Palestine Polytechnic University

College of Engineering

Civil & Arch Engineering Department

Building Engineering

Graduation Project

Analysis and Design of Different Types of Earth Retaining Structures for Different Soil Properties and Load Conditions in Hebron

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Submitted to the College of Engineering and Technology in partial fulfillment of the requirements for the degree of Bachelor degree in civil Engineering

Hebron – Palestine

January, 2021

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Abstract

This project aims to analyze and design different types of earth retaining structures according to various soil conditions and cases of loading.

The analysis and design were for two types of earth retaining structure. The first one was retaining walls, and the second one was piles walls. The retaining walls were analyzed by the Rankine method and checked for stability. Then, the walls were designed according to ACI 318-19, by calculating the reinforcement area required for each wall element and choosing the appropriate numbers and diameters of the bars. As for piles walls, only one type has been studied in one case, which is the contiguous piles wall. It was analyzed by using equilibrium equations and then designed by using Sp column program after considering it as a circular column [1].

The final results were the conclusion of the effect of the loads on the reinforcement area and the analysis of the results of the design contiguous piles wall. In addition, the final results were summarized in an excel sheet to verify hand calculations.

Key words:

Retaining walls, Contiguous piles, Earth retaining structures, Rankine.

Project Supervisor	Department Chairman
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تحليل وتصميم أنواع مختلفة من المنشأت الساندة لتربة في تربة ذات خصائص مختلفة وقيم أحمال مختلفة في الخليل

عمل الطالبة: ديما خليل الكراجه

تحت أشراف: الدكتور محمد طه السيد أحمد

الملخص

الهدف من هذا المشروع هو تصميم الأنواع المختلفة من المنشأت الساندة لتربة بأنواعها المختلفة مثل الجدران الأستنادية و جدران الخوازيق حسب الأنواع المختلفة للتربة وحسب حالات التحميل المختلفة.

تم تصميم الجدران الأستنادية بعد تحليلها باستخدام Rankine's method و التأكد من إستقرار هذه الجدران . ثم تم التصميم باستخدام الكود الامريكي ACI 318- 19 بحساب مساحة الحديد المطلوبة لكل عنصر من الجدران وإختيار أعداد و أقطار القضبان المناسبة. أما جدران الخوازيق تم دراسة نوع واحد فقط في حالة واحدة وهو جدار الخوازيق المتجاورة. تحليلها تم بإستخدام معادلات الأتزان, ثم تصميمها باستخدام برنامج Sp column بعد إعتبارها أعمدة دائرية المقطع.

ثم تحليل نتائج كميات الحديد المطلوبة للجدران الأستنادية وإستنتاج العلاقة ما بينها و بين معاملات التربة وقيمة الأحمال الواقعة على الجدران . و تحليل نتائج جدار الخوازيق المتباعدة.

وكان من النتائج النهائية أيضا القيام بعمل ورقة (excel sheet) لتحقق من النتائج التصيم اليدوي.

رئيس الدائرة **المشرف**

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Dedication

I thank God almighty my creator, my source of strength, inspiration and wisdom.

I want to dedication this project to my family who is encouraging me all the way.

Acknowledgements

I want to express my great gratitude to my supervisor, Dr. Mohamed Taha Alsayyed Ahmad for his effort and his information to complete this project.

I am also grateful to the staff of The Civil and Arch Engineering Department at the Palestine Polytechnic University, who assisted me with this project and offered their time to answers my questions.

I am also especially grateful to Prof. Nabil Al-Joulani and Dr. Nasr Abboushi for their assistance to me.

Finally, I would like to thank everyone for contributing to achieving this project.

Table of Contents

Contents:

List of figures.

List of Tables

List of Abbreviations

γ: Unit weight of soil.

γ_c: Unit weight of concrete.

γ_w: Unit weight of water.

q: Uniform surcharge load.

H: Height of the walls.

B: Width of the base wall.

Pp: Passive force.

$\sum$Pi: Sum of Sliding forces.

qmax: Maximum bearing capacity.

qmin: Minimum bearing capacity.

e: Eccentricity of acting loads.

Ka: Active coefficient of lateral earth pressure.

Kp: Passive coefficient of lateral earth pressure.

D.L: Dead load.

L.L: Live load.

Φ: Internal angle of friction.

C: Concrete cover.

As req: The required area of flexural reinforcement.

As min: Minimum area of flexural reinforcement.

b: Width of compression face of members.

d: Distance from extreme compression fiber to centroid of longitudinal tension reinforcement.

fc': Specified compression strength of concrete.

fy: Specified yield strength for no prestressed reinforcement.

Ktr: Transverse reinforcement index.

L: span length.

La: Additional embedment length beyond centerline of support or point of inflection.

Ld: Development length of tension of deformed bar.

Lsc: Compression lap splice length.

Lst: Tension lap splice length.

Mn: Nominal flexural strength at section.

Mu: Factored moment at section.

Vn: Nominal shear strength.

Vu: Factored shear forces at section.

ʎ: Modification factor to reflect the reduced mechanical properties of lightweight concrete to relative to normal weight concrete of the same compression strength.

Φ: Strength reduction factor.

ψe: Factor used to modify development length based on reinforcement coating.

ψs: Factor used to modify development length based on reinforcement size.

ψt: Factor used to modify development length based for casting location in tension.

db: Dimeter of bars.

ds: Dimeter of spiral.

a: Distance between bars.

ρ: Ratio of reinforcement.

ρ_{max}: max ratio of steel.

Ag: Gross area.

Ac: Area of section without cover.

as: Area of spiral.

Sreq: Spacing between spiral.

Chapter 1 : Introduction

1.1- Introduction.

1.2- Project objective.

1.3- Project problem.

1.4- Project methodology.

1.5- Project scope.

1.6- Project time schedule.

1.1- Introduction:

Excavation is the first step in building any structure, whether above or below the surface of the earth. But the excavated process may cause problems in the soil, such as collapsing. So, to prevent a collapse of the soil, and hold back the soil, the earth retaining structures are used.

There are many types of earth retaining structures, such as retaining walls, piles walls, and sheet piles. Some of these are temporary, like sheet piles, and others are permanent, like retaining walls. Each of these types has many conditions to use, advantages, and disadvantages. Therefore, the engineer must select the best-suited type to the excavation area.

This project includes the analysis and design of two main types of earth retaining structures. The first type is retaining walls, and the second type is piles wall. The retaining walls that are included for analysis and design are Gravity walls, Cantilever walls, Counterforted walls, and Buttressed walls. Also, The Contiguous piles wall was analyzed and designed.

1.2- Project problem:

The effect of changing properties of the soil surrounding the retaining walls and loads values on the stability, and structural design of the investigated retaining walls. In addition, the contiguous piles wall stability study and design it.

1.3- Project objective:

The objective of this project is to analyze and design different types of earth retaining structures for various cases.

1.4- Project methodology:

This project passed through many steps, which are:

1. Calculation of loads and forces effect on earth retaining structures.

2. Start to design earth retaining structures after checking of stability.

3. Using excel to calculate the area of steel for all types of earth retaining structures.

4. Analysis of results.

5. Writing research in accordance with the requirements of construction engineering.

1.5- Project scope:

This project is composed of the following chapters:

- Chapter one: A general introduction to the project.
- Chapter two: Structural description of different types of earth retaining structures.
- Chapter three: Structural analysis and design.
- Chapter four: The results that have been reached and discussion of results.
- Chapter five: Conclusion and recommendations.

1.6- Project time schedule.

Table 1-1:Project time schedule.

Activates / Week	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16
Collect related data	■															
Design of Cantilever wall		■														
Design of Gravity wall			■													
Design of Counterfort wall				■	■	■										
Design of Buttressed wall					■	■	■									
Preparing Chapter 2								■								
Collect piles data									■	■						
Analysis contiguous pile											■					
Design contiguous piles wall												■	■			
Analysis results														■		
Preparing Chapter 3 and 5															■	
Writing the project															■	
The delivery of the project																■

Chapter 2 : Structure description.

2.1 Introduction.

2.2 The goal of the structural design.

2.3 Scientific tests.

2.4 Loads affecting the walls.

2.5 Structural elements of the retaining walls.

2.6 Contiguous piles wall description.

2.1 Introduction:

After the analysis of the retaining walls and the selection of the appropriate dimensions for each element in retaining walls in the introduction project. The next step is the design stage to find all reinforcement requirements for all elements necessary for the wall. Besides, analyze and design contiguous piles wall.

2.2 The goal of the structural design:

The purpose of structural design is to find a safe structure to resist all the forces that affect the structure, such as dead and live loads, or external forces, such as lateral earth pressure.

When designing any element of an earth retaining structure, there should be taken into consideration the following standards:

1. Safety is the essential element that must be satisfied in the design, so choosing the appropriate element of each region to resist loads that act on them.

2. Economy must be considered when working on the selection of appropriate materials, and sufficient for its desired purpose and appropriate quantity, with the lowest cost and highest quantity.

3. Serviceability, this term implies that the wall performs its required function. in this project, the main function of the earth retaining structures is to protect the soil from collapsing.

2.3 Scientific tests:

Before the start of the design of any structure, some tests must be conducted. For example, a test of the properties of the soil in front and under the retaining wall as well as behind the retaining wall. These properties are obtained according to the International Center for Geotechnical Engineering Studies (ICGES).

2.4 Loads affecting the walls:

Before the design, loads acting on the walls should be determined due to their significant influence on the design process. There are various types of loads and forces acting on wall, which are:

1. Lateral Earth Pressure:

   The pressure which acts in the horizontal direction of the soil. There are three types of lateral load applied on a retaining wall based on the behavior of the wall. [2]

   a) Active earth pressure (Pa). Force is applied by the soil on the wall when the wall is free to deflect. The soil is moving towards the wall.
   b) Passive earth pressure (Pp). Pressure earth pressure is developed when the wall is pressing the soil. Wall moving towards the soil.
   c) At rest pressure or at rest condition. This is a special case and the pressure varies between the active pressure and the passive pressure. The structure does not move.

      Figure (2.1) shows the earth pressure diagram.

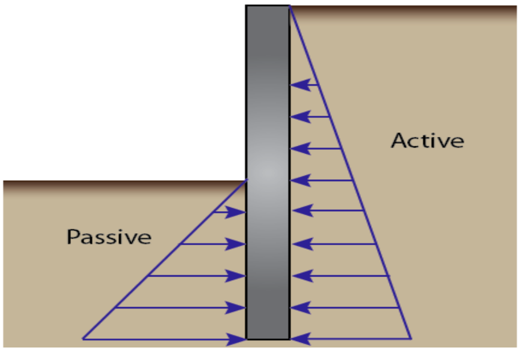

Figure 2-1:(Lateral earth pressure)

2. Surcharge Loads:

   This type of additional vertical loads could be a form of line load or uniformly distributed load. The correct estimation of the surcharge load is very important in the design, and an incorrect estimate could lead to wall failure. It is considered as a live load in the design stage, so we use the live load factor to calculate the factored load for it. [2]

3. Water Pressure:

   It is very important to take into account the forces acting on a wall due to the pore water pressure to design the wall. Incorrect estimation of the water pressure could lead to failures of the wall. Water pressure makes a significant change in the factor of safety. Therefore, much attention shall be made to consider the water pressure in the design and when possible to arrange adequate drainages. Figure (2.2) shows water pressure and surcharge pressure on wall. [2]

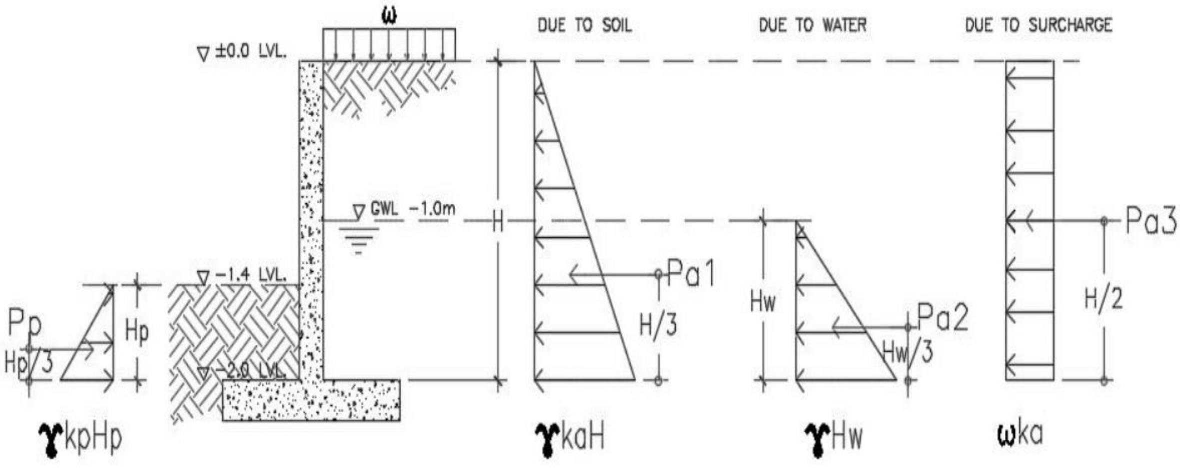

Figure 2-2: (Water pressure and surcharge pressure)

2.5 Structural elements of retaining walls:

There are many structural elements used in the retaining walls as the stem, base, toe, heel, key, counterfort, and buttress in some types of walls.

- Stem:

  The stem is the vertical member holding the backfill and resists earth pressure from the backfill side. It should be designed as a cantilever structure in cantilever walls and as a solid slab in counterfort and buttress walls. [3]

- Base slab:

  The base slab forms the foundation of the retaining wall. It consists of a heel slab and a toe slab. The heel slab in cantilever and buttressed walls acts as a horizontal cantilever under the weight of the retaining earth from the top, but in counterfort walls, it acts as a continuous solid slab. The toe slab also in counterfort and cantilever walls acts as a cantilever under the action of the soil pressure upward, but in buttressed walls, it acts as a continuous solid slab. [3]

- Key:

  Key projects down under the footing. It is used when the resistance provided does not give adequate safety against sliding, the depth of key should be provided to develop a passive pressure large enough to resist the excess force that causes sliding. Another function of the key is to provide sufficient development length for the dowels of the stem. The key can be set at any place in the base but the best place is under the heel because of the increase in friction force. The figure below shows the geometry of a typical retaining wall. [4]

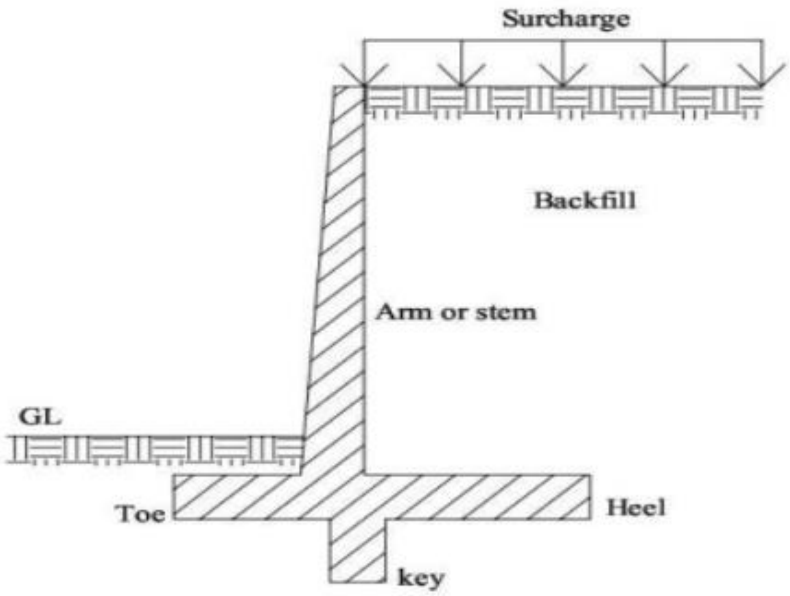

Figure 2-3: (Structure elements of retaining wall)

- Counterforts:

  Counterforts are vertical concrete webs at regular intervals along the backside of the wall. The counterforts tie the slab and base together, and the purpose of them is to reduce the shear forces and bending moments imposed on the wall by the soil. A secondary effect is to increase the weight of the wall from the added concrete. It acts as a tension member so should be designed as a T-section. [3]

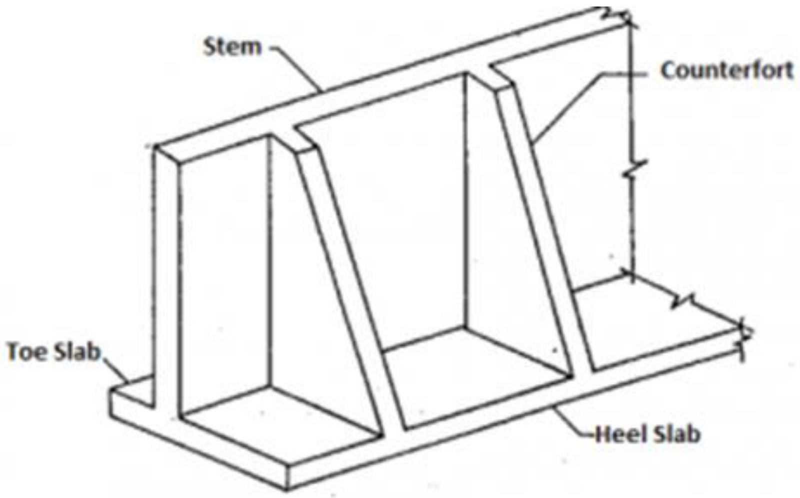

Figure 2-4:(Counterforted retaining wall)

- Buttresses:

  Buttresses are vertical concrete webs at regular intervals along the front side of the wall. The buttresses tie the slab and base together, buttresses act as compression members. It should be designed as a rectangular section. [4]

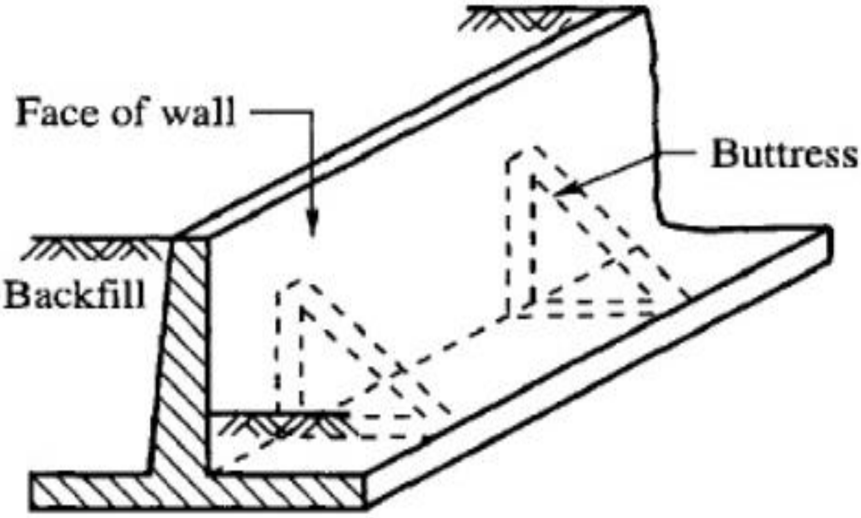

Figure 2-5:(Buttressed retaining wall)

2.6 Contiguous piles wall description:

The Contiguous piles wall is composed of a series of circular structural elements executed in the ground. It is used for high altitudes and can be used in granular soils, cohesive soils, and soft rock. The contiguous piles wall can only be used where groundwater is not a hazard. Contiguous piles wall consists of some of the piles arranged in a line with a small gap between adjacent piles. The diameter and spacing of the piles are decided based on soil type and magnitude of design pressure. Contiguous piles diameter ranges from 30 cm to 100 cm. The stability of the contiguous piles wall depends on penetration depth, which is calculated using equilibrium equations like sheet piles. Also, the wall of adjacent piles should be designed as a circular column under horizontal forces. Figure (2.6) shows the contiguous piles wall. [5]

Figure 2-6: (Contiguous piles wall)

Chapter 3 : Structural analysis and design.

3.1 Introduction.

3.2 Design requirements.

3.3 Factored load.

3.4 Design of Gravity walls.

3.5 Design of Cantilever walls.

3.6 Design of Counterfort walls.

3.7 Design of Buttressed walls.

3.8 Analysis and design of Contiguous piles wall.

3.9 Example of excel sheet.

3.1 Introduction:

The analysis of the retaining wall was done in the introduction project by using the Rankine method, but the walls that were not stable will be re-analyzed and then designed by ACI 318-19 code. Also, will analyze the contiguous piles wall by using the Rankine method and design it by using ACI 318-19 code.

3.2 Design requirements:

Were used is ACI 318-19 code for the calculation required structural element dimensions and reinforcement.

3.3 Factored Loads:

The structure may be exposed to different loads such as dead and live loads. The value of the load depends on the structure type and the intended use. The factored loads on which the structural analysis and design are based for the members of our projects are determined as [1]

$qu = 1.4DL$ ACI – 318 – 19 (5.3.1 a)

$qu = 1.2DL + 1.6L$ ACI – 318 – 19 (5.3.1 b)

3.4 Design of Gravity walls:

The structure of this type of walls does not include steel, it is just plain concrete, so we checked for the design strength of the concrete section such as the front part of base if it was adequate ($\Phi Mn > Mu$) it is not necessary to check any other sections.

Example:

Case1:

This wall was unstable, the factor of safety of sliding was less than 1.5 and the soil was subject to tension. To solve these problems, the base width was increased from 2.1m to 2.6m and designed key with depth =120 cm and width equal to the width of the stem after that re-analyzed then started to design. Figure (3.1) shows this.

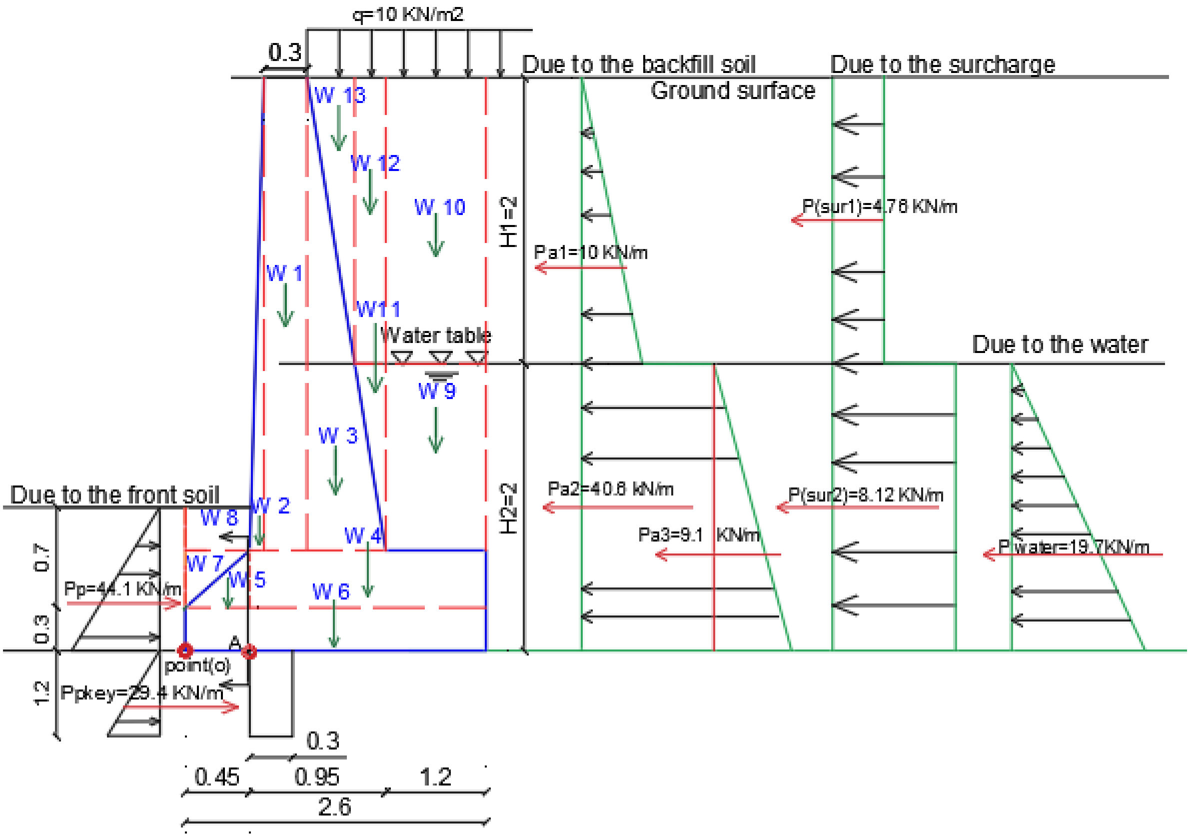

Figure 3-1:(Forces and lateral pressure of Gravity wall)

Analysis:

Coefficient of lateral earth pressure of key:

$Kp_{(key)}$ = (1+sin φ)/ (1-sin φ).

= (1+sin20)/ (1-sin 20).

= 2.04.

Passive force due to the key:

$Pp_{(key)} = 0.5*\gamma* Kp_{(key)}*H^2$.

$= 0.5*20*2.04*1.2^2 = 29.37$ KN/m.

- ∑ Passive pressure = Pp+ $Pp_{(key)}$= 44.1+29.37=73.47 KN/m.
- ∑ Vertical loads = ∑ W = 213.5 KN/m.

Check of sliding the wall:

Sliding forces: ∑(Pi).

Sliding forces = $Pa_1 + Pa_2 + Pa_3 + P_{surcharge1} + P_{surcharge2} + P_{water}$.

= 10+40.6+9.1+4.76+8.12+19.7 = 92.3 KN/m.

Resisting forces:

= (∑Vertical loads* tan (2* φ/3) + B*2/3* c+ Pp).

Φ and c to the soil under the wall.

Resist forces= (213.5*tan (2* 20/3) + 2.6*2/3* 13.3+ 73.47).

=147 KN/m.

The factor of safety for sliding ($F.s_{(sliding)}$) = (resisting forces)/ (sliding forces).

$F.s_{(sliding)}$ = 147/92.3

= 1.59 > 1.5 (Safe).

Check for bearing pressure under the wall:

$q_{(max)} = ((\sum V/B) *(1+(6*e/B)) \leq q_{(all)}$.

Calculating eccentricity of acting loads (e):

e = (B/2)- ((∑MR-∑Mo)/ ∑V).

= (2.6/2)- ((302-109)/213.5)

= 0.4m.

B/6 = 2.6/6 = 0.433>e.

$q_{@toe} = q_{max} = (\sum V/B*(1+(6*e/B))$.

= (213.5/2.6) *(1+(6*0.4/2.6).

=158 KN/m^2.

$q_{@heel} = q_{(min)} = (\sum V/B*(1-(6*e/B))$.

= (213.5/2.6) *(1-(6*0.4/2.6).

= 6.3 KN/m^2.

Design:

Solution:

Usually in the analysis and design of retaining walls consider a 1.0-meter length of the wall.

Data:

The surcharge equals 10 KN/m2.

Material

Fc': 20.00Mpa

The unit weight for wall parts equal unit weight of plain concrete:

γ_c =24 KN/m^3.

Unit weight of the backfill soil is above the water table is indicated by layer 1:

$\gamma_{s1:}$ =21 KN/m^3.

Unit weight of backfill soil below the water table is indicated by layer 2:

γ_{s2} =25 KN/m^3.

From stability analysis we found the following:

Actie coefficient of lateral earth pressure for layer 1:

Ka1 = 0.238.

Active coefficient of lateral earth pressure for layer 2:

Ka2=0.406

Kp =4.2.

$q_{@toe} = q_{max}$ = 158 KN/m^2.

$q_{@heel} = q_{min}$ = 6.3 KN/m^2.

Check the design strength of concrete section at point A of the base.

a. Soil pressure at A is:

The slope = $(q_{max} – q_{min})$/B.

= (158-6.3)/2.6.

= 58.3.

$q_{(A)} = q_{max}$ – slope* width of toe.

=158-58.3*0.45.

=131.76 KN/m^2.

b. M_A = 1.6*((158-131.76) *0.5*0.45*(2/3) *0.45+131.76*0.5*0.45^2) – 0.9*(24*0.3*0.45*0.5*0.45+24*0.4*0.45*0.45*2/3).

M_A = 22.35 KN.m.

c. The design moment strength of plain concrete is.

$\Phi M_n = \Phi*0.42*\sqrt{fc'}*b*h^2/6$. ACI – 318 - 19- Eq (14.5.2.1 a)

= $0.55*0.42*\sqrt{20}*1000*0.7^2/6$.

= 84.36 KN.m.

- ❖ $\Phi M_n \geq M_A$. ACI – 318 - 19- Eq (7.5.1.1 a)

84.36 ≥ 22.35.

The section is adequate. No other section to be checked.

3.5 Design of Cantilever walls.

Example:

Case 1:

This wall was unstable, the factor of safety of sliding was less than 1.5 and the soil was subject to tension. To solve these problems, the base width was increased from 2.1m to 2.6m and designed key with depth=120cm and width equal to the width of the stem after that re-analyzed then started to design. Figure (3.2) shows this.

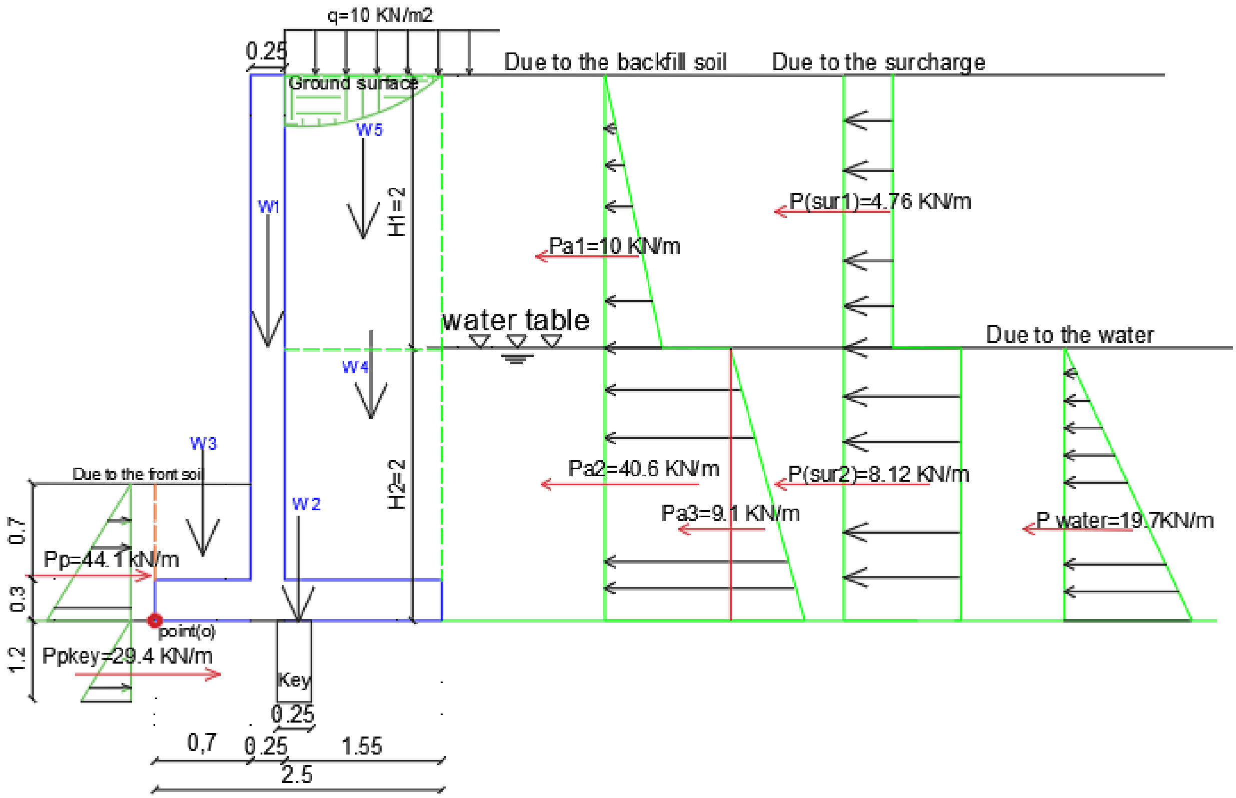

Figure 3-2: (Forces and lateral pressure of Cantilever wall)

Analysis:

Coefficient of lateral earth pressure of key:

$Kp_{(key)} = (1+\sin\varphi)/(1-\sin\varphi)$.

$= (1+\sin 20)/(1-\sin 20)$.

$=2.04$.

Passive force due to the key:

$Pp_{(key)} = 0.5*\gamma*Kp_{(key)}*H^2$.

$= 0.5*20*2.04*1.2^2 = 29.37$ KN/m.

- ❖ $\sum$ Passive pressure $=Pp+Pp_{(key)}= 44.1+29.37=73.47$ KN/m.
- ❖ $\sum$ Vertical loads $= \sum W = 190.65$ KN/m.

Check of sliding the wall:

Sliding forces: ∑(Pi).

Sliding forces= $Pa_1 + Pa_2 + Pa_3 + P_{surcharge1} + P_{surcharge2} + P_{water}$.

= 10+40.6+9.1+4.76+8.12+19.7 = 92.3 KN/m.

Resisting forces:

= (∑vertical loads* tan (2* φ/3) + B*2/3* c+ Pp).

Φ and c to the soil under the wall.

Resist forces= (190.65*tan (2* 20/3) + 2.5*2/3* 13.3+ 73.47).

=140.82 KN/m.

The factor of safety for sliding ($F.s_{(sliding)}$) = (resisting forces)/ (sliding forces).

$F.s_{(sliding)}$ = 140.82/92.3

= 1.53 > 1.5 (Safe).

Check for bearing pressure under the wall (check stress on soil):

$q_{(max)} = ((\sum V/B) *(1+(6*e/B)) \leq q_{(all)}$.

Calculating eccentricity of acting loads (e):

e=(B/2)- ((∑MR-∑Mo)/ ∑V).

= (2.5/2)- ((278.24-109.7)/190.65)

= 0.36 m.

B/6 = 2.5/6 = 0.417>e.

$q_{@toe} = q_{max} = (\sum V/B*(1+(6*e/B))$.

= (190.65/2.5) *(1+(6*0.36/2.5).

= 142.15 KN/m^2.

$q_{@heel} = q_{(min)} = (\sum V/B*(1-(6*e/B))$.

= (190.65/2.5) *(1-(6*0.36/2.5).

= 10.37 KN/m^2.

Design:

Solution:

Usually in the analysis and design of retaining walls consider a 1.0-meter length of the wall.

Data:

The surcharge equals 10 KN/m2.

Material

Fc': 24.00Mpa.

Fy: 420MPa

The unit weight for wall parts equal unit weight of reinforced concrete:

γ_c =25 KN/m^3.

Unit weight of the backfill soil is above the water table is indicated by layer 1:

$\gamma_{s1:}$ =21 KN/m^3.

Unit weight of backfill soil below the water table is indicated by layer 2:

γ_{s2} =25 KN/m^3.

γ_w =9.81 KN/m^3.

Maximum diameter of bars Φ20.

Cover = 7.5cm.

From stability analysis we found the following:

Active coefficient of lateral earth pressure for layer 1:

Ka1 = 0.238.

Active coefficient of lateral earth pressure for layer 2:

Ka2=0.406

Kp =4.2.

$q_{@toe} = q_{max}$ = 142.15/m^2.

$q_{@heel} = q_{(min)}$ = 10.37 KN/m^2.

Design of stem:

a. Design for shear:

$Pa_1 = 0.5 * \gamma_1 * ka1 * H_1^{(2)}$.

$=0.5*21*0.238*2^2$= 10 KN/m.

$Pa_2 = 0.5 * \gamma_2 * ka2 * H_2^{(2)}$.

$=0.5*25*0.406*1.7^2$= 29.33 KN/m.

$Pa_3 = 0.5*(\sigma_{a3} - \sigma_{a2}) * H_2$

=0.5*1.7*(21*2*0.406+(25-9.81) *1.7*0.406 – 25*1.7*0.406) = 8.74 KN/m.

$Pa_{(s1)} = q*k_a1*H_1$.

=10*0.238*2=4.76 KN/m.

$Pa_{(s2)} = q*k_a2*H_2$.

=10*0.406*1.7=6.902 KN/m.

$Pa_{(w)} = 0.5 * \gamma_w * H_2^2$.

$= 0.5*9.81*1.7^2 = 14.175 KN/m$.

$Pa_{(total)}$ = 10+29.33+8.74+4.76+6.902+14.175.

= 74 KN/m.

V_u at the face of the stem is:

$V_u = 1.6 * Pa_{(total)}$.

=1.6*74=118.4KN.

d = depth - cover – (diameter of bar/ 2).

d = 250-75-(20/2) = 165mm.

$\Phi V_c = \Phi * \sqrt{fc'} * b * d/6$. ACI – 318 - 19- Eq (9.6.3.1)

$= 0.75 * \sqrt{24} * 1 * 165/6$.

= 101.04. KN.

$0.5 * \Phi V_c$ =50.52 KN.

V_u = 118.4KN > ΦV_c = 101.04. KN. ACI – 318 - 19- Eq (7.5.1.1.b)

The thickness of 25 cm at the stem end is inadequate.

- To find the critical adequate depth we use the following equation:

$\Phi V_c = V_u$.

$0.75 * \sqrt{24} * 1 * d/6 = 118.4$ KN.

d= 193.35 mm.

Depth = d + cover + Φ/2.

= 193.35 + 75 + 20/2.

= 27.8 cm $\Longrightarrow$ Select depth = 30 cm.

a. Redesign for shear:

$Pa_{(total)}$ = 10+29.33+8.74+4.76+6.902+14.175.

= 74 KN/m.

V_u at the face of the stem is:

$V_u = 1.6 * Pa_{(total)}$.

=1.6*74=118.4KN.

d = depth - cover – (diameter of bar/ 2).

d = 300-75-(20/2) = 215mm.

$\Phi V_c = \Phi * \sqrt{fc'} * b * d/6.$

$= 0.75 * \sqrt{24} * 1 * 215/6.$

= 131.7. KN.

$\Phi V_c \geq Vu.$ ACI – 318 - 19- Eq (7.5.1.1.b)

118.4KN ≤131.7 KN.

❖ The thickness of 30 cm at the stem end is adequate enough.

b. Design for flexure:

M_u =1.6*(10*2.37+29.33*0.5*1.7+8.74*1.7/3+4.76*(2/2+1.7) +6.902*0.5*1.7+14.175*1.7/3).

=128.53 KN.m.

Take ϕ =0.9 for flexure. ACI – 318 - 19- Table (21.2.1)

$Rn = \frac{M_u}{\emptyset bd^2} = \frac{128.53\times10^6}{0.9\times1000\times215^2} = 3.09$ Mpa

$m = \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*24} = 20.6$

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$

$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*3.09*20.6}{420}}\right) = 0.00802$

$\rightarrow A_{s(req)} = \rho * b * d = 0.00802 * 1000 * 215 = 1724.3 mm^2/m.$

$As_{min} = \frac{\sqrt{f_c'}}{4\ (f_y)} * b_w * d \ \geq \ \frac{1.4}{f_y} * b_w * d$ ACI – 318 - 19 - Eq (9.6.1.2 a, b)

$= \frac{\sqrt{24}}{4*420} * 1000 * 215 \ \geq \ \frac{1.4}{420} * 1000 * 215.$

$= 627\ mm^2/m < 716.67\ mm^2/m$ (Larger value is control).

$As_{min} = 716.67\ mm^2/m < A_{s(req)} = 1724.3\ mm^2/m.$

Try ϕ16/100mm with $A_s = 2009.6\ mm^2/m > A_{s(req)} = 1724.3mm^2/m.$

Temperature and shrinkage reinforcement:

As_{min} horizontal = 0.002*b*h. ACI – 318 – 19- Table (7.6.1.1)

$= 0.002*1000*300 = 600\ mm^2/m.$

Use one-half of the horizontal bars at the external face of the wall.

0.5* As_{min} horizontal=300 mm^2/m.

Use ϕ12/300mm with A_s = 376.8 mm^2/m ≥ 300mm^2/m. At both surface of the wall.

Use ϕ12/300mm vertical bars at the front face of the wall support the horizontal temperature and shrinkage reinforcement.

Design of toe:

The download pressure due to self-weight of the toe slab = γ_c*h=25*0.3=7.5 KN/m^2.

The slope = (q_{max}-q_{min})/B

= (142.15-10.37)/2.5.

= 52.712.

The pressure at the face of toe:

q_A=142.15-0.7*52.712=105.25 KN/m^2.

d= 300-75-20/2

= 215 mm.

a. Design for shear:

The pressure at distance d from the face:

q_{ud} = 142.15- (0.7-0.215) *52.712.

= 116.58 KN/m^2.

V_{ud} = 1.6*((142.15+116.58) *0.5*0.485) – (0.9*7.5*0.485).

= 97.11 KN.

ΦV_c= $\Phi*\sqrt{fc'}$*b*d/6.

=0.75*$\sqrt{24}$*1*215/6 =131.66 KN.

0.5* ΦV_c=65.83 KN.

0.5* ΦV_c< V_{ud} ≤ ΦV_c.

65.83 KN < 93.45 KN ≤ 131.66 KN.

❖ The thickness of 30 cm of the toe slab is adequate enough.

b. Design for flexure:

M_u = 1.6*((142.15-105.25) *0.5*0.7^2*2/3+105.25*0.7^2*0.5) – 0.9*7.5*0.7^2*0.5.

= 49.25 KN.m.

Take ϕ =0.9 for flexure.

Rn $= \frac{M_u}{\emptyset b d^2} = \frac{49.25\times10^6}{0.9\times1000\times215^2} = 1.18$ Mpa

m = $\frac{f_y}{0.85\ f'_c} = \frac{420}{0.85*24}$ =20.6

ρ = $\frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$

= $\frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*1.18*20.6}{420}}\right)$ = 0.0029

→$A_{s(req)}$= ρ * b *d = 0.0029* 1000*215 = 623.5 mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f'_c}}{4\ (f_y)} * b_w * d \ \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 215 \ \geq \frac{1.4}{420} * 1000 * 215.$$

=627 mm^2/m < 716.67 mm^2/m (Larger value is control).

As_{min} = 716.67 mm^2 /m > $A_{s(req)}$ = 623.5mm^2 /m.

$A_{s(req)}$ = As_{min} = 716.67 mm^2 /m.

Try ϕ14/200mm with A_s = 769.3mm^2/m > $A_{s(req)}$ = 716.67 mm^2 /m.

Minimum shrinkage A_{sh} =0.0018*b*h. ACI – 318 – 19-Table (7.6.1.1)

=540 mm^2 /m.

Temperature and shrinkage reinforcement in the longitudinal direction is ϕ12/200mm.

Design of heel:

a. Design for shear:

The downward pressure due to self-weight of the heel slab and soil backfill is:

W_u = 1.2*(21*2+25*1.7+0.3*25) +1.6*10.

= 126.4 KN/m^2.

V_u = 126.4*1*1.5 = 189.6 KN.

d = 300-75-20/2.

=215mm.

ΦV_c= Φ*$\sqrt{fc'}$*b*d/6.

=0.75*√24*1*215/6 =131.66 KN.

$0.5*\Phi V_c$=65.83 KN.

V_u =189.6 KN > ΦV_c =131.66 KN.

The thickness of 30 cm of the heel slab is inadequate must be increased.

- To find the critical adequate depth we use the following equation:

$\Phi V_c = V_u$.

$0.75*\sqrt{24}*1*d/6$ =189.6 KN.

d= 309.6 mm.

Depth = d + cover + Φ/2.

= 309.6 + 75 + 20/2.

= 40.5 cm $\Longrightarrow$ Select depth = 45cm.

Since the wall was stable, it is not necessary to re-analyze the wall and redesign the stem.

- **Redesign of toe:**

The download pressure due to self-weight of the toe slab = γ_c*h=25*0.45=11.25 KN/m^2.

The slope = $(q_{max}-q_{min})/B$

= (142.15-10.37)/2.5.

= 52.712.

The pressure at the face of toe:

q_A=142.15-0.7*52.712 = 105.25 KN/m^2.

d= 450-75-20/2

= 365 mm.

a. Design for shear:

The pressure at distance d from the face:

q_{ud} = 142.15- (0.7-0.365) *52.712.

= 124.5KN/m^2.

V_{ud} = 1.6*((142.15+124.5) *0.5*0.335) – (0.9*11.25*0.335).

=68.1 KN.

$\Phi V_c = \Phi*\sqrt{fc'}*b*d/6$.

$=0.75*\sqrt{24}*1*365/6$ =223.52KN.

$0.5*\Phi V_c = 111.76$ KN.

$V_{ud} \leq 0.5*\Phi V_c$.

68.1 KN ≤ 111.76 KN.

❖ The thickness of 45 cm of the toe slab is adequate enough.

b. Design for flexure:

$M_u = 1.6*((142.15 - 105.25)*0.5*0.7^2*2/3+105.25*0.7^2*0.5) - 0.9*11.25*0.7^2*0.5.$

$=48.42$ KN.m.

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{48.42\times10^6}{0.9\times1000\times365^2} = 0.404 \text{ Mpa}$$

$$m = \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85*24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1-\frac{2*R_n*m}{f_y}}\right)$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1-\frac{2*0.404*20.6}{420}}\right) = 0.001$$

$\rightarrow A_{s(req)} = \rho * b * d = 0.001*1000*365 = 365 \text{mm}^2/\text{m}$.

$$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \ \geq \ \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 365 \ \geq \ \frac{1.4}{420} * 1000 * 365.$$

$=1064.4 \text{ mm}^2/\text{m} < 1216.7 \text{ mm}^2/\text{m}$ (Larger value is control).

$As_{min} = 1216.7 \text{ mm}^2/\text{m} > A_{s(req)} = 365 \text{ mm}^2/\text{m}$.

$A_{s(req)} = As_{min} = 1216.7 \text{ mm}^2/\text{m}$.

Try ϕ14/125 mm with $A_s = 1230.9 \text{mm}^2/\text{m} > A_{s(req)} = 1216.7 \text{ mm}^2/\text{m}$.

Minimum shrinkage $A_{sh} = 0.0018*b*h$.

$=810 \text{ mm}^2/\text{m}$

Temperature and shrinkage reinforcement in the longitudinal direction is ϕ14/175 mm.

- **Redesign of heel:**

a. Design for shear:

The downward pressure due to self-weight of the heel slab and soil backfill is.

W_u = 1.2*(21*2+25*1.55+0.45*25) +1.6*10.

= 126.4 KN/m^2.

V_u = 126.4 *1*1.5 = 190 KN

d = 450-75-20/2.

=365mm.

$\Phi V_c = \Phi * \sqrt{fc'} * b * d/6$.

$= 0.75 * \sqrt{24} * 1 * 365/6 = 223.5$KN.

0.5* ΦV_c=111.75 KN.

0.5* $\Phi V_c < V_u \le \Phi V_c$.

111.75 KN < 190 KN ≤ 223.5KN.

❖ The thickness of 45 cm of the heel slab is adequate enough.

b. Design for flexure:

M_u = 126.4 *1.5^2*0.5.

=142.2 KN.m.

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{142.2 \times 10^6}{0.9 \times 1000 \times 365^2} = 1.19 \text{ Mpa}$$

$$m = \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85*24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}}\right)$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*1.19*20.6}{420}}\right) = 0.0029$$

→$A_{s(req)} = \rho * b * d = 0.0029 * 1000 * 365 = 1062.5 \text{ mm}^2/\text{m}$.

$$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \;\ge\; \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 365 \;\ge\; \frac{1.4}{420} * 1000 * 365.$$

=1064.4 mm^2/m < 1216.7mm^2/m (Larger value is control).

As_{min} = 1216.7 mm^2 /m > $A_{s(req)}$ = 1102.3mm^2 /m.

$A_{s(req)}$ = As_{min} = 1216.7 mm^2 /m.

Try ϕ14/125mm with A_s = 1230.9mm^2/m > $A_{s(req)}$ = 1216.7 mm^2 /m.

Temperature and shrinkage reinforcement A_{sh} = 0.0018*b*h.

= 0.0018*1000*450=810 mm^2 /m.

In the longitudinal direction use ϕ14/175mm.

Ass: additional reinforcement in edge between stem and heel.

a) Check if needed it or not:

$$\rho_{max} = \text{max of} \begin{cases} \dfrac{\text{As of stem}}{\text{Area of stem section}} \\ \\ \dfrac{\text{As of heel}}{\text{Area of heel section}} \end{cases}$$

$$= \text{max of} \begin{cases} \dfrac{1724.3}{1000*300} \\ \\ \dfrac{1062.5}{1000*450} \end{cases} = \begin{cases} 0.0057 \\ \\ 0.0024 \end{cases}$$

ρ_{max} = 0.0057 > 0.004 ⟹ Needed Ass.

$$\text{Ass} = \text{max of} \begin{cases} 0.5 * \text{As stem} \\ 0.5 * \text{As heel} \end{cases} = \begin{cases} 0.5 * 1724.3 \\ 0.5 * 1062.5 \end{cases}$$

$$\text{Ass} = \text{max of} \begin{cases} 862.15\ mm^2 \\ 351.3\ mm^2 \end{cases}$$

Ass = 862.15 mm^2 ⟹ Select 8ϕ12.

$$Ld = \frac{fy}{2*\sqrt{fc'}} *db.$$

$$= \frac{420}{2*\sqrt{24}} *12 = 515 \text{ mm.}$$

Design of key:

It will be extended the main reinforcement from the stem to the face of the key, so it is just needed horizontal reinforcement equals 2ϕ14 and vertical reinforcement in the other face of the key equals ϕ14/300mm. [4]

Figure (3.3) shows all reinforcement details in each element in Cantilever retaining wall.

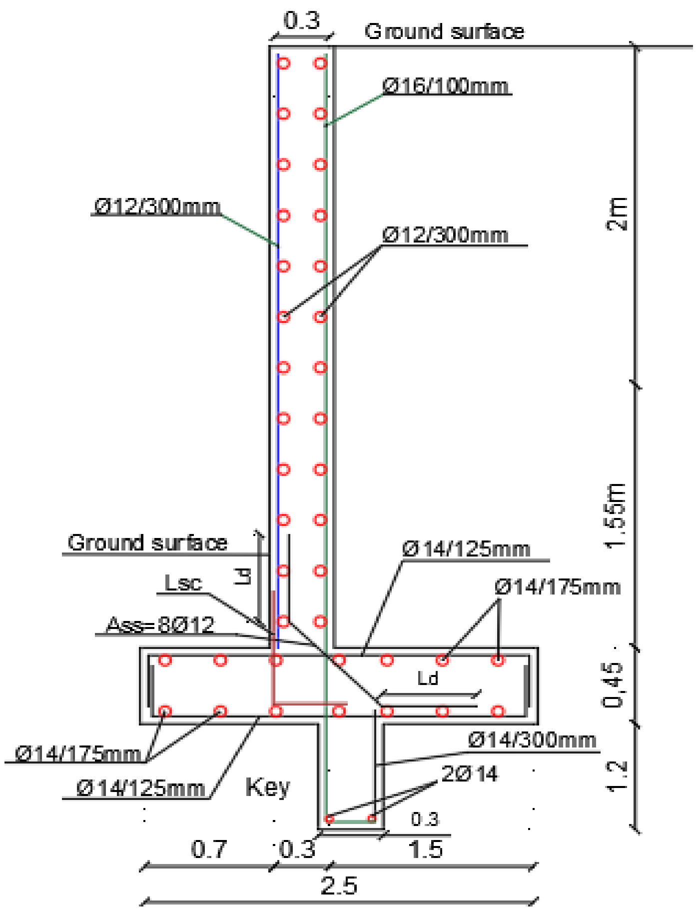

Figure 3-3: (Reinforcement details of Cantilever wall)

3.6 Design of Counterfort walls.

Examples:

Case 1 and Case 2:

These walls were unstable, the factor of safety of sliding was less than 1.5 and the soil was subject to tension. To solve these problems the base width was increased from 5.0-m to 5.5-m and designed a key. But after designing the key the depth was 4-m, which is illogical, so to solve this problem, there should be a design of piled foundation under the wall or the soil could be replaced with better soil or the soil under the wall could be improved by mixing the soil with asphalt, cement, or lime.

Case 3:

Solution:

Usually in the analysis and design of retaining walls consider a 1.0-meter length of the wall.

Data:

The surcharge equals 10 KN/m2.

Material

Fc': 24.00Mpa.

Fy: 420MPa

The unit weight for wall parts equal unit weight of reinforced concrete:

γ_c =25 KN/m^3.

γ_s =24 KN/m^3.

Maximum diameter of bars Φ20.

Cover = 7.5cm.

Distance between center to center of counterfort =3m.

Thickness of counterfort =0.3m.

From stability analysis we found the following:

Ka = 0.283.

Kp =4.2.

$q_{@toe} = q_{max}$ = 820.1 KN/m2/$(3m)^2$. = 273.4 KN/m^2

$q_{@heel} = q_{(min)}$ = 156.1 KN/m2/$(3m)^2$= 52.1 KN/m^2

Design of stem:

Pressure at base of stem:

$\sigma_a = \gamma_1$ * H *Ka.

= 24*8.5*0.283

= 57.7 $KN/(m)^2$.

$\sigma_{(surcharge)}$ =q*ka.

=10*0.283= 2.83 $KN/(m)^2$.

$\sigma_{(total)}$= 57.7 +2.83= 60.5 KN/m^2.

Forces at base of stem:

Pa1 = 0.5*σa1*H.

=0.5*57.7*8.5=245.2 KN/m.

P(surcharge) = $\sigma_{(surcharge)}$*H.

=2.83*8.5=24.1 KN/m.

$P_{(total)}$ = 269.3 KN/m.

- Moment at support (Negative moment):

  Mu =1.6*($W*L^2$)/12. ACI – 318 – 19-Table (6.5.2)

  =1.6* ($60.5*2.7^2$)/12

  = 58.8 KN.m

- Moment at middle span (positive moment):

  Mu =1.6*($W*L^2$)/16. ACI – 318 – 19-Table (6.5.2)

  = 1.6*($60.5*2.7^2$)/16.

  = 44.1 KN.m

- d= depth - cover – (diameter of bar/ 2).

  d = 500-75-(20/2) = 415mm.

a. Design for shear:

Maximum shear at the face of counterfort (face of support):

Vu = 1.6*(W*L)/2. ACI – 318 – 19-Table (6.5.4)

=1.6*(60.5*2.7)/2

=130.7 KN.

$\Phi V_c = \Phi * \sqrt{fc'} * b * d/6.$

$= 0.75 * \sqrt{24} * 1 * 415/6.$

= 254.13. KN.

ΦV_c= 254.13. KN > Vu=130.7 KN.

❖ The thickness of 50 cm at the stem end is adequate enough.

b. Design for flexure:

At support:

Take ϕ =0.9 for flexure.

$Rn = \frac{M_u}{\varnothing bd^2} = \frac{58.8\times10^6}{0.9\times1000\times415^2} = 0.38$ Mpa

$m = \frac{f_y}{0.85\ f'_c} = \frac{420}{0.85*24} = 20.6$

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$

$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.38*20.6}{420}}\right) = 0.001$

$\rightarrow A_{s(req)} = \rho * b * d = 0.001 * 1000 * 415 = 415\ mm^2/m.$

$$As_{min} = \frac{\sqrt{f'_c}}{4\,(f_y)} * b_w * d \;\; \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 415 \;\; \geq \frac{1.4}{420} * 1000 * 415.$$

= 1210.16 mm^2/m < 1383.3 mm^2/m (Larger value is control).

As_{min} = 1383.3 mm^2 /m >$A_{s(req)}$ = 415 mm^2 /m.

Or

As_{min} = 4* $A_{s(req)}$/3.

=553.3 mm^2 /m.

$A_{s(req)}$ = As_{min} =553.3 mm^2 /m.

Try ϕ12/200 mm with A_s = 565.2 mm^2/m > $A_{s(req)}$ = 553.3 mm^2 /m.

Temperature and shrinkage reinforcement:

As_{min} horizontal = 0.002*b*h.

= 0.0018*1000*500= 900 mm^2 /m.

Use ϕ12/100mm with A_s = 1130.4 mm^2 /m ≥ 900 mm^2 /m.

At middle span:

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{44.1\times10^6}{0.9\times1000\times415^2} = 0.28Mpa$$

$$= \frac{f_y}{0.85\,f'_c} = \frac{420}{0.85*24} = 20.6m$$

$$= \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})\rho$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.28*20.6}{420}}\right) = 0.00067$$

→$A_{s(req)}$= ρ * b *d = 0.00067* 1000*415 = 278.1 mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f'_c}}{4\,(f_y)} * b_w * d \;\; \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 415 \;\; \geq \frac{1.4}{420} * 1000 * 415.$$

= 1210.16 mm^2/m < 1383.3 mm^2/m (Larger value is control).

As_{min} = 1383.3 mm^2 /m >$A_{s(req)}$ = 278.1 mm^2 /m.

Or

As_{min} = 4* $A_{s(req)}$/3.

=371 mm^2 /m

$A_{s(req)}$ =As_{min} =371 mm^2 /m.

Try ϕ12/250 mm with A_s = 452.2 mm^2/m > $A_{s(req)}$ = 371 mm^2 /m.

Temperature and shrinkage reinforcement:

As_{min} horizontal = 0.0018*b*h.

= 0.0018*1000*500= 900 mm^2 /m.

Use ϕ12/100mm with A_s = 1130.4 mm^2 /m ≥ 900 mm^2 /m.

c. Development length.

Design of tension development length (Ldt)

$Ldt = \frac{9*Fy*\psi t*\psi e*\psi s}{10*\sqrt{Fc'}*\lambda*(cb+ktr)/db} * db \geq 300mm.$ ACI – 318 – 19- Eq (25.4.2.3.9)

According ACI:

$\psi t = 1, \ \psi e = 1, \ \psi s = 0.8 \quad \lambda = 1.$ ACI – 318 – 19-Table (25.4.2.5)

$ktr = 0.$ ACI – 318 – 19-Section (25.4.2.4)

cb is smallest of:

1) Cover + db/2 ACI – 318 – 19-Section (25.4.2.4)

=75+12/2 =81mm.

2) 0.5* Distance between bars = 0.5*a. ACI – 318 – 19-Section (25.4.2.4)

a =250*0.5 = 125 mm.

cb = 81mm.

(cb+Ktr)/db ≤ 2.5mm.

= (81+0)/12 =6.75 > 2.5 $\longrightarrow$ select = 2.5mm.

$Ldt_{(req)} = \frac{9*420*1*1*0.8*12}{10*\sqrt{24}*1*2.5}$ =296.3 mm < 300 mm.

Select $Ldt_{(req)}$ = 300 mm.

$Ldt_{(avl)} = 2*La + \frac{Mn}{Vu}$. ACI – 318 – 19- Eq (7.7.3.8.3.b)

La = the larger of (12* db or d). ACI – 318 - 19- Section (7.7.3.3)

12*db = 12*12 = 144mm.

d = d of stem =415mm.

- La = 415mm.
- Vu = factored shear at the support = 1.6*W*L/2.

=1.6*60.5*2.7/2 =130.7 KN.

- Mn = As * Fy *(d-a/2).

As = 4*3.14*12^2/4 = 452.2 mm^2.

a = $\frac{As*Fy}{0.85*Fc'*b}$ = 9.3 mm.

Mn = 452.2*420*(415-9.3/2)

=78 KN.m

$Ldt_{(avl)}$ = 2*415+(78/130.7) = 831 mm.

$Ldt_{(avl)} > Ldt_{(req)}$.

d. Connection of stem with counterfort.

Horizontal stirrups resist tension forces.

Horizontal pressure at any depth Y from top:

$\sigma_a = \gamma_1$ * H *Ka.

= 24*Y*0.283

=6.8*Y KN/$(m)^2$.

$\sigma_{(surcharge)}$ =q*ka.

=10*0.238= 2.38 kN/$(m)^2$.

$\sigma_{(total)}$= 6.8*Y+2.38. KN/m^2.

Forces at base of stem:

Pa1 = 0.5*σa1*H.

=0.5*6.8*Y^2=3.4*Y^2 KN/m.

P(surcharge) = $\sigma_{(surcharge)}$*H.

=2.38*Y KN/m.

Total shear force at Y = (6.8*Y+2.38) *2.7 KN/m.

Factored shear force = 1.6*(6.8*Y+2.38) *2.7

As = shear force /(0.9*Fy)

At Y= 8.2 m (at base of stem)

As = 664.5 mm^2.

Select 5ϕ10-2 Legs with As = 785 mm^2.

At Y=5.2 m (3m from base)

As = shear force /(0.9*Fy)

= 431.3 mm^2.

Select 3ϕ10-2 Legs with As = 471 mm^2.

Design of toe:

The download pressure due to self-weight of the toe slab = γ_c*h=25*0.8 =12.5KN/m^2.

The slope = $(q_{max}-q_{min})/B$

= (273.4-52.1)/5.

=44.3.

The pressure at the face of toe:

q_A=273.4-1.5*44.3=207 KN/m^2.

d= 500-75-20/2

= 415 mm.

a. Design for shear:

The pressure at distance d from the face:

q_{ud} = 273.4- (1.5-0.415) *44.3.

=225.3 KN/m^2.

V_{ud} = 1.6*((273.4+225.3) *0.5*1.085) – (0.9*12.5*1.085).

= 420.7 KN.

$\Phi V_c = \Phi * \sqrt{fc'} * b * d/6$.

$=0.75*\sqrt{24}*1*415/6$ =254.13 KN.

$V_{ud} > \Phi V_c$.

❖ The thickness of 50 cm of the toe slab is inadequate.

- To find the critical adequate depth we use the following equation:

$\Phi V_c = V_{ud}$.

$0.75*\sqrt{24}*1*d/6$ = 420.7 KN.

d= 687 mm.

Depth = d + cover + Φ/2.

= 687+ 75 + 20/2.

= 77.2 cm ⟶ Select depth = 80 cm.

Since the wall was stable, it is not necessary to re-analyze the wall and redesign the stem.

The download pressure due to self-weight of the toe slab = γ_c*h=25*0.8 =20KN/m^2.

d= 800-75-20/2

= 715 mm.

a. Redesign for shear:

The pressure at distance d from the face

q_{ud} = 273.4- (1.5-0.715) *44.3.

= 238.6 KN/m^2.

V_{ud} = 1.6*((273.4+238.6) *0.5*0.785) – (0.9*20*0.785).

= 307.4 KN.

$\Phi V_c = \Phi * \sqrt{fc'} * b * d/6$.

$=0.75*\sqrt{24}*1*715/6$ =437.85 KN.

$V_{ud} < \Phi V_c$.

❖ The thickness of 80 cm of the toe slab is adequate.

b. Design for flexure:

M_u = 1.6*((273.4-207) *0.5*1.5^2*2/3+207*1.5^2*0.5) – 0.9*20*1.5^2*0.5.

= 432 KN.m.

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{432\times10^6}{0.9\times1000\times715^2} = 0.94\text{Mpa}$$

$$m = \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85*24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}}\right)$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.94*20.6}{420}}\right) = 0.0023$$

→$A_{s(req)}$= ρ * b *d = 0.0023* 1000*715 = 1644.5 mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \;\geq\; \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 715 \;\geq\; \frac{1.4}{420} * 1000 * 715.$$

=2085 mm^2/m < 2383.3 mm^2/m (Larger value is control).

As_{min} = 2383.3 mm^2 /m > $A_{s(req)}$ = 1644.5 mm^2 /m.

Or:

As_{min} = 4*1644.5/3 = 2192.7

$A_{s(req)}$ = As_{min} = 2192.7 mm^2/m.

Try ϕ18/100 mm with A_s = 2543.4 mm^2/m > $A_{s(req)}$ = 2192.7 mm^2/m.

Minimum shrinkage A_{sh} =0.0018*b*h.

=1440 mm^2/m

Temperature and shrinkage reinforcement in the longitudinal direction is ϕ14/100mm.

c. Development length.

Design of tension development length (Ldt)

$Ldt_{(avl)}$ = width of toe – cover.

=1500-75 =1425 mm.

$Ldt_{(req)} = \frac{9*Fy*\psi t*\psi e*\psi s}{10*\sqrt{Fc'}*\lambda*(cb+ktr)/db}*db \geq 300mm.$

According ACI:

$\psi t = 1$, $\psi e = 1$, $\psi s = 0.8$ $\lambda = 1$, $ktr = 0$.

cb is smallest of:

1) Cover + db/2

=75+18/2 =84mm.

2) 0.5* Distance between bars = 0.5*a.

a =0.5*100 = 50 mm.

cb = 50mm.

(cb+Ktr)/db ≤ 2.5mm.

= (50+0)/18 =2.78 > 2.5 $\longrightarrow$ select = 2.5mm.

$Ldt_{(req)} = \frac{9*420*1*1*0.8*18}{10*\sqrt{24}*1*2.5}$ = 444.4 mm < 300 mm.

$Ldt_{(req)}$ = 444.4 mm.

$Ldt_{(avl)}$ > $Ldt_{(req)}$. OK

Design of heel slab:

Width of heel slab = 3m.

The downward pressure due to self-weight of the heel slab and soil backfill is:

W_u = 1.2*(24*8.2+0.8*25) +1.6*10.

= 276 KN/m^2.

a. Design for shear:

V_u = (W*L)/2= 372.6 KN.

d = 800-75-20/2.

=715mm.

$\Phi V_c = \Phi * \sqrt{fc'} * b*d/6$.

$= 0.75*\sqrt{24}*1*715/6$ =437.85 KN.

V_u =372.6KN < ΦV_c =437.85 KN.

The thickness of 80 cm of the heel slab is adequate.

b. Design for flexure:

At support:

$M_u = (W*L^2)/12$.

$= (276*2.7^2)/12$ = 167.7 KN.m.

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{167.7\times10^6}{0.9\times1000\times715^2} = 0.36 \text{ Mpa}$$

$$= \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85*24} = 20.6\text{m}$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.36*20.6}{420}}\right) = 0.00086$$

$\rightarrow A_{s(req)} = \rho * b * d$ = 0.00086* 1000*715 = 615 mm^2/m.

$$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \;\geq\; \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 715 \;\geq\; \frac{1.4}{420} * 1000 * 715.$$

=2085 mm^2/m < 2383.3 mm^2/m (Larger value is control).

As_{min} = 2383.3 mm^2/m > $A_{s(req)}$ = 615 mm^2/m.

Or:

As_{min} = 4* $A_{s(req)}$ /3.

= 820 mm^2 /m.

$A_{s(req)}$ =As_{min} = 820 mm^2 /m.

Try ϕ14/150 mm with A_s = 1026 mm^2/m > $A_{s(req)}$ = 820 mm^2 /m.

Temperature and shrinkage reinforcement A_{sh} = 0.0018*b*h.

= 0.0018*1000*800 = 1440 mm^2 /m.

In the longitudinal direction use ϕ14/100mm.

At middle span:

M_u =(W*L^2)/16.

= (276*2.7^2)/16 = 125.8 KN.m.

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{125.8\times10^6}{0.9\times1000\times715^2} = 0.27\text{Mpa}$$

$$m = \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*24} = 20.6$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.27*20.6}{420}}\right) = 0.00065$$

→$A_{s(req)}$= ρ * b *d = 0.00065* 1000*715 = 465 mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f_c'}}{4\ (f_y)} * b_w * d \ \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 715 \geq \frac{1.4}{420} * 1000 * 715.$$

=2085 mm^2/m < 2383.3 mm^2/m (Larger value is control).

As_{min} = 2383.3 mm^2 /m > $A_{s(req)}$ = 465 mm^2 /m.

Or:

As_{min} = 4* $A_{s(req)}$ /3.

= 620 mm^2 /m.

$A_{s(req)}$ =As_{min} = 620 mm^2 /m.

Try ϕ14/200mm with A_s = 769.3 mm^2/m > $A_{s(req)}$ = 620 mm^2 /m.

Temperature and shrinkage reinforcement A_{sh} = 0.0018*b*h.

= 0.0018*1000*800 = 1440 mm^2 /m.

In the longitudinal direction use ϕ14/100mm.

c. Development length:

Design of tension development length (Ldt)

Ldt = $\frac{9*Fy*\psi t*\psi e*\psi s}{10*\sqrt{Fc'}*\lambda*(cb+ktr)/db} * db \geq 300mm.$

According ACI:

$\psi t = 1\ ,\ \ \psi e = 1,\ \ \psi s = 0.8 \quad \lambda = 1, \quad ktr = 0.$

cb is smallest of:

1) Cover + db/2

   =75+14/2 =82 mm.

2) 0.5* Distance between bars = 0.5*a.

   a =0.5*200 = 100 mm.

cb = 82 mm.

(cb+Ktr)/db ≤ 2.5mm.

= (82+0)/14 =5.86 > 2.5 $\Longrightarrow$ select = 2.5mm.

$Ldt_{(req)}$ = $\frac{9*420*1*1*0.8*14}{10*\sqrt{24}*1*2.5}$ =345.7 mm > 300 mm.

Select $Ldt_{(req)}$ = 350 mm.

$Ldt_{(avl)}$ =2*La + $\frac{Mn}{Vu}$.

La = the larger of (12* db or d).

12*db = 12*14 = 168mm.

d = d of stem =715mm.

- La = 715mm.
- Vu = factored shear at the support = 1.6*W*L/2.

  =276*2.7/2 =372.6 KN.

- Mn = As * Fy *(d-a/2).

As = 5*3.14*14^2/4 = 769.3 mm^2.

a = $\frac{As*Fy}{0.85*Fc'*b}$ = 15.84 mm.

$Mn = 769.3*420*(715-15.84/2)$

$=228.5$ KN.m

$Ldt_{(avl)} = 2*715+(228.5/372.6) = 1431$ mm.

$Ldt_{(avl)} > Ldt_{(req)}$.

d. Connection of heel with counterfort.

Vertical stirrups resist tension forces.

As = shear force /(0.9*Fy).

Shear force = Vu = 1.6* W*L/2.

=276*2.7/2 =372.6 KN

As = (372.6*1000)/ (0.9*420) =986 mm^2.

Select 7ϕ10-2 Legs with As = 1099 mm^2.

Design of counterfort:

Designs the same design as a T- section with b = b counterfort = 30cm.

Tan ∂ = 3/8.2.

∂ =20.1.

- At Y= 2.2 from top:

Sin ∂ = D/2.2.

D= 0.76m.

d=760-75-10=675mm.

- At Y=5.2

Sin ∂ = D/5.2.

D= 1.8m.

d=1800-75-10=1715mm.

- At Y = 8.2.

Sin ∂ = D/8.2.

D= 2.82m.

d=2820-75-10=2735mm.

Pressure at the base of wall:

$\sigma_a = \gamma_1 * H *Ka$.

= 24*9*0.283

=61.13 KN/$(m)^2$.

$\sigma_{(surcharge)}$ =q*ka.

=10*0.283= 2.83 KN/(m)^2.

$\sigma_{(total)}$= 61.13+2.83= 64 KN/m^2.

Forces at base of stem:

Pa1 = 0.5*σa1*H.

=0.5*64*9=288 KN/m.

P(surcharge) = $\sigma_{(surcharge)}$*H.

=2.83*9=25.5 KN/m.

$P_{(total)}$ = 313.5 KN/m.

Load on counter at base = 288*3 +25.5*3= 940.5 KN.

a. Design for flexure:

At Y=9m from top.

Mu = 1.6*((864*9/3) +(76.5*9/2)) =4698 KN

d=2820-75-10 =2735 mm

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{4698\times10^6}{0.9\times300\times(2735)^2} = 2.33 \text{ Mpa}$$

$$m = \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*24} = 20.6$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*2.33*20.6}{420}}\right) = 0.006$$

$\rightarrow A_{s(req)}$= ρ * b *d = 0.006* 300*2735 = 4923 mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f_c'}}{4\ (f_y)} * b_w * d \ \geq \ \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 300 * 2735 \geq \frac{1.4}{420} * 300 * 2735.$$

=2393 mm^2/m < 2735 mm^2/m (Larger value is control).

$A_{s(req)}$ = 4923 mm^2 /m >As_{min} =2735 mm^2/m

$A_{s(req)}$ = 4923 mm^2 /m.

#No = $A_{s(req)}$ / A_s ϕ20.

=4923/314.

15.6 bars $\Longrightarrow$ select 16 bars.

Check of spacing between bars:

It is the largest of:

Clear space ≥ 2.5 cm.

≥ db = 2 cm

Clear space = 2.5 cm.

In thickness of counterfort = 30 cm cannot be put 16 bars in one layer because the clear span equals:

Clear span = (300-2*75-16*20)/15 < 2.5 cm.

Try distribution it in 3 layers in first layer put 6 bars and distribution 10 bars in layer 2 and 3:

Clear span = (300-2*75-6*20)/5 = 6 cm > 2.5 cm.

At Y=5.2m from top.

Forces at 5.2m from top:

Pa1 = 0.5* γ_1 *Ka * H^2.

=0.5*24*0.283*5.2^2=97.83 KN/m.

P(surcharge) = $\sigma_{(surcharge)}$*H.

=2.83*5.2=14.72 KN/m.

$P_{(total)}$ = 112.6 KN/m.

Load on counter at base = 97.83*3 +14.72*3= 337.65 KN.

Mu = 1.6*((293.5*5.2/3) +(44.16*5.2/2) =997.7 KN

Take ϕ =0.9 for flexure.

d=1715 mm

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{997.7\times10^6}{0.9\times300\times(1715)^2} = 1.26 \text{ Mpa}$$

$$m = \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*24} = 20.6$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*1.26*20.6}{420}}\right) = 0.0031$$

$\rightarrow A_{s(req)} = \rho * b * d = 0.0031 * 300 * 1715 = 1595\ mm^2/m.$

$$As_{min} = \frac{\sqrt{f'_c}}{4\,(f_y)} * b_w * d \ \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 300 * 1715 \geq \frac{1.4}{420} * 300 * 1715.$$

$=1500.3\ mm^2/m < 1715\ mm^2/m$ (Larger value is control).

$A_{s(req)} = 1595\ mm^2/m < As_{min} = 1715\ mm^2/m$

$A_{s(req)} = < As_{min} = 1715\ mm^2/m$

#No = $A_{s(req)} / A_s\ \phi 20$.

=1715/314.

5.5 bars $\Longrightarrow$ select 6 bars.

Check of spacing between bars:

It is the largest of:

Clear space ≥ 2.5 cm.

≥ db.

Clear space = 2.5 cm.

In thickness of counterfort = 30 cm can be put 6 bars in one layer because the clear span equals:

clear span = (300-2*75-6*20)/5 = 6 cm > 2.5 cm. OK

At Y=2.2 m from top:

Forces at 2.2 m from top:

$Pa1 = 0.5 * \gamma_1 * Ka * H^2.$

$= 0.5*24*0.283*2.2^2 = 16.44$ kN/m.

P(surcharge) = $\sigma_{(surcharge)}$*H.

=2.83*2.2=6.23KN/m.

$P_{(total)}$ = 22.67 KN/m.

Load on counter at base = 16.44*3 +6.23*3= 68 KN.

Mu = 1.6*((49.32*2.2/3) +(18.7*2.2/2) =90.76 KN

Take ϕ =0.9 for flexure.

d=695 mm

$$Rn = \frac{M_u}{\emptyset b d^2} = \frac{90.76\times10^6}{0.9\times300\times(675)^2} = 0.74 Mpa$$

$m = \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*24}$ =20.6

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$

$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.74*20.6}{420}}\right)$= 0.0018

→$A_{s(req)}$= ρ * b *d = 0.0018* 300*675 = 364.5 mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f_c'}}{4\ (f_y)} * b_w * d \ \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 300 * 675 \geq \frac{1.4}{420} * 300 * 675.$$

=591 mm^2/m < 675 mm^2/m (Larger value is control).

Or:

As_{min} = 4* $A_{s(req)}$ /3.

= 486 mm^2 /m

$A_{s(req)}$ = 486 mm^2 /m.

#No = $A_{s(req)}$) / A_s ф20.

= 486/314.

1.5 bars ⟹ select 2 bars.

Check of spacing between bars:

It is the largest of:

Clear space ≥ 2.5 cm.

≥ db.

Clear space = 2.5 cm.

In thickness of counterfort = 30 cm can be put 2 bars in one layer because the clear span equals:

clear span = (300-2*75-2*20)/1 = 11 cm.

11 cm > 2.5 cm. OK.

The following Figures (3.4) show all reinforcement details in each element in the Counterforted retaining wall.

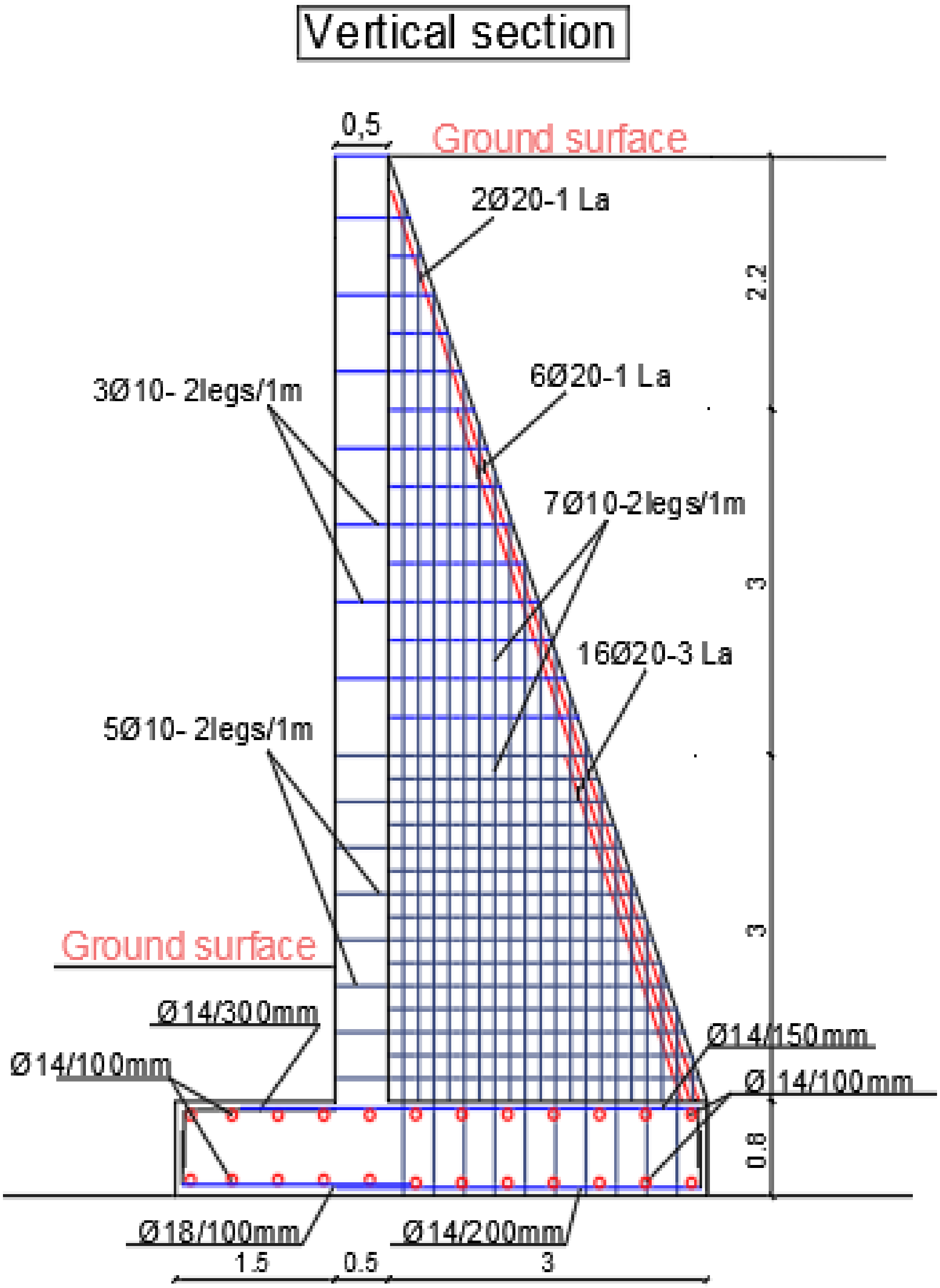

Figure 3-4: (Reinforcement details of counterfort and toe)

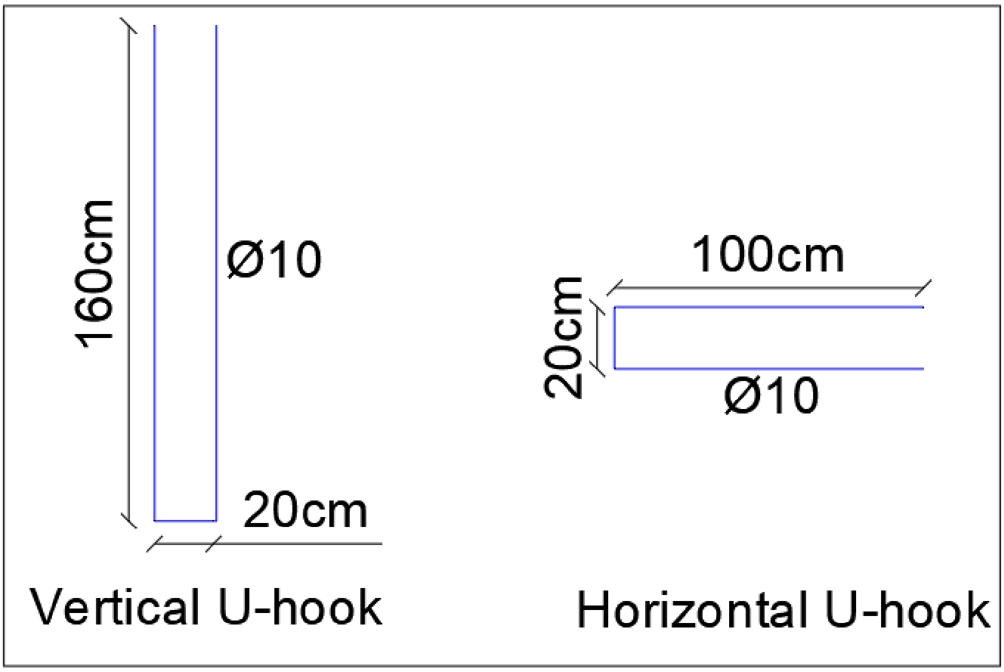

Figure 3-5:(Details of U-hook).

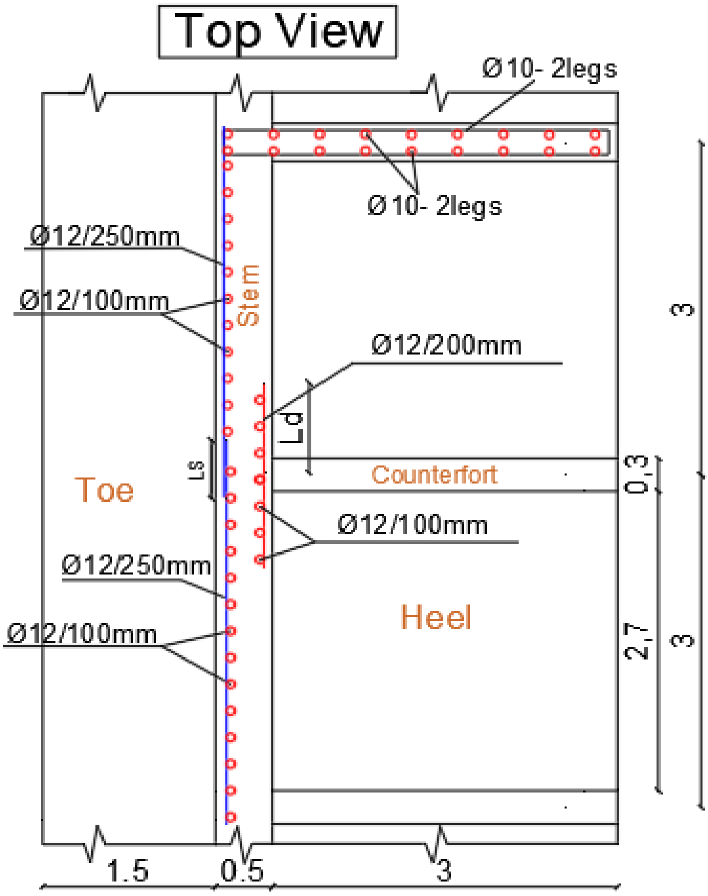

Figure 3-6 : (Reinforcement details of stem)

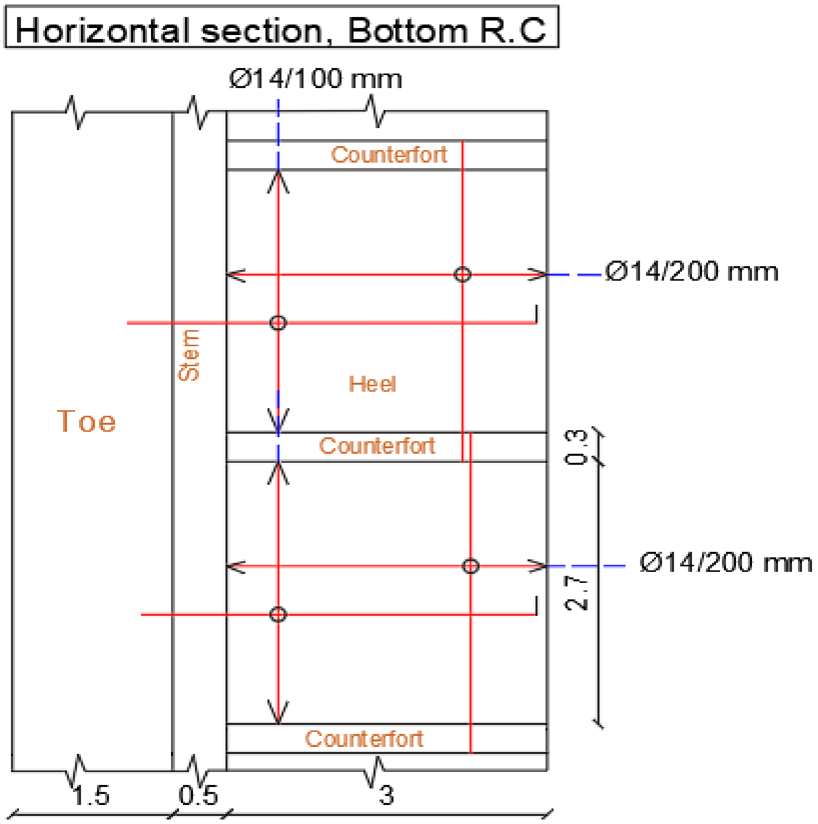

Figure 3-7: (Bottom Reinforcement details of heel)

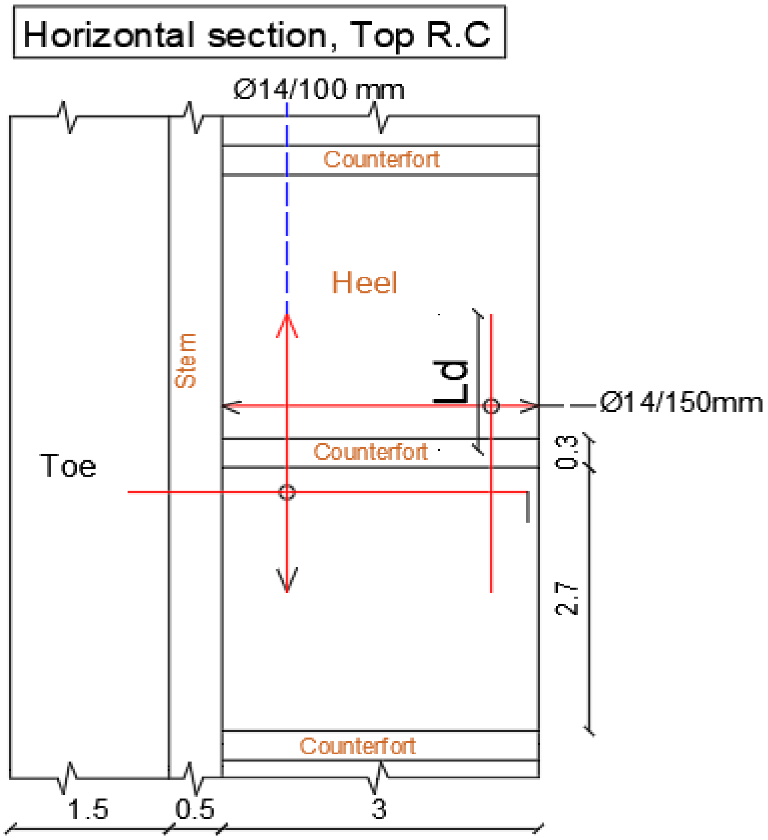

Figure 3-8: (Top Reinforcement details of heel)

3.7 Design of Buttressed walls.

Examples:

Case 1 and Case 2:

These walls were unstable, the factor of safety of sliding was less than 1.5 and the soil was subject to tension. To solve these problems the base width was increased from 5.0-m to 5.5-m and designed a key. But after designing the key the depth was 4-m, which is illogical, so to solve this problem, there should be a design of piled foundation under the wall or the soil could be replaced with better soil or the soil under the wall could be improved by mixing the soil with asphalt, cement, or lime.

Case 3:

Solution:

Usually in the analysis and design of retaining walls consider a 1.0-meter length of the wall.

Data:

The surcharge equals 10 KN/m2.

Material

Fc': 24.00Mpa.

Fy: 420MPa

The unit weight for wall parts equal unit weight of reinforced concrete:

γ_c =25 KN/m^3.

γ_s =24 KN/m^3.

Maximum diameter of bars Φ20.

Cover = 7.5cm.

Distance between center to center of buttress = 3m.

Thickness of buttress = 0.3m

From stability analysis we found the following:

Ka = 0.283.

Kp =4.2.

$q_{@toe} = q_{max}$ = 583.8 KN/m^2/3m. = 194.6 KN/m^2.

$q_{@heel} = q_{(min)}$ = 288.5 KN/m^2/3m. = 96.2 KN/m^2.

Design of stem:

Pressure at base of stem:

$\sigma_a = \gamma_1 * H * Ka$.

$= 24*8.5*0.283$

$= 57.7$ KN/(m)^2.

$\sigma_{(surcharge)} = q*ka$.

$=10*0.283= 2.83$ KN/(m)^2.

$\sigma_{(total)}= 57.7 +2.83= 60.5$ KN/m^2.

Forces at base of stem:

Pa1 = $0.5*\sigma a1*H$.

$=0.5*57.7*8.5=245.2$ KN/m.

P(surcharge) = $\sigma_{(surcharge)}*H$.

$=2.83*8.5=24.1$ KN/m.

$P_{(total)} = 269.3$ KN/m.

- Moment at support (Negative moment):

  $Mu=1.6*(W*L^2)/12$.

  $=1.6* (60.5*2.7^2)/12$

  = 58.8 KN.m

- Moment at middle span (positive moment):

  $Mu=1.6*(W*L^2)/16$.

  $= 1.6*(60.5*2.7^2)/16$.

  = 44.1 KN.m

- d= depth - cover – (diameter of bar/ 2).

  d = 500-75-(20/2) = 415mm.

a. Design for shear:

Max shear at the face of counterfort (face of support):

Vu = 1.6*(W*L)/2.

=1.6*(60.5*2.7)/2

=130.7 KN.

$\Phi V_c= \Phi*\sqrt{fc'}*b*d/6$.

$=0.75*\sqrt{24}*1*415/6$.

= 254.13. KN.

ΦV_c= 254.13. KN > Vu=130.7 KN.

❖ The thickness of 50 cm at the stem end is adequate enough.

b. Design for flexure:

At support (Positive):

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{58.8\times10^6}{0.9\times1000\times415^2} = 0.38 \text{ Mpa}$$

$$m = \frac{f_y}{0.85\ f'_c} = \frac{420}{0.85*24} = 20.6$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.38*20.6}{420}}\right) = 0.001$$

→$A_{s(req)}$= ρ * b *d = 0.001* 1000*415 = 415 mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f'_c}}{4\ (f_y)} * b_w * d \ \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 415 \ \geq \frac{1.4}{420} * 1000 * 415.$$

= 1210.16 mm^2/m < 1383.3 mm^2/m (Larger value is control).

As_{min} = 1383.3 mm^2 /m >$A_{s(req)}$ = 415 mm^2 /m.

Or

As_{min} = 4* $A_{s(req)}$/3.

=553.3 mm^2 /m.

$A_{s(req)}$ = As_{min} =553.3 mm^2 /m.

Try ϕ12/200 mm with A_s = 565.2 mm^2/m > $A_{s(req)}$ = 553.3 mm^2 /m.

Temperature and shrinkage reinforcement:

As_{min} horizontal = 0.002*b*h.

= 0.0018*1000*500= 900 mm^2 /m.

Use ϕ12/100mm with A_s = 1130.4 mm^2 /m ≥ 900 mm^2 /m.

At middle span:(Negative)

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{44.1\times10^6}{0.9\times1000\times415^2} = 0.28\text{Mpa}$$

$m = \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*24} = 20.6$

$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$

$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.28*20.6}{420}}\right) = 0.00067$

$\rightarrow A_{s(req)} = \rho * b * d = 0.00067 * 1000 * 415 = 278.1\ mm^2/m.$

$$As_{min} = \frac{\sqrt{f_c'}}{4\ (f_y)} * b_w * d \ \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 415 \ \geq \frac{1.4}{420} * 1000 * 415.$$

$= 1210.16\ mm^2/m < 1383.3\ mm^2/m$ (Larger value is control).

$As_{min} = 1383.3\ mm^2/m > A_{s(req)} = 278.1\ mm^2/m.$

Or

$As_{min} = 4 * A_{s(req)}/3.$

$= 371\ mm^2/m$

$A_{s(req)} = As_{min} = 371\ mm^2/m.$

Try ϕ12/250 mm with $A_s = 452.2\ mm^2/m > A_{s(req)} = 371\ mm^2/m.$

Temperature and shrinkage reinforcement:

As_{min} horizontal = 0.0018*b*h.

= 0.0018*1000*500= 900 mm^2/m.

Use ϕ12/100mm with $A_s = 1130.4\ mm^2/m \geq 900\ mm^2/m.$

c. Development length:

Design of tension development length (Ldt)

$$Ldt = \frac{9*Fy*\psi t*\psi e*\psi s}{10*\sqrt{Fc'}*\lambda*(cb+ktr)/db} * db \ \geq 300mm.$$

According ACI:

$\psi t = 1,\ \ \psi e = 1,\ \ \psi s = 0.8\ \ \ \lambda = 1,\ \ \ ktr = 0.$

cb is smallest of:

1) Cover + db/2

=75+12/2 =81mm.

2) 0.5* Distance between bars = 0.5*a.

a =250*0.5 = 125 mm.

cb = 81mm.

(cb+Ktr)/db ≤ 2.5mm.

= (81+0)/12 =6.75 > 2.5 $\Longrightarrow$ select = 2.5mm.

$Ldt_{(req)} = \frac{9*420*1*1*0.8*12}{10*\sqrt{24}*1*2.5}$ =296.3 mm < 300 mm.

Select $Ldt_{(req)}$ = 300 mm.

$Ldt_{(avl)} = 2*La + \frac{Mn}{Vu}$.

La = the larger of (12* db or d).

12*db = 12*12 = 144mm.

d = d of stem =415mm.

- La = 415mm.
- Vu = factored shear at the support = 1.6*W*L/2.

=1.6*60.5*2.7/2 =130.7 KN.

- Mn = As * Fy *(d-a/2).

As = 4*3.14*12^2/4 = 452.2 mm^2.

$a = \frac{As*Fy}{0.85*Fc'*b}$ = 9.3 mm.

Mn = 452.2*420*(415-9.3/2)

=78 KN.m

$Ldt_{(avl)}$ = 2*415+(78/130.7) = 831 mm.

$Ldt_{(avl)} > Ldt_{(req)}$.

Design of heel:

a. Design for shear:

The downward pressure due to self-weight of the heel slab and soil backfill is:

W_u = 1.2*(7.4*24+1.6*25) +1.6*10.

= 277 KN/m^2.

V_u = 277*1*3 = 831 KN.

d = 1600-75-20/2.

= 1515mm.

$\Phi V_c = \Phi * \sqrt{fc'} * b * d/6.$

$= 0.75 * \sqrt{24} * 1 * 1515/6 = 928$ KN.

$0.5 * \Phi V_c = 464$ KN.

$V_u < \Phi V_c$.

831 KN < 928 KN

The thickness of 160 cm of the heel slab is adequate.

b. Design for flexure:

$M_u = 277 * 3^2 * 0.5 = 1246.5$ KN.m.

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{1246.5 \times 10^6}{0.9 \times 1000 \times 1515^2} = 0.6\text{Mpa}$$

$$m = \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85 * 24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2 * R_n * m}{f_y}}\right)$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2 * 0.6 * 20.6}{420}}\right) = 0.00145$$

$\rightarrow A_{s(req)} = \rho * b * d = 0.00145 * 1000 * 1515 = 2197$ mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \;\; \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4 * 420} * 1000 * 1515 \;\; \geq \frac{1.4}{420} * 1000 * 1515.$$

$= 4418$ mm^2/m < 5050 mm^2/m (Larger value is control).

$As_{min} = 5050$ mm^2 /m > $A_{s(req)} = 2197$ mm^2 /m.

Or

$As_{min} = 4 * A_{s(req)} / 3 = 2929$

$A_{s(req)} = 2929$ mm^2 /m.

Try ϕ18/80 mm with $A_s = 3179$ mm^2/m > $A_{s(req)} = 2929$ mm^2 /m.

Temperature and shrinkage reinforcement $A_{sh} = 0.0018 * b * h$.

$= 0.0018 * 1000 * 1600 = 2880$ mm^2 /m.

In the longitudinal direction use ϕ18/100mm.

c. Development length.

Design of tension development length (Ldt)

$Ldt_{(avl)}$ = width of heel – cover.

=3000-75 =2925 mm.

$$Ldt_{(req)} = \frac{9*Fy*\psi t*\psi e*\psi s}{10*\sqrt{Fc'}*\lambda*(cb+ktr)/db}*db \geq 300mm.$$

According ACI:

$\psi t = 1$, $\psi e = 1$, $\psi s = 0.8$ $\lambda = 1$, $ktr = 0$.

cb is smallest of:

1) Cover + db/2

   =75+18/2 =84 mm.

2) 0.5* Distance between bars = 0.5*a.

a =0.5*80 = 40 mm.

cb = 50mm.

(cb+Ktr)/db ≤ 2.5mm.

= (340+0)/18 =2.2 < 2.5 $\longrightarrow$ select = 2.1mm.

$$Ldt_{(req)} = \frac{9*420*1*1*0.8*18}{10*\sqrt{24}*1*2.2} = 505 \text{ mm} > 300 \text{ mm}.$$

$Ldt_{(req)}$ = 505 mm.

$Ldt_{(avl)} > Ldt_{(req)}$. OK

Design of toe slab:

Width of toe slab = 2m.

The slope = $(q_{max}-q_{min})/B$.

= (194.6*1.6-96.2*1.6)/5.5 = 28.6.

The pressure at the face of toe:

q at face of toe =311.4-2*28.6=254.2 KN/m^2.

q at face of toe = 254.2 KN/m^2.

q at end of toe = 311.4 KN/m^2.

d= 1600-75-20/2

= 1515 mm.

W= pressure due rectangular + pressure due triangle.

W pressure due rectangular = 254.2 KN/m^2.

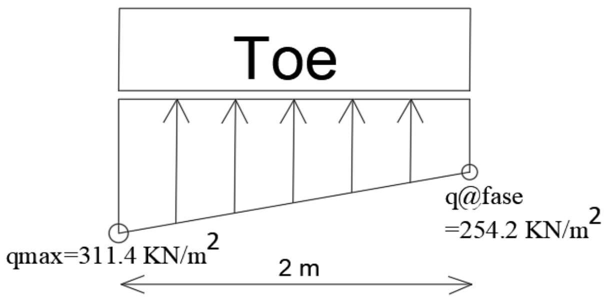

Figure 3-9: (Bearing pressure under the toe)

W pressure due triangle = (311.4-254.2) = 57.2 KN/m^2.

Select W = maximum pressure under the toe = 311.4 KN/m^2.

a. Design for shear:

V_u = (W*L)/2= 420.4 KN.

d = 1600-75-20/2.

= 1515 mm.

$\Phi V_c = \Phi * \sqrt{fc'} * b * d/6$.

$= 0.75 * \sqrt{24} * 1 * 1515/6 = 928$ KN.

V_u =420.4 KN < 0.5*ΦV_c =464 KN.

The thickness of 160 cm of the toe slab is adequate.

b. Design for flexure:

At support:(Positive)

$M_u = (W*L^2)/12$.

$= (311.4*2.7^2)/12 = 189.2$ KN.m.

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{189.2\times10^6}{0.9\times1000\times1515^2} = 0.1\text{Mpa}$$

$$= \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*24} = 20.6 m$$

$$\rho = \frac{1}{m}(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}})$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.1*20.6}{420}}\right) = 0.00024$$

→$A_{s(req)}$= ρ * b *d = 0.00024* 1000*1515 = 363.6 mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f_c'}}{4\ (f_y)} * b_w * d \ \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 1515 \ \geq \frac{1.4}{420} * 1000 * 1515.$$

=4418 mm^2/m < 5050 mm^2/m (Larger value is control).

As_{min} = 5050 mm^2 /m > $A_{s(req)}$ = 363.6 mm^2 /m.

Or:

As_{min} = 4* $A_{s(req)}$ /3.

= 484.8 mm^2 /m.

$A_{s(req)} = As_{min}$ = 484.8 mm^2 /m.

Try ϕ12/200 mm with A_s = 565.2 mm^2/m > $A_{s(req)}$ = 484.8 mm^2 /m.

Temperature and shrinkage reinforcement A_{sh} = 0.0018*b*h.

= 0.0018*1000*1600 = 2880 mm^2 /m.

In the longitudinal direction use ϕ18/100mm.

At middle span (Negative):

M_u =(W*L^2)/16.

= (311.4*2.7^2)/16 = 142 KN.m.

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\phi bd^2} = \frac{142\times10^6}{0.9\times1000\times1515^2} = 0.07 \text{ Mpa}$$

$$= \frac{f_y}{0.85\, f_c'} = \frac{420}{0.85*24} = 20.6m$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}}\right)$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*0.07*20.6}{420}}\right) = 0.00017$$

→$A_{s(req)}$= ρ * b *d = 0.00017* 1000*1515 = 257.6 mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f_c'}}{4\,(f_y)} * b_w * d \;\; \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 1000 * 1515 \geq \frac{1.4}{420} * 1000 * 1515.$$

=4418 mm^2/m < 5050 mm^2/m (Larger value is control).

As_{min} = 5050 mm^2 /m > $A_{s(req)}$ = 257.6 mm^2 /m.

Or:

As_{min} = 4* $A_{s(req)}$ /3.

= 343.4 mm^2 /m.

$A_{s(req)} = As_{min}$ = 343.4 mm^2 /m.

Try ϕ12/300 mm with A_s = 377 mm^2/m > $A_{s(req)}$ = 343.4 mm^2 /m.

Temperature and shrinkage reinforcement A_{sh} = 0.0018*b*h.

= 0.0018*1000*1600 = 2880 mm^2 /m.

In the longitudinal direction use ϕ18/100mm.

c. Development length.

Design of tension development length (Ldt):

Ldt $_{(req)}$ = $\frac{9*Fy*\psi t*\psi e*\psi s}{10*\sqrt{Fc'}*\lambda*(cb+ktr)/db}*db \geq 300mm.$

According ACI:

$\psi t = 1\ ,\ \ \psi e = 1,\ \ \psi s = 0.8\ \ \ \lambda = 1,\ \ \ ktr = 0.$

cb is smallest of:

1) Cover + db/2

=75+12/2 =81 mm.

2) 0.5* Distance between bars = 0.5*a.

a =0.5*300 = 150 mm.

Cb = 81 mm.

(Cb+Ktr)/db ≤ 2.5mm.

= (81+0)/12 =6.75 > 2.5 $\Longrightarrow$ select = 2.5mm.

Ldt$_{(req)}$ = $\frac{9*420*1*1*0.8*12}{10*\sqrt{24}*1*2.5}$ =296.3 mm < 300 mm.

Select Ldt$_{(req)}$ = 300 mm.

Ldt$_{(avl)}$ =2* La + $\frac{Mn}{Vu}$.

La = the larger of (12* db or d).

12*db = 12*12 = 144 mm.

d = d of stem =1515mm.

- La = 1515mm.
- Vu = factored shear at the support = 1.6*W*L/2.

=311.4*2.7/2 =420.4 KN.

- Mn = As * Fy *(d-a/2).

As = 4*3.14*12^2/4 = 452.16 mm^2.

a = $\frac{As*Fy}{0.85*Fc'*b}$ = 9.31 mm.

Mn = 452.16*420*(1515-9.31/2)

=287 KN.m

Ldt$_{(avl)}$ = 2*1515+(287/420.4) = 3030 mm.

Ldt$_{(avl)}$ > Ldt$_{(req)}$.

d. Design of stirrups:

1. Connection of toe with buttressed.

   Shear force due horizontal pressure:

   Horizontal pressure at any depth Y from top:

   $\sigma_a = \gamma_1 * H * Ka$.

   $= 24*Y*0.283$

   $= 6.8*Y$ KN/(m)^2.

   $\sigma_{(surcharge)} = q*ka$.

   $= 10*0.283 = 2.83$ kN/(m)^2.

   $\sigma_{(total)} = 6.8*Y+2.83$. KN/m^2.

   Total shear force at Y = (6.8*Y+2.83) *2.7 KN/m.

   Factored shear force = 1.6*(6.8*Y+2.83) *2.7

   As = shear force /(0.9*Fy)

   At Y=7.4 m (at base of stem)

   As =230 mm^2.

   Select 2ϕ10-2 Legs with As = 316 mm^2.

   But will be select 4ϕ10-2 Legs in each 1-meter along the width of the toe. Thus, will the distances between the stirrups are less than 300mm.

2. Connection of stem with buttresses.

   In this place, the cause of the shearing is the bearing pressure, so shear force equal:

   Shear force = Vu = 1.6* W*L/2.

   =311.4*2.7/2 =420.4 KN

   As = shear force /(0.9*Fy).

   As = (420.4*1000)/ (0.9*420) =1112.1 mm^2.

   Select 8ϕ10-2 Legs with As = 1264 mm^2.

   In each 1-meter along the stem, 8stirrups will be placed.

Design of buttress:

Designs the same design as a rectangle section with b = b web = b buttress = 30cm.

Calculation the effective depth (d):

Tan ∂ = 2/7.4.

∂ =15. 1.

- At Y = 7.4 at base of stem.

Sin ∂ = D/7.4.

D= 1.93 m.

d=1.93-75-10=1845 mm.

Pressure at the base of stem h=7.4m:

$\sigma_a = \gamma_1 * H * Ka$.

= 24*7.4*0.283

=50.3 KN/(m)^2.

$\sigma_{(surcharge)}$ =q*ka.

=10*0.283= 2.83 KN/(m)^2.

$\sigma_{(total)}$= 50.3+2.83= 53.13 KN/m^2.

Forces at base of stem:

Pa1 = 0.5*σa1*H.

=0.5*50.3*7.4=186.1 KN/m.

P(surcharge) = $\sigma_{(surcharge)}$*H.

=2.83*7.4=21 KN/m.

$P_{(total)}$ = 207.1 KN/m.

Load on counter at base = 186.1*3 +21*3= 621.3 KN.

a. Design for flexure:

At Y=7.4m from top.

Mu = 1.6*((558.3*7.4/3) +(63*7.4/2)) =2576.4KN

d=1930-75-10 =1845 mm

Take ϕ =0.9 for flexure.

$$Rn = \frac{M_u}{\emptyset bd^2} = \frac{2576.4\times10^6}{0.9\times300\times(1845)^2} = 2.8 \text{ Mpa}$$

$$m = \frac{f_y}{0.85\ f_c'} = \frac{420}{0.85*24} = 20.6$$

$$\rho = \frac{1}{m}\left(1 - \sqrt{1 - \frac{2*R_n*m}{f_y}}\right)$$

$$= \frac{1}{20.6}\left(1 - \sqrt{1 - \frac{2*2.8*20.6}{420}}\right) = 0.0072$$

→$A_{s(req)}$= ρ * b *d = 0.0072* 300*1845 = 3985.2 mm ^{2}/m.

$$As_{min} = \frac{\sqrt{f'_c}}{4\,(f_y)} * b_w * d \;\; \geq \frac{1.4}{f_y} * b_w * d$$

$$= \frac{\sqrt{24}}{4*420} * 300 * 1845 \geq \frac{1.4}{420} * 300 * 1845.$$

$=1614\ mm^2/m < 1845\ mm^2/m$ (Larger value is control).

$A_{s(req)} = 3985.2\ mm^2/m > As_{min}=1845$

$A_{s(req)} = 3985.2\ mm^2/m.$

#No = $A_{s(req)}$ / A_s ϕ12.

=3985.2/113.

35.3 bars $\Longrightarrow$ select 36 bars.

Will be put 2 bars in the width of buttress =30 cm, in the horizontal direction along the stem with spacing between bars in vertical direction approximately equal 300mm.

The following Figures (3.10) show all reinforcement details in each element in the Buttressed retaining wall.

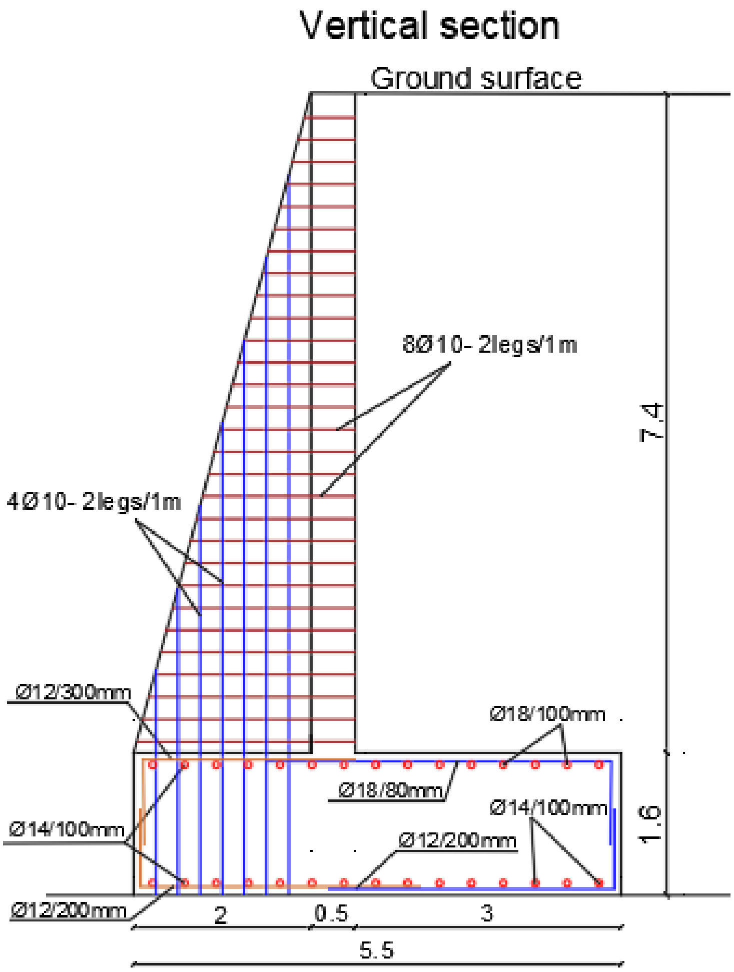

Figure 3-10: (Reinforcement details of Buttress and base)

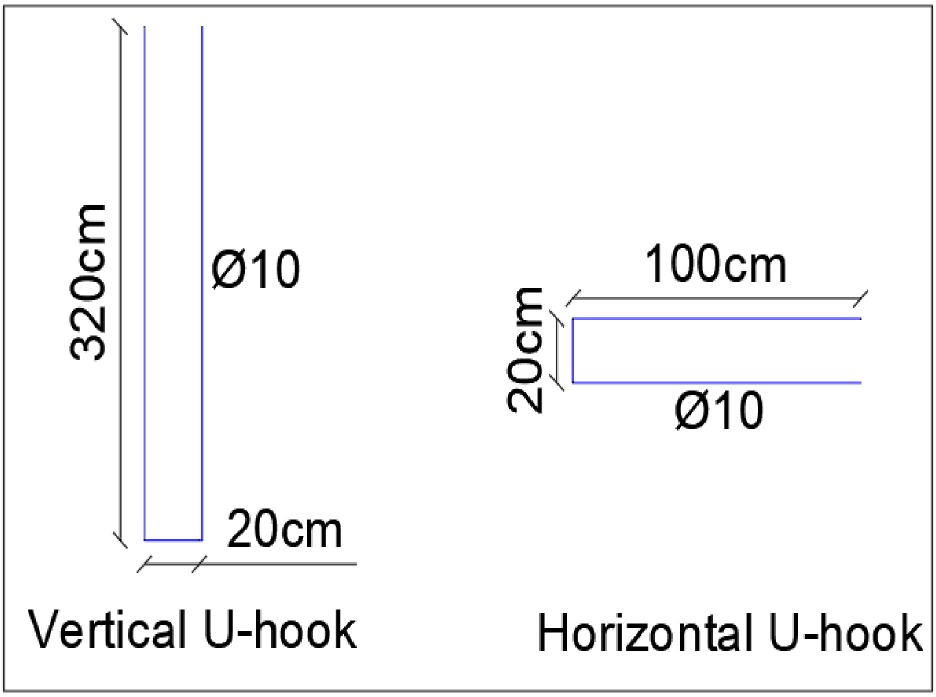

Figure 3-11:(Details of U-hook).

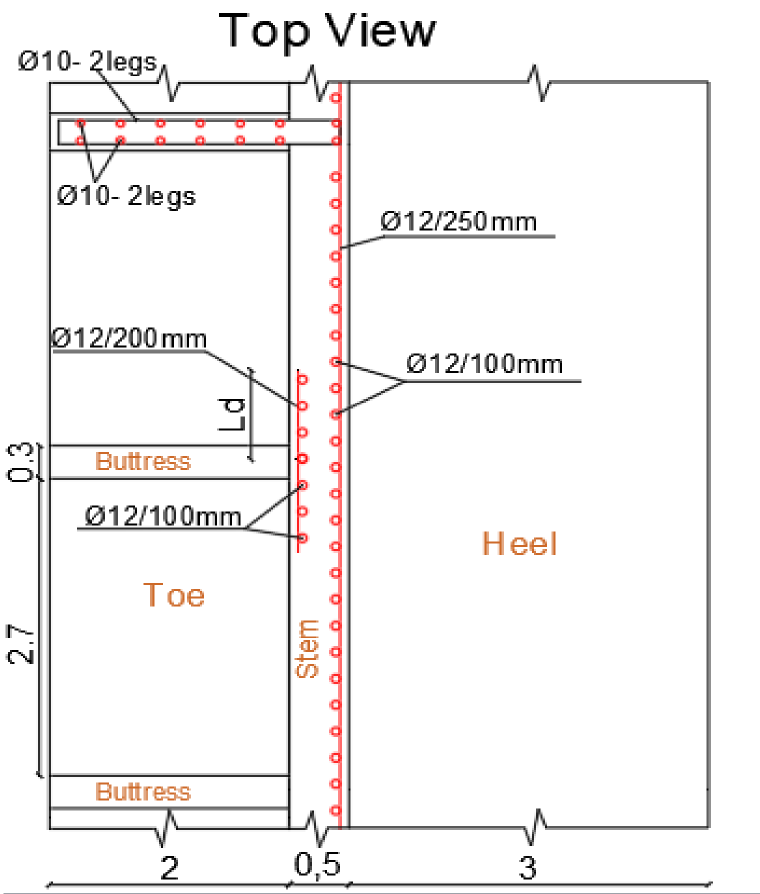

Figure 3-12: (Reinforcement details of stem)

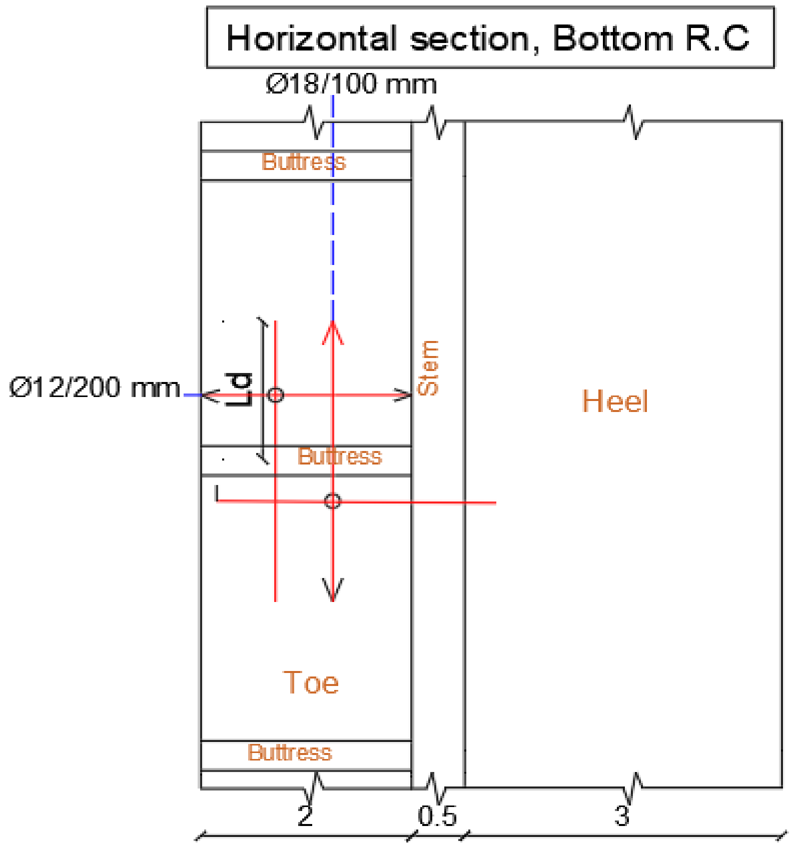

Figure 3-13 : (Bottom Reinforcement details of toe)

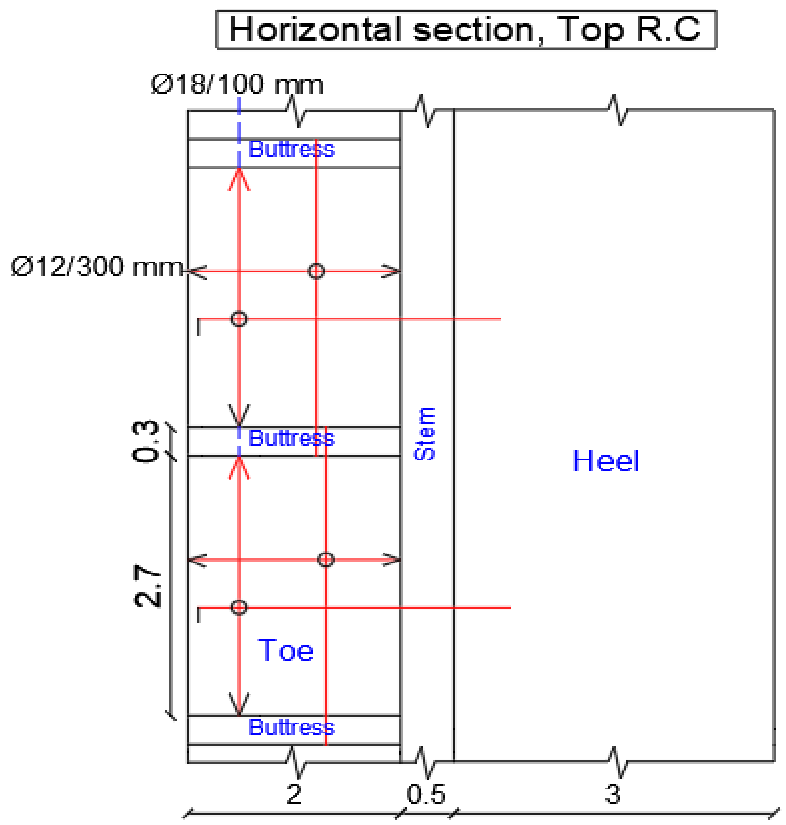

Figure 3-14:(Top Reinforcement details of toe)

3.8 Analysis and design Contiguous piles wall:

Contiguous piles wall:

The following figure shows the earth pressure distribution of Contiguous piles wall with clayey sand soil backfill.

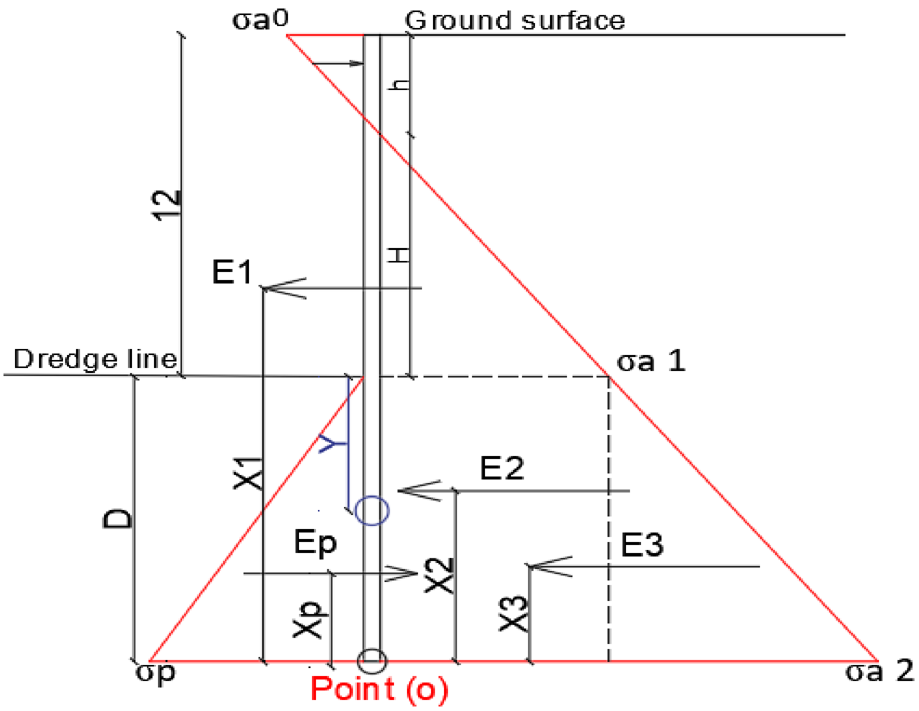

Figure 3-15: (Distribution of forces and lateral pressure on Contiguous pile)

Solution:

The length of the wall equal diameter of pile plus clear spacing between two piles.

Data:

The diameter selected for the piles = 0.8 m.

The distance from the center to the center of the piles = 0.95 m.

Clear spacing between piles = 0.15 m.

The surcharge equals 0 KN/m2.

Material

Fc': 30.00Mpa.

Fy: 420MPa

The unit weight for wall parts equal unit weight of reinforced concrete:

γ_c =25 KN/m^3.

γ_s =20 KN/m^3.

c = 20 KN/m^2.

φ =30 deg.

Maximum diameter of bars Φ20.

Cover = 7.5cm.

ds: diameter of spiral = 12 mm.

- **Analysis of Contiguous piles**:

Calculation of coefficient of lateral earth pressure:

Using Rankine method for active and passive coefficient of lateral earth pressure:

$Ka = \tan^2 (45- \varphi/2)$ or $= (1-\sin \varphi)/ (1+\sin \varphi)$.

$= \tan^2(45- 30/2) = 0.333$

$Kp = \tan^2 (45+ \varphi/2)$ or $= (1+\sin \varphi)/ (1-\sin \varphi)$.

$= \tan^2(45 + 38/2) = 3$

Calculation of h:

$h = \frac{2*c}{\gamma*\sqrt{Ka}} = 3.5$ m

H = 12-3.5 = 8.5m

Calculation of lateral pressure due to overburden pressure:

@ 0 meters.

$\sigma_{a0} = ka* H *\gamma -2*c*\sqrt{Ka} = -23.1$ KN/m^2

@ -12 meters.

$\sigma_{a\,1}=Ka*H* \gamma - 2*c*\sqrt{Ka}$.

$= (20*12*0.333 – 2*20*\sqrt{0.333})$.

$=56.84$ KN/m^2.

@ (-12 + D) meters.

$\sigma_{a\,2} = \sigma_{a\,1} + (\gamma * D*Ka)$.

$= 56.84 +(20*0.333*D)$

$=56.84+6.67$ D KN/m^2.

Calculation of lateral pressure due to soil on the passive side of the wall:

$\sigma_P = \gamma * D*Kp +2*c*\sqrt{Kp}$.

$= 60 D + 2*20*\sqrt{0.333}$.

$=23.1+ 60$ D KN/m^2.

Calculation of forces:

Due to the backfill soil:

$E1 = 0.5*8.5*56.84 = 241.6$ KN/m.

$E2 = D*56.84 = 56.84\ D$ KN/m.

$E3 = 0.5*D*6.67\ D = 3.335\ D^2$ KN/m.

Due to the front soil:

$Ep = 0.5*D*(23.1+60D)$

$= (11.55\ D + 30\ D^2)$ KN/m.

Calculation of moment at point (o):

By used equilibrium equation at any point:

Assume point of zero moment at distance D from dredge line.

1- $\sum M = 0$

Forces:

$E1 = 241.6$ KN/m.

$E2 = 56.84\ D$ KN/m.

$E3 = 3.335\ D^2$ KN/m.

$Ep = -(11.55\ D + 30\ D^2)$ KN/m.

Arms:

$X\ 1 = 2.83+D$

$X\ 2 = 0.5*D$

$X\ 3 = 0.333*D$

$X\ p = 0.333\ D$

$\sum M = 683.73 + 241.6*D + 28.42*D^2 + 1.11*D^3 - 3.85*D^2 - 10*D^3.$

$-8.89\ D^3 + 24.57\ D^2 + 241.6*D + 683.73 = 0$

$D = 7.64$ m $\Longrightarrow$ $1.2*7.64 = 9.2$ m.

Total length = 9.2+12 = 21.2 m

Select total length = 22 m.

Maximum moment at zero shear:

Assume point of zero shear at distance Y from dredge line.

2- $\sum F = 0$.

F = E1+E2+E3- Ep.

$0 = 241.6+56.84*Y+3.335*Y^2-11.55*Y-30*Y^{2.}$

$-26.665*Y^2 + 45.31*Y + 241.6 = 0$

Y = 4 m.

Maximum moment at (Y):

$M = -8.89\ Y^3 + 24.57\ Y^2 + 241.6*Y + 683.73$

M = 1474.3 KN.m.

Design of contiguous piles:

Weight of pile = area*height*unit weight of concrete.

$$= \frac{3.14*0.8^2}{4} *22*25 = 276.32 \text{ KN.}$$

Ultimate weight (Pu) = 1.4*DL = 1.4*276.32 = 387 KN.

Maximum moment = M *(The distance from the center of the pile to half the clear distance between two piles in both directions) = 1474.3*(0.8+0.15) = 1400.6 KN.m/Pile.

Ultimate maximum moment (Mu) = 1.6*1400.6= 2241 KN.m

- To design the contiguous piles will be used Sp column program. The design of the piles will be the same as the design of the circular columns. The following figure shows the details of the reinforcement in the pile section.

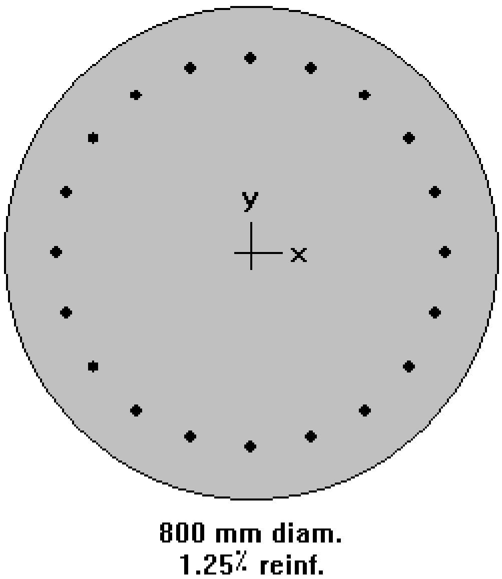

Figure 3-16: (Reinforcement detail in Pile section).

Number of bars = 20 ϕ 20. With area of steel = 6280 mm^2.

Area of section (Ag) = 502400 mm^2.

Reinforcement ratio ρ = 1.25% > 1 %
< 8 %. (OK)

- Design of spiral:

  Dc = D – 2*C.

  = 800-2*75 = 650 mm.

  Ag = 502400 mm^2.

  $Ac = \frac{\pi * Dc^2}{4}$ = 331662.5 mm^2.

  $\rho\ req = 0.45 * \frac{Fc'}{Fy} * (\frac{Ag}{Ac} - 1).$

  = 0.0165

  $\rho\ req = \frac{\pi * as * (Dc - ds) * 4}{Sreq * \pi * Dc^2}$

  $as = \frac{\pi * ds^2}{4}$ = 113 mm^2.

  $0.0165 = \frac{3.14 * 113 * (650 - 12) * 4}{Sreq * 3.14 * 650^2}$

  Sreq = 41.4 mm $\Longrightarrow$ Select Sreq = 40 mm > 25 mm.
  < 75 mm. (OK)

Figure (3.15) shows the spiral in pile.

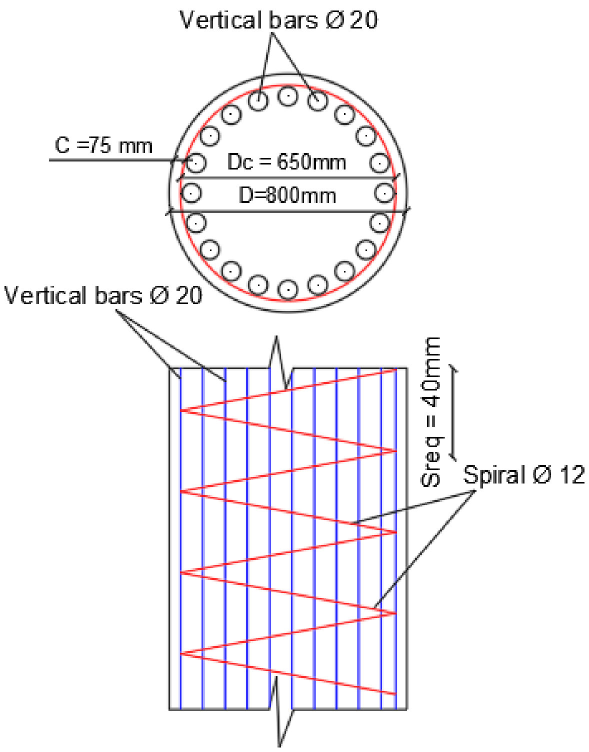

Figure 3-17: (Vertical section in pile).

3.9 Example of excel sheet:

Excel sheets were used to verify the hand calculations. The following is an example from the Excel sheet used in the design of the cantilever earth retaining structure.

Case 1

Wall details:

Retaining wall geometrical information		
Total heigh of retaining wall	4	m
Width of base	2.5	m
Heigh of base	0.45	m
Heigh of stem	3.55	m
Width of toe	0.7	m
Width of stem	0.3	m
Width of heel	1.5	m

Concrete detail:

Reinforced concrete	Unit weight (KN/m^3)
γc	25

Key detail:

Depth of key	1.2	m
Width of key	0.3	m

Soil details:

Soil		(φ) (degree)	c (KN/m2)	(γ) (KN/m^3)	Height layer of soil above wall (H) (m)	Height layer of soil from base of wall (m)	(qall) (KN/m^2)
Soil in backfill side	layer 1	38	0	21	2	2	-
	layer 2	25	0	25	1.7	2	-
Soil in front side		38	0	21	0.7	1	-
Soil in under the wall		20	13.3	20	-	-	250

Loading:

Applied surcharge load	KN/m^2
q	10

Water detail:

Unit weight (KN/m^3) (γw)	height of water (m)
9.81	2

Calculation of coefficient of lateral earth pressure:

Using Rankine method:

Type of coefficient		(φ)	K
Active coefficient (Ka)	layer1 (Ka1)	38	0.238
	layer2 (Ka2)	25	0.406
Passive coefficient (Kp)		38	4.20

Calculation of lateral pressure:

Lateral pressure due soil:

Lateral pressure due active soil (KN/m^2)								
At the end of layer 1				At the start of layer 2				At the end of layer 2
γ	H	Ka1	σa1	γ	H	Ka2	σa2	σa3
21	2	0.238	9.99	25	2	0.406	20.29	29.38

Lateral pressure due passive soil (KN/m^2)			
γ	H	Kp	σp
21	1	4.20	88.3

Lateral pressure due key (KN/m^2)			
γ	H	Kp	σp
20	1.2	2.04	49.0

Lateral pressure due water and surcharge:

Lateral pressure due surcharge (KN/m^2)					
In the layer 1			In the layer 2		
q	Ka	$\sigma_{surcharge}$	q	Ka	$\sigma_{surcharge}$
10	0.238	2.38	10	0.406	4.06

Lateral pressure due water (KN/m^2)		
γ_w	H	σ_{water}
9.81	2	19.62

Calculating forces and point of affect it:

Forces due soil:

Due active soil (KN/m)									
From the beginning to the end of layer 1				From the beginning to the end layer 2					
σa1	H	Pa 1	Y(m)	σa2	H	Pa 2	Y(m)	Pa 3	Y(m)
9.99	2	10.0	2.7	20.29	2	40.6	1	9.1	0.67

Due Passive soil (KN/m)			
σp	H	Pp	Y(m)
88.3	1	44.1	0.33

Due key (KN/m)			
σp	H	Pp	Y(m)
49.0	1.2	29.4	***

Forces due water and surcharge:

Due surcharge (KN/m)							
From the beginning to the end of layer 1				From the beginning to the end layer 2			
$\sigma_{surcharge}$	H	$P_{surcharge\ 1}$	Y(m)	$\sigma_{surcharge}$	H	$P_{surcharge\ 2}$	Y(m)
2.38	2	4.76	3.0	4.06	2	8.12	1.0

Due Water (KN/m)			
σ_{water}	H	P_{water}	Y(m)
19.62	2	19.62	0.67

Calculation of the vertical loads:

weight of:		area (m^2)	unit weight (KN/m^3)	weight per unit length (KN/m)
weight of stem (w1)		1.065	25	26.625
weight of base (w2)		1.125	25	28.125
weight of front soil (w3)		0.49	21	10.29
weight of backfill soil	layer 1 (w5)	3	21	63
	layer 2(w4)	2.33	25	58
weight of key (w6)		0.36	25	9
vertical loads sum				195.2

Check of Overturning about point:

Overturning moment (Mo):

Moment due	Force (P) (KN/m)		Vertical distance (Y)(m)	Mo (KN.m/m)
Backfill soil	Pa1	9.99	2.67	26.64
	Pa2	40.59	1.00	40.59
	Pa3	9.08	0.67	6.06
Surcharge	$P_{surcharge\ 1}$	4.76	3.00	14.27
	$P_{surcharge\ 2}$	8.12	1.00	8.12
Water	P_{water}	19.62	0.67	13.08
Sum of Mo				108.75

Resisting moment (MR):

Weight (KN/m)		X (m)	MR(KN.m/m)
w1	26.625	0.825	21.97
w2	28.125	1.25	35.2
w3	10.29	0.35	3.6
w4	58	1.725	100.3
w5	63	1.725	108.7
w6	9	0.825	7.4
Sum of MR			278.3

Check of sliding:

Sliding forces:

Sliding forces		KN/m
Active forces	Pa1	9.99
	Pa2	40.59
	Pa3	9.08
Surcharge forces	$P_{surcharge\ 1}$	4.76
	$P_{surcharge\ 2}$	8.12
Water	P_{water}	19.62
Sum of sliding forces		92.2

Resisting forces:

(φ) of soil under wall soil	C of soil under wall soil	Passive forces (KN/m)	Resist forces (KN/m)
20	13.3	73.5	141.9

Check for bearing pressure under the wall:

Equ 1 q(max) = (∑V/B*(1+(6*e/B)).

Equ 2 qmax= (4*∑V)/(3*L(B-2*e)).

Calculating eccentricity of acting loads (e):

∑MR (KN.m)	∑Mo (KN.m)	∑Mv (KN/m)	e (m)	B/6 (m)	Check (e<B/2)
278.3	108.8	195.2	0.38	0.41666667	use equ 1

Calculation qmax:

q max	149.50
qmin	6.63

Check stability of retaining wall:

a) Check of Overturning:

The factor of safety for overturning	∑MR (KN.m)	∑Mo (KN.m)	F.S(ov)=∑MR/∑Mo	Check F.S(ove)>2
	278.3	108.8	2.56	safe

b) Check of sliding:

The factor of safety for sliding	Resist forces (KN/m)	Sliding forces (KN/m)	F.s(sliding)=Resist force/sliding force	Check F.S(sliding)>1.5
	141.9	92.2	1.54	safe

c) Check for bearing pressure:

Maximum bearing capacity (KN/m^2)	Allowable bearing capacity (qall) (KN/m^2)	Check q (qmax<qall)
149.50	250	safe

Design of wall:

General details:

Fc'	24
Fy	420
Φbars	20
cover	75
Φflexure	0.9

Stem design:

Design for shear:

Pa1	62.3
Pa(sur)	11.66
Pa(total)	74.0

Shear force (Vu):

Vu	118.40
d	215
ΦVc	131.66
0.5ΦVc	65.83
check:	adequate

Design for flexure:

Mu	128.53
Rn	3.1
m	20.6
p	0.0080
As(req)	1723.8
As(min)	627.0
	716.7
As(min)	716.7
As (req)	1723.8

Toe design:

Design for shear:

Self-weight	11.25
Slope	57.1
q at the face	109.50
d	365

Shear design:

q at distance d	130.36
Vud	71.61
ΦVc	223.52
0.5ΦVc	111.76
check:	adequate

Design for flexure:

Mu	55.38
Rn	0.462
m	20.6
p	0.0011
As(req)	406.0
As(min)	1064.4
	1216.67
As(min)	1216.7
As (req)	1216.7

Heel design:

Design foe shear:

Self-weight	11.25
Soil backfills	80.8
Wu	126.40
Vu	189.6
d	365
ΦVc	223.52
0.5ΦVc	111.76
check:	adequate

Design for flexure:

<table>
<tr><td>Mu</td><td>142.20</td></tr>
<tr><td>Rn</td><td>1.186</td></tr>
<tr><td>m</td><td>20.6</td></tr>
<tr><td>p</td><td>0.00291</td></tr>
<tr><td>As(req)</td><td>1062.5</td></tr>
<tr><td rowspan="2">As(min)</td><td>1064.4</td></tr>
<tr><td>1216.67</td></tr>
<tr><td>As(min)</td><td>1216.67</td></tr>
<tr><td>As (req)</td><td>1216.67</td></tr>
</table>

Chapter 4 : Results and discussion.

4.1 Results of analysis and design of Cantilever walls.

4.2 Results of analysis and design of Counterfort walls.

4.3 Results of analysis and design of Buttressed walls.

4.4 Results of analysis and design of Contiguous piles walls.

4.1 Results of analysis and design of Cantilever walls:

Table (4.1) shows the results of Cantilever wall analysis and design in the different cases.

Table 4-1: The values of the area of steel required for different cases of Cantilever walls.

Number of cases		Thickness of stem (cm)	Depth of base (cm)	Total forces (KN/m)	q max (KN/m2)	Weight above the heel (KN/m^2)	As of stem (mm^2/m)	As of toe (mm^2/m)	As of heel (mm^2/m)
The cases with surcharge equal 10 KN/m^2	Case 1	30	45	74	142.15	126.4	1724.3	365	1062.5
	Case 3	25	36	57	143.06	131.7	2293.5	472.9	1029
	Case 5	25	35	47.91	131.7	131.6	1867.8	452.2	910.6
	Case 7	25	35	47.91	131.7	131.6	1867.8	452.2	910.6
	Case 9	25	35	46.28	128	127.24	1806.75	438.7	869.3
	Case 11	25	35	43.02	122.6	118.5	1674.75	418.1	807.4
The cases without surcharge	Case 2	30	45	62.2	120.18	128.8	1294.3	306.6	1083.3
	Case 4	25	36	46.5	120.5	135	1650	405.8	1045
	Case 6	25	35	39.1	105.6	134.9	1353	370.1	927.5
	Case 8	25	35	39.1	105.6	134.9	1353	370.1	927.5
	Case 10	25	35	37.47	102.8	130	129.3	357.8	901
	Case 12	25	35	34.2	97.5	119.6	1165	339.4	821.5

The following figures show the effect of forces, weight, maximum pressure and load conditions on the area of steel.

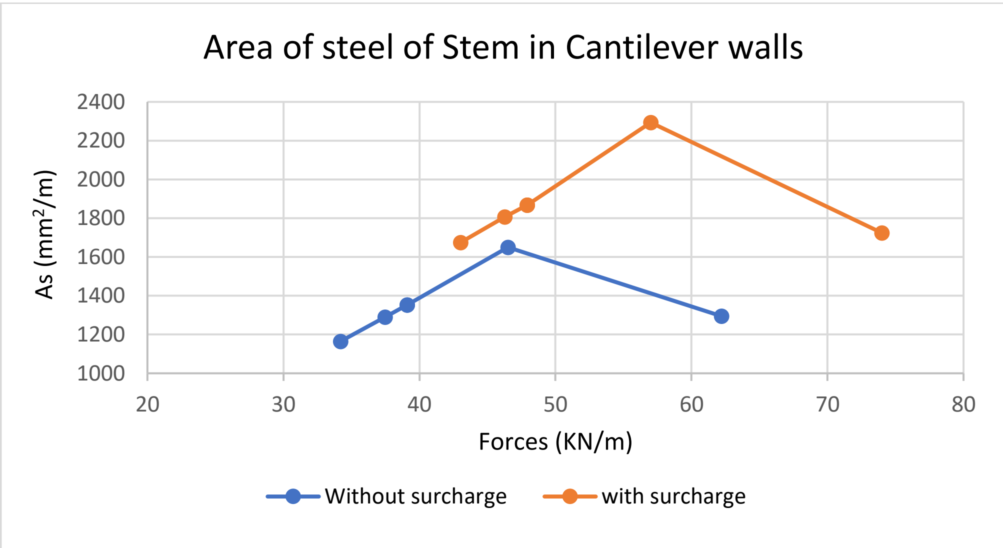

Figure 4-1: (The relation ship between the area of steel of stem and forces in Cantilever walls)

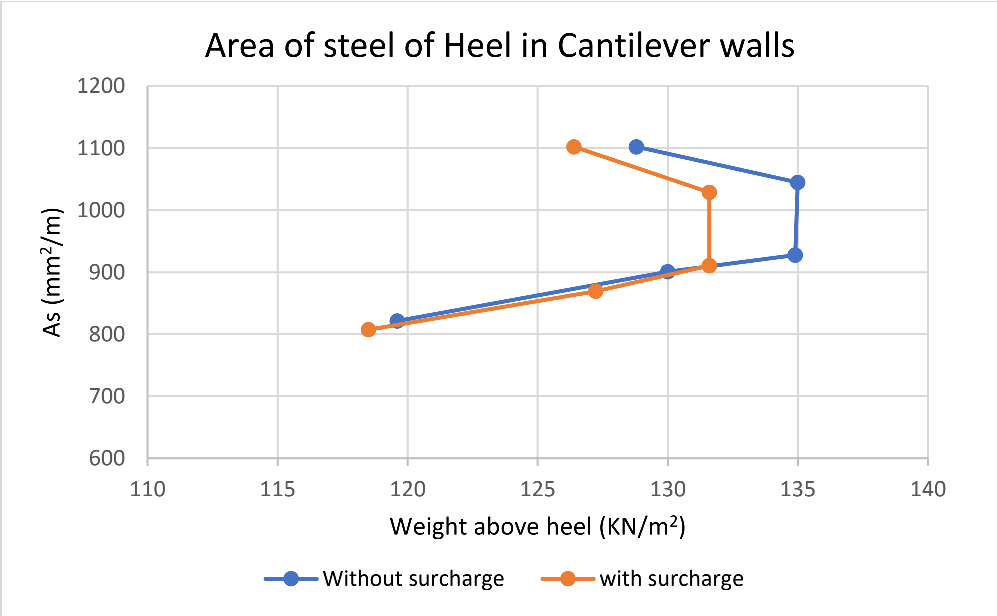

Figure 4-2: (The relationship between the area of steel of heel and weight above heel in Cantilever walls).

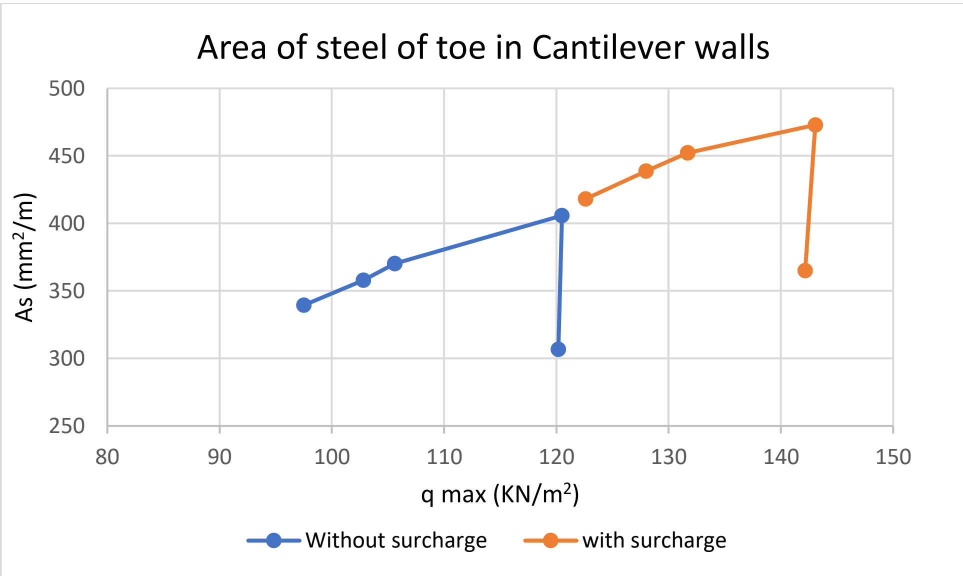

Figure 4-3: (The relationship between the area of steel of toe and maximum pressure in Cantilever walls).

4.2 Results of analysis and design of Counterfort walls:

Table (4.2) shows the results of Counterfort wall analysis and design in the different cases.

Table 4-2:The values of the area of steel required for different cases of Counterfort walls.

Number of cases		Thickness of stem (cm)	Depth of base (cm)	Total forces (KN/m)	q max (KN/m2)	Weight above heel (KN/m2)	Place	As of stem (mm2/m)	As of heel (mm2/m)	As of toe (mm2/m2)	As of counterfort (mm2/m)
The cases with surcharge equal 10 KN/m^2	Case 3	50	80	269.3	273.4	276	At support	415	615	1644.5	4923
							At middle span	278.1	465		
	Case 5	50	80	226.6	237.83	276	At support	318	615	1430	3856.4
							At middle span	238	465		
	Case 7	50	80	226.6	237.83	276	At support	318	615	1430	3856.4
							At middle span	238	465		
	Case 9	50	75	218	231.8	266.2	At support	307.1	665	1530	3692.3
							At middle span	228.3	486.5		
	Case 11	50	70	200.86	200.87	246	At support	278	650.4	1439.1	2924.8
							At middle span	207.5	486		
The cases without surcharge	Case 4	50	80	245.2	246	303.5	At support	357	715	1501.5	4266.6
							At middle span	270	515		
	Case 6	50	80	206.34	214.5	303.5	At support	303.4	715	1358.5	3364
							At middle span	228.3	515		
	Case 8	50	80	206.34	214.5	303.5	At support	303.4	715	1358.5	3364
							At middle span	228.3	515		
	Case 10	50	75	197.75	208.7	292	At support	299	731.5	1396.5	3282
							At middle span	220	532		
	Case 12	50	70	180.63	181.76	268.5	At support	270	738	1316	2547.5
							At middle span	200	529		

The following figures show the effect of forces, weight, maximum pressure and load conditions on area of steel.

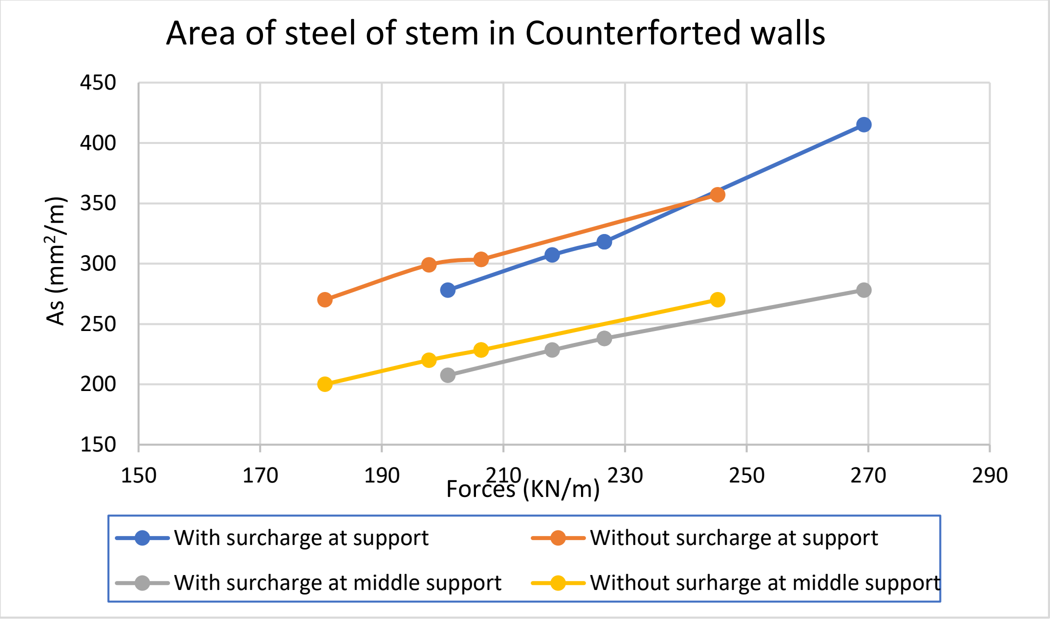

Figure 4-4: (The relationship between the area of steel of stem and forces in Counterforted walls)

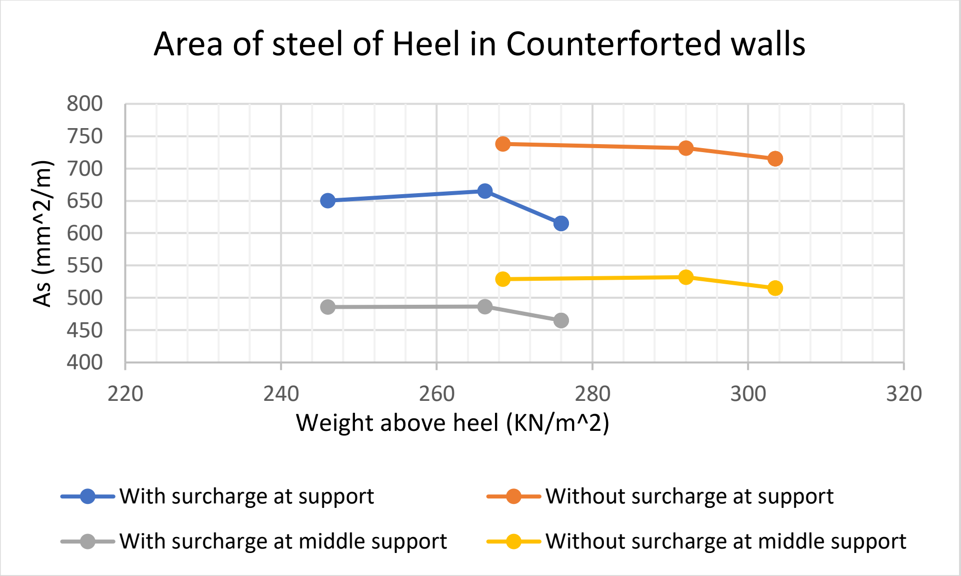

Figure 4-5:(The relationship between the area of steel of heel and weight above heel in Counterforted walls).

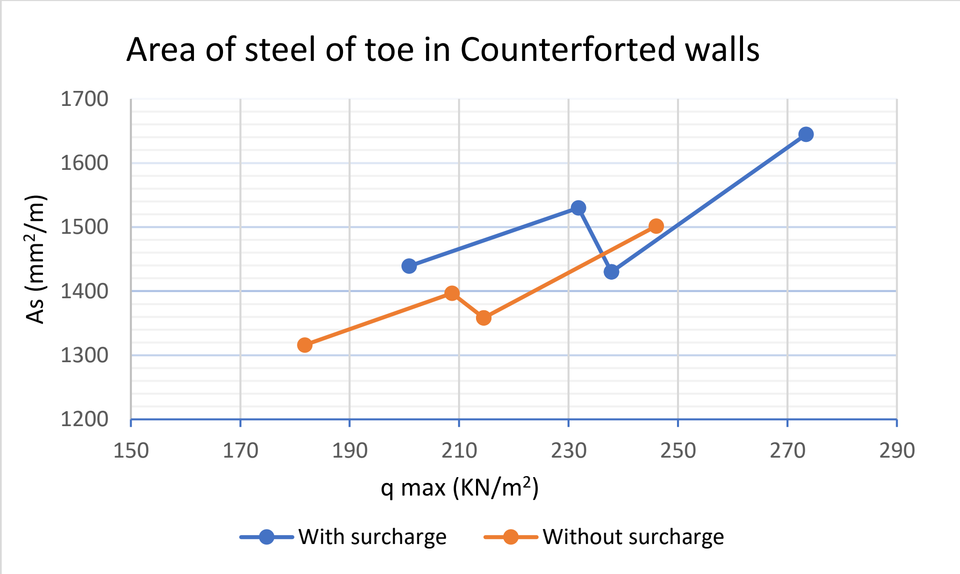

Figure 4-6: (The relationship between the area of steel of toe and maximum pressure in Counterforted walls).

4.3 Results of analysis and design of Buttressed walls:

Table (4.3) shows the results of Buttressed wall analysis and design in the different cases.

Table 4-3: **The values of the area of steel required for different cases of Buttressed walls.**

Number of cases		Thickness of stem (cm)	Depth of base (cm)	Total forces (KN/m)	q max (KN/m2)	Weight above heel (KN/m2)	Place	As of stem (mm2/m)	As of toe (mm2/m)	As of heel (mm2/m)	As of buttress (mm2/m)
The cases with surcharge equal 10 KN/m^2	Case 3	50	160	269.3	194.6	277	At support	415	363.6	2197	3985.2
							At middle span	278.1	257.6		
	Case 5	50	160	226.6	193.81	277	At support	318	333.3	2197	3314.6
							At middle span	238	258		
	Case 7	50	160	226.6	193.81	277	At support	318	333.3	2197	3314.6
							At middle span	238	258		
	Case 9	50	160	218	184.6	268	At support	307.1	318.2	2136.2	3210.3
							At middle span	228.3	242.4		
	Case 11	50	150	200.86	176.5	250	At support	278	321.5	2122.5	3100
							At middle span	207.5	239.14		
The cases without surcharge	Case 4	50	160	245.2	171.9	305	At support	357	303	2424	3376.4
							At middle span	270	227.3		
	Case 6	50	160	206.34	169.5	304.6	At support	303	288	2424	2767.5
							At middle span	227	216.6		
	Case 8	50	160	206.34	169.5	304.6	At support	303	288	2424	2767.5
							At middle span	227	216.6		
	Case 10	50	160	197.75	165.5	294	At support	299	282	2272.5	2656.8
							At middle span	220	209.1		
	Case 12	50	150	180.63	157.4	273	At support	270	283	2264	2546.1
							At middle span	200	212.3		

The following figures show the effect of force, weight, maximum pressure and load conditions on area of steel.

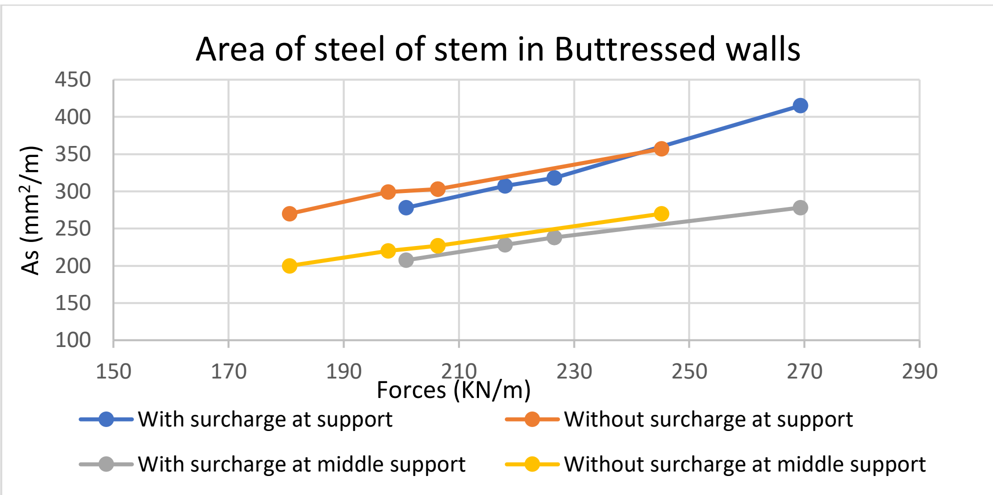

Figure 4-7: (The relationship between the area of steel of stem and forces in Buttressed walls)

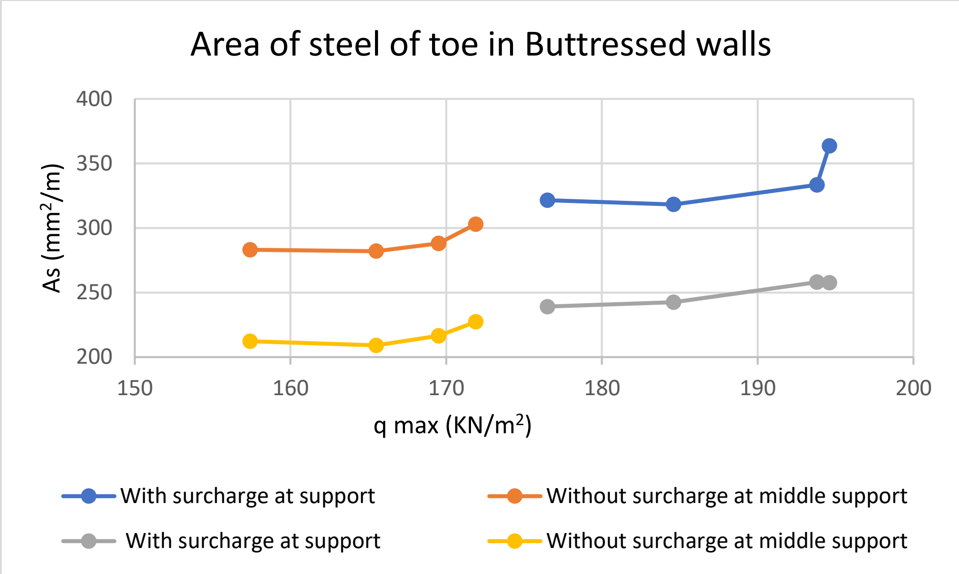

Figure 4-8: (The relationship between the area of steel of toe and maximum pressure in Buttressed walls).

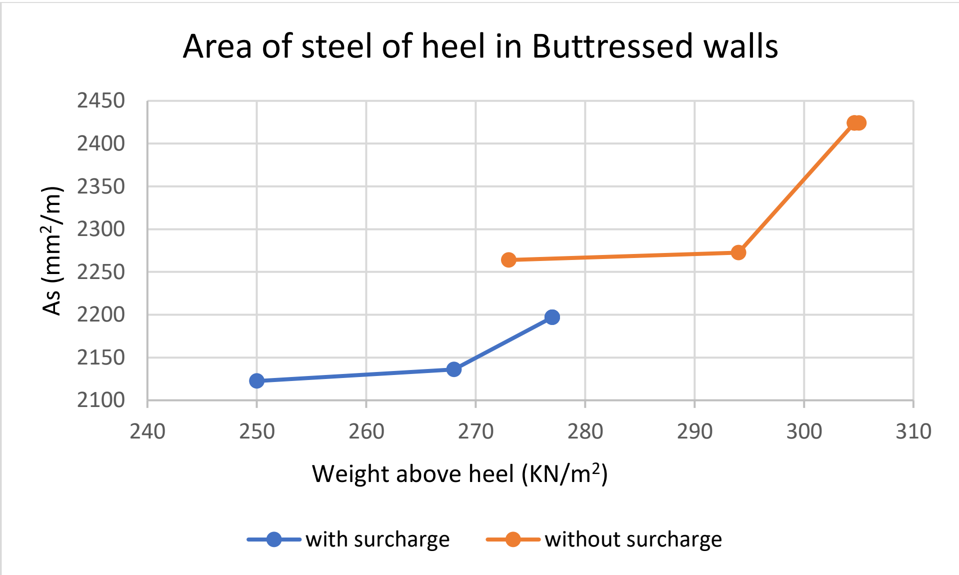

Figure 4-9: (The relationship between the area of steel of heel and weight above heel in Buttressed walls)

The conclusion:

After studying the results for all types of retaining walls and their representation in the previous figures, was noted the following:

1- The area of steel required for the stem increases by increasing the force affecting the wall, as shown in figures (4.1,4.4 and 4.7).

2- The area of steel required for the toe increases by increasing the maximum pressure, as shown in figures (4.3,4.6 and 4.8).

3- The area of steel required for the heel increases by increasing the weight above it, as shown in figures (4.2,4.5 and 4.9).

4- The area of steel required for wall elements increases if the backfill soil is content with water.

5- Increasing the concrete dimensions of the wall elements reduces the required steel area.

6- The area of steel required for wall elements increases with an increasing surcharge load.

4.4 Results of analysis and design of Contiguous piles wall:

This type of wall is used in the case of high excavation, as it is the most appropriate, safer, and easier to implement type compared to other types of earth retaining structures if used for the same height.

Chapter 5 : Conclusion and Recommendations.

5.1- conclusion.

5.2- Recommendations.

5.1 Conclusion:

By analyzing different types of walls by excel and hand calculations, it is concluded that the change of soil parameters and the change of surcharge value on earth retaining structures, influences the stability of retaining structures and the design of it. This means that it is essential to classify the soil correctly when designing any earth retaining structures. When classifying the soil correctly, the engineer can choose the most suitable type of soil retaining structure. At the same time, it is so important to make sure that the material used in the construction is the same as that used in the design. If for any reason, the material was different, then redesign of the wall should be applied.

5.2- Recommendations:

In this project different types of earth retaining structures were studied with changing various soil conditions and cases of loading using the Rankine method. Other studies can be done on the stability and design of the earth retaining structures such as the following:

1. Conduct a study by using Coulomb theory.
2. Conduct a study on the effect of soil condition and cases of load on stability and reinforcement of other types of piles walls; (secant and tangent).
3. Conduct a study on the effect of earthquake load on stability and design of earth retaining structures.

Chapter 6 : References

[1] American Concrete Institute, Building Code Requirements for Structural Concre(ACI 318-19)Commentary., Farminagton Hills: ACI Committee 318, 2019.

[2] Structural Guide, "Forces Acting on a Retaining Wall," Structural Guide, 2020. [Online]. Available: https://www.structuralguide.com/forces-acting-on-a-retaining-wall/. [Accessed 1 November 2020].

[3] ع. ا. القصبي, هندسة الأساسات. تصميم وتنفيذ الأساسات العميقة و الخاصة, القاهرة: دار الكتب العلمية للنشر و التوزيع, ٢٠٢٠.

[4] C.K.Wang and C.G.Salmon, REINFORCED CONCRETE DESIGN, Third ed., Ney York: Harper and Row, 1979.

[5] G. V. R, D.MALLAVALLI, R. PEDDI, N.KATRAGADDA and P.MULPURU., "Contiguous Pile Wall as a Deep Excavation Supporting System," *Leonardo Electronic Journal of Practices and Technologies,* no. 1583-1078, pp. 144-160, 2011.