SOFTWARE DESIGN OF RETAINING WALLS

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ABSTRACT

In the past four decades, civil engineering in general and structural engineering in particular have achieved remarkable successes in adapting successive generations of computational support technologies and devloping Computational tools for specific process steps, notably analysis.

One of the problem areas is the problems associated with lateral earth pressure and retaining wall stability. This segment of the soil mechanics has been receiving wide spread attention from engineers for a long time, The efforts to devise proc and formulate methodologies for analysis and design for the effect of the earth pressure dates back to over three centuries. Many of these theories developed by some of these early investigators serve as the basis for the present day study of the retaining walls. In the view of the great advantages occurring from the use of computers, like ease and promptness in performing various complex calculations, they are being introduced in every field of life. For Engineers, the computers have come as a big help.

Great leaps have been made in the field of engineering using computers. The civil engineers are employing these machines in almost all of their Work. The computers are accurate, precise and much faster in calculation and adapted to the changes which may be linked for incorporating from time to time. Thus the tedium of many calculations for any design process,formulation of planes etc can be removed.

This helps in saving time and efforts required in calculation by hand. It makes the optimum design or plan to be found quickly. Manually, it tail's quite a lot of tune but with the help of computer program developed one can achieve entire same objective in a matter of seconds. As an example, the computer program has been developed in this project for the Design of Gravity and Cantilever Retaining walls.This topic contains a vast area yet to be covered But the effort of putting civil engineering knowledge and computer skills together has taken a start with a pray that we will do better next.

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Chapter 1

INTRODUCTION

1.1 BACKGROUND

In the era of rapidly increasing land cost, civil engineering in general and the construction industry in particular are facing the immense challenge of economical and cost effective land reclamation solutions. The problem is more serious in mountainous regions where roads etc have to be constructed over retained sub-structures. Over the past few decades Geotechnical and structural engineering have evolved different solutions to address this problem by designing a number of earth retaining structures, one of the solutions is the Retaining walls. Retaining walls are very common in highway construction, bridge abutment, landscaping, and basement walls and in many other ways as the situations warrants.

As the man excelled in the field of science and technology and new construction materials were introduced the design of such structures also became complex and time consuming .But the revolutionary progress in the field of computers and information technology have eased our job. Now a days lengthy and laborious calculations, design and analysis procedures can be under taken in few minutes using design softwares. The topic under study is also an effort to demonstrate the use of computers software in designing of reinforced cement concrete Cantilever and Gravity retaining walls.

1.2 PROBLEM STATEMENT

Gravity and Cantilevere retaining walls are designed by trial and error. Trial section dimension are assumed and earth pressures acting on that section are computed based on some earth pressure theory. The stability at the wall is checked against overturning, sliding and bearing capacity failures. The section dimensions are revised until the wall is externally stable against these failure modes with a reasonable safety margin and reasonable economy. Then computations for bending moments, shears and required reinforcement are performed to ensure internal stability of the wall. For the safe and economical design of such retaining walls, a geotechnical engineer is required to make use of his experience and judgment besides using theoretical and empirical methods

This study critically assessed basic thumb rules required to implement the design of gravity and cantilever retaining walls. Indeed manual computational methods available for the design of cantilever retaining walls are tedious and time consuming and can be viewed as things of past in this age of computers. There is a need to make an effort to reduce the time involved in design and analysis of retaining walls and other structures. Computer software are available now days, but, may not suit user requirements. However, by understanding a computer language a user friendly and cheap "Design Software" can be developed by civil engineers.

1.3 OBJECTIVES OF STUDY

The project was under taken with following goals in mind:

- Fully understand the soil mechanics involved in design of cantilever and gravity retaining walls.
- Carry out structural design of cantilever and gravity retaining walls with cohesion less back fill.
- Comprehend and fabricate objective oriented software.
- Draw necessary conclusions and give recommendations on the basis of the outcome and design experience of such walls.

1.4 METHODOLOGY OF STUDY

A variety of materials are available covering many aspects of the subject. Many good books were consulted. Useful and relevant material was sifted by our syndicate in consultation with our Sponsor DS. Although a few minor variations were found in the structural design but the basics were same. Many examples were manually solved and analyzed.

The scope of the project was set and a number of algorithms were prepared to convert into computer language of visual basic. Again apart from books a few professionals were also consulted for the program coding.

After coding the software was run on available solved examples and results were matched. On basis of above procedures and problems faced the conclusions and recommendations were chalked out and finally the script was formulated with the help of syndicate DS.

1.5 SCOPE AND LIMITATIONS

The study encompasses the design of retaining walls for different cases encountered in the field. Special cases and other complex problems associated with lateral earth pressure and retaining walls, which deserves some more amount of study, have not been given their due share, owing to the limitation of time and weightage assigned to the project work.

The design aspects and methods are those taught at the under graduate level, therefore the program tailored to deal only the general problems. For most of us, Visual Basic was complex programming language, considerable time was spend on learning, yet it is a very versatile language and requires a lot of practice to harness its full benefits.

The data should be realistic and be within following limits. **(ACI-CODE)**

- Height of Gravity Retaining Wall = 10 to 15ft

- Height of Cantilever Retaining Wall = 10 to 25ft
- Unit weight of Soil = 80 to 130 pcf
- 28 days cylinder strength of concrete fc' =2500 to 6000 Psi
- Yield strength of steel fy = 36000 to 80000 Psi
- Angle of internal friction = 20 to 45 degrees
- Cohesion = 0 to 375 Psf
- Angle of inclination = 1 to 45 Degrees
- Surcharge load = 0 to 10ft of soil
- Bearing Capacity of soil = 1500 to 50000 Psf
- Co-efficient of friction = 0.2 to 0.65
- Structural design of Cantilever Retaining Wall was based on ACI 318-02

Chapter 2

SOIL MECHANICS

2.1 INTRODUCTION

The soil has a greater influence on behavior of earth retaining structures. Soil also does not have a homogenous structure, hence behave in the same way and further changes with addition of water. Owing to the complex nature of soil and its behavior under different conditions, the scope of the study has been limited to only granular backfill soils. Soil has characteristics of both solid and liquid due to which it exerts tremendous amount of lateral thrust on retaining structures. This property of soil is very important in soil engineering practices. The magnitude of lateral earth pressure varies considerably with the physical properties of soil, the soil conditions, external loads and many other parameters.

2.2 LATERAL EARTH PRESSURE

"Lateral earth pressure is the force which is exerted by the soil mass and which acts upon earth wall interface. The magnitude of lateral earth pressure is determined by the physical properties of the soil, the physical interaction between the soil and the structure and the value of displacements and deformations.

To simplify lateral earth pressure predictions, many designers assume an active state of stress on the driving side (heel side) of the wall (Bowles 1988, Das 1984). However some engineers (e.g., Matsuo et al. 1978) and agencies (e.g., U.S. Army, 1989) recommend assuming an at-rest state of stress. The active state corresponds to the minimum lateral pressure that can develop and at-rest state implies that no deformations or displacements occur. In reality an intermediate state of stress (a state of stress between active state and at rest state) may exist on the driving side but it is difficult to predict earth pressure under this state of stress. The passive state or stress is the greatest stress that may exist on the resisting side (toe side) of the wall (Lambe and Whitman 1969) and corresponds to lateral compression to failure.

2.2.1 At Rest Earth Pressure

Non-yielding walls such as bridge abutments experience at-rest or even higher pressures because of their rigidity. For such walls with normally consolidated cohesion less backfills for horizontal backfill surface, the coefficient of at-rest surface Ko can estimated using Jaky's equation as (Jaky 1944):

$$Ko = I\text{-}\sin \Phi \qquad (2.1)$$

Where Φ is drained friction angle of the backfill.

Experimental results suggest that Jaky's equation is valid only when backfill is deposited at its loosest state behind retaining walls (Sherif et al. 1984). For normally consolidated sloping backfills, the equation proposed in the Danish Code (Danish Geotechnical Institute 1978) can be used for computing at-rest earth pressure coefficient, KO as:

$$KO = K\,(1 + \sin p) \qquad (2.2)$$

The stress distribution behind non-yielding walls is assumed linear. Where a water table is not present, the resultant force acts at one-third of wall height from the wall base.

2.2.2 Earth Pressure at Intermediate Active State

An intermediate active state corresponds to the state of stress between active and at rest states. The prediction of lateral earth pressure at intermediate active state is difficult to estimate due to partial mobilization of shear strength of the backfill. Some researchers made use of finite element methods or developed analytical methods to predict earth pressure at any intermediate active state. However, these methods are not being used in routine design of retaining structures due to uncertainty in assumptions on which these methods are based.

2.2.3 Active Earth Pressure

Active earth pressure is the minimum pressure on the wall as it moves away from the backfill it is supporting. When lateral pressure approaches Pa soil behind the wall fails along a rupture plane. The mass of soil overlaying this rupture plane is in a state of plastic equilibrium i.e. at all points in this active zone shear resistance having been fully mobilized. In this state the major principal stress is horizontal. A body of soil is said to be in plastic equilibrium if every point of it is on the verge of failure. When the soil is in elastic equilibrium (i.e. at rest) the ratio of horizontal to vertical stress is called co-efficient of earth pressure at rest.

Passive Soil Pressure

Active Soil Pressure

Figure 2.1 Active and Passive Soil Pressure

2.2.4 Passive Earth Pressure

Passive earth pressure Pa is maximum pressure experienced by the wall as it is pushed towards the soil mass it is supporting. The pressure on the wall does not increase with the further movement as the soil has already failed along a rupture plane. The mass of soil over laying this rupture is in state of a plastic equilibrium i.e. every point in the mass of soil is on verge of failure. This zone of soil is called passive zone. At all points in this zone shear resistance have been fully mobilized. In this case the major principal stress is the horizontal stress where as the minor principal stress is vertical stress.

2.3 FACTORS AFFECTING ACTUAL EARTH PRESSURE

The main factors that affect actual earth pressure include wall movement, wall friction later in the backfill, and compaction of the backfill, surcharges and seasonal variation. These are discussed briefly in the following sections.

2.3.1 Wall Movement

Under service conditions a rigid retaining wall possibly can rotate about its base, its top, its middle, or another point; it can translate or there can be combination of these modes. It is also possible that wall may not yield at all. Terzaghi (1934), is believed to be the first to conclude from his large-scale earth pressure tests that the lateral earth pressure distribution behind retaining walls is associated with the type and the magnitude of wall movement

Earth pressure distribution behind a wall rotating about its base is considered as linear (Terzaghi 1934). For wall rotating about its top, Wu (1966) derived an expression that yields a semi-parabolic pressure distribution.

2.3.2 Wall Friction

A relative movement between a retaining wall and its backfill develops shear forces between the face of the wall and the backfill. These are termed wall friction (Lambe and Whitman 1969). The wall friction depends on roughness of the wall, amount and direction or the wall movement, soil properties of the backfill and inclination of ground surface behind the wall. The wall friction is also influenced by the settlement of the structure and the backfill soil (Grandi11987).

The maximum value of wall friction may not occur simultaneously with the maximum shearing resistance and the value of wall friction may vary across the wall (Bowles 1982). It was found that wall friction angle from 1/3 to 2/3 is a good approximation for computing lateral earth pressure using Coulomb's theory. The wall friction affects the magnitude and the direction of earth pressure

2.3.3 Water in the Backfill

The presence cf water in the wall backfill without compensating water in front substantially increases total pressure because the coefficient K for water is unity. The distribution of pressure on a retaining wall due to water is hydrostatic and the resultant water force acts at one-third the water height from wall base. The effective earth force acts above one-third of wall height due to the presence of water in the backfill. The combine force due to soil and water acts below the one-third of wall height Because of the buoyant effect of water on soil particles, the weight of submerged soil is reduced by an amount equal to the weight of water displaced by the soil particles.

2.3.4 Surcharge

Surcharge such as infinite uniformly distributed loads, concentrated line loads and strip loads increase lateral earth pressure on retaining structures. Only the effect of uniformly distributed loads on retaining walls is considered in this study. The lateral earth pressure exerted by a uniformly distributed surcharge q from the surface of the backfill downward is given as:

$$Pa=Kq \tag{2.4}$$

Where K is coefficient of earth pressure and has the same value as that used for the determination of lateral earth pressure exerted by the soil. A uniform surcharge results in a rectangular pressure distribution diagram. Which should be added to the lateral earth pressure diagram of the backfi1l. Methods for computing the increase in lateral earth pressure due to other types of surcharge and a load is not discussed here.

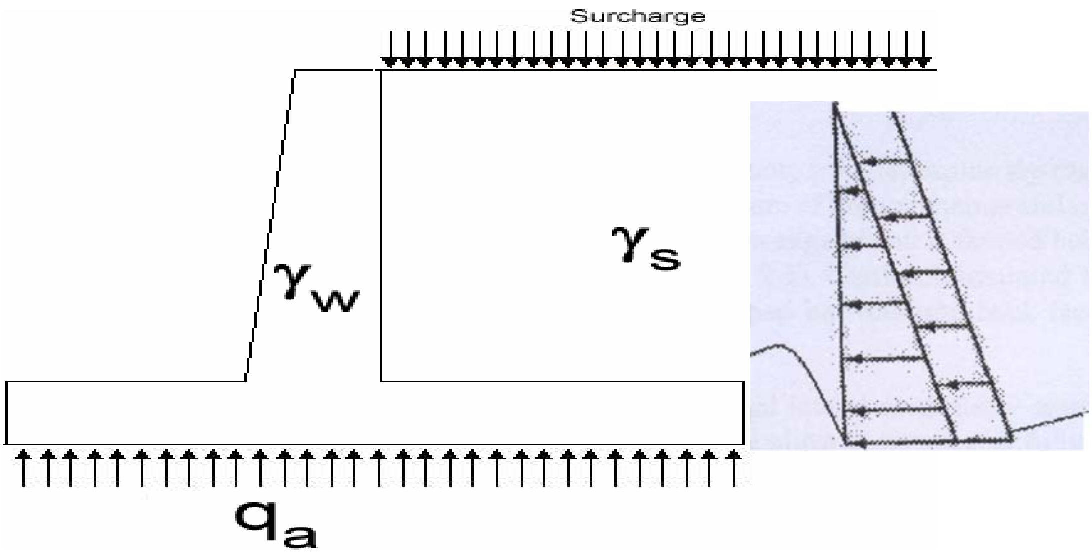

Figure 2.2 Effect of Surcharge

2.3.5 Seasonal Variation

Seasonal variations affect the magnitude and the distribution of earth pressure mainly because of changes in temperature, freezing of backfi11 and heavy rainfall. Field test results show about 30 percent variation of earth pressure due to seasonal variations (Terzaghi and Peck 1967). The earth pressure usually Increases in the summer and decreases in the winter (Duncan et al., 1990). The warm front face of a wall in the summer expands relative to the back face and causes the wall to deflect against the fill. The result is an increase in earth Pressure. In the winter the pressure is reversed. However, the earth pressure also increases in winter due to expansion of water in the backfill voids due to freezing It is also observed about 50 percent increase in earth pressure due to simulated rain fall at the rate of 5 inches per hours.

2.4 FIELD MEASUREMENT OF LATERAL EARTH PRESSURE

Many researchers have reported measured earth pressure on walls as higher than active pressures (Goulds 1970, Broms and Angelson 1971). Field measurements of lateral earth pressure were conducted by Coyle et al. (1974) and Wright et al. (1975) on an 18.25 feet high retaining wall with cohesion less backfill. The wall was founded on piles and the base was restrained to a great extent due to pile foundation. The measured pressures agreed with theoretical pressures in the upper part of the wall but were found to be close to at rest values in the lower part of the wall. This is expected due to the safety factors incorporated in the design; the wall did not move far enough for the pressure to reduce to active value.

2.5 ACTIVE AND PASSIVE EARTH PRESSURE THEORIES

The classical earth pressure theories of Coulomb and Rankine were developed in 1776 and 1857 respectively. In one way or the other, these theories are still in use for the prediction of lateral earth pressure behind retaining structures. To check if these theories are applicable for earth pressure prediction against cantilever retaining walls, a brief review is presented in the following sections.

2.5.1 Coulomb's Theory (1776)

The basic approach of Coulomb's theory is to determine the minimum and maximum earth force consistent with limiting equilibrium of homogeneous and isotropic soil mass. For cohesion less soils coulomb assumed that a wedge of soil is fanned behind walls with planar back face at minimum lateral force (Figure 2.3). Coulomb assumed that the failure wedge acts as a rigid body and friction is developed between the back face of the wall and the failure wedge.

Coulomb's theory solves for the total lateral earth force assuming that the rupture surface is a plane (Figure 2.3) and that the shear resistance is fully mobilized along this plane. For the active case shear on the failure plane acts against gravity and the largest total lateral earth force computed for any orientation e represents the smallest value of lateral earth force on the back of a retaining wall. Coulomb's theory yields only the resultant force; the state of stress within the soil mass is not obtained. As moment equilibrium is not considered, the location of resultant with respect to wall height is unknown.

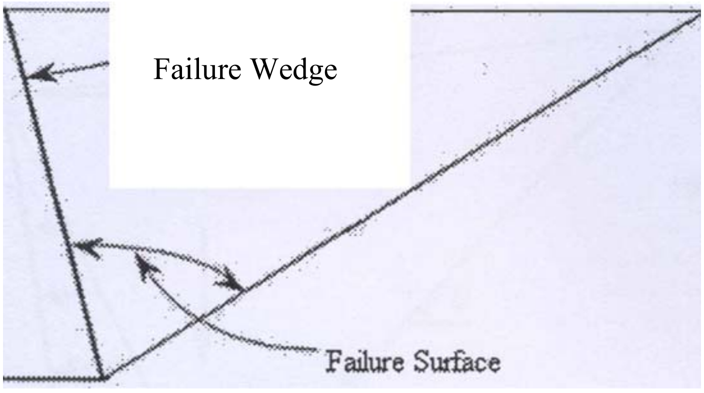

Figure 2.3 Coulomb Failure Wedge

A linear pressure distribution increasing with depth is usually assumed behind a wall increasing with depth for the case of a level or uniformly sloping ground without surcharge or water table provided the soil mass is in a state of failure throughout the wedge. This solution places the point of application of the resultant lateral earth force at one-third of wall height.

The magnitude of the active earth force against the wall is obtained by considering the equilibrium of the failure wedge (Figure 2.3). W is the weight of soil with in the failure wedge. Pa is the resultant active force acting at the wall friction angle 0 from the normal. R is the resultant of the normal and shear stresses along the rupture surface and acts at an angle of shear resistance from the normal. The analytical solution of forces acting on failure wedge yields

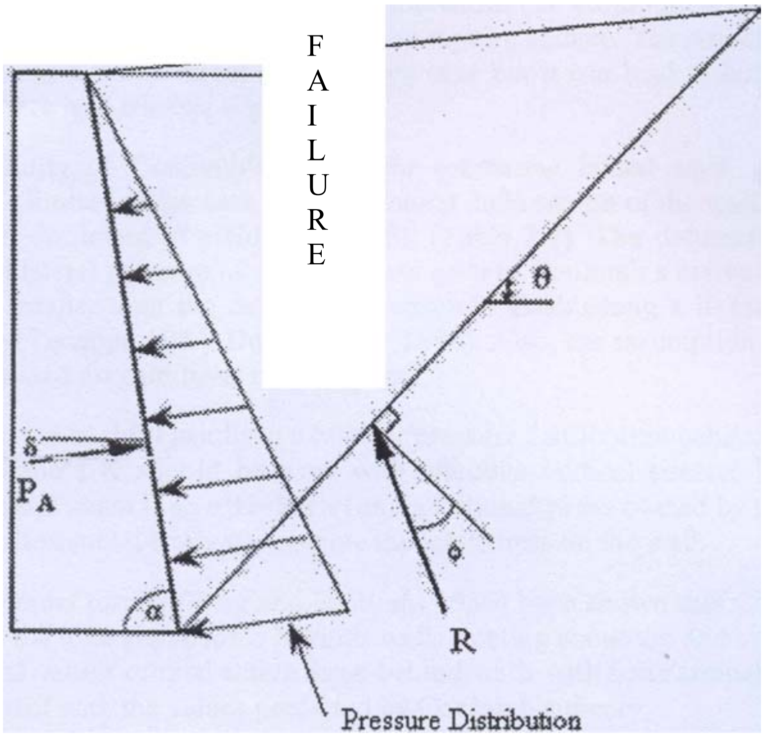

Figure 2.4 Assmned Conditions for Failure : Coulomb

$$\mathbf{Pa = Ka\gamma H^2/2} \qquad (2.1)$$

Where γ is unit weight of the backfill, H is the wall height, K is a coefficient of active earth pressure and is given as:

$$\mathbf{Ka = \frac{\sin^2(\alpha+\Phi)}{\sin^2\alpha \sin(\alpha-\beta)\left[1+\sqrt{\{\sin(\Phi+\delta)\sin(\Phi-\beta)/\sin(\alpha-\beta)\sin(\alpha+\beta)\}}\right]^2}} \qquad (2.2)$$

Where *a,* β, Φ and δ are angles as shown in Figure 2.3. Similarly, the expression for the passive earth force Pp is written as:

$$\mathbf{Pp = Kp\gamma H} \qquad (2.3)$$

Where Kp is coefficient of passive earth pressure and is given as:

$$\mathbf{Kp = \frac{\sin^2(\alpha-\Phi)}{\sin^2\alpha \sin(\alpha+\delta)\left[1-\sqrt{\{\sin(\Phi+\delta)\sin(\Phi+\beta)/\sin(\alpha+\delta)\sin(\alpha+\beta)\}}\right]^2}} \qquad (2.4)$$

For horizontal backfill, vertical wall back face (α=90 deg) and no wall friction, the coefficients of active and passive earth pressure reduce to $Ka=\tan^2(45-\Phi/2)$ and

Kp=tan^2(45+Φ/2) respectively. The main deficiencies in Coulomb's theory are in the assumptions involving the ideal soil and a plane rupture surface. The assumption of a plane rupture surface has a minor effect in the active case but it can lead to large errors for the passive case where wall friction is present.

The validity of Coulomb's theory for estimating lateral earth pressure behind retaining wall is limited to the case in which lateral deformation of the wall exceeds certain minimum limits sufficient to yield the backfill (Table 2.2). The deformation required to reduce the total lateral pressure of cohesion less soils to Coulomb's active state of stress is about 5 times smaller than the deformation required establishing a linear distribution of lateral pressure (Terzaghi 1934, Duncan et al. 1990). Also, the assumption of a planar wall back face is violated for cantilever retaining wall.

Water in the backfill results in a bilinear pressure distribution behind the wall. In this situation, Coulomb's K should be used with effective vertical stresses to compute effective horizontal stress (i.e., σ'H=Kσ'v) and additional stress caused by the water should be added to the horizontal stress to compute the total stress on the wall.

Expenmental results (Fang and Ishibashi 1986) have shown that Coulomb's theory underestimates the total active force behinds walls rotating about top and bottom. However, the experimental values of total active force behind walls with translational movements are in close agreement with the values predicted by Coulomb's theory.

2.5.2 Rankine's Theory (1857)

Rankine considered a semi-infinite cohesion less soil mass that can develop a state of failure with the slightest deformation. He examined the effect of lateral expansion and compression of such a soil mass. The state of stress under active and passive conditions are called Rankin's states and require that there be no shearing stresses on vertical planes in a soil mass with a horizontal ground surface. In reality no semi-infinite mass of cohesion less soil exists. Even if it exists, it is not possible to develop a state of failure in the whole soil mass with a slight deformation as known from relation between stress and strain in the soil. However, Rankin's state of stress can be induced in wedge-shaped zone ABC (Figure 2.5).

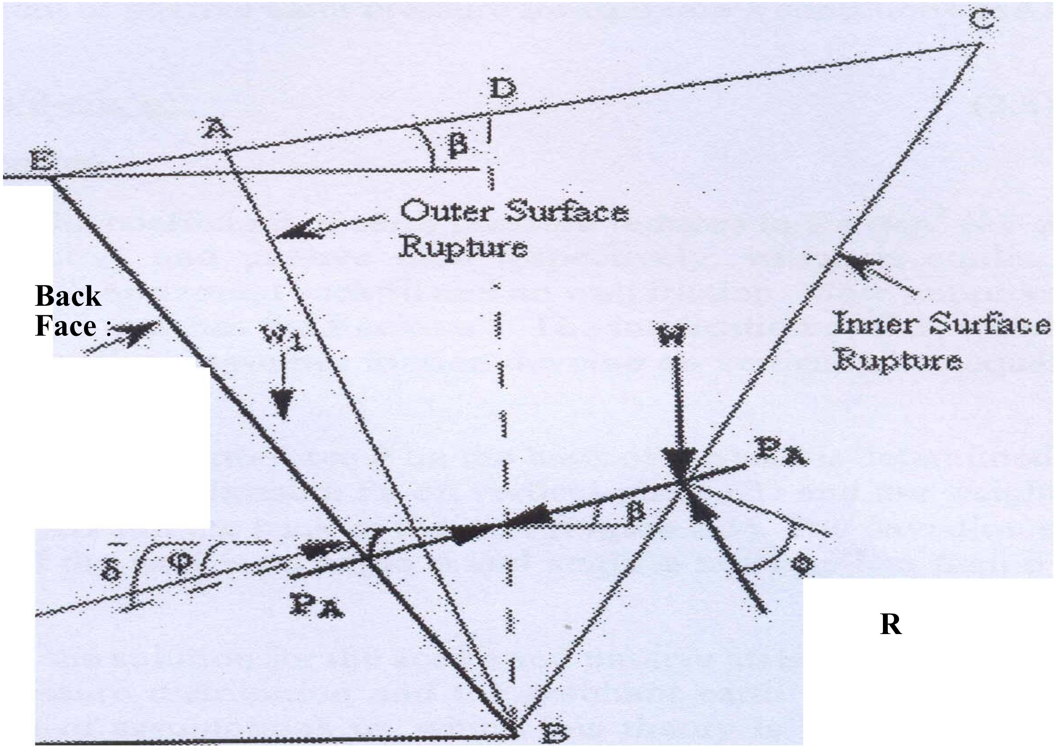

Figure 2.5 **Rankine's Active State of Stress**

When applying Rankine's theory to retaining walls, the inner and outer rupture surfaces must be within the soil. This implies that no sliding occurs between the back face of the wall and the soil. The soil wedge ABC remains with the wall in an elastic state of stress. The failure wedge would slide along the outer rupture surface AB instead of back face of the wall EB. As no rupture surface occurs along the back face of retaining wall; the lateral earth pressure is determined on vertical plane which passes through B. Then the resultant of active earth pressures Pa against vertical plane BD is given as:

Pa =KaγH'^2/2 (2.5)

Where Ka is a coefficient of active earth pressure for Rankin's conditions and is given as:

Ka = <u>**cosB cosB- (cos^2B-cos^2Φ)1/2**</u> (2.6)

cosB+(cos^2-cos^2Φ)1/2

Where B is backfill slope angle (Figure 2.5). Similarly the resultant passive force Pp can be computed by analogy as:

Pp=KpγH'^2/2 (2.7)

Where Kp is a coefficient of passive earth pressure for Rankine's conditions and it is given as:

$$\mathbf{Kp} = \frac{\cos B \cos B - (\cos^2 B - \cos^2 \Phi)^{1/2}}{\cos B + (\cos^2 B - \cos^2 \Phi)^{1/2}} \tag{2.8}$$

For horizontal backfill, the coefficient of earth pressure reduces to Ka=tan^2 (45-Φ/2) and Kp= tan^2 (45+Φ/2) for active and passive case respectively, which is similar to Coulomb's solution for walls with horizontal backfill and no wall friction. More importantly when δ=p in Coulomb solution, it matches the Rankine's. The implication of this, which is often miss stated is that Rankine analysis assumes friction develop on vertical plane equal to slope angle.

The magnitude of the resultant earth force P on the back of the wall is determined by combining the resultant of lateral earth pressure Pa on vertical plane BD and the weight of the soil Wl between the plane AB and the back of the wall (Figure 2.5). The deviation of P from the normal to the back of the wall is an angle Φ and angle Φ must be less than δ for Rankine's theory to apply.

Rankine's theory yields the solution for the active and passive state of stress in a soil mass. Therefore, the earth pressure distribution and the resultant earth force are known, provided there is no violation of assumptions on which this theory is based. Rankine's theory implies linear distribution of earth pressure with depth behind the wall and places the resultant of forces one-third of height H for the case of a wall without considering the effect of water presence in the backfill (Figure 2.5).

Chapter 3

RETAINING WALLS

3.1 INTRODUCTION

A retaining wall is a structure built for the purpose of holding back or retaining or providing one-sided lateral confinement for soil or other loose material. The loose material being retained pushes against the wall, tending to overturn and slide it. Retaining walls are used in many design situations where there are abrupt changes in the ground slope. Perhaps the most obvious examples to the reader occur along highway or railroad cuts and fills. Often retaining walls are used in these locations to reduce the quantities of cut and fill as well as the right-of-way width required if the soils were allowed to assume their natural slopes. Retaining walls are used in many other locations as well, such as for bridge abutments, basement walls, and culverts.

3.2 TYPES OF RETAINING WALLS

Retaining walls are generally classed as being gravity or cantilever types, with several variations possible. These are described in the paragraphs to follow:

3.2.1 Gravity Retaining Wall

Gravity retaining wall, Figure 3.1(a), is used for walls of up to about 10 to 15ft in height. It is usually constructed with plain concrete, stone, bricks, or block masonry for stability against sliding and overturning. It is usually so massive that it is unreinforced.

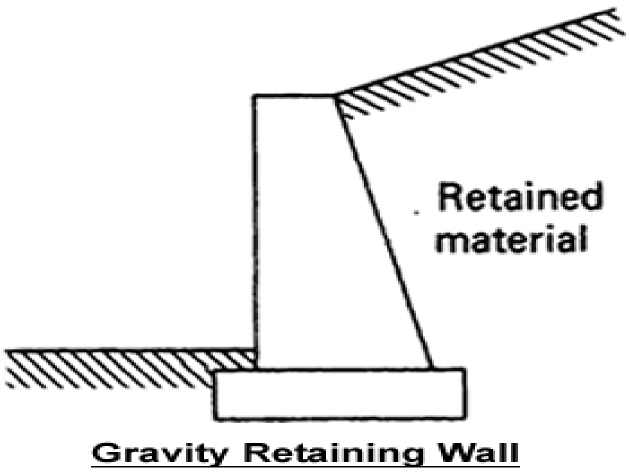

Figure 3.1(a) Gravity Retaining Wall

3.2.2 Semi Gravity Retaining Wall

Semi gravity retaining walls, Figure 3.1 (b), fall in between the gravity and cantilever types. They depend on their own weights plus the weight of some soil behind the wall to provide stability. Semi gravity walls are used for approximately the same range of heights as the gravity walls and usually have some light reinforcement.

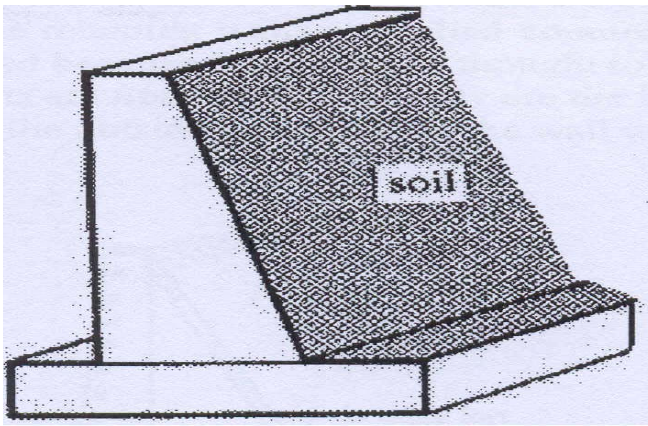

Figure 3.1(b) Semi-Gravity Retaining wall

3.2.3 Cantilever Retaining Wall

The cantilever retaining wall or one of its variations is the most common type of retaining wall. Such walls are generally used for heights from about 10 to 25 ft. In discussing retaining walls the vertical wall is referred to as the stem. The part of the footing that is pressed down into the soil is called the toe, while the part that tends to be lifted is called the heel. These parts are indicated for the cantilever retaining wall of Figure 13.1 (c).Since all components of wall act as cantilever hence called cantilever retaining wall.

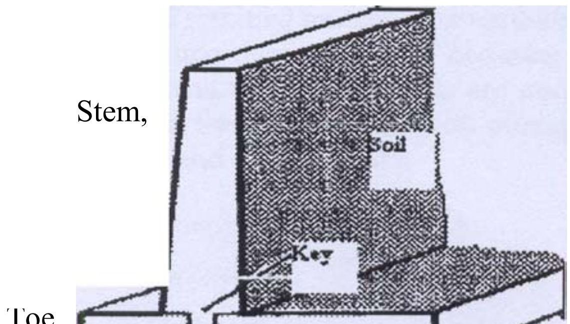

Figure 3.I(e) Cantilever Retaining Wall

3.2.4 Counter fort Retaining Wall

When it is necessary to construct retaining walls of greater heights than approximately 20 to 25 ft the bending moments at the junction of the stem and footing become so large that the designer will, from economic necessity, have to consider other types of walls to handle the moments. This can be done by introducing cross walls on the front or back of the stem. If the cross walls are behind the stem (that is, inside the soil) and not visible, the retaining walls are called counter fort walls. The counter fort type is more commonly used because it is normally thought to be more attractive because the cross walls or counter forts are not visible. Not only are the buttresses visible on the toe side, but their protrusion on the outside or toe side of the wall will use up valuable space.

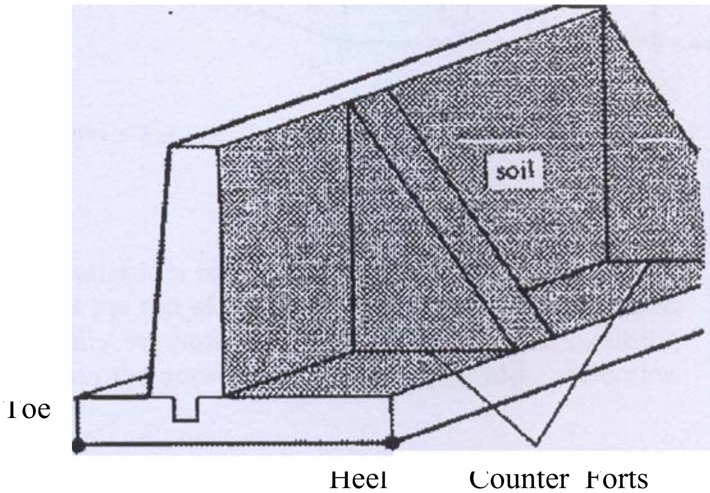

Figure 3.1(d) Counter fort Retaining wall

3.2.5 Buttress Retaining Wall

A buttress wall is similar to the counter fort except that the transverse supports are on opposite side of retained materia1. Nevertheless, buttresses are somewhat more efficient than counter forts since they consist of concrete that is put in compression by the overturning moments, whereas the counter forts are concrete members used in a tension situation and they need to be tied to the wall with stirrups. Occasionally, high walls are designed with both buttresses and counter forts.

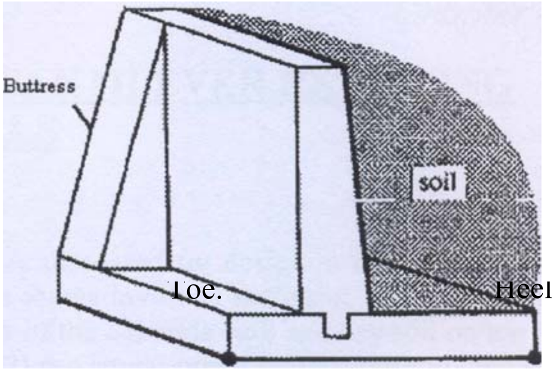

Figure 3. 1 (e) Buttress Retaining Wall

3.2.6 Bridge Abutment

A wall type bridge abutment acts similar to a retaining wall except that bridge deck provides an additional horizontal restraint at the top of the stem, thus abutment is designed as a beam fixed at the bottom and partially supported at the top .These are retaining structures with wing wall extensions to retain the approach fill and provide protection against erosion.

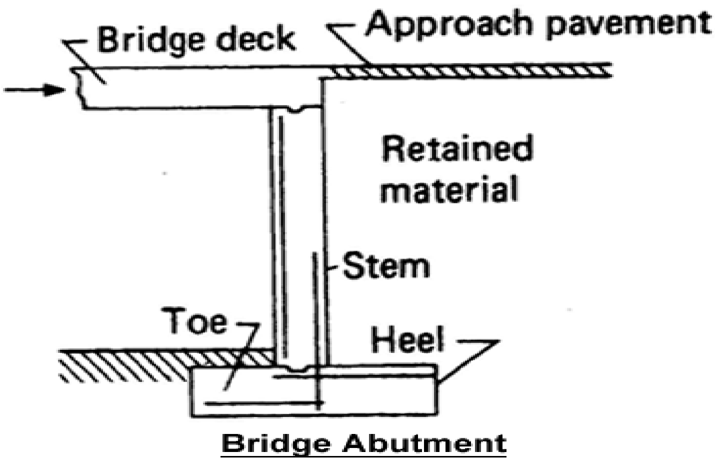

Figure 3.1(f) Bridge Abutment

Chapter 4

SOFTWARE DESIGN DEVELOPMENT

4.1 BACKGROUND

Numerous procedural computer programs have been used in the past for the design of retaining Walls. Newman (1976) used a procedural program to produce a structural design of cantilever wall for various loading conditions and heights. A computer program was used by Rhomberg and Street (1981) to compile a design aid for concrete cantilever retaining wall design with in 5% of absolute minimum cost. The U.S. Army Corps of Engineers (U.S. Army, 1989) makes extensive use of computer programs for the design of retaining structures.

In these programs, most if not all of the decision-making process is left to the user or alternatively, procedural programs and use numerous iterations for optimum design. In addition, they have a limited knowledge base and can't explain their working and justify their answers. Recently developed software have overcome some of the limitations of procedural programs.

4.2 PREVIOUS GEOTECHNICAL APPLICATIONS

Some previous applications of expert system technology to geotechnical engineering problems are briefly described in the following sections.

4.2.1 RETWALL

RETWALL (Hutchinson et al 1987) is an expert system for selecting and sizing earth retaining walls. RETWALL was implemented on SUN2 Microsystems using the expert system shell, a backward-chaining production rule system. The system also employs graphical procedures written in C. The user interface is through multiple windows and graphics. RETWALL provides extensive explanations in the form of 'Why', 'note list' and 'explain' predicates.

The Knowledge-base of **RETWALL** contains knowledge of various prototype retaining wall cross sections and how to select among them. This knowledge is not well documented in literature so it was obtained from specialist engineers in the field through survey. The selection of the most appropriate type of retaining wall is based on type of application, soil and topographical conditions and designer preferences.

The system has two main modules, the high level and low level. The purpose of high level is the selection of the particular retaining structure for a given application. At present, the system can choose among 10 available wall types. The lower level module is designed to perform the preliminary design of the selected retaining structure. Presently, the only lower level routine available is that which designs block work walls.

4.2.2 CONE

CONE (Mullarkey 1985) is a knowledge-based system that classifies soils based on cone penetrometer data and infers shear strength of the soil. The knowledge incorporated into CONE is embodied in a series of production rules using the OPS5 programming language and LISP functions.

The knowledge base of **CONE** consists of three main modules: information gathering, soil classification and shear strength estimation. Both backward and forward chaining control strategies are used in CONE. The information gathering process uses backward chaining strategy and the soil classification and shear strength estimation uses a forward chaining strategy. CONE uses fuzzy logic to represent the uncertainty in the empirica1 interpretation of the data.

4.2.3 RETAIN

RETAIN (Adams et aI., 1988 and Adam et a11990) is a KBES for retaining wall failure diagnosis and rehabilitation ign synthesis. RETAIN consists of a database implemented in DBMS INFORMIX and series of modules including site identification, failure diagnosis, design synthesis, cost estimation and evaluate/consistency- The database integrates OPS83 production rules, C language algorithmic functions and INFORMIX ESQL database queries. The RETAIN system uses DIGR inference engine to traverse the network. RETAIN system was developed on a Microwaves workstation but the entire system has also been transported to a PC.

The knowledge base for retaining wall failure diagnosis contains possible failure modes of all possible wall types as well as possible evidence associated with each failure mode. The heuristic knowledge base for design synthesis is represented as relational database tables. For problem-solving, the RETAIN system uses the derivation approach for retaining wall diagnosis and the formation approach for the synthesis or rehabilitation design (In the derivation approach, a solution of the problem at hand is derived from a list of predefined solution stored in the knowledge base; In the formation approach, a solution is formed from the eligible solution components stored in the knowledge base).

4.3 VISUAL BASIC

As already mentioned, Visual Basic was a new programming language, but was adopted for its versatility and lots of options that it offers. Work for developing computer program was started after carrying out detail study of the theoretical aspects of the subject matter. A brief introduction and major characteristic of language are given below.

Microsoft Visual Basic is the fastest and easiest way to create applications for Microsoft Windows. Visual provides with a complete set of tools to simplify rapid application development. The 'Visual' part refers to the method used to create the graphical user interface (GUI). Rather than writing numerous lines of codes to describe the appearance and location of interface elements, you simply add rebuilt objects into place on screen without having most of the skills necessary to create an effective user interface. The"Basic" part refers to the BASIC (Beginners All-Purpose Symbolic

Instruction Code) language used by more programmers than any other

language of computing.
Visual Basic has evolved from the original BASIC language and now contains several hundred statements, functions and keywords, many of which relate directly to the Windows (GUI). Beginners can create applications by learning just a few of the keywords, yet the power of the language allows professionals to accomplish anything than can be accomplished by other Windows programming language. The Visual Basic programming language is not unique to Visual Basic. The Visual Basic programming system, Application Edition included in Microsoft Excel, Microsoft Access, and many other Windows applications using the same language.

4.4 IMPORTANT FEATURES

Data access features allow you to create databases, front-end applications and scalable server-side components for most popular database formats, including Microsoft SQL Server and other enterprise-level databases. ActiveX(tm) technologies allow you to use the functionality provided by other applications, such as Microsoft Word Processor, Microsoft Excel spreadsheet and other Windows applications. You can even automate applications and objects created using the Professional or Enterprise editions of visual basic.

Internet capabilities make it easy to provide access to documents and applications across the Internet or intranet from within your application, or to create Internet server applications.

4.5 ADVANTAGES OF VISUAL BASIC 6.0

Following are the advantages of using visual basic-6:

- Microsoft Visual Basic is the fastest and easiest way to create applications for Microsoft Windows.
- Visual Basic provides with a complete set of tools to simplify rapid application development.
- Rather than writing numerous lines of code to describe the appearance and location of interface elements, you simply add rebuilt objects into screen.
- The Visual Basic Scripting Edition (VB Script) is a widely used scripting language and a subset of the Visual Basic language.
- The language has the Integrated Development Environment programming and one of the most powerful tools of Windows programming in the world today.
- Very Productive in providing appropriate tools for different

- aspects of user interface which allow you to create attractive and useful applications,
- Strong Data handling,
- Supports Windows GUI.
- Well matched to the application area of the proposed project
- Clear and simple, and displays a high degree of orthogonally.
- Has a syntax that is consistent and natural, and that promotes the readability of programs.
- Provide a small but powerful set of control abstractions.
- Provide an adequate set of primitive data abstractions.
- Support strong typing.
- Provide support for scoping and information hiding,
- Provide high-level support for functional and data abstraction.

4.6 PRELIMINARIES OF WALLS

Gravity and cantilever retaining walls are discussed for design in the next section, but whichever type is used, there will be three forces involved that must be brought into equilibrium. These include (1) the gravity loads of the concrete wall and any soil on top of the footing (the so-called developed weight), (2) the lateral pressure from the soil, and (3) the bearing resistance of the soil. In addition, the stresses within the structure have to be within permissible values and the loads must be supported in a manner such that undue settlements do not occur.

4.7 GRAVITY RETAINING WALLS

Retaining wall design proceeds with the selection of tentative dimension, which are then analyzed for stability and structural requirements and are revised as required. Since this is trial process, several solutions to the problem may be obtained all of which are satisfactory. A computer solution greatly simplifies the work in retaining wall design and provides the only practical mean to optimize the design procedure.

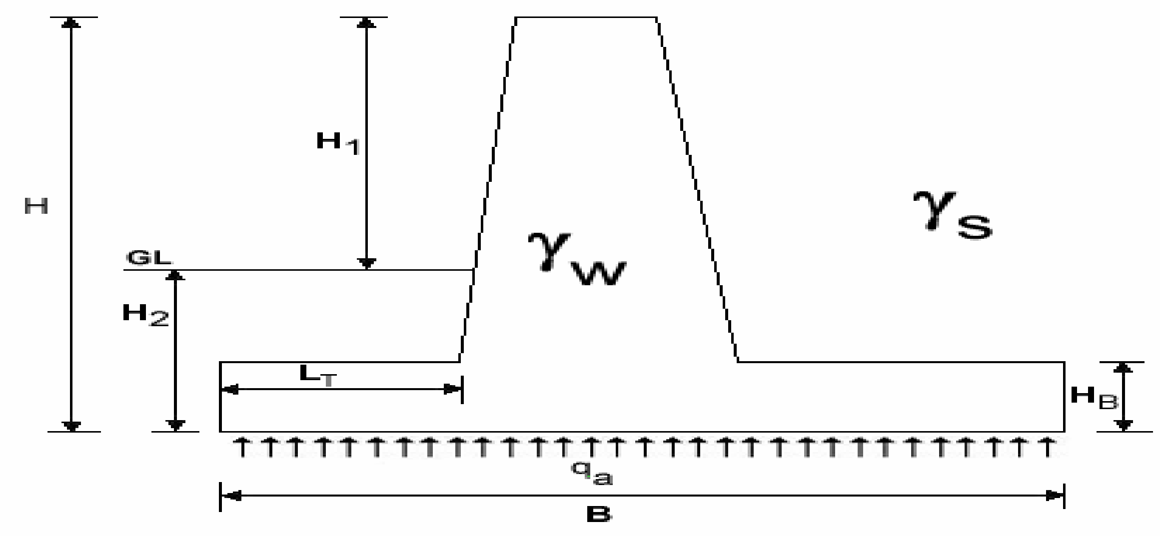

Figure 4.1 Gravity Retaining Wall

Gravity wall dimensions may be taken as shown in figure below. Gravity walls, generally, are trapezoidal shaped but, also may be built with broken backs. The base and other dimensions should be such that the resultant falls within the middle one third of the base. The top width of the stem should be on the order of 1 foot. If the heel projection is only 4 inches to 6 inches, the coulomb equation may be used for evaluating the lateral earth pressure, with the surface of sliding taken along the back face of the wall. The Rankine solution may be used along the back face of the wall, and on a Section taken through the heel Because 0f the massive proportions and resulting low concrete stresses, low strength concrete can generally be used for wall construction.

4.7.1 Stability of Wall.

Gravity retaining walls must provide adequate stability against sliding. The soil in front of the wall provides earth pressure resistance as the wall tends to slide into it. If the soil is excavated or eroded after the wall is built, the passive pressure component is not available and sliding instability may occur. If there is certainty of no loss of toe soil, the designer may use the passive pressure in this zone as part of the sliding resistance. Additional sliding stability may be derived from the use of key beneath the base.

4.7.2 Design of Gravity Retaining Wall

Gravity retaining wall is a structure which resists lateral earth pressure by its weight. A retaining wall is considered safe, if it is safe against

- Overturning
- Sliding
- Bearing failure

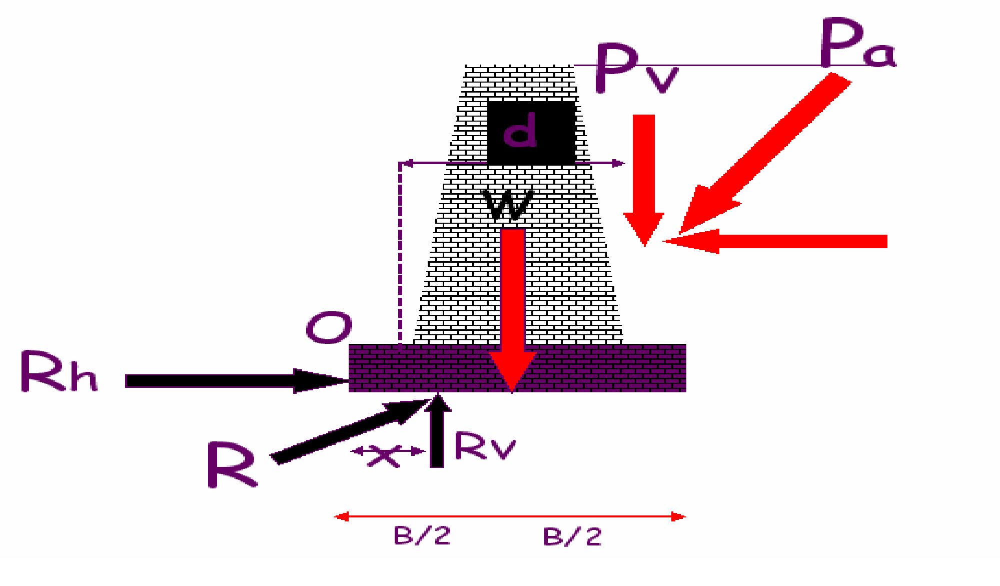

Figure 4.2 Forces On Gravity Wall

Consider a retaining wall as shown in the figure. Let R be the reaction from the base acting at distance X from toe of the wall O. the total active force can be split into its horizontal and vertical components Ph and Pv respectively. The passive earth pressure Pp is ignored. The wall will be safe against overturning if the resultant passes through middle third point of the base. Taking moment of all forces about 0,

X= {(Pv* d)-(Ph*b)}/Rv (4.1)

Where **Rv =W+Pv** (4.2)

The eccentricity of R from the centre of base is given as:

e=(B/2 -X) (4.3)

We must ensure that **e < B /6** so that there is no overturning.

4.7.2.1 FOS Against Overturning

FOS = Resisting moment / Overturning moment

= **(W*a + Pv*d) /Ph* b > 1.5** (4.4)

4.7.2.2 FOS Against Sliding

FOS = **(Rv* tanδPh) > 1.5** (4.5)

Where δ = friction angle between wall base and soil.

4.7.2.3 FOS Against Bearing Capacity

f max = Rv / B [1 + (6e / B)] (4.6)

f min = Rv / B [1 - (6e / B)] (4.7)

The maximum stress should not exceed bearing capacity of soil.

4.8 CANTILEVER RETAINING WALL

4.8.1. Development

The terms associated with a cantilever retaining wall cross section are shown in Figure 4.3. The stem acts as a cantilever beam which must resist lateral pressure caused by the backfill material. The base is the structural component that must transmit vertical forces and to some extent, horizontal forces, to the foundation soil. The heel is the portion of the base on the back side of the wall and the toe is the base portion on the front side of the wall. A base key may be provided to increase lateral resistance in case the foundation soil is weak. The key can be at different locations along the base. The backfill is material placed to elevate the ground surface to the design elevation, elevate the ground surface to the design elevation.

The general approach to the design of a cantilever retaining wall is to analyze conditions that would exist at the time of impending collapse and to apply suitable factors of safety to prevent such a collapse. This approach is known as limit design and requires limiting equilibrium mechanics. Whitman 1969). The design must ensure external and internal stability of the wall. External stability refers to wall stability against overturning, sliding and bearing capacity failures. The wall is considered internally stable when all its parts have sufficient flexural and shear strength to resist applied bending moments and shear.

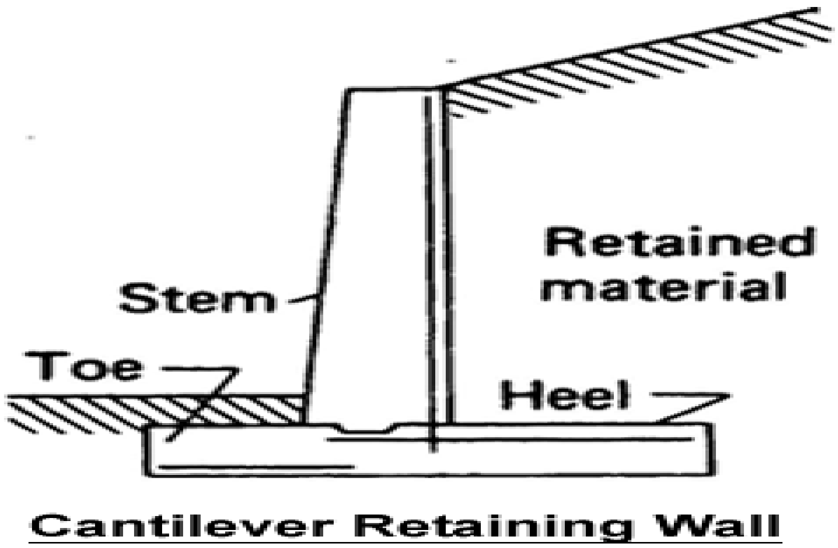

Figure 4.3 Cantilever Retaining Wall

4.8.2 Preliminary Proportions.

Site conditions impose certain constraints on wall proportions. The height of wall depends on the elevation difference across the wall and the depth of required cover on the toe side of the wall. The stem thickness and reinforcement must be sufficient to resist shears and moments due to earth pressure against the wall. The stem thickness at the top of the wall should be sufficient to permit easy placement of concrete. The dimensions of the base, heel, and toe are a function of result from the stability and strength requirements established in the design procedure. The required base width is a function of strength properties of backfill and foundation soils and the slopes of backfill. The thickness of the base depends on shears and moments at sections located at the front and the back of the stem. The base in front of the wall should be placed below the depth of frost action, zone of seasonal variation and the depth of scour. The toe and heel lengths are governed by overturning and sliding stability requirements. Recommended dimensions are given below.

Table: 4.1 Recommended Trial Dimensions For Cantilever Walls

Parts of wall	Peck(1974)	Das(1984)	Bowe1s(1988)
<u>Base</u>			
Width	0.4 to .65H	0.5 to 0.7 H	0.4 to 0.7 H
Thickness	H/12 to H/8	0.1 H	H/12 to H/lO
Toe length	B/3	0.1 H	B/3
<u>Stem</u>			
Top thickness	Sufficient For placement Of concrete	Minimum 30 cm	Minimum 20 cm Prefer 30 cm
Bottom thickness	Increase Thickness with depth by 1/4 to ¾ in/ft	0.1 H (same as base thickness)	H/12 to H/lO (same as base thickness)

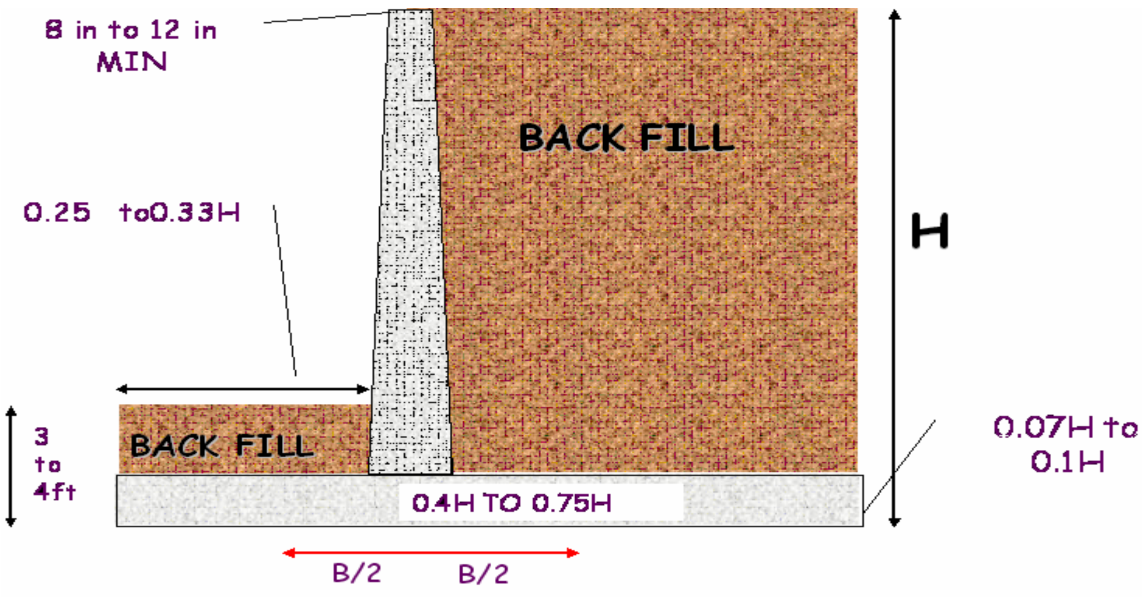

Figure 4.4 Wall Proportioning

4.8.3 Forces Acting on the Wall.

The force considered for the stability analysis of a unit width of a cantilever retaining wall include the driving-side soil force Pa against the vertical section AB at the end of the heel, the resisting-side soil force S against the horizontal section OB at the base, the effective soil force N acts vertically on the base OB and the total weight the wall components and the soil above the base. The principal structural elements of the wall are the stem, heel and toe. The forces acting on the stem are different from those acting on the wall due to difference in height.

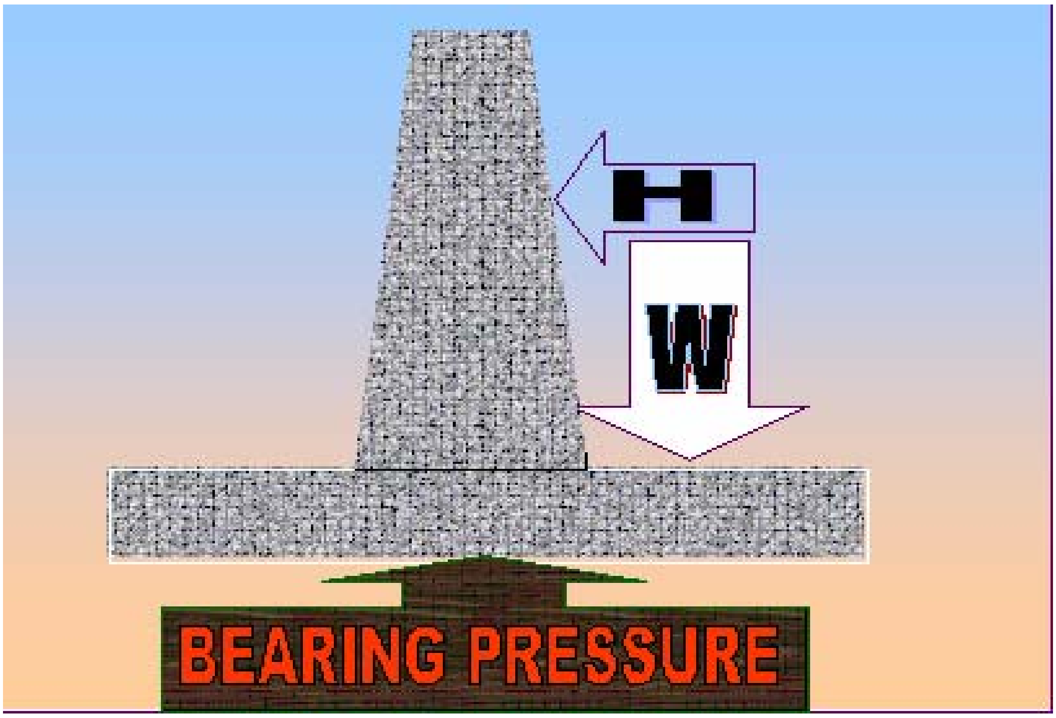

Figure 4.5 Forces on Wall

4.8.4 Structural Design Basis.

The structural design of a cantilever wall is based on factored service loads in recognition of the possibility of an increase above service conditions. The American Concrete Institute code 318-319 specifies that cantilever walls should be designed according to flexural design provisions of code but, the application of overload factors is not directly apparent. An overload factor is a number from 0.9 to 1.7 that is to be multiplied with the structural loads to satisfy the safety provisions of ACI Code. The overload factors given in ACI 9.2.4 for the design of members to resist earth pressure cannot be applied as easily to retaining walls as to the members in building.

There can be many different combinations of factored dead load D and live load L and the earth pressure. The ACI Code interpretation is:

- The dead load that increases design moment should be multiplied by 1.4, such as for heel.
- The dead load such as earth over heel that reduces the effect of earth pressure should be multiplied by 0.9.
- The effect of earth pressure such as soil pressure under heel should be multiplied by 1.7 (ACI 318-89). Obviously, these factored load states seem to be quite confusing to the designer and perhaps contradictory.

To simplify the Code interpretation, many engineers (Wang et aI., 1985, Ferguson et al., 1988, Nilson et al., 1986) neglect loads over the heel. The other reasons given for this neglect are that the actual soil pressure under the heel is very small and its distribution is non linear. This reasoning has been generally based on a pressure distribution.

There is also a possibility that the upward soil pressure under the heel is distributed. This distribution results when the resultant of forces R acts on the heel side from mid point of the base. In this case, neglection of soil pressure under the heel can lead to very conservative heel thickness. The overload factors commonly recommended by structural engineers for the design of cantilever retaining walls are given in table above. (Wang et aI., 1985, McCormack 1986). Many engineers (e.g., Newman 1976, Bowles 1988) recommended not neglecting soil pressure under the heel for heel designed subtracting it from loads over the heel before applying overload factors. Similarly, for toe design, the loads over the toe are subtracted from the upward soil pressure before applying load factors. Generally, the same load factors are used for the design of stem, heel and toe. The overload factors can vary between 1.7 to 1.9, depending upon importance of retaining structures and assumptions, such as active or at-rest, made for earth pressure computations

Table 4.2 Load Factors

Wall Pa	**Tvpe of loading**	**Load factor**
Stem	**Earth pressure**	**1.7**
Heel	**Soil and water over heel**	**1.4**
	Weight of heel	**1.4**
	Surcharge	**1.7**
Toe	**Weight of Toe**	**0.9**
	Soil pressure under toe	**1.7**

4.8.5 The Structural Design

The design of cantilever retaining wall is based on the analysis of typical one foot slice The Structural Design of stem, heel and toe is discussed in the following sections. The design is restricted to the calculation of pressure, shears, and moments required area of steel and development length. The discussion on splicing and other minor details is beyond the scope of this study.

4.8.5.1 Stem Design.

The stem is considered as a cantilever beam of unit width for computing steel area and treated as a cantilever slab while making checks for shear (Nilson et al., 1986). The shear and moment at the base of stem due to lateral earth pressure are computed and are used to determine the stem thickness and required steel reinforcement. The critical section for bending moment is taken at the bottom of stem and critical section for shear can be taken at distance d from the bottom of stem (ACI - 11.1.3.1), where d is a distance fron mid of reinforcement to the front face of the stem where stem joins the base.

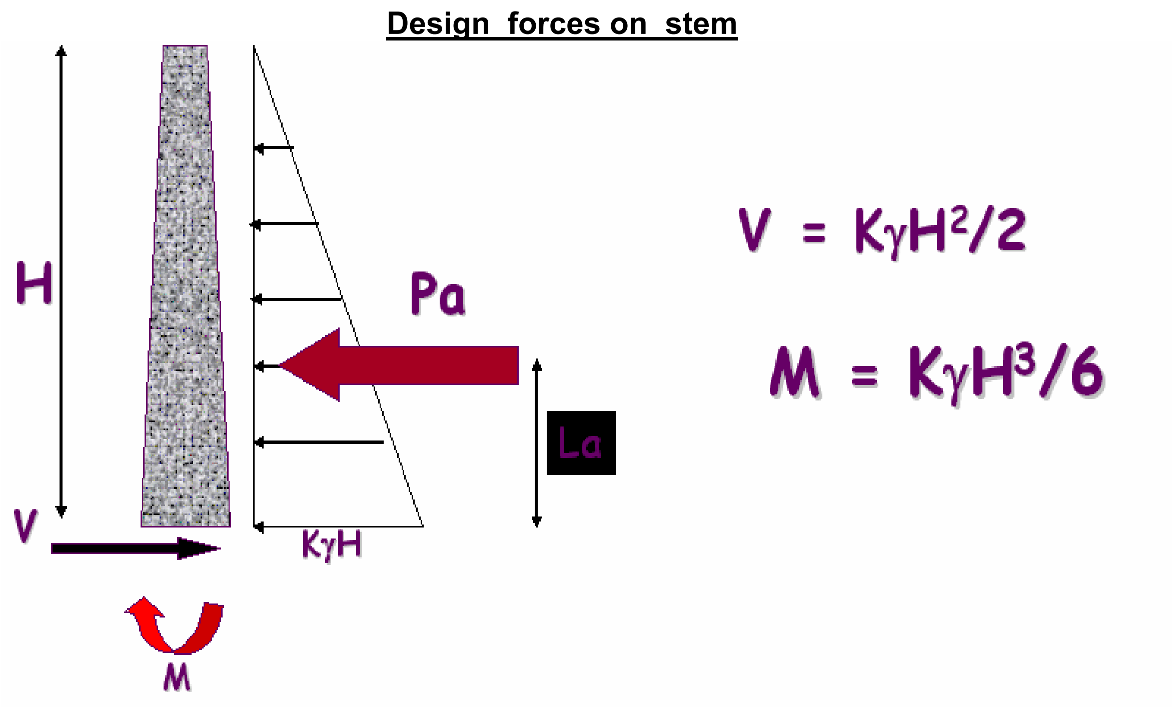

Figure 4.6 Forces on Stem

To make the calculations simple, many engineers usually take critical section for both shear and moment at the bottom of stem (McCormack 1986, Bowles 1988). The critical section for shear taken at stem bottom results in a slightly thicker stem which helps the stability of the wall but may reduce wall movement essential to develop active conditions behind the wall. The earth pressure on the stem due to backfill is assumed to vary bilinearly in the presence of water with depth from wall top and attains maximum value at a depth equal to the height of stem. The weight of stem causes some axial compression which can be neglected being very small. Then, the maximum pressure that acts at the bottom of the stem is given as:

$$P = K\,\gamma H \qquad (4.8)$$

Where K is coefficient of earth pressure which can vary between active and at-rest values, γ unit weights of backfill soil respectively and b is base thickness. The shears and moments at the bottom of the stem can be given as:

$$M=(K\gamma H)*La \qquad (4.9)$$

$$V= K\,\gamma H \qquad (4.10\}$$

Where Pa is lateral earth force on driving side of stem 0 is wall friction angle. La and B moment arms as shown in figure. Lateral earth force on toe side is neglected because of uncertainties associated with its presence.

The bending moment requires the use of vertical reinforcement on the backfill side of the stem. In addition, temperature and shrinkage reinforcement must be provided according to ACI- 14.3.3 Two third of temperature and shrinkage steel may be used on it on the face because of higher temperature variation. There should be just enough vertical steel on the face to support the horizontal reinforcement.

4.8.5.2 Heel Design.

The heel is designed as cantilever beam for downward forces acting over the heel and upward soil pressure which acts under the heel. The forces that are considered over the heel including the weight of the backfill, weight of heel itself, vertical component of lateral earth force unit weight of water. The forces acting over the heel tend to push the heel downward into the foundation soil. The necessary upward reaction to hold it attached with the stem is provided by the upward soil and water pressure under the heel and a shear force at the intersection with the stem.

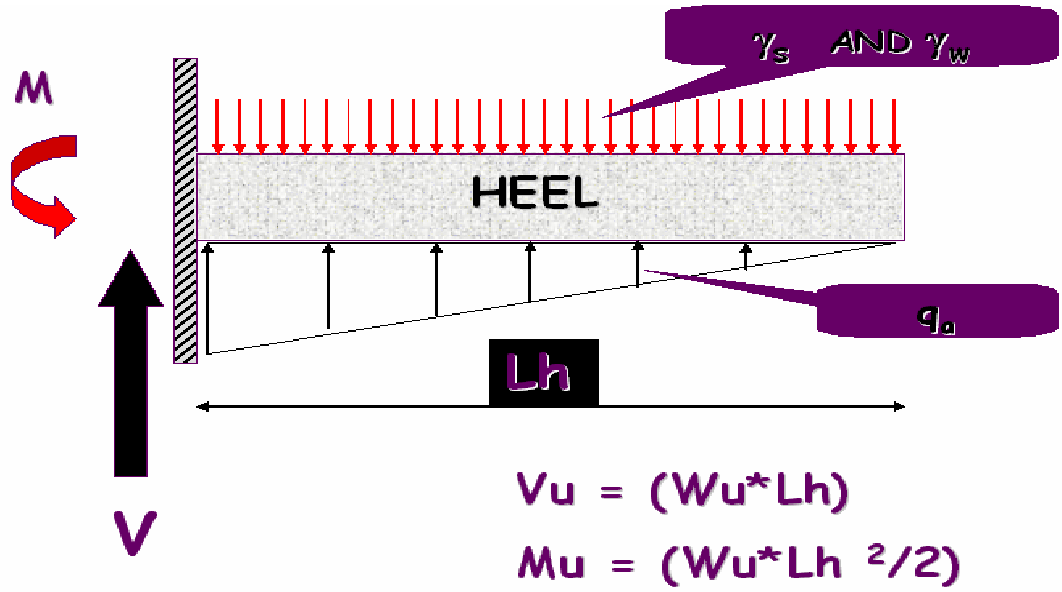

Figure 4.7 Forces on Heel

The critical section for the bending moment is taken at the centre of the stem steel because the plane of weakness is likely to occur at stem steel rather than at the back face of stem where it joins heel (Wang et aI., 1985). The code ACI 11.1.3 allows the critical section for the shear to be taken at distance d from the face only when compression is induced in the end region due to support reaction. In the present case, tension is induced in the concrete where heel joins the stem. Therefore, many designers consider the critical section for shear at a point where heel joins the stem. The shear usually controls the heel thickness (Wang, et aI., 1985, McCormack 1986). The shear and moment at the critical sections are computed to determine heel thicknesses and steel reinforcement. The expression for computing pressure q, shear V and moment M, at the critical sections are given as:

$$\mathbf{V = (Ws+Wc)*Lh} \tag{4.11}$$

$$\mathbf{M = (Ws+Wc)*Lh/2} \tag{4.12}$$

Where Ws and W c are unit weights of heel and soil over the free end of heel, q min is upward soil pressure at the end of heel Lh is the heel length . The base is not subjected to extreme temperature as it is well below the ground surface. Therefore, the requirement of temperature and shrinkage steel is very small and mainly this steel acts as binder bars for the main flexural steel. (McCormack 1986, Nilsson et aI., 1986).

4.8.5.3 Toe Design.

Toe is also designed as a cantilever beam for the weight of soil, water over the toe and the weight of toe itself all acting downward and the soil pressure under the soil pressure under the toe acting upward. The critical section for moment is taken at a point where toe joins the stem and the critical section for shear can be taken at a distance d from the face of the stem (Wang et aI., 1985). It is convenient to take the section at the face of support because usually the toe length is small. The thickness of toe need not be same as the thickness of heel but most designers make it same (McCormack 1986, Bowles 1988) for ease of construction. The shear and moment at the critical section are computed to determine toe thickness and reinforcement.

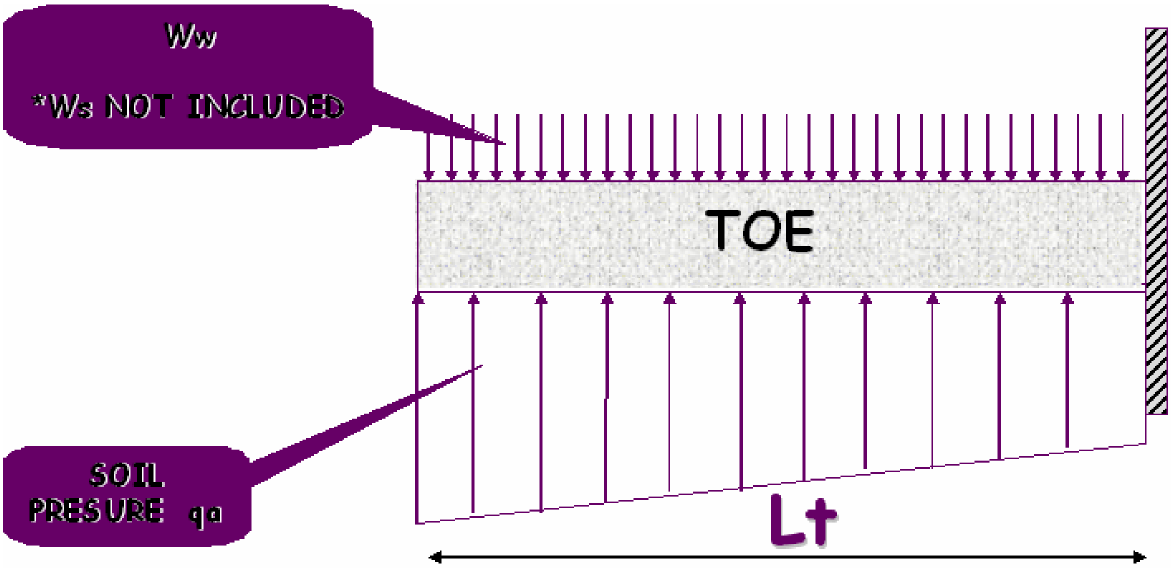

Figure 4.8 Forces on Toe

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4.8.5.4 Moment Capacity

In this study, the cantilever retaining wall is designed for shear and bending in accordance with strength design method of the ACI code. For computing nominal moment capacity (ΦMn), the stem, heel and toe are treated as cantilever beams of unit width and steel area is calculated according to the provisions of ACI code. The wall is considered safe against bending if nominal moment capacity of each component of wall exceeds the factored maximum moment (Mu) on that component. The expression for computing nominal moment capacity is given as:

$$\mathbf{\Phi Mn = \Phi\ As\ fy\ (\ d - a/2)} \qquad (4.13)$$

$$\mathbf{a = As\ fy\ /\ (\ 0.85\ f\ c' * b)} \qquad (4.14)$$

Where

A = steel area

fy '= Yield strength of steel

f c' = allowable compressive strength of concrete

d = distance from extreme fiber to the mid of steel
b = unit width

4.8.5.5 Checks for Shear.

Stirrups are not commonly used in the design of cantilever retaining wall. Therefore, each part of the wall must have enough strength to Stand against shear without using shear reinforcement. While making checks for shear, the stem, heel and toe are treated as a cantilever slab. The wall is considered safe against shear if nominal shear strength (ΦVc) of each part of the wall is greater than the maximum factored shear (Vu) acting on that part. The expression for computing the nominal shear strength of concrete is given as:

$$\mathbf{\Phi Vc = 2\ \Phi\sqrt{V}\ f'c\ b} \qquad (4.15)$$

Where Φ is strength reduction factor and is taken as 0.85 for shear design.

4.8.5.6 Soil Properties.

The soil properties of the foundation soil and the backfill must be defined for design in order to obtain an efficient and safe design. Lateral earth pressure depends on unit weight and friction angle of backfill. Although, probably stronger, the friction angle of typical clean granular backfill material used in retaining wall problems is often taken between 30 to 36 degrees and unit weights are often in the range of 100 to 120 pcf The shear strength and compressibility of foundation soil must be known to design against bearing capacity failures or excessive settlements. The parameters required for the foundation soil are the friction angle, unit weight and cohesion. These parameters can best be estimated using direct shear test since retaining wall is plane strain case. However, these parameters are often inferred from N value of SPT tests or from designer's experience.

4.9 DRAINAGE AND OTHER DETAILS

As already discussed that presence of water greatly influences the design of retaining walls, hence an effort is made to prevent development of hydraulic pressure under any circumstances. Apart from structural damage it results in uneconomical design. At times, the water pressure together with submerged back fill will produce a much larger force than a dry backfill. To lower this force (and to produce a more economical design) engineers provide drains or weep holes to reduce the buildup of water pressure behind walls.

Weep holes are 6 to 8in diameter drain pipes that run through the wall. They are typically spaced 8 to 10ft apart both horizontally and vertically. If water draining through the wall creates an unsightly appearance or leads to softening of the soil under the toe, a continuous back drain to carry the water to a drainage ditch or sewer may he used. The back drain, positioned behind the wail at the heel, consists of a perforated soil pipe surrounded by a graded stone filter. A filter prevents in flow of sand or silt that would clog the drain. Also to reduce the likelihood of water pressure building up behind a wall from rain or snow, many designers often slope the surface of the backfill down and away from the back of the wall or place an impermeable layer of soil (i.e. one with a high clay content) near the ground surface.

Chapter 5

CONCLUSION AND FUTURE WORK

5.1 SUMMARY

The topic of gravity and cantilever retaining walls is vast and worthy of more research. However, the purpose of this project was to cover the pertinent aspects specially related to computer aided design of gravity and cantilever retaining walls. Recognizing the power and utility of modem computers in technology reference to computer aided design has been appropriately made in the project. However it is believed that the study and assimilation of basic principles is of the highest priority. Once these are understood and mastered, then the computer program can be used with out much difficulty. This is the reason that the more emphasis has been placed in the practical aspects of the design, rather than on design theory and programming.

This dissertation has presented the development of computer software for the preliminary design of gravity and cantilever retaining walls. The design of these walls includes proportioning the wall dimensions to ensure external stability and providing sufficient steel (in case of cantilever walls) and concrete area to ensure internal stability. The developed system is structured to ensure external stability and, at the user's option, internal stability of the wall. The development stages of this program included basic knowledge collection from the literature, development of a prototype, its analysis, knowledge elicitation from instructors on wall design, development, testing and validation of the program. The present program is limited to cohesion less backfills and foundation soils not having significant settlement problems.

5.2 CONCLUSION

The objectives of the study have been achieved with the development of such software, but there is a dire need of joining two streams of knowledge, Civil Engineering and Computer Skills as one powerful tool for enhancing research and design capabilities of civil engineering profession. This Project work is just a very small step in that direction. Following conclusions are drawn from this experience:

- Visual Basic - 6 is quite a user friendly computer programming language, which can be utilized at its best with a sincere effort in Civil Engineering problems etc.
- This program is in the simplest form of programming language, with very vast area of Design yet to be covered.
- This program enables its users to exercise the Stability, Overturning,

Sliding and Bearing Capacity Checks in a comfortable way.

. This kind of programs cut short the laborious calculations for design and stability checks to benefit the users in terms of time and efforts.

. The development of this program demonstrates the viability of knowledge based techniques as an effective problem solving tool for design tasks that combine heuristic knowledge and extensive computations such as the design of retaining walls.

. In the process of developing program, the approaches used for the collection and formalization of the knowledge, development of the algorithms and finally the expansion of the developed algorithms by incorporating knowledge have proven successful. The same approach appears fruitful for similar design problems in civil engineering.

5.3 RECOMMENDATIONS

The interdisciplinary nature of this research has raised many questions; some of them remained unanswered and became required avenues for future research. These include the study of models to represent uncertainty in program's fuzzy logic and its application to design problems in geotechnical engineering and the development of knowledge-based systems for a variety of tasks in geotechnical engineering. The following areas appear to offer the greatest potential for further research:

. Further extension of the developed system to other types of structures including foundations, overhead water tanks, dams, canals and reinforced earth walls etc.

. Expansion computerized programs to estimate shear strength parameters of backfill and foundation soil from standard penetration test (SPT) or cone penetrometer test results etc.

. Retaining Wall Design has a number of cases, which due to paucity of time could not be covered in this project. These cases can be furthur worked upon and a comprehensive program regarding Design of Retaining Walls can be build by compiling these all efforts.

. A computer research department should be established in college where students can seek guidance for developing of small software.

. Like SAP2000, AUTOCAD etc. it is a step to learn and broaden the mental horizon pertaining to software development and its applications.

APPENDIX **1**

EXAMPLE NO 1

· **Statement**

A gravity retaining wall is required to retain a bank 11 ft 6" high. The top surface is subjected to live load surcharge of 400 Psf Unit weight of soil is 120 Pcf Angle of intemal friction is 30°. μ=0.55.The allowable soil pressure is 4 Ksf Unit of concrete weight is 150 Ib/ft3. Design a Gravity Retaining Wall.

· **Solution**

· **Trial Section**

Height of Retaining Wall above Ground level=11 '6"

Depth of Wall below Gr level=3'4" Say 3.5'

Total height of the wall = 15'

Width of Base=10'
Base Thickness = 2'

Width at Top = 2'

. **Earth Pressure**

Ka=(I-SinΦ)/(I +SinΦ) =0.33

P1 =Ka*γ*h^2/2=0.33*120*1 5*1 5/2= 4.45 Kips

P1 = 4.5k acing 5' above base i.e.h/3

P2=Ka*γ*hs*H=0.33*120*3.33*15=2 Kips acing at 7.5 foot from base i.e. H/2

. **Over Turning Moment**

OTM= 4.5*5+2*7.5=37.5 K ft

. **Stabilizing Moment**

<u>SM=(W2*2.5)+(WI *5)+(W3*5.5)+(W4*7.5)+(W5*9.75)</u>

Load(for 1ft) , K	Lever Arm ft	SM K'
Wl= 10*2*1 *150/1000=3	5	15
W2=13 *2* 1 *150/1000=3.9	2.5	9.75
W3=1/2*6* 13* 150/1000=5.85	5.5	32.18
W4=1/2*6* 13*120/1000=4.68	7.5	35.1
W5=0.5*13*120/1000=0.78	9.75	7.6
Total W=18.2i		99.6

. **FOS Against Overturning**

FOS= *SM / OTM* = 99.63/37.5 = 2.65>1.5 **(OK)**

. **FOS Against Sliding**

Sliding Force= P1 +P2= 6.5 K

Resisting Force= μ RV=μW=0.55*18.21=10.0IK

FOS=RF/SF= 10.01/6.5=1.54> 1.5 **(OK)**

. **Maximum Soil Pressure**

Case 1. No Surcharge between A and B

Resultant Rv= W=18.21K

Loc of Resultant from point '0'= (99.63-37.5)/18.21=3.41'

e=1.59'

q= Rv/L (H+-6e/L) = 18.21/10(1 +6*1.59/10)

qmax = 3.56Ksi

qmin = 0.084Ksi

Case 2. Surcharge between A and B

W= 18.21 +(6.5*400)/1000= 20.81K

SM= 4000*6*S/1000*6. 75+99.63=117.15K ft

Resultant Rv= 20.81 K ft

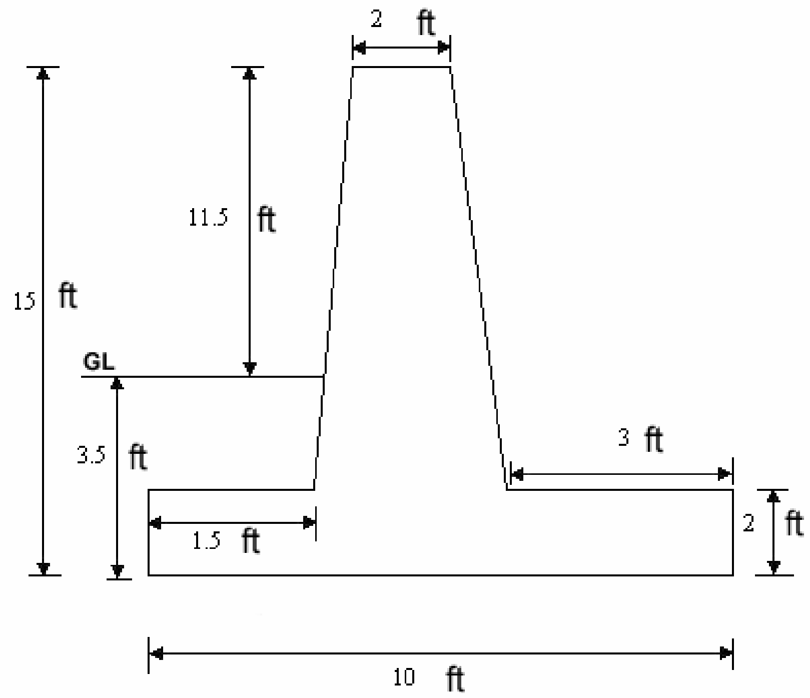
Loc of New Resultant= (117.15-37.5)/20.81 =3.83'

e= 5-3.83= 1.17'

q_{max}= 3.54 Ksf

q_{min}= 0.62 Ksf

Same as above



Final Dimensions

EXAMPLE NO 2

- **Statement**

A cantilever retaining wall is required to support a bank of earth 16 ft above ground level. The surcharge on the level backfill is equivalent to 8 ft of soil. Soil density 120 pcf and angle of internal friction is 35° and the co-efficient of friction b/w soil and concrete is 0.40 f c = 4 ksi and fy = 60 ksi. Max allowable soil pressure is to be limited to 5000 psi Design the cantilever retaining wall using strength method of ACI code.

- **Required:**

Design the cantilever retaining wall by strength design method.

- **Solution~**

  - **Trial Dimensions**:

Height of soil above ground level = 16ft

Assume 4ft depth of footing to prevent frost action etc.

Total height of retaining wall = 16 + 4 = 20ft

Assume base thickness (7 to 10 % of H) = 2 ft

Height of stem = 20-2 = 18 ft

Assume thickness of stem on top = 12 in

  - **Check Thickness of Stem**:

Co-efficient of active earth pressure.

$Ka = (1- \sin \Phi)/(1 + \sin \Phi) = (1- \sin 35)/(1 + \sin 35)=0.271$

Horizontal earth pressure:

$P1 =1/2 * Ka*\gamma*H^2 =1/2* 0.271 *120*18*18 = 5.27$ k/sqft

P1 is acting at 6ft above base

$P2 = Ka* \gamma *Hs*H=0.271 *8 *18*120 = 4.68$ k/sqft

P2 is acting at 9ft above base level

$P = (P1 +P2) = 9.95$ k/sqft

Moment at the base:

The stem behaves as a cantilever structure.

Mu = (5.27*6+4.68*9)*1.7=125.358K

Steel Ratio

p (max) = 0.75*0.85*β1 *fc *I* fy*87/(87+60)

= 0.75*0.85*0.85*4/60*87 (87+60)

p (max) = 0.0214

Let p = 0.5 p (max) = 0.5*0.0214 = 0.0107

- **Check Thickness of wall Slab From Flexure Criteria**

Mu=Φ*p*b*d2*fy(I-0.59 *p*fy/fc)

125.35*12 = 0.9*12*d2*0.0139*40(1-0.59*0.0139*40/3
d=16.77

Assume 2 in clear cover and #8 bars being used.

H=16. 77+2+ 1/2= 19 .27"=20"

d = 20-1/2-2 = 17.50

- **Check Thickness of Slab from Shear Criteria**

Maximum shear force = Vu = (P1 +P2) * 1.7 = 16.91K (ACl11.3.1.1)

Shear Capacity = Φ*Vc = 0.85*2*√fc*b*d

Φ*Vc =19.55K
Φ*Vc > Vu **(OK)**

- **Area of Steel**

BM at various levels

My = 0.092*3 √y+0.221√y (Gen eq)

Effective Depth

dy = 9.5 + 8y/18

Value of p

Mu*12 = 0.9* p*12*d2*40(1-.59*40/3*p)

. **Area of Steel**

Loc from Top	Mu (K)	D In	P	As In^2	Remarks
18	125.4	17.5	0.0126	2.65	
15	80.58	16.17	0.00923	1.79	
12	47.59	14.83	0.00632	1.12	
9	24.53	13.5	0.0050	0.81	P (min)
6	9.91	12.1 7	0.0050	0.73	P (min)
3	2.24	10.83	0.0050	0.65	P (min)

. **Temperature and Shrinkage Steel**

As (min) = 0.0025*b*h

= 0.0025*12*(20+12)/2=0.48in^2

Use #4 @ 10"c/c

. **Development Length**

ld = 3/40*40000/ -√*3000*1* *1/2.5*9/8=25"

. **Splice Length**

1.3*25= 32.5 =36"

. **Length of the Base**

PI' = Ka*γ*H^2/2 = 0.271 *120/1000*20*20/2 = 6.52K

P2' = Ka* γ*hs*H=0.271 *12*20*8=5.2K

W = (20+.8)*0.12*x = 3.36*x

By Taking Moment about Pt B

X=7.53ft

L = 1.5x = 1.5*7.53 = 11.5ft

- **Conditions of External Stability**

Stability Moment

Length of toe = L/3-L/4 = 3ft

Taking Moments About Pt 0

Weight (W) Kips	Lever Arm (ft)	Moment (K)
WI =(18+8)*0.12*6.83=21.31	8.09	172.40
W2=(1 *18)*1.5=2.7	4.17	11.26
W3=1/2*(0.67*18)*1.5=0.90	3.45	3.10
W4=2*11.5*0.15=3.45	5.57	19.84

∑W=28.36 ∑M=206.6

Over Turning Moment

Taking Moment about Pt 0

OTM=5.2*10+6.5*20/3=95.33K-ft

- **Factor of Safety against Over Turning**

FOS=SM/OTM=206.6/95.33

= 2.17>2.0 **(OK)**

- **Check For Max Soil Pressure**

By Takmg Moment about Pt 0

Rv*x+95.33=206.60
X=3.92

e=L/2-x= 1. 83

q = Rv/L(I+-6e/L)

q(max) = 28.36/11.5(1 +6* 1.83/l1.5)=4.82Ksf <qa **(OK)**

q(Min) = 28.36/11.5(1-6*1.83/11.5)=O.11Ksf **(OK)**

- **FOS Against Sliding**

Sliding Force = (PI +P2) = 11.70K

Resisting Moment= μ*Rv=0.4*28.36=11.34K

FOS= Resisting Force/Sliding Force = 11.34/11.70 = 0.97 **(Not OK)**
Add base key

.

<u>Design of Base Key</u>

FOS= Force Resisting Sliding/Force Causing Sliding

Force Resisting Sliding= μW+Pp=0.4*28.36

Force Resisting Sliding=11.70*1.50=17.55K

By Above Two Eq

Pp=6.21K

Hp=5.29K

A=Hp-3= 2.29ft say2.5ft
Provide base key Ift*2.5ft

. **Design of Heel**

Loads

SelfWeight=2*1 *1 *0. 15=0.30k/ft

Soil weight= 18* 1 * 1 *0. 12=2. 16k/ft

Surcharge Load=8*1 *1 *0. 12=0.96K/ft

Total Wu= 5.08K/ft

Shear Force

Vu= Wu*L=5.08*6.83=34. 7K

d=24-3-l/2=20.5"

Φ*Vc =Φ*2 √f'c*b*d

*0.85*2*d *√3000*12*20.05/1* 000=22.90K (**Not OK**)
Now increase Base Thickness

*34.7=0.85*2*d3000** 12*d/l 000

d=31"

h=31 +3+ 1/2=36"

Design for Flexure

Mu=Wu*L^2/2=5.08*6.83*6.83/2=118.5K ft

P=200/fy=200/40000=0.005

118.5*12=0.9*p*12*32.5*32.5*40(1-0.59p*40/3)

p=0.00319<p (min)

Use p (min)

As(min) = 0.005*12*32.5=1.95cm^2

Use #9 @ 6"c/c

Development Length

ld= 24.64*1.3=32"

Temperature and Shrinkage Steel

Asl=0.002*12*36=0.80 Use Use #8 @ 10"c/c

As2=0.002*12*24=0.57 Use Use #8@ 14"c/c

.

Design of Toe

Loads

Selfweight = 2*0. 15=0.30K/ft

Soil Pressure at the Front Edge of the Stem
= 0.11+(4.82-11)/17.5 *8.5=3.59Ksf

Shear Force

Vu = 1.7(3*(4.82+3.59)/2)-3*0.3*1.4= 20.1 **0K**

Φ*Vc =0.85*2* √3000*20.5*12=22.90K **(OK)**

Design for Flexure

Mu=(3 .59*3 *7.5)+(4.83-3.59)/2*3 *2/3*3)*1. 7-
(0.3*3*3/2)*1.4)

=31.85Kft

p (min)=200/fy=0.005

31.85*12=0.90* p*12*40*20.5*20.5(1-0.59 p*40/3)

p=0.00214 Use p (min)

As=0.005*20.5*12=1.23in^2

Use #9 @ 9"c/c

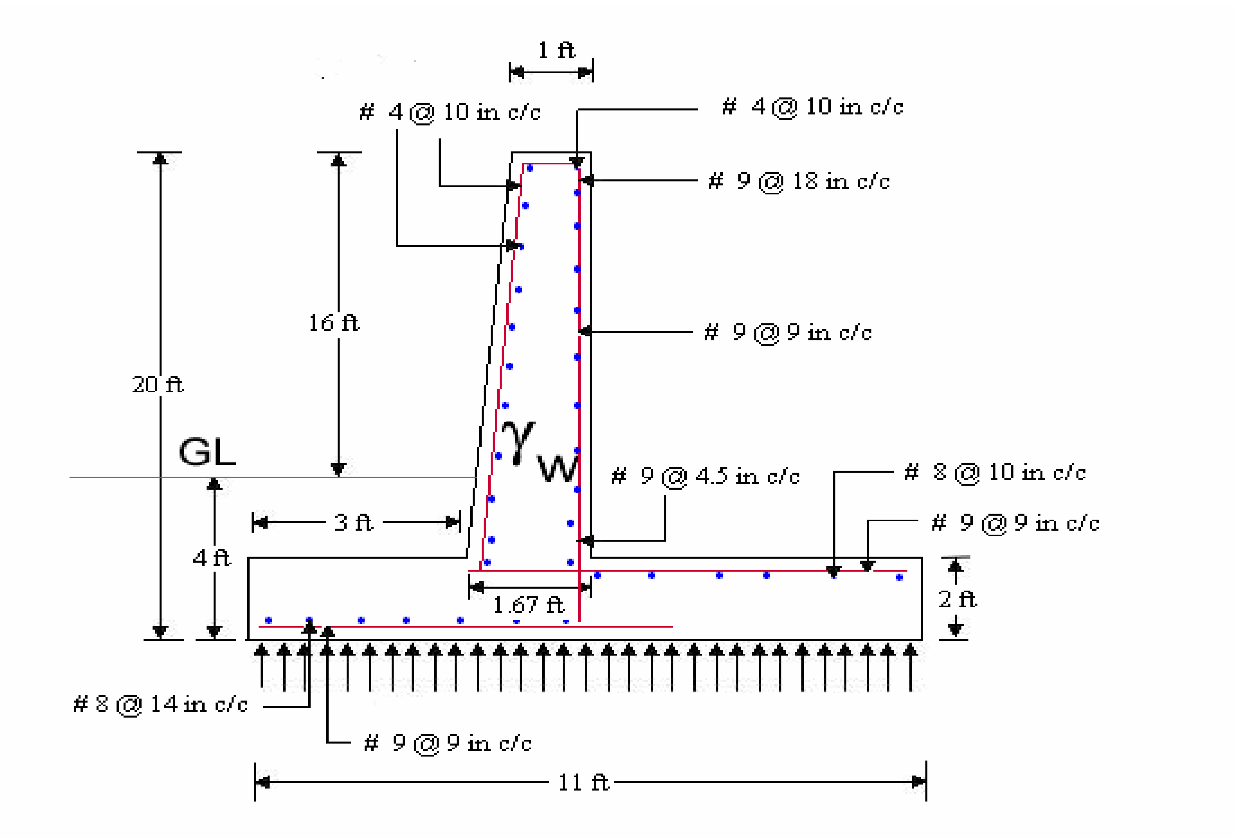
Development Length

Id=25"

Temperature and Shrinkage Steel

As1 =0.002*12*24.50=0.576in^2

Use #8 @ 14"c/c



Final Sketch

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