# Musqat Primary School for Girls – Kufr Raei

# AN NAJAH NATIONAL UNIVERSITY

## Faculty of Engineering

**DESIGN OF FOUNADATION FOR**

**MUSCAT PRIMARY SCHOOL – KUFUR RA'I**

Prepared by

***Ehab Waleed Diab***

***Abdul Rahim Na'em Melhim***

Supervised by

***Dr. Mohammad Ghazal***

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# إهداء

اليكَ يا ذلك النور المسلول من دجية الكون , يا من عانق الكدَّ ورافق التعب, ,يا من وقف في وجه دوامة الحياة , وقارع لأجلي المستحيل ....

اليـــــكَ أبـــــي الحبيب ....

واليكِ انتِ , يا أجمل لحن يرقص له قلبي , وأرق همسة ٍ تغفو بها عيني , اليك ِ يا من جافيتِ مضجعك حتى تكتحل عيناكِ ونحن نغذي خطانا نحو الغد...

اليـــــكِ أمـــــي الحبيبه ....

اليكم انتم , يا من ابحرتم معي في سفينة الحياه, وشاطرتموني العبء الثقيل, وتقاسمتم معي أهوال الأيام, لنرفع على جبين الوطن شعار المجد والأصاله....

اليـــــكم اخوتـــــي .... واليـــــكم أصدقائـــــي....

واليكم ,,, يا أقماراً أنارت دربي , ويا سادة نقشوا على جدران الذاكرة ايات علم وحكمه , ويا كراما أعطوا , وبفضل عطائهم تفتحت الورود في جنات امالي ....

اليـــــكم أساتذتـــــي الأفاضـــــل ....

أهدي هذا العمل

**مع شكري وتقديري لكل من ساهم في انجازه, وكان عونا وسندا....**

**راجياً من الله تعالى.. أن يوفقنا لما فيه خير لهذه الامه ......**

**ايهاب وليد ذياب**

**عبد الرحيم ملحم**

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# CHAPTER ONE

## *Introduction*

All constructions must be carried by foundation. Foundations are structural elements that connect the structure with soil. This element is not visible to the public eye since it lies down the surface of soil (substructure), and it is main function is to transfer the loads of structure to larger area so that the pressure acts on soil is reduced as we move downward till it becomes less than its bearing capacity. So foundations must be designed as safe as possible to be compatible with its duty and function of bearing.

Soil Pressure

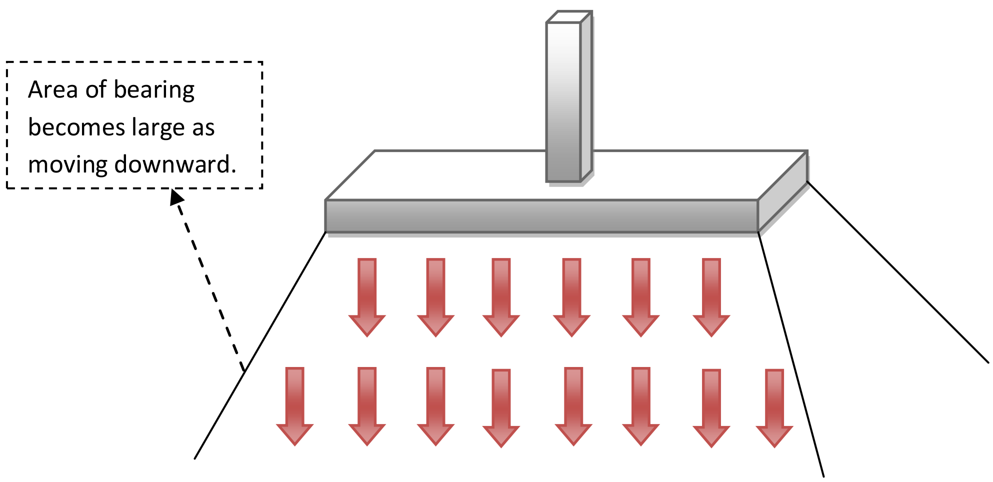
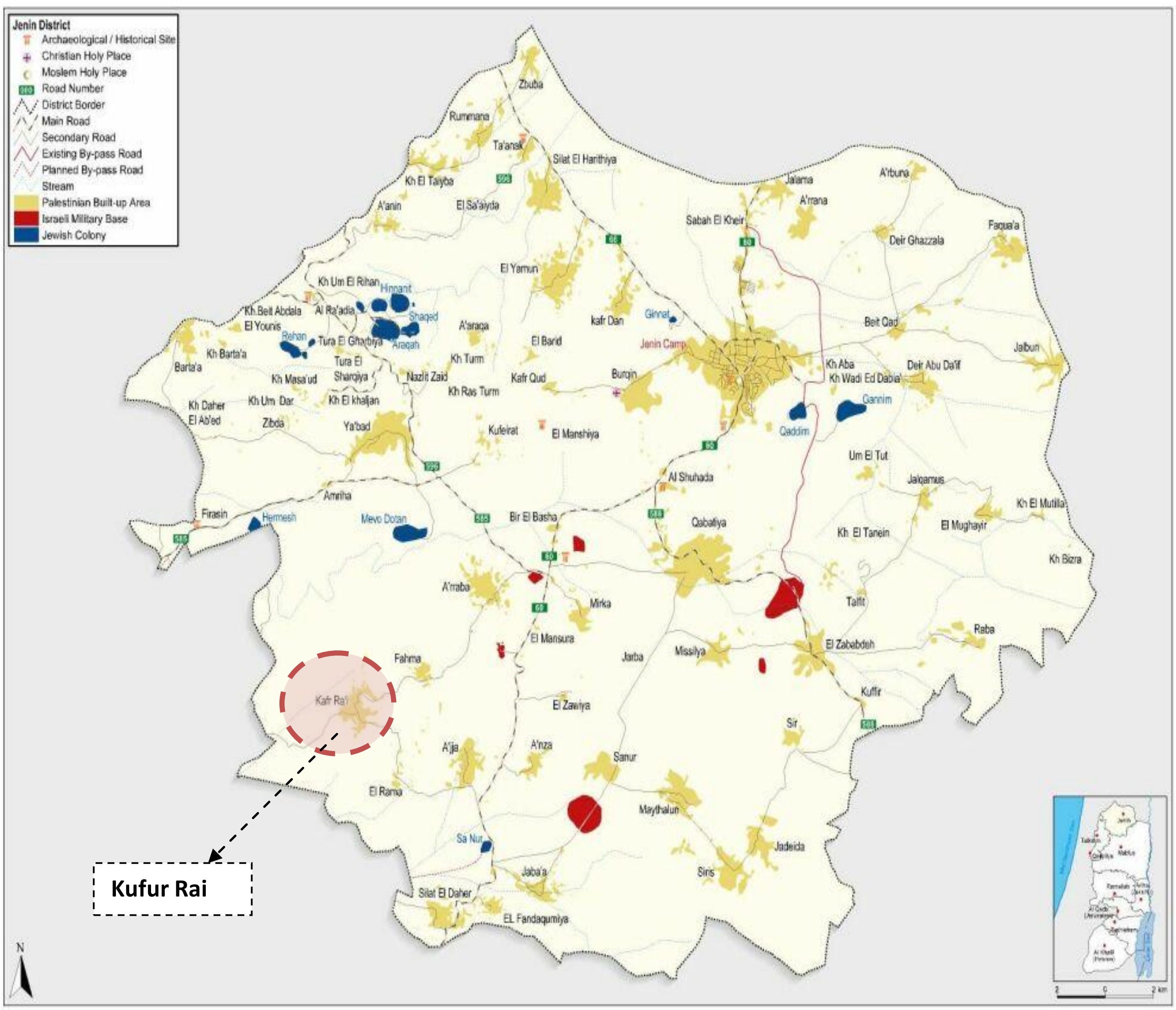



*Figure (1:1) soil pressure under footing*

## 1.1 THE SITE

The site is located in **Jenin** district where it falls in the north ward of the west bank as shown in the map below:


Jenin district population is about 250,000 capita, and it has more than 180 schools building, they are spread out all over the city and villages there. The site specificly in **Kufur Rai** town where located in the south west of the jenin district, about 20 km from the center of distric,also in the middle distance between Talkarem and Jenin. See the figure below




Kufr Rai town area is about 36 km2, half of this area is covered by trees. The population there is about 7500 . Four schools is already built there with some similar charesteristics and constructional properties, but this site is slightly different due to the quality of the land where the project will take palce.

The site is loacted in the north part of the town, opposite to the main street, section No. 21 basin No. 15. Area available  about 5000 m2, where only 1023 m2 will be occupied by the project.

According to the location of project, it has a different features than the other locations of existing projects, since it is surrounding by three mountains, and as a result of soil erosion of these mountains in winter to the location, clay layers is formed there which gives a weak surface of land to be able to carry the building with the available conditions and limited funds, so a proper way must be considered in construction so that fulfill with limitations and conditions there.

## 1.2 THE PROJECT

The project is a school building designed as four floors where a two floor only is carried out with 2046 $m^2$ area, the school has a mark of Omani buildings, as it is a grant from the Omani government.

Since the interest is in designing the foundation, special types of footing will be performed considering all the architectural features and geological properties, also all modification and improvement of section of the land, single footing selected in design as a main component of construction, in addition to the shear and retaining walls.

## 1.3 SCOPE OF PROJECT:

- Evaluation of foundations.
- Selection of the proper foundations
- Design of foundations.

# CHAPTER TWO

## *Literature Review*

## 2.1 SITE INVISTIGATION

### ***2.1.1 GENERAL REQUIREMENTS*:**

The site investigation process of the exploration program includes many steps:-

***- Planning.***

***- Making test boreholes.***

***- Collecting soil samples at desired intervals for subsequent observation and laboratory.***

The approximate required minimum depth of the borings should be predetermined. The depth can be change during the drilling operation, depending on the subsoil encountered.

### ***2.1.2 OBJECTIVES OF INVESTIGATION:-***

***There are many objectives can be achieved from the exploration of the site and neighborhood:-***

1) Evaluating the bearing.

2) Selection of the type of foundation required in the site.

3) Settlement which could be happened there.

4) Determination of problems potential.

5) Location of water table, and its effect on project.

6) Lateral earth pressure.

7) Construction method that will be adopted.

### ***2.1.3 INFORMATION REQUIRED FROM SITE INVISTAGATION:***

The following information should be obtained in the site investigation for foundation engineering purposes:

1) The general topography of the site as it affects foundation and construction, e.g. surface configuration, adjacent property.

2) The location of buried services such as electric power, water mains.

3) The previous history and use of the site including information on any defects or failure of exiting, or method of construction for the neighbor projects which can provide a serious background about the problems that could be happen and the best solution to avoid.

4) General geology of the area with particular Reference to the main geological formations underlying the site and the possibility of subsidence from mineral extraction.

5) A detailed record of the soil and rock strata and ground-water conditions within the zones affected by foundation bearing pressures and construction operations.

6) Results from lab test.

And to get the required information about the site, boreholes must be taken there considering all conditionals and precautions.

Next tables show spacing between boreholes and depth of boreholes in table (2:1) & (2:2) respectively

| Type of project | Spacing (m) | Spacing(ft) |
|---|---|---|
| **Multistory building** | **10- 30** | **30 – 100** |
| **One – story industrial plants** | **20- 60** | **60 – 200** |
| **Highways** | **250 - 500** | **800 – 1600** |
| **Residential subdivision** | **250 - 500** | **800 – 1600** |
| **Dams and dikes** | **40 - 80** | **130 – 260** |

*Table (2:1) Spacing between boreholes*

## Approximate depths of Boreholes

| No. of stories | Boring Depth | |
|---|---|---|
| **1** | **3.5 m** | **11 ft** |
| **2** | **6 m** | **20 ft** |
| **3** | **10 m** | **33 ft** |
| **4** | **16 m** | **53 ft** |
| **5** | **24 m** | **79 ft** |

*Table (2:2) Depth of boreholes*

After taking the sample from the site of project, and make the required test and analysis, bearing capacity of soil must be found first for the high necessity of the determination of the type of foundations that will adopt.

## 2.2 BEARING CAPACITY

Bearing Capacity is the ability of a soil to resist the pressure from foundation without causing a shear failure or excessive settlement. The sign of Bearing Capacity (B.C) where its unit is a pressure's unit (**ton/m$^2$, KN/ m$^2$, Kg/cm$^2$, lb/ft$^2$ etc…**) so it could be called the Bearing Pressure.

The conventional method of designing foundation is based on the concept of bearing capacity. One meaning of the verb to bear is to support or hold up. Generally therefore bearing capacity refers to the ability of a soil to support or hold up a foundation and structure. The ultimate bearing capacity ($q_{ult}$) is the value of bearing stress which causes a sudden catastrophic settlement of the foundation (due to shear failure).

The allowable bearing capacity ($q_a$) is the maximum bearing stress that can be applied to the foundation such that it is safe against instability due to shear failure and the maximum tolerable settlement is not exceeded. The allowable bearing capacity is normally calculated from the ultimate bearing capacity using a factor of safety ($F_s$). When excavating for a foundation, the stress at founding level is relieved by the removal of the weight of soil. The net bearing pressure ($q_n$) is the increase in stress on the soil.

$q_n = q - q_o$

$q_o = \gamma D$

where D is the founding depth and γ is the unit weight of the soil removed.

### ***2.2.1 DEFENITIONS:***

**1. Ultimate B.C ( qult ):** It's the gross pressure at the base of foundation at which the soil fail shear. It's not used for design because it has a big value

**2. Net ultimate B.C ( qu net ):** It's net increase in pressure at the base of foundation cause the failure…

$$q_{u\,net} = q_{ult} - \gamma DF$$

***Where:***

$\gamma DF$ = Over burden pressure at foundation level

$q_{ult}$ = Ultimate B.C

$q_{u\,net}$ = Net ultimate B.C

**3. Net safe Bearing Capacity ( qn.s *):***It's the pressure at which foundation designed

$$qn.s = \frac{q_{n.ult}}{F.S}$$

***Where:***

F.S = Factor of safety equal from (2 to 5)

**4. Safe Bearing Capacity ( qs *):***It means the gross safe Bearing Capacity which used in design.

$$qs = qgs = qns + \gamma D = \frac{q_{n.ult}}{F.S} + \gamma D$$

**5. Net allowable B.C qn.all:** It's the net pressure which can be used for the design of foundation, which ensure that there is not shearing failure, or the settlement within reach the limit, to choose the allowable B.C ( qn.all ).

If the net safe settlement pressure more than net safe B.c

qn.p > qn. qall = qn.s

If the net safe B.C more than the net safe settlement pressure the the allowable B.C equal the net safe settlement pressure.

qn.s > qn.p

qall = qn.p

***Where:***

qn.s = Net Safe B.C

qn.p = Net settlement pressure

qall = Net allowable B.C (Design allowable B.C)

## ***2.2.2 TERZAGHI'S BEARING CAPACITY THEORY:***

As the load is applied, the footing undergoes a certain amount of settlement as it is pushed downward, and a wedge of soil directly below the footing's base moves downward with the footing. The soil's downward movement is resisted by shear resistance of the foundation soil along slip surfaces cde and cfg and by the weight of the soil in sliding wedges acfg and bcde for each set of assumed slip surfaces, the corresponding load that would because failure can be determined. The set of slip surface giving the least applied load (that would cause failure) is the most critical

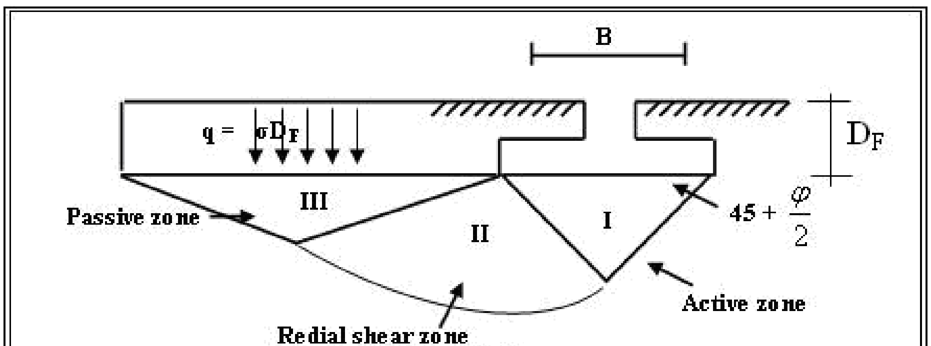



*Fig.* (2.1)

## ***2.2.3 BEARING CAPACITY EQUATION:***

I. Under concentrated vertical central load …

$q_{ult} = CN_c\lambda_c + qN_q\lambda_q + \gamma_2 BN_\gamma\lambda_\gamma$

***Where***:

$q_{ult}$ = Ultimate Bearing Capacity.

C = Cohesive stress.

q = Over burden pressure above (F-L) = $\gamma_1 D_F$

$\gamma_1$ = Unit weight of soil above footing level

$\gamma_2$ = Unit weight of soil at the base of foundation

B = Width of foundation

$N_c$, $N_q$, $N_\gamma$ = Bearing Capacity (B.C) factors depend on φ (angle of internal friction) $N_q = e^{\pi \tan\varphi} . \tan^2 (45 + \frac{\varphi}{2})$

$NC = (Nq - 1) \cot\varphi$

$N\gamma = (Nq - 1) \tan\varphi$

$Nc, Nq, N\gamma = F(\varphi)$

$\lambda_C, \lambda_q, \lambda_\gamma$ = Factors depend on the shape of foundation Shape dimension [B, L].

| Foundation | $\lambda_C$ - $\lambda_q$ | $\lambda_\gamma$ |
|---|---|---|
| **Strip** | **1.0** | **1.0** |
| **Rectangular** | **1 + 0.3 B/L** | **1 – 0.3 B/L** |
| **Square & Circle** | **1.3** | **0.7** |

The value of shape Factor $D_F$ = Depth of foundation.

## 2.3 FOUNDATIONS

### ***2.3.1 INTRODUCTION FOR FOOTING***

Footing foundation is one of the oldest and most popular types of shallow foundations. A footing is an enlargement of the base of a column or wall for the purpose of distributing the load on the supporting soil to a bigger area so that load becomes less than the bearing capacity of the soil at particular depth.

### ***2.3.2 GENERAL REQUIREMENT FOR FOOTINGS:-***

Several precautions and requirements should be taken into consideration for design of footings

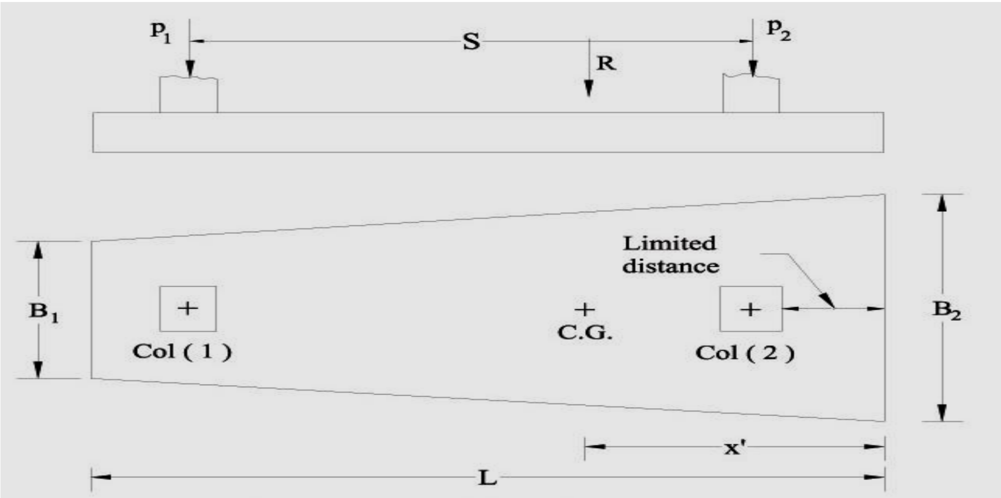
1) Footing area should be suitable to be able to carry the loads acting on it.
2) Footing depth should be adequate to resist the shear, volumetric changes in soil and other weather conditions.
3) Footing should resist the structural movement (sliding, overturning).
4) Footing should be strong as enough as possible to prevent deformation.
5) Footings should be able to resist the high settling and partially settling.
6) Footings should be design taking into the consideration ground water or any saturated layers of soil.
7) Economical conditions and safety should be also considered during design.

## ***2.3.3 TYPES OF FOUNDATIONS***

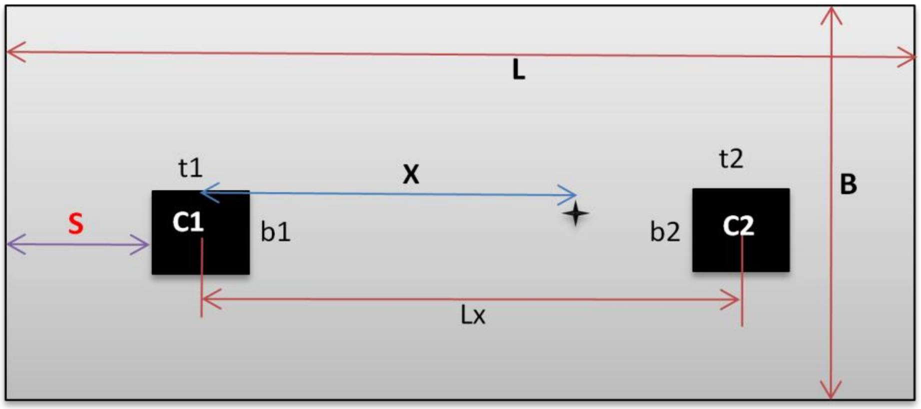
Depending on the structure and soil encountered, various types of foundations are used:-

**A) Isolated footing**: these footings are used to carry individual columns. They may be square, rectangular, or circular. Isolated footing is frequently used to support individual or multiple column loads for building.

**B) Combined footing**: This type is used to support two or more column loads. They may be continues with a rectangular or trapezoidal plan. The combined footing becomes necessary in situation where a wall column has to be placed on a property line that may be common in urban areas. Under such condition an isolated footing may not be suitable since it would have to be eccentrically loaded. Also the combined footing considered to be more economical in cases of having closely spaced columns.

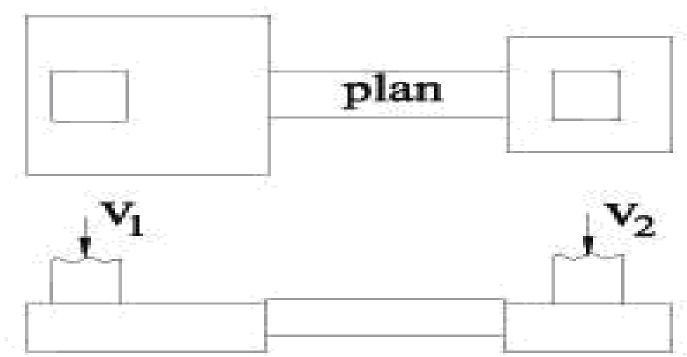


Trapezoidal combined footing

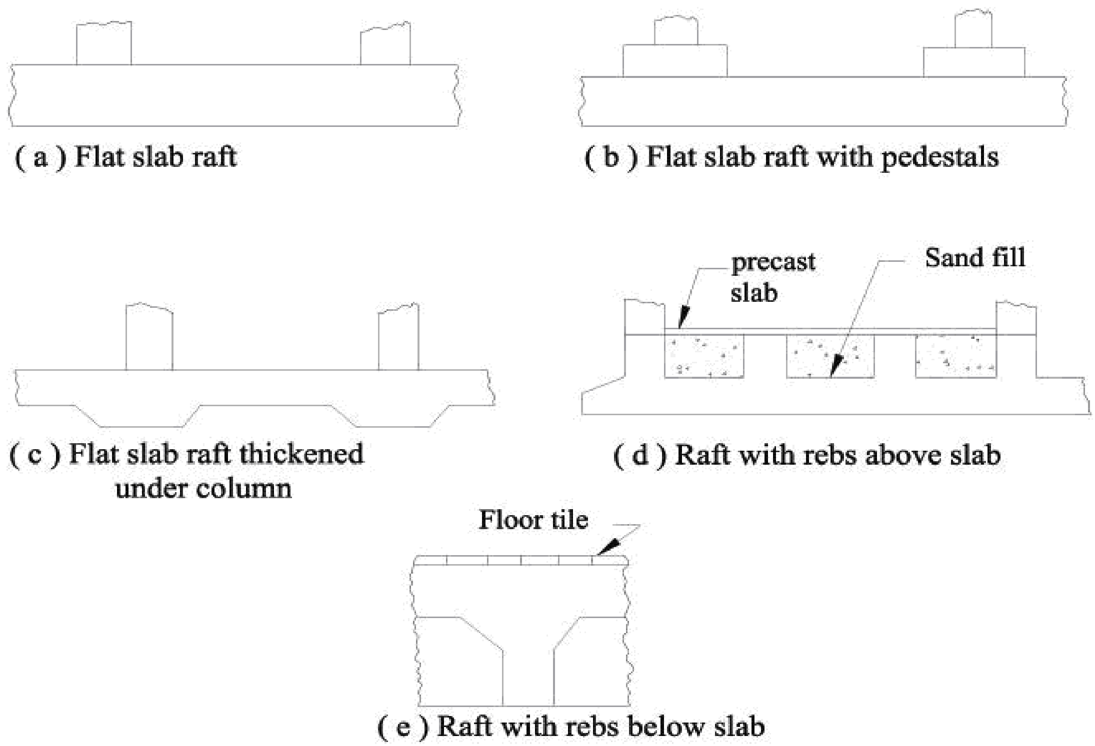



*Rectangular combined footing*

**C) Strap or Cantilever Footings:** A strap footing maybe used where the distance between columns is so great that a combined or trapezoid footing becomes quite narrow, with resulting high bending moments. A strap footing consists in two column footings connected by a member termed a strap, beam, or cantilever which transmits the moment from the exterior footing.



**D) Mat or Raft Foundations:** The raft foundation is continuous footing that covers the entire area beneath a structure and supports all the walls and columns. The term mat is also used for foundation of this type. It is used generally on soil of low bearing capacity and where the area covered by spread footings is more than half the area covered by the structure. Raft foundations can be constructed near the ground surface, or at the bottom of basements. In high-rise buildings, mat-slab foundations can be several meters thick, with extensive reinforcing to ensure relatively uniform load transfer.




Types of raft foundations .

**E) Retaining Wall:** Is a structure that holds back soil or rock from a building, structure or area. Retaining walls prevent down slope movement or erosion and provide support for vertical or near-vertical grade changes. Cofferdams and bulkheads, structures that hold back water, are sometimes also considered retaining walls. Retaining walls are generally made of masonry, stone, brick, concrete, vinyl, steel or timber. Once popular as an inexpensive retaining material, railroad ties have fallen out of favor due to environmental concerns.

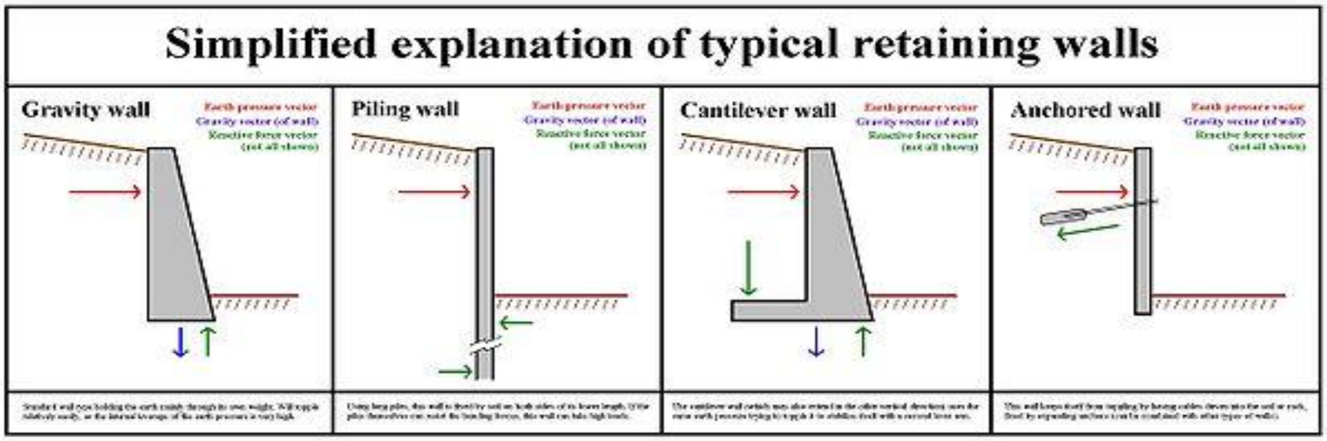
The most important consideration in proper design and installation of retaining walls is that the retained material is attempting to move forward and down slope due to gravity. This creates lateral earth pressure behind the wall which depends on the angle of internal friction (phi) and the cohesive strength (c) of the retained material, as well as the direction and magnitude of movement the retaining structure undergoes.

Lateral earth pressures are typically smallest at the top of the wall and increase toward the bottom. Earth pressures will push the wall forward or overturn it if not properly addressed. Also, any groundwater behind the wall that is not dissipated by a drainage system causes an additional horizontal hydrostatic pressure on the wall.[1]

As an example, the International Building Code requires retaining walls to be designed to ensure stability against overturning, sliding, excessive foundation pressure and water uplift; and that they be designed for a safety factor of 1.5 against lateral sliding and overturning.

**TYPES OF RETAINING WALL**

1) Gravity
2) Sheet piling
3) Cantilevered
4) Anchored



# CHAPTER THREE

## *Geotechnical Investigation*

## 3.1 INTRODUCTION

### ***3.1.1 GENERAL***

This Chapter presents the outcome of the site investigation carried out for the proposed construction site of the New Muskat School in Kufr Ra'i (Omani Aid).

### ***3.1.2 PROJECT DESCRIPTION***

The project consists of the construction of a two story building with plan construction area of about 1000 square meters.

### ***3.1.3 PURPOSE AND SCOPE***

Investigation of the underground conditions at a site is prerequisite to the economical design of the substructure elements. It is also necessary to obtain sufficient information for feasibility and economic studies for any project.

It should be also noted that the scarcity of construction sites in the urban areas of the West Bank with considerable urban renewal and the accompanying backfill, often with no quality control, affects the underground conditions and results significant variation within a few meters in any direction.

For this particular project, and due to the known weak soil conditions of the area and the type of proposed structures, the site investigation becomes of special importance to obtain sufficient information about the geotechnical parameters of the ground.

In general, **the purpose of this site investigation** was to provide the following:

1- Information to determine the type of foundation required (shallow or deep).
2- Information to allow the geotechnical consultant to make a recommendation on the allowable bearing capacity of the soil.
3- Sufficient data/ laboratory tests to make settlement and swelling predictions.
4- Location of the groundwater level.
5- Information so that the identification and solution of excavation problems can be made.
6- Information regarding permeability and compaction properties of the encountered materials.

## 3.2 SITE CONDITIONS

### ***3.2.1 DESCRIPTION***

The project site lies in **Kufr Rai** and is bordered by a 5m road from the east as shown on the attached plan-layout.

No high voltage, electrical or telephone poles, sewer or water pipes were observed within the depths of the drilled boreholes.

A general site plan showing the locations of boreholes is presented in Fig.1.

### ***3.2.2 SUBSURFACE CONDITIONS :***

The studied area, after its preparation for construction, is approximately flat. The general soil formation within the depths of the borings consists mostly of light brown, plastic silty clay with dry to medium moist condition followed by medium hard formation of creamy marlstone.

The drilled boreholes for this study were pre-determined by the Consultant (Khatib & Alami). They reflect the described above general conditions and are enough, in our opinion, to represent the whole area of the proposed construction. They are discussed in more detail in subsequent sections of this Chapter.

### ***3.2.3 GROUNDWATER AND CAVITATION***

No fixed ground water table was observed. No cavities or other kind of weaknesses were encountered within the depths of the drilled boreholes.

### ***3.2.4 SUMMARY OF RESULTS:***

| **Layer** | **Type of soil** | **Depth (m)** | | | |
|---|---|---|---|---|---|
| | | **BH1** | **BH2** | **BH3** | **BH4** |
| 1 | Dry deposits of silt clay | 0-2 | 0-1.5 | 0-3.5 | 0-3.5 |
| 2 | Soft and moist formation of creamy marlstone formation | 2-4.5 | 1.5-4.5 | 3.5-6 | 3.5-6 |
| 3 | Dry, medium hard formation of creamy marlstone formation | 4.5-10 | 4.5-10 | - | - |
| End of boring | | 10 | 10 | 6 | 6 |

*Table (3:1) boreholes summary*

| Borehole No. | Depth (m) | Moisture Content (%) | % passing sieve #200 | Liquid limit (%) | Plasticity Index (PI) |
|---|---|---|---|---|---|
| 1 | 0-2 | 5.8 | 65.4 | 49.9 | 24.1 |
| | 2-4.5 | 9.5 | 68.7 | 51.2 | 25.1 |
| | 4.5-10 | 7.1 | 38.4 | NP | NP |
| 2 | 0-1.5 | 5.6 | 66.8 | 48.4 | 22.4 |
| | 1.5-4.5 | 21.3 | 76.5 | 52.9 | 26.4 |
| | 4.5-10 | 8 | 37.4 | NP | NP |
| 3 | 0-3.5 | 5.4 | 69.8 | 48.8 | 23.0 |
| | 3.5-6 | 6.3 | 34.5 | NP | NP |
| 4 | 0-3.5 | 5.5 | 72.3 | 51.0 | 23.4 |
| | 3.5-6 | 6.1 | 35.1 | NP | NP |

*Table (3:2) Summary of test results*

| Borehole No. | Cohesion ($Kg/cm^2$) | Angle of internal Friction $\Phi(^o)$ |
|---|---|---|
| 1 | 43 | 15 |
| 2 | 45 | 15 |
| 3 | 45 | 15 |
| 4 | 46 | 14 |

*Table (3:3) Summary of shear test results*

## 3.3 BEARING CAPACITY ANALYSIS

***3.3.1 FOR THE UPPER SILTY CALY LAYER (the school building area):***

Using the shear test parameters of cohesion and angle of internal friction and the soil density, the following well known Terzaghi equation with correction terms suggested by Schultze can be used to calculate the bearing capacity of rectangular foundation of any sides ratio B:L

$$q_{ult} = (1 + 0.3\ B/L)\ CN_c + \gamma_o\ DN_q + (1 - 0.2\ B/L)\ (\gamma_1 B/2)\ N_\gamma$$

where:

$\gamma_o$ - Unit weight of soil above foundation level in $KN/m^3$.

$\gamma_1$ - Unit weight of soil below foundation level in $KN/m^3$.

C,Ø - Strength parameters of the soil below foundation level in $KN/m^2$ and degrees respectively.

B - Width of foundation in (m).

L - Length of foundation in (m).

$N_C$, $N_q$, $N_\gamma$ - Bearing capacity coefficients dependent on the angle of internal friction

of the soil below foundation level (dimensionless).

D - Depth of foundation (m).

### ***3.3.2 CALCULATION FOR THE ASSUMED SINGLE FOOTING :***

***Considering:***

B = 2.5 m

L = 2.5 m

D = 2.0 m

$\gamma_o$= 17 KN/m³

$\gamma_1$ = 17 KN/m³

C = 42 KN/m² (average)

Ø = 14 º

and reducing C and Ø according to Terzaghi to

$C'$ = 2/3 C = 28.1 KN/$m^2$

tan $Ø'$ = 2/3 tan Ø , $Ø'$ = 10º

The bearing capacity was computed by a special computer program using both **Terzaghi** and **Vesic** methods. The sheet with computation is attached. Based on the calculations, a bearing capacity range of **1.53-1.78 Kg/cm$^2$** was obtained at a depth of 2m from the existing ground for shallow isolated footings on the top layer of silty clay.

### ***3.3.3 FOR THE MARLESTONE LAYER (average depth=4.0-4.5m from the surface) (the sanitary unit's area):***

According to the known codes of engineering practice, the bearing capacity of rocks is taken as a percentage of the unconfined compressive strength of rock core samples tested in accordance with ASTM D-2938.

Following the Jordanian Code for Foundations and Retaining Walls (Amman- 1992) [§3/7/1-2], the mentioned percentage is 5% for rocks with RQD ≤ 75 % and the bearing capacity should not exceed 10 $Kg/cm^2$.

Taking the lowest compressive strength value of rock core specimens from the table (2) and applying the percentage of 5%, the strength will be:

**$Q_{all}$ = 5 % x 73 = 3.7 $Kg/cm^2$**

## 3.4 SELECTION OF FOUNDATION TYPE

According to the nature and characteristics of the materials encountered in the drilled boreholes (top layer of silty clay followed by medium hard marlstone), we recommend the following solutions for foundations:

### ***3.4.1 UNDER THE SCHOOL BUILDING:***

It is recommended to excavate the ground to a depth of at least 2.0m from the existing surface, compact the reached ground to at least 95% of maximum dry density (modified Proctor), and then:

- Spread and properly compact a granular material (base course) of at least 60cm thickness over the excavated level. The replacement can consist of one rockfill layer and two base course layers each compacted to at least 98% of maximum dry density (modified Proctor).
- Use of isolated and/or continuous footings (with tie beams) on the compacted surface. **The recommended bearing capacity is 2.0 $kg/cm^2$ over the compacted granular fill.**

***The other alternative for the foundations of the school building is to excavate the top layer of silty clay and lay foundations at any depth on or below -3.5m from the existing ground (on the medium hard marlstone) with bearing capacity of not more than 3.7 kg/cm$^2$.***

#### ***3.4.2 UNDER THE SANITARY UNITS:***

It is recommended to excavate the ground and penetrate the top silty clay layer reaching the layer of medium hard marlstone and use isolated footings.

**The recommended bearing capacity within this layer is 3.5 kg/cm$^2$.**

## 3.5 ENGINEERING RECOMMENDATIONS

As a result of field and laboratory activities carried out and the analysis of the available data and test results, the following engineering recommendations can be made:

#### ***3.5.1 TYPE AND DEPTH OF FOUNDATIONS***

Owing to the encountered subsurface conditions, which are discussed in this report, it is recommended to consider foundations as described in 3.4 above.

#### ***3.5.2 MATERIALS FOR BACKFILLING – COMPACTION CRITERIA***

The top materials encountered in the drilled boreholes are not satisfactory for using in backfilling purposes due to the high plasticity. In general, materials for the backfilling should be granular, not containing rocks or lumps over 15 cm in greatest dimension, free from organic matter, with plasticity index (PI) not more than 20. The backfill material should be laid in lifts not exceeding 25 cm in loose thickness and compacted to at least 95 percent of the maximum dry density at optimum moisture content as determined by modified compaction tests (Proctor) (ASTM D-1557).

### ***3.5.3 SLAB ON GRADE***

Slab on grade should be laid on properly compacted granular materials (as minimum, a 30cm layer of base course should be applied). It is recommended in this regard to compact the natural ground prior to the application of fill layers and compact the upper base course to at least 98% of maximum dry density at optimum moisture content according to modified Proctor test (AASHTO T-180).

### ***3.5.4 DRAINAGE OF THE SITE***

It is recommended to design an effective rainwater drainage system to get rid of the consequences of the rainwater percolation into the layers. The site should be graded so as to direct rainwater and water away from all planned structures.

### ***3.5.5 SWELLING AND SHRINKAGE:***

- The test results show that the silty clay materials encountered at the site has a low to medium potential for expansiveness. This mean that this material could swell and shrink with the increase and decrease of moisture content, respectively. Swelling and shrinkage of the subsurface soil could cause development of small cracks in the structure walls and ground floor slab. The dimensions of these cracks increase with the repetitive variation of moisture content of the soil, which could lead to serious problems. Therefore, the ground moisture changes should be minimized.
- The various environmental factors which can locally influence the ground volume change can be grouped broadly as those which put water into the ground and those which take water out of the ground.

The former will usually consist of broken water pipes, leaking taps, leaking septic tanks, leaking drains, rain water pipes discharging into the ground and irrigation of gardens; the second group usually consists only of trees, but could also include furnaces which heat the ground and dry it out.

***The following simple features of design to minimize moisture accumulation beneath the school building under consideration are recommended:***

a- sloping the ground down and away from the building so that any water run-off flows away from it,

b- Ensuring that septic tanks are reinforced to minimize cracking and have adequate flexible water-proofing,

c- Ensuring that water supply pipes and sewer pipes are sufficiently flexible, or are flexibly connected, to accommodate movements,

d- Ducting all rainwater falling onto roofs well away from the foundations.

e- Ensuring that large trees are kept away from the building. To remove all possible tree root influence the distance x between tree and building should not be less than the tree height, h. However, no significant movements are likely provided x is greater than h/2.

f- Ensuring that the perimeter of the building is paved. This helps to reduce the seasonal moisture change. The laying of an impermeable cover around the building, say 2 m wide, also helps to reduce long term differential heave.

### ***3.5.6 SEISMIC CONSIDERATIONS***

As far as the seismic activity in the region has not witnessed any serious earthquakes in the last 70 years, the last series of earthquakes since February 2004 in Palestine and neighboring Middle East countries and their serious consequences made it necessary to consider a seismic precautive factor in the design of the project structures. Referring to the Unified Building Code

Research in Jordan, the area can be considered within Zone B, which corresponds to an intensity of VI to VIII according to the Mercalli Scale (4-6 Richter scale respectively).

According to the seismic zoning chart prepared by An-Najah National University for Palestine, the seismic gravity acceleration factor for area (Zone III)

$z = 0.18\text{-}0.20\ g$ , where g – gravity acceleration.

# CHAPTER FOUR

## *Structural Design*

## 4.1 STRUCTURAL ANALYSIS

Columns loads calculated using hand calculations (Tributary area), where the structure subjected to the following loads:

1) Dead Load (own weight).
2) Super imposed dead load $=250kg/m^2$.
3) Live loads $=400kg/m^2$.

Using ACI code, the ultimate loads calculated considering load combination:

$P_u = 1.2Dead + 1.6Live.$

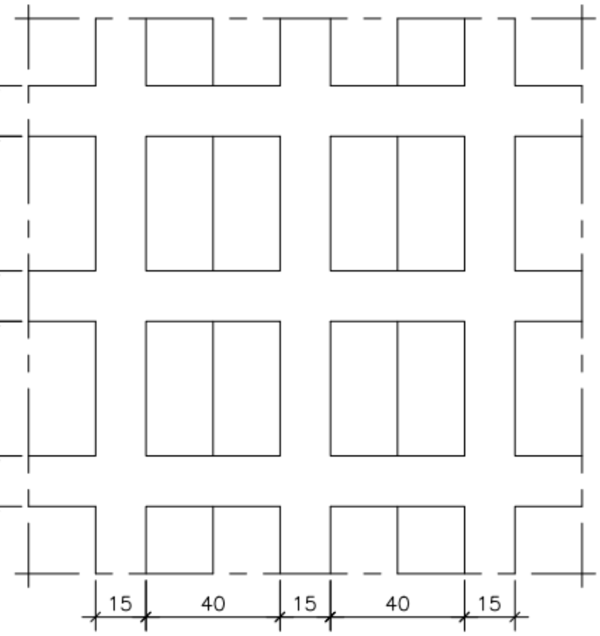
Using in this project the following materials characteristics:

$f'_c = 240kg/cm^2$ (B 300)

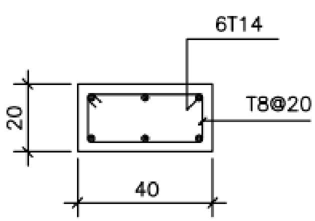
Where: f'c is the compressive strength of concrete

$f_y = 4200\ kg/cm^2$

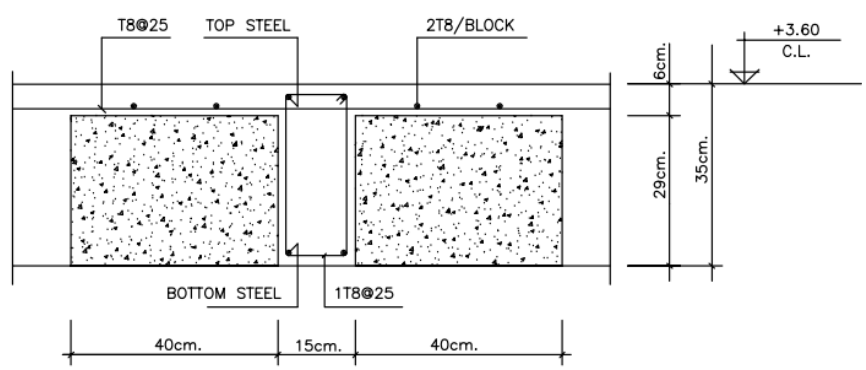
Where: fy is the yield strength of steel




TYPICAL TWO WAY RIBBED SLAB PLAN




DETAIL-1




TYPICAL RIB SECTION

**From the figures shown above and sections details:-**

Number of **blocks** =18

<u>*Volume of blocks*</u> = 18*(0.4*0.2*0.29) = **0.4176** $m^3$

<u>*Volume of the concrete*</u> = (1.65*1.65*0.35) = **0.953 - 0.4176** $m^3$

= **0.5354**

Weight of **concrete** = 0.5354*2.5=**1.3385** ton

Wight of **blocks** =0.4176 * 1.2 **= 0.50** ton

**<u>So total weight of the slab section</u>**

= 1.3385 + 0.50 = **1.839** ton

**So the load per unit area** = 1.839/ (1.65 * 1.65) = **0.6753** ton\m²

***<u>Consider super imposed dead load + partial walls</u>*** = 0.15 + 0.1 = **0.25** ton\m²

So the ***<u>total dead</u>*** load = 0.6753 +0.25 =**0.925** ton\m²

The ***<u>live load</u>*** considered to be **0.4** ton\m²

**<u>So the Ultimate Load</u>**

((1.2* 0.925) + (1.6*0.4))= **1.75**

<u>Ultimate factor</u> = 1.75/ (0.925+0.4) = **1.32**

## SUMMARY OF LOADS ACTING ON THE FOOTINGS

This table shows the dimensions and the load acting by column and walls on the footings, where the shaded cells represent shear walls or columns attached with walls in the same footings (eccentric).

| Col No. | Dimensions (cm) | Service Load | Col No. | Dimensions (cm) | Service Load | Col No. | Dimensions (cm) | Service Load(tn) |
|---|---|---|---|---|---|---|---|---|
| C1 | 20*80 | 66.41 | C22 | 20*60 | 49.50 | C43(w) | 20\m | 50\m |
| C2 | 20*80 | 100.49 | C23 | 20*80 | 56.10 | C44(w) | 20\m | 50\m |
| C3 | 20*80 | 100.49 | C24 | 20*60 | 49.50 | C45(w) | 20\m | 50 \m |
| C4 | 20*80 | 100.49 | C25 | 20*80 | 56.10 | C46(w) | 20\m | 50\m |
| C5 | 20*80 | 100.49 | C26 | 20*60 | 49.50 | C47 | 20*60 | 100.80 |
| C6 | 20*80 | 100.49 | C27 | 20*80 | 56.10 | C48 | 60*60 | 367.50 |
| C7 | 20*80 | 100.49 | C28 | 20*60 | 49.50 | C49 | 20*60 | 76.80 |
| C8 | 20*80 | 100.49 | C29 | 20*80 | 56.10 | C50 | 20*60 | 32.40 |
| C9 | 20*80 | 100.49 | C30 | 20*60 | 35.10 | C51 | 60*60 | 319.20 |
| C10 | 20*80 | 91.35 | C31 | 60*60 | 108.00 | C52 | 20*60 | 73.50 |
| C11 | 20*80 | 56.10 | C32 | 60*60 | 135.00 | C53 | 20*80 | 91.35 |
| C12 | 20*100 | 117.00 | C33 | 20*70 | 138.00 | C54 | 20*60 | 57.60 |
| C13 | 20*120 | 174.00 | C34 | 20*80 | 102.30 | C55 | 20*80 | 47.85 |
| C14 | 20*120 | 174.00 | C35 | 85*85 | 270.00 | C56 | 20*80 | 102.30 |
| C15 | 20*120 | 174.00 | C36 | 20*90 | 132.00 | C57 | 20*80 | 73.50 |
| C16 | 20*120 | 174.00 | C37 | 20*120 | 174.00 | C58 | 20*85 | 97.65 |
| C17 | 20*120 | 174.00 | C38 | 20*60 | 85.50 | C59 | 20*80 | 56.10 |
| C18 | 20*120 | 174.00 | C39 | 20*40 | 26.25 | C60(w) | 20\m | 50\m |
| C19 | 20*120 | 174.00 | C40 | 20*50 | 49.50 | C61 | 20*80 | 134.64 |
| C20 | 20*120 | 174.00 | C41 | 20*90 | 122.40 | | | |
| C21 | 20*120 | 174.00 | C42 | 20*60 | 57.60 | | | |

*Table (4:1) service loads*

## 4.2 DESIGNS OF SINGLE FOOTINGS

### ***4.2.1) DETERMINE AREA OF FOOTING***

Soil report recommended using single footing, so the initial design based on single footing, and adjustments provided when necessary.

The allowable bearing capacity of the soil equal to **3.5kg/cm$^2$ (35ton/m$^2$)** which will be use to determine the required footings' area, and to complete other calculations.

Required area = Service load/$q_u$ ………..… equu(4:1)

Required area = area which allowable for settlement ..…………equ(4:2)

Net $q_u$ = $q_u$ - $\gamma$* $D_f$ ……..……..equ(4:3)

***Where:***

$\gamma$: soil unit weight

$D_f$: Depth of footing

Net $q_u$ = 26.5 ton\m$^2$

Assumed depth =1.5m

There for required area and initial dimensions will be shown next table of ***non eccentric single*** footing or wall attach (non shaded cells):

| Footing | service load | Required area | B | L | provide area |
|---|---|---|---|---|---|
| F1 | 100.49 | 2.85 | 1.5 | 2 | 3 |
| F2 | 91.35 | 2.6 | 1.5 | 2 | 3 |
| F3 | 56.1 | 1.63 | 1 | 2 | 2 |
| F4 | 117 | 3.35 | 1.5 | 2.4 | 3.6 |
| F5 | 174 | 4.92 | 1.8 | 2.8 | 5.04 |
| F6 | 49.5 | 1.401 | 1 | 1.5 | 1.5 |
| F7 | 56.1 | 1.63 | 1 | 2 | 2 |
| F8 | 35.10 | 1.09 | 1 | 1.5 | 1.5 |
| F9 | 108.00 | 3.18 | 2 | 2 | 4 |
| F10 | 135.00 | 3.89 | 2 | 2 | 4 |
| F11 | 138.00 | 3.86 | 2 | 2.4 | 4.8 |
| F12 | 102.30 | 2.9 | 1.5 | 2 | 3 |
| F13 | 270.00 | 7.75 | 3 | 3 | 9 |
| F14 | 132.00 | 3.81 | 1.5 | 2.4 | 3.6 |
| F15 | 174.00 | 2.43 | 1.5 | 2 | 3 |
| F16 | 122.40 | 3.64 | 1.5 | 2.4 | 3.6 |
| F17 | 57.60 | 1.71 | 1 | 2 | 2 |
| F18 | 100.80 | 2.89 | 1.5 | 2 | 3 |
| F19 | 367.50 | 10.83 | 3.3 | 3.3 | 10.89 |
| F20 | 76.80 | 2.16 | 1.5 | 1.5 | 2.25 |
| F21 | 32.40 | .88 | 1 | 1 | 1 |
| F22 | 319.20 | 9.24 | 3 | 3 | 9 |
| F23 | 73.50 | 1.95 | 1.5 | 1.5 | 2.25 |
| F24 | 91.35 | 2.61 | 1 | 2.4 | 2.4 |
| F25 | 57.60 | 1.75 | 1.2 | 1.5 | 2.25 |
| F26 | 47.85 | 1.42 | 1 | 1.5 | 1.5 |
| F27 | 102.30 | 2.9 | 1.5 | 2 | 3 |
| F28 | 134.64 | 3.81 | 1.5 | 2.4 | 3.6 |

*Table (4:2) Required area & Initial dimensions*

## 4.2.2 THICKNESSES OF FOOTINGS

Depth of footing will be controlled by wide beam shear (one way action) and punching shear (two way action).

**Wide beam shear:**

Shear cracks are form at distance “d” from the face of column, and extend to the compression zone, the compression zone will be fails due to combination of compression and shear stress.

Max shear will be occurs at distance “d” from face of the column, where "d” is the effective depth of footing and the shear represented by this formula:

$V_u = q_u * L$ ………....equ(4 :4)

Where :

$V_u$ : max shear.

$q_u$= ultimate load /area of footing. ....………..equ(4:5)

Ultimate capacity by concrete given by this formula:

$\Phi *V_c = 0.75 * 0.53\sqrt{f'_c} *B*d *10$ …...……equ(4:6)

Where :

Φ: Safety reduction factor = 0.75 for shear.

$f'_c$ : Compressive strength of concrete (cylinder)(Kg\cm$^2$).

B: Footing width.

D: Footing depth.

$\Phi *V_c > V_u$ ………..….equ(4:7)

**Punching shear:**

Formation of inclined cracks around the perimeter of the concentrated load may cause failure of footing.

Max. shear will be given in this formula :

$V_u = q_u (B*L – ( (b+d)*(h+d) )$ ………….equ(4 : 8)

Where**:**

$V_u$: Max punching shear.

B: Footing width.

L: Footing length.

b:Column width.

h:Column depth.

d:Footing depth.

Concrete shear strength is the smallest of:

$\Phi*Vc = \Phi/6\ (1+(2/\beta_c)\sqrt{f'_c}*b_o*d$ ………….equ(4:9

$\Phi*Vc = \Phi/12\ (\ 2+(\alpha_s/(b_o/d))\ )\sqrt{f'_c}*b_o*d$ ………….equ(4:10

$\Phi*Vc = \Phi\ (0.33)\sqrt{f'_c}*b_o*d$ ………….equ(4:11

Units in N & mm

***Where:***

$b_0$ : Perimeter of the critical section taken at " **d/2** " from the face of column.

$\beta_c$: Ratio of longer length to the shorter length of the loaded member (column).

$\alpha_s$ : = 40 (Interior footing ).

= 30 (Edge footing ).

= 20 (Corner footing ).

**Note :**

**For** $\beta_c$ <=2 & $(b_0/d)$ <=20

Critical :

$\Phi*Vc = \Phi\,(0.34)\sqrt{f'_c}*b_o*d$ ...…………….equ(4:12

**Thickness of F1**

$F'_c$ = 240 Kg /cm$^2$ (B300)

$F_y$ = 4200 Kg /cm$^2$

The design based on 1 meter

Service load =100.49 ton , Ultimate load =132.64 ton

= 1.4*2.1 = 2.94 m$^2$

**Wide beam shear:**

$q_u$= 132.64/2.94 = 45.115 ton /m$^2$

$V_u$ = 45.115 * (.65-d)

$\Phi *V_c = 0.75 * 0.53\sqrt{240} *1.0 * d *10$

$\Phi *V_c >= V_u$ from that d=27 cm

**Punching shear:**

*Punching shear:

$P_{u(punching)} \leq \varphi V_{cp}$

$P_{u(punching)} = P_{u(column)} - q_{ult}((b+d)(h+d))$

$=132.64-45.115((.2+.27)(0.8+.27))$

$=109.95ton$

$V_{cp}=1.06*\sqrt{fc}*A_0*d$

Where,

$A_0$: parmeter of the critical section (at distance (d/2) from the face of column)

$\varphi V_{cp}=0.75*1.06*\sqrt{240}*10*(2(.2+.27)+2(0.8+.27))*.27$

$=102.42ton<109.95ton$ (**Not Ok**)

Try a new depth (d=.32)

$P_{u(punching)}=106.36ton$

$\varphi V_{cp}=0.75*1.06*\sqrt{240}*10*(2(.2+0.32)+2(0.8+0.32))*0.32$

$=129.27\ ton>106.36$ (**Ok**)

Take cover=8cm

So that, total depth of footing=h=40cm

**d =32cm** **cover = 8 cm**

**Thickness of F2:**

Service load =91.35 ton , Ultimate load =120.582ton

Area = 1.35 * 2 =2.7 $m^2$

**Wide beam shear:**

$q_u$=120.582/2.7 = 44.66 ton /$m^2$

$V_u$ = 44.66 * (.6-d)

$\Phi *V_c = 0.75 * 0.53\sqrt{240} *1.0 * d *10$

$\Phi *V_c >= V_u$ , from that d=.27 cm

**Punching shear:**

$P_{u(punching)} \leq \varphi V_{cp}$

$P_{u(punching)} = P_{u(column)} - q_{ult}((b+d)(h+d))$

=120.58-44.66((.2+.27)(0.8+.27))

=99.96ton

$V_{cp}=1.06*\sqrt{fc} *A_0*d$

Where,

$A_0$: parmeter of the critical section (at distance (**d/2**) from the face of column)

$\varphi V_{cp}=0.75*1.06*\sqrt{240} *10*(2(.2+.27)+2(0.8+.27))*.27$

=102.42ton>99.96ton (**Ok**)

Take cover=8cm

So that, total depth of footing=h=35cm

**d =27cm** **cover= 8 cm**

## *4.2.3 Steel Reinforcement (Flexural):*

Isolated footing represented as cantilever, so the max moment occurs at the face of the column:

$$M_u = q_u * L^2 / 2 \qquad \text{............equ(4:13)}$$

$$\rho = \frac{0.85\, f'_c}{f_y}\left(1-\left(\sqrt{\left(1-\frac{2.61 * M_u}{f'_c * b * d^2}\right)}\right)\right) \qquad \text{...........equ(4:14)}$$

***where :***

ρ: Ratio of steel in the section.

$M_u$ : Ultimate moment.

$F'_c$ : Concrete compressive strength.

$F_y$ : Yield strength of steel.

Minimum ratio of steel is given by:

$A_{st-P}$

$_{min} =$ 0.0018 Ag

***Where:***

Ag: gross section area

**Development length:**

It is important to check minimum length that the steel must be extended behind to the point that is not required.

$$L_d = \frac{0.73\, f_y\, d_b}{\sqrt{f'_c}} \qquad \text{................equ(4:15}$$

Units: N & mm

***Where:***

$L_D$: development length in tension

$d_b$: diameter of bar

$$L_{dc} = \frac{0.24\, f_y\, d_b}{\sqrt{f'_c}} \qquad \text{……..……….equ(4:16}$$

Units: N & mm

***Where:***

$L_{dc}$: development length in compression

**Steel reinforcement of F1:**

Isolated footing represented as cantilever, so the max moment occurs at the face of the column:

Ultimate moment at the face of the column ($M_{ult}$) $=q_{ult}*[(\frac{L-h}{2})^2/2$ $=45.115*[(\frac{2.1-0.8}{2})^2/2] = 9.53$ ton.m

$M_n = \frac{Mu}{\Phi}$, where, $\Phi$=0.9

$M_n$ =10.58ton.m

$M_n = R_n bd^2$

10.58ton. m=$R_n$x(100cm)x(32cm)$^2$

$R_n$ =10.33kg/cm$^2$

$$\rho = \frac{1}{m}(1-\sqrt{1-\frac{2mRn}{fy}})$$

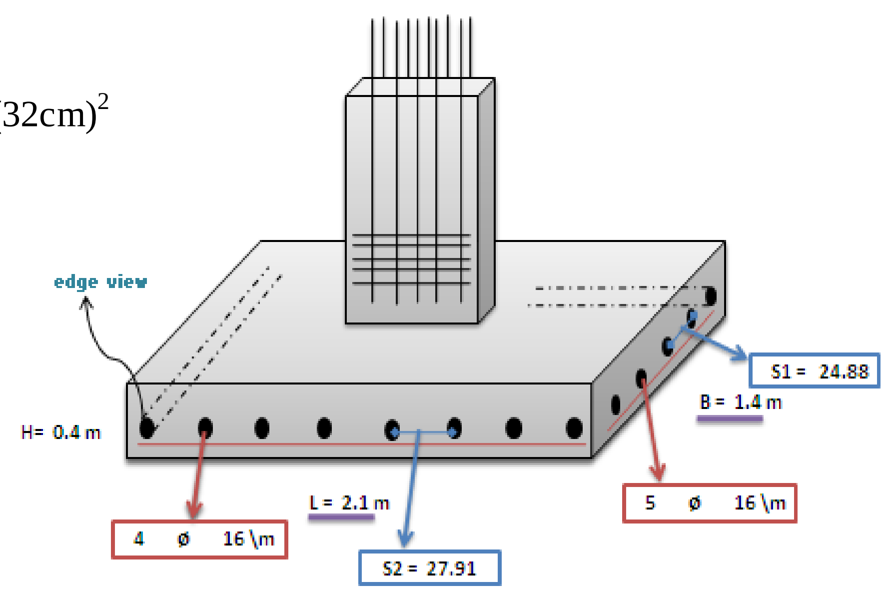
***Where***:-

ρ: Steel ratio.

$m= \frac{fy}{0.85\, f'c}$ =20.59

so that ρ=.0025

$A_{s.=}$ 8.08




illustrative view (not scaled)

$A_{s.min}$=0.0018*100*60= 7.2$cm^2$

$A_{s.}$used=8.08$cm^2$

**<u>Use 5 Φ 16</u> (1 Φ16 /25cm c/c)** **provide 8.08 $cm^2$**

**In the other direction:**

Ultimate moment at the face of the column

$(M_{ult})=q_{ult}*[(\frac{B-t}{2})^2/2]=45.115*[(\frac{1.4-0.2}{2})^2/2]=8.12$ ton.m

$M_n=\frac{Mu}{\Phi}$, where, Φ=0.9

$M_n$ =9.02ton.m

$M_n=R_n bd^2$

9.02x100 ton.cm=$R_n$x(100cm)x(32cm)$^2$

$R_n$ =8.8kg/$cm^2$

$\rho=\frac{1}{m}(1-\sqrt{1-\frac{2mRn}{fy}})$

***where:***

ρ : Steel ratio.

m= $\frac{fy}{0.85\,f'c}$ =20.59

so that ρ= .0021

$A_s$=6.85

$A_{s.min}$=0.0018*100*40= 7.2$cm^2$

**<u>Use 4 Φ 16</u> (1 Φ16 /28cm c/c)** **provide 7.2 $cm^2$**

**Check Development length:**

$L_d = \dfrac{0.73 * 420 * 16}{\sqrt{24}} = 1001.3$ mm ( for Φ16 mm in tension )

Provided = 1400 mm in short direction and 2100 mm in long direction **ok**

**Steel reinforcement of F2:**

Isolated footing represented as cantilever, so the max moment occurs at the face of the column:

Ultimate moment at the face of the column ($M_{ult}$)=$q_{ult}$*[$(\frac{L-h}{2})^2/2$ =44.66 *[$(\frac{2-0.8}{2})^2/2$]= 8.04 ton.m

$M_n = \frac{Mu}{\Phi}$, where, Φ=0.9

$M_n$ =8.933ton.m

$M_n = R_n bd^2$

8.933ton. m=$R_n$x(100cm)x(27cm)$^2$

$R_n$ =12.25kg/cm$^2$

$\rho = \frac{1}{m}(1-\sqrt{1-\frac{2mRn}{fy}})$

ρ=.0030

$A_s$=8.11

$A_{s.min}$=0.0018*100*35= 6.3cm$^2$

**<u>Use 5 Φ 16</u> (1 Φ16 /25cm c/c)** **provide 8.11 cm$^2$**

**In the other direction:**

Ultimate moment at the face of the column $(M_{ult})=q_{ult}*[(\frac{B-t}{2})^2/2]=44.66 *[(\frac{1.35-0.2}{2})^2/2]= 7.38$ton.m

$M_n = \frac{Mu}{\Phi}$, where, $\Phi=0.9$

$M_n = 8.2$ton.m

$M_n = R_n bd^2$

$8.2x100$ ton.cm$=R_n x(100cm)x(27cm)^2$

$R_n = 11.24kg/cm^2$

$\rho = \frac{1}{m}(1-\sqrt{1-\frac{2mRn}{fy}})$

$\rho=.0028$

$A_s=7.43$

**<u>Use 4 Φ 16</u> (1 Φ16 /27cm c/c)** **provide 7.43 $cm^2$**

**Check Development length:**

$L_d = \frac{0.73 * 420 * 16}{\sqrt{24}} = 1001.35$ mm ( for Φ16 mm in tension )

Provided = 1350 in short direction and 2000 in long direction **(ok)**

**Note: For anchoring, a hook can be provided**

## SUMMARY OF DIMENSIONS

| Footing | B (m) | L (m) | d (m) | h (m) |
|---|---|---|---|---|
| F1 | 1.4 | 2.1 | .32 | .4 |
| F2 | 1.35 | 2 | .27 | .35 |
| F3 | 1 | 1.7 | .22 | .3 |
| F4 | 1.5 | 2.3 | .32 | .4 |
| F5 | 1.8 | 2.8 | .37 | .45 |
| F6 | 1 | 1.45 | .22 | .3 |
| F7 | 1 | 1.7 | .22 | .3 |
| F8 | 1 | 1.2 | .17 | .25 |
| F9 | 1.8 | 1.8 | .27 | .35 |
| F10 | 2 | 2 | .32 | .4 |
| F11 | 1.8 | 2.2 | .42 | .5 |
| F12 | 1.4 | 2.1 | .32 | .4 |
| F13 | 2.8 | 2.8 | .47 | .55 |
| F14 | 1.65 | 2.35 | .37 | .45 |
| F15 | 1.15 | 2.15 | .37 | .45 |
| F16 | 1.6 | 2.3 | .32 | .4 |
| F17 | 1.1 | 1.6 | .22 | .3 |
| F18 | 1.5 | 1.95 | .32 | .4 |
| F19 | 3.3 | 3.3 | .67 | .75 |
| F20 | 1.3 | 1.7 | .27 | .35 |
| F21 | 0.8 | 1.2 | .17 | .25 |
| F22 | 3 | 3.1 | .62 | .7 |
| F23 | 1.25 | 1.7 | .27 | .35 |
| F24 | 1.35 | 2 | .27 | .35 |
| F25 | 1.2 | 1.5 | .22 | .3 |
| F26 | 1 | 1.5 | .17 | .25 |
| F27 | 1.45 | 2.1 | .32 | .4 |
| F28 | 1.7 | 2.3 | .37 | .45 |

*Table (4:6) summary of dimensions*

**Note:***Provide 10 cm plain concrete under footings.

The following table shows summary of ***single footing reinforcement***:

| Footing | Reinforcement in short direction/cm/m | Reinforcement in long direction/cm/m |
|---|---|---|
| F1 | 1ø16/25 | 1ø16/27 |
| F2 | 1ø16/25 | 1ø16/27 |
| F3 | 1ø14/28 | 1ø14/28 |
| F4 | 1ø16/28 | 1ø16/18 |
| F5 | 1ø16/ 19 | 1ø16/19 |
| F6 | 1ø14/28 | 1ø14/28 |
| F7 | 1ø14/ 28 | 1ø14/28 |
| F8 | 1ø12/ 25 | 1ø12/23 |
| F9 | 1ø16/25 | 1ø14/19 |
| F10 | 1ø16/ 22 | 1ø16/22 |
| F11 | 1ø16/ 22 | 1ø16/21 |
| F12 | 1ø16/ 25 | 1ø16/28 |
| F13 | 1ø16/ 16 | 1ø16/16 |
| F14 | 1ø16/ 23 | 1ø16/23 |
| F15 | 1ø16/ 19 | 1ø16/19 |
| F16 | 1ø16 / 22 | 1ø16/22 |
| F17 | 1ø14/32 | 1ø14/29 |
| F18 | 1ø14/ 18 | 1ø14/19 |
| F19 | 1ø25/ 30 | 1ø25/30 |
| F20 | 1ø14/ 22 | 1ø14/22 |
| F21 | 1ø12/ 25 | 1ø12/25 |
| F22 | 1ø20/20 | 1ø20/22 |
| F23 | 1ø16/29 | 1ø16/30 |
| F24 | 1ø16/25 | 1ø16/27 |
| F25 | 1ø16/28 | 1ø16/30 |
| F26 | 1ø12/25 | 1ø12/20 |
| F27 | 1ø16/25 | 1ø16/27 |
| F28 | 1ø16/21 | 1ø16/21 |

*Table (4:7) single footing reinforcement*

## 4.2.4 ELASTIC SETTELMENT:

Settlement calculations essential to check whether the settlement of the footing with initial dimensions is acceptable or not.

$$\mathbf{Se = q_0 * \alpha * B' * (1-\mu_s^2)/E_s * I_s * I_f} \quad \text{......... equ(4:17)}$$

***Where:***

$q_0$ : Net applied pressure at foundation level ($KN/m^2$)

$\alpha$ : = 4 for center , = 1 for corner

B' : = (B/2) for center , = B for corner

$\mu_s$ : Soil poisson ratio.

$E_s$ : Average modules of elasticity of soil

$I_s$ : shape factor

$$I_s = F_1 + ( (1-2\mu_s)/(1-\mu_s) ) * F_2 \quad \text{...........equ(4:18)}$$

$F_1$ & $F_2$ = f( m' , n' )

m' = (L/B) for center and corner

n' = H/(B/2) for center , = (H/B) for corner

$I_f$: Depth factor , = f( ($D_f$/B) , $\mu_s$ , (L/B) )

Note: f1, f2 & If from “Principle of Foundation Engineering 5th edition”[1].

**Footing 19:**

$\mathbf{S_e = q_0*\alpha *B'* (1-\mu_s^2) *I_s *I_f/E_s}$

$q_0$ = (367.5*9.81) / (3.3*3.3) =331.05 $KN/m^2$

$\alpha$: = 4 for center

= 1.65 (assume $D_f$ =1.5 m) B': = (B/2) for center

$\mu_s$:= 0.3 $KN/m^2$ $E_s = 25 *10^3$

$I_s$ : shape factor

$F_1$ & $F_2$ = 0.498,0 .016respectively

m' = 1 , n' = 10

Therefore:

$I_s$ = 0.498 + ((1- 2*0.3)/ (1-0.3))*0.016 = 0.5

$I_f$ = f (($D_f$/B), $\mu_s$ , (L/B) ) = 0. 606

And, Se = .024 * 1000 = 24 mm

Settlement calculations carried out based on settlement of center of footing which more than settlement of corner , in addition to calculate settlement as a flexible footing , but if the footing is rigid , the settlement value will be reduce.

*All values of settlement less than 25 mm is acceptable value.*

## 4.3 DESIN OF TIE BEAMS

Tie beams are beams used to connect between columns necks, its function to provide resistance moments applied on the columns, Tie beams useful in building that designed to resist earthquakes load to provide limitation of footings movement, and it’s also important if settlement occurs.

The moment applied on the columns is low, so assuming a tie beam with dimensions of 30 cm width and 60 cm depth.

**b=30 cm**

**h=60cm**

**Cover=5cm**

**Steel reinforcement:**

Use minimum area of steel:

$As_{min} = \rho_{min}\, b\, d$

$\rho_{min} = 14/f_y$ …..………equ(4:19)

$\rho_{min}=0.0033$

$As = 0.0033*30*55 = 5.5 cm^2$

**Use 1 Φ 12 /20cm top steel.**

**Use1 Φ 12 /20cm bottom steel.**

## 4.4 DESIGN OF WALL FOOTING

Shear walls which using in project supported by wall footing, this wall footing carried a distributed load (load per meter), and so the design of wall footing will be per meter.

The design steps are the same as design of single footing, but thickness will be controlled by wide-beam shear.

Ultimate load /m' = 66t/m' (From SAP2000)

Service load /m' = 50t/m' (from SAP2000)

Wall width = service load /allowable bearing capacity

B = 50 /35 = 1.42 m

**Use B=1.5 m**

**Check shear:**

$q_u = 66/1.5 = 44\ t/m^2$

$V_u = q_u * L$

$= 44\ (0.65 – d)$

$\Phi * V_c = 0.75 * 0.53\sqrt{240} * d * 10$

From that: **d= 27**cm **(try H=35 , d=30)**

**Steel reinforcement:**

Max moment in wall footing occurs at in the middle between center line of wall and face of wall

So, Bending Moment = $q_u$*$L^2$/2

M = 44 (.4225)/2 =9.295ton.m

$$\rho = \frac{0.85 * 240}{4200} \left( 1 - \left( \sqrt{1 - \left( \frac{2.61 * 9.295 * 10^5}{240 * 100 * 30^2} \right)} \right) \right) = 0.0028$$

$A_{st}$ =0.0028 * 100*30 = 8.4 $cm^2$/m

**(1 Φ14 /18 cm c/c)**

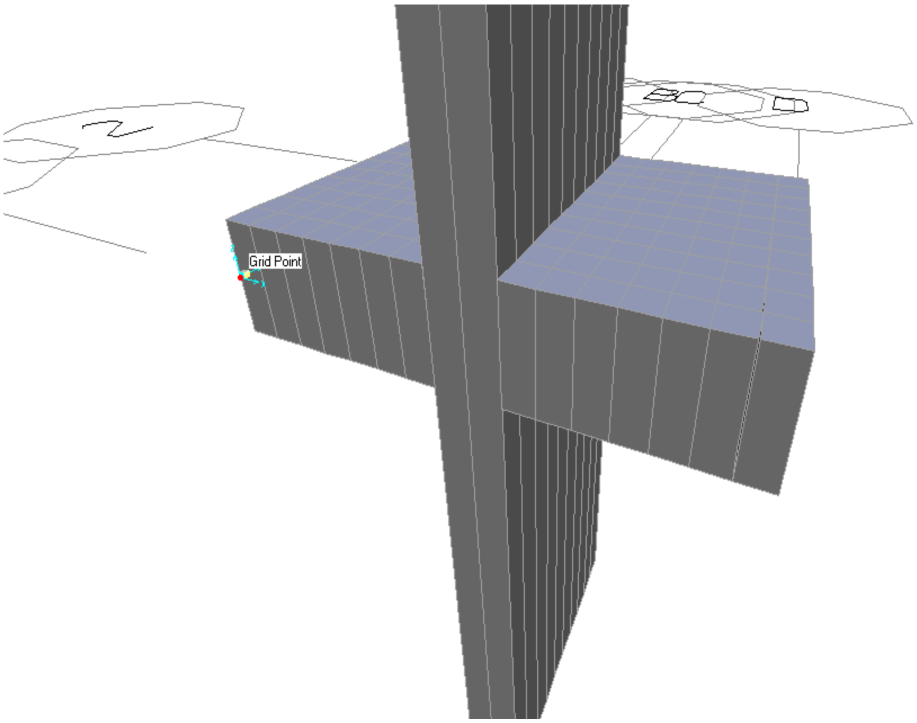
In the other direction use min area of steel

$A_{stmin}$ = 0.0018(100)(35) =6.3 $cm^2$

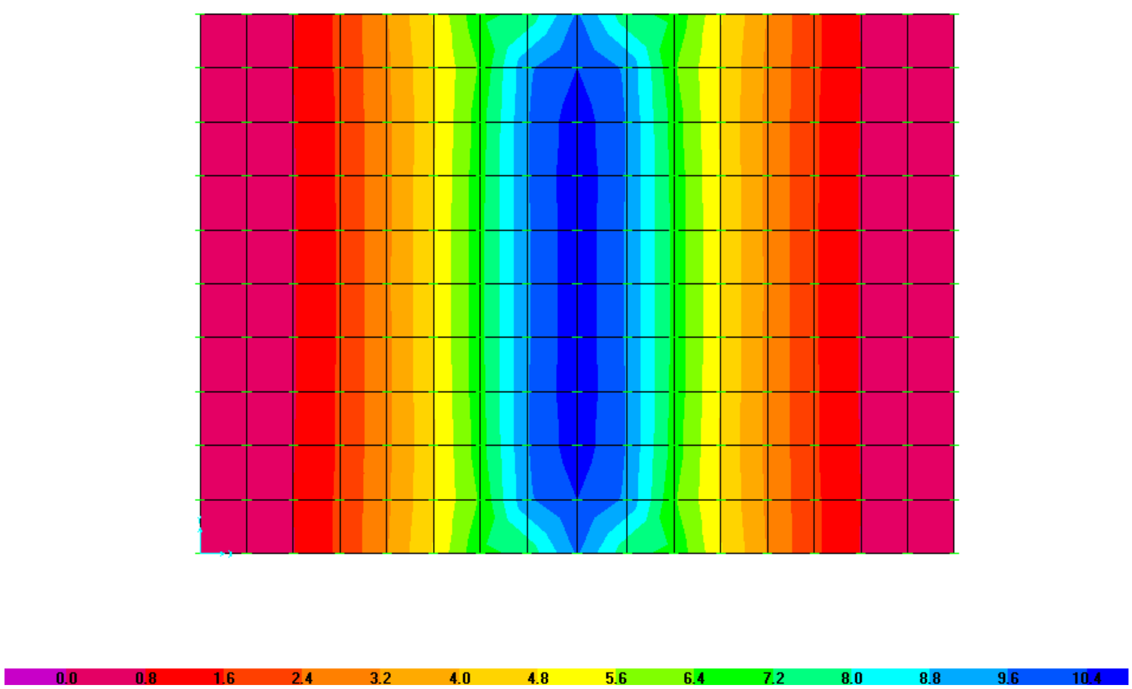
**(1 Φ14 /24 cm c/c)**

# 4.5 SAP ANALYSIS AND DESIGN:

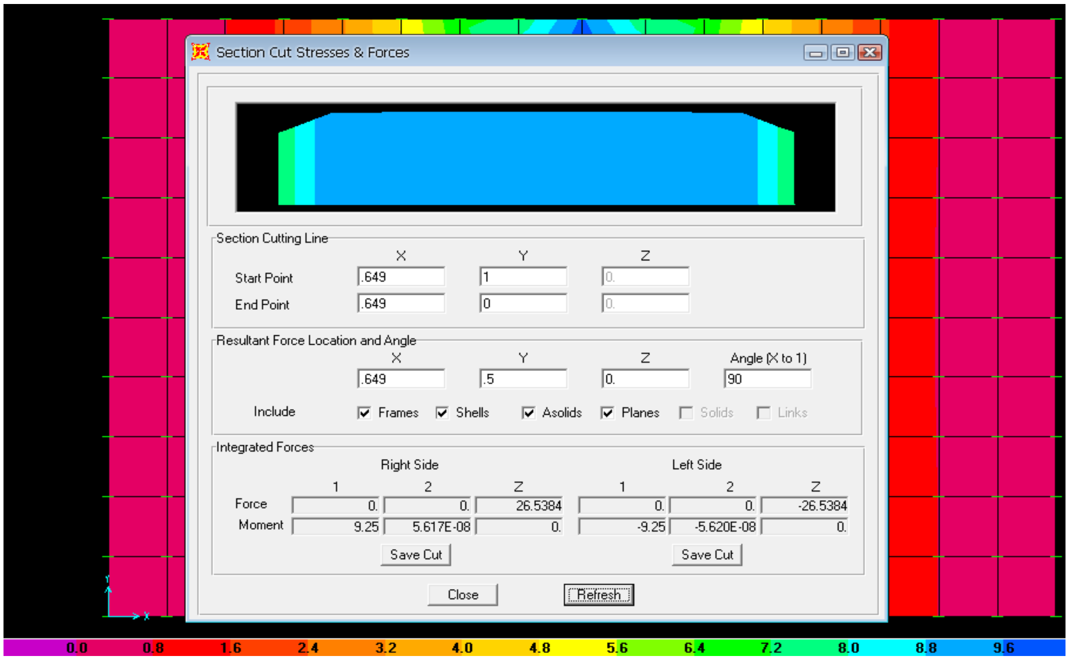
## ***4.5.1 Wall Footing:***




The figure below show bending moment in x-direction using SAP2000:



The figure below shows the values of moment at distance d from the face of the column **(section cut)**:




***<u>Steel Reinforcement</u>***

From the figure above the moment is 9.25

$\rho$= 0.00279

$A_{st}$ =0.00279 * 100*30 = 8.3 $cm^2/m$

**(1 Φ14 /18 cm c/c)**

In the other direction use min area of steel

$A_{stmin}$ = 0.0018(100) (35) =6.3 $cm^2$

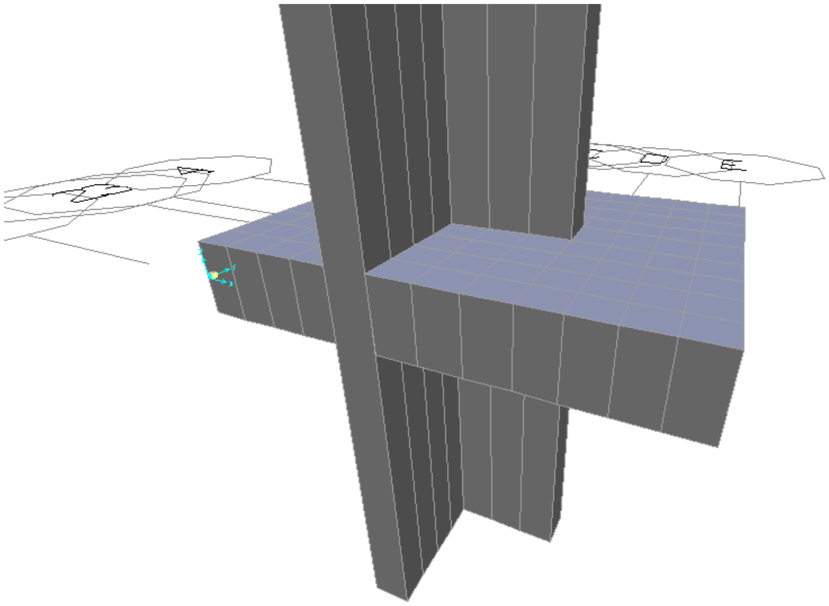
**(1 Φ14 /24 cm c/c)**

### *4.5.2 Eccentric footing:*

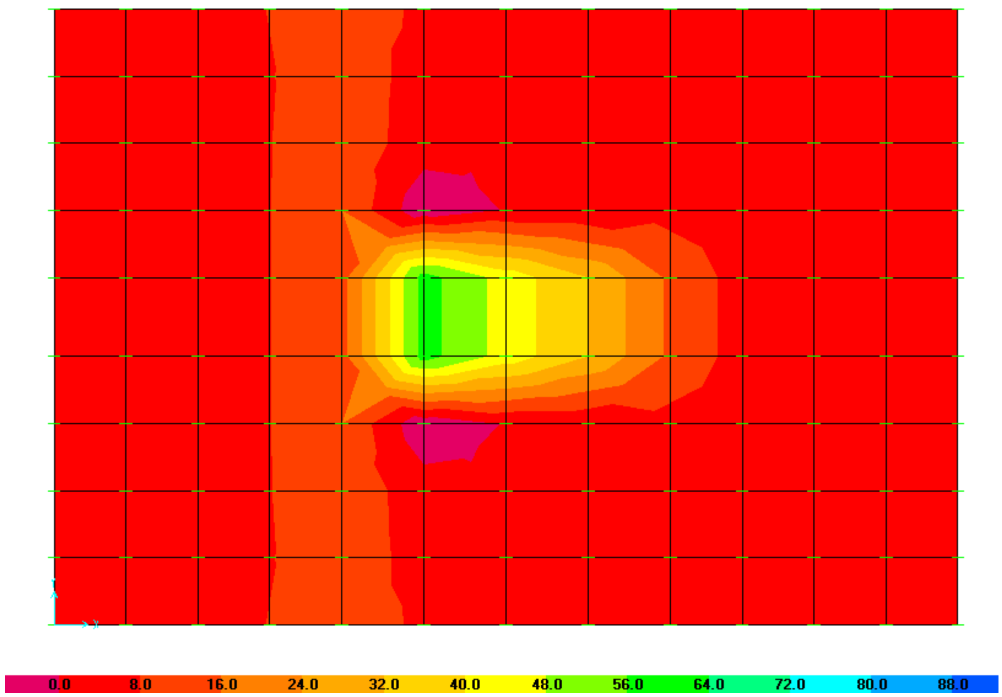
The shaded footing in the table of loading shows footings have an eccentric loads due to attachment with walls which will be design using sap 2000 program:-

**For footing 1** (L=2.2 m , B = 1.55 m , h =0.4 m)

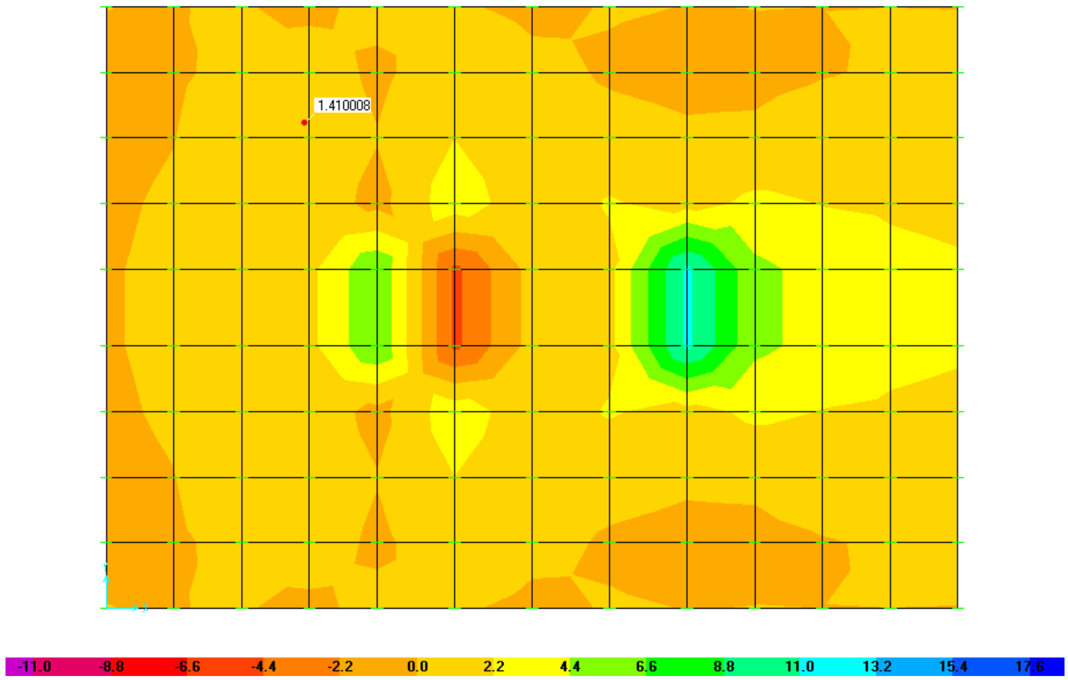
(The figure below shows the attachment of the wall and column)



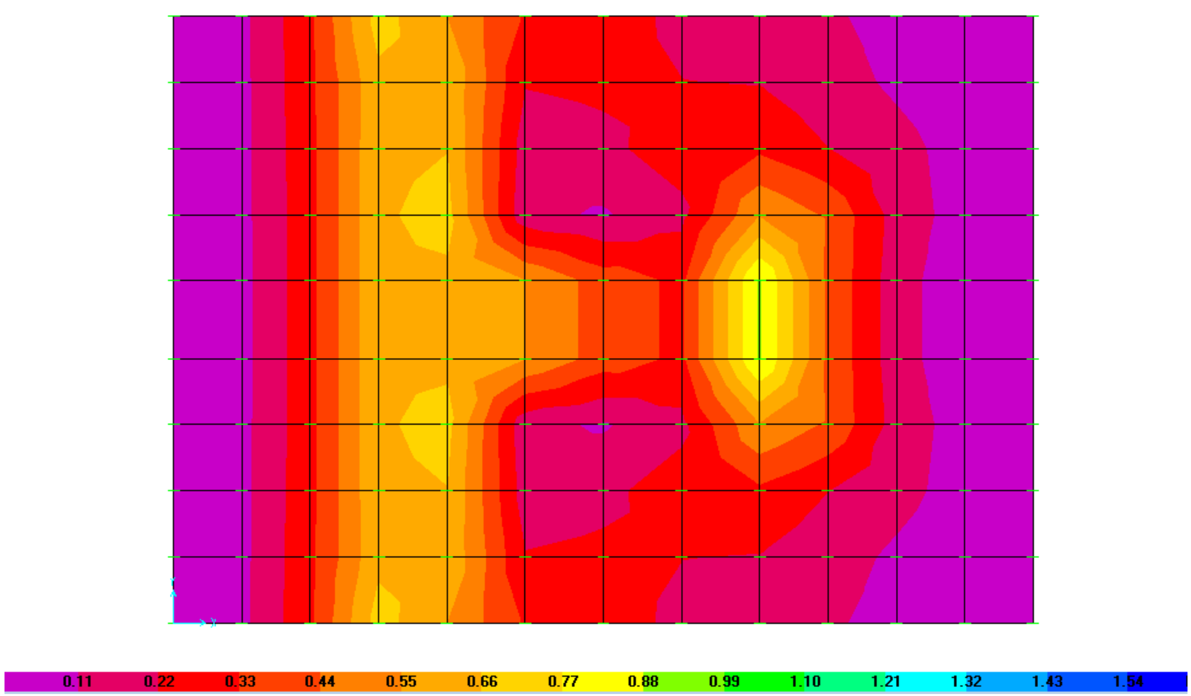
The moment in the **x-direction** shown in the figure:



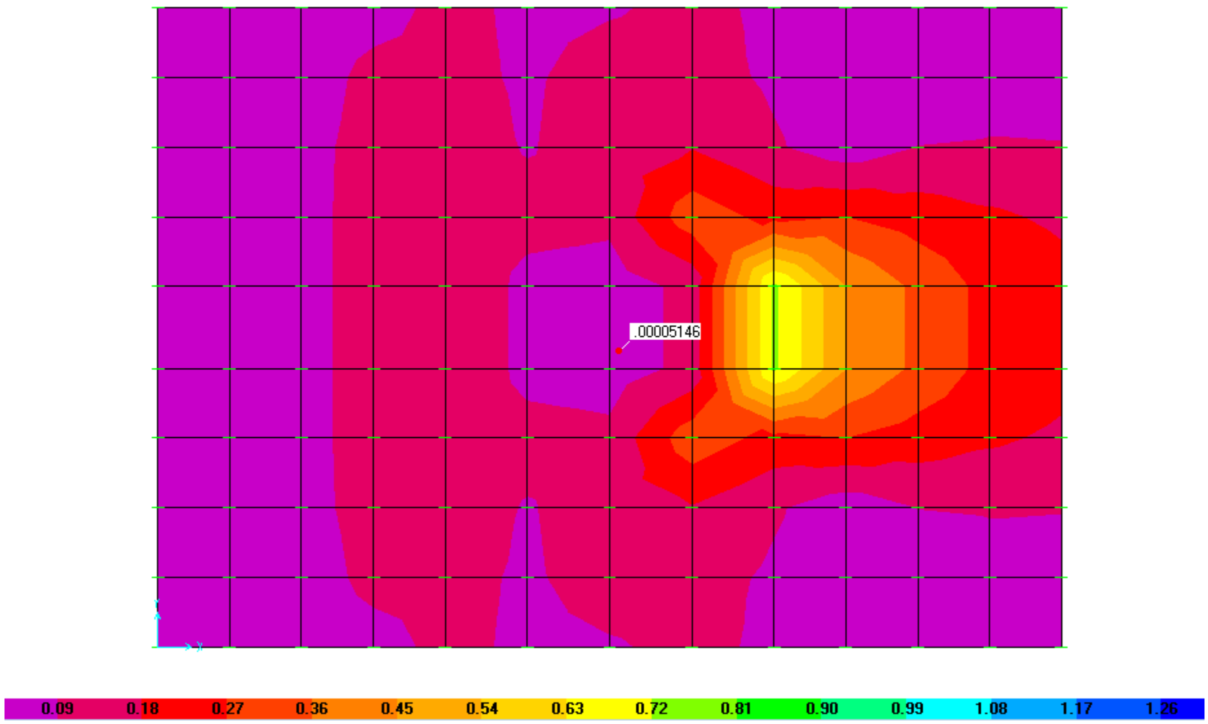
The moment in the **y-direction** below shown in the figure




The figure below shows the area of steel (As1) in **x- direction** that recommended in the design:



The figure below shows the reinforcement in the **y-direction** (As2):-



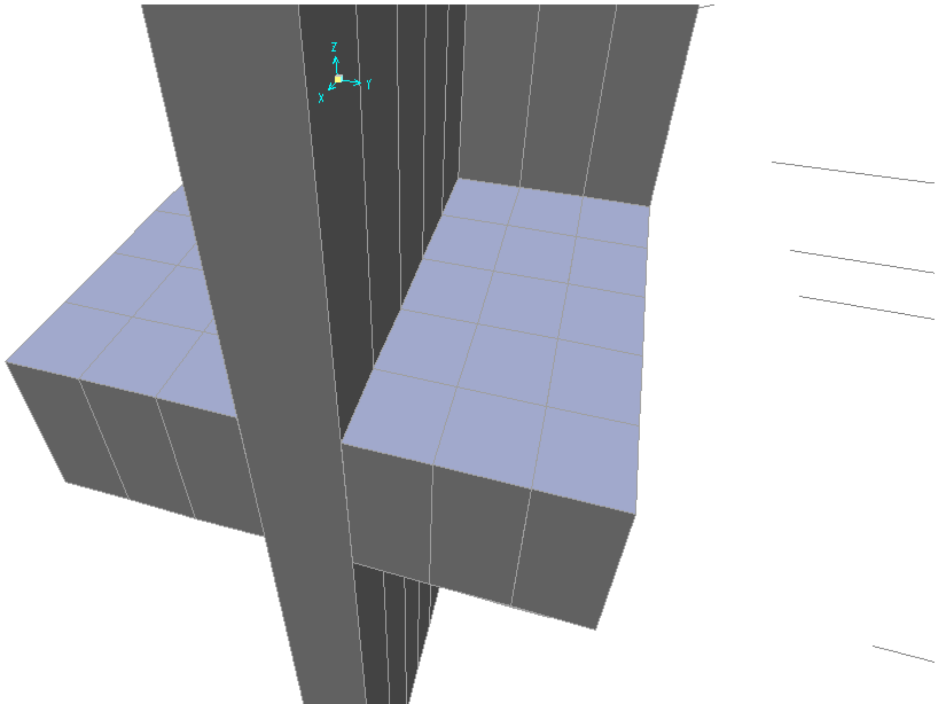

So, the maximum area of steel in the **x-direction** As1= 7.8 cm2

**1 ø 14 \20 cm.**

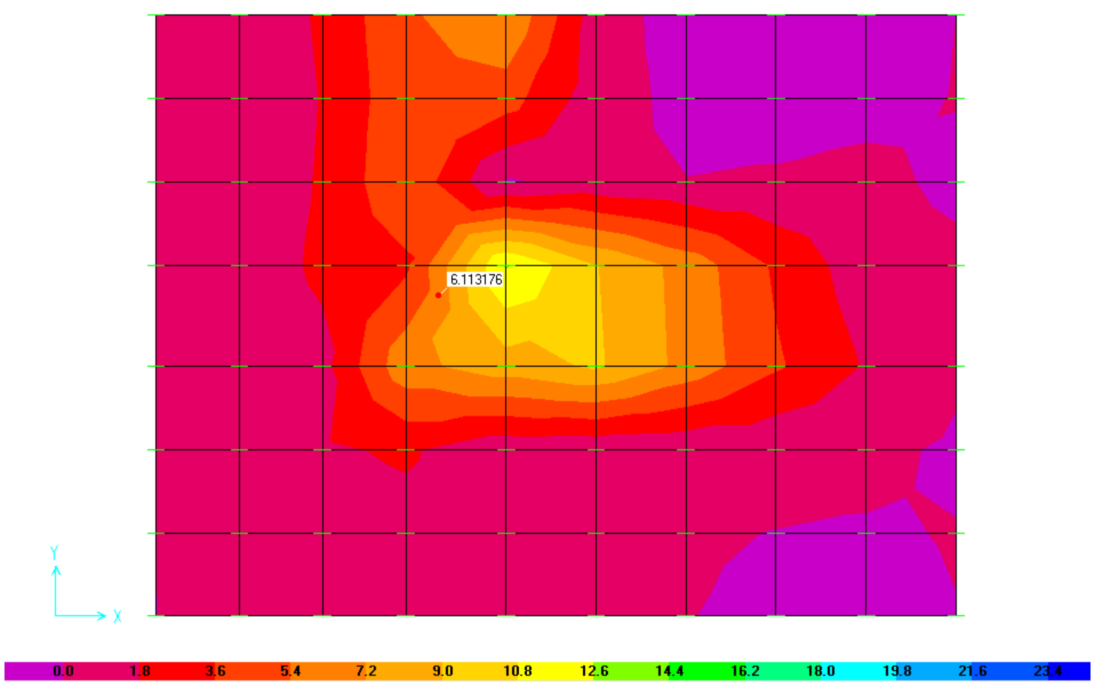
The maximum area of steel in the **y-direction** As2 = 7 cm2

**1 ø 17\22 cm.**

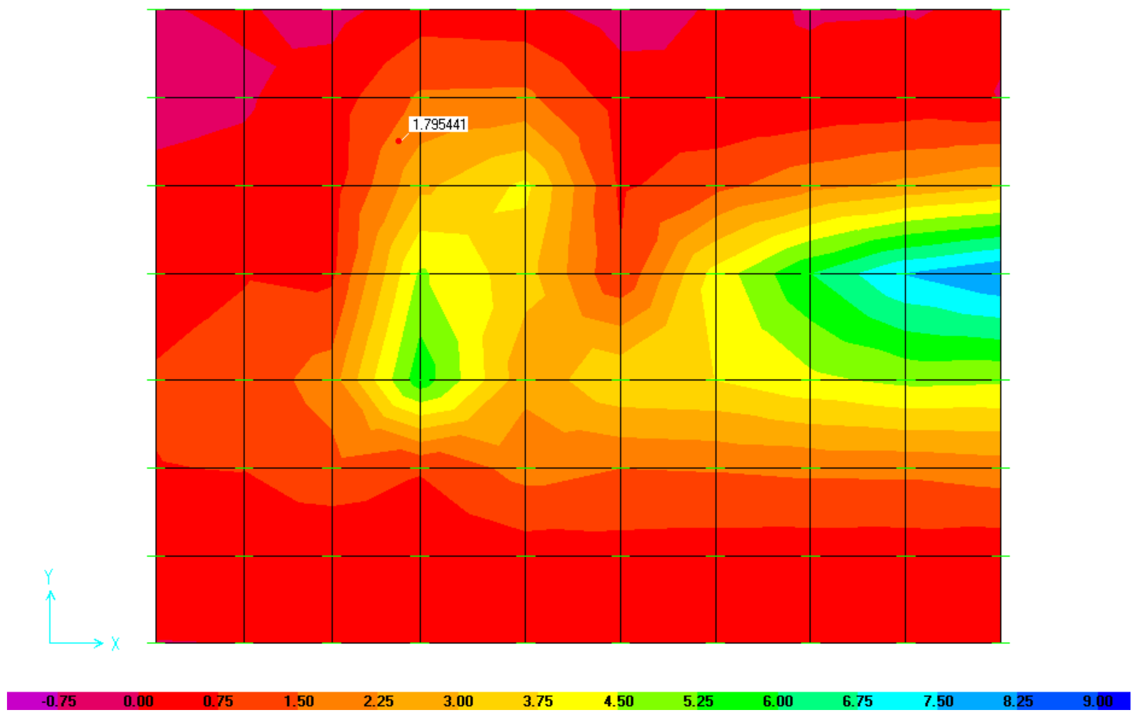
**For footing 2** (L= 1.6 m, B= 1.2 m, d= 0.4 m)



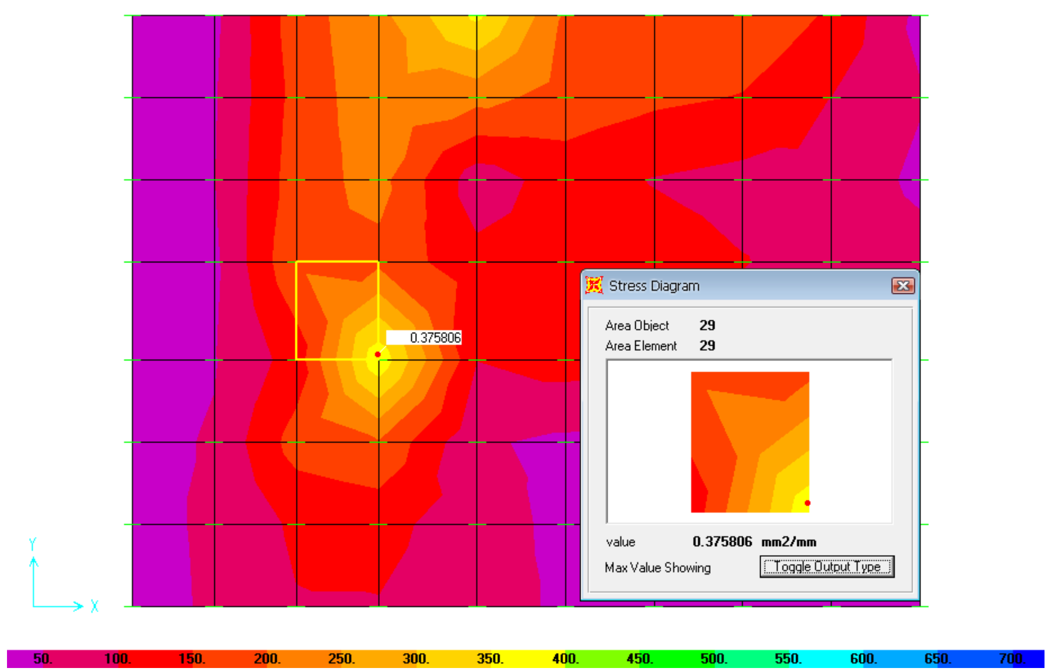
The moment in the **x-direction** shown in the figure:



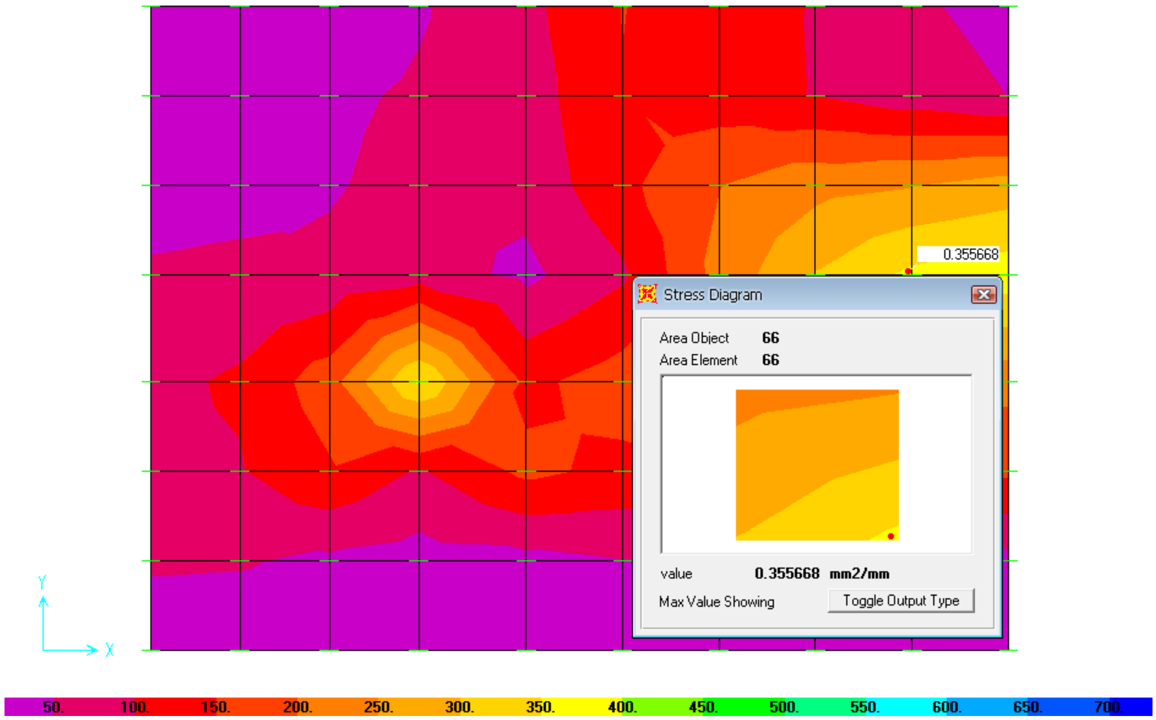

The moment in the **y-direction** below shown in the figure




The figure below shows the area of steel (As1) in **x- direction** that recommended in the design:




The figure below shows the reinforcement in the **y-direction** (As2):-




The maximum area of steel in the **x-direction** As1= 3.8 cm2

The maximum area of steel in the **y-direction** As2 = 3.6 cm2

Since both areas in x and y is too small, it is recommended now to use the minimum area of reinforcement:

As min = 0.0018 * 100 *40 = 7.2 cm2

**1 ø 14 \ 21 cm**

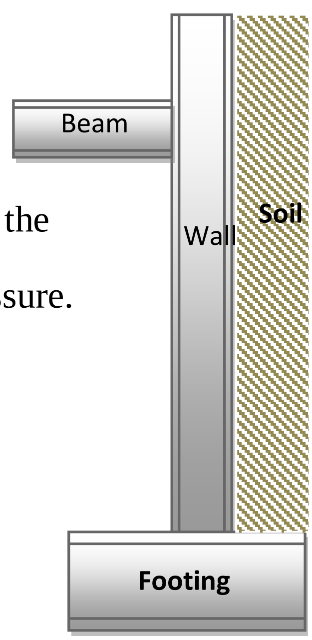
**Summary of reinforcement for the eccentric footings**

| Footing | L * B * d (m) | Reinforcement in L | Reinforcement in B |
|---|---|---|---|
| F1 | 1.8 * 1.4 * 0.4 | 1ø14/21 cm | 1ø14/21 cm |
| F52 | 1.35*1.8*0.35 | 1ø14/20 cm | 1ø14/21 cm |
| F53 | 1.5 * 2 *0.4 | 1ø16/28 cm | 1ø14/20 cm |
| F55 | 1* 1.6 * 0.3 | 1ø14/29 cm | 1ø14/29 cm |
| F57 | 1.3*1.9*0.35 | 1ø14/24 cm | 1ø14/24 cm |
| F58 | 1.5*2.2*0.4 | 1ø16/25 cm | 1ø14/20 cm |
| F59 | 1.1*1.7*0.3 | 1ø12/21 cm | 1ø12/21 cm |

## 4.6 PROKON ANALYSIS AND DESIGN:

***Retaining shear walls:***

The figure beside shows an illustrative view of the retaining wall in the project, where it carrying a part of building in addition of countering the lateral soil pressure.




This model designed using ***Prokon Calc Pad,***

Considering the **axial load** acts on the footing

comes from the wall and the building, in addition to the

**bending moment** resulting from the lateral soil pressure.

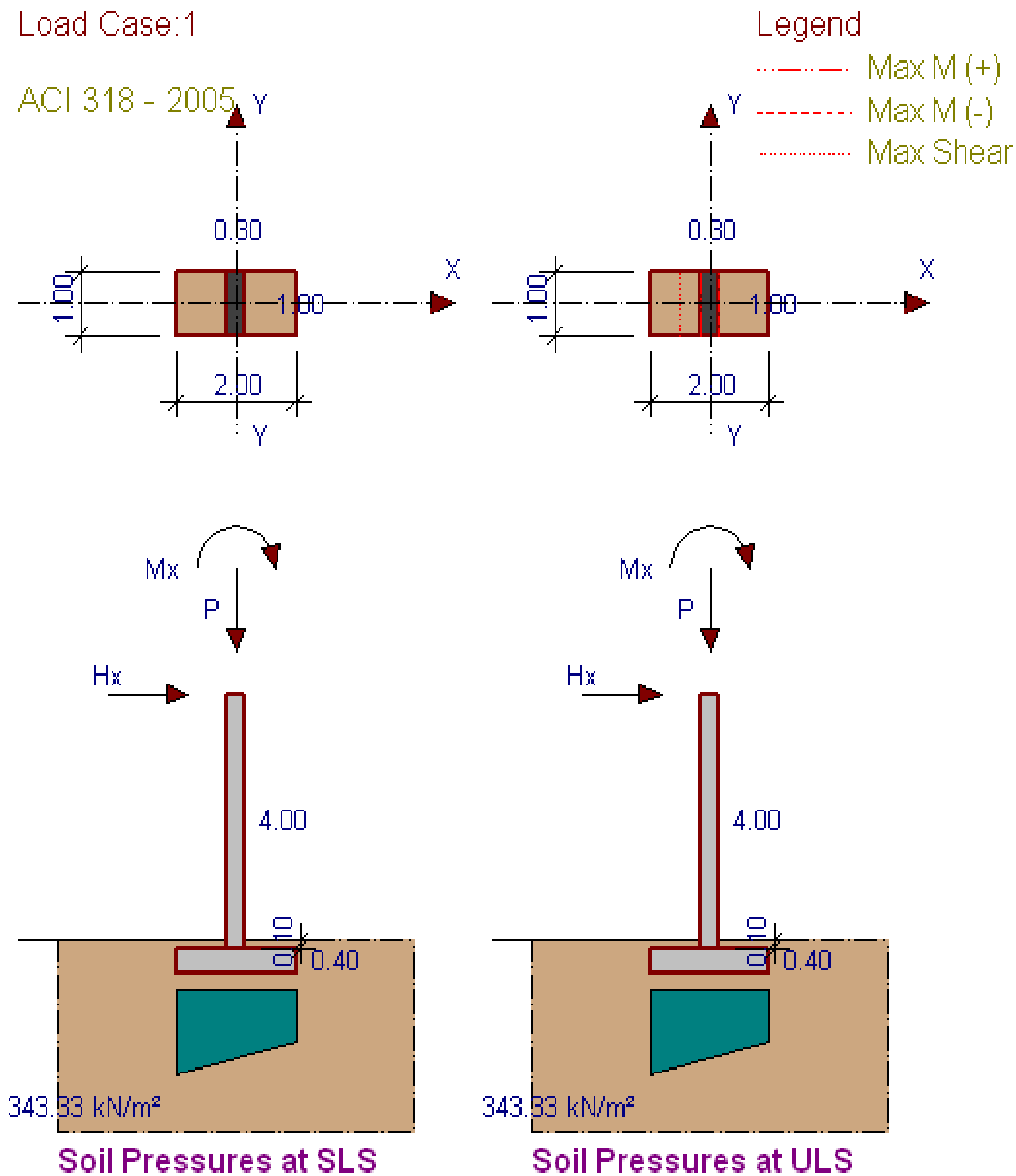
q = 4* 17 * 0.61= **41.5 KN.m**

Ka = 0.61

$\gamma$ = 17 KN\ $m^3$

Moment = 41.5 * $4^2$ /15 = **44.6 KN.m**

The figure below shows an illustrative sketch and details of the wall existing in the projects.



# Column Base Design :

## Input Data

| | | |
|---|---|---|
| Base length A (m) | | 2.00 |
| Base width B (m) | | 1.00 |
| Column(s) | Col 1 | Col 2 |
| C (m) | .3 | |
| D (m) | 1 | |
| E (m) | | |
| F (m) | | |
| Stub column height X (m) | | 4 |
| Base depth Y (m) | | .4 |
| Soil cover Z (m) | | .1 |
| Concrete density ($kN/m^3$) | | 25 |
| Soil density ($kN/m^3$) | | 17 |
| Soil friction angle (°) | | 14 |
| Base friction constant | | 0.5 |
| Rebar depth top X (mm) | | 50 |
| Rebar depth top Y (mm) | | 50 |
| Rebar depth bottom X (mm) | | 50 |
| Rebar depth bottom Y (mm) | | 50 |
| ULS ovt. LF: Self weight | | 1.0 |
| ULS LF: Self weight | | 1.0 |
| Max. SLS bearing pr. ($kN/m^2$) | | 350 |
| S.F. Overturning (ULS) | | 1 |
| S.F. Slip (ULS) | | 1 |
| fc' base (MPa) | | 24 |
| fc' columns (MPa) | | 24 |
| fy (MPa) | | 420 |

| | | Unfactored Loads | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Load Case | Col no. | LF ULS ovt | LF ULS | P (kN) | Hx (kN) | Hy (kN) | Mx (kNm) | My (kNm) |
| 1 | 1 | 1 | 1 | 500 | | | -44.6 | |

| Bending Schedule Parameters | | Rebar | (mm²/m) | |
|---|---|---|---|---|
| Schedule file name | BaseBS | Entered | Required | Nominal |
| Bars: Bottom X-direction | Y25@400 | 1227 | 876 | 1149 |
| Bars: Bottom Y-direction | Y25@400 | 1227 | 0 | 1149 |
| Bars: Top X-direction | | 0 | 0 | 1149 |
| Bars: Top Y-direction | | 0 | 0 | 1149 |

| Column Parameters | Column 1 | Column 2 |
|---|---|---|
| Type:(C)ol,(S)tub,(N)one | S | |
| Main Bars | 4Y16 | |
| Middle bars vert faces | 4Y12 | |
| Middle bars hor faces | 2Y16 | |
| Lap length factor | 45 | |
| Link diameter (mm) | 12 | |
| Link width (mm) | 240 | |
| Link height (mm) | 940 | |
| No. of Links | 3 | |
| Column names | col1 | |

| Cover (mm) | |
|---|---|
| Bottom | 75 |
| Sides | 50 |
| Top | 50 |
| Cols | 30 |

BS Picture
- ( ) A4
- (•) A5

First Bar mark A  Language E/A  [Reset defaults]

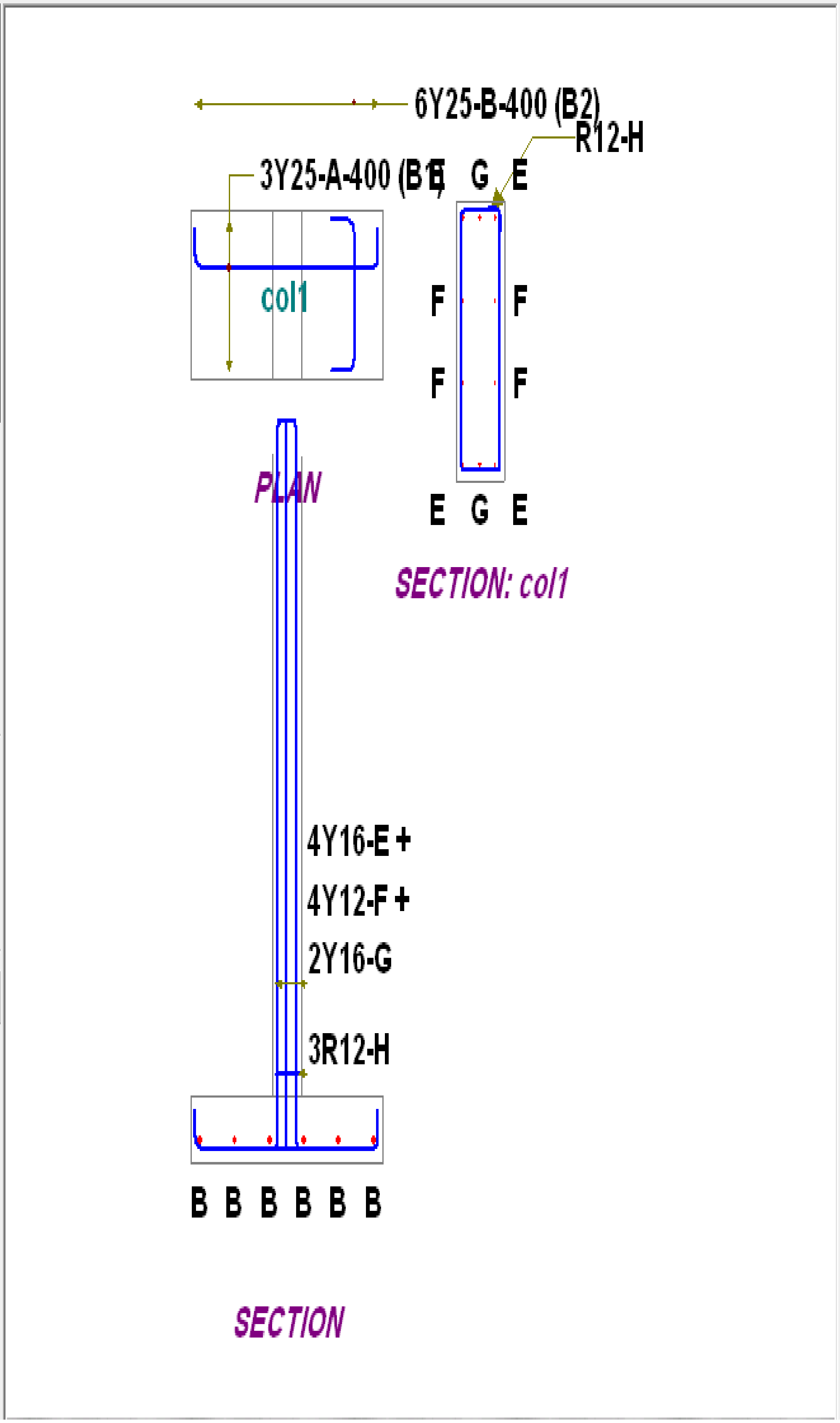
Top bars configuration
- (•) SC 52 ABR
- ( ) SC 38 ABR
- ( ) SC 55
- ( ) SC 35 + stools
- ( ) SC 34 ABR + stool
- ( ) none

Bottom bars configuration
- (•) SC 35
- ( ) SC 34 ABR
- ( ) SC 55
- ( ) SC 60 X-dir; SC 35 Y-dir.
- ( ) SC 60 Y-dir; SC 35 X-dir.

[Generate Schedule]

☐ Save bending schedule parameters with input

NB:Check starter bars sizes with column design



| Output for Load Case 1 | |
|---|---|
| Soil pressure (ULS) (kN/m²) | 343.33 |
| Soil pressure (SLS) (kN/m²) | 343.33 |
| SF overturning (SLS) | 12.40 |
| SF overturning (ULS) | 12.40 |
| Safety Factor slip (ULS) | >100 |
| Safety Factor uplift (ULS) | >100 |
| Bottom | |
| Design moment X (kNm/m) | 112.93 |
| Reinforcement X (mm²/m) | 876 |
| Design moment Y (kNm/m) | 0.00 |
| Reinforcement Y (mm²/m) | 0 |
| Top | |
| Design moment X (kNm/m) | 0.00 |
| Reinforcement X (mm²/m) | 0 |
| Design moment Y (kNm/m) | 0.00 |
| Reinforcement Y (mm²/m) | 0 |
| Linear Shear X (MPa) | 0.433 |
| vc (MPa) | 0.612 |
| Linear Shear Y (MPa) | 0.000 |
| vc (MPa) | 0.580 |
| Linear Shear Other (MPa) | 0.000 |
| Punching Shear (MPa) | 0.567 |
| vc (MPa) | 1.220 |
| Cost | 0.00 |

Load case 1

Previous Next

# APPENDECIS

# BOREHOLE LOG

| | | | | | |
|---|---|---|---|---|---|
| *Project* | Muskat School – Kufr Rae'i – Jenin | | | *Location* | Kufr Rae'i |
| **Borehole No.** | 1 | **Page No.** | 1/1 | *Drilling Date* | 10-7-2008 |
| *Ground level* | 0.0 | | | *Weather* | ***Sunny*** |
| **Drill Rig** | Mobile B-31 | | | *Operator* | Adnan |

| Scale **(m)** | Sam-pler Type | Sym-bol | Description of soil strata | USCS | SPT (No. of blows) 15 | 15 | 15 | N |
|---|---|---|---|---|---|---|---|---|
| 0.0 | | | Dry deposits of silty clay | CL | | | | |
| | | | | | 8 | 8 | 10 | 18 |
| 1.0 | | | | | | | | |
| | | | | | | | | |
| 2.0 | | | Medium moist deposits of silty clay | CL | 9 | 9 | 12 | 21 |
| | | | | | | | | |
| 3.0 | | | | | | | | |
| | | | | | | | | |
| 4.0 | | | | | | | | |
| | | | Medium hard formation of creamy marlstone formation | CL | | | | R |
| 5.0 | | | | | | | | |
| | | | | | | | | |
| 6.0 | | | | | | | | |
| | | | | | | | | |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| 7.0 | | | | | | | | | |
| | | | | | | | | | |
| 8.0 | | | | | | | | | |
| | | | | | | | | | |
| 9.0 | | | | | | | | | |
| | | | | | | | | | |
| 10.0 | | | | | | | | | |
| | | | | | | | | | |
| 11.0 | | | | | | | | | |
| | | | | | | | | | |
| 12.0 | | | | End of boring @ 10 m | | | | | |

*Water Record*

| | | | |
|---|---|---|---|
| **Level, at which water was encountered** | None | Color of water | - |
| *Water level 24hrs. after completion* | | None | |

**Remarks :**

**USCS-** Unified Soil Classification System

**R-** Refusal

| | | |
|---|---|---|
| **Approved :** | **Dr. Sami A. Hijjawi** | |

# BOREHOLE LOG

| | | | | | |
|---|---|---|---|---|---|
| *Project* | Muskat School – Kufr Rae'i – Jenin | | | *Location* | Kufr Rae'i |
| **Borehole No.** | 2 | **Page No.** | 1/1 | *Drilling Date* | 10-7-2008 |
| *Ground level* | 0.0 | | | *Weather* | ***Sunny*** |
| **Drill Rig** | Mobile B-31 | | | *Operator* | Adnan |

| Scale **(m)** | Sam-pler Type | Sym-bol | Description of soil strata | USCS | SPT (No. of blows) 15 | 15 | 15 | N |
|---|---|---|---|---|---|---|---|---|
| 0.0 | | | Dry deposits of silty clay | CL | | | | |
| | | | | | 8 | 8 | 9 | 17 |
| 1.0 | | | | | | | | |
| | | | Soft and moist formation of creamy marlstone formation | CL | | | | |
| 2.0 | | | | | | | | R |
| | | | | | | | | |
| 3.0 | | | | | | | | |
| | | | | | | | | |
| 4.0 | | | | | | | | |
| | | | Dry, medium hard formation of creamy marlstone formation | CL | | | | R |
| 5.0 | | | | | | | | |
| | | | | | | | | |
| 6.0 | | | | | | | | |
| | | | | | | | | |
| 7.0 | | | | | | | | |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | |
| 8.0 | | | | | | | | | |
| | | | | | | | | | |
| 9.0 | | | | | | | | | |
| | | | | | | | | | |
| 10.0 | | | | | | | | | |
| | | | | | | | | | |
| 11.0 | | | | | | | | | |
| | | | | | | | | | |
| 12.0 | | | | End of boring @ 10 m | | | | | |

| | | | |
|---|---|---|---|
| *Water Record* | | | |
| **Level, at which water was encountered** | None | Color of water | - |
| *Water level 24hrs. after completion* | | None | |

**Remarks :**

**USCS-** Unified Soil Classification System

**R-** Refusal

| | | |
|---|---|---|
| **Approved :** | **Dr. Sami A. Hijjawi** | |

# BOREHOLE LOG

| | | | | | |
|---|---|---|---|---|---|
| *Project* | Muskat School – Kufr Rae'i – Jenin | | | *Location* | Kufr Rae'i |
| **Borehole No.** | 3 | **Page No.** | 1/1 | *Drilling Date* | 10-7-2008 |
| *Ground level* | 0.0 | | | *Weather* | ***Sunny*** |
| **Drill Rig** | Mobile B-31 | | | *Operator* | Adnan |

| Scale **(m)** | Sam-pler Type | Sym-bol | Description of soil strata | USCS | SPT (No. of blows) 15 | 15 | 15 | N |
|---|---|---|---|---|---|---|---|---|
| 0.0 | | | Dry deposits of silty clay | CL | | | | |
| | | | | | | | | |
| 1.0 | | | | | 9 | 8 | 9 | 17 |
| | | | | | | | | |
| 2.0 | | | | | | | | |
| | | | | | | | | |
| 3.0 | | | | | | | | |
| | | | Medium hard formation of creamy marlstone formation | CL | | | | R |
| 4.0 | | | | | | | | |
| | | | | | | | | |
| 5.0 | | | | | | | | |
| | | | | | | | | |
| 6.0 | | | | | | | | |
| | | | | | | | | |
| 7.0 | | | | | | | | |

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | | |
| 8.0 | | | | | | | | | | |
| | | | | | | | | | | |
| 9.0 | | | | | | | | | | |
| | | | | | | | | | | |
| 10.0 | | | | | | | | | | |
| | | | | | | | | | | |
| 11.0 | | | | | | | | | | |
| | | | | | | | | | | |
| 12.0 | | | | End of boring @ 6 m | | | | | | |

| *Water Record* | | | |
|---|---|---|---|
| **Level, at which water was encountered** | None | Color of water | **-** |
| *Water level 24hrs. after completion* | | None | |

**Remarks :**

**USCS-** Unified Soil Classification System

**R-** Refusal

| | | |
|---|---|---|
| **Approved :** | **Dr. Sami A. Hijjawi** | |

# BOREHOLE LOG

| | | | | | |
|---|---|---|---|---|---|
| *Project* | Muskat School – Kufr Rae'i – Jenin | | | *Location* | Kufr Rae'i |
| **Borehole No.** | 4 | **Page No.** | 1/1 | *Drilling Date* | 10-7-2008 |
| *Ground level* | 0.0 | | | *Weather* | ***Sunny*** |
| **Drill Rig** | Mobile B-31 | | | *Operator* | Adnan |

| Scale **(m)** | **Sam-pler Type** | **Sym-bol** | **Description of soil strata** | **USCS** | SPT (No. of blows) **15** | **15** | **15** | **N** |
|---|---|---|---|---|---|---|---|---|
| 0.0 | | | | | | | | |
| | | | | | | | | |
| 1.0 | | | | | 9 | 8 | 8 | 16 |
| | | | Dry deposits of silty clay | CL | | | | |
| 2.0 | | | | | | | | |
| | | | | | | | | |
| 3.0 | | | | | | | | |
| | | | | | | | | R |
| 4.0 | | | | | | | | |
| | | | Medium hard formation of creamy marlstone formation | CL | | | | |
| 5.0 | | | | | | | | |
| | | | | | | | | |
| 6.0 | | | | | | | | |
| | | | | | | | | |
| 7.0 | | | | | | | | |

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | |
| 8.0 | | | | | | | | | |
| | | | | | | | | | |
| 9.0 | | | | | | | | | |
| | | | | | | | | | |
| 10.0 | | | | | | | | | |
| | | | | | | | | | |
| 11.0 | | | | | | | | | |
| | | | | | | | | | |
| 12.0 | | | | End of boring @ 6 m | | | | | |

*Water Record*

| | | | |
|---|---|---|---|
| **Level, at which water was encountered** | None | Color of water | - |
| *Water level 24hrs. after completion* | | None | |

**Remarks :**

**USCS-** Unified Soil Classification System

**R-** Refusal

| | | |
|---|---|---|
| **Approved :** | **Dr. Sami A. Hijjawi** | |

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