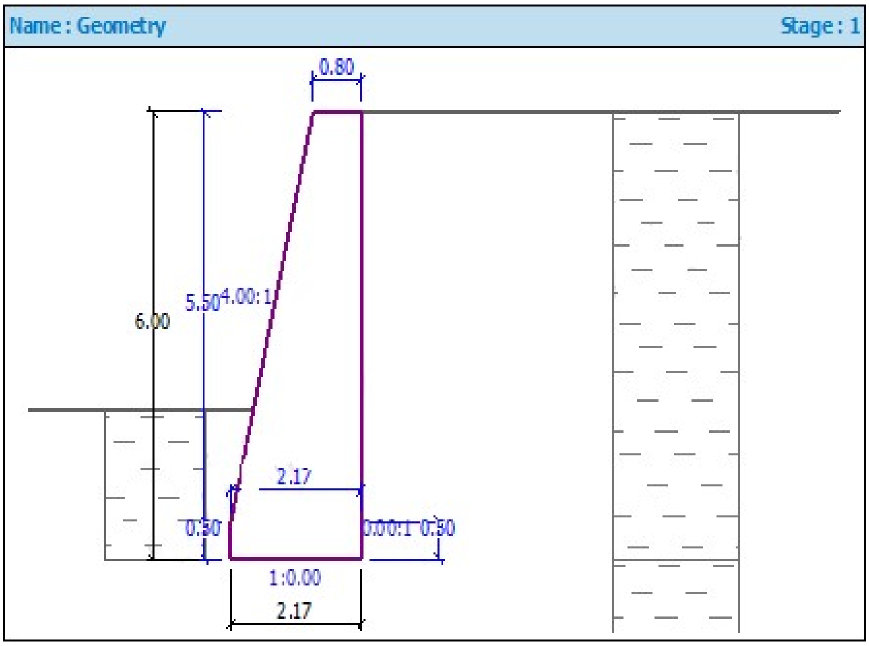

Figure 4.3 Dimension of 6m Gravity wall

Slope stability

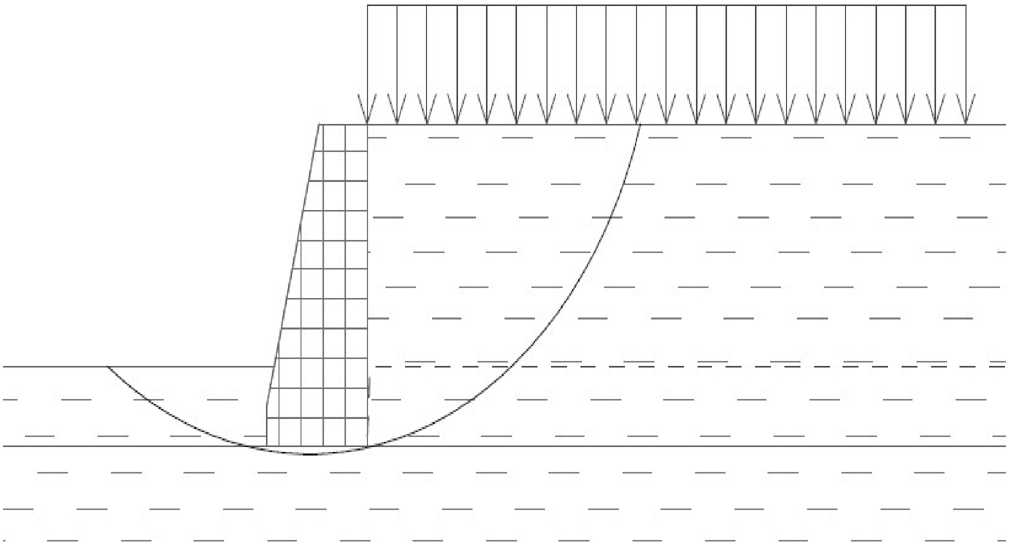

Figure 4.4 Slope Stability Analysis

Verification load condition of wall

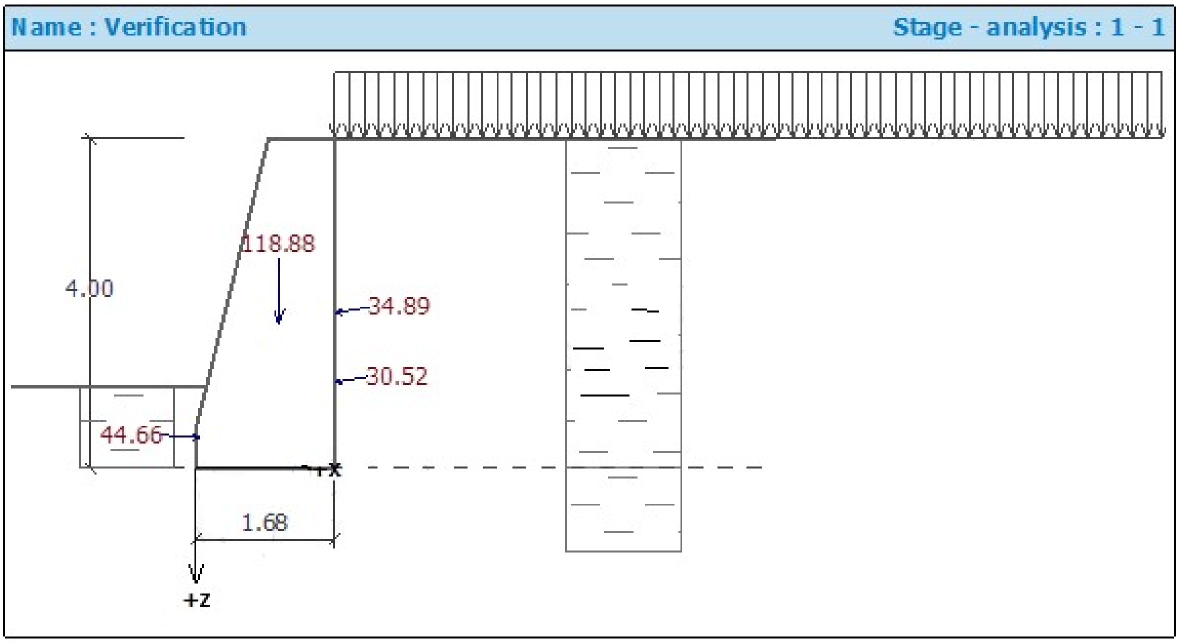

Figure 4.5 Verification load

Bearing capacity of soil

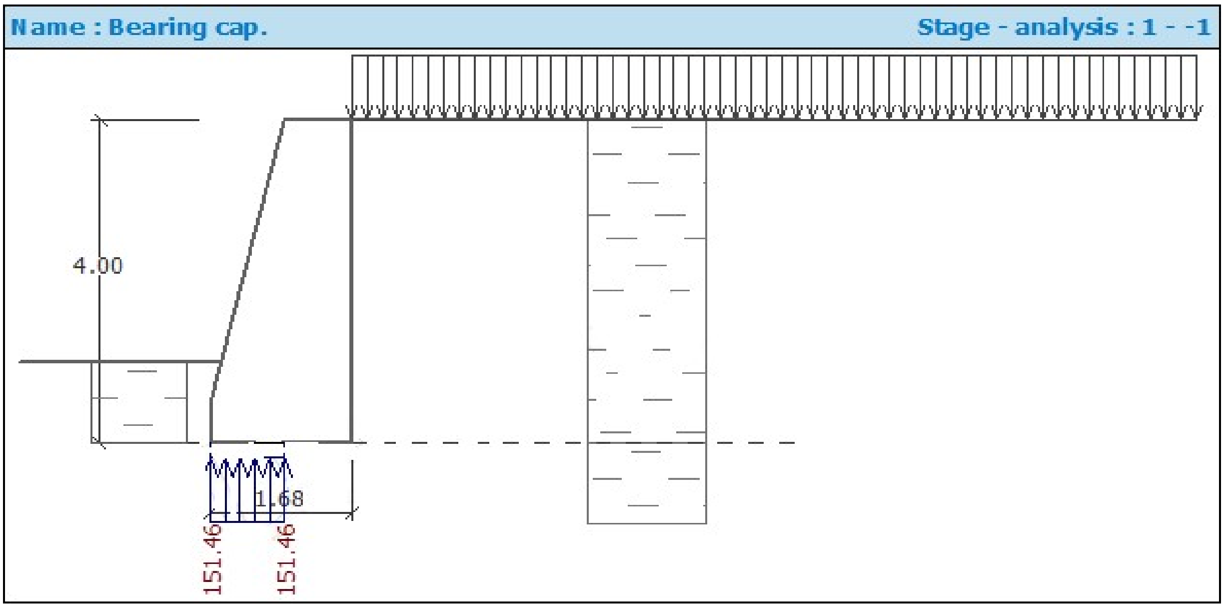

Figure 4.6 Load given by soil

Dimensioning Load condition

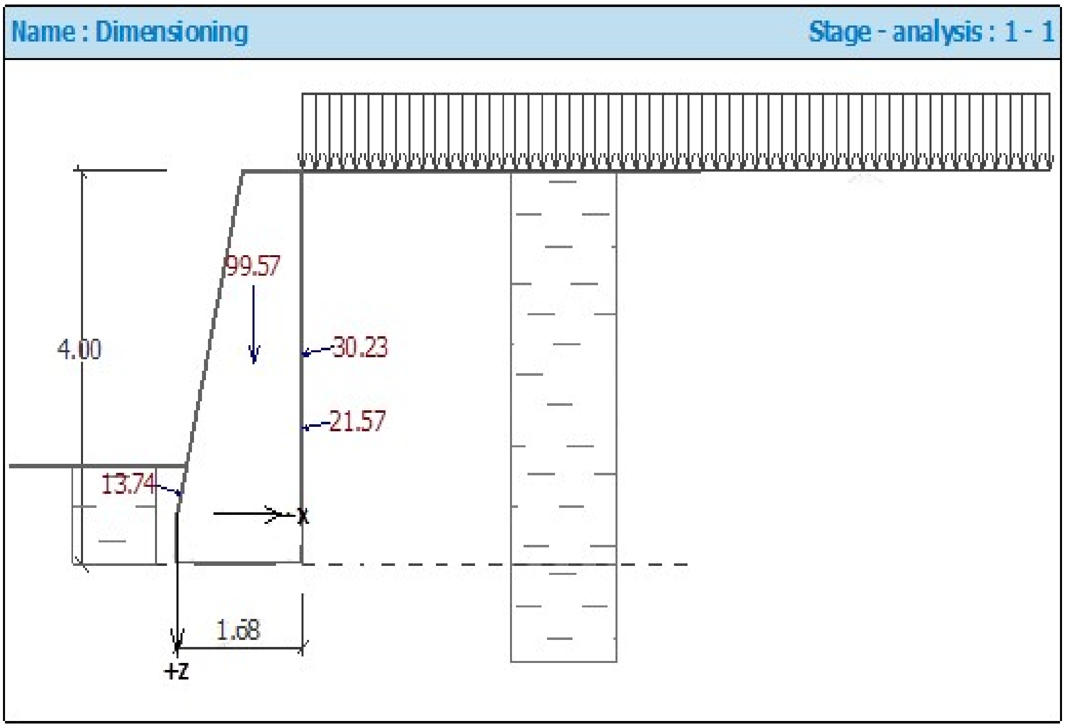

Fig 4.7 Dimensioning load

4.1 Results

Case:-1. 4m Gravity wall with Clay soil, surcharge=30kpa is variable and Horizontal terrain, FFres=1m

Table - 4.1

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	4m BS
Verification	Overturning	Mres	119.49	98.45	112.48	119.49
		Movr	-45.56	-2.12	-45.07	-45.56
		Satis	Y	Y	Y	Y
	Slip	Hres	75.38	75.38	75.38	75.38
		Hact	-92.18	-53.10	-91.23	-92.18
		Satis	Y	Y	Y	Y
	Force Acting	M	-55.86	-5.41	-53.04	-55.86
		N	130.37	113.62	124.79	130.37
		Q	-92.18	-53.10	-91.23	-92.18
		Satis	Y	Y	Y	Y
	Overall Satisfactory		Y	Y	Y	Y
Bearing Capacity Of Foundation Soil	Eccentricity	E	0	O	0	0
		Ealw	552.8	552.8	552.8	552.8
		Satis	Y	Y	Y	Y
	Foundation Soil	б	77.83	67.83	74.50	77.83
		Rd	250	250	250	250
		Satis	Y	Y	Y	Y
	Overall Satisfactory		Y	Y	Y	Y
Wall Stem Check	Shear	Vu	-45.06	-14.63	-44.55	-45.06
		Vrd	468.91	468.91	468.91	468.91
		Satis	Y	Y	Y	Y
	Pressure + Flexure	Mu	-21.48	10.29	-20.18	-21.48
		Pu	111.03	97.09	108.25	111.03
		Prd	11504.19	13067.51	11629.54	11504.19
	Overall Satisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		187.79	192.69	188.33	187.79
	Fp in KN/m		493.99	474.59	493.13	493.99
	Ma in KN-m/m		1087.42	1024.78	1087.60	1087.42
	MP in KN-m/m		2860.48	2524.00	2847.75	2860.48
	FOS		2.63	2.46	2.62	2.63
	Satisfactory		Y	Y	Y	Y

Table 4.2

Case:-2. 4m Gravity wall with Clay soil, surcharge=30kpa is variable and 5^0 slope FF res=1m

S.N			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	4m BS	4m BS & 1Mfs	4m BS, 3m FS	4m BS & 4mFS
Verification	Overturning	Mres	119.49	98.45	112.48	119.49	112.47	117.14	119.49
		Movr	-45.39	-2.12	-44.97	-45.39	-88.94	-45.61	-45.39
		Satis	Y	Y	Y	Y	Y	Y	Y
	Slip	Hres	75.38	75.38	75.38	75.38	75.38	75.38	75.38
		Hact	-91.77	-53.10	-90.90	-91.77	-132.3	-92.38	-91.77
		Satis	Y	Y	Y	Y	Y	Y	Y
	Force Acting	M	-55.70	-5.41	-52.94	-55.70	-106.4	-58.47	-55.70
		N	130.37	113.62	124.79	130.37	113.36	124.52	130.37
		Q	-91.77	-53.10	-90.90	-91.77	-132.3	-92.38	-91.77
		Satis	Y	Y	Y	Y	Y	Y	Y
	OverallSatisfactory		Y	Y	Y	Y	Y	Y	Y
Bearing Capacity Of Foundation Soil	Eccentricity	E	0	0	0	0	0	0	0
		Ealw	552.8	552.8	552.8	552.8	552.8	552.8	552.8
		Satis	Y	Y	Y	Y	Y	Y	Y
	Foundation Soil	Ϭ	77.83	67.83	74.75	77.83	67.68	74.34	77.83
		Rd	250	250	250	250	250	250	250
		Satis	Y	Y	Y	Y	Y	Y	Y
	OverallSatisfactory		Y	Y	Y	Y	Y	Y	Y
Wall Stem Check	Shear	Vu	-44.90	-14.63	-44.48	-44.90	-75.05	-45.08	-44.90
		Vrd	468.91	468.91	468.91	468.91	468.91	468.91	468.91
		Satis	Y	Y	Y	Y	Y	Y	Y
	Pressure + Flexure	Mu	-21.46	10.29	-20.18	-21.46	-53.33	-22.86	-21.46
		Pu	111.03	97.09	108.25	111.03	96.82	107.98	111.03
		Prd	11507.98	13067.51	11630.29	11507.98	5118.34	11177.58	11507.98
	Overall Satisfactory		Y	Y	Y	Y	Y	Y	Y
Slope Stability	Fa in KN/m		403.52	454.08	432.02	403.51	202.51	421.53	403.52
	Fp in KN/m		998.17	1067.80	1058.37	998.17	588.12	1049.09	998.17
	Ma in KN-m/m		4411.85	5040.44	4850.62	4411.85	1520.12	4790.17	4411.85
	MP in KN-m/m		10913.36	11852.90	11883.09	10913.36	4414.68	11921.57	10913.36
	FOS		2.47	2.35	2.45	2.47	2.90	2.49	2.47
	Satisfactory		Y	Y	Y	Y	Y	Y	Y

Table 4.3

Case:-3. 4m Gravity wall with clay soil, surcharge=30kpa is variable and 10^{O} slope FF res=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	5m BS
Verification	Overturning	Mres	119.49	98.45	112.48	119.49
		Movr	-45.39	-2.12	-44.97	-45.39
		Satis	Y	Y	Y	Y
	Slip	Hres	75.38	75.38	75.38	75.38
		Hact	-91.77	-53.10	-90.90	-91.77
		Satis	Y	Y	Y	Y
	Force Acting	M	-55.70	-5.41	-52.94	-55.70
		N	130.37	113.62	124.79	130.37
		Q	-91.77	-53.10	-90.90	-91.77
		Satis	Y	Y	Y	Y
	Overall Satisfactory		Y	Y	Y	Y
Bearing Capacity Of Foundation Soil	Eccentricity	E	0	0	0	0
		ealw	552.8	552.8	552.8	552.8
		Satis	Y	Y	Y	Y
	Foundation Soil	Ϭ	77.83	67.83	74.75	77.83
		Rd	250	250	250	250
		Satis	Y	Y	Y	Y
	Overall Satisfactory		Y	Y	Y	Y
Wall Stem Check	Shear	Vu	-44.90	-14.63	-44.48	-44.90
		Vrd	468.91	468.91	468.91	468.91
		Satis	Y	Y	Y	Y
	Pressure + Flexure	Mu	-21.46	10.29	-20.18	-21.46
		Pu	111.03	97.09	108.25	111.03
		Prd	11507.98	13067.51	11630.29	11507.98
	Overall Satisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		415.70	462.52	406.13	415.70
	Fp in KN/m		932.70	994.35	906.65	932.70
	Ma in KN-m/m		4293.70	4878.66	4199.82	4293.70
	MP in KN-m/m		9633.72	10488.36	9375.64	9633.72
	FOS		2.24	2.15	2.23	2.24
	Satisfactory		Y	Y	Y	Y

Table 4.4

Case:-1. 4m Gravity wall with C-phi soil, surcharge=30kpa is variable and Horizontal terrain condition, FF res =1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	4m BS
Verification	Overturning	Mres	142.30	116.88	134.62	142.30
		Movr	43.23	68.13	44.13	43.23
		Satis	Y	Y	Y	Y
	Slip	Hres	57.54	42.61	54.97	57.54
		Hact	-1.05	23.99	1.71	-1.05
		Satis	Y	Y	Y	Y
	Force Acting	M	8.37	42.48	11.93	8.37
		N	128.28	108.92	122.30	128.28
		Q	-1.05	23.99	1.71	-1.05
		Satis	Y	Y	Y	Y
	Overall Satisfactory		Y	Y	Y	Y
Bearing Capacity Of Foundation Soil	Eccentricity	E	65.3	390.0	97.6	65.3
		ealw	552.8	552.8	552.8	552.8
		Satis	Y	Y	Y	Y
	Foundation Soil	б	83.06	121.70	82.65	83.06
		Rd	300	300	300	300
		Satis	Y	N	Y	Y
	Overall Satisfactory		Y	N	Y	Y
Wall Stem Check	Shear	Vu	19.18	36.53	19.84	19.18
		Vrd	468.91	468.91	468.91	468.91
		Satis	Y	Y	Y	Y
	Pressure + Flexure	Mu	11.12	32.58	12.47	11.12
		Pu	112.52	97.03	109.64	112.52
		Prd	13194.76	8960.85	12929.21	13194.76
	Overall Satisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		179.73	176.81	179.93	179.94
	Fp in KN/m		296.42	253.81	291.40	295.61
	Ma in KN-m/m		980.86	804.65	968.78	981.80
	MP in KN-m/m		1617.69	1155.05	1568.96	1612.93
	FOS		1.65	1.44	1.62	1.64
	Satisfactory		Y	N	Y	Y

Table 4.5

Case:-2. 4m Gravity wall with C-phi soil, surcharge=30kpa is variable and 5^{O} slope FF res =1m

S.N			1	2	3	4	5	6	7
Water Condition			**No water**	**1m BS**	**3m BS**	**4m BS**	**4m BS & 1m FS**	**4m BS & 3m FS**	**4m BS & 4m FS**
Verification	Overturning	Mres	144.61	118.45	136.87	144.61	137.45	142.26	144.61
		Movr	52.83	75.25	53.64	52.83	9.73	53.87	52.83
		Satis	Y	Y	Y	Y	Y	Y	Y
	Slip	Hres	55.98	41.23	53.33	55.98	55.63	54.72	55.98
		Hact	6.26	29.09	8.80	6.26	-30.61	9.75	6.26
		Satis	Y	N	Y	Y	Y	Y	Y
	Force Acting	M	16.81	48.81	20.32	16.81	-32.36	16.38	16.81
		N	129.67	109.86	123.65	129.67	113.86	125.10	129.67
		Q	6.26	29.09	8.80	6.26	-30.61	9.75	6.26
		Satis	Y	N	Y	Y	Y	Y	Y
	Overall Satisfactory		Y	N	Y	Y	Y	Y	Y
Bearing Capacity Of Foundation Soil	Eccentricity	E	129.6	444.3	164.4	129.6	0	131.0	129.6
		Ealw	552.8	552.8	552.8	552.8	552.8	552.8	552.8
		Satis	Y	Y	Y	Y	Y	Y	Y
	Foundation Soil	Ϭ	91.59	139.70	91.85	91.59	67.97	88.53	91.59
		Rd	300	300	300	300	300	300	300
		Satis	Y	N	Y	Y	Y	Y	Y
	Overall Satisfactory		Y	N	Y	Y	Y	Y	Y
Wall Stem Check	Shear	Vu	24.76	40.52	25.37	24.76	-4.89	25.41	24.76
		Vrd	468.91	468.91	468.91	468.91	468.91	468.91	468.91
		Satis	Y	Y	Y	Y	Y	Y	Y
	Pressure + Flexure	Mu	16.59	36.81	17.93	16.59	-15.63	15.41	16.59
		Pu	113.60	97.78	110.71	113.60	99.44	110.68	113.60
		Prd	12351.58	8234.62	12067.11	12351.58	12151.57	12473.46	12351.58
	Overall Satisfactory		Y	Y	Y	Y	Y	Y	Y
Slope Stability	Fa in KN/m		202.25	203.18	202.61	198.18	198.23	204.34	198.18
	Fp in KN/m		323.06	282.72	318.51	315.55	339.50	314.97	315.55
	Ma in KN-m/m		1233.16	1094.46	1224.02	1162.06	1247.23	1284.21	1162.06
	MP in KN-m/m		1969.8	1522.93	1924.17	1850.31	2136.11	1979.46	1850.3
	FOS		1.60	1.39	1.57	1.59	1.71	1.54	1.59
	Satisfactory		Y	N	Y	Y	Y	Y	Y

Table 4.6

Case:-3. 4m Gravity wall with C-phi soil, surcharge=30kpa is variable and 10^{O} slope FF res =1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	4m BS
Verification	Overturning	Mres	148.06	120.68	140.22	148.06
		Movr	68.79	87.58	69.49	68.79
		Satis	Y	N	Y	Y
	Slip	Hres	53.35	38.72	50.56	53.35
		Hact	18.16	37.67	20.37	18.16
		Satis	Y	Y	Y	Y
	Force Acting	M	31.05	60.03	34.49	31.05
		N	131.72	111.19	125.65	131.72
		Q	18.16	37.67	20.37	18.16
		Satis	Y	N	Y	Y
	Overall Satisfactory		Y	N	Y	Y
Bearing Capacity Of Foundation Soil	Eccentricity	e	235.7	539.9	274.5	235.7
		ealw	552.8	552.8	552.8	552.8
		Satis	Y	Y	Y	Y
	Foundation Soil	б	109.44	186.79	111.59	109.44
		Rd	300	300	300	300
		Satis	Y	N	Y	Y
	Overall Satisfactory		Y	N	Y	Y
Wall Stem Check	Shear	Vu	33.92	47.31	34.44	33.92
		Vrd	468.91	468.91	468.91	468.91
		Satis	Y	Y	Y	Y
	Pressure + Flexure	Mu	25.97	44.45	27.30	25.97
		Pu	115.18	98.79	112.26	115.18
		Prd	10931.63	6922.66	10615.20	10931.63
	Overall Satisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		235.86	251.89	238.34	234.51
	Fp in KN/m		362.33	345.89	360.83	359.48
	Ma in KM-m/m		1644.96	1766.92	1672.86	1623.63
	MP in KN-m/m		2527.04	2426.28	2532.62	2488.85
	FOS		1.54	1.37	1.51	1.53
	Satisfactory		Y	N	Y	Y

Table 4.7

Case:-1. 4m Gravity wall with Sand soil, surcharge=30kpa is variable and Horizontal terrain with FF res=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	4m BS
Verification	Overturning	Mres	146.75	121.65	139.29	146.75
		Movr	64.26	101.34	65.64	64.26
		Satis	Y	N	Y	Y
	Slip	Hres	94.62	78.82	89.80	94.62
		Hact	-24.77	12.31	-20.65	-24.77
		Satis	Y	Y	Y	Y
	Force Acting	M	13.64	59.77	17.58	13.64
		N	114.78	95.61	108.93	114.78
		Q	-24.77	12.31	-20.65	-24.77
		Satis	Y	N	Y	Y
	OverallSatisfactory		Y	N	Y	Y
Bearing Capacity Of Foundation Soil	Eccentricity	E	118.9	625.1	161.4	118.9
		Ealw	552.8	552.8	552.8	552.8
		Satis	Y	N	Y	Y
	Foundation Soil	Ϭ	79.86	225.05	80.56	79.86
		Rd	450	450	450	450
		Satis	Y	N	Y	Y
	OverallSatisfactory		Y	N	Y	Y
Wall Stem Check	Shear	Vu	30.73	56.46	31.76	30.73
		Vrd	468.91	468.91	468.91	468.91
		Satis	Y	Y	Y	Y
	Pressure + Flexure	Mu	29.84	58.51	31.23	29.84
		Pu	112.43	96.80	109.58	112.43
		Prd	10219.17	4161.31	9868.81	10219.17
	OverallSatisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		202.13	198.48	188.88	181.89
	Fp in KN/m		444.88	347.96	394.65	388.06
	Ma in KN-m/m		1273.76	1056.48	1024.22	934.11
	MP in KN-m/m		2803.53	1852.16	2139.95	1992.88
	FOS		2.20	1.75	2.09	2.13
	Satisfactory		Y	Y	Y	Y

Table 4.8

Case:-2. 4m Gravity wall with C-phi soil, surcharge=30kpa is variable and 5^{O} slope FF res =1m

S.N			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	4m BS	4m BS & 1m FS	4m BS & 3m FS	4m BS & 4m FS
Verification	Overturning	Mres	148.10	122.81	140.62	148.10	141.10	145.78	148.10
		Movr	68.57	105.27	69.93	68.57	34.25	77.59	68.57
		Satis	Y	Y	Y	Y	Y	N	Y
	Slip	Hres	95.29	79.39	90.45	95.29	89.20	98.40	95.29
		Hact	-22.13	14.56	-18.05	-22.13	-31.38	8.62	-22.13
		Satis	Y	Y	Y	Y	Y	Y	Y
	Force Acting	M	17.28	63.11	21.21	17.28	-16.23	31.78	17.28
		N	115.59	96.30	109.73	115.59	108.21	119.37	115.59
		Q	-22.13	14.56	-18.05	-22.13	-31.38	8.62	-22.13
		Satis	Y	N	Y	Y	Y	Y	Y
	Overall Satisfactory		Y	N	Y	Y	Y	Y	Y
Bearing Capacity Of Foundation Soil	Eccentricity	E	149.5	655.3	193.3	149.5	0	266.3	149.5
		ealw	552.8	552.8	552.8	552.8	552.8	552.8	552.8
		Satis	Y	N	Y	Y	Y	Y	Y
	Foundation Soil	б	84.00	264.35	85.16	84.00	64.60	104.48	84.00
		Rd	450	450	450	450	450	450	450
		Satis	Y	N	Y	Y	Y	Y	Y
	Overall Satisfactory		Y	N	Y	Y	Y	Y	Y
Wall Stem Check	Shear	Vu	32.88	58.34	33.89	32.88	7.79	37.77	32.88
		Vrd	468.91	468.91	468.91	468.91	468.91	468.91	468.91
		Satis	Y	Y	Y	Y	Y	Y	Y
	Pressure + Flexure	Mu	32.40	60.92	33.79	32.40	1.82	32.30	32.40
		Pu	113.08	97.37	110.23	113.08	99.45	110.62	113.08
		Prd	9842.17	3783.40	9483.82	9842.17	13397.54	9744.25	9842.17
	Overall Satisfactory		Y	Y	Y	Y	Y	Y	Y
Slope Stability	Fa in KN/m		204.85	199.14	205.36	204.07	212.78	216.09	204.07
	Fp in KN/m		435.83	353.04	420.89	425.95	438.22	401.04	425.95
	Ma in KN-m/m		1171.08	1028.95	1186.18	1172.74	1798.21	1489.57	1172.74
	MP in KN-m/m		2491.50	1824.17	2431.10	2447.82	3703.45	2764.50	2447.82
	FOS		2.13	1.77	2.05	2.09	2.06	1.86	2.09
	Satisfactory		Y	Y	Y	Y	Y	Y	Y

Table 4.9

Case:-3. 4m Gravity wall with Sand soil, surcharge=30kpa is variable and 10^{O} slope FF res =1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	4m BS
Verification	Overturning	Mres	149.65	124.13	142.14	149.65
		Movr	73.47	109.73	74.82	73.47
		Satis	Y	N	Y	Y
	Slip	Hres	96.05	80.04	91.20	96.05
		Hact	-19.11	17.14	-15.09	-19.11
		Satis	Y	Y	Y	Y
	Force Acting	M	21.41	66.91	25.34	21.41
		N	116.52	97.09	110.63	116.52
		Q	-19.11	17.14	-15.09	-19.11
		Satis	Y	N	Y	Y
	Overall Satisfactory		Y	N	Y	Y
Bearing Capacity Of Foundation Soil	Eccentricity	e	183.7	689.2	229.0	183.7
		ealw	552.8	552.8	552.8	552.8
		Satis	Y	N	Y	Y
	Foundation Soil	б	89.11	327.28	90.91	89.11
		Rd	450	450	450	450
		Satis	Y	N	Y	Y
	Overall Satisfactory		Y	N	Y	Y
Wall Stem Check	Shear	Vu	35.33	60.48	36.33	35.33
		Vrd	468.91	468.91	468.91	468.91
		Satis	Y	Y	Y	Y
	Pressure + Flexure	Mu	35.30	63.65	36.70	35.30
		Pu	113.83	98.03	110.97	113.83
		Prd	9419.57	3360.62	9052.40	9419.57
	Overall Satisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		273.16	244.06	232.14	254.52
	Fp in KN/m		585.78	417.85	464.01	525.90
	Ma in KN-m/m		2439.08	1572.29	1489.44	1992.84
	MP in KN-m/m		5230.46	2691.92	2977.19	4117.64
	FOS		2.14	1.71	2.00	2.07
	Satisfactory		Y	Y	Y	Y

4.2 Non- Homogeneous ConditionResults

Table 4.10

Case:-1; 4m Gravity wall Non-Homogeneous (Clay, Sand, C-Phi)condition, surcharge=30kpa is Horizontal terrain and FF resistance=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	4m BS
Verification	Overturning	Mres	94.27	75.71	89.0	94.27
		Movr	50.09	77.59	50.98	50.09
		Satis	N	N	N	N
	Slip	Hres	41.40	24.73	38.87	41.40
		Hact	10.11	35.86	12.88	10.11
		Satis	Y	N	Y	Y
	Force Acting	M	30.91	65.32	33.65	30.91
		N	109.22	92.29	104.24	109.22
		Q	10.11	35.86	12.88	10.11
	OverallSatisfactory		N	N	N	N
Bearing Capacity of foundation Soil	Eccentricity	e	283.0	707.8	322.8	283.0
		ealw	453.8	453.8	453.8	453.8
		Satis	Y	N	Y	Y
	Foundation Soil	б	135.01	10000	142.91	135.01
		Rd	450	450	450	450
		Satis	Y	N	Y	Y
	OverallSatisfactory		Y	N	Y	Y
Wall Stem check	Shear	Vu	24.83	43.05	25.50	24.83
		Vrd	384.91	384.91	384.91	384.91
	Pressure + Flexure	Mu	23.92	45.82	24.86	23.92
		Pu	90.47	76.94	88.08	90.47
		Prd	7557.56	1640.75	7237.45	7557.56
	OverallSatisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		165.97	177.56	174.13	167.75
	Fp in KN/m		282.38	254.26	292.40	284.49
	Ma in KN-m/m		752.94	767.46	841.09	777.49
	MP in KN-m/m		1281.03	1099.01	1412.37	1318.48
	F.O.S		1.70	1.43	1.68	1.70
	Satisfactory		Y	N	Y	Y

Table No. 4.11

Case:-2; 4m Gravity wall Non-Homogeneous (Clay, Sand, C-Phi)condition, surcharge=30kpa is varying with 5^0 Slope angle and FFresistance=1m

S.N			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	4m BS	4m BS & 1mFS	4m BS& 3mFS	4m BS & 4m FS
Verification	Overturning	Mres	95.47	76.32	90.15	95.47	90.73	93.87	95.47
		Movr	54.65	80.48	55.47	54.65	11.89	55.19	54.65
		Satis	N	N	N	N	Y	N	N
	Slip	Hres	40.62	24.85	38.06	40.62	46.33	39.20	40.62
		Hact	14.79	38.62	17.33	14.79	-23.29	16.68	14.79
		Satis	Y	N	Y	Y	Y	Y	Y
	Force Acting	M	34.87	67.90	37.56	34.87	-12.87	33.58	34.87
		N	110.09	92.73	105.07	110.09	95.95	105.11	110.09
		Q	14.79	38.62	17.33	14.79	-23.29	16.68	14.79
	OverallSatisfactory		N	N	N	N	Y	Y	N
Bearing Capacity of foundation Soil	Eccentricity	e	316.8	732.3	357.5	316.8	0	319.5	316.8
		ealw	453.8	453.8	453.8	453.8	453.8	453.8	453.8
		Satis	Y	N	Y	Y	Y	Y	Y
	Foundation Soil	б	148.47	10000	159.19	148.47	69.78	142.80	148.47
		Rd	450	450	450	450	450	450	450
		Satis	Y	N	Y	Y	Y	Y	Y
	OverallSatisfactory		Y	N	Y	Y	Y	Y	Y
Wall Stem check	Shear	Vu	28.06	45	28.67	28.06	-1.58	28.37	28.06
		Vrd	384.91	384.91	384.91	384.91	384.91	384.91	384.91
	Pressure + Flexure	Mu	26.08	47.32	27.02	26.08	-3.85	25.08	26.08
		Pu	91.09	77.24	88.69	91.09	79.24	88.40	91.09
		Prd	7164.84	1334.14	6836.35	7164.84	10997	7212.6	7164.84
	OverallSatisfactory		Y	Y	Y	Y	Y	Y	Y
Slope Stability	Fa in KN/m		173.72	213.97	179.35	174.52	183.04	171.37	174.52
	Fp in KN/m		296.76	317.79	301.74	297.26	366.89	283.80	297.26
	Ma in KN-m/m		792.41	1083.62	845.34	806.73	1187.9	774.84	806.73
	MP in KN-m/m		1353.69	1609.46	1422.20	1374.06	2381.2	1283.2	1374.06
	F.O.S		1.71	1.49	1.68	1.70	2.00	1.66	1.70
	Satisfactory		Y	N	Y	Y	Y	Y	Y

Table no.4.12

Case:-1;4m Gravity wall Non-Homogeneous (C-Phi, Clay, Sand)condition, surcharge=30kpa is varying with Horizontal terrain and FFresistance=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	4m BS
Verification	Overturning	Mres	94.72	77.61	89.62	94.72
		Movr	11.87	51.07	13.24	11.87
		Satis	Y	N	Y	Y
	Slip	Hres	68.91	55.82	64.91	68.91
		Hact	-44.06	-6.03	-39.94	-44.06
		Satis	Y	Y	Y	Y
	Force Acting	M	-25.37	20.02	-22.24	-25.37
		N	83.60	67.72	78.75	83.60
		Q	-44.06	-6.03	-39.94	-44.06
	OverallSatisfactory		Y	N	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	e	0	295.6	0	0
		ealw	453.8	453.8	453.8	453.8
		Satis	Y	Y	Y	Y
	Foundation Soil	Ϭ	60.80	86.40	57.27	60.80
		Rd	450	450	450	450
		Satis	Y	Y	Y	Y
	OverallSatisfactory		Y	Y	Y	Y
Wall Stem check	Shear	Vu	11.75	38.40	12.77	11.75
		Vrd	384.91	384.91	384.91	384.91
	Pressure + Flexure	Mu	-3.15	24.82	-2.14	-3.15
		Pu	84.79	71.95	82.44	84.79
		Prd	10997.5	6116.1	10997.5	10997.5
	OverallSatisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		175.93	209.94	171.83	175.74
	Fp in KN/m		401.90	398.57	376.06	394.30
	Ma in KN-m/m		918.44	1435.7	856.48	915.52
	MP in KN-m/m		2098.12	2725.7	1874.5	2054.14
	F.O.S		2.28	1.90	2.19	2.24
	Satisfactory		Y	Y	Y	Y

Table no.4.13

Case:-2; 4m Gravity wall Non-Homogeneous (C-Phi, Clay, Sand)condition, surcharge=30kpa is varying with 5^0 Slope angle and FFresistance=1m

			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	4m BS	4m BS& 1mFS	4m BS & 3m FS	4m BS & 4m FS
Verification	Overturning	Mres	95.74	78.50	90.63	95.74	91.03	94.18	95.74
		Movr	14.61	53.56	15.97	14.61	-19.71	23.62	14.61
		Satis	Y	N	Y	Y	Y	Y	Y
	Slip	Hres	69.53	56.36	65.52	69.53	65.91	73.47	69.53
		Hact	-41.95	-4.23	-37.87	-41.95	-51.21	-11.21	-41.95
		Satis	Y	Y	Y	Y	Y	Y	Y
	Force Acting	M	-23.15	22.06	-20.02	-23.15	-55.77	-9.28	-23.15
		N	84.34	68.37	79.48	84.34	79.96	89.12	84.34
		Q	-41.95	-4.23	-37.87	-41.95	-51.21	-11.21	-41.95
	Overall Satisfactory		Y	N	Y	Y	Y	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	E	0	322.6	0	0	0	0	0
		ealw	453.8	453.8	453.8	453.8	453.8	453.8	453.8
		Satis	Y	Y	Y	Y	Y	Y	Y
	Foundation Soil	б	61.34	93.69	57.80	61.34	58.15	64.82	61.34
		Rd	450	450	450	450	450	450	450
		Satis	Y	Y	Y	Y	Y	Y	Y
	Overall Satisfactory		Y	Y	Y	Y	Y	Y	Y
Wall Stem check	Shear	Vu	13.34	39.80	14.36	13.34	-11.74	18.23	13.34
		Vrd	384.91	384.91	384.91	384.91	384.91	384.91	384.91
	Pressure + Flexure	Mu	-1.74	26.15	-0.74	-1.74	-30.45	-1.51	-1.74
		Pu	85.38	72.47	83.02	85.38	74.25	83.41	85.38
		Prd	10997.5	5834.6	10997.5	10997.5	4952.6	10997.5	10997.5
	Overall Satisfactory		Y	Y	Y	Y	Y	Y	Y
Slope Stability	Fa in KN/m		210.92	220.17	207.87	203.55	175.97	208.41	203.55
	Fp in KN/m		462.30	400.14	439.24	439.42	395.51	400.48	439.42
	Ma in KN-m/m		1375.73	1439.4	1333.4	1276.5	1177.7	1384.5	1276.5
	MP in KN-m/m		3015.39	2615.9	2817.6	2755.6	2647.1	2660.6	2755.6
	F.O.S		2.19	1.82	2.11	2.16	2.25	1.92	2.16
	Satisfactory		Y	Y	Y	Y	Y	Y	Y

Table no. 4.14

Case:-1; 4m Gravity wall Non-Homogeneous (Sand, C-Phi, Clay)condition, surcharge=30kpa is varying with Horizontal terrain and FFresistance=1m

<table>
<tr><td>S.N</td><td colspan="2"></td><td>1</td><td>2</td><td>3</td><td>4</td></tr>
<tr><td>Water Condition</td><td colspan="2"></td><td>No water</td><td>1m BS</td><td>3m BS</td><td>4m BS</td></tr>
<tr><td rowspan="10">Verification</td><td rowspan="3">Overturning</td><td>Mres</td><td>84.41</td><td>70.22</td><td>79.68</td><td>84.41</td></tr>
<tr><td>Movr</td><td>7.03</td><td>45.23</td><td>7.51</td><td>7.03</td></tr>
<tr><td>Satis</td><td>Y</td><td>N</td><td>Y</td><td>Y</td></tr>
<tr><td rowspan="3">Slip</td><td>Hres</td><td>61.68</td><td>23.90</td><td>61.88</td><td>61.68</td></tr>
<tr><td>Hact</td><td>-73.65</td><td>-36.87</td><td>-72.71</td><td>-73.65</td></tr>
<tr><td>Satis</td><td>Y</td><td>Y</td><td>Y</td><td>Y</td></tr>
<tr><td rowspan="3">Force Acting</td><td>M</td><td>-3.22</td><td>39.71</td><td>-1.17</td><td>-3.22</td></tr>
<tr><td>N</td><td>107.87</td><td>94.11</td><td>103.29</td><td>107.87</td></tr>
<tr><td>Q</td><td>-73.65</td><td>-36.87</td><td>-72.71</td><td>-73.65</td></tr>
<tr><td colspan="2">Overall Satisfactory</td><td>Y</td><td>Y</td><td>Y</td><td>Y</td></tr>
<tr><td rowspan="7">Bearing Capacity of foundation Soil</td><td rowspan="3">Eccentricity</td><td>e</td><td>0</td><td>421.9</td><td>0</td><td>0</td></tr>
<tr><td>ealw</td><td>453.8</td><td>453.8</td><td>453.8</td><td>453.8</td></tr>
<tr><td>Satis</td><td>Y</td><td>Y</td><td>Y</td><td>Y</td></tr>
<tr><td rowspan="3">Foundation Soil</td><td>σ</td><td>78.45</td><td>177.20</td><td>75.12</td><td>78.45</td></tr>
<tr><td>Rd</td><td>450</td><td>450</td><td>450</td><td>450</td></tr>
<tr><td>Satis</td><td>Y</td><td>Y</td><td>Y</td><td>Y</td></tr>
<tr><td colspan="2">Overall Satisfactory</td><td>Y</td><td>Y</td><td>Y</td><td>Y</td></tr>
<tr><td rowspan="6">Wall Stem check</td><td rowspan="2">Shear</td><td>Vu</td><td>-26.56</td><td>1.61</td><td>-26.07</td><td>-26.56</td></tr>
<tr><td>Vrd</td><td>384.91</td><td>384.91</td><td>384.91</td><td>384.91</td></tr>
<tr><td rowspan="3">Pressure + Flexure</td><td>Mu</td><td>21.89</td><td>47.65</td><td>22.80</td><td>21.89</td></tr>
<tr><td>Pu</td><td>91.99</td><td>80.53</td><td>89.70</td><td>91.99</td></tr>
<tr><td>Prd</td><td>8029.39</td><td>1708.0</td><td>7738.5</td><td>8029.39</td></tr>
<tr><td colspan="2">Overall Satisfactory</td><td>Y</td><td>Y</td><td>Y</td><td>Y</td></tr>
<tr><td rowspan="6">Slope Stability</td><td colspan="2">Fa in KN/m</td><td>179.91</td><td>183.81</td><td>178.78</td><td>179.91</td></tr>
<tr><td colspan="2">Fp in KN/m</td><td>417.18</td><td>391.86</td><td>413.38</td><td>417.18</td></tr>
<tr><td colspan="2">Ma in KN-m/m</td><td>954.32</td><td>870.19</td><td>913.93</td><td>954.32</td></tr>
<tr><td colspan="2">MP in KN-m/m</td><td>2214.44</td><td>1855.6</td><td>2138.6</td><td>2214.44</td></tr>
<tr><td colspan="2">F.O.S</td><td>2.32</td><td>2.13</td><td>2.31</td><td>2.32</td></tr>
<tr><td colspan="2">Satisfactory</td><td>Y</td><td>Y</td><td>Y</td><td>Y</td></tr>
</table>

Table no. 4.15

Case:-2; 4m Gravity wall Non-Homogeneous (Sand, C-Phi, Clay)condition, surcharge=30kpa is varying with 5^0 slope angle and FFresistance=1m

S.N			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	4m BS	4m BS & 1m FS	4m BS & 3m FS	4m BS & 4m FS
Verification	Overturning	Mres	84.76	70.57	80.03	84.76	80.02	83.17	84.76
		Movr	11.63	48.81	11.95	11.63	-31.92	11.41	11.63
		Satis	Y	Y	Y	Y	Y	Y	Y
	Slip	Hres	60.87	20.75	59.18	60.87	61.88	61.88	60.87
		Hact	-70.97	-35.57	-70.24	-70.97	-111.59	-71.59	-70.97
		Satis	Y	Y	Y	Y	Y	Y	Y
	Force Acting	M	1.21	43.12	3.10	1.21	-47.24	-0.76	1.21
		N	108.12	94.36	103.54	108.12	94.11	103.27	108.12
		Q	-70.97	-35.57	-70.24	-70.97	-111.59	-71.59	-70.97
	Overall Satisfactory		Y	Y	Y	Y	Y	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	E	11.2	457.0	30.0	11.2	0	0	11.2
		Ealw	453.8	453.8	453.8	453.8	453.8	453.8	453.8
		Satis	Y	N	Y	Y	Y	Y	Y
	Foundation Soil	б	79.93	204.69	78.74	79.93	68.44	75.11	79.93
		Rd	450	450	450	450	450	450	450
		Satis	Y	N	Y	Y	Y	Y	Y
	Overall Satisfactory		Y	N	Y	Y	Y	Y	Y
Wall Stem check	Shear	Vu	-24.58	2.91	-24.29	-24.58	-54.73	-24.76	-24.58
		Vrd	384.91	384.91	384.91	384.91	384.91	384.91	384.91
	Pressure + Flexure	Mu	25.15	50.42	26.01	25.15	-4.79	24.16	25.15
		Pu	92.24	80.78	89.96	92.24	80.53	89.69	92.24
		Prd	7410.06	1129.91	7114.75	7410.06	10997.54	7468.12	7410.06
	Overall Satisfactory		Y	Y	Y	Y	Y	Y	Y
Slope Stability	Fa in KN/m		213.39	257.11	381.00	213.39	194.94	215.92	213.39
	Fp in KN/m		465.61	536.68	860.37	465.61	498.55	472.58	465.61
	Ma in KN-m/m		1313.71	1779.79	3887.37	1313.71	1333.14	1375.70	1313.71
	MP in KN-m/m		2866.48	3715.05	8778.51	2866.48	3409.47	3010.96	2866.48
	F.O.S		2.18	2.09	2.26	2.18	2.56	2.19	2.18
	Satisfactory		Y	Y	Y	Y	Y	Y	Y

4.3 CANTILEVER WALL RESULTS

Dimension of Cantilever wall

Here; K=0.5, h= 5.25m, xx= 0.75m, V1=1.20m, V2= 2.50m, S1=0

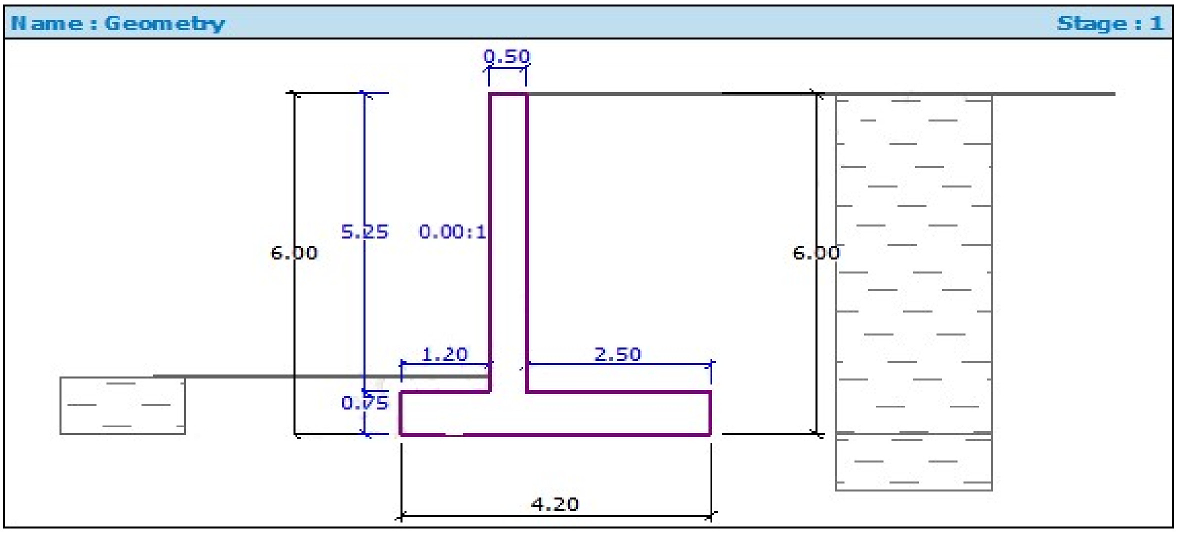

Fig 4.8 Dimension of 6m cantilever wall

Here; K=0.6, h= 6.20m, xx= 0.8m, V_1=1.40m, V_2= 2.50m, S_1=0

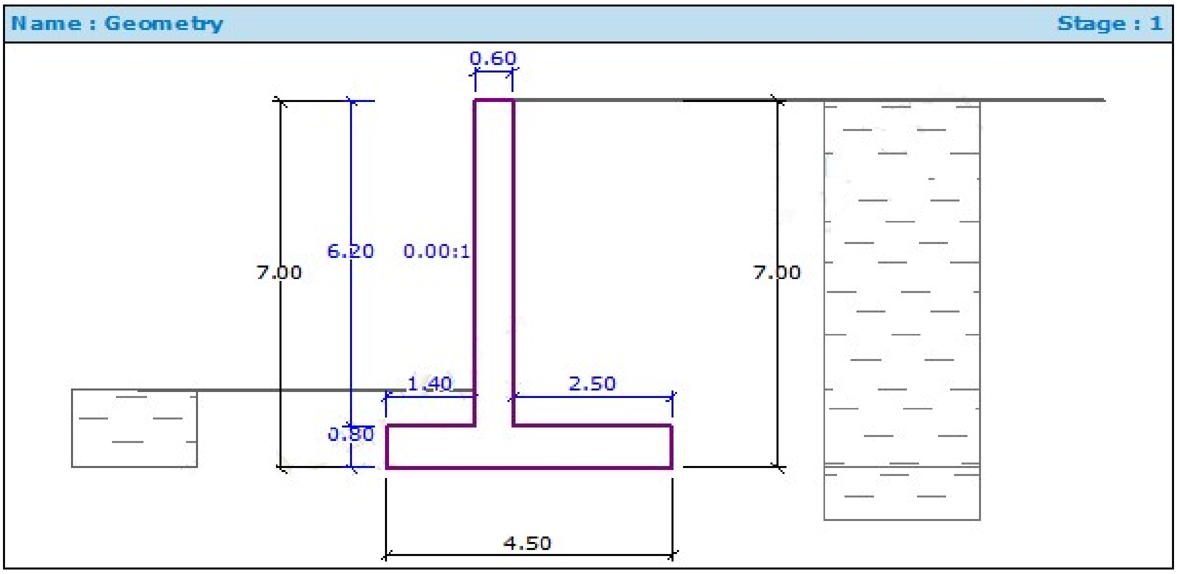

Fig 4.9 Dimension of 7m cantilever wall

Here; K=0.6, h= 7.0m, xx= 1.0m, V_1=2.0m, V_2= 3.0m, S_1=0

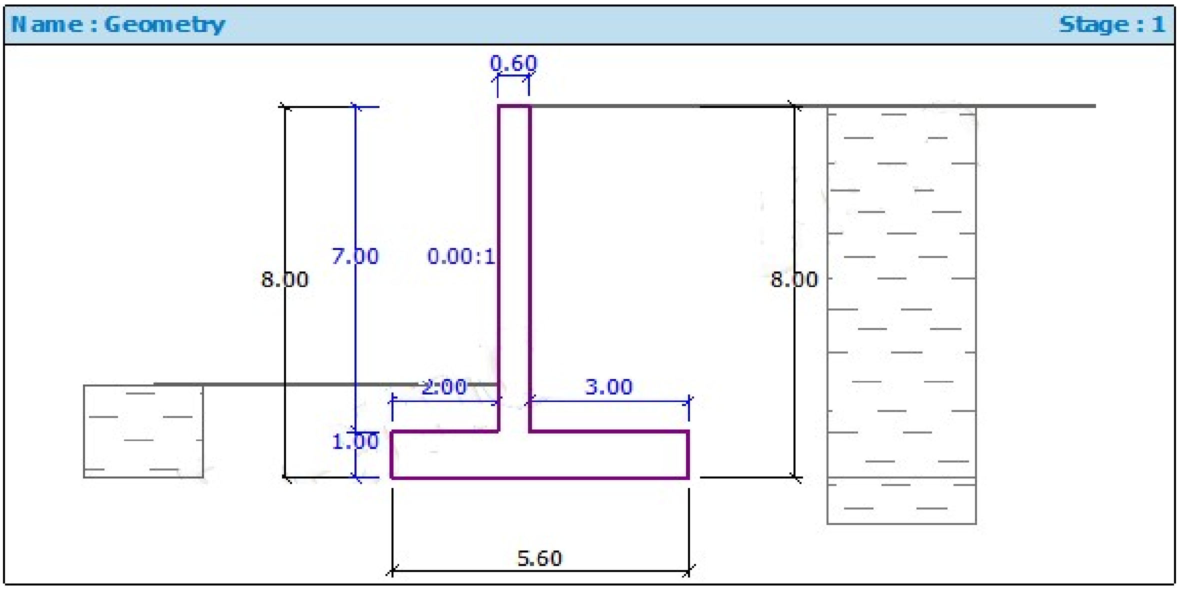

Fig 4.10 Dimension of 8m cantilever wall

Slope Stability analysis

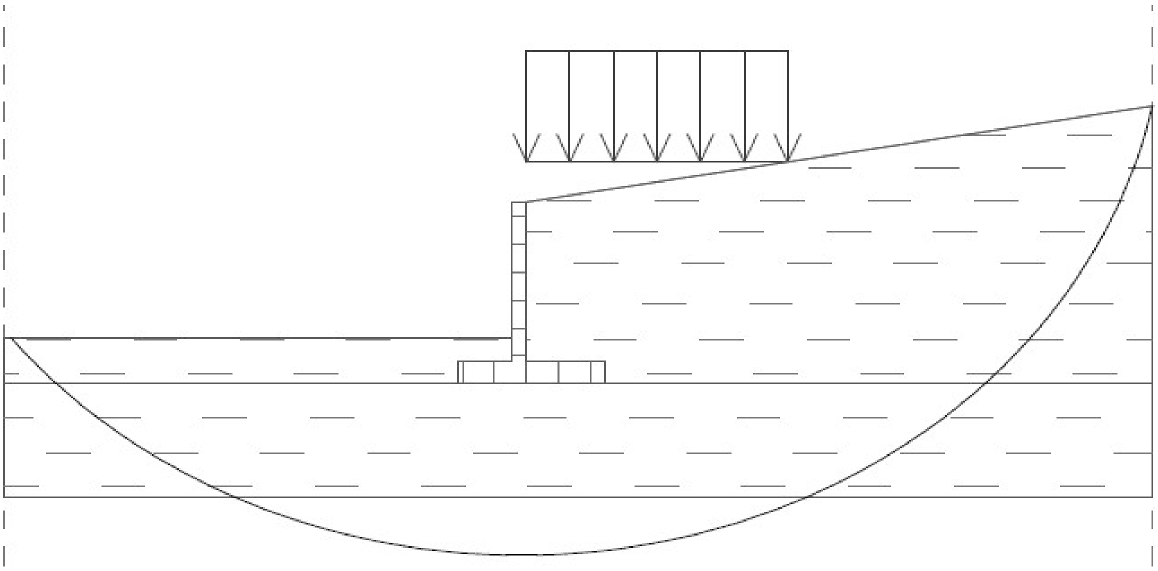

Fig 4.11 Slope stability analysis for cantilever wall

Table 4.16

Case:-1; 6m Cantilever wall, with Clay soil condition, surcharge=30kpa is varying with 5^0 Slope angle and FF resistance=1m

S.N			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	6m BS	6m BS & 1m FS	6m BS & 3m FS	6m BS & 6m FS
Verification	Overturning	Mres	992.91	495.65	714.09	992.91	919.41	948.81	992.91
		Movr	38.11	163.77	43.38	38.11	-168.7	-5.42	38.11
		Satis	Y	Y	Y	Y	Y	Y	Y
	Slip	Hres	189	133.52	189	189	189.0	189.0	189
		Hact	-13.48	33.87	-8.21	-13.48	-134.0	-54.07	-13.48
		Satis	Y	Y	Y	Y	Y	Y	Y
	Force Acting	M	-141.0	137.90	-49.42	-141.0	-421.4	-228.6	-141.0
		N	387.52	223.70	295.85	387.52	317.52	345.52	387.52
		Q	-13.48	33.87	-8.21	-13.48	-134.0	-54.07	-13.48
	OverallSatisfactory		Y	Y	Y	Y	Y	Y	
Bearing Capacity of foundation Soil	Eccentricity	E	0	616.4	0	0	0	0	0
		ealw	1386	1386	1386	1386	1386	1386	1386
		Satis	Y	Y	Y	Y	Y	Y	Y
	Foundation Soil	б	92.27	75.39	70.44	92.27	75.60	82.27	92.27
		Rd	250	250	250	250	250	250	250
		Satis	Y	Y	Y	Y	Y	Y	Y
	OverallSatisfactory		Y	Y	Y	Y	Y	Y	
Slope Stability	Fa in KN/m		468.52	666.57	478.73	468.52	397.74	458.78	468.52
	Fp in KN/m		815.05	1099.4	814.07	815.05	1177.9	886.03	815.05
	Ma in KN-m/m		4794.4	7498.7	4850.0	4794.4	4655.7	5475.0	4794.4
	MP in KN-m/m		8340.4	12368	8247.2	8340.4	13788	10574	8340.4
	F.O.S		1.74	1.65	1.70	1.74	2.96	1.93	1.74
	Satisfactory		Y	Y	Y	Y	Y	Y	Y

Table No. 4.17

Case:-2; 6m Cantilever wall, with Clay soil condition, surcharge=30kpa is varying with 10^0 Slope angle and FF resistance=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	6m BS
Verification	Overturning	Mres	1010.67	513.41	731.85	1010.67
		Movr	58.66	168.45	63.93	58.66
		Satis	Y	Y	Y	Y
	Slip	Hres	189	135.58	189.0	189
		Hact	-0.31	40.14	4.96	-0.31
		Satis	Y	Y	Y	Y
	Force Acting	M	-127.13	135.90	-35.54	-127.13
		N	392.8	228.98	301.13	392.8
		Q	-0.31	40.14	4.96	-0.31
	OverallSatisfactory		Y	Y	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	e	0	593.5	0	0
		ealw	1386	1386	1386	1386
		Satis	Y	Y	Y	Y
	Foundation Soil	σ	93.52	76.0	93.52	93.52
		Rd	250	250	250	250
		Satis	Y	Y	Y	Y
	OverallSatisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		533.0	605.39	536.07	533.0
	Fp in KN/m		867.89	928.44	855.13	867.89
	Ma in KN-m/m		5500.0	6352.46	5421.99	5500.0
	MP in KN-m/m		8956.76	9742.30	8649.02	8956.76
	F.O.S		1.63	1.53	1.60	1.63
	Satisfactory		Y	Y	Y	Y

Table no. 4.18

Case:-1; 6m Cantilever wall, with Sand soil condition, surcharge=30kpa is varying with 5^0 Slope angle and FF resistance=1m

S.N			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	6m BS	6m BS & 1m FS	6m BS & 3m FS	6m BS & 6m FS
Verification	Overturning	Mres	1041.12	551.38	765.37	1041.12	976.76	1003.43	1041.12
		Movr	194.99	358.41	230.38	194.99	-0.22	163.12	194.99
		Satis	Y	N	Y	Y	Y	Y	Y
	Slip	Hres	315.48	174.87	236.43	315.48	270.45	292.43	315.48
		Hact	15.61	114.42	51.38	15.61	-69.98	10.02	15.61
		Satis	Y	Y	Y	Y	Y	Y	Y
	Force Acting	M	-80.71	231.30	38.64	-80.71	-320.8	-130.82	-80.71
		N	382.71	212.14	286.81	382.71	328.08	354.75	382.71
		Q	15.61	114.42	51.38	15.61	-69.98	10.02	15.61
	Overall Satisfactory		Y	N	Y	Y	Y	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	e	0	1090.3	134.7	0	0	0	0
		ealw	1320.0	1320.0	1320.0	1320.0	1320.0	1320.0	1320.0
		Satis	Y	Y	Y	Y	Y	Y	Y
	Foundation Soil	Ϭ	95.68	116.60	76.88	95.68	82.02	88.69	95.68
		Rd	450	450	450	450	450	450	450
		Satis	Y	Y	Y	Y	Y	Y	Y
	Overall Satisfactory		Y	Y	Y	Y	Y	Y	Y
Slope Stability	Fa in KN/m		484.59	500.96	469.84	503.11	432.23	457.32	503.11
	Fp in KN/m		1031.12	707.85	820.42	1020.93	863.07	862.96	1020.93
	Ma in KN-m/m		4583.32	4229.34	3978.92	5206.43	4548.24	4440.12	5206.43
	MP in KN-m/m		9752.53	5975.95	6947.87	10565.0	9081.83	8378.50	10565.0
	F.O.S		2.13	1.41	1.75	2.03	2.00	1.89	2.03
	Satisfactory		Y	Y	Y	Y	Y	Y	Y

Table No. 4.19

Case:-2; 6m Cantilever wall, with Sand soil condition, surcharge=30kpa is varying with 10^0 Slope angle and FF resistance=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	6m BS
Verification	Overturning	Mres	1085.68	584.34	806.13	1085.68
		Movr	214.94	374.77	249.58	214.94
		Satis	Y	N	Y	Y
	Slip	Hres	326.11	182.73	246.21	326.11
		Hact	23.98	120.81	59.08	23.98
		Satis	Y	Y	Y	Y
	Force Acting	M	-79.53	233.78	40.81	-79.53
		N	395.60	221.67	298.68	395.60
		Q	23.98	120.81	59.08	23.98
	OverallSatisfactory		Y	N	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	e	0	1054.6	136.6	0
		ealw	1320	1320	1320	1320
		Satis	Y	Y	Y	Y
	Foundation Soil	σ	98.90	117.24	80.15	98.90
		Rd	450	450	450	450
		Satis	Y	Y	Y	Y
	OverallSatisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		543.40	531.65	540.89	551.78
	Fp in KN/m		1114.56	738.28	936.13	1084.94
	Ma in KN-m/m		5434.48	4501.07	5172.11	5760.21
	MP in KN-m/m		11146.7	6250.41	8951.50	11326.0
	F.O.S		2.05	1.39	1.73	1.97
	Satisfactory		Y	N	Y	Y

Table No. 4.20

Case:-1; 6m Cantilever wall, with C-Phi soil condition, surcharge=30kpa is varying with 10^0 Slope angle and FF resistance=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	5m BS
Verification	Overturning	Mres	1094.62	552.12	800.09	1032.70
		Movr	416.69	493.77	431.29	417.16
		Satis	Y	N	N	Y
	Slip	Hres	158.50	68.89	118.50	150.92
		Hact	158.55	207.40	176.06	160.68
		Satis	N	N	N	N
	Force Acting	M	169.11	421.50	278.66	189.90
		N	403.36	228.50	308.32	383.54
		Q	158.55	207.40	176.06	160.68
	OverallSatisfactory		N	N	N	N
Bearing Capacity of foundation Soil	Eccentricity	e	419.3	1844.6	903.8	495.1
		e_{alw}	1386	1386	1386	1386
		Satis	Y	N	Y	Y
	Foundation Soil	σ	119.99	447.42	128.88	119.49
		Rd	300	300	300	300
		Satis	Y	N	N	Y
	OverallSatisfactory		Y	N	N	Y
Slope Stability	F_a in KN/m		528.59	502.49	495.49	520.78
	F_p in KN/m		667.94	496.71	566.72	635.09
	M_a in KN-m/m		5689.41	4206.3	4520.68	5461.92
	MP in KN-m/m		7189.29	4157.8	5170.57	6660.73
	F.O.S		1.26	0.99	1.14	1.22
	Satisfactory		N	N	N	N

4.4 EARTHQUAKERESULTS

Table 4.21

Case:-1. 4m Gravity wall with Clay soil, surcharge=30kpa is variable and 5^0 slope FF res=1m

S.N			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	4m BS	4m BS& 1mFS	4mBS & 3m FS	4m BS & 4m FS
Verification	Overturning	Mres	114.50	93.46	107.49	114.50	107.48	112.15	114.50
		Movr	-27.74	15.48	-27.34	-27.74	-66.00	-27.76	-27.74
		Satis	Y	Y	Y	Y	Y	Y	Y
	Slip	Hres	75.38	64.61	75.38	75.38	75.38	75.38	75.38
		Hact	-81.72	-43.08	-80.87	-81.72	-117.9	-81.85	-81.72
		Satis	Y	Y	Y	Y	Y	Y	Y
	Force Acting	M	-37.23	12.99	-34.50	-37.23	-82.72	-39.81	-37.23
		N	125.38	108.63	119.80	125.38	108.36	119.53	125.38
		Q	-81.72	-43.08	-80.87	-81.72	-117.9	-81.85	-81.72
	Overall Satisfactory		Y	Y	Y	Y	Y	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	E	0	119.6	0	0	0	0	0
		ealw	552.8	552.8	552.8	552.8	552.8	552.8	552.8
		Satis	Y	Y	Y	Y	Y	Y	Y
	Foundation Soil	б	74.85	75.66	71.52	74.85	64.69	71.36	74.85
		Rd	250	250	250	250	250	250	250
		Satis	Y	Y	Y	Y	Y	Y	Y
	Overall Satisfactory		Y	Y	Y	Y	Y	Y	Y
Slope Stability	Fa in KN/m		468.13	488.26	475.14	470.40	409.48	469.0	470.40
	Fp in KN/m		993.99	989.85	998.71	995.14	992.58	993.45	995.14
	Ma in KN-m/m		5171.63	5266.22	5075.98	5197.51	4485.2	5200.0	5197.51
	MP in KN-m/m		10981.1	10676.3	10669.3	10995.3	10872	11015	10995.3
	F.O.S		2.12	2.03	2.10	2.12	2.42	2.12	2.12
	Satisfactory		Y	Y	Y	Y	Y	Y	Y

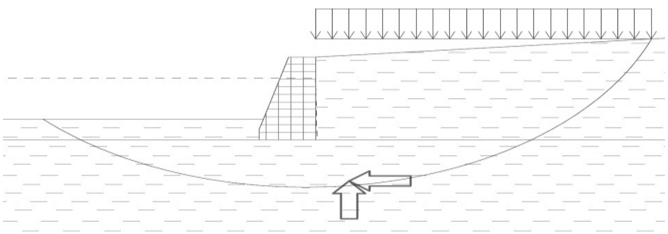

Fig 4.12 Slope stability analysis during EQ.

Table No.4.22

Case:-2. 4m Gravity wall with Clay soil, surcharge=30kpa is variable and 10^0 slope FF res=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	4m BS
Verification	Overturning	Mres	113.46	92.42	106.44	113.46
		Movr	-24.07	19.14	-23.67	-24.07
		Satis	Y	Y	Y	Y
	Slip	Hres	75.38	61.30	75.38	75.38
		Hact	-79.63	-40.99	-78.78	-79.63
		Satis	Y	Y	Y	Y
	Force Acting	M	-33.40	16.83	-30.66	-33.40
		N	124.33	107.58	118.75	124.33
		Q	-79.63	-40.99	-78.78	-79.63
	OverallSatisfactory		Y	Y	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	e	0	156.4	0	0
		ealw	552.8	552.8	552.8	552.8
		Satis	Y	Y	Y	Y
	Foundation Soil	б	74.23	78.98	70.90	74.23
		Rd	250	250	250	250
		Satis	Y	Y	Y	Y
	OverallSatisfactory		Y	Y	Y	Y
Slope Stability	Fa in KN/m		437.98	491.84	575.60	532.58
	Fp in KN/m		845.93	910.43	1086.85	1016.14
	Ma in KN-m/m		4570.61	5101.15	6464.53	5700.21
	MP in KN-m/m		8827.9	9442.47	12206	10875
	F.O.S		1.93	1.85	1.89	1.91
	Satisfactory		Y	Y	Y	Y

Table No.4. 23

Case:-1. 4m Gravity wall with Sand soil, surcharge=30kpa is variable and 5^0 slope FF res=1m

S.N			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	4m BS	4m BS & 1m FS	4m BS & 3m FS	4m BS & 4m FS
Verification	Overturning	Mres	146.90	121.3	138.81	146.90	139.90	144.58	146.90
		Movr	106.47	140.1	104.76	106.47	77.50	115.74	106.47
		Satis	N	N	N	N	N	N	N
	Slip	Hres	93.17	77.12	88.04	93.17	87.09	96.30	93.17
		Hact	-3.62	32.48	-0.72	-3.62	-8.38	27.70	-3.62
		Satis	Y	Y	Y	Y	Y	Y	Y
	Force Acting	M	54.23	97.15	55.40	54.23	26.09	69.00	54.23
		N	113.03	93.56	106.80	113.03	105.65	116.82	113.03
		Q	-3.62	32.48	-0.72	-3.62	-8.38	27.70	-3.62
	OverallSatisfactory		N	N	N	N	N	N	N
Bearing Capacity of foundation Soil	Eccentricity	e	479.8	1038.5	518.7	479.8	246.9	590.7	479.8
		ealw	552.8	552.8	552.8	552.8	552.8	552.8	552.8
		Satis	Y	N	Y	Y	Y	N	Y
	Foundation Soil	б	158.0	10000	167.5	158.0	89.45	236.64	158.0
		Rd	450	450	450	450	450	450	450
		Satis	Y	N	Y	Y	Y	N	Y
	OverallSatisfactory		Y	N	Y	Y	Y	N	Y
Slope Stability	Fa in KN/m		241.35	248.34	238.06	251.08	239.26	254.36	251.08
	Fp in KN/m		451.85	380.76	426.04	458.97	398.79	403.23	458.97
	Ma in KN-m/m		1612.47	1593.38	1592.3	1870.6	2025.3	2049.9	1870.6
	MP in KN-m/m		3018.77	2442.99	2849.5	3419.6	3375.8	3249.7	3419.6
	F.O.S		1.87	1.53	1.79	1.83	1.67	1.59	1.83
	Satisfactory		Y	Y	Y	Y	Y	Y	Y

Table No. 4.24

Case:-2. 4m Gravity wall with Sand soil, surcharge=30kpa is variable and 10^0 slope FF res=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	8m BS
Verification	Overturning	Mres	148.84	123.04	140.51	148.84
		Movr	122.68	155.27	119.81	122.68
		Satis	N	N	N	N
	Slip	Hres	93.81	77.66	88.56	93.81
		Hact	4.54	40.25	6.98	4.54
		Satis	Y	Y	Y	Y
	Force Acting	M	69.15	113.13	69.27	69.15
		N	113.80	94.21	107.43	113.80
		Q	4.54	40.25	6.93	4.54
	OverallSatisfactory		N	N	N	N
Bearing Capacity of foundation Soil	Eccentricity	e	607.6	1179.6	644.8	607.6
		ealw	552.8	552.8	552.8	552.8
		Satis	N	N	N	N
	Foundation Soil	б	247.54	1000	278.73	247.54
		Rd	450	450	450	450
		Satis	N	N	N	N
	OverallSatisfactory		N	N	N	N
Slope Stability	Fa in KN/m		251.38	274.84	282.90	283.71
	Fp in KN/m		416.87	395.86	474.91	486.07
	Ma In KN-m/m		2153.89	1890.44	2172.44	2200.8
	MP in KN-m/m		3571.91	2722.8	3646.83	3770.6
	F.O.S		1.66	1.44	1.68	1.71
	Satisfactory		Y	N	Y	Y

Table 4.25

Case:-1. 4m Gravity wall with C-Phi soil, surcharge=30kpa is variable and 5^0 slope FF res=1m

S.N			1	2	3	4	5
Water Condition			No water	1m BS	4m BS	4m BS & 1m FS	4m BS & 4m FS
Verification	Overturning	Mres	144.48	120.67	144.48	137.32	144.48
		Movr	114.78	148.79	114.78	77.13	114.78
		Satis	N	N	N	N	N
	Slip	Hres	41.17	29.25	41.17	46.10	41.17
		Hact	33.55	64.33	33.55	1.31	33.55
		Satis	N	N	N	Y	N
	Force Acting	M	77.15	119.56	77.15	33.42	77.15
		N	127.58	109.18	127.58	111.77	127.58
		Q	33.58	64.33	33.58	1.31	33.58
	Overall Satisfactory		Y	N	Y	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	e	604.7	1095.1	604.7	299	604.7
		ealw	552.8	552.8	552.8	552.8	552.8
		Satis	N	N	N	Y	N
	Foundation Soil	б	274.0	10000	274.0	103.77	274.0
		Rd	300	300	300	300	300
		Satis	N	N	N	Y	N
	Overall Satisfactory		N	N	N	Y	N
Slope Stability	Fa in KN/m		240.60	286.61	239.01	244.18	239.01
	Fp in KN/m		345.73	375.78	342.79	359.64	342.79
	Ma in KN-m/m		1749.52	2551.86	1719.16	2036.3	1719.16
	MP in KN-m/m		2513.99	3345.79	2465.66	2999.2	2465.66
	F.O.S		1.44	1.31	1.43	1.47	1.43
	Satisfactory		N	N	N	N	N

Table 4.26

Case:-2. 4m Gravity wall with C-Phi soil, surcharge=30kpa is variable and 10^0 slope FF res=1m

S.N			1	2	3	4
Water Condition			No water	4m BS	4m BS & 1m FS	4m BS & 4mFS
Verification	Overturning	Mres	125.25	156.90	149.86	157.13
		Movr	189.38	211.71	189.98	227.05
		Satis	N	N	N	N
	Slip	Hres	29.99	36.17	31.84	36.10
		Hact	80.90	75.82	50.95	82.44
		Satis	N	N	N	N
	Force Acting	M	157.85	167.87	139.64	182.74
		N	111.91	134.99	118.84	134.71
		Q	80.90	75.82	50.95	82.44
	Overall Satisfactory		N	Y	N	N
Bearing Capacity of foundation Soil	Eccentricity	e	1410.5	1243.5	1175.1	1356.6
		ealw	552.8	552.8	552.8	552.8
		Satis	N	N	N	N
	Foundation Soil	б	10000	10000	10000	10000
		Rd	300	300	300	300
		Satis	N	N	N	N
	Overall Satisfactory		N	N	N	N
Slope Stability	Fa in KN/m		467.10	300.34	382.19	325.15
	Fp in KN/m		580.56	411.02	555.76	431.65
	Ma in KN-m/m		5574.95	2721.67	5071.25	3179.80
	MP in KN-m/m		6929.05	3724.56	7374.25	4221.29
	F.O.S		1.24	1.37	1.45	1.33
	Satisfactory		N	N	N	N

4.5 NON- HOMOGENEOUS RESULT

Table 4.27

Case:-1. 4m Gravity wall with (C-phi, Clay, Sand), surcharge=30kpa is variable and 5^0 slope FF res=1m

S.N			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	4m BS	4m BS & 1m FS	4m BS & 3m FS	4m BS & 4m FS
Verification	Overturning	Mres	95.18	77.32	89.68	95.18	90.47	93.62	95.18
		Movr	58.06	86.64	55.24	58.06	29.09	67.33	58.06
		Satis	Y	N	N	Y	Y	N	Y
	Slip	Hres	68.10	54.56	63.86	68.10	64.50	72.05	68.10
		Hact	-23.51	11.43	-20.80	-23.51	-28.26	7.82	-23.51
		Satis	Y	Y	Y	Y	Y	Y	Y
	Force Acting	M	19.68	54.82	18.82	19.68	-7.59	33.80	19.68
		N	82.61	66.19	77.46	82.61	78.24	87.41	82.61
		Q	-23.51	11.43	-20.80	-23.51	-28.26	7.82	-23.51
	OverallSatisfactory		N	N	N	N	N	N	N
Bearing Capacity of foundation Soil	Eccentricity	e	238.2	828.3	242.9	238.2	0	386.7	238.2
		ealw	453.8	453.8	453.8	453.8	453.8	453.8	453.8
		Satis	Y	N	Y	Y	Y	Y	Y
	Foundation Soil	σ	91.93	10000	87.12	91.93	56.90	145.31	91.93
		Rd	450	450	450	450	450	450	450
		Satis	Y	N	Y	Y	Y	Y	Y
	OverallSatisfactory		Y	N	Y	Y	Y	Y	Y
Slope Stability	Fa in KN/m		238.73	239.63	241.97	257.99	214.52	248.0	257.99
	Fp in KN/m		460.71	383.50	449.61	492.41	408.79	411.01	492.41
	Ma in KN-m/m		1717.46	1563.05	1811.3	2258.34	1829.6	2057.9	2258.34
	MP in KN-m/m		3314.39	2501.42	3365.6	4310.26	3486.7	3410.7	4310.26
	F.O.S		1.93	1.60	1.86	1.91	1.91	1.63	1.91
	Satisfactory		Y	Y	Y	Y	Y	Y	Y

Table 4.28

Case:-2. 4m Gravity wall with (C-phi, Clay, Sand), surcharge=30kpa is variable and 10^0 slope FF res=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	4m BS
Verification	Overturning	Mres	99.96	80.48	93.86	99.96
		Movr	123.60	129.58	112.97	123.60
		Satis	N	N	N	N
	Slip	Hres	70.74	56.23	66.14	70.74
		Hact	-1.84	26.50	-1.48	-1.84
		Satis	Y	N	Y	Y
	Force Acting	M	82.64	95.99	74.27	82.64
		N	85.81	68.21	80.23	85.81
		Q	-1.87	26.50	-1.48	-1.87
	OverallSatisfactory		Y	N	N	Y
Bearing Capacity of foundation Soil	Eccentricity	e	963.0	1407.3	925.7	963.0
		ealw	453.8	453.8	453.8	453.8
		Satis	N	N	N	N
	Foundation Soil	б	10000	10000	10000	10000
		Rd	450	450	450	450
		Satis	N	N	N	N
	OverallSatisfactory		N	N	N	N
Slope Stability	Fa in KN/m		315.04	306.25	327.73	315.90
	Fp in KN/m		560.78	459.30	565.11	555.16
	Ma in KN-m/m		3176.91	2718.97	3466.66	3281.14
	MP in KN-m/m		5654.99	4077.83	5977.60	5766.25
	F.O.S		1.78	1.50	1.72	1.76
	Satisfactory		Y	Y	Y	Y

4.6 EARTHQUAKE RESULT OF CANTILEVER WALL

Table 4.29

Case:-1. 6m Cantilever wall with Clay soil, surcharge=30kpa is variable and 5^0 slope FF res=1m

S.N			1	2	3	4	5	6	7
Water Condition			No water	1m BS	3m BS	6m BS	6m BS & 1m FS	6m BS & 3m FS	6m BS & 6m FS
Verification	Overturning	Mres	944.54	447.27	665.72	944.54	871.65	901.05	944.54
		Movr	143.34	268.77	148.44	143.34	-35.88	105.78	143.34
		Satis	Y	N	Y	Y	Y	Y	Y
	Slip	Hres	189.0	79.08	169.10	189.0	189.0	189.0	189.0
		Hact	23.39	70.68	28.61	23.39	-83.38	-12.23	23.39
		Satis	Y	N	Y	Y	Y	Y	Y
	Force Acting	M	-30.55	248.12	60.86	-30.55	-281.7	-110.7	-30.55
		N	366.97	203.16	275.31	366.97	297.99	325.99	366.97
		Q	23.39	70.68	28.61	23.39	-83.38	-12.23	23.39
	Overall Satisfactory		Y	N	Y	Y	Y	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	e	0	1221.3	221.1	0	0	0	0
		ealw	1386	1386.0	1386	1386	1386	1386	1386
		Satis	Y	Y	Y	Y	Y	Y	Y
	Foundation Soil	Ϭ	87.37	115.61	73.26	87.37	70.95	77.62	87.37
		Rd	250	250	250	250	250	250	250
		Satis	Y	N	Y	Y	Y	Y	Y
	Overall Satisfactory		Y	N	Y	Y	Y	Y	Y
Slope Stability	Fa in KN/m		1152.67	738.01	1154.86	1163.52	919.58	981.25	1163.52
	Fp in KN/m		1863.67	1102.03	1813.33	1864.86	1764.31	1875.97	1864.86
	Ma in KN-m/m		21672.7	8419.58	20390.9	21859.1	14991.4	18300.7	21859.1
	MP in KN-m/m		35041.1	12572.4	32017.3	35035.1	28762.5	34987.4	35035.1
	F.O.S		1.62	1.49	1.57	1.60	1.92	1.91	1.60
	Satisfactory		Y	N	Y	Y	Y	Y	Y

Table No. 4.30

Case:-2. 6m Cantilever wall with Clay soil surcharge=30kpa is variable and 10^0 slope FF res=1m

S.N			1	2	3	4
Water Condition			No water	1m BS	3m BS	6m BS
Verification	Overturning	Mres	961.46	464.19	682.63	961.46
		Movr	189.36	298.89	194.45	189.36
		Satis	Y	N	Y	Y
	Slip	Hres	186.80	71.46	156.73	186.80
		Hact	44.76	85.15	49.98	44.76
		Satis	Y	N	Y	Y
	Force Acting	M	9.11	271.88	100.51	9.11
		N	372	208.18	280.33	372
		Q	44.76	85.15	49.98	44.76
	OverallSatisfactory		Y	N	Y	Y
Bearing Capacity of foundation Soil	Eccentricity	e	24.5	1306	358.5	24.5
		ealw	1386	1386	1386	1386
		Satis	Y	Y	Y	Y
	Foundation Soil	σ	89.62	131.09	80.49	89.62
		Rd	250	250	250	250
		Satis	Y	N	Y	Y
	OverallSatisfactory		Y	N	Y	Y
Slope Stability	Fa in KN/m		1355.70	1425.36	1361.27	1367.24
	Fp in KN/m		1902.84	1888.37	1858.30	1902.84
	Ma in KN-m/m		25374.9	26222.62	25534.58	25590.94
	MP in KN-m/m		35615.9	34740.58	34857.97	35615.99
	F.O.S		1.40	1.32	1.37	1.39
	Satisfactory		N	N	N	N

4.7 FEMRESULTS

During analysis of earthquake first we find out the value of K_h and K_v, which is already given in table number 1.

Figure related to FEM analysis

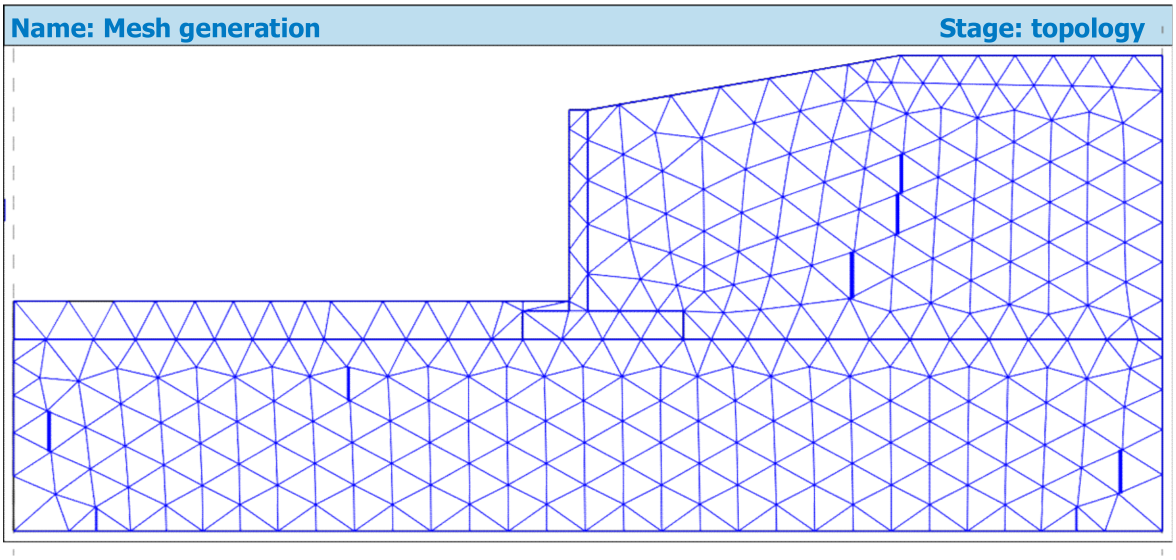

Fig.4.13 Mesh Generation

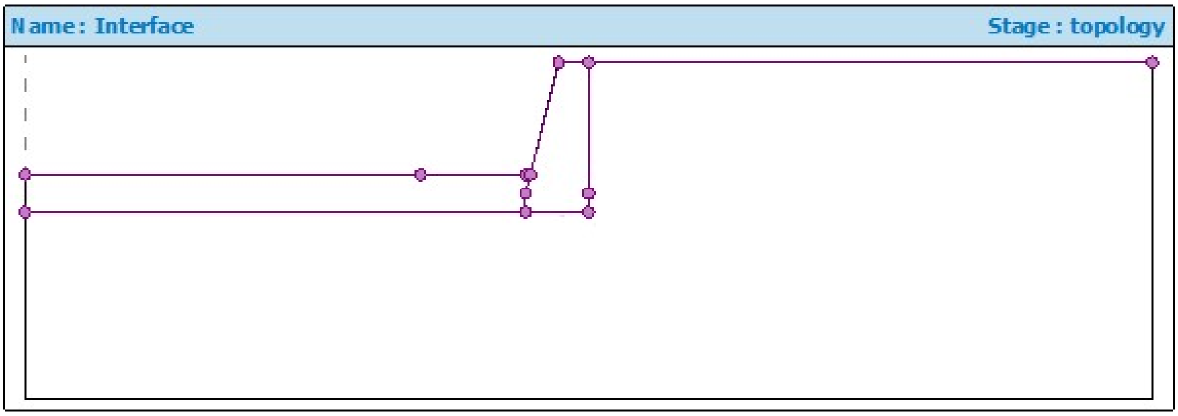

Fig. 4.14 Interface

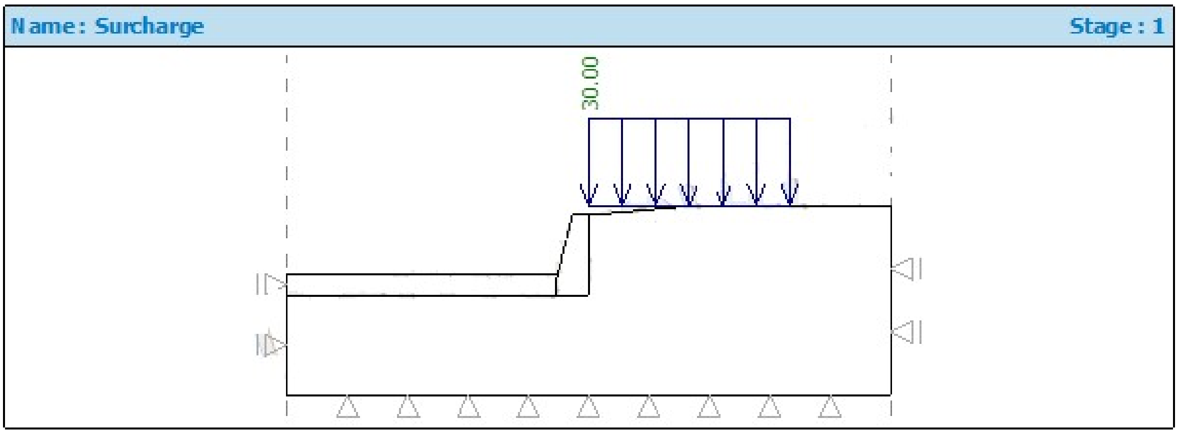

Fig. 4.15 Surcharges on Wall

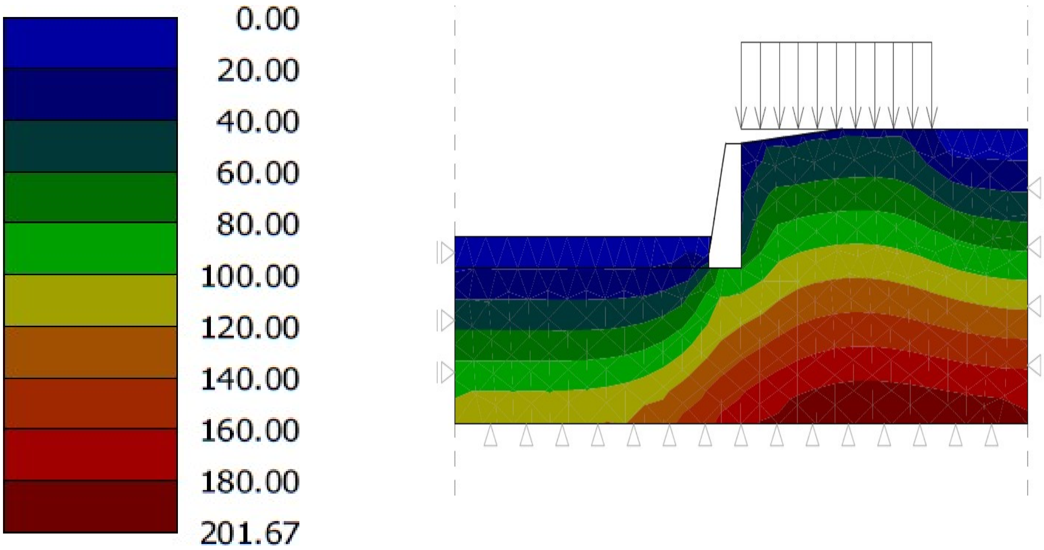

Fig. 4,16 Loading Condition in KPa

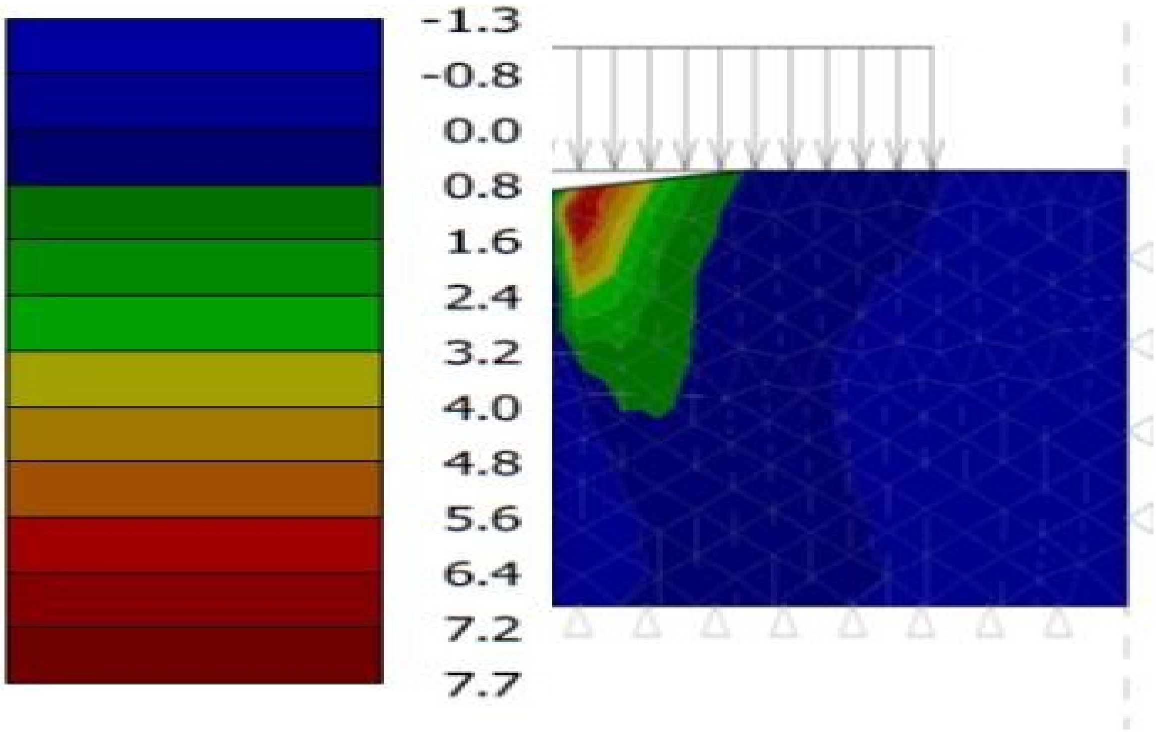

Figure 4.17 Settlement Condition in mm

GRAVITY WALL

Table No. 4.31

Case1:- 4m Gravity wall for all soil condition

S.N		HORIZONTAL TERRAIN	5^0SLOPE ANGLE	10^0 SLOPE ANGLE
1	No. of nodes	1490	1492	1539
2	No. of element	893	893	918
3	Region	489	489	510
4	Beam	101	101	102
5	Interface	303	303	306

Number of nodes, elements, regions, beam and interfaces are equal in 4m retaining wall due to reason of same geometry condition. If geometry condition is same then there is no effect on nodes,elements.

4m Gravity wall

Horizontal terrain condition

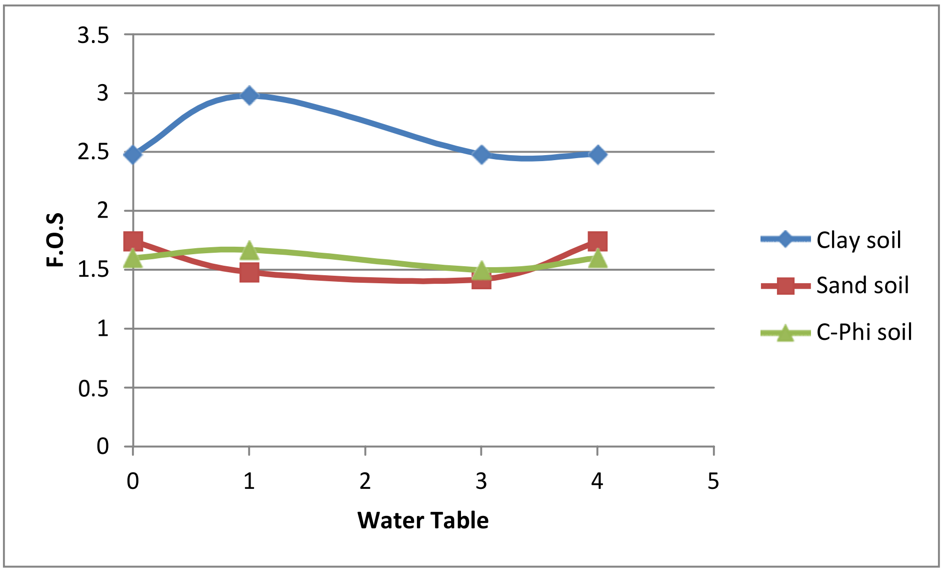

Figure 4.18 4m horizontal terrain

5^0 Slope anglecondition

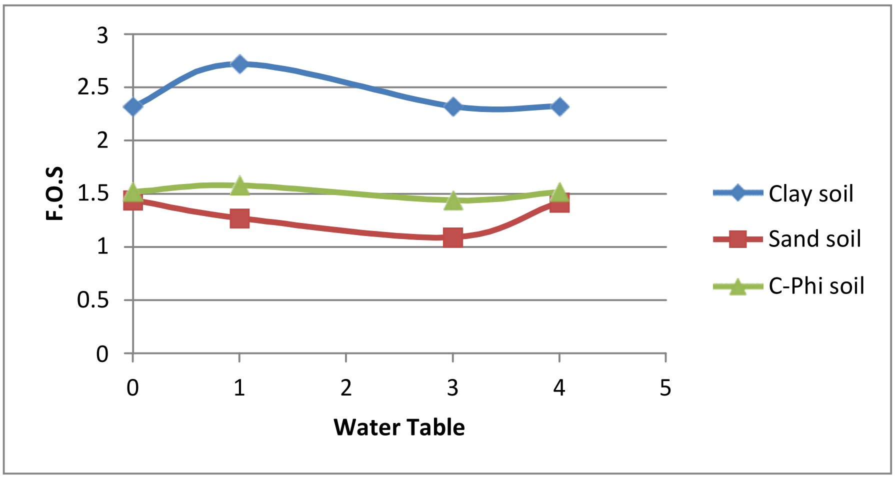

Figure 4.19 4m 5^0 Slope angle

10^0 Slope anglecondition

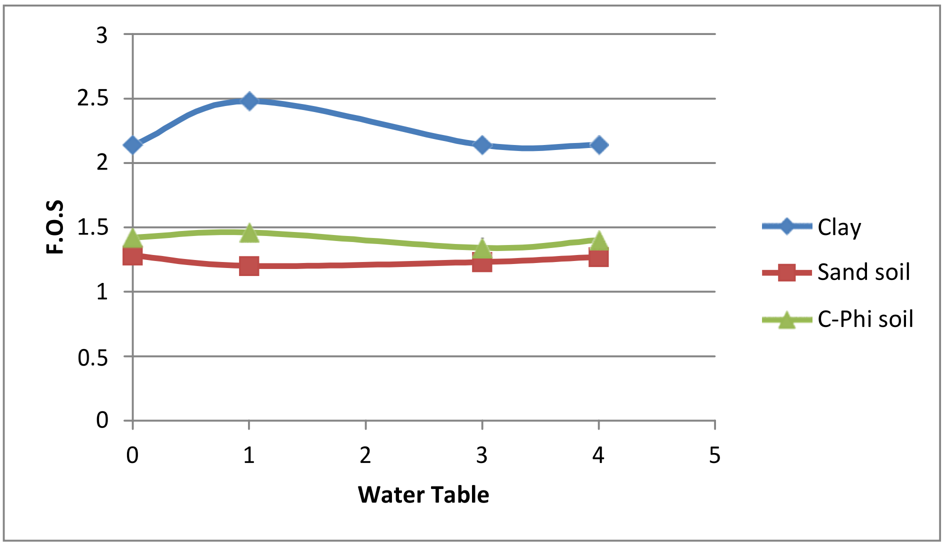

Figure 4.20 4m 10^0 slope angle

Table No. 4.32

5m Gravity wall Gravity wall for all soil condition

S.N		Horizontal terrain	5 degree slope	10 degree slope
1.	No. of nodes	1614	1623	1659
2.	No. of element	959	964	982
3.	Region	547	548	566
4.	Beam	103	104	104
5.	Interface	309	312	312

Horizontal terrain condition

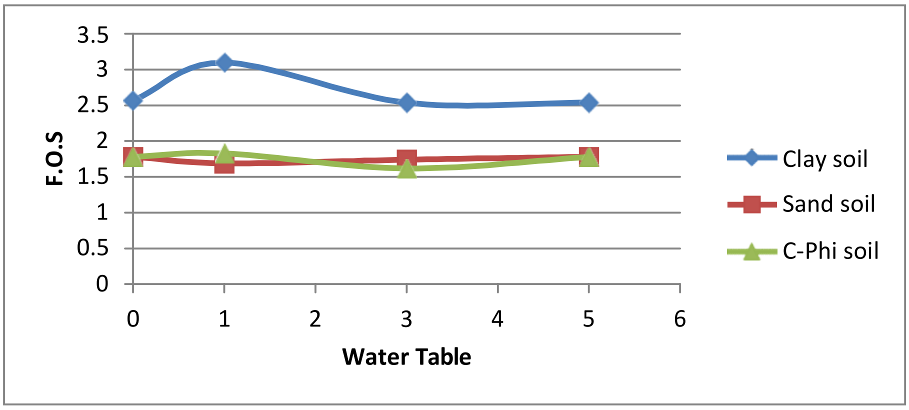

Figure 4.21 5m Horizontal terrain

5^0 Slope anglecondition

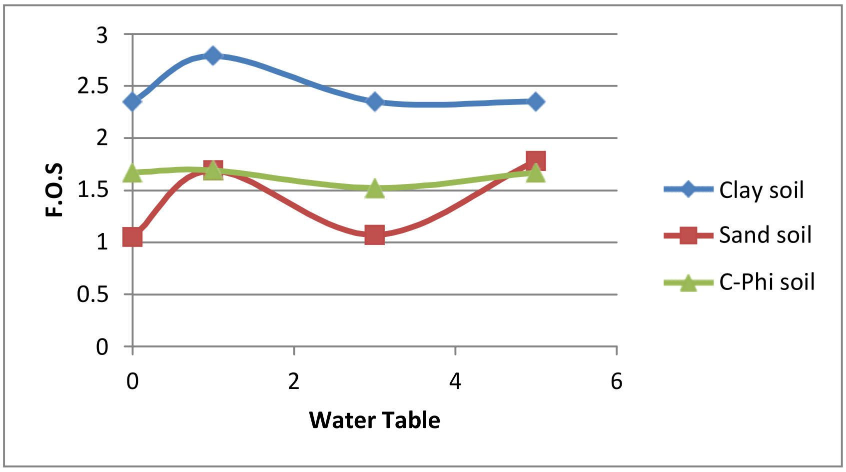

Figure 4.22 5m 5^0 slope angle

10^0 Slope anglecondition

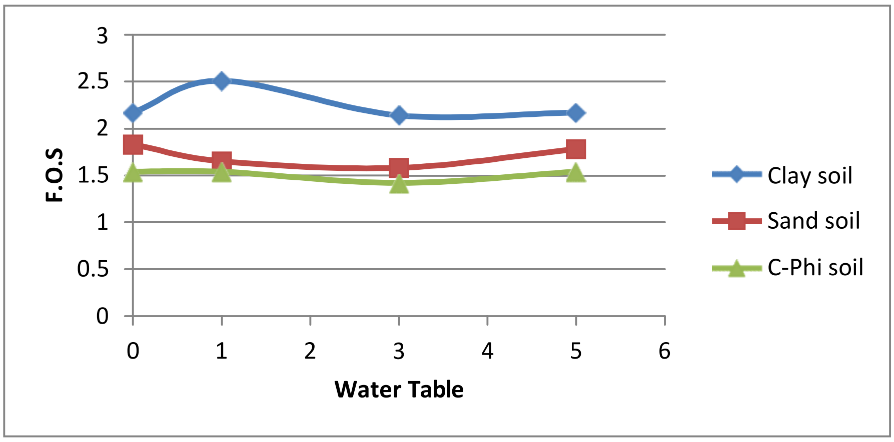

Figure 4.23 5m 10^0 slope angle

Table No.4.33

6m Gravity wall for all soil condition

S.N		Horizontal terrain	5 degree slope	10 degree slope
1.	No. of nodes	1696	1733	1749
2.	No. of element	1005	1024	1032
3.	Region	581	596	604
4.	Beam	106	107	107
5.	Interface	318	321	321

Horizontal terrain condition

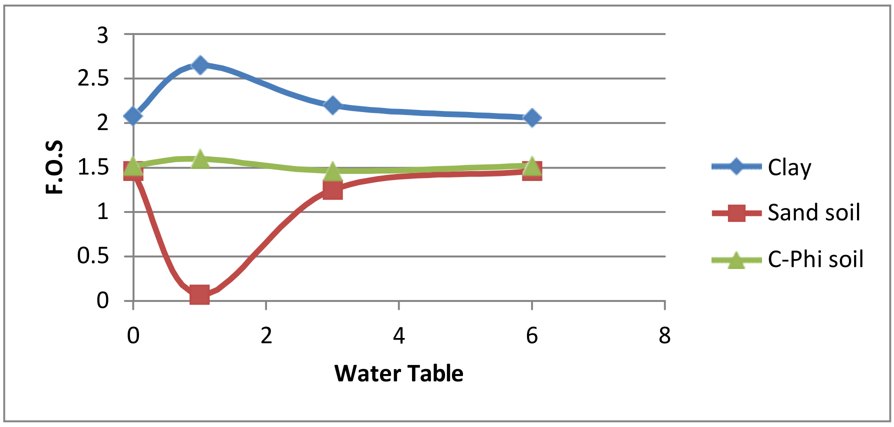

Figure 4.24 6m Gravity wall for Horizontal terrain

5^0 Slope anglecondition

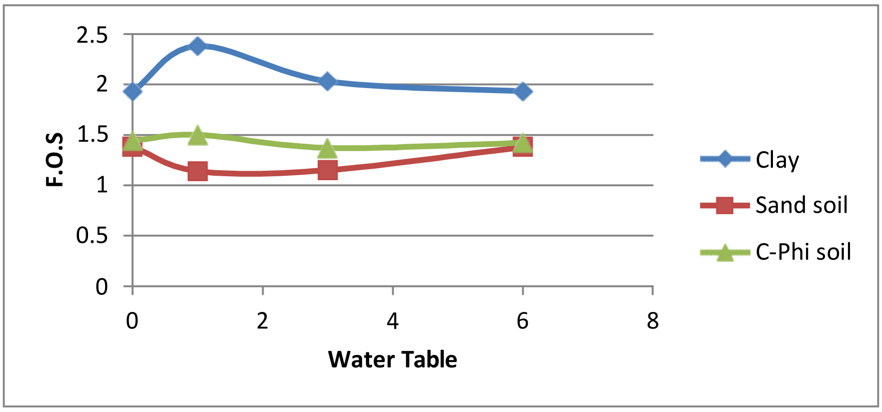

Figure 4.25 6m 5^0 slope angle

10^0 Slope anglecondition

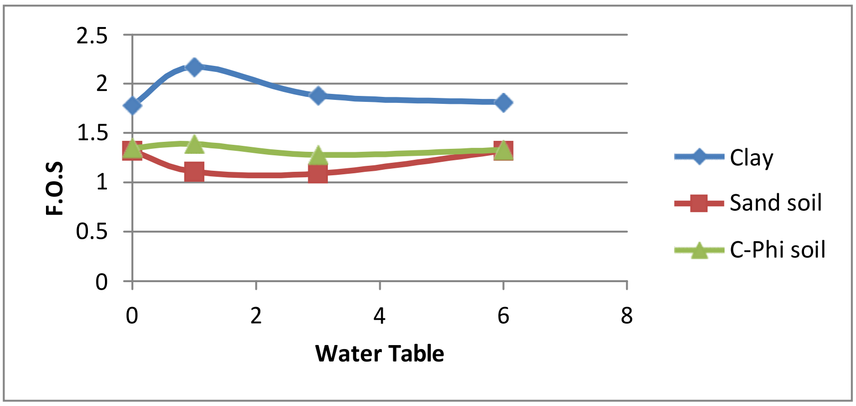

Figure 4.26 6m 10^0 slope angle

CANTILEVER WALL

Table No.4.34

Case1:- 6m Cantilever wall for all soil condition

S.N		Horizontal terrain	5 degree slope	10 degree slope
1.	No. of nodes	1667	1722	1754
2.	No. of element	1022	1051	1067
3.	Region	526	551	567
4.	Beam	124	125	125
5.	Interface	372	375	375

Horizontal terrain condition

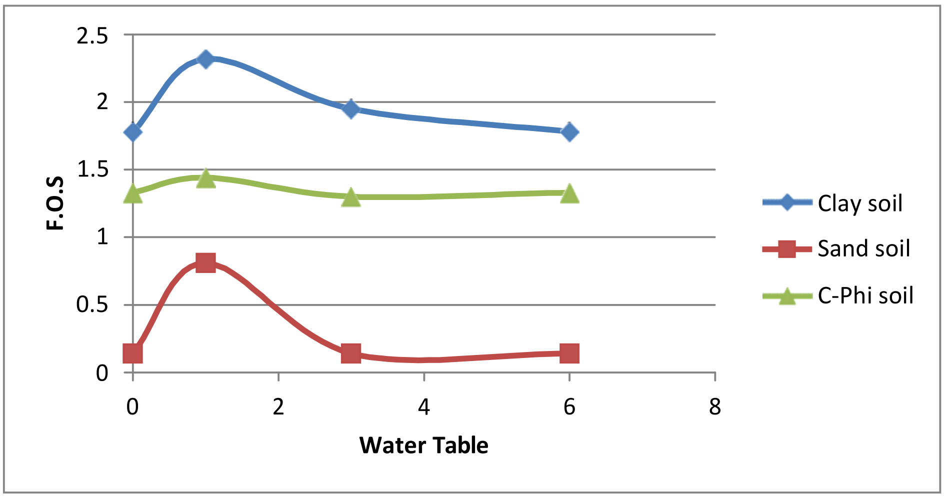

Figure 4.27 6m cantilever wall Horizontal terrain

5^0 Slope anglecondition

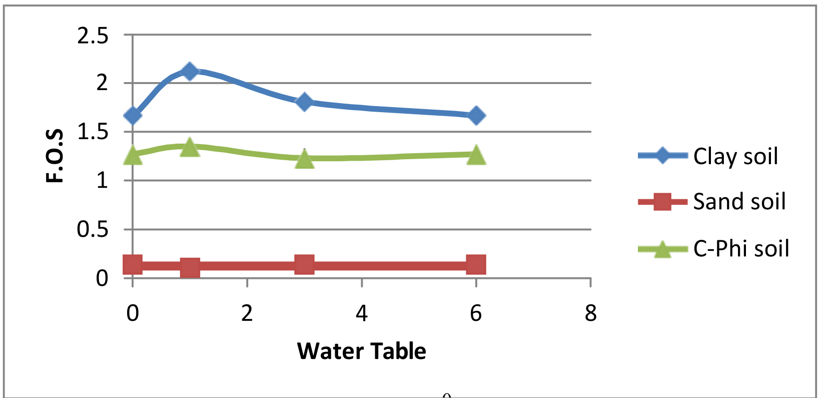

Figure 4.28 6m 5^0 slope angle

10^0 Slope anglecondition

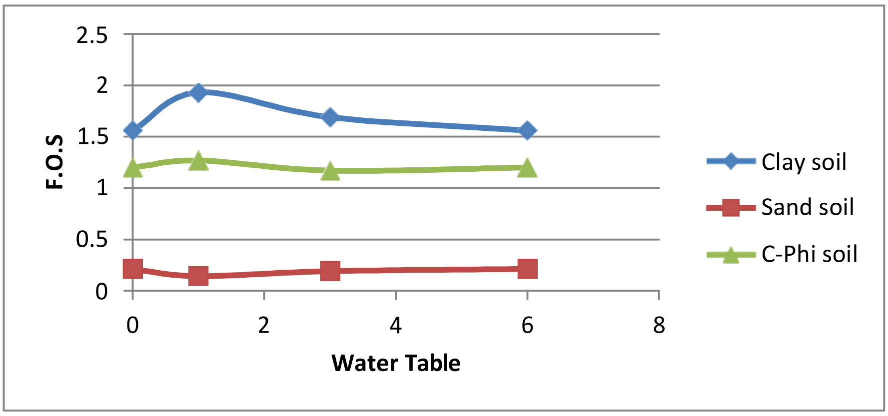

Figure 4.29 6m 10^0 slope angle

7m Cantilever wall

Table No.4.35

Case1:- 7m Cantilever wall for Clay and C-Phi soil condition

S.N		Horizontal terrain	5 degree slope	10 degree slope
1.	No. of nodes	1760	1815	1858
2.	No. of element	1077	1106	1129
3.	Region	561	586	605
4.	Beam	129	130	131
5.	Interface	387	390	393

Horizontal terrain condition

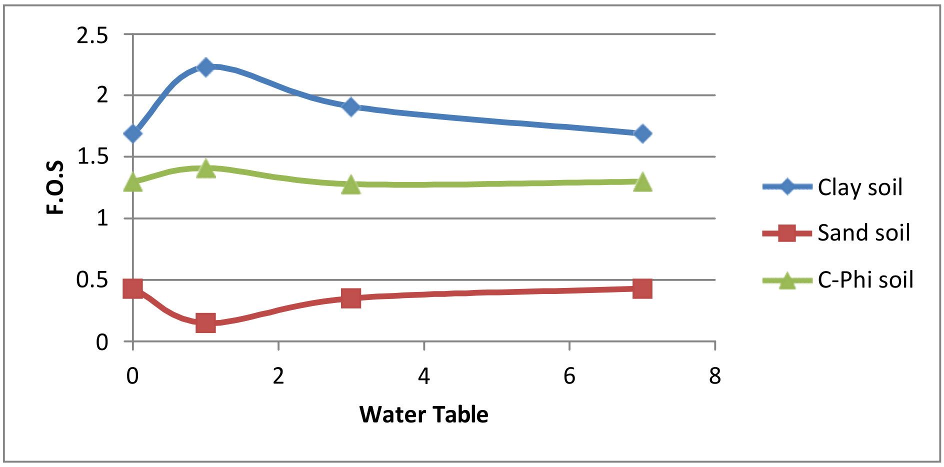

Figure 4.30 7m Horizontal terrain

5^0 Slope anglecondition

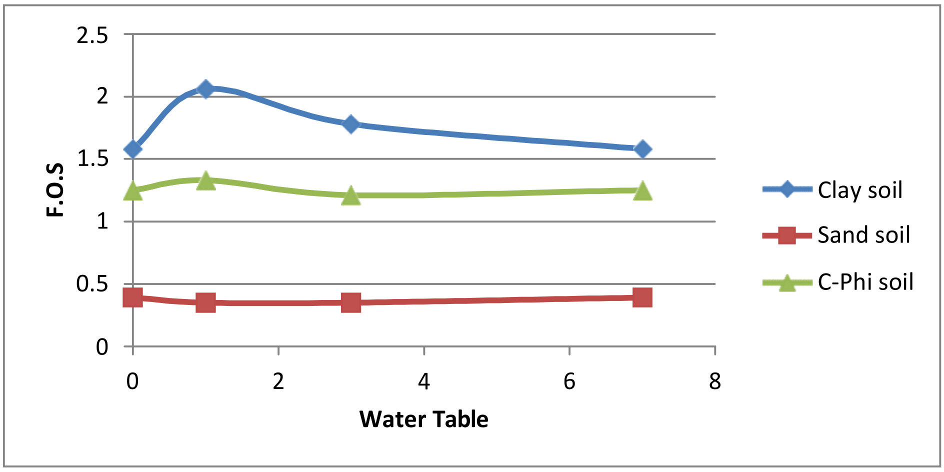

Figure 4.31 7m 5^0 slope angle

10^0 Slope anglecondition

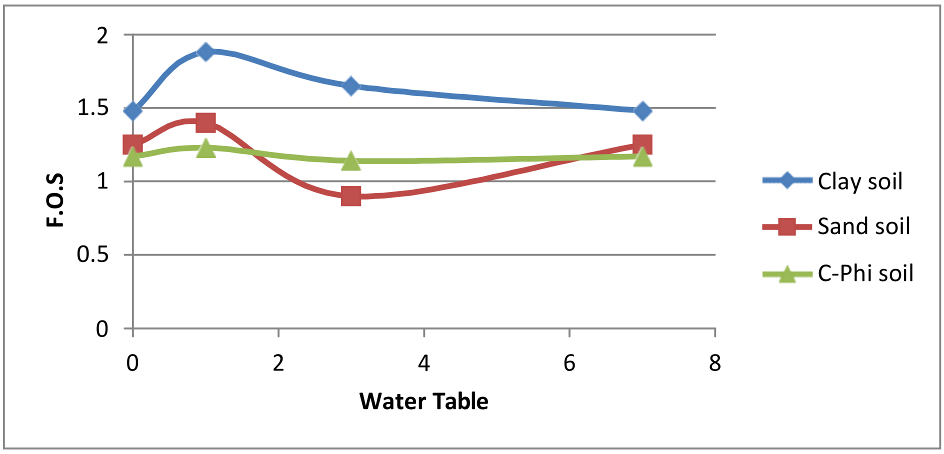

Figure 4.32 7m 10^0 slope angle

8m Cantilever wall

Table No. 4.36

Case1:- 8m Cantilever wall for all soil condition

S.N		Horizontal terrain	5 degree slope	10 degree slope
1.	No. of nodes	1871	1930	1973
2.	No. of element	1136	1167	1190
3.	Region	612	639	658
4.	Beam	131	132	133
5.	Interface	393	396	399

Horizontal terrain condition

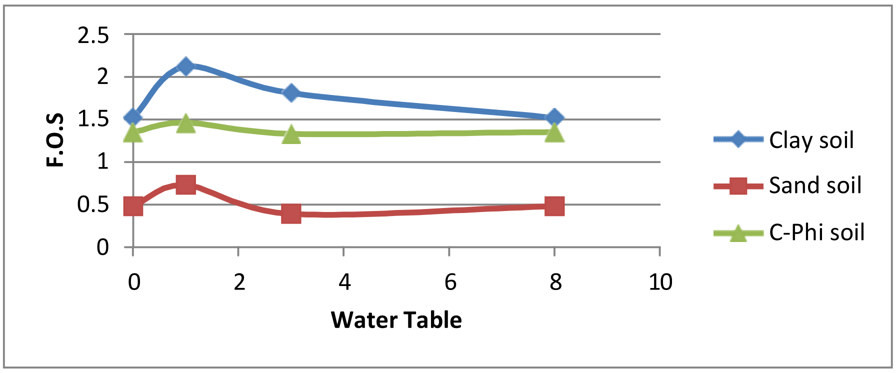

Figure 4.33 8m Horizontal terrain

5^0 Slope anglecondition

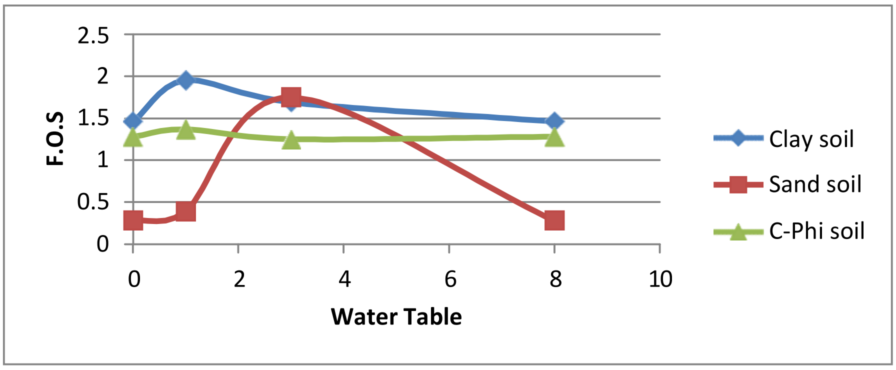

Figure 4.34 8m 5^0 slope angle

10^0 Slope anglecondition

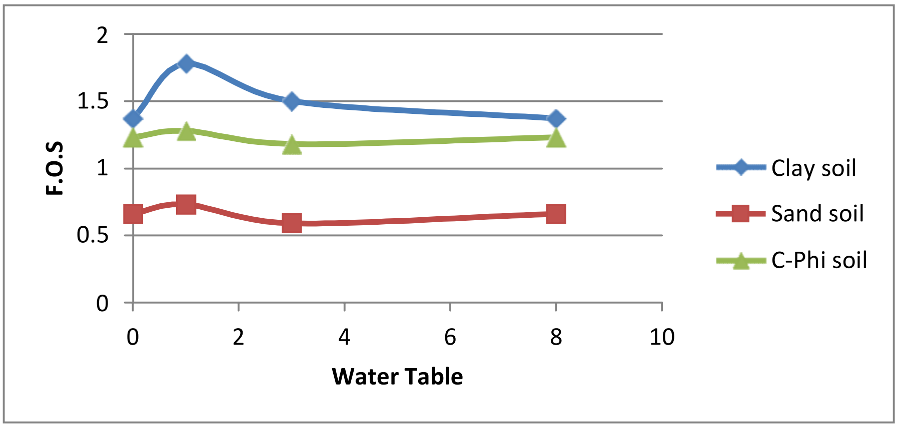

Figure 4.35 8m 10^0 slope angle

4.8 Discussions:-

On the basis of results different points to be noted:-

1. There is no change in moment, force, eccentricity and wall stem condition when water is not available, and water is at the base level of backward side and forward side. But the factor of safety will be changed.
2. Factor of safety is equal, when water table is at base of retaining wall on backward side and forward side.
3. When there is no water, load & moment are maximum and water is at 1 m depth below the top layer parameters of load and moment are minimized. When water goes downward direction parameters is increasing and reach its maximum value.
4. When backfilling soil is clay in a gravity wall, then there is no change in load parameter with the change in slope, but the factor of safety of retaining wall will be changed.
5. Maximum allowable eccentricity is not depending on the change of water table variation and slope angle.
6. Increasing the factor of K_h results in corresponding decrease of the factor of safety.
7. If earthquake load is applied on retaining wall then there is decrease in load parameters.
8. The finite element approach uses reduces soil parameter C and φ, while the classical approach discussed above uses effective soil parameter. The reduction in the value of C and φ is based on ground conditions and thus the results from finite element method are more realistic.
9. On the basis of classical and numerical approach method to analyses of retaining wall, the change in factor of safety of clay and C – φ soil are not significance but in the case of sand is significant.
10. On the basis of study for any retaining wall clay soil is better option, due to its cohesive properties.
11. On the basis of study, during earthquake analysis, clay and sand soil are acceptable in slope stability but C – φ soil is not accepted.

CHPTER -5

CONCLUSION

5.1 General

On the basis of present study following conclusions may be drawn:-

1. The factor of safety is decreasing with increase in slope angle.
2. Force and moment are increasing with increasing in slope angle.
3. The comparative study of finite element method and classical method as indicated that finite element approach is more near to the realistic condition and it evaluate the other parameter like stress intensity, shear strain deformation and FEM is also better due to in classical approach more assumption will be required, but in finite element method is that no assumption needs to be made in advance about the shape or location of the failure surface, slice side forces and their directions.

5.2 Scope of future:-

There are so many works on retaining wall will be done.

1. For analysis of retaining wall for varying loading condition and change of load condition.
2. For analysis of retaining wall for geometry condition.
3. Analysis of retaining wall can be done from other classical methods.
4. To study and analysis of other type of retaining wall.

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