

CHAPTER ONE

INTRODUCTION

1.1. General Introduction:

Reinforced concrete is a strong durable building material that can be formed into many varied shapes and sizes. It has established itself as one of the major construction materials. Today concrete structures (including buildings, bridges, power plant, dams, etc.) constitute a large part of the modern civil infrastructure.

Concrete is defined as a material formed by mixing cement, coarse and fine aggregate and water. It's produced by hardening of the cement/paste (cement and water); beside these basic components, it may also contain admixtures and/or additions. The resulting composite material is strong in compression but relatively weak in tension, and therefore steel reinforcement is often incorporated to carry the tensile stresses. The mechanical properties of the hardened concrete and the rheological properties of the fresh concrete are relatively varying within certain limits. The density produced with regular aggregates is around 2400 kg/m^3 ; the compressive strength, which can be achieved without any special considerations, ranges from 20 up to 100MPa, and the modulus of elasticity typically ranges from 25 to 45GPa. Furthermore, using lightweight aggregates, with a density below 1500 kg/m^3 it is possible to produce concrete with densities below 2000 kg/m^3 having compressive strength up to 65MPa and a modulus of elasticity of 25GPa (see Fib Bulletin 2000).

In international standards, concretes generally are classified according to their compressive strength. However a distinction is often made between different types of concrete depending on the composition, state of hardening or special properties, of concrete in particular.

As a construction material, concrete has advantages and disadvantages; some of these are listed in Table 1.1. The advantages of concrete are what make this material so ubiquitous, for example: the cost of concrete is relatively low (cost per unit volume); concrete is moisture-resilient and can be made water-impermeable; concrete is non-combustible and can resist high temperatures; concrete is, due to its high density, sound absorbing and capable of thermal storage; concrete structures can also be made durable, although this requires experienced designers, work executed with good quality, and a proper mix design. The disadvantages, on the other hand, are responsible for problems in infrastructure deterioration, service load failures by excessive cracking and deflections. Concrete is an intrinsically brittle material with low ductility and, what is more, high-strength concrete is even more brittle. The tensile strain capacity is low, and the tensile strength is only about 5% to 10% of its compressive strength. Concrete is not volume-stable over time; it shrinks, swells, and, when subjected to an external action, creeps. Concrete has a low strength-to-density ratio. Concrete requires a formwork to support it until it has hardened. Concrete, when newly cast, may be sensitive to early age drying and plastic shrinkage cracks may form. Desiccation of moisture requires time, which sometimes may be important for the speed of construction (Thomas Telford; 1986).

Table 1.1: Advantages and disadvantages of concrete, from Mindess et al. (2003)

Advantages	Disadvantages
Ability to be cast	Low tensile strength
Economical	Low ductility
Durable	Volume instability
Fire-resistant	Low strength-to-weight ratio
Energy-efficient	
On-site fabrication	
Aesthetic properties	

To make the most of the advantages and avoid or reduce the effects of the disadvantages, a proper mix design should be made.

Cracking of the concrete section cannot be prevented; however, the size of and location of the cracks can be limited and controlled by reinforcement, placement of control joints, the curing methodology and the mix design of the concrete. Cracking defects can allow moisture to penetrate and corrode the reinforcement. This is a serviceability failure in limit state design. Cracking is normally the result of an inadequate quantity of reinforcement, spaced at too great distance. The concrete then cracks either under excess loading or due to internal effects such as early thermal shrinkage when it cures.

Ultimate failure leading to collapse can be caused by crushing of the concrete, when compressive stresses exceed its strength; by yielding or

failure of the bar, when bending or shear stresses exceed the strength of the reinforcement; or by bond failure between the concrete and the bar.

The urgency of repairs, strengthening or replacement must be evaluated in a technical sense along with realistic cost estimates, so that proper priorities can be established in budget planning.

The information obtained from structural assessment can then be used to determine whether or not corrective work is required or is economically feasible (when compared to the cost of demolition and replacement). If so, how it can best be accomplished. Without prior planning and proper assessment, any programmed of corrective work is likely to prove ineffective.

Concrete structures need to be strengthened for any of the following reasons (Thomas Telford; 1991):

- Load increases due to higher live loads, increased wheel loads, installations of heavy machinery, or vibrations.
- Damage to structural parts due to aging of construction materials or fire damage, corrosion of steel reinforcement, and/or impact of vehicles.
- Improvements in suitability for use due to limitation of deflections, reduction of stress in steel reinforcement and/or reduction of crack widths.
- Modification of structural system due to elimination of walls/columns and/or openings cut through slabs.

- Errors in planning or construction due to insufficient design dimensions and/or insufficient reinforcing steel.

There are many ways for increasing or restoring the structural capacity of structures, including code changes, changes in use (which increase service loads), deficiencies within the structure caused by errors in design or construction, or loss of capacity due to deterioration. Regardless of the need, Structural can assist in the development and implementation of value-added structural upgrade and strengthening solutions.

The most important step in the design of repair and/or strengthening work is a careful assessment of the existing structure. The purpose of this assessment is to identify all defects and damage, to diagnose their causes and hence to assess the present and likely future adequacy of the structure.

Some evaluations will relate to whether or not an economically effective repair can limit or contain damage; hence increasing the effective life of structure. In some instances an evaluation will be concerned with the degree of urgency required to implement a repair, because of the advanced stage of damage.

The degree of restoration will depend on whether or not it is required to restore the original or impart greater load-carrying capacity. If, for technical and/or economical reasons, it is not feasible to achieve complete restoration to the original capacity. At the same time total replacement is not an acceptable option, a reduction of the applied live load becomes mandatory.

The owner, in reaching a decision as to a course of action, will have to evaluate not only the technically feasible options available, but also the costs of each option, political considerations (economical impact to communities served by the facility), life expectancy associated with the various options available, any historical significance of the structure, any risks that may be involved with any changes in safety level or reduction in load carrying capacity, etc.

1.2. Objective of the research:

This research investigates the strengthening of reinforced concrete structures (Columns, Beams, Foundations, slabs and RC walls) and take a local building as a case study. To achieve the target of the research, the following specific objectives are proposed:

1. To have an adequate knowledge about the concrete as structural materials and also adequate information about the advantages and disadvantages of concrete, and the mechanical failure of RC material.
2. To have good background about the problems which exposed the slabs, beams, columns foundations and RC walls to failure, and the structural solutions and the method of construction strengthening.
3. To re-analyse and assess the building which is subjected to failure due to bad construction and the different between the design detailing and the structural section for existing building. To achieve this objective, the following specific objective should be achieved:
 - To re-design the columns sections to carry the expected correct columns load under load combination 1 Ultimate load (1.4 DL+1.6 LL).

- To re-design the foundations of the building for adequacy to carry the obtained footing loads under load combination 2 Service Load (DL+ LL).
- To re-design all structural elements of the building.

1.3. Organization of the research:

This research has been organized as follow:

The concrete as structural materials, and the advantages and disadvantages of concrete, and the mechanical failure are presented. This is content of chapter 1.

The problems which exposed to the slabs, beams, columns foundations and RC walls, and the structural solutions are discussed. This is the content of chapter 2.

The main objective of this study (case study), is re-analysis and assessment of the building which is subjected to failure due to bad construction and the different between the design detailing and the structural section for the existing building is discussed. This is the content of chapter 3.

Strengthening design calculations for Columns, Foundations and Slabs are designed. This is the content of chapter 4.

The research summary and conclusions with recommendation for future researches are presented. This is the content of chapter 5.

CHAPTER TWO

LITERATURE REVIEW OF STRENGTHENING TECHNIQUES

2.1 Introduction

This chapter deals with the problems which are related to the slabs, beams, columns, foundations and R.C walls and the structural solutions and the methods of constructions and strengthening.

Many strengthening techniques are available depending on the purpose needed from strengthening. Some of those techniques are explained in details in the following sections. The strengthening or retrofit buildings for many reasons, (Thomas Telford; 1991):

- Increase in the applied loads (additional floors or increase in live load and/or dead load).
- Mistakes of the unsafe design or lack of quality control i.e. (not reaching target strengths).
- Reinforcement steel corrosion or insufficient number of bars.
- Cracks in concrete or stress less than the design stress.
- The inclination of the column is more than the allowable.
- The settlement in the foundation is more than the allowable.
- Errors in planning or construction due to insufficient design dimensions and/or insufficient reinforcing steel.

2.2. Problems related to the slabs:

The load-bearing capacity of a slab can be limited by the moment capacity or by shear capacity. Normally the bending moment capacity is decisive but in heavily-loaded thick slabs supported on columns or in slabs with highly concentrated load. The punching shear capacity may also be critical. In a serviceability limit state, deformations are often decisive for the load to be carried. Cracking can also limit the serviceability of a slab. Slabs are normally under-reinforced. In these cases, supplementary tensile reinforcement can increase the moment capacity.

Overlays can either be assumed to interact with the old slab or to function independently. If interaction is to take place, shear stresses must be transferred between the new and old concrete. The shear stresses are normally fairly low in slabs in which case mechanical anchorages between new and old concrete are not necessary. In certain cases mechanical connectors, such as stirrups or studs can be used.

In some cases, the slabs exposed to several problems due to increasing the applied loads on slabs and these problems are summarized in the following points:

- Case (1): In some cases, the slab is unable to carry the negative moment and the lower steel is sufficient.
- Case (2): Also in another case when the slab is unable to carry the positive moment or when the dead load (that will be added to the slab) is much less than the live load carried by the slab.

2.2.1 Strengthening of Reinforced Concrete Slabs:

The strengthening of the structures should be designed and constructed in accordance with appropriate building codes. To overcome the problem discussed in the previous paragraph the treatment is to add upper steel mesh with a new concrete layer in Case (1), and a new concrete layer on the bottom of the slab should be added in Case (2).

When adding concrete to the compressive zone, it must be remembered that the new concrete normally shrinks more than the old concrete. If the bond is satisfactory, the new concrete may then have a closely-spaced pattern of cracks due to shrinkage. This should not affect the ultimate load to any appreciable extent, but the stiffness of the slab will be less than for a corresponding monolithic slab.

When the tensile zone is supplemented with extra tensile reinforcement, this reinforcement should be covered with concrete. This concrete provides fire and corrosion protection for the reinforcement. In addition to the transfer of shear forces.

The new reinforcement should always be firmly fixed to the slab. The reinforcement can, for example, be welded or tied to expansion bolts which have been bolted tightly into the concrete. The spacing between the slab and the reinforcement should be at least 10mm.

In order to implement the previous solutions, the following steps should be made as shown in *Fig 2.1* and *Fig 2.2*:

- Removing the old concrete cover.

- Cleaning the reinforcing steel bars using a wire brush or a sand compressor.
- Coating the steel bars with an epoxy material that would prevent corrosion.
- In presence of corrosion in the steel bars, a new steel mesh, designed according to the codes' requirements, must be added.
- The new reinforcing steel mesh is then installed and fastened vertically to the slab of the roof and horizontally to the surrounding beams, using steel dowels.
- Coating the concrete surface with an appropriate epoxy material that would guarantee the bond between the old and new concrete.
- Before the epoxy dries, the concrete is poured with the required thickness. Additional materials that would lower the shrinkage should be added to the concrete.

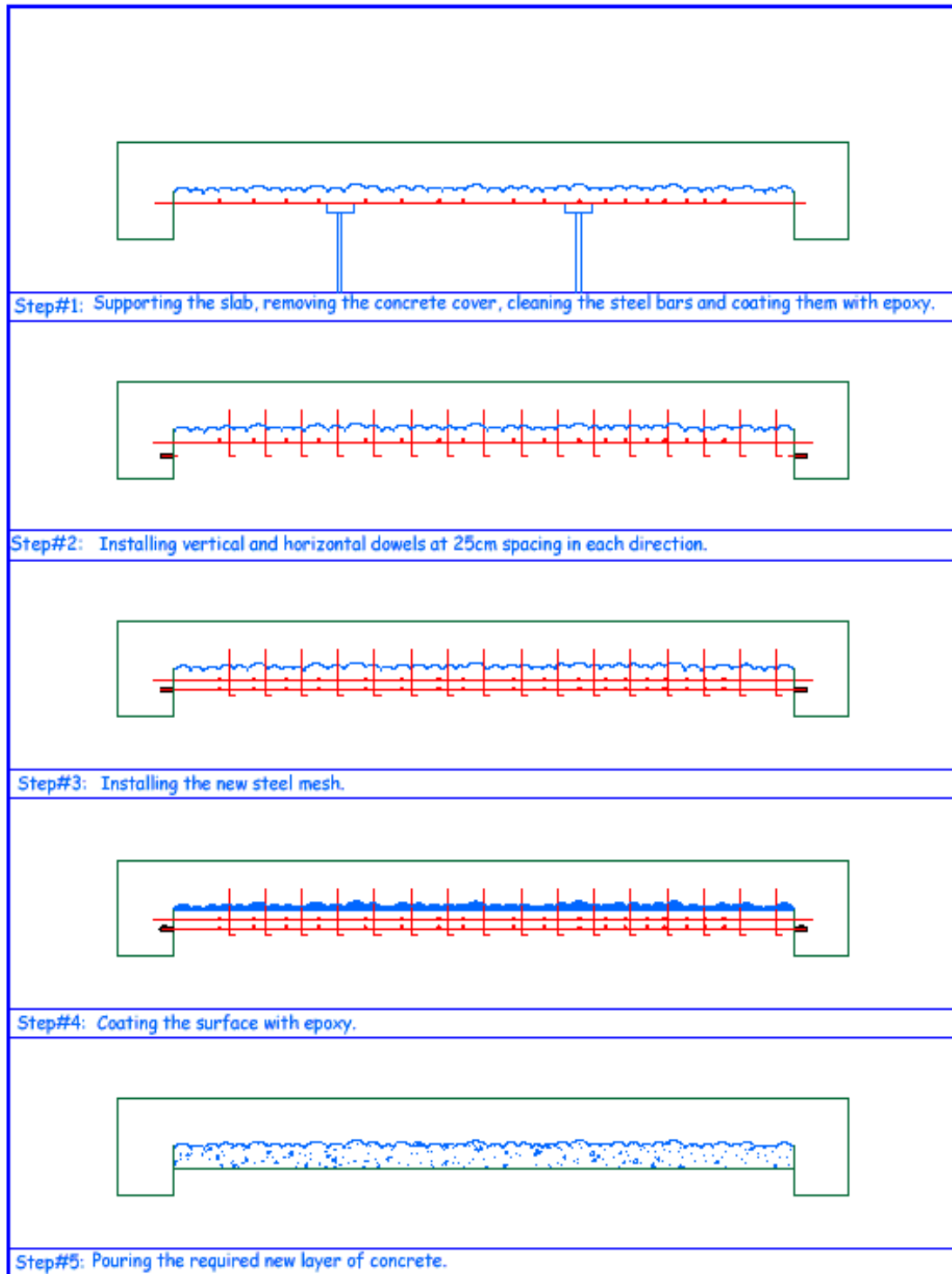


Figure (2.1): Strengthening a slab by increasing its depth from bottom

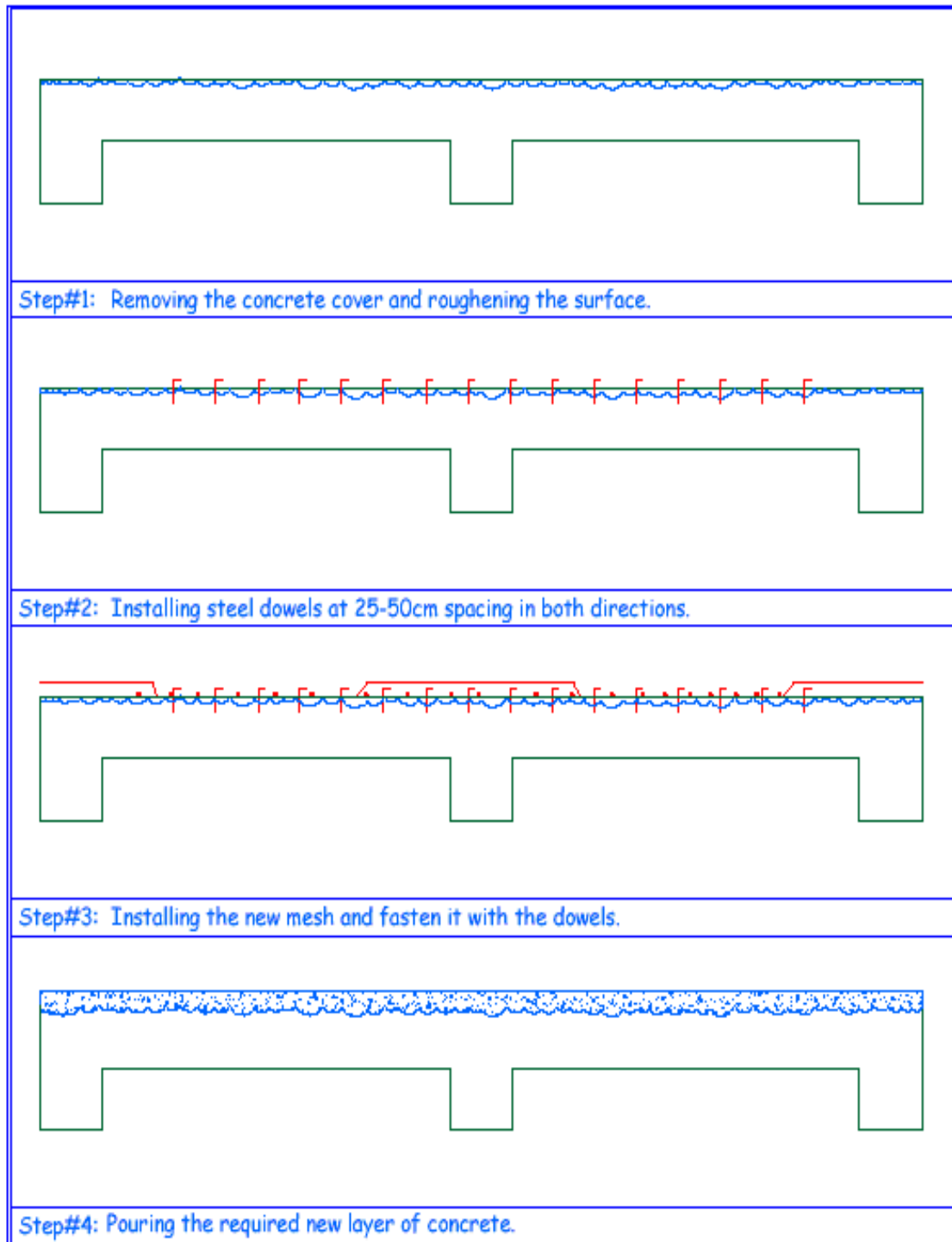


Figure (2.2): Strengthening a slab by increasing its depth from top.

There are also some other techniques used for strengthening a reinforced concrete slab such as:

- Increasing the shear capacity of the slab by adding steel plates strengthened by vertical screw bolts. Strengthening of the slab by post stressed reinforcement.
- Adding steel beams.

In case of hollow slabs, reinforced concrete is added inside the holes of the slab.

2.3 Problems related to the beams:

Reinforced concrete beams need strengthening when the existing steel bars in the beam are unsafe or insufficient, or when the loads applied to the beam are increased.

2.3.1 Strengthening of Reinforced Concrete Beams:

The moment capacity of a beam can be limited by compressive failure in the concrete (over-reinforced beam) or by yielding of the reinforcement (under-reinforced beam). In both cases the moment capacity can be improved by increasing the effective depth, i.e., by adding concrete to the compression zone. The most efficient way to strengthen an under-reinforced beam, however, is to add new tensile reinforcement, especially if this also means increasing effective depth (D.P. Singh;1992).

Shear strengthening is normally achieved by supplementing the beam with vertical or inclined stirrups. Special attention should be paid to the

anchorage of the additional shear reinforcement. The anchorage of the main reinforcement may have to be improved, e.g. by exposing the reinforcement and providing it with end anchors.

Additional concrete can either be assumed to interact with the old beam, in which case shear forces must be transferred in the interface between the new concrete and the old concrete, or to act independently, in which case the deformations are of the same magnitude for the new part and for the old part. If interaction can be achieved, a simpler, less expensive and less bulky construction can normally be obtained.

The strengthening can be accomplished by the addition of concrete in the compressive zone, in the tensile zone, or in both. Additional concrete in the tensile zone is of course meaningful only if it is provided with reinforcement. The shear stresses are usually fairly high in beams. The normal solution should therefore be to anchor the additional concrete with reinforcement, which is placed in accordance with the shear force diagram, i.e., normally more closely-spaced towards the ends of the beam. When additional concrete is added to the compression zone, it must be borne in mind that the additional concrete will normally have a greater shrinkage than the old concrete, even if the old concrete has been pre-moistened.

The tensile zone is supplemented with extra reinforcement surrounded with concrete, which provides the reinforcement with protection against corrosion and fire, and which transfers shear forces to the old beam. Supplementing the tensile zone mostly applies to the lower part of the beam. This means that conventional concreting is difficult. The normal solution is to use

shotcrete. The new reinforcement should normally be surrounded by stirrups anchored in the old beam or enclosing it. If the shear stresses in the joint are extremely large, shear keys can be chipped into the old beam.

If there is sufficient shear strength in the joint between the new and the old concrete, it should be possible to assume complete interaction in the ultimate state. Experiments have shown that the ultimate load for beams strengthened in this way does not deviate in any significant manner from the ultimate load for a corresponding monolithic beam. The stiffness, on the other hand, is normally lower.

In such cases, there are different solutions that could be followed:

I- ADDING REINFORCEMENT STEEL BARS TO THE MAIN STEEL WITHOUT INCREASING THE BEAM'S CROSS SECTIONAL AREA

This solution is carried out when the reinforcing steel bars are not capable to carry the stresses applied to the beam. The following steps should be followed:

- The concrete cover is removed for both the upper and lower steel bars.
- The steel bars are well cleaned and coated with an appropriate material that would prevent corrosion.
- Holes are made, in the whole span of the beam under the slab, as shown in Fig (2.3), 15-25cm apart, a diameter of 1.3cm and extend to the total width of the beam.

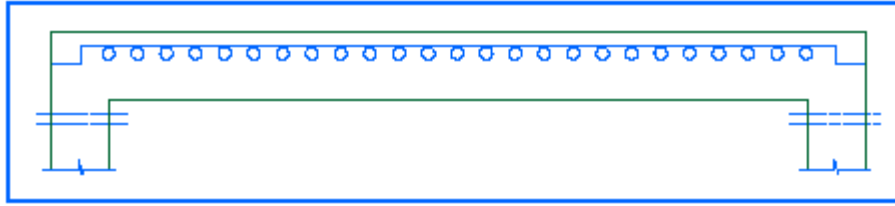


Figure (2.3): Holes in the span of a beam.

- The holes are filled with an epoxy material with low viscosity and installing steel connectors for fastening the new stirrups.
- Steel connectors are installed into the columns in order to fasten the steel bars added to the beam.
- The added stirrups are closed using steel wires and the new steel is installed into these stirrups.
- The surface is then coated with a bonding epoxy material.
- The concrete cover is poured over the new steel and the new stirrups.

The previous steps are illustrated in Fig (2.4).

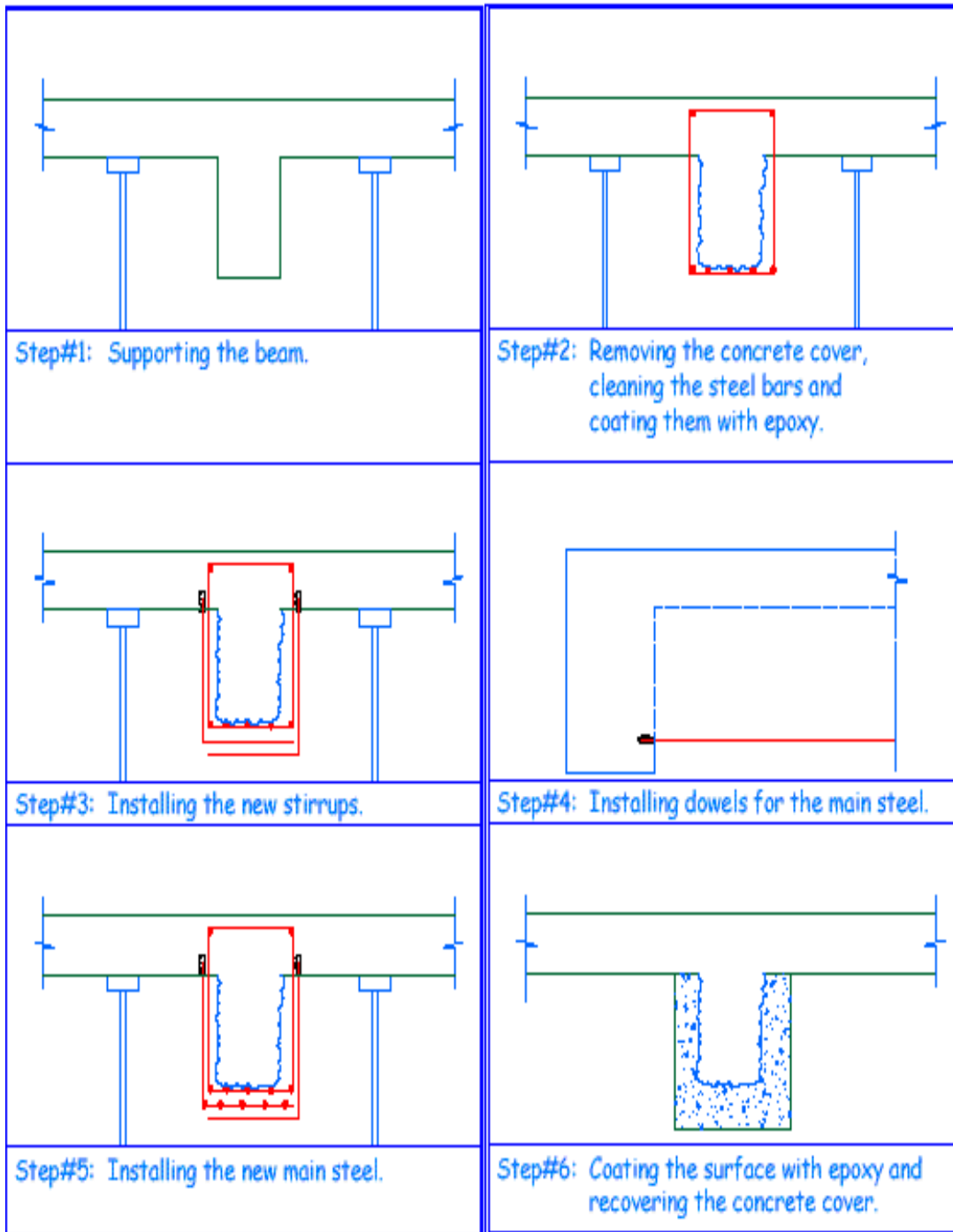


Figure (2.4): Strengthening a beam without increasing the cross sectional area.

II- INCREASING BOTH THE REINFORCING STEEL BARS AND THE CROSS SECTIONAL AREA OF CONCRETE:

This solution is chosen when both the steel and concrete are not able to carry the additional loads applied to the beam. In such cases the following steps should be followed as in *Fig (2.5)*.

- Removing the concrete cover, roughing the beams surface, cleaning the reinforcement steel bars and coating them with an appropriate material that would prevent corrosion.
- Making holes in the whole span and width of the beam under the slab at 15-25cm.
- Filling the holes with cement mortar with low viscosity and installing steel connectors for fastening the new stirrups.
- Installing the steel connectors into the columns in order to fasten the steel bars added to the beam.
- Closing the added stirrups using steel wires and the new steel is installed into these stirrups.
- Coating the concrete surface with an appropriate epoxy material that would guarantee the bond between the old and new concrete, exactly before pouring the concrete.
- Pouring the concrete jacket using low shrinkage concrete.

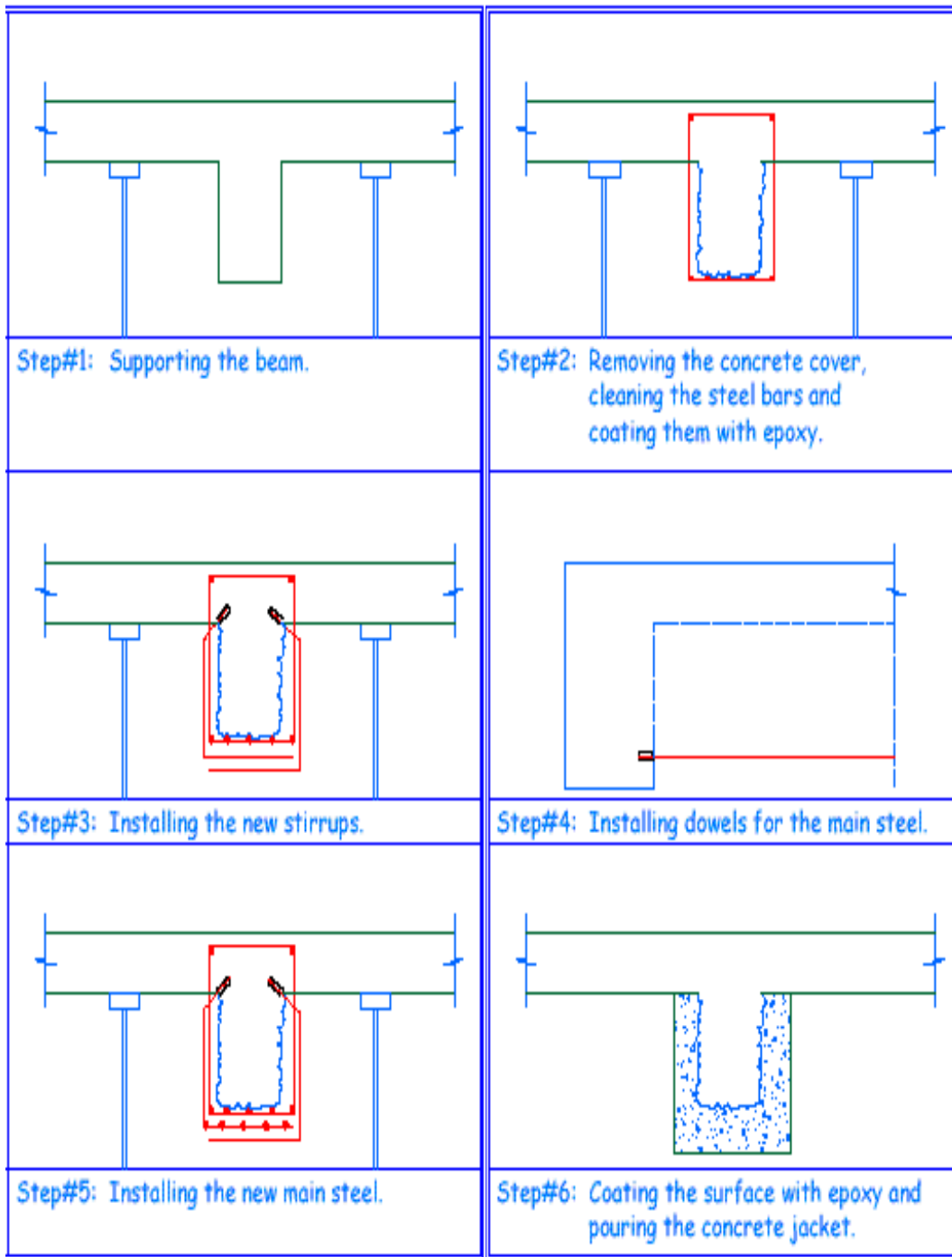


Figure (2.5): Strengthening a beam by increasing the cross sectional area and the bars.

III-ADDING STEEL PLATES TO THE BEAM:

When it is required to strengthen the beam's resistance against the applied moment or shear stress, steel plates are designed with the appropriate size and thickness.

Then those plates are attached to the beam as follows:

- Roughing and cleaning the concrete surfaces where the plates will be attached.
- Coating the concrete surfaces with a bonding epoxy material.
- Making holes in the concrete surfaces and plates.
- Putting a layer of epoxy mortar on top of the plates with a 5mm thickness.
- Attaching the steel plates to the concrete using bolts.

The previous steps are illustrated in Fig (2.6) which shows steel plates attached to slab.

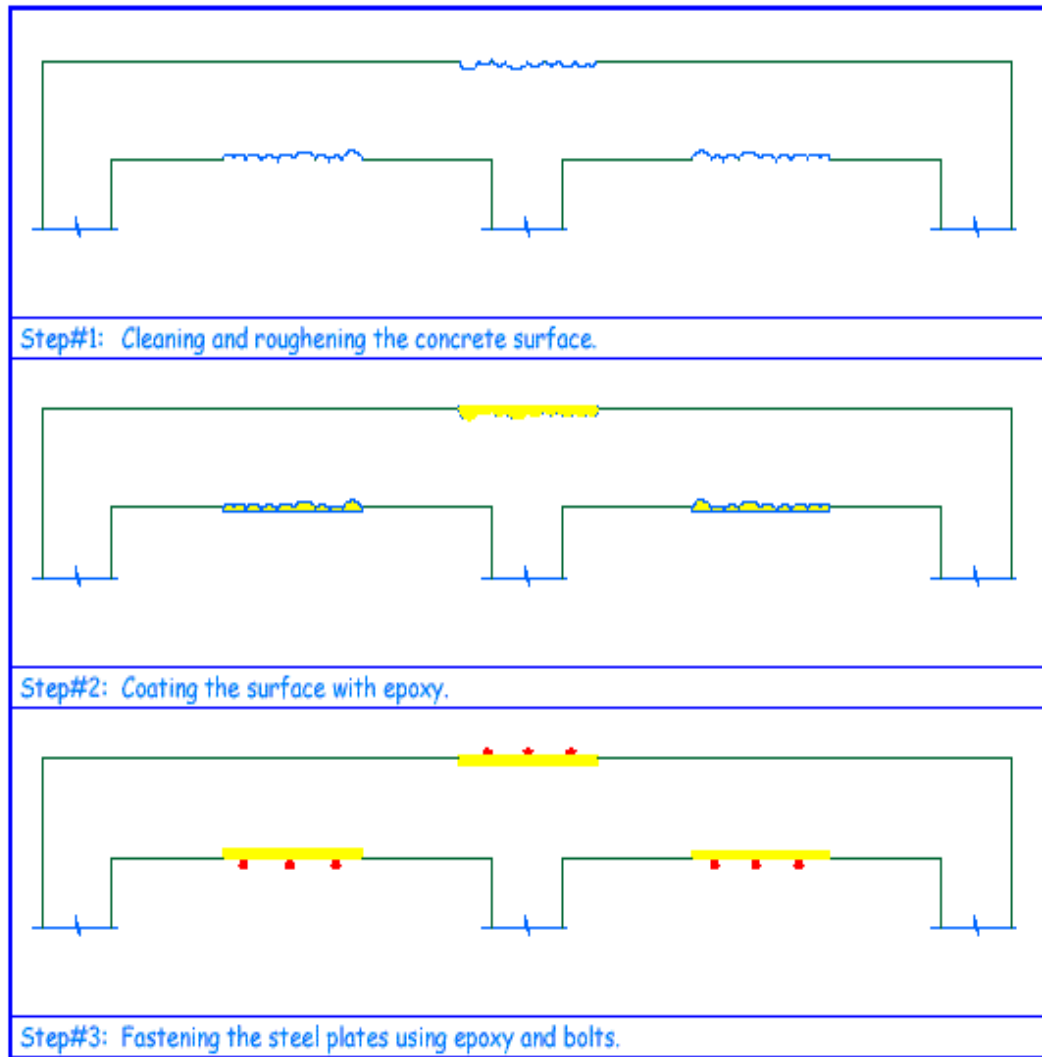


Figure (2.6): Strengthening a beam by adding steel plates.

In some cases, it is needed to reduce the load on the beam that needs strengthening before implementing the previous steps, either partial or complete unloading. This is made by putting steel beams on top or below the concrete beams, as shown in *Fig (2.7)*.

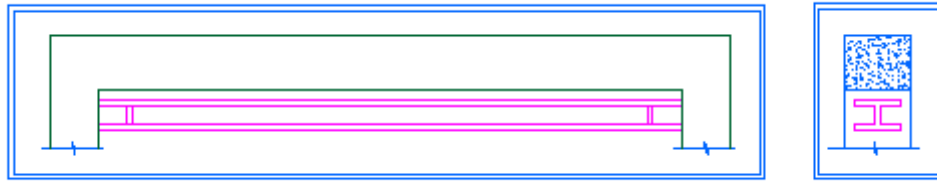


Figure (2.7): Reducing the load on the beam using steel beams.

The following *Figure (2.8)* were taken during strengthening an existing building; they present the practical method of implementing some strengthening techniques.



Strengthening a beam , slab and column.



.Strengthening a beam and Slab.



. Jacketing a beam by increasing bars and cross section.



Strengthening by steel plates.

Figure (2.8): Strengthening an existing building.

2.4 Problems which encountered in columns:

There are many types of problems:

- The load carried by the column is increased due to either increasing the number of floors or due to mistakes in the design.
- The compressive strength of the concrete or the percent and type of reinforcement are not according to the codes' requirements.
- The inclination of the column is more than the allowable.
- The settlement in the foundation is more than the allowable.

2.4.1 Strengthening of Reinforced Concrete Columns:

The load-carrying capacity of a column can be increased by encasing the existing column with concrete or providing a supplementary steel structure which will relieve the original column of part or all of its load and/or make the column stiffer (Hayashi T;1980).

The additional concrete should preferably enclose the column and should be provided with closed stirrups. The corners of the existing column should be chipped off before concreting, to reduce stress concentrations in the new concrete due to shrinkage, and to improve its interaction with the old concrete. Jacketing with extra stirrups can also be used as a means of increasing the ductility of existing columns in earthquake regions. Sometimes complete jacketing is impracticable, in the case of wall or columns and at expansion joints for example.

The fully satisfactory interface can be obtained between the old and new concrete. The strengthened column will not behave as a homogeneous

concrete column with the same cross-section. The additional concrete will shrink more than the old concrete, so that here is a risk of cracking in the new concrete. Unloading the column during the strengthening work may, to some extent, compensate for this. Also saturating the old concrete with water before adding the new concrete may also have some beneficial effect. However, the effect of shrinkage can seldom be completely compensated for strengthening of RC columns is needed to increase the capacity of columns to carry the obtained column loads.

There are two major techniques for strengthening RC columns:

2.4.1.1 Reinforced concrete jacket:

The jacket size, the number and diameter of the steel bars used in the jacketing process depend on the structural analysis that was made to the column. In some cases, before this technique is carried out, we need to reduce or even eliminate temporarily the loads applied to the column; this is done by the following steps:

- Putting mechanical jacks between floors.
- Putting additional props between floors.

Moreover, in some cases, where corrosion in the reinforcement steel bars was found, the following steps should be carried out:

- Remove the concrete cover.
- Clean the steel bars using a wire brush or sand compressor.
- Coat the steel bars with an epoxy material that would prevent corrosion.

When there was no need for the previous steps, the jacketing process could start by the following steps:

- Adding steel connectors into the existing column in order to fasten the new stirrups of the jacket in both the vertical and horizontal directions at spaces not more than 50cm. Those connectors are added into the column by making holes 3-4mm larger than the diameter of the used steel connectors and 10-15cm depth.
- Filling the holes with an appropriate epoxy material then inserting the connectors into the holes.
- Adding vertical steel connectors to fasten the vertical steel bars of the jacket following the same procedure in step 1 and 2.
- Installing the new vertical steel bars and stirrups of the jacket according to the designed dimensions and diameters.
- Coating the existing column with an appropriate epoxy material that would guarantee the bond between the old and new concrete.
- Pouring the concrete of the jacket before the epoxy material dries. The concrete used should be of low shrinkage and consists of small aggregates, sand, cement and additional materials to prevent shrinkage.

The previous steps are illustrated in *Fig (2.9)*.

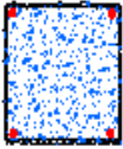
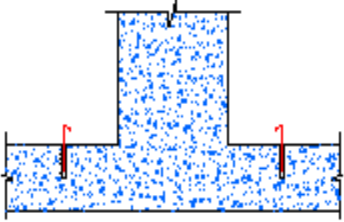
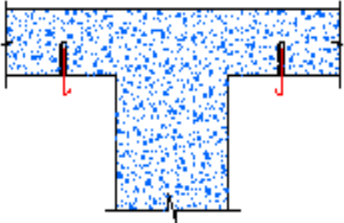
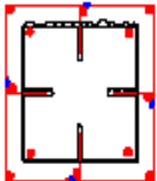
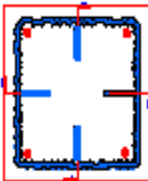
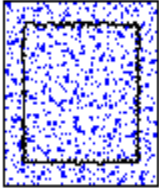
	
Step#1: Cleaning and roughening the surface.	Step#2: Installing dowels for fastening the stirrups.
	
Step#3: Installing dowels for fastening the lower main steel.	Step#4: Installing dowels for fastening the upper main steel.
	
Step#5: Installing the new stirrups and steel bars.	Step#6: Coating the surface with epoxy.
	
Step#7: Pouring the new concrete layer.	

Figure (2.9): Increasing the cross sectional area of a column by RC jacketing.

2.4.1.2 Steel jacket:

This technique is chosen when the loads applied to the column will be increased, and at the same time, increasing the cross sectional area of the column is not permitted.

This technique is implemented by the following steps as shown in *Fig (2.10)* (Edward.G.Nawy; 2003):

- Removing the concrete cover.
- Cleaning the reinforcement steel bars using a wire brush or a sand compressor.
- Coating the steel bars with an epoxy material that would prevent corrosion.
- Installing the steel jacket with the required size and thickness, according to the design, and making openings to pour through them the epoxy material that would guarantee the needed bond between the concrete column and the steel jacket.
- Filling the space between the concrete column and the steel jacket with an appropriate epoxy material.

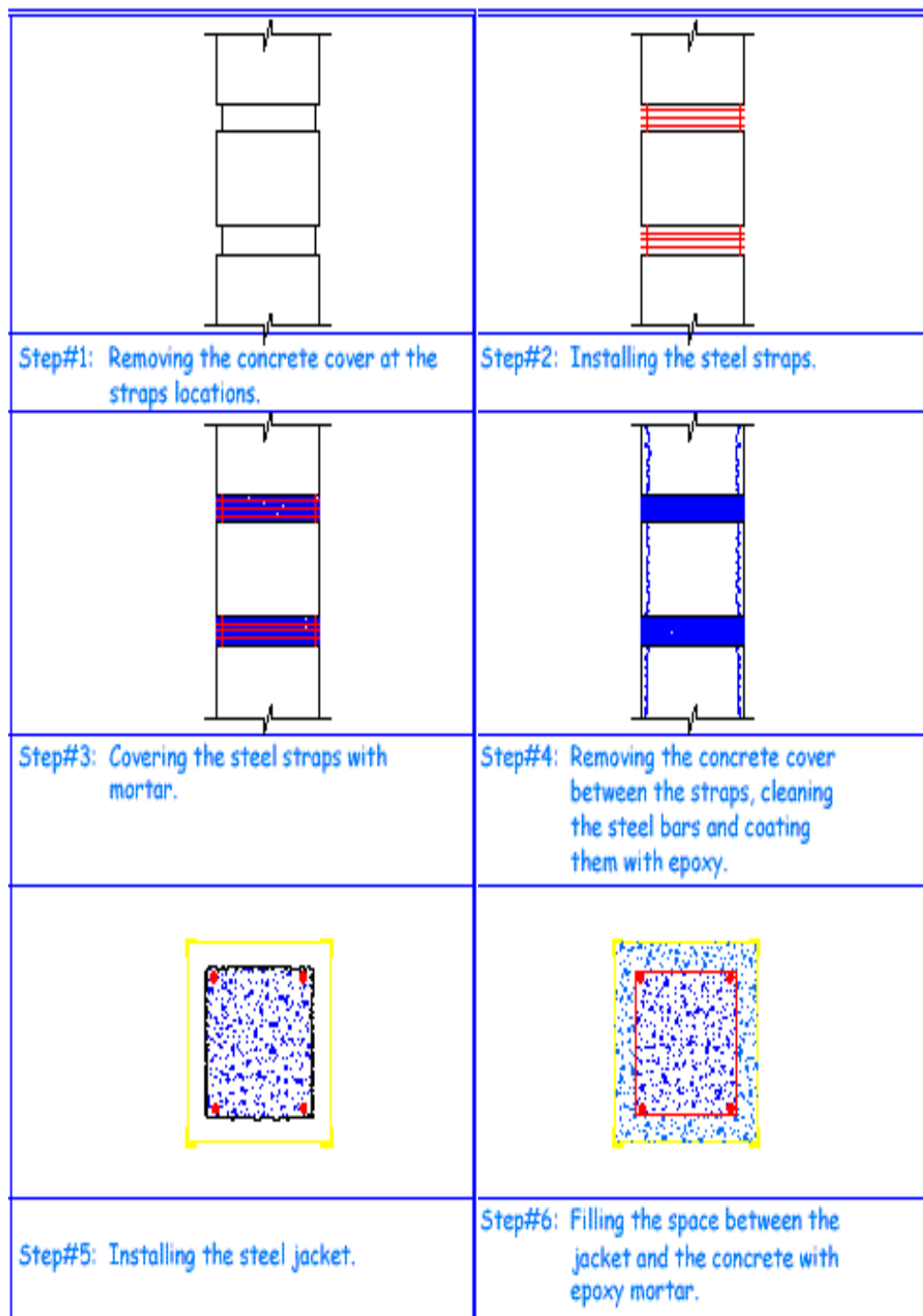


Figure (2.10): Increasing the cross sectional area of a column by steel jacketing.

In some cases, where the column is needed to carry bending moment and transfer it successfully through the floors, one should install a steel collar at the neck of the column by means of bolts or a suitable bonding material.

2.5 Problems encountered in foundations:

Foundation settlement and movement can be caused by building on expansive clay. Compressible soils may contain organic material, soft clay or other components that cause the soil to settle when loaded. Columns foundations also need strengthening in the case of applying additional loads.

2.5.1 Strengthening of Reinforced Concrete Foundations:

Widening and strengthening of existing foundations may be carried out by constructing a concrete jacket to the existing footings. The new jacket should be properly anchored to the existing footing and column neck in order to guarantee proper transfer of loads. The size of the "jacket" shall be selected such that the average maximum foundation pressure does not exceed the recommended allowable value. Attention shall be given during construction in order that the excavations for the new "jackets" do not affect the existing adjacent foundations.

An isolated footing is strengthened by increasing the size of the footing and the reinforcement steel bars as follows:

- Excavating around the footing
- Cleaning and roughening the concrete surface.
- Installing dowels at 25-30cm spacing in both directions using an appropriate epoxy material.

- Fastening the new steel bars with the dowels using steel wires. The diameter and number of steel bars should be according to the design.
- Coating the footing surface with a bonding agent in order to achieve the required bond between old and new concrete.
- Pouring the new concrete before the bonding agent dries. The new concrete should contain a non-shrinkage material.

The previous steps are illustrated in *Fig (2.11)*.

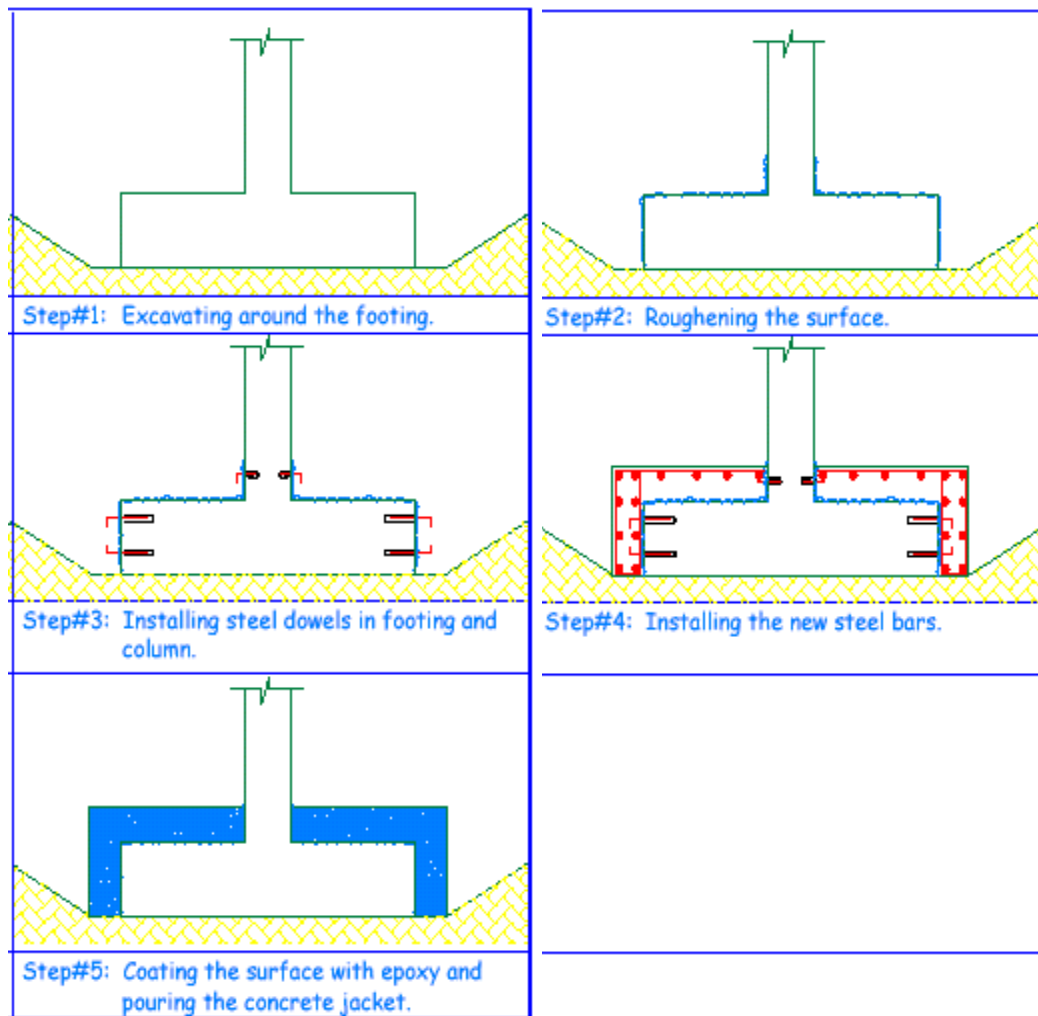


Figure (2.11): Strengthening of an isolated footing.

The following *figure (2.12)* illustrate the practical way of jacketing a footing by reinforced concrete.



Excavating around the footing.



hardening the surface and installing the dowels.



Installing the main steel.



Completing the jacket.

Figure (2.12): practical way of jacketing a footing by reinforced concrete.

2.6 Strengthening of R.C Walls:

The strengthening of the R.C Walls should be designed and constructed in accordance with appropriate building codes and to overcome the problem discussed in the previous paragraph the treatment is to increase the dimension of the wall and its reinforcement.

The dimensions of the wall and its reinforcement are increased by the following steps:

- Roughing the total area of the concrete surface.
- Installing steel connectors for the whole surface at 25-30cm spaces in both directions. The diameter of the steel connectors is determined according to the design and their depth should be 5-7 times their diameter.
- Installing steel connectors into the wall footings, with the same number and diameter of the main vertical steel bars, using an epoxy material.
- Installing the steel mesh and fasten it by steel wires to the steel connectors.
- Coating the surface of the wall with an appropriate epoxy material.
- Pouring the concrete jacket using low shrinkage concrete before drying of the epoxy material.

The previous steps are illustrated in *Fig (2.13)*.

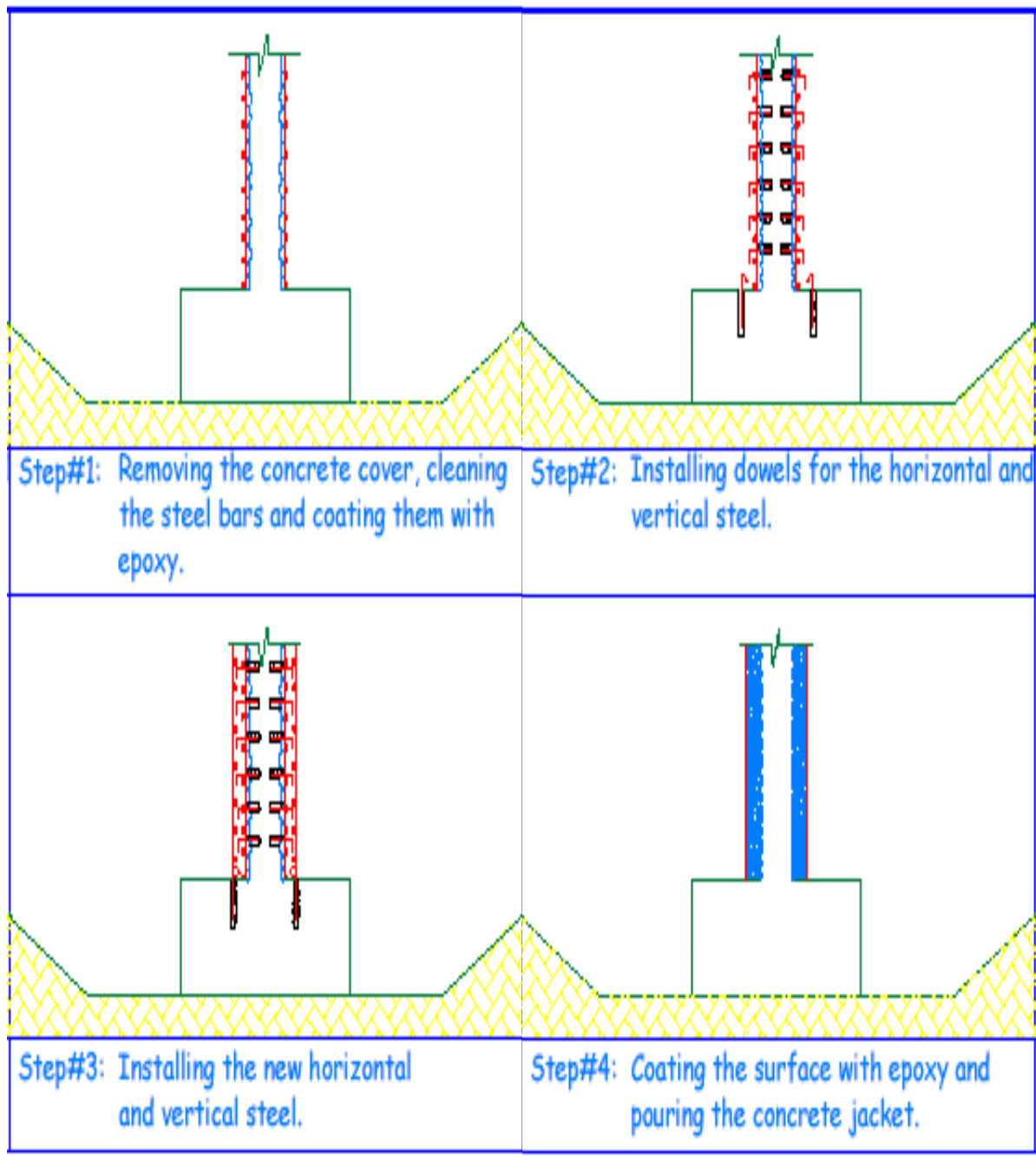


Figure (2.13): Strengthening of reinforced concrete walls

CHAPTER THREE

CASE STUDY:

STRUCTURAL ANALYSIS AND ASSESSMENT OF LOCAL BUILDING

3.1 General Description

This chapter deals with the re-analysis and assessment of local building which is subjected to failure due to bad construction and the difference between the actual design detailing and the structural section for existing building.

It consists of six stories reinforced concrete building. The building is composed of a flat slab system that is supported by reinforced concrete columns which transfer the structure's loads to 100 isolated footings, four combined footings and three small raft foundations.

The purpose of an evaluation of damaged structure is not only to determine the effect of damage to the structure's life expectancy but also, and perhaps more importantly, a determination, as far as is possible, of its cause so as to determine an effective retrofit. Before an effective repair can be implemented, the cause of the damage has to be removed or the repair measure has to be designed to accommodate the cause, otherwise the risk of damage repetition exists

The complexity and magnitude of the repair procedure will depend on whether or not the cause of the damage has merely to be removed or whether or not the structure must be restored to its original condition. There are

several options to be considered in an evaluation of restoration of the functionality of a damaged structure

- a) Total replacement of the structure
- b) A combination of partial replacement and repair, based upon the severity.
- c) Extensive strengthening measures.

The most important step in the design of repair and/or strengthening work is a careful assessment of the existing structure. The purpose of this assessment is to identify all defects and damage, to diagnose their causes and hence to assess the present and likely future adequacy of the structure.

The information obtained from structural assessment can then be used to determine whether or not corrective work is required or is economically feasible (when compared to the cost of demolition and replacement) and, if so, how it can best be accomplished. Without prior planning and proper assessment, any programmed of corrective work is likely to prove ineffective.

3.2 Loading Conditions and Materials Properties:

The properties of a material must be known in order to establish its load carrying capacity. Minimum requirements for the properties of a given material are given in various national material specifications, such as the American Society for Testing and Materials (ASTM), German industry Standards (DIN), etc. However, the actual material properties may be somewhat different from the minimum requirements.

Therefore, before assessing the load-carrying capacity of an existing structure, the actual properties of the materials must be determined as they may have a significant effect in the determination of the repair and/or strengthening method to be applied.

Damaged material should be tested or evaluated as to the extent of damage and whether it can be retained in the structure or must be replaced. For example, the extent to which a prestressing tendon has been strained (is it still within its elastic limit or has it exceeded the yield strength?), the remaining fatigue resistance, etc.

The effect of loads imposed on the structure can be evaluated and compared to the load-carrying capacity of the structure. Due account should be taken of various possible structural systems (redistribution caused by cracking, plastic hinges, etc.) and any reserve of the structural system (transverse distribution). Calculations can then be made to determine stresses and crack widths under conditions of actual use and the safety factors in the calculated conditions of failure. In this manner, it is possible to approximate the load-carrying behavior and effect of any strengthening which may be necessary.

3.2.1 Loading Conditions:

The design loads used to analyze the building were estimated according to material properties and building utilization in accordance with BS 6399-1996.

Load combinations for design assessment purposes were chosen according to BS 8110- 1985 the load categories considered in determining the design loads are:

1. Self weight of the structure.

2. Dead load from partitions and floors.
3. Live loads emanating from the intended use of each floor.

Due to the fact that the overall building height is less than 19m, the effect of wind and seismic loads was neglected.

3.2.1.1 Dead and live loads:

The self weight of the structure was computed using a density of 2400 kg/m³ for concrete. The weight of each member in the structure was computed by using the member's dimensions to calculate the volume and hence the weight under a gravitational acceleration value of 9.81 m/s².

Finishing for floors were assumed to be 1.5 kN/m² for each floor, while live loads of 13.5 kN/m were used to simulate the effect of wall partitions in the locations identified from the architectural drawings supplied by the client.

The dead weight of the one brick walls was estimated assuming a wall density of 18.0 kN/m³ and a wall thickness of 0.25 m (one brick wall). Live loads for the buildings were adopted as shown in table 3.1.

Table 3.1: Design live Loads

Floor usage	Live load
rooms	2.5kN/m ²

3.2.1.2 Load Combinations:

In order to calculate the stresses to which the individual members of the structure will be subjected during the structure's life, the load combination shown in table 3.2 were used. These combinations were chosen in accordance with BS 8110 - 1998.

Table 3.2: Design load combination

Floor usage	Live load
Load combination 1	1.4 D +1.6 L
Load Combination 2	1.0 D +1.0 L

D = Dead Load

L = Live Load

3.2.2 Material Properties:

Material properties used in the analysis and design assessment of the structure are shown in table 3.3.

Table 3.3: Material Properties

Property	Adopted Value	Source
Concrete Compressive Strength (F_{CU})	25 N/mm ²	Client
Steel Tensile Strength (F_Y)	420 N/mm ²	Client
Allowable Soil Bearing Capacity	180 N/mm ²	Client

3.3 Building Modeling:

A three dimensional finite element model of the building in question was generated for one half of the symmetrical building using Axis VM software and the drawings provided by the client (Appendix A & B). The model was used to compute the internal forces in the building structural members due to the different load combinations.

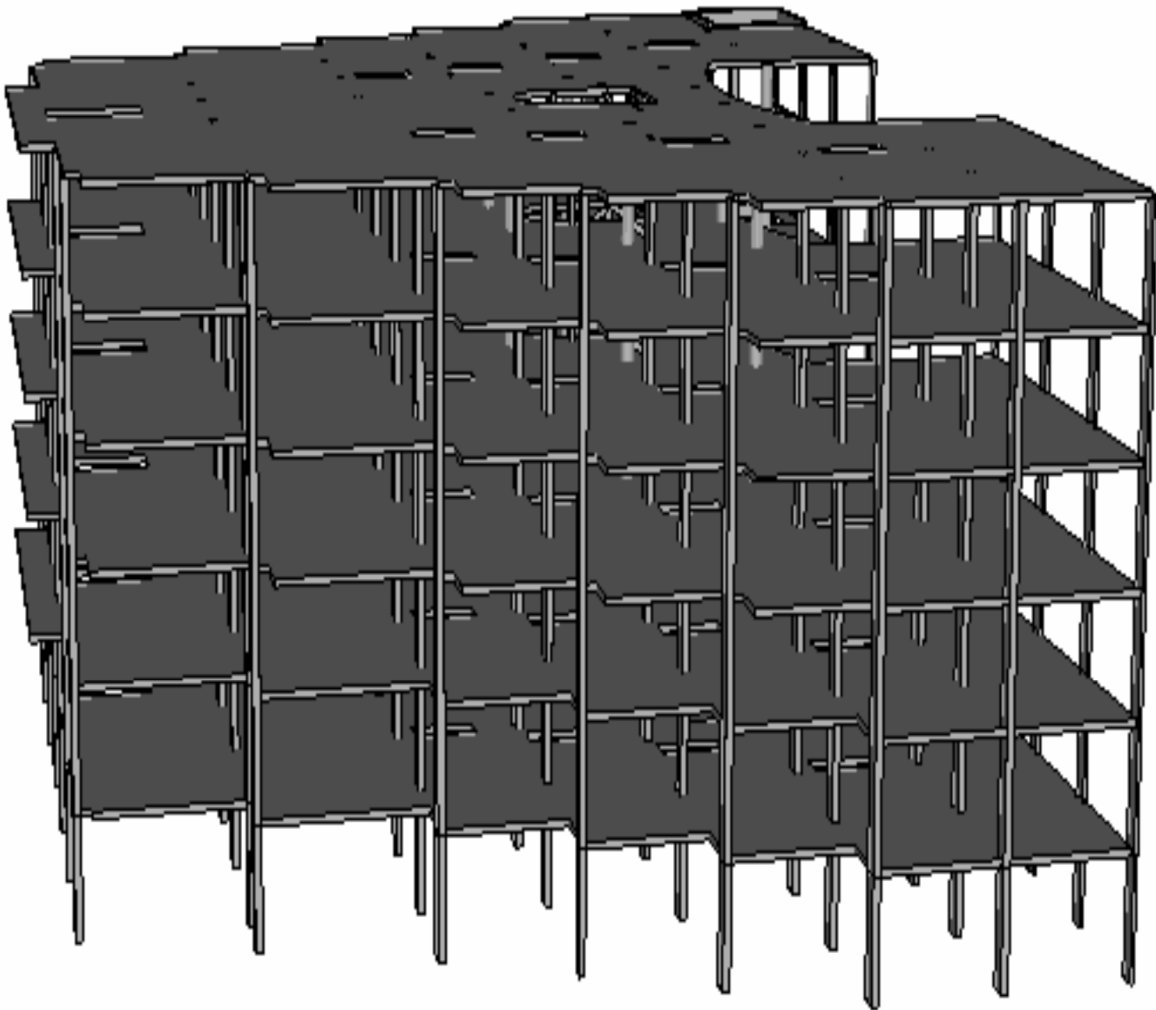


Figure 3.1: Rendered View of Model

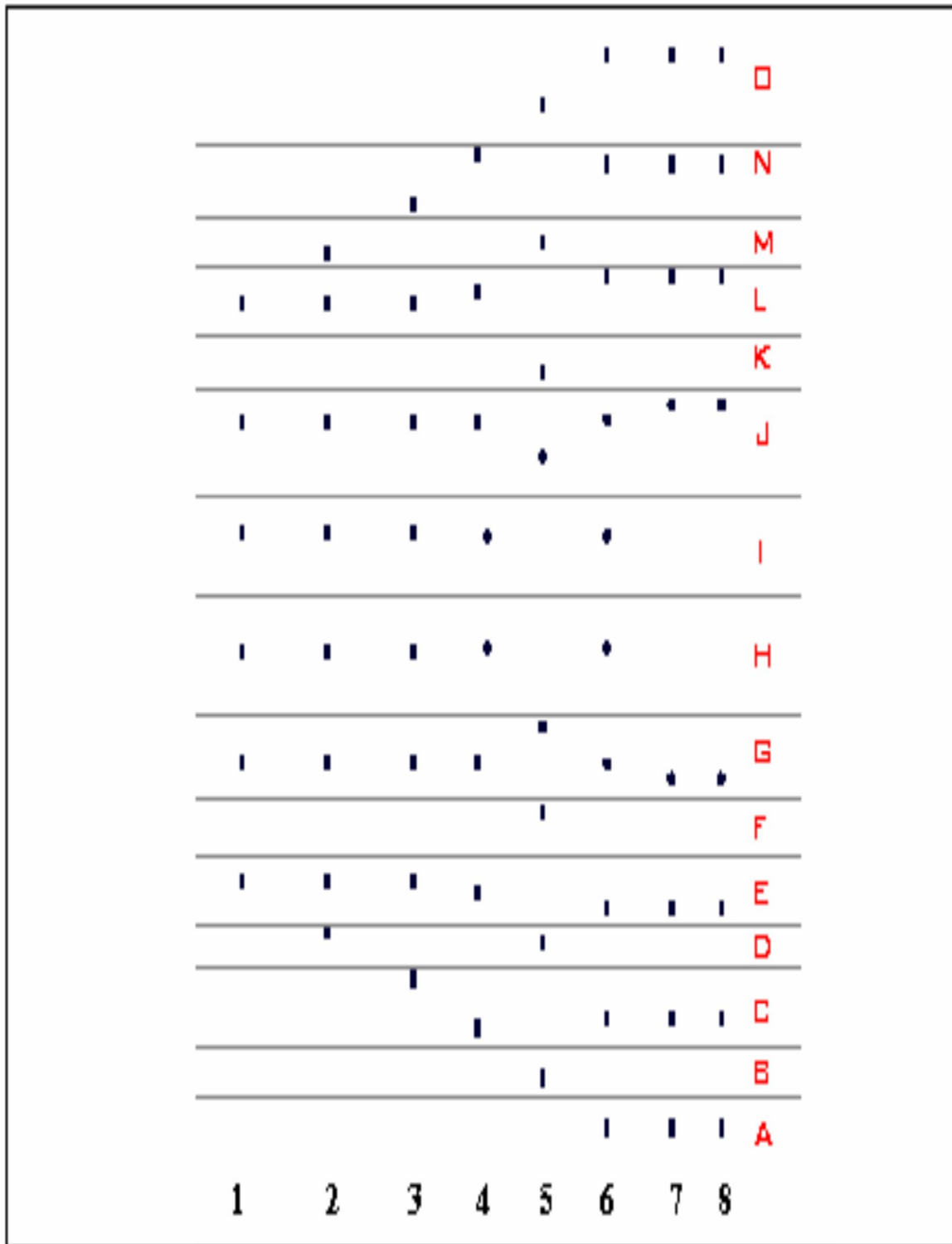


Figure 3.2: Footing designation

3.4 Modeling Results:

The model results were summarized to compute the internal forces in the structure components constituting the building (isolated footing, columns, and slabs). Assessment of each of these components for building is shown hereafter.

3.4.1 Foundation Assessment:

The footings of the building are a mix of type F1 (2.20m×2.20m×0.5m) and F2 (2.40m×2.40m×0.5m). The capacity of the footings was checked for adequacy to carry the obtained footing loads under load combination 2 (Service Load). The results show that 30 of the 60 footing in the analyzed part of the building were inadequately sized by more than 30% and some of footing (14) are severally undersized to the extent that the pressure they apply on the soil exceed twice the allowable bearing capacity.

Table 3.4: Foundation forces due to service load.

Footing No.	Footing Type.	R(x) [kN]	R(y) [kN]	R(z) [kN]	R(xx) [kNm]	R(yy) [kNm]	utilization
A6	F1	-3.668	-4.467	-806.986	4.765	-3.913	97%
A7	F1	1.771	-6.529	-947.204	6.964	1.889	113%
A8	F1	1.388	-1.33	-412.774	1.419	1.481	97%
B5	F1	-4.939	-5.639	-1157.14	6.014	-5.268	138%
C3	F1	-6.196	-5.234	-1109.43	5.583	-6.609	133%
C4	F1	-6.424	-6.368	-1219.00	6.793	-6.853	146%
C6	F2	2.579	4.019	-1680.47	-4.15	2.751	165%
C7	F2	-1.298	1.24	-1509.99	-1.2	-1.384	148%
C8	F2	2.513	0.442	-674.246	-0.417	2.68	133%
D5	F2	2.949	4.646	-2111.72	-4.784	2.934	207%
E1	F1	-8.179	-6.037	-957.169	6.439	-8.724	116%
E3	F2	4.427	4.336	-2085.87	-4.625	4.722	205%
E4	F2	-0.469	3.207	-2109.29	-3.25	-0.704	206%
E6	F2	0.614	-3.119	-1638.12	3.327	0.654	161%
E7	F2	0.052	-1.22	-1607.189	1.432	0.056	157%
E8	F2	2.523	-0.76	-672.614	0.865	2.692	132%
F5	F2	1.198	5.594	-1696.823	-5.829	0.972	167%
G1	F1	-9.569	0.787	-1304.432	-0.84	-10.207	156%
G2	F2	2.776	1.37	-2535.316	-1.461	2.961	247%
G3	F2	4.526	-0.958	-2214.558	1.022	4.828	217%
G4	F2	1.191	3.421	-1696.967	-3.649	0.971	167%
G5	F2	-2.852	-2.671	-1584.203	2.849	-3.042	156%
G6	F2	2.26	2.786	-1423.45	-2.972	2.41	140%
G7	F2	-4.156	-0.18	-972.278	0.192	-4.547	97%
G8	F2	5.143	-1.156	-389.111	1.233	5.486	79%
H1	F1	-14.467	-0.544	-1571.544	0.58	-15.431	189%
H2	F2	4.584	-4.248	-2613.403	4.531	4.889	252%
H3	F2	4.724	-4.099	-2120.09	4.372	5.038	208%
H4	F2	5.518	-0.013	-1749.447	0.014	4.685	172%
H6	F2	-0.568	1.358	-858.668	-1.448	-0.606	85%
H7	F2	-2.257	3.408	-238.967	-3.635	-2.407	26%
H8	F2	5.285	1.649	-84.358	-1.759	5.631	11%
I1	F1	-14.459	0.564	-1570.858	-0.602	-15.423	189%
I2	F2	4.603	4.322	-2612.27	-4.61	4.909	256%
I3	F2	4.659	4.094	2135.336	-4.367	4.969	208%
I4	F2	4.764	1.903	-1760.308	-2.03	3.874	172%
I6	F2	-0.4	-1.645	-863.541	1.755	-0.427	85%
I7	F2	-2.427	-4.814	-235.443	5.134	-2.589	26%
I8	F2	5.299	-1.545	-82.928	1.648	5.646	11%

J1	F1	-9.605	-0.833	-1303.254	0.889	-10.245	156%
J2	F2	2.717	-1.73	-2550.093	1.845	2.898	247%
J3	F2	4.493	-1.653	-2334.838	1.763	4.793	217%
J4	F2	1.283	-5.212	-1740.353	5.559	1.061	167%
J5	F2	-2.49	2.654	-1580.331	-2.831	-2.656	156%
J6	F2	2.528	-3.75	-1412.87	4	2.696	140%
J7	F2	-3.994	-1.054	-985.943	1.125	-4.376	97%
J8	F2	5.258	0.775	-397.587	-0.826	5.609	79%
K5	F2	1.624	-8.022	-1667.287	8.422	1.432	167%
L1	F1	-8.311	6.127	-951.7	-6.536	-8.865	116%
L3	F2	4.779	-1.611	-2142.217	1.718	5.098	205%
L4	F2	-1.788	-1.186	-2130.42	1.092	-2.113	206%
L6	F2	3.058	3.841	-1665.016	-4.097	3.262	161%
L7	F2	-0.313	2.073	-1551.094	-2.337	-0.334	157%
L8	F2	3.19	1.87	-667.234	-2.049	3.403	132%
M5	F2	1.36	-2.129	-2131.234	2.098	1.237	207%
N3	F1	-6.348	5.572	-1087.007	-5.943	-6.771	133%
N4	F1	-6.396	6.423	-1192.538	-6.852	-6.822	146%
N6	F2	3.659	-4.538	-1663.458	4.706	3.903	165%
O5	F1	-5.625	6.083	-1146.897	-6.488	-6	138%
O6	F1	-2.967	4.513	-854.674	-4.813	-3.165	97%

* Utilization = (Max Stress)/ (Allowable Bearing Capacity (B.C.)) × 100%

Example calculation for Utilization: foundation H²

Max Stress = 2613.403 / (2.4x2.4) = 453.72 kN/m²

Allowable Soil Bearing Capacity = 180 N/mm²

Utilization = (453.72 / 180) x 100% = 252 %

So the best way to solve this problem to strengthen all this foundation and enlarge it by using the Raft Foundation in this structure to be safe.

3.4.2 Columns Assessment:

The internal forces for each of the columns of the building are shown in table 3.5. These forces are presented using the designation shown in figure 3.3.

All of the existing columns of the building are a mix of type C1 (0.25m×0.6m) for the internal columns, type C2 (0.3m×0.6m) for external column and C3(round 0.5mm).The capacity of columns was checked for adequacy to carry the obtained column loads under load combination 1 (Ultimate Load). This check was conducted by using spreadsheets. The spreadsheet results show that Columns number A8, C6, C8, D5, E2, E3, E4, E6, E8, F5, G2, G3, G4, H1, H2,H3, I1, I2, I3, J2, J3, J4, K5, L2, L3, L4, L6, L8, M5, N6, N8 and O8 were inadequately sized at the short columns level. A section 900mm×550mm with 4Ø20mm and 8Ø12mm instead of the used 600mm×250mm with 8 Ø 20mm should have been adopted.

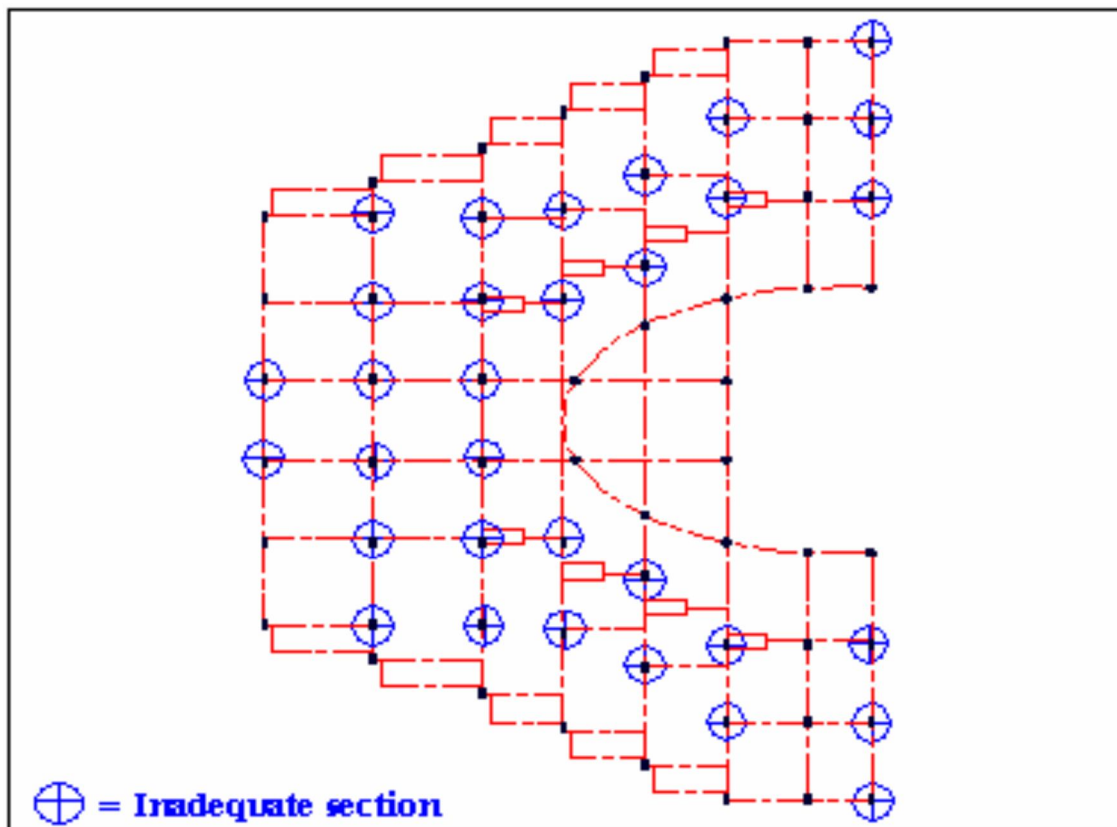


Figure 3.3: Inadequately sized short columns

At the ground floor level columns number D5, E3,E4,G2, G3, H1, H2, H3, I1, I2, I3, J2, J3, L3, L4, L5, M5, A8, C8, E8, L8, N8 and O8 were inadequately sized. A section 900mm×550mm with 4Ø20mm and 8Ø12mm instead of the adopted 600×mm250mm with 8 Ø 20mm should have been used.

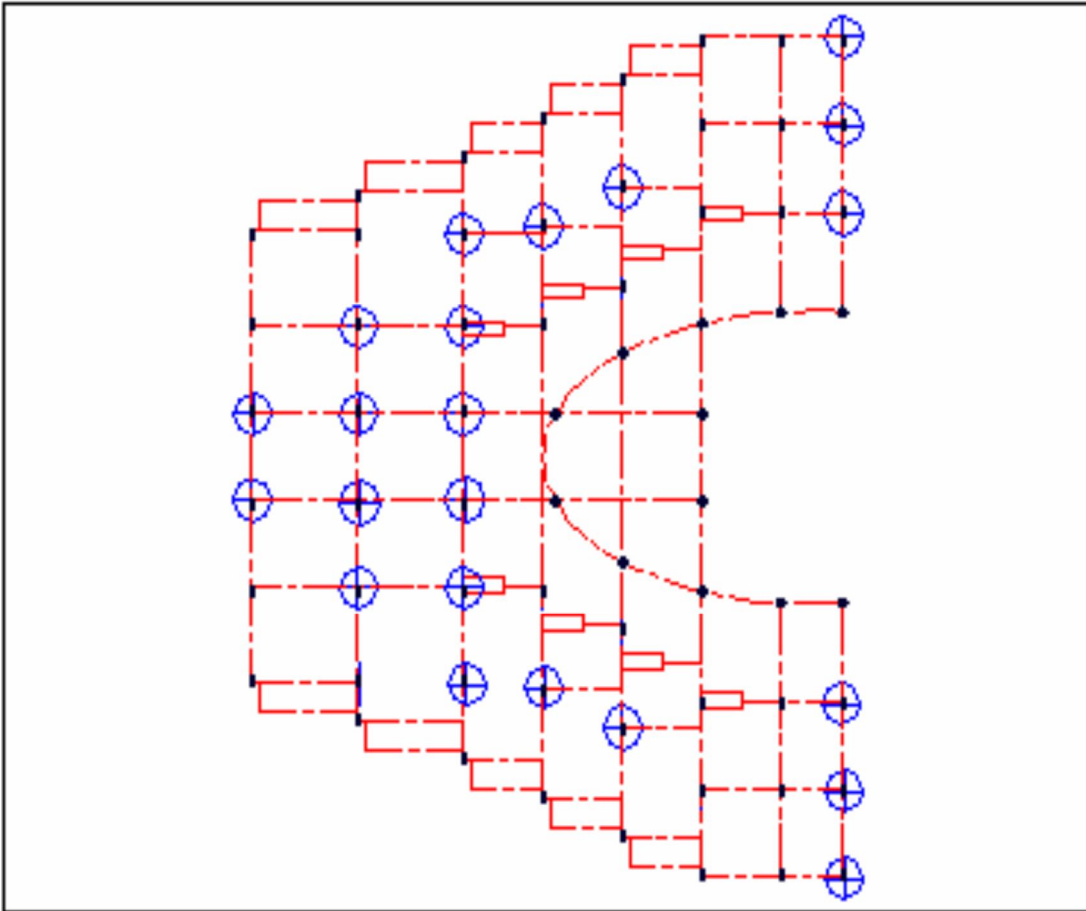


Figure 3.4: Inadequately sized ground floor columns

At the first floor columns level columns number G2, G3, H2, H3, I2, I3, J2, J3, A8, C8, E8, L8, N8 AND O8 were inadequately sized. A section

600mm×300mm with 4Ø20mm and 8Ø12mm instead of the used 600×250 with 8 Ø 20mm should have been adopted .

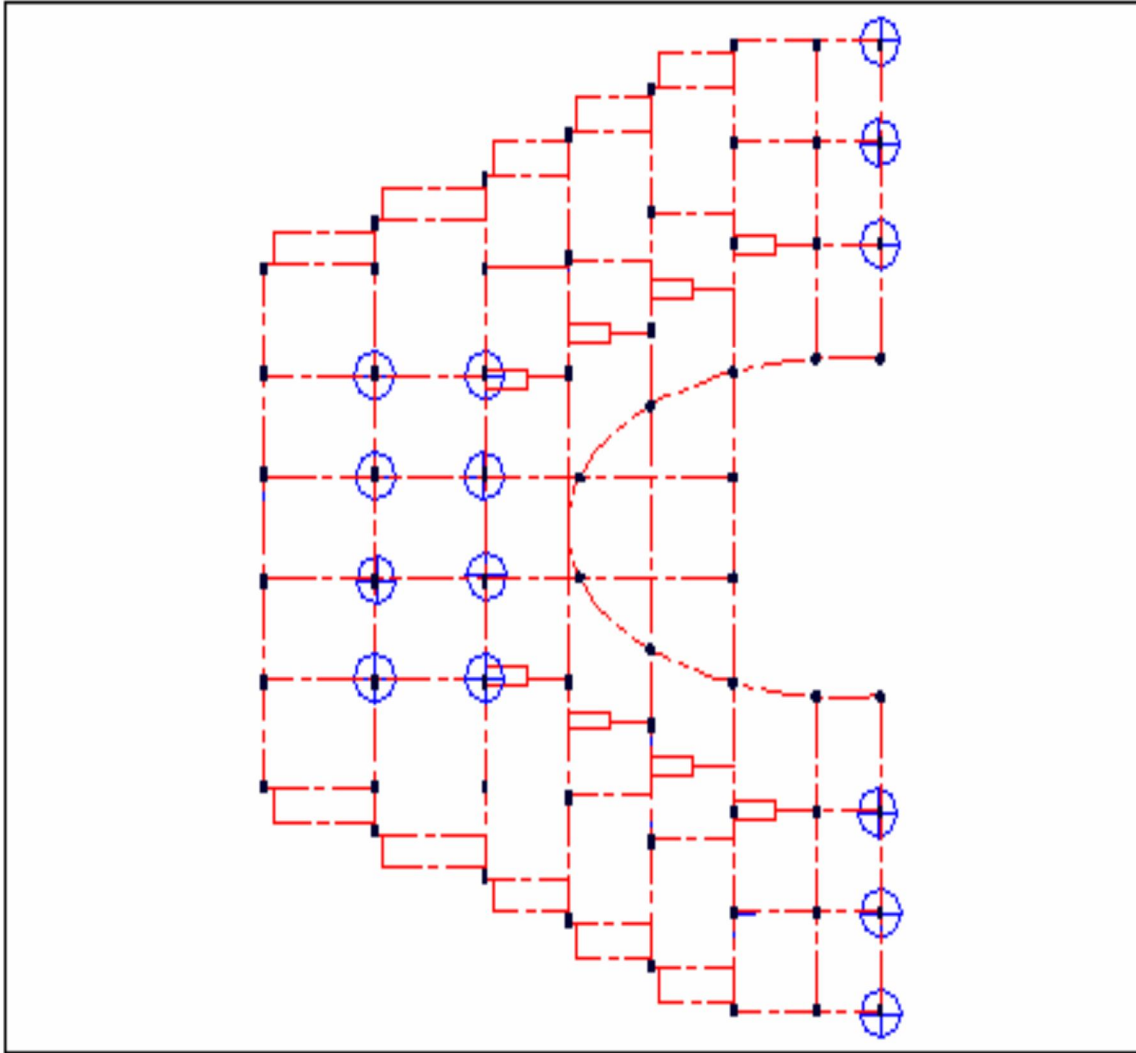


Figure 3.5: Inadequately first floor columns sized

Columns of the second floor and above floors were adequate except for columns number A8, C8, E8, L8, N8 and O8. A section that is

600mm×200mm with 8 Ø 20mm should have been adopted instead of the used 600mm×125mm with 8 Ø 20mm.

3.4.2.1 Columns Result:

3.4.2.1.1 Short columns results:

Table 3.5: short columns results

Column No:	height (m)	N _x (kN)
A6	3.0	-1147.76
A7	3.0	-1354.33
A8	3.0	-589.19
B5	3.0	-1648.58
C3	3.0	-1579.45
C4	3.0	-1736.96
C6	3.0	-2415.62
C7	3.0	-2177.29
C8	3.0	-968.63
D2	3.0	-1175.08
D5	3.0	-3029.80
E1	3.0	-1362.84
E2	3.0	-2586.97
E3	3.0	-2973.39
E4	3.0	-3022.05
E6	3.0	-2353.38
E7	3.0	-2315.87
E8	3.0	-968.365

F5	3.0	-2427.88
G1	3.0	-1867.03
G2	3.0	-3618.68
G3	3.0	-3169.00
G4	3.0	-2426.64
G5	3.0	-2267.72
G6	3.0	-2051.70
G7	3.0	-1403.18
G8	3.0	-561.244
H1	3.0	-2243.08
H2	3.0	-3735.78
H3	3.0	-3038.60
H4	3.0	-2503.66
H6	3.0	-1233.89
H7	3.0	-347.143
H8	3.0	-122.399
I1	3.0	-2242.12
I2	3.0	-3734.18
I3	3.0	-3060.27
I4	3.0	-2519.00
I6	3.0	-1240.63
I7	3.0	-342.035
I8	3.0	-120.328
J1	3.0	-1865.39
J2	3.0	-3656.34
J3	3.0	-3337.31

J4	3.0	-2487.97
J5	3.0	-2262.36
J6	3.0	-2036.98
J7	3.0	-1422.57
J8	3.0	-573.25
K5	3.0	-2386.71
L1	3.0	-1354.94
L2	3.0	-2579.82
L3	3.0	-3068.8
L4	3.0	-3051.56
L6	3.0	-2391.47
L7	3.0	-2236.75
L8	3.0	-960.756
M2	3.0	-1171.77
M5	3.0	-3057.24
N3	3.0	-1548.06
N4	3.0	-1699.89
N6	3.0	-2391.94
N7	3.0	-677.225
N8	3.0	-615.28
O5	3.0	-1617.91
O6	3.0	-1215.62
O7	3.0	-453.135
O8	3.0	-136.79

3.4.2.1.2 Ground floor Columns results:

Table 3.6: Ground floor columns forces due to ultimate load

Column No:	height (m)	Nx (kN)
A8	3.2	-524.299
C6	3.2	-2032.11
C8	3.2	-847.776
D5	3.2	-2496.56-
E2	3.2	-2157.07
E3	3.2	-2534.15
E4	3.2	-2515.71
E6	3.2	-1963.13
E8	3.2	-840.134
F5	3.2	-1972.05
G2	3.2	-3078.92
G3	3.2	-2656.35
G4	3.2	-1983.03
H1	3.2	-1928.28
H2	3.2	-3091.79
H3	3.2	-2525.39
I1	3.2	-1927.58
I2	3.2	-3091.49
I3	3.2	2547.75
J2	3.2	-3043.26
J3	3.2	-2781.89
J4	3.2	-2021.76

J5	3.2	-1879.67
L2	3.2	-2170.99
L3	3.2	-2565.5
L4	3.2	-2486.51
L6	3.2	-1912.39
L8	3.2	-804.733
M5	3.2	-2440.16
N6	3.2	-1970.68
N8	3.2	-512.738
O8	3.2	-113.999

3.4.2.1.3 First floor Columns results:

Table 3.7: First floor columns forces due to ultimate load

Column No:	height (m)	N _x (kN)
A8	3.0	-390.132
C8	3.0	-629.925
D5	3.0	-1915.11
E3	3.0	-1935.02
E4	3.0	-1932.24
E8	3.0	-635.897
G2	3.0	-2539.17
G3	3.0	-2026.03
H1	3.0	-1541.21

H2	3.0	-2453.86
H3	3.0	-1995.25
I1	3.0	-1540.87
I2	3.0	-2451.15
I3	3.0	-2010.95
J2	3.0	-2386.95
J3	3.0	-2265.45
L3	3.0	-2058.48
L4	3.0	-2019.91
L8	3.0	-624.387
M5	3.0	-1954.02
N8	3.0	-367.515
O8	3.0	-93.236

3.4.2.1.4 Second floor column results:

Table 3.8: Second floor columns forces due to ultimate load

Column No:	height (m)	Nx (kN)
A8	3.0	-332.062
C8	3.0	-513.282
E8	3.0	-504.031
G2	3.0	-1999.40
G3	3.0	-1427.64
H2	3.0	-1754.32
H3	3.0	-1425.03

I2	3.0	-1752.14
I3	3.0	-1436.63
J2	3.0	-1706.38
J3	3.0	-1593.35
L8	3.0	-452.254
N8	3.0	-266.315
O8	3.0	-67.560

CHAPTER FOUR

RESULTS & DISCUSSION

4.1 Introduction:

The Strategies for Structural Strengthening is when a structure is going to be strengthened there are several aspects to be considered, such as the structure had inadequate load bearing capacity due to a design fault already present before it was taken into service. It was then strengthened slightly above the desired performance level. The damages were then repaired to a new satisfactory performance level. Later, the demands on the structure were changed, higher load bearing capacity was required, and the structure needed to be strengthened to a higher performance level to meet these demands and keep the structure in service.

The strengthening of the structures should be designed and constructed in accordance with appropriate building codes. If special codes for strengthening exist, there will of course be an assistance to the designers and contractors. However, this is seldom the case, and many problems in connection with strengthening are not dealt with in the normal building codes.

Typical problems of this kind are the transfer of shear forces between the old and new concrete applied for strengthening reinforcement. The post tensioning of the existing structure which in some respects is different from the post-tensioning of a new structure.

Satisfactory interaction between existing concrete and new concrete is generally required in strengthening and repair. As a rule, the aim is to get the structural parts, composed of the different concretes, to act as a homogeneously- cast structural component. To achieve this, the joint between the old concrete and the new concrete must be capable of transferring shear stresses without relative movements of such a magnitude that the static behavior is significantly affected. Furthermore, the joint must be durable for the environment in question, i.e. the composite structural component must not change its mode of action with time.

4.2 Evaluations of results of the structural assessment:

Data resulting from the investigation of a damaged structure form the basis for the decision as to what corrective actions must be undertaken. These depend on type and extent of the damage.

An initial concern should be as to whether or not there is a risk of failure to the damaged structure. If this risk is present, a first course of action must be to provide immediately an adequate auxiliary support mechanism to remove the risk. Where minor damage exists a determination must be made as to whether the damage is stable or whether it will propagate with subsequent service loading. This is often a difficult subjective assessment, made on the basis of a visual examination, until such time as verifying calculations can be accomplished.

The elements of time and/or the harshness of the environment become important parameters when either the load-carrying capacity is being diminished by deterioration (corrosion, etc.) or when the load-carrying capacity has to be increased (increased traffic volume).

Some evaluations will relate to whether or not an economically effective repair can limit or contain damage and can thus increase the effective life of structure. In some instances an evaluation will be concerned with the degree of urgency required to implement a repair, because of the advanced stage of damage.

The urgency of repairs, strengthening or replacement must be evaluated in a technical sense along with realistic cost estimates, so that proper priorities can be established in budget planning. The question of urgency must also consider an estimation of remaining life expectancy. Although assumptions in a time-dependent damage process are subjective at best, this evaluation will make an assessment of the degree of urgency required less complex.

4.3 Columns problems and solutions:

All the internal and external Columns in this building (case study) are under sized due to construction faults or errors in planning and construction due to insufficient design detailings. so the capacity of columns cannot carry the obtained column loads under load combination 1 (Ultimate Load). In this chapter we strengthen the internal and external Columns to carry the ultimate load , and determine the new Columns sections.

4.3.1 Sample Design Calculation for Strengthening of Internal Column (H2):

The existing Internal columns section is 250 x 600 mm with steel reinforcement 8 Ø 20mm

Take the most critical section for the internal Columns and use the same design section for all internal columns.

- Concrete density = 2400 kg/m³

- Finishing for floors = 1.5 kN/m^2
- Partitions (wall) = 13.5 kN/m
- Live load (rooms) = 2.5 kN/m^2

Load combination (1): $1.4 D + 1.6 L$

Load combination (2): $1.0 D + 1.0 L$

Loaded Area for column H2 = $4.75 \times 6 = 28.5 \text{ m}^2$

Roof load :

Self weight = $0.2 \times 24 = 4.8 \text{ kN/m}^2$

Finishes = 1.5 kN/m^2

Live load = 2.5 kN/m^2

Total load on roof = 8.8 kN/m^2

Typical floor:

Self weight = $0.2 \times 24 = 4.8 \text{ kN/m}^2$

Finishes = 1.5 kN/m^2

Walls = 4.5 kN/m^2

Live load = 2.5 kN/m^2

Total floor load = 13.3 kN/m^2

Use column weight of $600 \times 250 \times 3000 \text{ mm}$ height

$$= 0.60 \times 0.25 \times 3.0 \times 24 = 10.8 \text{ kN}$$

For 7 columns = $7 \times 10.8 = 75.6 \text{ Kn}$

Service load:

$$S = 28.5 (8.8 + (13.3 \times 6)) + 75.6$$

$$= 2607 \text{ kN}$$

Ultimate load:

$$\text{Roof load} = (1.4 \times 6.3) + (1.6 \times 2.5) = 12.82 \text{ kN/m}^2$$

$$\text{Floor load} = (1.4 \times 10.8) + (1.6 \times 2.5) = 19.12 \text{ kN/m}^2$$

$$\begin{aligned}\text{Ultimate load} &= (28.5 \times (12.82 + 6 \times 19.12)) + (1.4 \times 75.6) \\ &= 3740 \text{ kN}\end{aligned}$$

1. Short Column H2

$$N = 0.35 f_{cu} \times A_c + 0.67 f_y \times A_{sc}$$

Existing column 250 x 600 with steel reinforcement 8 Ø 20mm.

$$\begin{aligned}&= 0.35 \times 25 \times 250 \times 600 + 0.67 \times 420 \times 2512 \\ &= 2019 \text{ kN} < 3740 \text{ kN}\end{aligned}$$

The Short column is inadequately.

By increase the column size to 550x 900 mm and steel bar to (4 Ø 20mm + 8 Ø 12mm) .

$$\begin{aligned}N &= 0.35 \times 25 \times 550 \times 900 + 0.67 \times 420 \times 4672 \\ &= 5646 \text{ kN} > 3618.68 \text{ kN which is ok}\end{aligned}$$

2. Ground floor Column H2

$$\begin{aligned}\text{Ultimate load} &= (28.5 \times (12.82 + 5 \times 19.12)) + (1.4 \times 64.8) \\ &= 3180 \text{ kN} > 2019 \text{ kN}\end{aligned}$$

The ground floor column 250 x 600 is in adequately.

Use the same short column section 550 x 900 with steel reinforcement (4 Ø 20mm + 8 Ø 12mm).

$$N = 5646 \text{ kN} > 3180 \text{ kN}$$

Then the column is ok.

3. First Floor column H2

$$\text{Ultimate load} = (28.5 \times (12.82 + 4 \times 19.12)) + (1.4 \times 54)$$

$$= 2621 \text{ kN} > 2019 \text{ kN}$$

The First Floor column 250 x 600 with steel reinforcement 8 Ø 20mm is inadequately.

Use the same column section 550 x 900 with steel reinforcement (4 Ø 20mm + 8 Ø 12mm).

Then the column is adequate to carry the load.

4. Second Floor column H2

$$\begin{aligned} \text{Ultimate load} &= (28.5 \times (12.82 + 3 \times 19.12)) + (1.4 \times 43.2) \\ &= 2061 \text{ kN} > 2019 \text{ kN} \end{aligned}$$

The second Floor column Section 250 x 600mm with steel reinforcement 8 Ø 20mm is inadequately.

Use the same column section 550 x 900 with steel reinforcement (4 Ø 20mm + 8 Ø 12mm).

Then the column is adequate to carry the load.

5. Third floor column H2

$$\begin{aligned} \text{Ultimate load} &= (28.5 \times (12.82 + 2 \times 19.12)) + (1.4 \times 32.4) \\ &= 1501 \text{ kN} \end{aligned}$$

Which is safe.

$$\begin{aligned} N &= 0.35 \times 25 \times 250 \times 600 + 0.67 \times 420 \times 2512 \\ &= 2019 \text{ kN} > 1501 \text{ kN} \end{aligned}$$

Then the third floor column 250 x 600 with steel reinforcement 8 Ø 20mm is Satisfy.

So we must to increase all the Internal Columns sections 250 x 600mm for (short, Ground, First and second floor columns) to column section 550 x 900

with steel reinforcement (4 Ø 20mm + 8 Ø 12mm). by Strengthening the Columns by Concrete jacket to increase the section capacity to carry the load.

The strengthening detailing shown in Appendix (C), and the Construction procedures for Strengthening the external Columns shown in Appendix (D1) & (D2).

Use the same section for all internal columns.

4.3.2 Sample Design Calculation for Strengthening of External Column (E2):

The existing External columns section is 300 x 600 mm with steel reinforcement 8 Ø 20mm.

Take the most critical section for the External Columns is E2

Concrete density = 2400 kg/m³

- Finishing for floors = 1.5 kN/m²
- Partitions (wall) = 13.5 kN/m
- Live load (rooms) = 2.5 kN/m²

Load combination (1): 1.4 D + 1.6 L

Load combination (2): 1.0 D + 1.0 L

Loaded Area for column E2 = 20.25 m²

Roof load :

Self weight = 0.2 x 24 = 4.8 kN/m²

Finishes = 1.5 kN/m²

Live load = 2.5 kN/m²

Total load on roof = 8.8 kN/m²

Typical floor:

Self weight = 0.2 x 24 = 4.8 kN/m²

Finishes = 1.5 kN/m²

Walls = 7 kN/m²

Live load = 2.5 kN/m²

Total floor load = 15.8 kN/m²

Use column weight of 600 x 300 x 3000mm height

$$= 0.60 \times 0.30 \times 3.0 \times 24 = 12.96 \text{ kN}$$

For 7 columns = 7 x 12.96 = 90.7 kN

Service load:

$$S = 20.25 (8.8 + (15.8 \times 6)) + 90.7$$

$$= 2188 \text{ kN}$$

Ultimate load:

$$\text{Roof load} = (1.4 \times 6.3) + (1.6 \times 2.5) = 12.82 \text{ kN/m}^2$$

$$\text{Floor load} = (1.4 \times 13.3) + (1.6 \times 2.5) = 22.62 \text{ kN/m}^2$$

$$\begin{aligned} \text{Ultimate load} &= (20.25 \times (12.82 + 6 \times 22.62)) + (1.4 \times 90.7) \\ &= 3135 \text{ kN} \end{aligned}$$

1. Short Column H2

$$N = 0.35 f_{cu} \times A_c + 0.67 f_y \times A_{sc}$$

Existing column 300 x 600 with steel reinforcement 8 Ø 20mm.

$$= 0.35 \times 25 \times 300 \times 600 + 0.67 \times 420 \times 2512$$

$$= 2281.9 \text{ kN} < 3135 \text{ kN}$$

The Short column is inadequately.

By increase the column size to 450 x 900 mm and steel bar to (4 Ø 20mm + 5 Ø 12mm) .

$$N = 0.35 \times 25 \times 450 \times 900 + 0.67 \times 420 \times 4342$$

$$= 4765 \text{ kN} > 3135 \text{ kN} \text{ which is ok}$$

2. Ground floor Column E2

$$\begin{aligned}\text{Ultimate load} &= (20.25 \times (12.82 + 5 \times 22.62)) + (1.4 \times 78) \\ &= 2632 \text{ kN} > 2281.9 \text{ kN}\end{aligned}$$

The ground floor column 300 x 600 is inadequate.

Use the same short column section 450 x 900 with steel reinforcement (4 Ø 20mm + 5 Ø 12mm).

$$N = 4765 \text{ kN} > 2632 \text{ kN}$$

Then the column is ok.

3. First Floor column E2

$$\begin{aligned}\text{Ultimate load} &= (20.25 \times (12.82 + 4 \times 22.62)) + (1.4 \times 65) \\ &= 2183 \text{ kN} > 2281.9 \text{ kN}\end{aligned}$$

The First Floor column 300 x 600 with steel reinforcement 8 Ø 20mm is inadequate.

Use the same column section 450 x 900 with steel reinforcement (4 Ø 20mm + 5 Ø 12mm).

Then the column is adequate to carry the load.

4. Second Floor column H2

For more safety use the same strengthening section 450 x 900 with steel reinforcement (4 Ø 20mm + 5 Ø 12mm) for the second floor.

So we must to increase all the External Columns sections 300 x 600mm for short, Ground, first and second floor columns to column section 450 x 900 with steel reinforcement (4 Ø 20mm + 5 Ø 12mm) by Strengthening the

Columns with Concrete jacket to increase the section capacity to carry the load.

The strengthening detailing shown in Appendix (E), and the Construction procedures for Strengthening the Columns shown in Appendix (F1) & (F2).

Use the same section for all external Columns.

The strengthening process for Columns are show in Appendix (M) & (N) & (O) & (P) & (Q) and (R).

4.4 Foundations problems and solutions:

The foundations for this case study are a mix of two type of pad foundations F1 (2.20m×2.20m×0.5m) and F2 (2.40m×2.40m×0.5m). The capacity of the footings was checked for adequacy to carry the obtained footing loads under load combination 2 (Service Load).

The results show that 30 of the 60 footing in the analyzed part of the building were inadequately sized by more than 30% and some of footing (14) are severally undersized to the extent that the pressure they apply on the soil exceed twice the allowable bearing capacity. Accordingly

So we must to solve this problem by changing the mechanism of the pad foundations to be in raft foundations to increase the bearing capacity and to save the foundations from the punching shear.

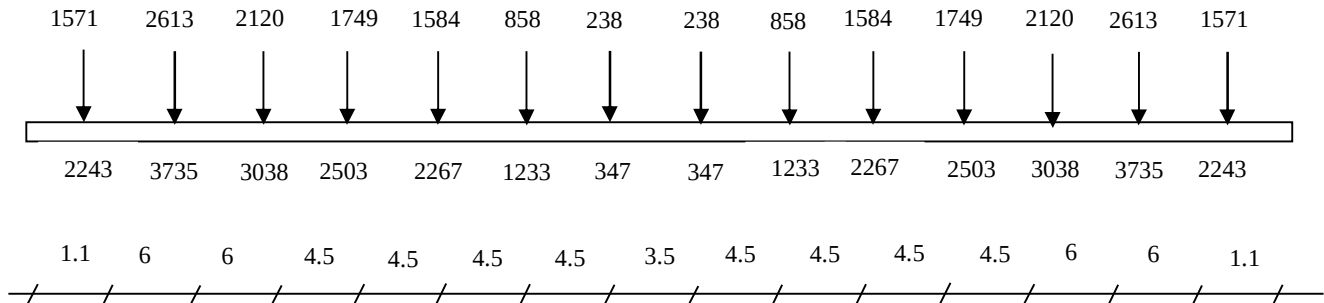
4.4.1 Design of foundation:

The footing of the building was checked for adequacy to carry the obtained footing loads under the service load combination , and the results show that 30 to the 60 footing often analysis were inadequately sized by more than 30% and some of the footing are severally undersized .

So the best solution is to enlarge the Pad Foundation to Raft Foundation to meet the code requirements, and make the building safe .

4.4.1.1 Design of Raft Foundation:

Take the critical strip to design the Raft Foundation



The following applied load per kN and the distance per meter Column dimension after strengthening 550*900 mm

allowable soil bearing capacity = 180 kN/m²

The width of strip for design = (4.5 + 4.75) / 2 = 4.625 m

Area of strip = 4.625 * 65.7 = 303.8 m²

$$\text{Net stress } \sigma_n = \frac{\sum p_s}{A}$$

$\sum p_s$ = total service load = 21466 kN

$$\sigma_n = \frac{21466}{303.8} + (0.9 \times 24) = 92.25 \text{ kN/m}^2 < \text{allowable stress}$$

180 kN/m² which is ok

Structural design of Raft foundation :

Use Raft of uniform thickness

The ultimate stress $\sigma_u = \frac{\Sigma p_u}{A}$

Σp_u = ultimate load = 30732 kN

$$\sigma_u = \frac{30732}{303.8} = 101.15 \text{ kN/m}^2$$

Absolute maximum design moment :

$$M = \frac{\sigma_u * l^2}{8} = \frac{101.15 * 6^2}{8} = 455 \text{ kN.m}$$

Assume the thickness of Raft foundation = 900 mm

The effective depth = 900 – 80 = 820 mm

Bending reinforcement

$$K = \frac{M}{b d^2 f_{cu}} = \frac{455 * 10^6}{1000 * 820^2 * 25} = 0.027$$

From the lever arm curve $l_a = 0.95$ therefore

Effective depth $z = l_a * d = 0.95 * 820 = 780 \text{ mm}$

$$A_s = \frac{M}{0.95 f_{yv} z} = \frac{455 * 10^6}{0.95 * 420 * 780} = 1461 \text{ mm}^2$$

Provide five T 20 bars . area = 1570 mm²

Check the punching shear:

1- At the column face:

The column section 550*900mm

Perimeter $u = 2 (550 + 900) = 2900 \text{ mm}$

Shear force $V = 3735 \text{ kN}$ in critical section

To allow for the effects of moment transfer :

V is increased by 15 percent . thus :

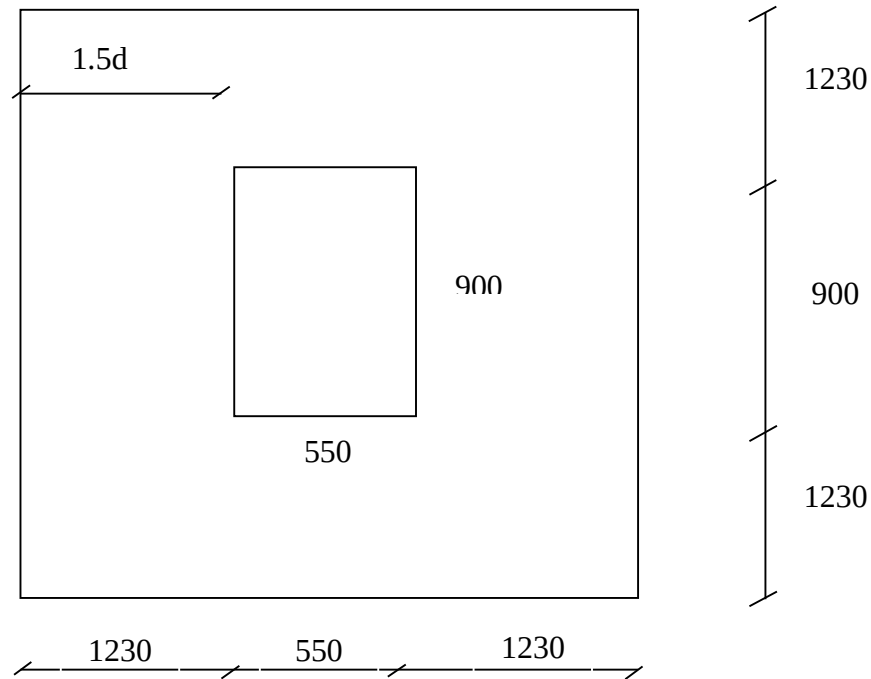
$$\gamma = \frac{1.15v}{ud} = \frac{1.15 * 3535 * 10^6}{2900 * 820} = 1.81 \text{ N/mm}^2 < 0.4 \sqrt{f_{cu}} = 4 \text{ N/mm}^2$$

2- The critical section for shear is 1.5 * effective depth

$$= 1.5 * 820 = 1230 \text{ mm From the column face}$$

Thus the length of perimeter

$$U = 2 (550 + 2 * 1230) + 2 (900 + 2 * 1230) = 12740 \text{ mm}$$



Ultimate shear force = 3735 kN

$$V = 3735 - \{ (0.55 + 2 * 1.230) * (0.9 + 2 * 1.230) * 101.15 \} = 2712 \text{ kN}$$

Thus the shear force :

$$\gamma = \frac{1.15 * 2712 * 10^3}{12740 * 820} = 0.30 \text{ N/mm}^2$$

$$\frac{100 A_s}{b d^2} = \frac{100 * 1570}{1000 * 820^2} = 0.2$$

By inspection of the table from code the value of ultimate shear stress $V_c = 0.39 \text{ N/mm}^2$.

Which is greater than shear stress = 0.3 N/mm^2

Thus Strengthen to Raft Foundation with depth 900 mm .

and steel reinforcement T 20 @ 200 mm c/c

So the Pad Foundation enlarge to Raft foundation to meet the code requirement and make the building safe .

The Raft foundation detail are shown in Appendix (G1 & G2 & G3 & G4 & G5 & G6). According to the different levels of the old foundations.

Appendix (H) :Detailing of the shear keys.

The enlarge process from isolated foundations to Raft Foundation are shown in Appendix (S) & (T) & (U) & (V) & (W) & (X) and (Y).

4.5 Flat slab Assessment:

The Slabs in the building (case study) were structurally analyzed under ultimate loads (load combination 1) using the three dimensional model.

Reinforcement module and the needed reinforcements for the slab section are summarized in table 12-5. The check results indicate that the slab design for this building is adequate.

Table 4.1: Slab reinforcement

Area of Reinforcement	Required reinforcement mm ² /m ²		Provided reinforcement mm ² /m ²		Status	
	area at X-X direction	area at YY direction	area at X- X direction	area at YY direction	x-x direction	y-y direction
Column strip (Top)	1281	1075	1340	1340	Adequately	Adequately
Column strip (Bottom)	560	716	750	750	Adequately	Adequately
Middle strip(Top)	640	478	670	670	Adequately	Adequately
Middle strip(Bottom)	689	573	750	750	Adequately	Adequately

Punching Shear check is taken in four critical areas as shown in the figure 4.1 below which include:

- Internal perimeter A&B
- Edge D
- Edge C

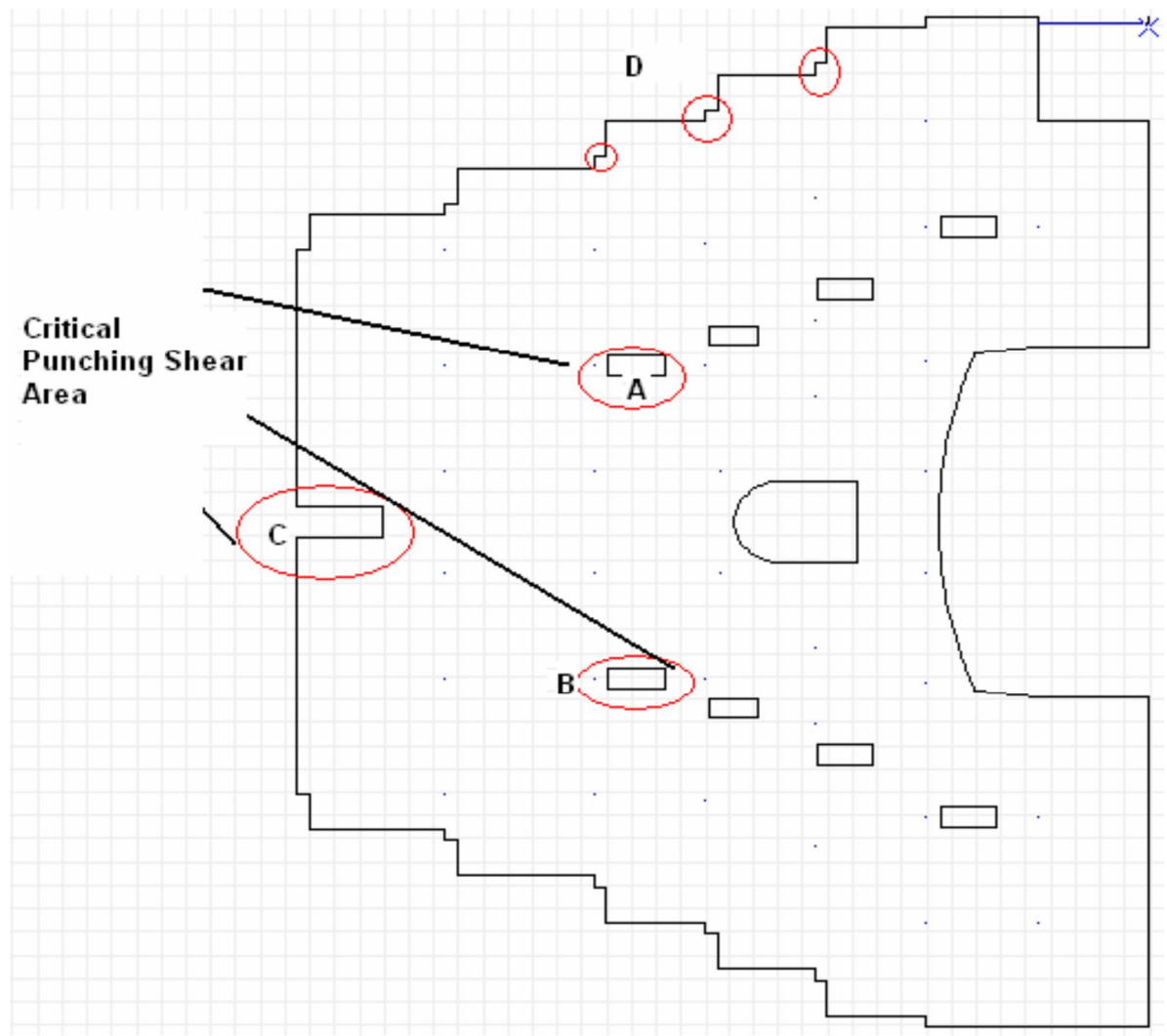


Figure 4.1: Critical Punching shear areas

4.5.1 Punching shear check for flat slab:

- Punching Shear Check for Critical Areas A and B are shown in Appendix (H).
- Punching shear check for critical area C are shown in Appendix (j).
- Punching shear check for critical area D are shown in Appendix (K).

CHAPTER FIVE

CONCLUSIONS AND RECOMMENDATIONS

5.1. The research summary:

This research deals with strengthening of reinforced concrete structure, (Columns, Beams, Foundations, Slabs and RC walls).

However, Concrete structures need to be strengthened for many cases such as the errors in planning or construction due to insufficient design dimensions and/or insufficient reinforcing steel or Damage to structural parts due to aging of construction materials or fire damage, corrosion of steel reinforcement, also Load increases due to higher live loads

In this research the main objective of the study is to re-analysis and assessment of local building which is subjected to failure due to bad construction and the difference between the actual design and the structural sections for existing building.

It comprises a six story reinforced concrete building. The building is composed of a flat slab system that is supported by reinforced concrete columns which transfer the structure's loads to 100 isolated footings.

The model results were summarized to compute the internal forces in the structure components constituting the building (isolated footing, columns, and slabs).

The footings of the building are a mix of type F1 (2.20m×2.20m×0.5m) and F2 (2.40m×2.40m×0.5m). The capacity of the footings was checked for adequacy to carry the obtained footing loads under load combination 2 (Service Load). The results show that 30 of the 60 footing in the analyzed part of the building were inadequately sized by more than 30% and some of footing (14) are severally undersized to the extent that the pressure they apply on the soil exceed twice the allowable bearing capacity.

After re-designed of the foundations, the Pad Foundation enlarge to Raft foundation to meet the code requirement and make the building safe .

All of the existing columns of the building are a mix of type C1 (0.25m×0.6m) for the internal columns, type C2 (0.3m×0.6m) for the external column and C3(round 0.5mm).The capacity of columns was checked for adequacy to carry the obtained column loads under load combination 1 (Ultimate Load). This check was conducted by using spreadsheets. The spreadsheet results show that Columns number A8, C6, C8, D5, E2, E3, E4, E6, E8, F5, G2, G3, G4, H1, H2,H3, I1, I2, I3, J2, J3, J4, K5, L2, L3, L4, L6, L8, M5, N6, N8 and O8 were inadequately sized at the short columns level.

The results after re-design is to increase all the Internal Columns sections 250 x 600mm for (short, Ground, First and second floor columns) to column section 550 x 900 with steel reinforcement (4 Ø 20mm + 8 Ø 12mm). by Strengthening the Columns with Concrete jacket to increase the section capacity to carry the load.

The strengthening detailing shown in Appendix (A).

And also to increase all the External Columns sections 300 x 600mm for short, Ground, first and second floor columns to column section 450 x 900 with steel reinforcement (4 Ø 20mm + 5 Ø 12mm) by Strengthening the Columns with Concrete jacket to increase the section capacity to carry the load.

The strengthening detailing shown in Appendix (D),

The Slabs in the building (case study) were structurally analyzed under ultimate loads (load combination 1) using the three dimensional model.

Reinforcement module and the needed reinforcements for the slab section are summarized in table 12-5. The check results indicates that the slab design for this building is adequate.

5.2. The research Conclusion:

Based on the analysis results of the local building for (isolated footing, columns, and slabs) the following conclusions can be reported:

- The capacity of the footings was checked for adequacy to carry the obtained footing loads under load combination 2 (Service Load). The results show that 30 of the 60 footing in the analyzed part of the building were inadequately sized by more than 30%. After re-designed of the foundations, the Pad Foundation enlarge to a Raft foundation to meet the code requirement and make the building safe.
- The results after re-design is to increase all the Internal Columns sections 250 x 600mm for (short, Ground, First and second floor columns) to column section 550 x 900 with steel reinforcement

(4 Ø 20mm + 8 Ø 12mm). by Strengthening the Columns with Concrete jacket to increase the section capacity to carry the load.

- And also to increase all the External Columns sections 300 x 600mm for(short, Ground, First and second floor columns) to column section 450 x 900mm with steel reinforcement (4 Ø 20mm + 5 Ø 12mm) by Strengthening the Columns with Concrete jacket to increase the section capacity to carry the load.
- The Slabs in the building (case study) were structurally analyzed under ultimate loads (load combination 1) using the three dimensional model. Reinforcement module and the needed reinforcements for the slab section are summarized in table 12-5. The check results indicates that the slab design for this building is adequate.

5.3. Recommendations:

This research is restricted for Strengthening of Reinforced Concrete Structures and the techniques of strengthening.

5.3. Recommendations from the research:

- 1) When reinforced concrete slabs or beams are strengthened by means of reinforced concrete jackets, it is generally recommended that the thickness of the new concrete layer should be less than one third of the existing concrete.
- 2) It is also recommended that in any case of strengthening a minimum number of shear connectors should be provided.
- 3) In the case of open stirrups or diagonal bars for shear strengthening, the safe transfer of forces acting in this additional reinforcement has to be secured by bond and additional mechanical heavy duty connectors (dowels, anchors, etc.).
- 4) The corners of the existing column should be chipped off before concreting, to reduce stress concentrations in the new concrete due to shrinkage, and to improve its interaction with the old concrete
- 5) The purpose of a quality assurance system is to ensure that the quality of a material meets the minimum required in specification documents and/or that the quality of fabrication or construction is without faults.

- 6) Staggering of lapped splices is recommended unless the distance between the bars is greater than 12 db (db being the diameter of the bars).
- 7) We need to qualify engineers in the field of strengthening process.
- 8) Interest and to ensure the structural design phase, and carefully review the detailed drawings. And then the construction phase, so we do not need strengthening.

5.3. Recommendations for future studies:

- 1) We need more research and studies to find alternative and cheap materials used in the strengthening process.
- 2) Methods also (e.g. assisting steel frame).

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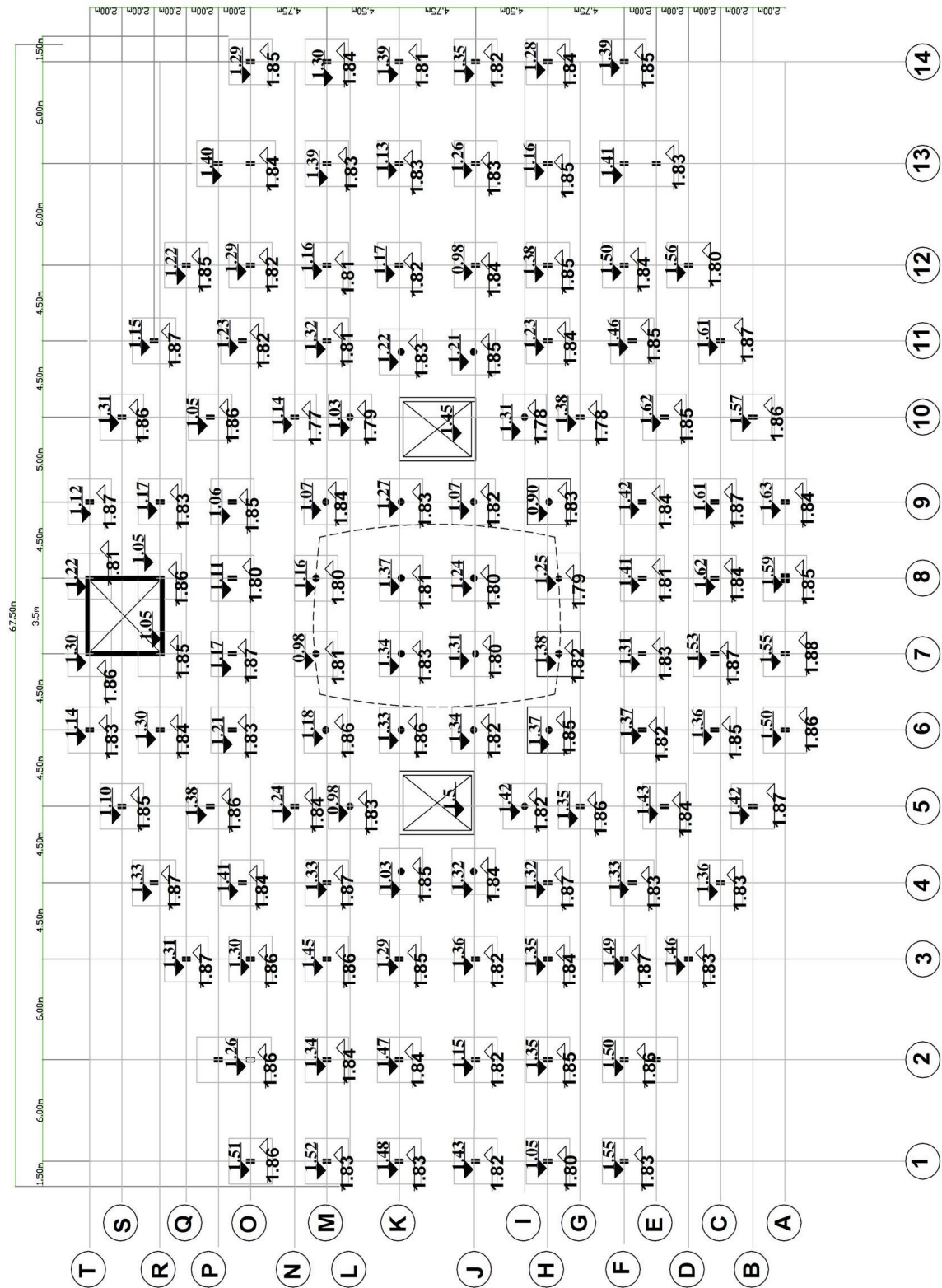
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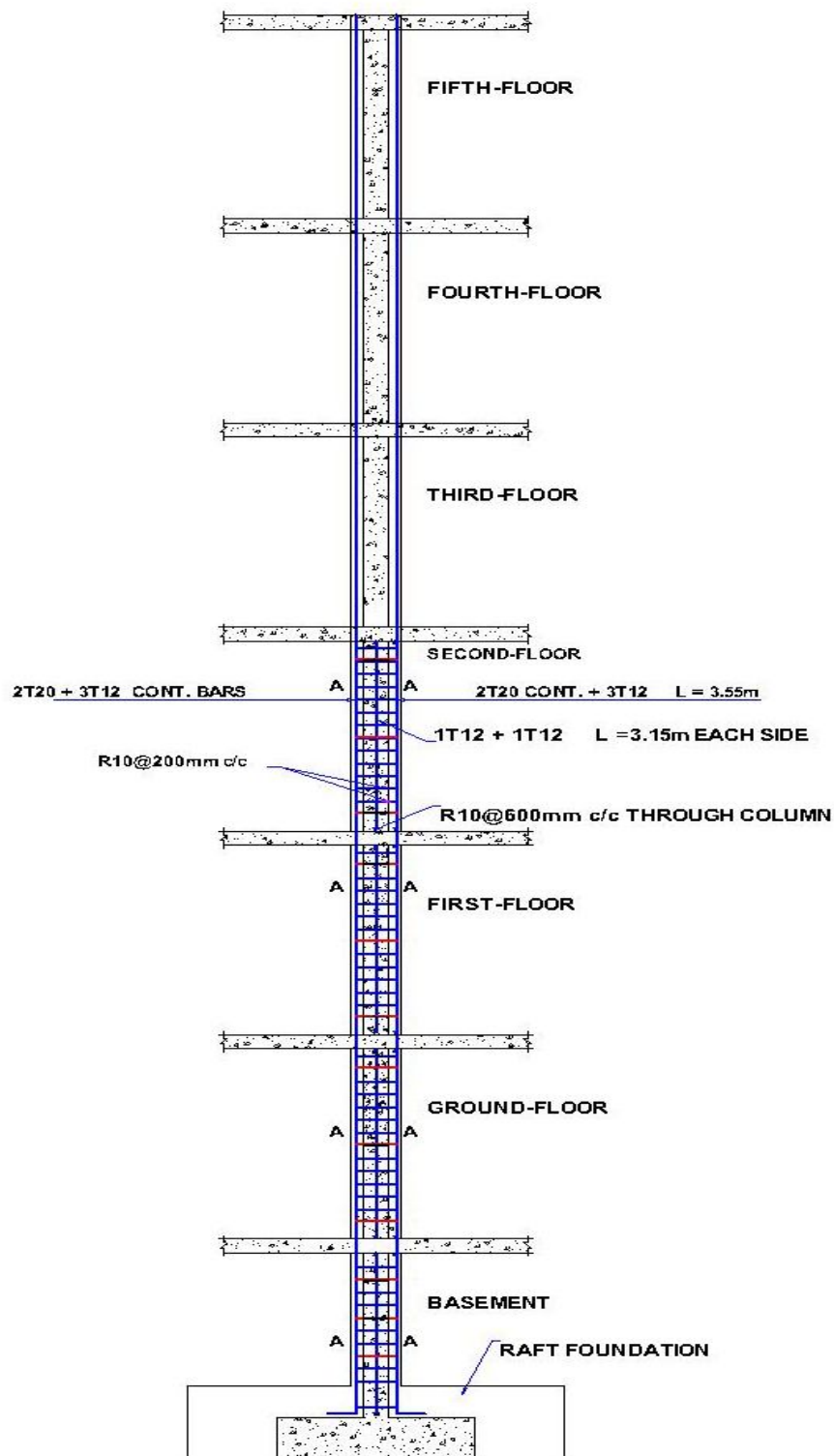
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APPENDICES

Appendix (B) : Plan of Police hotel building



Appendix (C) : Internal Columns Strengthening Details.

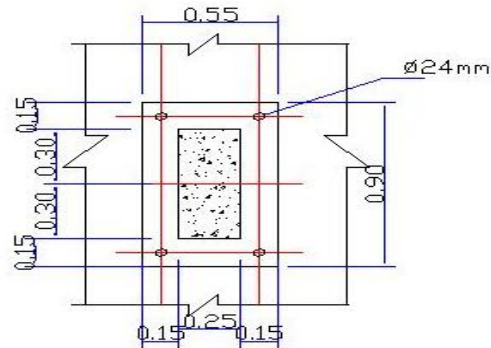


INTERNAL COLUMN STRENGTHENING DETAILS

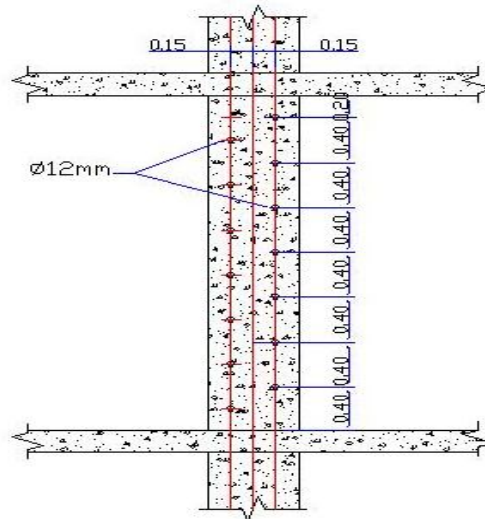
Appendix (D1) : Construction procedure for Strengthening of the Internal Columns.

CONSTRUCTION PROCEDURE:

- 1- DRILL 4 HOLES Ø24 mm THROUGH ALL SLABS, AS PER DRAWINGS.**



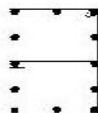
- 2- DRILL 14 HOLES Ø12 mm IN EACH COLUMN FOR TIES, AS PER DRAWINGS.**



- 3- PREPARE LINKS**



- 4- INSERT T20 , T12 BARS & R10 links
- USE CONCRETE SPACERS**



Appendix (D2) : Construction procedure for Strengthening of the Internal Columns.

5- PREPARE FORMS

*2 PIECES P.W.

0.60

*2 PIECES P.W.

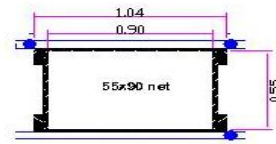
1.04

* 4 PIECES

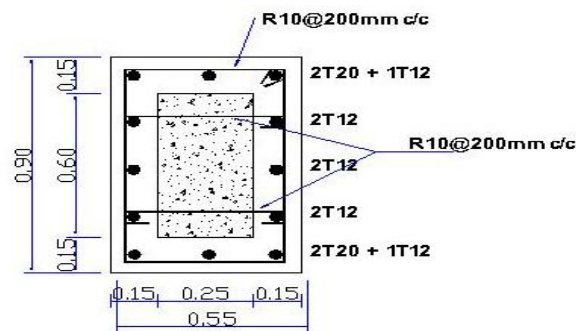
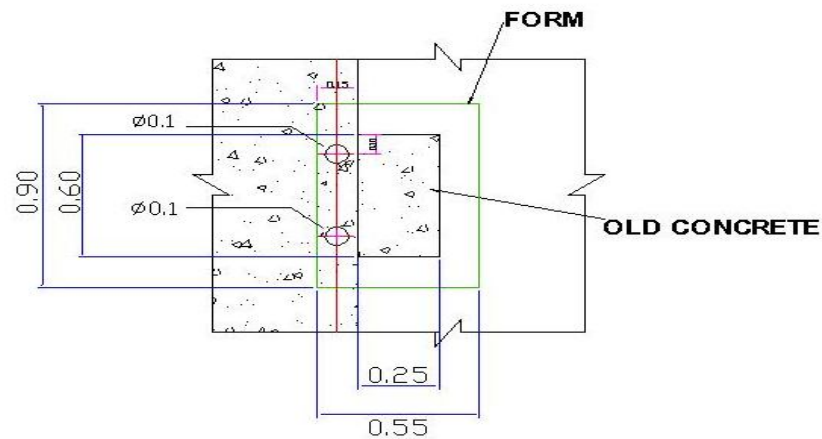
0.10 0.05

* 4 PIPES 2" PLUS JOINTS

* ASSEMBLE PROCEDURE

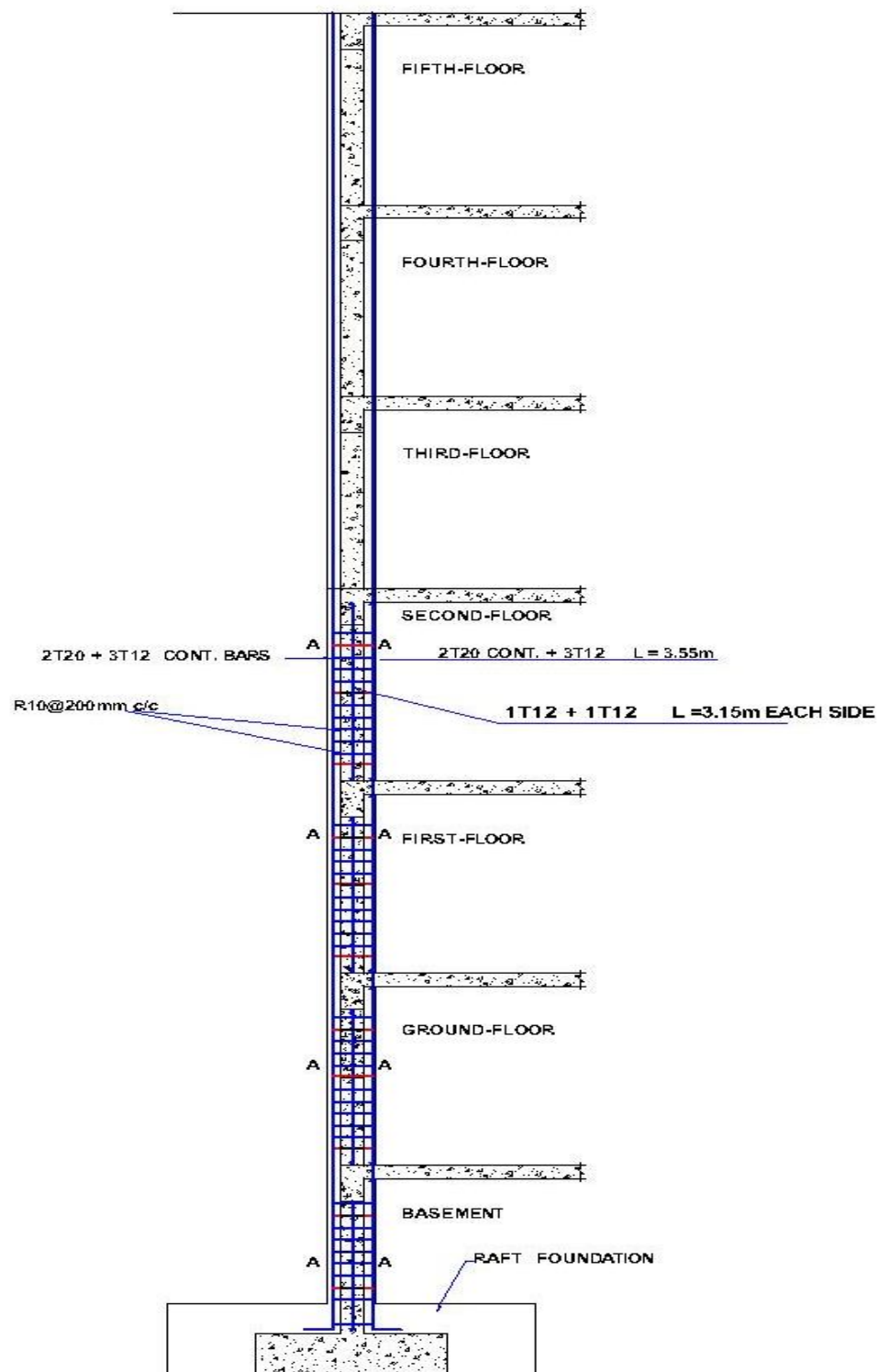


6- CONCRETING



A - A

Appendix (E) : External Columns Strengthening Details.

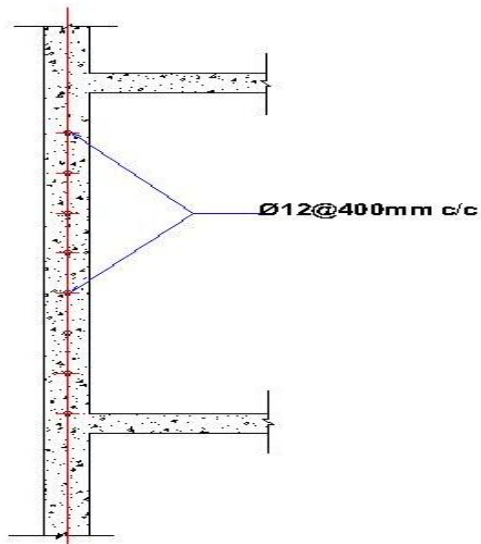
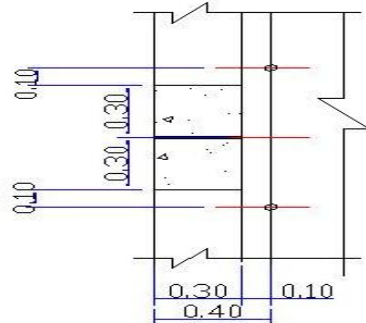


EXTERNAL COLUMN STRENGTHENING DETAILS

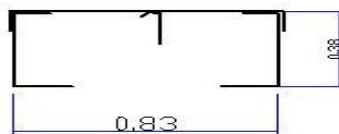
Appendix (F1) : Construction procedure for Strengthening of the Internal Columns.

CONSTRUCTION PROCEDURE:

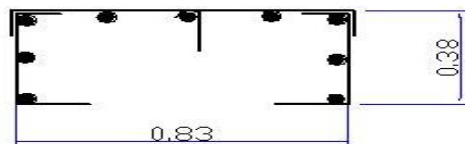
- 1- DRILL 2 HOLES Ø24 mm THROUGH ALL SLABS, AS PER DRAWINGS.



- 3- PREPARE LINKS

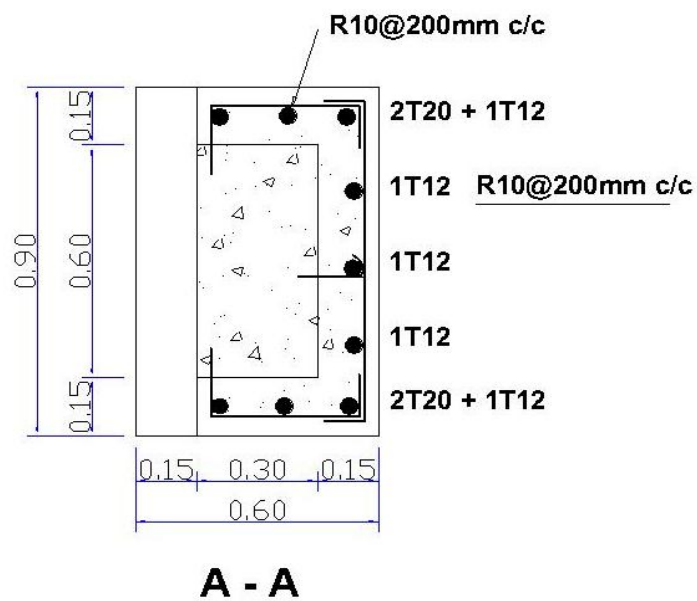
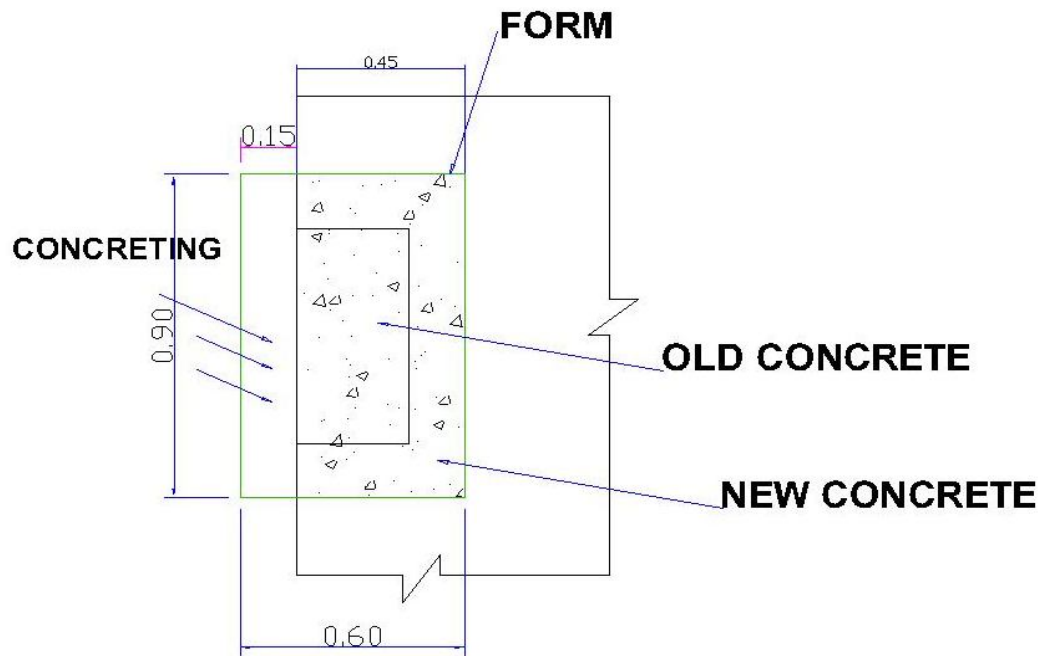


- 4- INSERT T20 , T12 BARS & R10 links
- USE CONCRETE SPACERS



Appendix (F2) : Construction procedure for Strengthening of the Internal Columns.

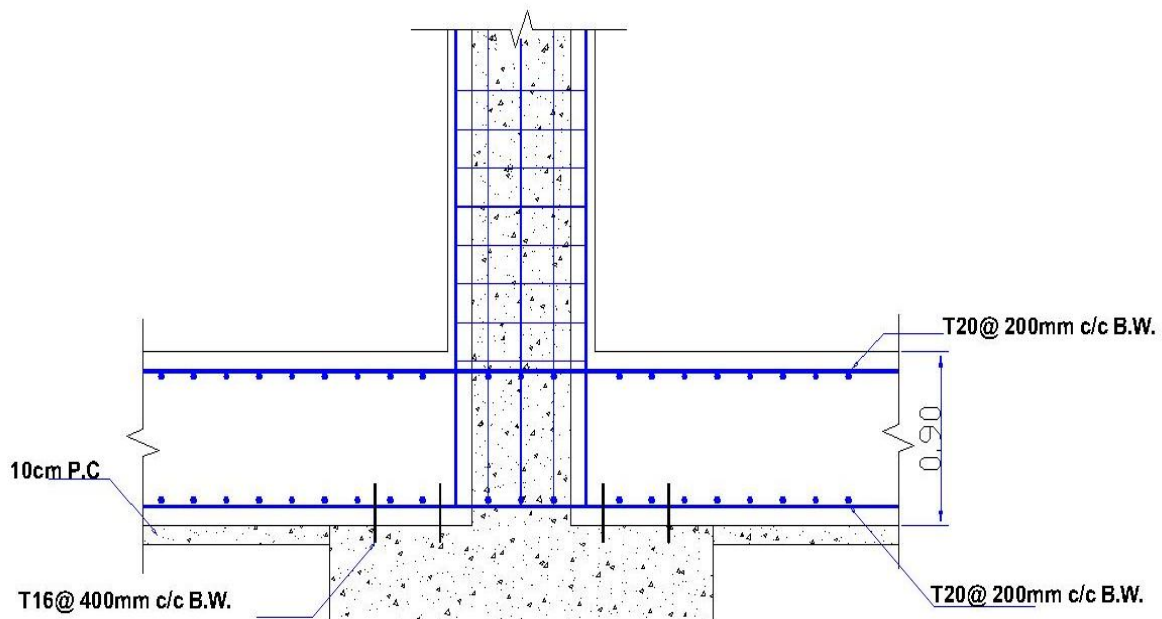
6- concreting



Appendix (G1) : Raft foundation details.

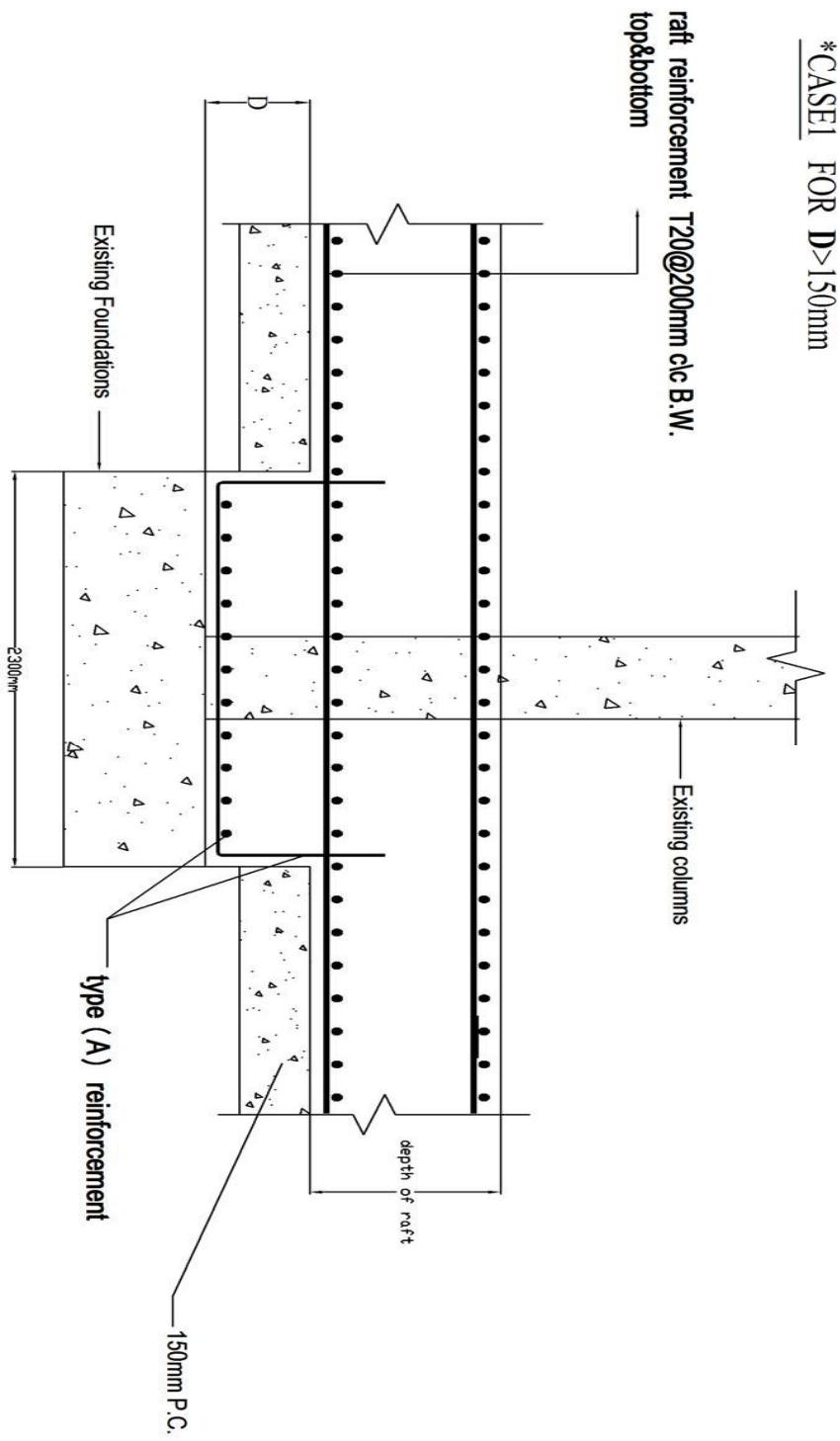
Notes

- * CONCRETE MIX: $1:1\frac{1}{2}:3$
- * MAX. AGGREGATE SIZE : 15mm
- * W/C = 0.5
- * SLUMP >120mm
- * $F_{cu.} > 25 \text{ N/mm}^2$
- * $F_y = 460 \text{ N/mm}^2$
- * THE LINKS DIMENSIONS ARE THE INTERNAL ONES.

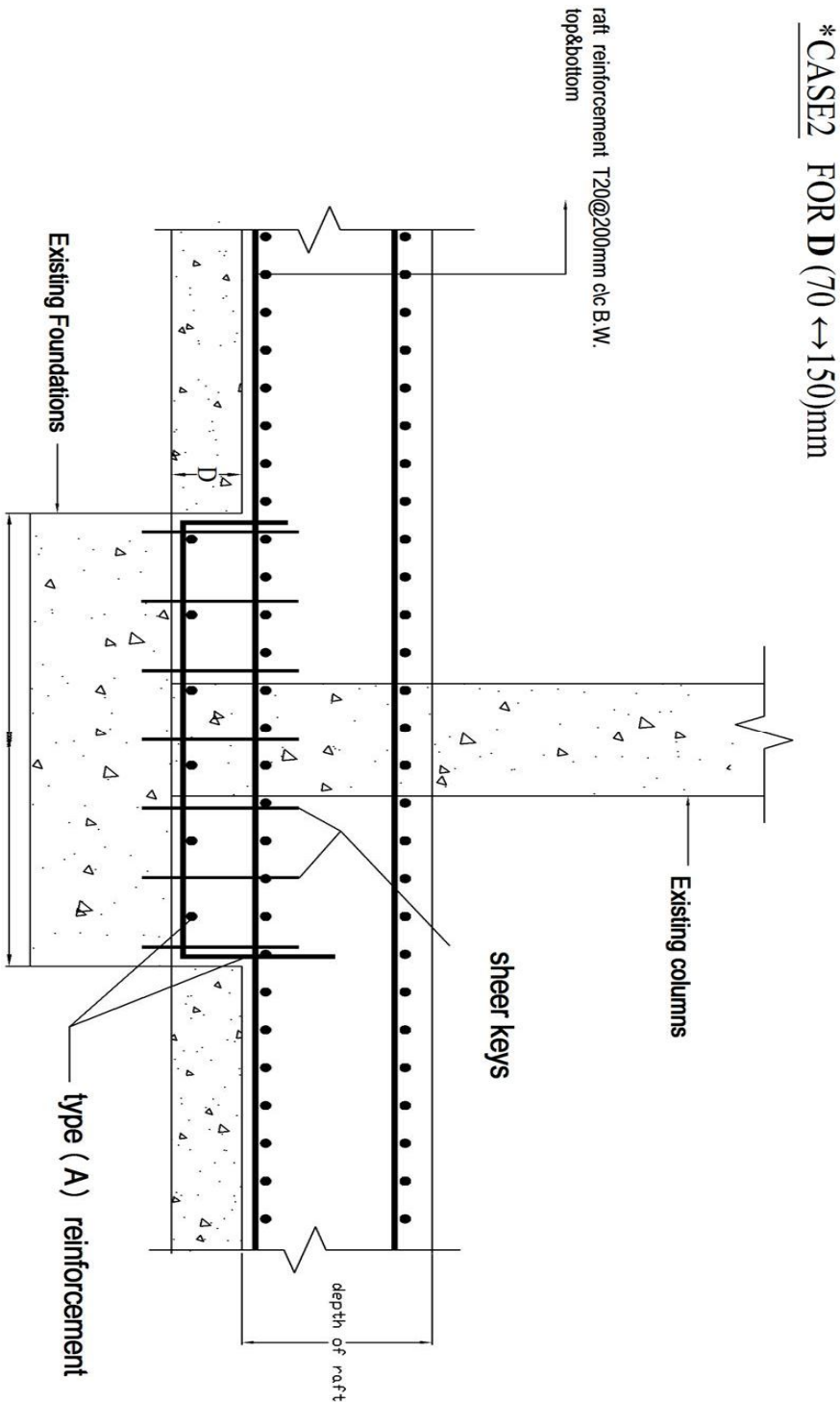


RAFT FOUNDATION DETAILS

Appendix (G2) : Raft foundation details.

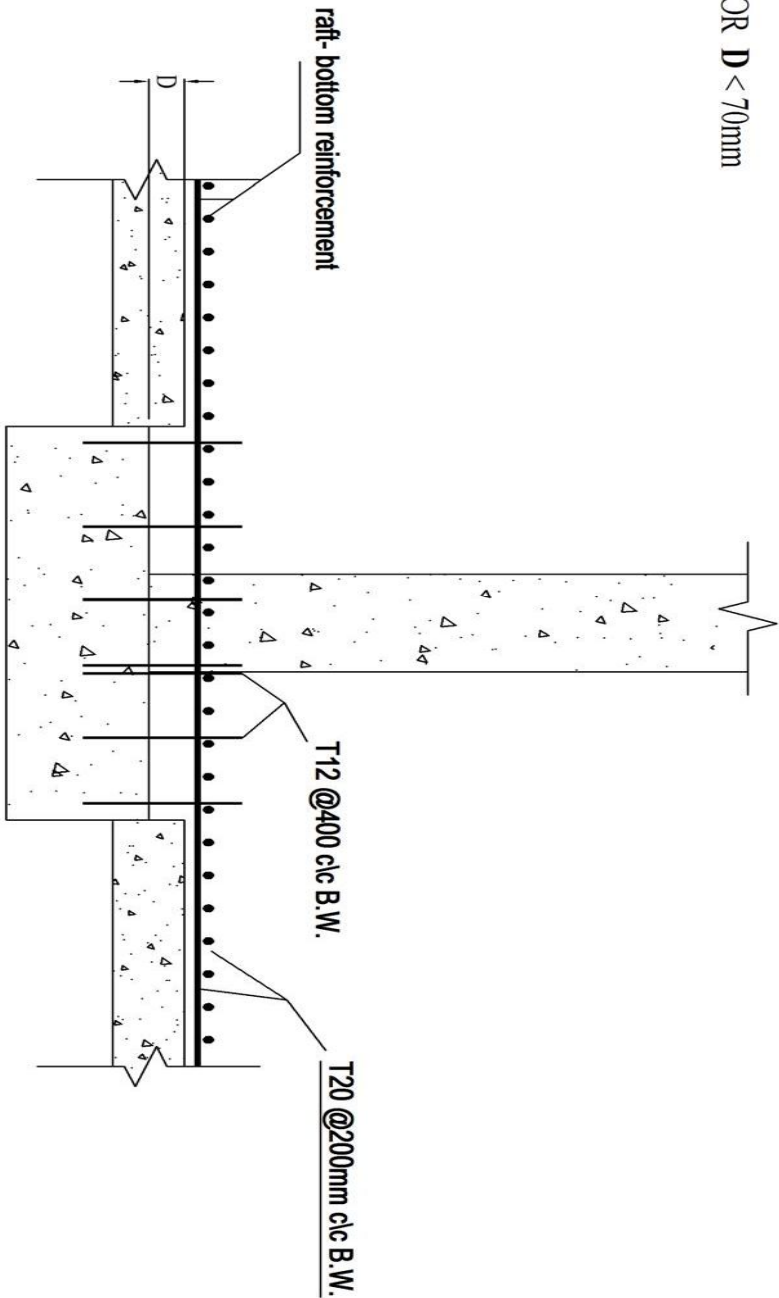


Appendix (G3) : Raft foundation details.



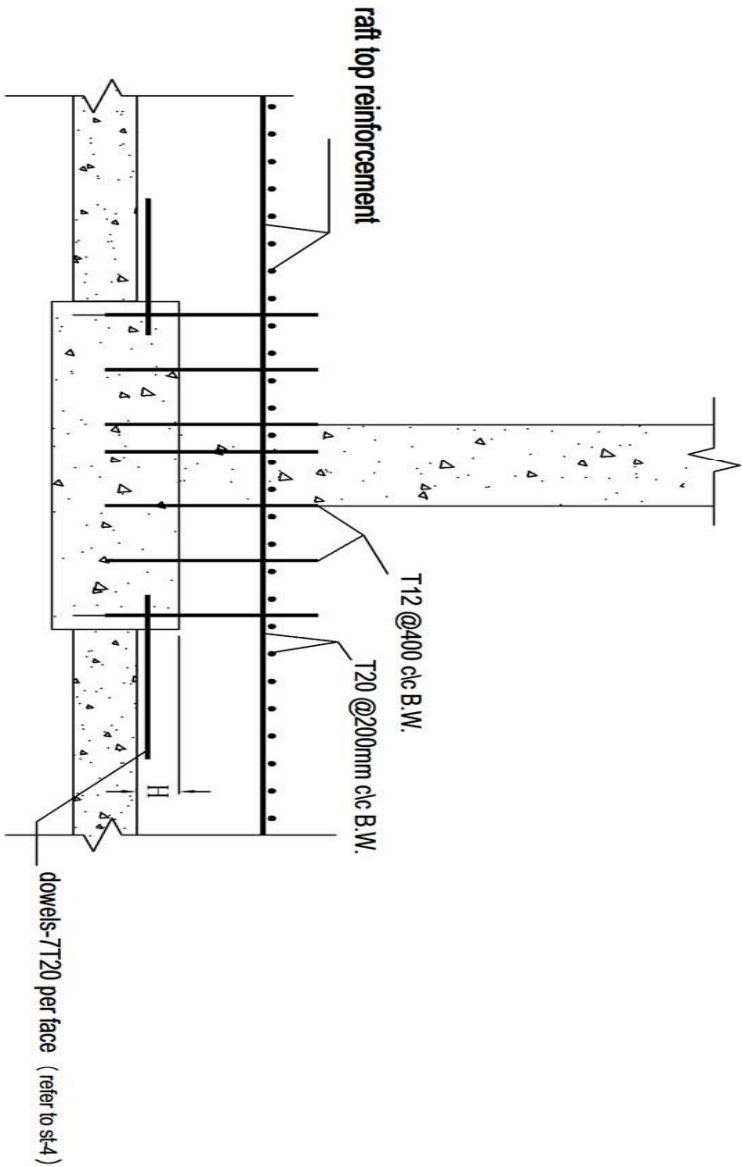
Appendix (G4) : Raft foundation details.

*CASE3 FOR **D** < 70mm



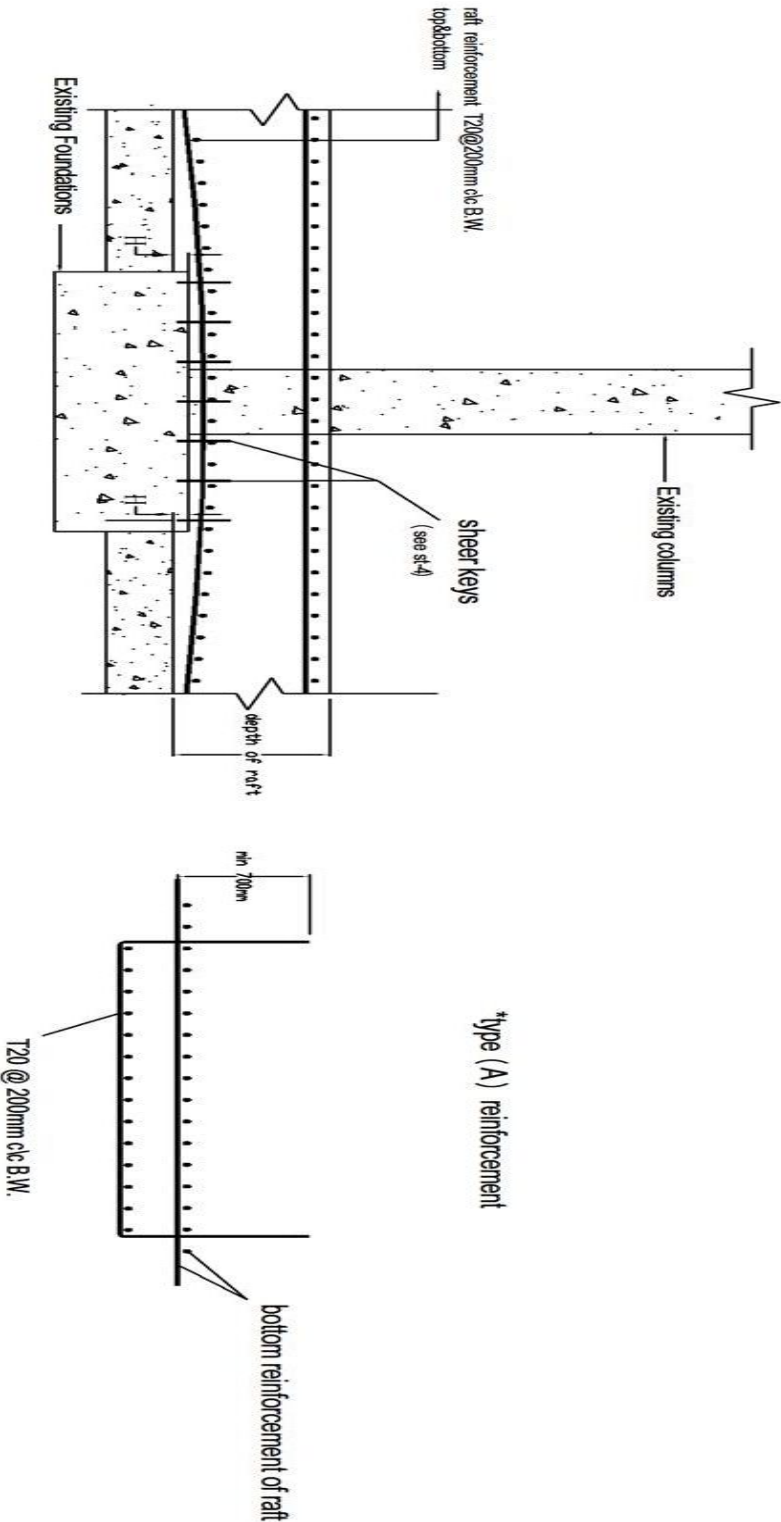
Appendix (G5) : Raft foundation details.

*CASE4 FOR $H > 100\text{mm}$

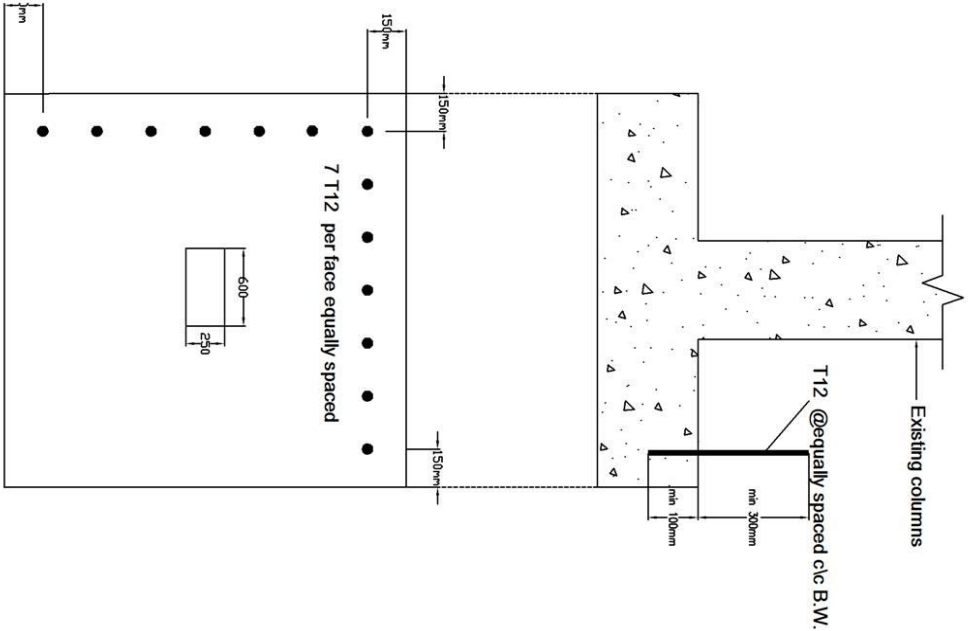


Appendix (G6) : Raft foundation details.

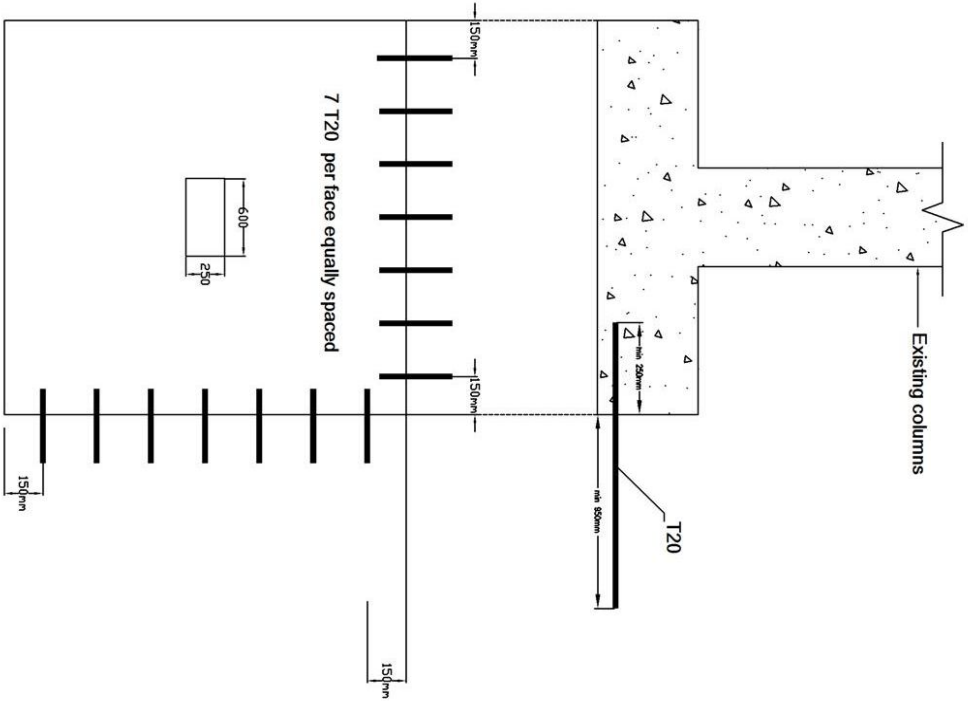
*CASE5 FOR $H < 100\text{mm}$



Appendix (H) : Shear keys in old foundations.

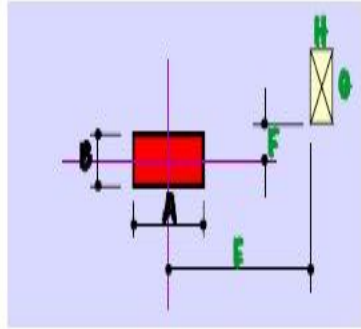
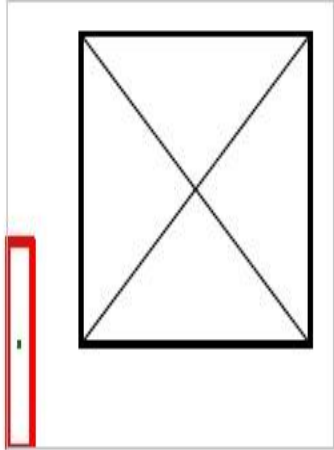


SHEER KEYS:



DOWELS:

Appendix (I): Punching Shear Check for Critical Areas A and B:

MATERIALS		f_{cu} N/mm ² <u>25</u> f_{yv} N/mm ² <u>420</u> link Ø mm <u>12</u>	STATUS VALID DESIGN		Legend
DIMENSIONS		A mm <u>250</u> B mm <u>600</u> G mm <u>900</u>	E mm <u>600</u> F mm <u>0</u> H mm <u>2250</u>		
LOADING		V_t kN <u>412</u> ult UDL kN/m ² <u>31.72</u>	$V_{eff} =$ kN <u>376.9</u>		
SLAB		h mm <u>200</u>	dx mm <u>165</u> dy mm <u>145</u> ave d mm <u>155</u>	A_{sx} mm ² /m <u>1340</u> A_{sy} mm ² /m <u>1340</u> ave A_s % <u>0.868</u>	
RESULTS		$V_{eff} = 376.9$ kN At col. face, $v_{max} = 1.414$ N/mm ²	$v_c = 0.7641$ N/mm ² (Table 3.8) At 1.5d perimeter, $v = 0.7380$ N/mm ² At 0d perimeter, $v = 0.5599$ N/mm ²		
PROVIDE LINKS 0		Links not required			
					
		Plan			

Appendix (j): Punching shear check for critical area C:

MATERIALS		fcu	N/mm ²	<u>25</u>	STATUS		VALID DESIGN		Legend	
		fyv	N/mm ²	<u>420</u>						
		link Ø	mm	<u>8</u>						
DIMENSIONS		A	mm	<u>400</u>	E	mm	<u>1500</u>			
		B	mm	<u>250</u>	F	mm	<u>0</u>			
		D	mm	<u>0</u>	G	mm	<u>2500</u>			
					H	mm	<u>1500</u>			
LOADING		Vt	kN	<u>308</u>						
		ult UDL	kN/m ²	<u>24.80</u>						
SLAB		h	mm	<u>200</u>	dx	mm	<u>165</u>	Asx	mm ² /m	<u>1340</u>
					dy	mm	<u>145</u>	Asy	mm ² /m	<u>1340</u>
					ave d	mm	<u>155</u>	ave As	%	<u>0.868</u>
RESULTS		Veff =	385.0	kN	vc =		0.7641	N/mm ²	(Table 3.8)	
		At col. face, v max =	2.738	N/mm ²	At 1.5d perimeter, v =		1.3117	N/mm ²		
					At 3.75d perimeter, v =		0.6896	N/mm ²		
PROVIDE LINKS (single leg)										
Perimeter 1		8 R8 @ 200	77	from col face						
Perimeter 2		10 R8 @ 205	194	from col face						
Perimeter 3		12 R8 @ 210	310	from col face						
Perimeter 4		14 R8 @ 215	426	from col face						
					Plan					

Appendix (K): Punching shear check for critical area D

MATERIALS		fcu	N/mm ²	<u>25</u>	STATUS		VALID DESIGN		Legend		
		fyv	N/mm ²	<u>420</u>							
		link Ø	mm	<u>8</u>							
DIMENSIONS		A	mm	<u>250</u>	E	mm	<u>0</u>				
		B	mm	<u>600</u>	F	mm	<u>0</u>				
		C	mm	<u>0</u>	G	mm	<u>0</u>				
		D	mm	<u>0</u>	H	mm	<u>0</u>				
LOADING		Vt	kN	<u>324.0</u>							
		ult UDL	kN/m ²	<u>16.10</u>							
SLAB		h	mm	<u>250</u>	dx	mm	<u>215</u>	Asx	mm ² /m	<u>1340</u>	
					dy	mm	<u>195</u>	Asy	mm ² /m	<u>1340</u>	
					ave d	mm	<u>205</u>	ave As	%	<u>0.655</u>	
RESULTS		Veff =	405.0	kN	vc =	0.6488	N/mm ²	(Table 3.8)			
		At col. face, v max =	1.153	N/mm ²	At 1.5d perimeter, v =	0.5433	N/mm ²				
					At 0d perimeter, v =	0.4256	N/mm ²				
PROVIDE LINKS		0			Links not required						
						Plan					

Appendix (L): *Shows the existing police hotel building before strengthening.*



Appendix (M): Described the Punching of columns in preparation for adding shear connectors for the concrete jacket



Appendix (N): Filling the holes with an appropriate epoxy material then inserting the shear connectors into the holes.



Appendix (O): Describe how to strengthen the column by adding the design reinforcement steel jacket and stirrups, before casting the concrete jacket.



Appendix (P): Shown the column jacket shuttering to increase the section capacity (Column strengthening).



Appendix (Q): Drilling hole on slab to fill the grout cement on the top of the column.



Appendix (R): Showing the strengthening of the external columns.



Appendix (S): Removal of the surface layer to implement the new foundation (Raft Foundation).



Appendix (T): Excavate the new foundation (Raft foundation).



Appendix (U): Demolish any concrete walls before laying the Raft Foundation.



Appendix (V): The site is ready for new foundation.



Appendix (W): Describe the foundations convert from isolated to Raft footing, and appears that the reinforcing bars fixed in the old foundation as a shear connectors with Epoxy material.

k



Appendix (X): Describe how to link the steel reinforcement between the Raft foundation and columns.



Appendix (Y): Filling the holes with epoxy material then inserting the shear connectors into the holes to connect (fixed) the Raft foundation with columns.

