CHAPTER ONE

INTRODUCTION

This chapter is devoted to describe the assigned project which is the Korean Industrial School at Jenin district. It concentrates also at investigation and understanding of the site and its surroundings. Finally, the scope of the project is defined and stated at the end of this chapter.

1.1 THE SITE

The proposed project was constructed at Jenin District which is located at the north of Palestine as shown in figure 1.1. Jenin city has many of villages around it, and the map in figure 1.2 shown how the villages surrounding around Jenin city.

Our project located on Bait Qad street as shown in the figure 1.3, and it’s far from Bait Qad street about 600 m, and from Jenin city about 3 km. The school is located in an agricultural areas as shown in figure 1.4, and the type of it’s soil is soft clay as mentioned in soil report test.
Figure 1.1 : Palestine Map
Figure 1.2 : Jenin District
Figure 1.3: location of the school relative to Jenin city
Figure 1.4: the location of the school related to Beit Qad street
1.2 THE PROJECT

The existing building consists of four floors, each has class rooms, corridors and various halls for educational purposes, the total area of the school is 641.6 m², the clear height of each floor is 3.44 m. See figure 1.5.

The building is divided into two parts, 2.5 cm expansion joint is separating the building- blocks, an expansion joint provides the required distance to absorb the temperature-induced expansion and, also it absorb vibration, and allows movement due to ground settlement or earthquakes. It also has two stairs of same area, each stair is surrounded by shear walls (i.e., work to keep the building rigid and braced), they are located in a suitable location that provides the building the needed rigidity. See figure 1.6.
Figure 1.5: The school building
Figure 1.6: Shear walls and expansion joint
1.3 SCOPE OF THE WORK

This project involves the following items:

1- Evaluation of the site and the project.
2- Selection of proper footings.
3- Design of the selected footings including combined footings, shear walls, and a retaining wall.
CHAPTER TWO

LITERATURE REVIEW

Foundation may be classed according to their nature, their function, the material they are constructed of, and according to method of their analysis. All foundation however, may be classed generally as (1) shallow (2) deep (3) special foundation.

If the foundation is laid directly on a component load bearing soil at minimum depth below the ground surface such that the foundation is safe against lateral expansion of soil from underneath the base of footing and is also safe against volume changes of soil due to frost action, swelling for examples, and shrinkage (settlement, heave), one then speaks of a shallow foundation. Shallow foundation applies mainly to buildings and certain engineering structures.

If, however, the load from the structure should be transmitted to a considerable depth by means of end bearing piles, piers, and caissons through weak soil to a geologic stratum component to support the structural load, or by means of friction piles where no underlying strong stratum is economically feasible, one then speaks of a deep foundation. Deep foundation applies mainly to engineering
structures such as bridges and hydraulic structures as well as to building if they have to be built on sites of poor soil. Wherever possible, deep be avoided because their cost increase rabidly with depth.

2.1: Shallow foundation

In shallow foundation the depth of foundation beneath natural ground level is very near. In average foundation depth is between 1.0 m and 2.5 m see (figure 2.1)

Figure 2.1: Shallow Foundation
We can use shallow foundation when the following conciliations are fulfilled:

1- When there is good bearing stratum near the ground surface and this layer can resist safely the stresses from footing.
2- When the expected settlements is within limit according to surface nature.
3- When the cost of construction of this type of footing is economic more than the other types.

There are two main types of shallow foundation:

1- When the good bearing soil stratum is near ground level $D_f = (1-2m)$ we shall put a layer of plain concrete over it then we shall put a reinforced concrete footing over P.C. footing.

2- When the good bearing silo stratum is relatively far from ground level ($D_f = 2.5-3m$) a part of the weak soil stratum is replaced by a well compacted sandy gravel layer over this replacement stratum we shall put the required footing (P.C.and R.C).
2.1.1 : Isolated (Spread) Foundation

A footing a carrying a single column is called isolated footing, since its function is to spread the column load laterally to the soil so that the stress intensity is reduced are sometimes called single or spread or spread footing.

Single footing may be accountant thickness or either stepped or sloped.

Stepped or sloped footing are most commonly used to reduce the quantity of concrete away from the column where the bending moments are small and when then the footing is not reinforced.

Isolated footing are designed to resist dead load delivered by column, the live load contribution may be either the full amount for one or two story buildings or reduced value as allowed by the focal building code for multistory structures.

Isolated footing can be square or rectangular or circular shape.

- If we have a square or rectangular column, is preferred to have a square footing.
- If we have a circular column, is preferred to have a circular footing.
2.1.2 : Combined Foundation

A combined footing is usually used to support two columns of unequal loads. In such a case, the resultant of the applied loads would not coincide with the centroid of the footing, and the consequent the soil pressure would not be uniform.

Another case where a combined footing is an efficient foundation solution is when there are two interior columns which are so close to each other that the two isolated footings stress zones in the soil areas would overlap.

The area of the combined footing may be proportioned for a uniform settlement by making its centroid coincide with the resultant of the column loads supported by the footing.

There are many instances when the load to be carried by a column and the soil bearing capacity are such that the standard spread footing design will require an extension of the column foundation beyond the property line. In such a case, two or more columns can be supported on a single rectangular foundation. If the net allowable soil pressure is known, the size of the foundation B x L can be determined. A third case of a useful application of a combined footing is if one (or several) columns are placed right at the property line. The footings for those columns can not be centered around the
columns. The consequent eccentric load would generate a large moment in the footing. By tying the exterior footing to an interior footing through a continuous footing, the moment can be substantially reduced, and a more efficient design is attained.

Figure 2.2 : Rectangular Combined Foundation
This type of combined footing, shown in Figure 2.3, is sometimes used as an isolated spread foundation for a column that is required to carry a large load in a tight space. The size of the trapezoidal footing that will generate a uniform pressure on the soil can be found through the following procedure.

![Figure 2.3: Trapezoidal Foundation](image-url)
A strap footing is used to connect an eccentrically loaded column footing to an interior column.

The strap is used to transmit the moment caused from an eccentricity to the interior column footing so that a uniform soil pressure is generated beneath both footings.

The strap footing may be used instead of a rectangular or trapezoidal combined footing if the distance between columns is large and/or the allowable soil pressure is relatively large so that the additional footing area is not needed.

Figure 2.4 : Strap Foundation
2.1.3 : Continuous (Wall) Foundation

It is a continuous concrete footing under wall in all of its directions. It is used to transmit loads from wall into soil bearing layer. This type of footing takes the following shapes.

![Wall Footing](image)

Figure 2.5 : Wall Footing

2.1.4 : Strip Foundation

It is one footing carrying more than two columns on longitudinal line. This type of footing is as beam calculating for it shearing force diagram, bending moment diagram due to existing column load and soil pressure then design it as a reinforced concrete section to determine $d$ and $A_s$. If the footing is concentrically loaded, the pressure is uniform. If the column loads are not equal or not uniformly spaced, moments of the distribution determined assuming that the pressure varies uniformly.
In the longitudinal direction, the footing may be approximately analyzed for moments and shears by either of two following methods:

1. Assume a rigid foundation. Then the shear at any section is the sum of the forces on the side of the section, and the moment at the section is the sum of the moments of the forces on the side of the section.

2. Assume the strip is an inverted continuous beam where the columns are the support and the earth pressure causes distributed loading.

A more accurate analysis may be made be if the flexibility of the footing and the assumed elastic response of the soil are taken into account. The pressure distribution will be uniform.

2.1.5: Mat (Raft) Foundation

Mat foundation is a type of shallow foundation, mat foundation are a foundation system in which essentially the entire building is placed on a large continuous footing. Mat foundations found some use as early as the nineteenth century, and have continued to be utilized to effectively resolve special soil or design conditions. In locations where the soil is weak and bed rock is extremely deep, floating or
compensated mat foundations are sometimes utilized. For this type of foundation, the amount of soil removed and the resulting uplift (on the foundation) caused by ground water is equalized by the downwards forces of the building and foundation. Yet another variation of the mat foundation is to use it in combination with caissons or piles.

**Mats are used to:**

1- Distributes loads from the periphery of the structure over the entire area of the structure.

2- Reduce concentration of high contact pressure on soil.

3- Reduce hydraulic head or pressure of water.

Relative to static and strength calculations and design, mat foundation may be classed in two broad groups.

1- Mats designed by the conventional method of analysis Rigid mats (stiff raft).

This type is also called box structures made of cellular constructions or rigid frames consisting of slabs and basement walls. They are capable to resist very large flexure stresses. It is used when the
substructure cannot resist the different settlement developed under of flexible raft.

2- Mats designed on the basis of the theory of elasticity.

**Flexible Mats**

1. Flat plate with constant thickness (usually from 1-2m) with top and bottom two way reinforcement steel meshes. This type is most suitable where the column loads are small or moderate and the column spacing is fairly small and uniform. This type is easy to be constructed.

2. Flat plate thickened under columns, or plate with pedestals. This type is used for large column loads to provide sufficient strength for shear (punch) and negative moment.

3. Inverted slap with girders. It is used when the bending stresses becomes large because of large column spacing and unequal column loads. The disadvantages of this type is needs large amount soil foundation depth excavation.

**Some common mats foundation used.**

* Flat plate with uniform thickened.

* Flat plate thickened under column.
* Inverted slaps with girders.

Mats sometimes supported by piles in situation such as high ground water or where the soil is susceptible to large settlement, the piles help in reducing the settlement of the structure located over highly compressible soil.

Figure 2.6: Types of Mat Foundation
2.2 : Deep Foundation

Deep foundations are used for heavy structures and/or weak soils. The most common types of deep foundations is the pile foundation.

2.2.1 Pile Foundation

Pile foundations are the part of a structure used to carry and transfer the load of the structure to the bearing ground located at some depth below ground surface. The main components of the foundation are the pile cap and the piles. Piles are long and slender members which transfer the load to deeper soil or rock of high bearing capacity avoiding shallow soil of low bearing capacity. The main types of materials used for piles are Wood, steel and concrete. Piles made from these materials are driven, drilled or jacked into the ground and connected to pile caps. Depending upon type of soil, pile material and load transmitting characteristic piles are classified accordingly. In the following chapter we learn about, classifications, functions and pros and cons of piles.

Functions of piles

As with other types of foundations, the purpose of a pile foundations is:

- To transmit a foundation load to a solid ground
To resist vertical, lateral and uplift load

A structure can be founded on piles if the soil immediately beneath its base does not have adequate bearing capacity. If the results of site investigation show that the shallow soil is unstable and weak or if the magnitude of the estimated settlement is not acceptable a pile foundation may become considered. Further, a cost estimate may indicate that a pile foundation may be cheaper than any other compared ground improvement costs.

In the cases of heavy constructions, it is likely that the bearing capacity of the shallow soil will not be satisfactory, and the construction should be built on pile foundations. Piles can also be used in normal ground conditions to resist horizontal loads. Piles are a convenient method of foundation for works over water, such as jetties or bridge piers.

**Classification of piles**

**Classification of pile with respect to load transmission and functional behavior**

- End bearing piles (point bearing piles)
- Friction piles (cohesion piles)
Combination of friction and cohesion piles

End bearing piles

These piles transfer their load on to a firm stratum located at a considerable depth below the base of the structure and they derive most of their carrying capacity from the penetration resistance of the soil at the toe of the pile (see figure 1.1). The pile behaves as an ordinary column and should be designed as such. Even in weak soil a pile will not fail by buckling and this effect need only be considered if part of the pile is unsupported, i.e. if it is in either air or water. Load is transmitted to the soil through friction or cohesion. But sometimes, the soil surrounding the pile may adhere to the surface of the pile and causes "Negative Skin Friction" on the pile. This, sometimes have considerable effect on the capacity of the pile. Negative skin friction is caused by the drainage of the ground water and consolidation of the soil. The founding depth of the pile is influenced by the results of the site investigate on and soil test.

Friction or cohesion piles

Carrying capacity is derived mainly from the adhesion or friction of the soil in contact with the shaft of the pile.
Friction or cohesion piles

Carrying capacity is derived mainly from the adhesion or friction of the soil in contact with the shaft of the pile.

Cohesion piles

These piles transmit most of their load to the soil through skin friction. This process of driving such piles close to each other in groups greatly reduces the porosity and compressibility of the soil within and around the groups. Therefore piles of this category are sometimes called compaction piles. During the process of driving the pile into the ground, the soil becomes molded and, as a result loses some of its strength. Therefore the pile is not able to transfer the exact amount of load which it is intended to immediately after it
has been driven. Usually, the soil regains some of its strength three to five months after it has been driven.

**Friction piles**

These piles also transfer their load to the ground through skin friction. The process of driving such piles does not compact the soil appreciably. These types of pile foundations are commonly known as floating pile foundations.

![Friction Piles](image)

**Figure 2.8 : Friction Piles**

**Combination of friction piles and cohesion piles**

An extension of the end bearing pile when the bearing stratum is not hard, such as a firm clay. The pile is driven far enough into the lower material to develop adequate frictional resistance. A farther variation
of the end bearing pile is piles with enlarged bearing areas. This is achieved by forcing a bulb of concrete into the soft stratum immediately above the firm layer to give an enlarged base. A similar effect is produced with bored piles by forming a large cone or bell at the bottom with a special reaming tool. Bored piles which are provided with a bell have a high tensile strength and can be used as tension piles.

Figure 2.9: Combination of Friction and Cohesion Piles

Classification of pile with respect to type of material

- Timber
- Concrete
- Steel
- Composite piles

Factors affecting choice of pile

- Location and type of structure.
There are many factors that can affect the choice of a piled foundations. All factors need to be considered and their relative importance taken into account before reaching a final decision.

2.3 : Special Foundation

- Foundation for tall structure (smoke stacks, radio and television towers, light houses)
- Foundation for subsurface and overland pipe lines.
- Foundation for port and maritime structures.
- Machine foundation.
- Vehicular and aqueous tunnels.
- Foundation on elastic support.
- Other foundation of elastic support.
2.4: General Capacity of Soils

Bearing Capacity is the ability of a soil to support a load from foundation without causing a shear failure or excessive settlement. The sign of Bearing Capacity (B.C) and this units as pressure’s unit ton/m², KN/m², Kg/cm², lb/ft² etc... so can called Bearing Pressure.

Its known from observation of foundation subjected to load bearing capacity failure occurs usually as shear failure of the soil supporting the footing. The three principles modes of shear under foundation have been as general shear failure, local shear failure, and punching shear failure.

Consider a strip foundation resting on the surface of a dense or stiff cohesive soil, as shown in the figure (2.12), with a width of B now, if the load is gradually applied to the foundation, settlement will increase.

The variation the load per unit area on the foundation, q, with the foundation settlement is also shown in figure (2.10). At the certain point when the load per unit area equals q, a sudden failure in the soil supporting the foundation will occur and the failure in the soil will extend to the ground surface. This load per unit area, q, is usually referred to as the ultimate bearing capacity of the foundation. When this type of sudden failure takes place, it is called the general shear failure.

If the foundation under consideration rest on sand or clays soil of medium compaction figure (2.9), an increase of load on foundation will be also accompanied by an increase of settlement, however, in this case the failure in the soil will
gradually extend outward from the foundation, as shown by the solid lines, in figure (2.9). When the load per unit area on the foundation equals, $q_u$, the foundation movement will be accompanied by sudden jerks. The load per unit area at which this happens is the ultimate bearing capacity, $q_u$. Beyond this point, increase of the load will be accompanied by a large increase of foundation settlement. The load per unit area of the foundation, $q_u$, is referred to as the first failure load. Note that a peak value of $q$ is not realized in this type of failure, which is called the local shear failure of the soil.

If the foundation is supported by a fairly loose soil the load -- settlement plot will be like the one in figure (2.11). In this case, the failure surface in the soil will not extend to the ground surface. Beyond the ultimate failure load $q_u$, the load settlement plot will be steep and partially linear. This type of failure in the soil is called punching shear failure.

![Figure 2.9: Local Shear Failure](image)

![Figure 2.10: General Shear](image)
Factors affecting modes of failure

According to experimental results from foundation resting on sand, the mode of failure likely to occur in any situation depends on the size of the foundation and the relative density of the soil.

Other factors might be:

- Permeability: relating to drained / untrained behavior compressibility.
- Shape e.g. strips can only rotates one way.
- Interaction between adjacent foundations and other structures.
- Incidence and relative magnitude of horizontal loading or moments.
- Presence of stiffer or weaker underlying lyres.
2.4.1 : Terzaghi’s Bearing Capacity Theory

In 1948, Terzaghi proposed a well-conceived theory to determine the ultimate bearing capacity of a shallow rough rigid continuous (strip) foundation supported by a homogeneous soil layer extending to a great depth. Terzaghi defined a shallow foundation as a foundation where the width, B, is equal to or less than its depth, Df. The failure surface in soil at ultimate load (that is, $q_u$, per unit area of the foundation) assumed by Terzaghi is shown in Fig. Referring to Fig, the failure area in the soil under the foundation can be divided into three major zones. They are:

1. Zone $ACD$ This is a triangular elastic zone located immediately below the bottom of the foundation. The inclination of sides $AC$ and $BC$ of the wedge with the horizontal is $\phi = \theta$ (soil friction angle).

2. Zone $ADF$ and $CED$ This zone is the Prandtl's radial shear zone.

3. Zone $AFH$ and $CEG$. This zone is the Rankine passive zone. The slip lines in this zone make angles of $\pm(45 - \phi/2)$ with the horizontal.
Note that the replacement of the soil above the bottom of the foundation by an equivalent surcharge $q$, the shear resistance of the soil along the failure surface GI and HG was neglected.

Using the equilibrium analysis Terzaghi expressed the ultimate bearing capacity in the form.

$$q_u = cN_c + qN_q + \frac{1}{2} \gamma BN_f$$

(Strip Foundation) \hspace{1cm} (2.1)

Where $c =$ cohesion of soil

$\gamma =$ unit weight of soil

$Q = \gamma Df$
where $N_c$, $N_q$, and $N_\gamma$ = bearing capacity factors that are non-dimensional and they are only functions of the soil friction angle, the bearing capacity factors are defined by:

$$N_\chi = \chi \sigma \tau \phi (N\theta - 1)$$  \hspace{1cm} (2.2)

$$N\theta = \varepsilon^2 (3\pi/4 - \phi /2) \tau \alpha \nu \phi / [2 \chi \sigma 2 (45 + \phi /2)]$$  \hspace{1cm} (2.3)

$$N_\gamma = (1/2) \tau \alpha \nu \phi (K\pi / \chi \sigma^2 \phi - 1)$$  \hspace{1cm} (2.4)

For foundations that are rectangular or circular in plan, a plane strain condition in soil at ultimate load does not exist. Therefore, Terzaghi proposed the following relationships for square and circular foundations.

**Square footings:**

$$\theta_v = 1.3 \chi N_\chi + \gamma \Delta N\theta + 0.4 \gamma B N_\gamma$$  \hspace{1cm} (2.5)

**Circular footings:**

$$\theta_v = 1.3 \chi N_\chi + \Delta N\theta + 0.3 \gamma B N_\gamma$$  \hspace{1cm} (2.6)

It is obvious that Terzaghi’s bearing capacity theory was obtained by assuming general shear failure in soil. However, the local shear failure in soil, Terzaghi suggested the following relationships:

$$\theta_v = 2/3 N_\chi' + \theta N\theta' + 1/2 \gamma B N_\gamma'\hspace{1cm}\text{strip foundation}$$  \hspace{1cm} (2.7)

$$\theta_v = 0.867\chi N_\chi' + \theta N\theta' + 0.4 \gamma B N_\gamma'\hspace{1cm}\text{square foundation}$$  \hspace{1cm} (2.8)

$$\theta_v = 0.867 \chi N_\chi' + \theta N\theta' + 0.3 \gamma B N_\gamma'\hspace{1cm}\text{circular foundation}$$  \hspace{1cm} (2.9)
2.4.2 General Bearing Capacity Equation

The ultimate bearing capacity equations presented are for continuous, square, and circular foundation only. They do not address the case of rectangular foundations (0<B/L<1). Also, the equations do not take into account the shearing resistance along the failure surface in soil above the bottom of the foundation of the failure surface marked as GI and HJ. In addition, the load on the foundation may be inclined. To account for all these shortcomings, Meyerhof (1963) suggested the following form bearing capacity equation.

\[ q_u = C N_c F_{cs} F_{cd} F_{ci} + q N_q F_{qs} F_{qd} F_{qi} + \frac{1}{2} \gamma B N_{\gamma} F_{\gamma_s} F_{\gamma_d} F_{\gamma_i} \]  

(2.10)

Where

\( C \) = cohesion.

\( q \) = effective stresses at level of the bottom of the foundation.

\( \gamma \) = soil unit weight.

\( B \) = width or diameter for circular foundation.

\( F_{cs}, F_{qs}, F_{\gamma_s} \) = shape factors.

\( F_{ci}, F_{qi}, F_{\gamma_i} \) = load inclination factor.

\( F_{cd}, F_{qd}, F_{\gamma_d} \) = depth factors.
Note that the original equation for ultimate bearing capacity equation is derived only for the plane – strain case (that is, for continuous foundation). The shape, depth and load inclination factors based on experimental data.

2.4.3: Bearing Capacity Factors

Based on laboratory and field studies of bearing capacity, the basic of the failure surface in soil suggested by Terzaghi now appears to be correct (Vesic, 1973). However, the angle $\alpha$ is closer to $45 + \theta$. If this change is accepted, the value of $N_c$, $N_q$, and $N_\gamma$ for a given soil friction angle will also change from those give with $\alpha = 45 + \theta/2$. The relations for $N_c$, $N_q$ can be derived as

$$N_q = \tan^2 \left(45 + \frac{\phi}{2}\right) e^{\tan 2 \phi}$$  \hspace{1cm} (2.11)

$$N_c = (N_q - 1) \cot \phi$$ \hspace{1cm} (2.12)

The relation for $N_q$ was presented by Reissner (1924) Caquot. Kersal (1953) and Vesic (1973) gave the relation for $N_\gamma$ as

$$N_\gamma = 2(N_q - 1) \tan \phi$$ \hspace{1cm} (2.13)
Table shows the variation of the proceeding bearing capacity factors with soil friction angle.

![Table](image)

**Table 2.1 : Bearing Capacity Factors**
The relationship for shape. Depth and inclination factors, recommended for uses are shown in Table 2. Other relationships generally found in many texts and references.

<table>
<thead>
<tr>
<th>Factor</th>
<th>Relationship</th>
<th>Source</th>
</tr>
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<tbody>
<tr>
<td><strong>Shape</strong></td>
<td></td>
<td>De Beer (1970)</td>
</tr>
<tr>
<td></td>
<td>( F_a = 1 + \frac{B N_q}{L N_c} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( F_w = 1 + \frac{B}{L} \tan \phi )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( F_y = 1 - 0.4 \frac{B}{L} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>where ( L ) = length of the foundation ((L &gt; B))</td>
<td></td>
</tr>
<tr>
<td><strong>Depth</strong></td>
<td><strong>Condition (a):</strong> ( D_f / B \leq 1 )</td>
<td>Hansen (1970)</td>
</tr>
<tr>
<td></td>
<td>( F_{cd} = 1 + 0.4 \frac{D_f}{B} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( F_{qd} = 1 + 2 \tan \phi(1 - \sin \phi)^2 \frac{D_f}{B} )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( F_{yd} = 1 )</td>
<td></td>
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<tr>
<td></td>
<td><strong>Condition (b):</strong> ( D_f / B &gt; 1 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( F_{cd} = 1 + (0.4) \tan^{-1} \left( \frac{D_f}{B} \right) )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( F_{qd} = 1 + 2 \tan \phi(1 - \sin \phi)^2 \tan^{-1} \left( \frac{D_f}{B} \right) )</td>
<td></td>
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<tr>
<td></td>
<td>( F_{yd} = 1 )</td>
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<tr>
<td><strong>Inclination</strong></td>
<td></td>
<td>Meyerhof (1963); Hanna and Meyerhof (1963)</td>
</tr>
<tr>
<td>( F_a = F_w )</td>
<td>( \left( 1 - \frac{B \cos \phi}{2 \tan \phi} \right)^2 )</td>
<td></td>
</tr>
<tr>
<td>( F_y )</td>
<td>( \left( 1 - \frac{B}{2 \phi} \right)^2 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>where ( \phi ) = inclination of the load on the foundation with respect to the vertical</td>
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</tbody>
</table>

Table 2.2 : Shape depth and inclination factors
2.5 : Soil Settlement

In design of most foundations, there are specifications for allowable levels of settlement. Refer to Fig.(2.3) which is a plot of load per unit area $q$ versus settlement $S$ for a foundation. The ultimate bearing capacity is realized at a settlement level of $S_u$. Let $S_{all}$ be the allowable level of settlement for the foundation and $q_{all(S)}$ be the corresponding allowable bearing capacity. If $FS$ is the factor of safety against bearing capacity failure, then the allowable bearing capacity is $q_{all(b)} = q_u / FS$.

![Figure 2.14 : Load settlement curve for shallow foundation](image)

The settlement corresponding to $q_{all(b)}$ is $S'$. For foundations with smaller widths of $B$, $S'$ may be less than $S_{all}$; however, for larger values of $B$, $S_{all} < S'$. Hence, for smaller foundation widths, the bearing capacity controls and, for larger foundation widths, the
allowable settlement controls. This section describes the procedures for estimating the settlement of foundations under load.

2.5.1 : Total and Differential Settlements

Settlement due to consolidation of the foundation soil is usually the most important consideration in the calculating the serviceability limit state or in assessing allowable bearing pressers where permissible stress methods are used.

Even though sinking of foundation as a result of shear failure of the soil been safeguarded by ultimate limit state calculations or by applying arbitrary safety factor.

on the calculated ultimate bearing capacity, it is still necessary to investigate the likelihood of settlements before the allowable bearing pressure can be fixed. In the following pages consideration will be given to the causes of settlement, the effects on the structure of total and differential settlement, methods of estimating settlement, and the design of the foundation to eliminate settlement or to minimize its effects.

Any structure built on soil is subject to settlement. Some settlement is inevitable and, depending on the situation, some settlements are tolerable. When building structures on top of soils, one needs to have some knowledge of how settlement occurs and predict how much and how fast settlement will occur in a given situation.

Important factors that influence settlement:

- Soil Permeability
- Soil Drainage
• Load to be placed on the soil
• History of loads placed upon the soil (normally or over-consolidated)
• Water Table

Settlement is caused both by soil compression and lateral yielding (movement of soil in the lateral direction) of the soils located under the loaded area. Cohesive soils usually settle from compression while cohesion less soils often settle from lateral yielding - however, both factors may play a role. Some other less common causes of settlement include dynamic forces, changes in the groundwater table, adjacent excavations, etc. Compressive deformation generally results from a reduction in the void volume, accompanied by the rearrangement of soil grains. The reduction in void volume and rearrangement of soil grains is a function of time. How these deformations develop with time depends on the type of soil and the strength of the externally applied load (or pressure). In soils of high permeability (e.g. coarse-grained soils), this process requires a short time interval for completion, and almost all settlement occurs by the time construction is complete. In low permeable soils (e.g. fine-grained soils) the process occurs very slowly. Thus, settlement takes place slowly and continues over a long period of time. In essence, a graph of the void ratio as a function of time for several different applied loads provides an enormous amount of information about the settlement characteristics of a soil.
2.5.2 : Types of Settlements

The settlement of a structural foundation consists of three parts:

1. The immediate settlement ($\rho_i$) this settlement takes place during application of loading as a result of elastic deformation of the soil without change in water content.

2. The consolidation settlement ($\rho_c$) this settlement takes place as a result of volume reduction of the soil caused by extrusion of some of the pore water from the soil.

3. The Creep or secondary settlement ($\rho_\alpha$) this settlement occurs over a very long period of years after completing the extrusion of excess pore water. It's caused by the viscous resistance of the soil particle to adjustment under compression.

4. the final settlement ($\rho_f$) this settlement is the sum of the ($\rho_i$) and ($\rho_\alpha$)

If deep excavation is required to reach the foundation level, swelling of the soil will take place as a result of removal of the pressure of the overburden. The magnitude of the swelling is depending on the depth of overburden removed and the time of foundations remains unloaded.
2.5.3 Estimation of Settlement of Foundation on Sand and Gravels

Settlements of foundation on sand, gravel and granular fill materials, takes place almost immediately as the foundation loading is imposed on them. Because of the difficulty of sampling these soils, there is no practicable laboratory test procedure for determining their consolidation characteristics.

From a review of a number of case records of the settlement of structures founded on these soils, Sutherland concluded that there is no reliable method for extrapolating the settlement of a standard plate to the settlement of an actual foundation at the same location. Consequently settlements of foundations on sand and gravel are estimated by semi-empirical method based on SPT or CPT values or on the results of pressure meter test. There are many tests to calculate the value of settlement in sand and gravel

1. Estimation of settlement from standard penetration tests
2. Estimation of settlement from static cone penetration tests
3. Estimation of settlement from pressure meter tests

* Estimation of settlement from standard penetration tests

Berland and Burbidge establish an empirical relationship based on the standard penetration test from which the settlement of foundation on sand and gravel can be calculated from the equation
2.5.4 Calculation of Secondary Consolidation Settlement

1) Secondary consolidation is the portion of time-dependent settlement that occurs at essentially constant effective stress.

2) The rate of secondary consolidation is not dependent on the flow of water or on clay layer thickness, and is relatively constant for normal engineering stress increases.

3) Secondary consolidation occurs slowly with a continually decreasing rate.

4) The secondary consolidation portion of the consolidation curve is approximately linear on an e-log time plot.

5) Secondary consolidation usually estimated from a lab consolidation test.

6) The exact cause of secondary consolidation is unknown, but is possibly a readjustment of the double water layer surrounding the clay particles.

7) Secondary consolidation may be more important than primary consolidation for organic and highly compressible inorganic clays.

8) The ratio of secondary to primary consolidation increases as the ratio of stress increment to initial stress decreases: i.e. watch out for small stress increases in thick clay layers.

\[ \frac{\sigma_s}{\sigma_c} \uparrow \; \text{as} \; \frac{\Delta \sigma_s}{\sigma_0} \downarrow \]
2.5.5 Settlement of Piles

The elastic settlement of a pile under a vertical working load, $Q_w$, is determined by three factors:

$$S_e = S_{e(1)} + S_{e(2)} + S_{e(3)}$$

(2.14)

Where $S_e$ = the total settlement in the pile

$S_{e(1)}$ = settlement of pile shaft

$S_{e(2)}$ = settlement of pile caused by the load at the pile point

$S_{e(3)}$ = settlement of pile caused by the load transmitted along the pile shaft

**Determination of $S_{e(1)}$**

If the pile material is assumed to be elastic, the deformation of the pile shaft can be evaluated using the fundamental principles of mechanics of materials:

$$S_{e(1)} = \frac{(Q_{wp} + \xi Q_{ws})L}{A_p E_p}$$

(2.15)

Where $Q_{wp}$ = load carried at the pile point under working load condition

$Q_{ws}$ = load carried by frictional (skin) resistance under working load condition

$A_p$ = area of the pile cross section

$L$ = length of the pile

$E_p$ = modulus of elasticity of the pile material
The magnitude of $\xi$ depends on the nature of the unit friction (skin) resistance distribution along the pile shaft ($f$). If the distribution of $f$ is uniform or parabolic, as shown in figures 2.19 a and b, $\xi = 0.5$. However, for a triangle distribution of $f$ (fig 2.19 c) the magnitude of $\xi$ is about 0.67.

**Fig 2.19 :** Various types of unit friction (skin) resistance distribution along the pile shaft

**Determination of $S_e^{(2)}$**

The settlement of a pile caused by the load carried at the pile point may express as:

$$S_e^{(2)} = \frac{q_{wp} D}{E_s} (1 - \mu_s^2) I_{wp}$$  \hspace{1cm} (2.16)

Where $D =$ width or diameter of the pile
\[ q_{wp} = \text{point load per unit area at the pile point} = \frac{Q_{wp}}{A_p} \]

\[ E_s = \text{modulus of elasticity of soil at or below the pile point} \]

\[ \mu_s = \text{Poisson’s ratio of soil} \]

\[ I_{wp} = \text{influence factor} = 0.85 \]

**Determination of \( S_e \)\(^{(3)}\)**

The settlement of a pile caused by the load carried along the pile shaft is given by a relation similar to Eq. (28) or

\[
S_{e(3)} = \left( \frac{Q_{ws}}{pL} \right) \frac{D}{E_s} \left( 1 - \mu_s^2 \right) I_{ws} \quad \text{..................} \quad (2.17)
\]

Where

\( p = \) perimeter of the pile

\( L = \) embedded length of the pile

\( I_{ws} = \) influence factor

Note that the term \( \frac{Q_{ws}}{pL} \) in Eq. 29 is the average value of \( f \) along the pile shaft.

The influence factor \( I_{ws} \), has a simple empirical relation

\[ I_{ws} = 2 + 0.35 \sqrt{\frac{L}{D}} \]
CHAPTER THREE
DESIGN AND CALCULATION

3.1 Calculation of loads on columns

Density:
Sand = 1.6t/m^3
Mortar and tiles = 2.5t/m^3
Concrete = 2.5t/m^3
Hollow block = 1.2t/m^3
Plaster = 2.5t/m^3
Ru = 0.52cm^2

For calculated wu:
* Mortar and tiles = (0.05*2.5*0.52*1) = 0.065 t/Ru
* sand = (0.1*1.6*0.52*1) = 0.0832 t/Ru
* hollow block = (0.4*0.17*1.2*1) = 0.0816 t/Ru
* concrete = ((0.08*0.52)+(0.12*0.17))*2.5*1 = 0.155 t/Ru
* plaster = (0.02*2.5*0.52*1) = 0.026 t/Ru
* Σ = 0.4108 t/Ru
→ for (1m) = 0.4108/0.52 = 0.79t/m^2
DL = portioning + material load
→ 0.125 + 0.79 = 0.915 t/m^2
L.L = 0.3t/m^2 for class room
L.L = 0.5t/m^2 for corridors
Wu for class room

Wu = (1.2*0.915)+(1.6*0.3) = 1.578t/m^2
Wu = (1.2*0.915)+(1.6*0.5) = 1.898t/m^2

* For column one

(1.35*1.6*1.578*4)+((0.25*1.04*2.5*1.6*4)+(1.6*0.2*1.2*16.2)+
(0.1*1.6*17*2.5)+(1.35*0.25*1.04*2.5*4)+(1.35*16.2*0.2*1.2)+
(0.3*0.5*16.2*2.5)+(0.1*1.35*17*2.5))*1.2= 57.86ton

* for column two

(4.725*2.7*1.578*4) +((2.7*0.25*1.04*2.5*4)+(4.725*0.25*0.7*4*2.5)
+ (2.7*0.2*16.2*1.2)+(0.1*2.7*17*2.5)+(0.3*0.5*16.2*2.5)+(0.02*2.7*
16.2*2.5))*1.2 = 132.5 ton

* for column three

(2.7*3.125*1.578*4)+((2.7*0.25*1.04*4*2.5)+(3.125*0.25*0.74*2.5)
+(0.2*2.7*16.2*1.2)+(0.1*2.7*17*2.5)+(0.02*2.7*13.76*2.5)+
(0.02*2*3.125*13.76*2.5)+(0.3*0.25*16.2*2.5)+(0.3*0.25*17*2.5))*
1.2 = 105 ton
3.2 Design of Footings

We designed footings as combined when we look to the allowable bearing capacity of the soil we find that it is moderate and the design of combined footing is possible and more economical than others, and those footings two of it has two columns and it designed manually, but the rest has many of columns and it designed by using SAP program.

3.2.1 Design of combined footings

This footing has two columns and it designed manually as shown below.

Q1 = 39.28 ton
Q2=39.28 ton
L3 = 3.35 m

Area = \((Q1+Q2)/ q_{all}\)
\( \text{Area} = \frac{(39.28 + 39.28)}{20} \)
\[ \text{Area} = 3.93 \text{ m}^2 \]

Take moment at A:
\( 39.28 \times 3.35 = (Q1+Q2) \times X \)
\[ X = 1.675 \text{ m} \]

\[ L = (1.675 + .25) \times 2 \]
\[ L = 3.85 + .55 = 4.4 \]
\[ L = 4.4 \text{ m} \]

\[ B = \frac{3.93}{4.4} \]
\[ B = .89 \]

\[ B = 1 \text{ m} \]
\[ L_2 = L_1 = .525 \text{ m} \]
• Punching Shear :

$$41.88 - 25 * (0.25 + d) = V_u$$

$$0.75 * 0.35 * \sqrt{240} * 10 * 10 * 1 * d = V_c$$

$$V_u = \Phi V_c$$

$$\Rightarrow d = 41 \text{ cm}$$

$$\Rightarrow h = 50 \text{ cm}$$
Checking Punching shear:

\[ C \# 1 : \]

\[ P_{up} = 55 - 25 \left( 0.5 + 0.41/2 \right) \left( 0.5 + 0.41 \right) \]

\[ = 46.98 \]

\[ \phi V_c = 0.75 \times 1.06 \times \sqrt[2]{240 \times 10 \times \left( 2 (0.5 + 0.41/2) + 0.5 + 0.41 \right)} \times 0.41 \]

\[ \Rightarrow 117.15 > 46.98 \text{ so it's ok} \]

\[ C \# 2 : \]

The same calculation for \( C \# 1 \)

- **Design of flexure**:

\[ M_u = 31.63 \]

\[ \rho = 0.85 \times 240/4200 \left( 1 - \sqrt{1 - \left( 2.61 \times 10^5 \times 31.63/100 \times 41^2 \times 240 \right)} \right) \]

\[ = 5.25 \times 10^{-3} \]

\[ \rho \text{ min} = 14/fy = 14/4200 = 3.33 \times 10^{-3} \]

\[ \rho > \rho \text{ min} \]
\[ A_s = \rho \times b \times d \]

\[ A_s = 21.525 \text{ cm}^2 \]

\[ \therefore \text{Use } 11 \varnothing 16 \]

**Bottom steel**

\[ M_u = 3.44 \]

\[ \rho = 0.85 \times 240 / 4200 \times (1 - \sqrt{1 - (2.61 \times 10^5 \times 3.44 / 100 \times 41^2 \times 240)}) = 5.43 \times 10^{-4} \]

\[ \rho_{\text{min}} = 14 / f_y = 14 / 4200 = 3.33 \times 10^{-3} \]

\[ \rho < \rho_{\text{min}} \]

\[ \therefore \text{Use } \rho_{\text{min}} \]

\[ A_s = \rho \times b \times d \]

\[ A_s = 13.53 \text{ cm}^2 \]

\[ \therefore \text{Use } 7 \varnothing 16 \]

\[ A_s \text{ shrinkage} = 0.0018 \times 100 \times 40 \]

\[ = 9 \text{ cm}^2 \]

\[ \therefore \text{Use Stirrup } 11 \varnothing 16 \]
• Elastic settlement

\[ p_i = q_i B \left(1 - \mu_s^2/E_s \right) I_p \quad \text{eq(6.7) from principle of geotechnical engineering book} \]

\[ q_{\text{load}} = \frac{(55 + 55)}{(4.4 \times 1)} \]
\[ q_{\text{load}} = 25 \text{ ton/m}^2 \]

\[ q_{\text{soil}} = 20 \text{ ton/m}^2 \]
\[ q_{\text{net}} = 25 - 20 \]
\[ = 5 \text{ ton/m}^2 \]
\[ q_{\text{net}} = 50 \text{ KN/m}^2 \]

\[ \mu_s = 0.2 \quad \text{……. From table (6.6) from principle of geotechnical engineering book} \]

\[ E_s = 2415 \text{ KN/m}^2 \quad \text{……. From table (6.5) from principle of geotechnical engineering book} \]
\[ I_p = 2 \quad \ldots \quad \text{From table (6.4) from principle of geotechnical engineering book} \]

\[ \rightarrow \text{from eq (6.7)} \]

\[ p_i = 0.039 \text{ m} \]

The calculated settlement is considered high. In order to reduce this value, certain measures can be undertaken including soil improvement by various methods. These methods may be soil excavation and replacement of better soil, soil stabilization by compaction and other methods.
3.2.2 Design of Combined footings (continuos)

The following footings has several columns and it designed by using SAP program to analysis and design all of the combined footings below.

A. Combined Footing (F2)

Area of footings = 798.2/20
⇒ Area = 39.91 m²

$$\sum M_A = 0$$
$$41.3*27 + 94.6*24.3 + 94.6*21.6 + 75.2*18.9 + 75.2*16.2$$
$$+ 75.2*13.5 + 75.2*10.8 + 75.2*8.1 + 75.2*5.4 + 75.2*2.7 = 798.2 * x$$

⇒ 11142.36 = 798.2 * x

∴ X = 13.96 m

∴ L = (13.96 + 0.125)*2
⇒ L = 28.17 m

∴ Take L = 28.5 m
B = Area / L  
B = 39.91/28.5

∴ B = 1.4 m

- **Punching Shear:**
  
The same calculations as combined footing before:

  \[ V_u = \phi V_c \]

  ➔ d = 41 cm
  ➔ h = 50 cm

- **Design of flexure:**

**Bottom Steel**

From SAP:

For Column # 60 & # 61
\[ M_u = 60 \text{ t.m} \]

\[ \rho = 0.85 \times \frac{240}{4200} \left(1 - \sqrt{1 - \left(\frac{2.61 \times 10^5 \times 60}{100 \times 41^2 \times 240}\right)}\right) = 10.5 \times 10^{-3} \]

\[ \rho_{\text{min}} = \frac{14}{f_y} = \frac{14}{4200} = 3.33 \times 10^{-3} \]

\[ \rho > \rho_{\text{min}} \]

Use \( \rho \)

\[ A_s = \rho \times b \times d \]

\[ A_s = 0.0105 \times 140 \times 41 \]

\[ A_s = 60.27 \text{ cm}^2 \]

\[ \therefore \text{Use } 13 \varnothing 25 \]

For other columns:
\[ M_u = 32 \text{ t.m} \]

\[ \rho = 0.85 \times \frac{240}{4200} \left(1 - \sqrt{1 - \left(\frac{2.61 \times 10^5 \times 32}{100 \times 41^2 \times 240}\right)}\right) = 5.26 \times 10^{-3} \]

\[ \rho_{\text{min}} = \frac{14}{f_y} = \frac{14}{4200} = 3.33 \times 10^{-3} \]

\[ \rho > \rho_{\text{min}} \]

Use \( \rho \)

\[ A_s = \rho \times b \times d \]

\[ A_s = 0.00526 \times 140 \times 41 \]

\[ A_s = 30.2 \text{ cm}^2 \]

\[ \therefore \text{Use } 10 \varnothing 20 \]

**Top Steel**
\[ M_u = 10 \text{ t.m} \]
\[ \rho = 0.85 \times \frac{240}{4200} (1 - \sqrt{1 - \frac{(2.61 \times 10^5 \times 10}{100 \times 41^2 \times 240})} = 1.45 \times 10^{-3} \]
\[ \rho_{min} = \frac{14}{fy} = \frac{14}{4200} = 3.33 \times 10^{-3} \]
\[ \rho < \rho_{min} \]

Use \( \rho_{min} \)

\[ A_s = \rho \times b \times d \]
\[ A_s = 0.00333 \times 140 \times 41 \]
\[ A_s = 18.94 \text{ cm}^2 \]

\[ \therefore \text{Use} \ 10 \, \varnothing \, 16 \]

\[ A_s \text{ shrinkage} = 0.0018 \times 140 \times 50 \]
\[ = 18.94 \text{ cm}^2 \]

\[ \therefore \text{Use} \ 9 \, \varnothing \, 12 \]

\[ \therefore 2 \, \varnothing \, 12 / \, 31 \text{ cm} \]
B. Combined Footing (F1)

Reinforcement of combined footing (F2)

Combined Footing (F1)
Area of footings = 672.2/20
Area = 31.36 m²

\[ \sum M_A = 0 \]
\[ 41.3 \times 21.6 + 94.6 \times 18.9 + 75 \times 16.2 + 75 \times 13.5 + 75 \times 10.8 + 75 \times 8.1 + 75 \times 5.4 + 75 \times 2.7 = 627.2 \times x \]

6932.52 = 627.2 \times x
\[ \therefore X = 11.05 \text{ m} \]

\[ \therefore L = (11.05 + 0.125) \times 2 \]
L = 22.35 m

\[ \therefore \text{Take } L = 23 \text{ m} \]

B = Area / L
B = 31.63/23

\[ \therefore B = 1.4 \text{ m} \]
• Punching Shear :

The same calculation as combined footing before :

\[ V_u = 0.75 \cdot V_c \]

\[ d = 41 \text{ cm} \]

\[ h = 50 \text{ cm} \]

• Design of flexure :

**Bottom Steel**

From SAP :

For Column 2
\[ M_u = 64 \text{ t.m} \]
\[ \rho = 0.85 \cdot \frac{240}{4200} \cdot (1 - \sqrt{1 - (2.61 \cdot 10^5 \cdot 64 / 100 \cdot 41^2 \cdot 240)}) = 11 \cdot 10^{-3} \]
\[ \rho_{\text{min}} = 14 / f_y = 14 / 4200 = 3.33 \cdot 10^{-3} \]
\[ \rho > \rho_{\text{min}} \]

Use \( \rho \)

\[ A_s = \rho \cdot b \cdot d \]
\[ A_s = 0.011 \cdot 140 \cdot 41 \]
\[ A_s = 63.14 \text{ cm}^2 \]

:. Use 13 Ø 25

For other columns :
\[ M_u = 31 \text{ t.m} \]
\[ \rho = 0.85 \cdot \frac{240}{4200} \cdot (1 - \sqrt{1 - (2.61 \cdot 10^5 \cdot 31 / 100 \cdot 41^2 \cdot 240)}) = 5.25 \cdot 10^{-3} \]
\( \rho_{\text{min}} = \frac{14}{f_y} = \frac{14}{4200} = 3.33 \times 10^{-3} \)

\( \rho > \rho_{\text{min}} \)

Use \( \rho \)

\[ A_s = \rho \times b \times d \]

\[ A_s = 0.00525 \times 140 \times 41 \]

\[ A_s = 30.13 \text{ cm}^2 \]

\( \therefore \) Use 10 \( \varnothing \) 20

**Top Steel**

\( M_u = 9 \text{ t.m} \)

\( \rho = 0.85 \times 240 / 4200 (1 - \sqrt{1 - (2.61 \times 10^5 \times 9 / 100 \times 41^2 \times 240)} = 1.43 \times 10^{-3} \)

\( \rho_{\text{min}} = \frac{14}{f_y} = \frac{14}{4200} = 3.33 \times 10^{-3} \)

\( \rho < \rho_{\text{min}} \)

Use \( \rho_{\text{min}} \)

\[ A_s = \rho_{\text{min}} \times b \times d \]

\[ A_s = 0.00333 \times 140 \times 41 \]

\[ A_s = 18.94 \text{ cm}^2 \]

\( \therefore \) Use 10 \( \varnothing \) 16

\( A_s \) shrinkage = 0.0018 \( \times \) 140 \( \times \) 50

\[ = 18.94 \text{ cm}^2 \]

\( \therefore \) Use 9 \( \varnothing \) 12

\( \therefore 2 \varnothing 12 / 31 \text{ cm} \)
C. Combined Footing (F3)

Reinforcement of combined footing (F1)

Combined Footing (F3)
Area of footings = $\frac{608.6}{20}$

$\Rightarrow$ Area = $30.43 \text{ m}^2$

$\sum M_A = 0$

$109.9 \times 14.4 + 129.6 \times 10.8 + 129.6 \times 7.2 + 129.6 \times 3.6 = 608.6 \times x$

$\therefore X = 7.2 \text{ m}$

$\therefore L = (7.2 + 0.125) \times 2$

$L = 14.65 \text{ m}$

$\therefore$ Take $L = 15.5 \text{ m}$

$B = \frac{\text{Area}}{L}$

$B = \frac{30.43}{15.5}$

$\therefore B = 2 \text{ m}$
- **Punching Shear**:  
  
  The same calculation as combined footing before:  
  
  \[ V_u = 0 \cdot V_c \]
  
  \[ d = 41 \text{ cm} \]
  
  \[ h = 50 \text{ cm} \]

- **Design of flexure**:  
  
  **Bottom Steel**  
  
  From SAP:  
  
  For Column 32  
  \[ M_u = 64 \text{ t.m} \]
  
  \[ \rho = 0.85 \times 240 / 4200 \times (1 - \sqrt{1 - (2.61 \times 10^5 \times 64 / 100 \times 41^2 \times 240)}) = 11 \times 10^{-3} \]
  
  \[ \rho_{\text{min}} = 14 / f_y = 14 / 4200 = 3.33 \times 10^{-3} \]
  
  \[ \rho > \rho_{\text{min}} \]
  
  Use \( \rho \)
  
  \[ A_s = \rho \times b \times d \]
  
  \[ A_s = 0.011 \times 200 \times 41 \]
  
  \[ A_s = 90.2 \text{ cm}^2 \]
  
  \[ \therefore \text{Use} \ 19 \varnothing 25 \]
  
  For other columns:  
  \[ M_u = 45 \text{ t.m} \]
  
  \[ \rho = 0.85 \times 240 / 4200 \times (1 - \sqrt{1 - (2.61 \times 10^5 \times 45 / 100 \times 41^2 \times 240)}) = 7.67 \times 10^{-3} \]
\[ \rho_{\text{min}} = \frac{14}{f_y} = \frac{14}{4200} = 3.33 \times 10^{-3} \]

\[ \rho > \rho_{\text{min}} \]

Use \( \rho \)

\[ A_s = \rho \times b \times d \]

\[ A_s = 0.00767 \times 200 \times 41 \]

\[ A_s = 62.89 \text{ cm}^2 \]

\[ \therefore \text{Use } 13 \varnothing 25 \]

**Top Steel**

\[ M_u = 33 \text{ t.m} \]

\[ \rho = \frac{.85 \times 240}{4200} \left(1 - \sqrt{1 - \left(2.61 \times 10^5 \times 33 / 100 \times 41^2 \times 240\right)}\right) = 5.49 \times 10^{-3} \]

\[ \rho_{\text{min}} = \frac{14}{f_y} = \frac{14}{4200} = 3.33 \times 10^{-3} \]

\[ \rho > \rho_{\text{min}} \]

Use \( \rho \)

\[ A_s = \rho \times b \times d \]

\[ A_s = 0.00549 \times 200 \times 41 \]

\[ A_s = 45.01 \text{ cm}^2 \]

\[ \therefore \text{Use } 15 \varnothing 16 \]

\( A_s \) shrinkage = \[ 0.0018 \times 200 \times 50 \]

\[ = 18 \text{ cm}^2 \]

\[ \therefore \text{Use } 16 \varnothing 12 \]

\[ \therefore 2 \varnothing 12 / 31 \text{ cm} \]
3.3 Design of Shear wall:

The ultimate axial load on walls is from the loads from adjoining slab (dead & live load), reactions of beams, in addition to own weight of wall.

\[ W_{beam} = 0.25 \times 0.70 \times 8.2 \times 2.5 \]
\[ = 3.5875 \text{ ton} \]

Load on shear wall (SH1) “Pu”

\[ P_u = (1.755 \times 1.8 \times 8.2 \times 0.25 \times 4) + (3.5875 \times 4) \times 1.2 + 1.2 \times (0.02 \times 2.5 \times 8.2 \times 16.2) + 1.85 \times 0.25 \times 6.9 \times 1.775 + 1.2 \times (0.02 \times 1.85 \times 6.9 \times 2.5 \times 16.8) \]

\[ \Rightarrow P_u = 70 \text{ ton} \]

According to ACI cpde (14.5)

\[ \Phi P_{nw} = 0.55 \Phi f' c A_g [1 - (K L_c / 32 h)^2] \]

\( \Phi P_{nw} \): nominal axial load strength of wall.
\( A_g \): gross area of section.
\( h \): over all thickness of member.
\( \Phi \): strength reduction factor which equal (0.7)

\( L_c \): vertical distance between supports.
\( K \): effective length factor, which less than 1, conservatively we will consider that \( K = 1 \)

\[ \Rightarrow \Phi P_{nw} = 0.55 \Phi f' c A_g [1 - (K L_c / 32 h)^2] \]

\[ \Phi P_{nw} = 0.55 \times 0.7 \times 240 \times (6.9 \times 0.3) \times [1 - (4 / 32 \times 0.3)^2] \]
= 158.06 ton > 70 ton → so it’s safe

- Vertical reinforcement:

  According to ACI code (14.3.2) minimum ratio of vertical reinforcement area to gross concrete area shall be 0.0012

  \[ A_s = \rho \times b \times d \]

  \[ A_s = 0.0012 \times 100 \times 30 \]

  \[ A_s = 3.6 \text{ cm}^2 \]

  :: Use Ø 14 / 40 cm

- Horizontal reinforcement:

  According to ACI code (14.3.3) minimum ratio of horizontal reinforcement area to gross concrete area shall be 0.002

  \[ A_s = \rho \times b \times d \]

  \[ A_s = 0.002 \times 100 \times 30 \]

  \[ A_s = 6 \text{ cm}^2 \]

  :: Use Ø 14 / 25 cm
3.4 Design of a Retaining wall:

\[ \gamma_1 = 18 \text{ kN/m}^2 \]
\[ C_1 = 0 \]
\[ \phi_1 = 30 \]

\[ \gamma_2 = 13.9 \text{ kN/m}^2 \]
\[ \phi_2 = 20 \]
\[ C_2 = 40 \]

(Retaining Wall)
\[ H' = 4 + 0.4 + 1.8 \tan 10^\circ = 4.68 \text{ m} \]

From table Active earth pressure coefficient \( K_a \)
\[ \alpha = 10^\circ, \quad \phi = 30^\circ \]
\[ \Rightarrow K_a = 0.35 \]

\[ P_a = 0.5 \times H'^2 \times \gamma \times K_a \]

\[ P_a = 0.5 \times 4.68^2 \times 18 \times 0.35 \]

\[ P_a = 68.99 \text{ KN/m} \]

\[ P_v = P_a \times \sin 10^\circ \]

\[ P_v = 11.97 \text{ KN/m} \]

\[ P_h = P_a \times \cos 10^\circ \]

\[ P_h = 67.94 \text{ KN/m} \]

From the figure “Retaining wall” on the previous page, the calculations as shown in the table below:

<table>
<thead>
<tr>
<th>Section No.</th>
<th>Area</th>
<th>Weight/ Unit length (KN/m)</th>
<th>Moment Arm point C</th>
<th>Moment (KN.m/m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.2</td>
<td>30</td>
<td>0.65</td>
<td>19.5</td>
</tr>
<tr>
<td>2</td>
<td>0.2</td>
<td>5</td>
<td>0.466</td>
<td>2.33</td>
</tr>
<tr>
<td>3</td>
<td>1.64</td>
<td>24</td>
<td>1.3</td>
<td>28.8</td>
</tr>
<tr>
<td>4</td>
<td>7.2</td>
<td>115.2</td>
<td>1.8</td>
<td>184.32</td>
</tr>
<tr>
<td>5</td>
<td>0.224</td>
<td>4.03</td>
<td>1.86</td>
<td>7.49</td>
</tr>
</tbody>
</table>

\[ P_v = 11.97 \]

\[ \sum = 190.2 \]

\[ \sum M_R = 242.44 \]
• The Overturning moment:

\[ M_0 = P_h \times (H/3) \]

\[ M_0 = 67.94 \times (4.68/3) \]
\[ M_0 = 105.98 \text{ KN/m} \]

\[ \text{FS}_{\text{overturning}} = \sum M_R / M_0 \]

\[ \text{FS}_{\text{overturning}} = 2.28 > 2 \text{ it's okay} \]

• The Sliding:

\[ K_p = \tan^2 (45 + \frac{\theta}{2}) \]
\[ = 2.04 \]

\[ D = 1.5 \text{ m} \]

\[ P_p = 0.5 \times K_p \times \gamma_2 \times D^2 + 2 \times C_2 \times \sqrt{K_p} \times D \]
\[ = 0.5 \times 2.04 \times 13.9 \times 1.5^2 + 2 \times 40 \times \sqrt{2.04} \times 1.5 \]

\[ \Rightarrow P_p = 203.29 \text{ KN/m} \]

\[ \text{FS}_{\text{sliding}} = \frac{\left( \sum v \times \tan k_1 \theta_1 + B \times K_2 \times C_2 + P_p \right)}{P_a \times \cos 10^\circ} \]

\[ K_1, K_2 : \text{range 0.5 → 2/3} \]
Let \( K_1 = K_2 = 2/3 \)

\[ \Rightarrow \text{FS}_{\text{sliding}} = \frac{(190.2 \times \tan 20^\circ \times 2/3 + (2.6 \times 2/3 \times 40 + 203.29)}{68.99 \times \cos 10^\circ} \]

\[ \Rightarrow \text{FS}_{\text{sliding}} = 4.6 > 1.5 \text{ it's okay} \]
• The safety against bearing capacity:

\[ e = B/2 - \left( \sum M_R - \sum M_o \right)/\sum V \]

\[ e = 0.486 > B/6 \]

\[ \therefore \text{It's safe} \]

\[ q_{\text{toe, heel}} = \sum V/B \left( 1 \pm 6 \frac{e}{B} \right) \]

\[ q_{\text{toe}} = 147.3 \text{ KN/m}^2 \]

\[ q_{\text{heel}} = 11.61 \text{ KN/m}^2 \]

\[ q_u = 2 \text{ kg/cm}^2 \rightarrow 196.2 \text{ KN/m}^2 \]
A. Design of Stem:

Max moment = $68.99 \times \frac{4}{3}$

Max moment = 91.98 KN.m

Max moment = 9.2 ton .m

$M_u = (1.6) \times (9.2)$
\[ M_u = 14.72 \text{ ton.m} \]

\[ \rho = 0.85 \times 240 / 4200 (1 - \sqrt{1 - (2.61 \times 10^5 \times 14.72 / b \times d^2 \times f'_c)} = 3.68 \times 10^{-3} \]

\[ \rho \text{ min} = 14 / f_y = 14 / 4200 = 3.33 \times 10^{-3} \]

\[ \rho > \rho \text{ min} \]

Use \( \rho \)

\[ A_s = \rho \times b \times d \]

\[ A_s = 0.0368 \times 100 \times 33 \]

\[ A_s = 12.14 \text{ cm}^2 / \text{m} \]

\[ \therefore \text{Use } 1 \varnothing 14 / 15 \text{ cm} \]

\[ A_s \text{ shrinkage} = 0.0018 \times 100 \times 40 \]

\[ = 7.2 \text{ cm}^2 / \text{m} \]

\[ \therefore \text{Use } 1 \varnothing 12 / 15 \text{ cm, in two dimensions.} \]

- Check shear at base of wall:

\[ V_a = P_h = 67.94 \]

\[ V_u = 67.94 \times 1.6 = 108.7 \text{ KN} \]

\[ V_u = 10.87 \text{ ton} \]

But \( \varnothing \ V_c = (0.75) (0.53) (\sqrt{240}) (33) (100) / 1000 \]

\[ \Rightarrow \varnothing \ V_c = 20.32 \text{ ton } > 10.86 \text{ ton } \rightarrow \text{it's okay} \]

**B. Design of Toe:**
Design of footings for Korean School

\[ q_1 \text{ (weight}_{\text{toe}} \text{) } = 0.4 \times 0.4 \times 2.5 = 0.4 \text{ ton} \]

\[ q_2 = \left( \frac{14.73 - 1.2}{2.6} \right) \times 2^2 + 1.2 \]

\[ q_2 = 12.64 \text{ ton/m} \]

\[ M_u = \left[ \frac{q_1 (1.2)(0.4)/2 + q_{\text{toe}} (1.6)(0.4)^2/2}{0.4}(0.4)(0.5)(0.4/3)(2) \right] \times 1.6 \]

\[ M_u = 0.096 + 1.885 + 0.178 \]

\[ M_u = 2.25 \text{ ton.m} \]

\[ \rho = 0.85 \times 240/4200(1 - \sqrt{1 - (2.61 \times 10^5 \times 2.25 / b \times d^2 \times f'_{c})}) = 5.35 \times 10^{-4} \]

\[ \rho_{\text{min}} = 14/f_y = 14/4200 = 3.33 \times 10^{-3} \]
\[ \rho < \rho_{\text{min}} \]

Use \( \rho_{\text{min}} \)

\[ A_s = \rho \times b \times d \]

\[ A_s = 0.0033 \times 100 \times 33 \]

\[ A_s = 10.89 \text{ cm}^2/\text{m} \]

\[ \therefore \text{Use } 1 \phi 12 / 10 \text{ cm} \]

\[ A_s \text{ shrinkage} = 0.0018 \times 100 \times 40 \]

\[ = 7.2 \text{ cm}^2/\text{m} \]

\[ \therefore \text{Use } 1 \phi 12 / 15 \text{ cm} \]

- Check shear

\[ V_u = (-0.4)(0.4)(1.2) + 12.64 \times 0.4 \times 1.6 + (14.73 - 12.64) \times (0.4)(0.5)(1.6) \]

\[ V_u = 8.56 \text{ ton} \]

But \( \phi V_c = (0.75)(0.53)(\sqrt{240})(33)(100)/1000 \)

\[ \phi V_c = 20.32 \text{ ton} > 8.56 \text{ ton} \rightarrow \text{it's okay} \]
C. Design of Heel:

\[ q_1 = 1.6 \times 0.4 \times 2.5 = 1.6 \text{ ton/m}^2 \]

\[ q_{1u} = 1.6 \times 1.2 \]

\[ q_{1u} = 1.92 \text{ ton/m}^2 \]

\[ q_2 = 1.6 \times 4 \times 1.8 = 11.52 \text{ ton/m}^2 \]

\[ q_{2u} = 18.43 \text{ ton/m}^2 \]

\[ q_3 = 1.2 \rightarrow q_{3u} = 1.92 \text{ ton/m}^2 \]

\[ M_u = (1.96)(1.6) / 2 + 18.43 \times 1.6 / 2 + 1.92 \times 1.6^2 / 2 + (15.24 - 1.72) \times (1.6 / 2) \times (1.6 / 3) \]

\[ M_u = 1.53 + 14.74 + 2.45 + 5.68 \]

\[ M_u = 24.4 \text{ ton.m} \]

\[ \rho = 0.85 \times 240 / 4200 (1 - \sqrt{1 - (2.61 \times 10^5 \times 24.4) / b \times d^2 \times f_c}) = 6.23 \times 10^{-3} \]

\[ \rho \min = 14 / f_y = 14 / 4200 = 3.33 \times 10^{-3} \]

\[ \rho > \rho \min \]

Use \( \rho \)

\[ A_s = \rho \times b \times d \]

\[ A_s = 0.00623 \times 100 \times 33 \]

\[ A_s = 20.55 \text{ cm}^2 \]

\[ \therefore \text{Use 1 } \varnothing 16 / 15 \text{ cm} \]
\[ A_s \text{ shrinkage} = 0.0018 \times 100 \times 40 \]
\[ = 7.2 \text{ cm}^2/\text{m} \]
\[ \therefore \text{Use} \ 1 \ Ø 12 / 15 \text{ cm} \]

- Check shear:

\[ V_u = (1.92 + 18.43) - (1.92 \times 1.5) - (15.24 - 1.92)(0.5)(1.6) \]
\[ V_u = 20.34 - 2.88 - 10.656 \]
\[ V_u = 6.80 \text{ ton} \]

But \[ \emptyset V_c = (0.75)(0.53)(\sqrt{240})(33)(100)/1000 \]
\[ \emptyset V_c = 20.32 \text{ ton} > 6.80 \text{ ton} \rightarrow \text{it’s okay} \]
( Reinforcement of the Retaining wall )
CHAPTER FOUR
SUMMARY AND CONCLUSION

This chapter is illustrated the summery and conclusion of the project. After all investigation and selection of proper foundations. The following points are stated:

1. The site is located in the north eastern bound of Jenin District beside Beit Qad street.
2. The area of the site is 18876 m².
3. The soil in the site is mainly clay.
4. The areas around the site are agricultural lands.
5. The school consists of four floors.
6. The total area of the school is 641.6 m².
7. The bearing capacity of the soil is 2 kg/cm².
8. Combined footings were used in the school.
9. Shear wall and a retaining wall were designed.
CHAPTER FIVE

REFERENCES

CHAPTER SIX

APPENDIX

- Table 6.1: Distribution of loads on columns

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<th>Column</th>
<th>Load (Ton)</th>
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<td>C2</td>
<td>130</td>
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Fig 6.1 : Map for footings and columns
Fig 6.2: 3D map for footing F3
Fig 6.3 : Moment distribution for footing F3