ADDIS ABABA UNIVERSITY
SCHOOL OF GRADUATE STUDIES

FACULTY OF TECHNOLOGY
DEPARTMENT OF CIVIL ENGINEERING

Assessment of flat plate Construction without shear walls with respect to the effectiveness against lateral load resisting Capacity.

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A Thesis Submitted to School of Graduate Studies of Addis Ababa University in Partial Fulfillment of the Requirements for the Degree of Masters of Science in Civil Engineering (Structures Major)

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BY EYOB TADESSE

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ABSTRACT

The overwhelming social and economic impacts of recent earthquakes in the world have resulted in an increased awareness of the potential seismic hazard and the corresponding vulnerability of existing structures. Greater effort has been given to reasonable estimates, predictions and mitigation of the risks associated with these potential losses. As being one of the special reinforced concrete structural forms, flat-slab systems need further attention. We are living in an era when flat slab construction activity is booming due to many advantages it posses in terms of architectural flexibility, use of space, easier formwork and shorter construction time, However the structural efficiency of the flat-slab construction is hindered by its poor performance under earthquake loading. This undesirable behavior has originated from the insufficient lateral resistance due to the absence of deep beams or shear walls in the flat-slab system. Hence, the effectiveness of flat slab structures without shear walls or other stiffening mechanisms needs to be investigated against seismic hazard.

Generally, flat-slabs can be designed by any procedure that satisfies equilibrium and geometric compatibility provided that every section has strength at least equal to the required strength (ACI-318). In this study various flat slab analysis methods that are pertinent to gravity and earthquake loading are reviewed along with different modeling techniques. Among many of flat slab analysis methods, Direct Design Method, Equivalent Frame Method and The finite analysis method are the most commonly used methods of analysis. And finally, FEM method is used to analyze flat slab buildings along with the effective beam width modeling method for gravity and seismic loading respectively. The regularity of the buildings in terms of mass and stiffness both in plan and elevation enabled the utilization of

two-dimensional analysis in the assessment of seismic response. Hence, in this study, the three dimensional flat-slab building is modeled as a series of two dimensional interconnected planar frames. For the modeling of the 2-D planar frames, the portion of the slab that will contribute to the frame analysis is determined as well as the width of the concealed beam within this slab portion.

In this study different flat slab structures with different aspect ratios are modeled, analyzed and designed to satisfy *strength requirements* and these structures designed for strength are evaluated against *drift capacity*. Building layout in plan is kept constant while slab depth and building height are used as a variable by increasing the number of stories up to the attainment of the largest practical slab depth.

Finally, Low to medium rise flat plate structures (*G+1* - *G+6*) having medium spans , located in seismic zone 2 which are designed and analyzed in this study found to be effective in seismic hazards resistance. How ever, the outcome of this study articulate that flat plate structures with more than six numbers of stories constructed in seismic zone II need to be combined with shear walls or other stiffening mechanisms.

NOTATIONS

`

A_c The area of slab at critical section

b_o The perimeter along the critical section.

C_1, C_2 Column dimensions in the bending and transverse direction.

d Effective Depth of a slab.

DL Service dead load.

DR Design drift ratio.

E_s Modulus of elasticity of steel.

E_c Modulus of elasticity of concrete.

f_c' Compressive cylinder strength of concrete

f_{yk}- Characteristic tensile strength of a steel.

h Total depth of a flat slab.

LL Service live load.

l_e Elastic effective beam widths

l_1, l_2 Span lengths in bending direction and transverse direction

M_u Design moment

V_g The direct shear force due to gravity load only.

VR Gravity shear ratio.

$\boldsymbol{V_u}$ Total factored shear force.

χ - Reduction factor accounting for gravity loads.

Φ Diameter of a reinforcing bars.

ϕ Strength reduction factor.

α_i ,α_e- Effective width coefficient for interior and exterior connections.

β The stiffness reduction factor to represent cracking resulting from service gravity loads and the combined effects of creep and shrinkage as well.

ξ Time dependant factor used to calculate additional long-term deflection resulting from creep and shrinkage. $0 \leq \xi \leq 1$.

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1. INTRODUCTION

1.1. BACKGROUND OF THE PROBLEM

The devastating social and economic impacts of recent earthquakes in the world have resulted in an increased awareness of the potential seismic hazard and the corresponding vulnerability of the built environment. Greater effort has been given to reasonable estimates, predictions and mitigation of the risks associated with these potential losses.

In order to be successful in mitigation efforts; the expected damage and the associated loss in urban areas caused by severe earthquakes should be properly estimated. It is also appropriate to consider the expected damage as a measure of seismic vulnerability. The determination of such a vulnerability measure requires the assessment of the seismic performances of all types of building structures typically constructed in an urban region when subjected to a variety of potential earthquakes.

As being one of the special reinforced concrete structural forms, flat-slab systems need further attention. They possess many advantages in terms of architectural flexibility, use of space, easier formwork and shorter construction time. However the structural efficiency of the flat-slab construction is hindered by its poor performance under earthquake loading. This undesirable behavior has originated from the insufficient lateral resistance due to the absence of deep beams or shear walls in the flat-slab system.

A frame consisting of columns and either flat plates or slabs with drop panels, but lacking shear walls or other bracing elements ,is insufficient in resisting lateral loads and is subjected to significant lateral drift deflection. These deflections are amplified by P - Δ effects resulting from gravity Loads. This gives rise to excessive deformations that cause damage in non-structural members even when subjected to earthquakes of moderate intensity. [10]

Practically unbraced Slab-column frames can some times be used only for low raised buildings or for the top few floors in tall building, where inter-storey drift may be reduced. However Medium to high-rise Flat-slab type of construction is in widespread use in some parts of the country including Addis Ababa which is in fact located in the low-seismicity zone; on the other hand Adama and Awassa which are known as high earthquake vulnerable regions. In these cases, the design of flat-slab buildings is typically carried out in a similar manner to ordinary frames. Hence it becomes more important to review the different methods used for modeling and analysis and investigate the vulnerability of this of this special structure with regard to resistance against earthquake forces.

1.2. OBJECTIVES OF THE STUDY

The objective of the research is

A - To study the different analysis methods and find out the effectiveness and extent of applicability of each analysis methods for laterally loaded flat slab Buildings.

B - To assess the effectiveness of flat slab construction without any stiffening structures such as structural wall or bracing systems with respect to lateral load resisting capacity and observe the number of stories or height limit up to which flat slab buildings can effectively and practically be constructed without any stiffening mechanisms to resist seismic action.

2. REVIEWS OF FLAT PLATE SYSTEMS

2.1. FLAT SLAB STRUCTURES

2.1.1. INTRODUCTION

A flat-plate structure, as shown in Figure 2.1, consists of a slab with uniform thickness supported on the columns with no beams or drop panels. The economy of flat plate buildings has lead to their wide spread utilization throughout the world. Conventionally flat-plate structure is generally used for lightly loaded structures such as apartments, hotels, and office buildings with relatively short spans, typically less than 6m. For longer spans or heavier loads, flat-slabs system with shear capitals or drop panels would be more feasible. [19]

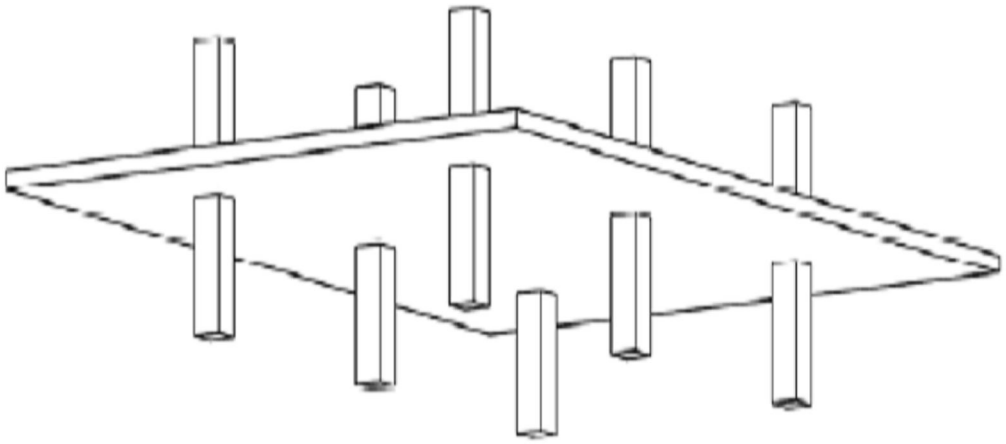


Figure *2.1*: Flat-plate structure. [19]

Flat-plates have been widely used due to the reduced construction cost associated with the simple formwork and simple arrangement of flexural reinforcement. An additional advantage of a flat-plate is reduced building story heights that result in more usable space in a building for a given or limited height and reduces lateral loads acting on the system, as well as mechanical, electrical, and cladding costs.

When a slab-column connection of flat plate building carries both gravity load and unbalanced moment induced by wind or seismic load, significant shear and bending moment are concentrated at one side of the column where punching failure may occur. The punching failure can occur with little or no warning and can lead to the collapse of a large area of the floor system and possibly a progressive collapse of the entire structure (see Figure 2.2). In the 1985 Mexico City earthquake, 44 flat plate buildings were severely damaged and 91 collapsed due to punching failure. [21]

Figure 2.2—Punched Slab to Column Connections in 1985 Mexico City Earthquake [7]

Second, flat-plate structures are inherently flexible. Under service level gravity loads, the slab may exhibit extensive cracking around the column that reduces the slab-column connection stiffness and may result in unacceptable slab deflection and joint rotation in the cases of seismic action.

2.1.2. OVERVIEW OF DESIGN AND ANALYSIS CONSIDERATIONS IN FLAT-SLAB CONSTRUCTION

One of the most common floor systems for the construction practice in many earthquake vulnerable parts of the world (Mediterranean, Middle East and some Parts of Ethiopia) is the flat-slab .This practice is generally used for relatively light loads and for spans from 4.0m to 6m. For heavy industrial loads and for larger spans, flat-slabs are recommended to be used with drop panels or column capitals. In such structural systems, the load is transferred to the column by thickening the slab near the column, using drop panels and/or by flaring the top of the column to form a column capital. The drop panel commonly extends about one sixth of the span each way from each column, giving extra strength in the column region while minimizing the amount of concrete at mid-span. [17] The flat-slab type of construction provides architectural flexibility, more clear space, less building height, easier formwork, and, consequently, shorter construction time. Furthermore, the absence of sharp corners improves fire resistance, so there is less danger of the concrete spalling and exposing the reinforcement. However, a serious problem that can arise in flat plates is brittle punching failure due to the transfer of shearing forces and unbalanced moments between slabs and columns.

[20] Under earthquake actions, the unbalanced moments can produce high shear stresses in the slab. In addition to this, flat-slabs are susceptible to significant reductions in stiffness as a consequence of slab cracking that can arise from construction loads, service gravity loads, temperature and shrinkage effects and lateral loads. [25] Hence in regions of high seismic risk, flat-slab construction should only be used as the vertical load carrying system in structures braced by frames or shear walls, which are responsible for the lateral capacity of the structure. [10] In such cases, slab-column connections must undergo the lateral deformations of the primary lateral load-resisting structural elements without punching failure in order to sustain the gravity loads acting at the instance of earthquake occurrence in addition to unbalanced moments resulting from

earthquake lateral forces. [20] suggested that the primary lateral load resisting structural elements, such as shear walls, should be combined with flat plates in seismic zones to keep the lateral drift ratio lower than 1.5%. Therefore the slab-column connections must be capable of withstanding 1.5 % drift ratio without punching failure. They also stated that for ductility and drift capacity, slab column connections without shear reinforcement must satisfy

$$V_u \leq 0.4V_o \qquad \textbf{(2.1)}$$

Where:-

V_u -is the factored axial force and

V_o -is the nominal shear capacity in the absence of unbalanced moment.

In the presence of shear reinforcement, the above criterion is no longer required. Thus, slab-column connections in seismic zones must possess

(A) Adequate strength against punching shear failure during and after earthquake occurrence and

(B) Adequate ductility to undergo allowable deformations without failure, that is, the ability to undergo a specified minimum lateral inter-story drift ratio. [19]

In spite of the above recommendations, flat-slabs are often adopted as the primary lateral load resisting system and their architectural flexibility made them popular in seismically active regions, such as most of the Mediterranean basin and some parts of our country. In these cases, the design of flat-slab buildings is typically carried out in a similar manner to ordinary frames. Where this practice is followed, the response under moderate earthquakes indicates extensive damage to non-structural elements even when code provisions for drift limitation are satisfied. [4] Prevention of such damage is important, as it accounts for the greatest portion of total repair costs. [28]

2.2. METHODS EMPLOYED FOR DESIGN OF FLAT-SLAB BUILDINGS

According to ACI-318, flat-slabs can be designed by any procedure that satisfies equilibrium and geometric compatibility provided that every section has strength at least equal to the required strength, and that the serviceability conditions are satisfied. Generally, some of the methods employed for the design of flat-slabs are,

A- Direct Design Method

B- Equivalent Frame Method,

C -The Yield-line method,

D-The Finite Element Method

A-Direct design method

The calculation of moment in the direct design method is based on the statical moment, M_o .In this method, the slab is considered panel by panel and equation 2.2 is used to compute the total moment in each panel. Then the statical moment is then divided up to between positive and negative moments, and these are divided between middle and column strips. [10]

For design, the slab is considered to be a series of frames in the two way direction, as shown in fig 2.3. In each span of each of the frames, it is necessary to compute the total statical Moment $M_{o.}$ Thus, we have

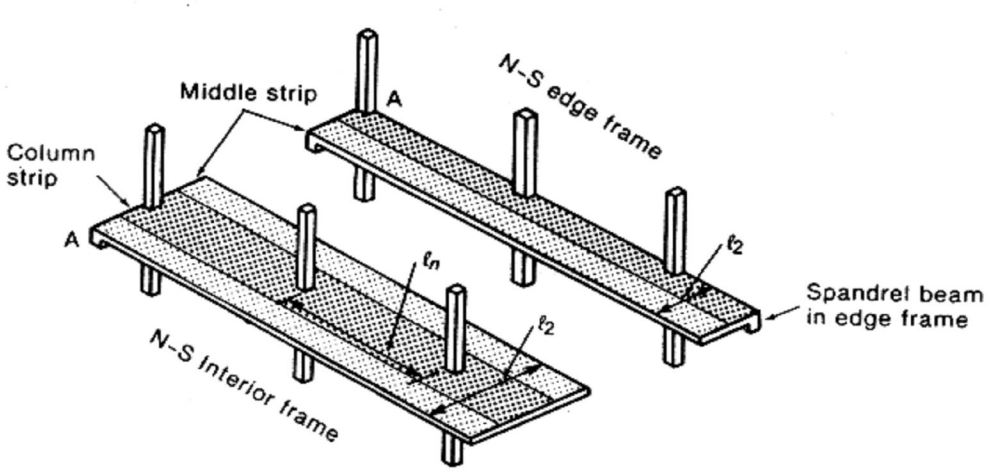
$$M_o = \frac{w_u l_2 l^2{}_n}{8} \qquad (2.2)$$

Where;

w_u - is factored load per unit area.

l_2 - is the transverse width of the strip.

l_n - Is the clear span between columns.



2.3 Division of slab into frame for design [10]

Positive and negative moments in Panels

The total factored moment, statical moment, M_o is divided into positive and negative factored moments according to rules given in ACI section 13.2.1.These are illustrated in fig 2.4.In interior spans, 65 percent of M_o is assigned to the negative moment region and 35 percent to the positive –moment regions.

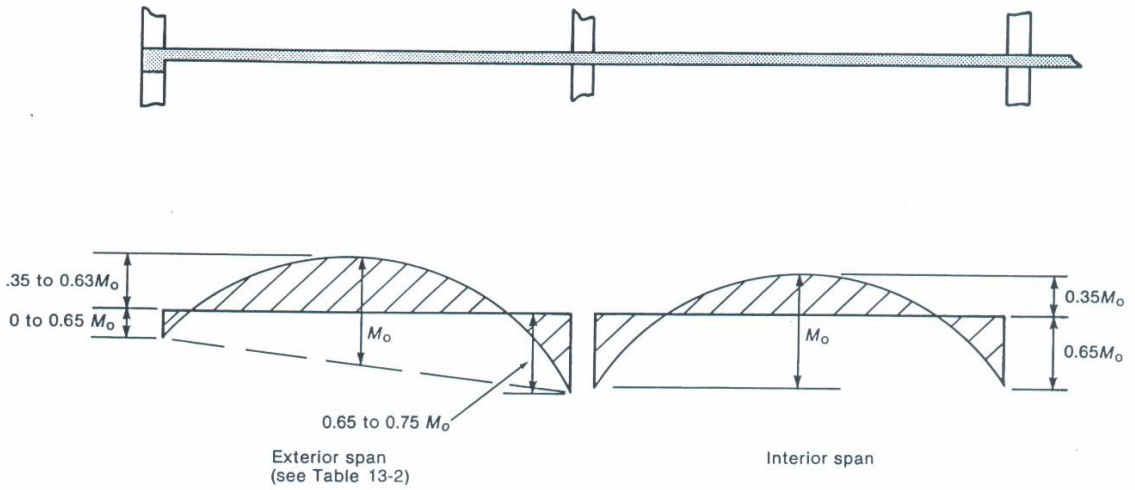


Fig 2.4 - Assignment of M_o to positive and negative moment region. [10]

The exterior end of an exterior span has considerably less fixity than the end at the interior support. The division of M_o in an end spans into positive and negative moment regions given in table 2.1

Division of moments between middle and column strips.

ACI Section 13.6.4 defines the fraction of the negative and positive moments assigned to the column strips. The remaining amount of negative and positive is assigned to the adjacent half-middle strips. In this case, 75 percent of the negative moment is in the column strip and the remaining 25 percent is divided equally between the two adjacent half-middle strips. Similarly, 60 percent of the positive moment is assigned to the column strip and the remaining 40 percent is divided, with 20 percent going to each adjacent half middle strip. [2]

Table 2.1- Distribution of Total Factored Static moment, M_o in an exterior Span[10]

	(1)	(2)	(3)	(4)	(5)
	Exterior edge unrestrained	Slab with beams between all supports	Slab without beams between interior supports		Exterior edge fully restrained
			Without edge beam	With edge beam	
Interior negative factored moment	0.75	0.7	0.7	0.7	0.65
positive factored moment	0.63	0.57	0.52	0.5	0.35
Exterior negative factored moment	0	0.16	0.26	0.3	0.65

Limitations of the use of the Direct Design method

1-There must be a minimum of three continuous spans in each direction. If they are fewer than three panels, the interior negative moments from the direct design method tends to be too small.

2- Rectangular panels must have a long-span/short-span ratio not greater than 2.

3- Successive span length in each direction shall not differ by more than one-third of the longer span.

4- Columns may be offset from the basis rectangular grid of the building by up to 0.1 times the span parallel to the offset.

5- All loads must be due to gravity only.

6- The service (un factored) live load shall not exceed two times the service dead load. [10]

B-Equivalent Frame Method

In the equivalent frame method the slab is divided in to a series of two dimensional frames, and the positive and negative moments are computed via an elastic-frame analysis. Once the positive and negative moments are known, they are divided up between middle and column strips in exactly the same way as in the direct design method.

As it is explained above, equivalent frame is a simple substitute of a two dimensional model for a three dimensional frame consisting of slabs and columns. Thus, the building frame is cut vertically from top to bottom, first in the longitudinal direction and transverse direction along the centerlines on the adjacent panels. In general, the frames used for the analysis of EFM method will consist of three members.

1- A series of slabs in the longitudinal direction.

2- Columns extending above and below the slab.

3-The transverse moment transfer elements (torsional members) consisting slab at the columns in the transverse direction.

For the exterior row of columns, the slab beam will consists of only one-half the interior panel. It should be noted that in EFM analysis the spans are considered from center-to-center of slabs, but the design moments are reduced to those at the face of the supports. The resulting two dimensional frames from the EFM are the equivalent frame and they should be analyzed by methods of elastic analysis such as moment distribution, slope deflection or matrix displacement methods. The frame over the over all building height or a frame representing single floor can be analyzed separately with the columns assumed fixed at the floor above and below. [26]

C -The Yield-line method,

The yield-line method will enable the most economic and uniform distribution of reinforcement to be achieved. It makes the use of uniform loose bar, or one- or two-way spanning mats very attractive, particularly for the sagging moment reinforcement. Top steel should be concentrated over supports to control cracking. It should be noted that since this method considers only possible collapse mechanisms, a separate elastic analysis of cracking and deflection may be required.

D-The Finite Element Method

Use of finite-element (FE) analysis has particular advantages when the floor has irregular supports or geometry, large openings or carries concentrated heavy loads. Care is needed in modeling the geometry, material properties and loads on the structure.

2.2.1. MODELING OF FLAT-SLAB FRAMES FOR SEISMIC LOADING

The regularity of the buildings in terms of mass and stiffness in both plan and elevation enables the utilization of two-dimensional analysis in the assessment of seismic response. However, dividing the building into small models should not be done without ensuring that the overall behavior is not changed significantly.

For the modeling of the slabs, the portion of the slab that will contribute to the frame analysis should be determined as well as the width of the concealed beam within this slab portion. Two simplified methods exist for the former, namely the Effective Beam Width method and the Equivalent column Method, in which the effect of the slab is accommodated by appropriate modification of the beam width or the column stiffness, respectively.

2.2.1. Equivalent Column Method

When the beam – and – column frame shown in fig 2.5a is loaded, the ends of the column and beam undergo equal rotations where they meet at the joint. If the flexural stiffness, $K=M/\Theta$, is known for the two members, it is possible to calculate the joint rotation and the moments in the members. Similarly, in the case shown fig 2.5b, the ends of the slab and the wall both undergo equal end rotations when the slab is loaded. When the flat plate is connected to column, as shown in fig 2.5c, the end rotation of the column is equal to the end rotation of the strip of slab C-D, which is attached to the column. The rotation at A of strip A-B is greater than the rotation at point C, however, because there is less restraint to the rotation of the slab at this point. In effect, the edge of the slab had twisted as shown in fig 2.5d.As a result; the average rotation of the edge of the slab is greater than the rotation of the end of the column.

To account for this effect in slab analysis, the column is assumed to be attached to the slab beam by the transverse torsional members A-C and C-A'. One way of including these members in the analysis is by the use of the concept of ***equivalent column***, which is a single element consisting of the column above and below the floor and attached torsional member, as shown in fig 2.5d. The stiffness of the equivalent column K_{ec} represents the combined stiffness of the columns and attached torsional members:

$$K_{ec} = \frac{M}{Average\ \ rotation\ \ of\ \ the\ \ edge\ \ beam} \qquad (2.3)$$

The inverse of the stiffness, 1/K, is called the flexibility. The flexibility of the equivalent column.1/ K_{ec}, is equal to the average rotation of the joint between the "edge beam" and the rest of the slab when a unit moment is transferred from the slab to the equivalent column. This average rotation is the rotation of the end of the columns, Θ_c, plus the average twist of the beam, $\Theta_{t,\ avg}$, both computed for a unit moment.

$$\theta_{ec} = \theta_c + \theta_{t,avg} \qquad (2.4)$$

Where;

Θ_c - is rotation for a unit moment and is given by $1/\sum K_c$,

$\sum K_c$ - Refers to the sum the flexural stiffness of the columns above and below the slab. And given by,

$$K_c = \frac{4 E_c I_c}{L_c} \qquad (2.5)$$

I_c - is the moment of inertia of the columns.

$\Theta_{t,\,avg}$ –Is a rotation for a unit moment and attain the value $1/K_{t,}$

K_t - is the torsional stiffness if the attached torsional members. And given by:

$$K_t = \frac{9EC}{l_2\left(1-c_2/l_2\right)^3} \qquad (2.6)$$

Substitution in to 2.4 gives;

$$\frac{1}{K_{ec}} = \frac{1}{\sum K_c} + \frac{1}{K_t} \qquad (2.7)$$

l_2 - Refers to the transverse spans on each side of the column.

C- Is Modulus of rigidity which is similar to polar moment of inertia.

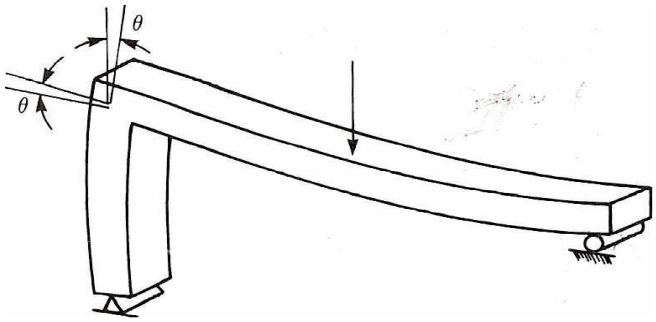
$$C = \sum\left[\left(1-0.63\frac{x}{y}\right)\frac{x^3 y}{3}\right] \qquad (2.8)$$

x- is the short side of the rectangle in the slab column connection joint.

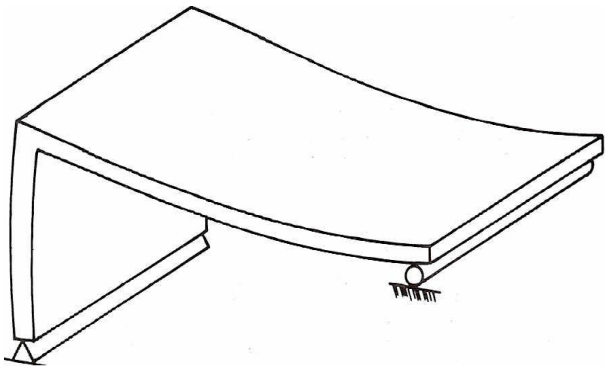
y- is the long side of the rectangle in the slab column connection joint.

E- Refers to modulus of elasticity of concrete.

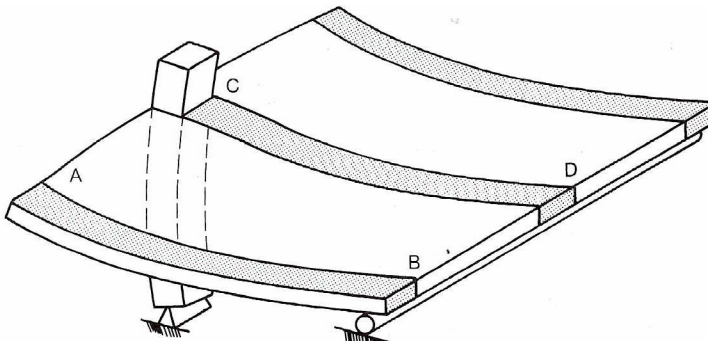
The equivalent column method recommended for design in ACI (1973) commentary should be distinguished from the alternate equivalent beam methods, which have since been developed; the present ACI code has omitted the equivalent column methods from the main body of the code. The equivalent column model is most commonly used for both gravity and lateral loads, even though it is most suited to gravity loads. [26]



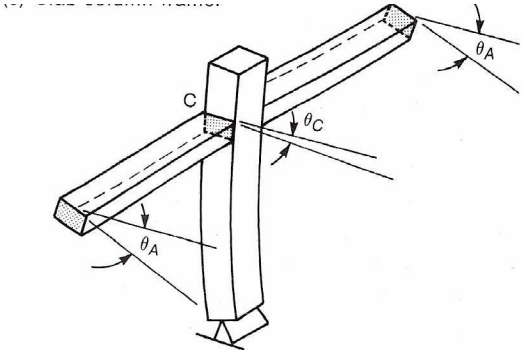
(a) Beam and column frame



(b) Slab and wall frame



(c) Slab column frame



(d) Column and edge of slab.

Fig 2.5 Frame action and twisting of edge beam. [10]

2.2.2. Effective Beam Width (*Slab-beams)* Method

In effective beam width models, an actual slab is replaced with a beam element, which rotates uniformly across its transverse width. And the effective beam width is defined as the slab width for which uniform rotation across its width gives the same column displacement as the original slab. In this method, the columns of the model structure remain the same, but floor slabs are replaced with equivalent slab-beams. Determination of an accurate effective width is an issue of paramount importance with respect to modeling of the structures. Overestimation of the beam width will result in a stronger and stiffer beam than appropriate, possibly leading to premature column failure. Furthermore, the increase inertia of the shallow beam, although related linearly with width, may lead to an underestimation of lateral drifts. On the other hand, underestimation of the effective width can trigger premature beam failure if the predicted capacity is exceeded by the imposed demand. [19]

Many analytical and experimental studies for obtaining the effective beam width have been performed. Some studies focused on defining the elastic widths of the effective beam according to connection geometries. Others concerned the stiffness reduction phenomenon due to cracking and vertical member rectangularity in connection geometries and the combined effects of creep and shrinkage in stiffness reduction in the analysis model. Each study was based on the elastic and non-elastic behavior of the slab.[19]

The use of effective beam width concept for the analysis of flat plate systems subjected to earthquake loading has been recommended by Luo and Durrani (1995a and 1995b), amongst others. For the purpose of developing a rational method for the selection of the effective width, the authors conducted extensive analytical research based on the results of numerous tests on slab-column connections reported in literatures. Their study considered as variables the column and slab aspect ratios, the magnitude of the gravity loads, the location of the connection (interior or exterior) and the presence of the stirrups and spandrel beams as part of the slab. The suggested expressions of the effective width coefficient for interior connections, α_i takes the form:

$$\alpha_i = \chi \frac{R_{12}\left(\frac{c_1}{l_2}\right)}{0.05 + 0.002\left(\frac{l_1}{l_2}\right)^4 - 2\left(\frac{c_1}{l_1}\right)^3 - 2.8\left(\frac{c_1}{l_1}\right)^2 + 1.1\left(\frac{c_1}{l_1}\right)} \qquad (2.9)$$

Where:

$$R_{12} = -0.0221\left(\frac{c_1}{c_2}\right)^4 + 0.0281\left(\frac{c_1}{c_2}\right)^3 + 0.1535\left(\frac{c_1}{c_2}\right)^2 + 0.773\left(\frac{c_1}{c_1}\right) + 0.0845 \quad \ldots\ldots \quad (2.10)$$

$$\chi = \left(1 - 0.4 \frac{V_g}{4 A_c \sqrt{f_c'}}\right) \qquad (2.11)$$

χ -is a Parameter for reduction factor accounting for gravity loads;

C_1 and C_2 - are the column dimensions in the bending direction and normal to the bending direction, respectively.

l_1 and l_2 -Represent span lengths in bending direction and normal / transverse to l_1, respectively, both are measured center to center of the columns.

V_g -is the direct shear force due to gravity load only,

A_c - is the area of slab critical section (when it is taken at d/2).

f_c' is the compressive concrete strength.

The expression for the exterior connections has a similar form. [16] In an analytical study made by Jung-Wook Choi and Jin-Gyu Song (2005) for obtaining the effective beam width, much concern is given to the effect of vertical member rectangularity in connection geometries and the combined effects of creep and shrinkage in stiffness reduction, i.e. in addition to defining the elastic widths of the effective beam according to connection geometries; it attempts to quantify stiffness reduction phenomenon due to cracking.

A common technique or approach for obtaining elastic effective beam widths *(l_e)* is to cut a typical interior connection under lateral load only from a continuous floor system and then to bound the connection by lines of inflection and symmetry as shown in *Fig. 2.6.* These lines are assumed to be located along the slab mid-spans perpendicular and parallel to the loading direction. Actually, the position of the inflection lines will change if the stiffness of adjacent vertical members located parallel to the loading direction is not the same. However, this change can generally be ignored for simplicity. [11] A joint region exists wherever the slab connects to a vertical member. The total flexural stiffness of the connection

increases with the stiffness of the joint region. As shown in *Fig. 2.7*, Aalami (1972) assumed two bounds on the stiffness of the joint region: one considers the slab within the joint region to be rigid; the other assigns a joint flexibility equal to that of the surrounding slab. The behavior of the joint region depends on the relative flexural stiffness between the joint region and the surrounding slab. In most practical cases, this relative stiffness is high, so most analytical investigations have idealized the joint region as rigid. [27] Based on this assumption, an elastic stiffness value for a connection can be obtained.

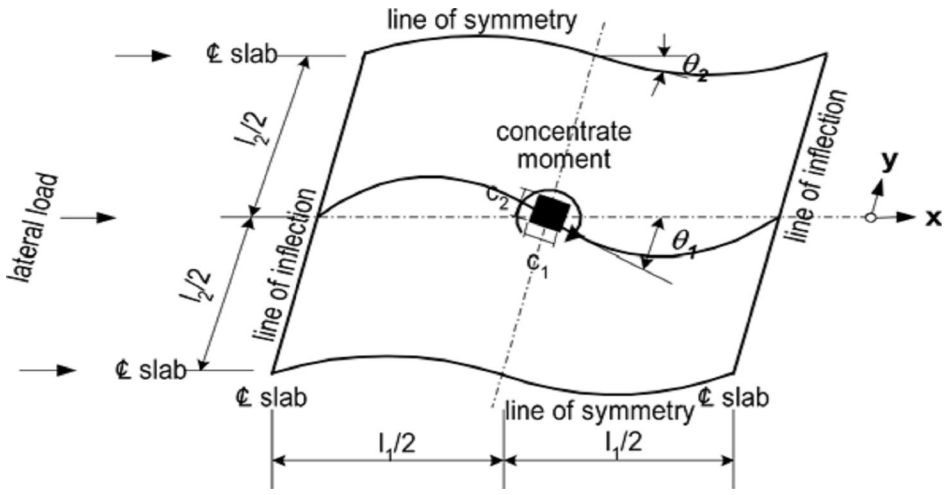
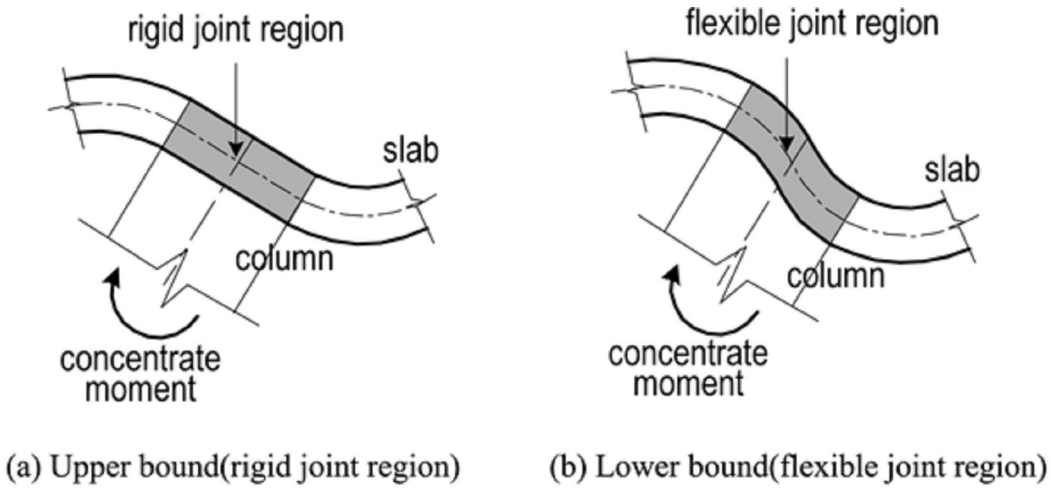


Fig. 2.6 A slab-vertical member connection. [11]



(a) Upper bound(rigid joint region) (b) Lower bound(flexible joint region)

Fig. 2.7 Two bounds on the stiffness of the joint region.[11]

An elastic effective beam width ***(l_e)*** represents the elastic stiffness of a slab connected to a vertical member. The elastic stiffness of the slabs is affected by connection geometries. To obtain the elastic widths, slabs with different geometric conditions are modeled by using the finite element technique. In the finite element model, applied loads transferred from a vertical member to a slab are represented by using the concentrated moment load (M) applied at the center of the joint region, which is treated rigidly. Structural analyses are performed by using general purpose software, and then with the results of the analyses, the slab rotations (θ_s) at the center of the joint region are acquired. The slab's flexible stiffness can be calculated by using M/θ_s.

The stiffness of the beam elements for rigid and flexible bar models can be obtained by dividing the moment load (M) by the beam rotations (θ_{br} and θ_{bf}). The elastic widths for both models can be calculated by equating the flexural stiffness of the beam element to that of the slab.

As flat plate structure includes two distinct substructures, the interior frame and the exterior frame, as shown in *Fig. 2.8.* For the connections in an interior frame, the elastic effective beam width for a rigid bar model is represented as;

$$l_e = 2c_1 + 0.3l_1 + 0.8\,(c_2 - c_1) \qquad (2.12)$$

Where;

l_e- elastic effective beam width.

C_1 – is the column dimensions in the bending direction

C_2 -are the column dimensions in the transverse direction.

l_1 -Represent span lengths in bending direction.

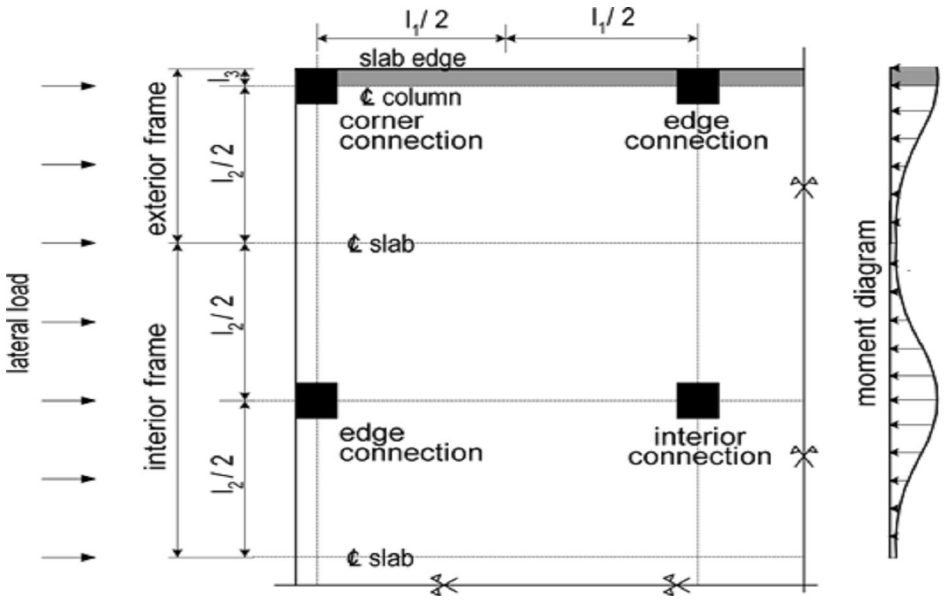


Fig. 2.8 Flat plate structures.[11]

For an exterior frame, some adjustment of Eq. (2.12) is required. As shown in fig 2.8, the dimension l_3 is the distance from the slab edge to the centerline of the vertical member. The slab rotation of the strip in the dimension l_3 is assumed to be equal to the uniform rotation of the effective beam element. Thus, the elastic

widths for the exterior frame can be calculated as the sum of half of the values obtained from Eq. (2.12) and l_3:

$$l_e = c_1 + 0.15l_1 + 0.4\,(c_2 - c_1) + l_3 \qquad (2.13)$$

The values obtained from Eqs. (2.12) and (2.13) are applicable to the rigid bar model, where rigid bars having the length of $c_1/2$ model the joint region at both ends of the beam element. If the rigid bars are not included in the model, the widths given by Eqs. (2.12) and (2.13) should be multiplied by the factor of $\mathbf{1/\,(1-c_1/l_1)^3}$.

The same Author suggests, the stiffness reduction β due cracking resulting from service gravity loads and the combined effects of creep and shrinkage can be defined by

$$\beta = \frac{DL + LL}{\xi L_{sus} + DL + LL} * \frac{17\, f_{ck} h^3}{L_{sus} l_n^4} \geq \frac{1}{4} \qquad (2.14)$$

Where;

β - Stiffness reduction factor due to cracking.

ξ - is the time dependant factor used to calculate additional long-term deflection resulting from creep and shrinkage. $0 \leq \xi \leq 1$

DL - Service dead load.

LL - Service live load.

L_{sup} - Superimposed Load.

L_{sus} – Sustained Load.

Kang and Wallace (2005) explains that the effective Slab width model is a common method used to model the lateral-load stiffness of the slab-column frame. In here the three-dimensional system is modeled as a two-dimensional frame using an effective slab width $\boldsymbol{\alpha l_2}$ and a conventional column, where the α-factor is derived using elastic plate theory to result in an equivalent slab width with uniform rotation across the effective slab width that yields the same rotation stiffness as the original system with non-uniform rotation. The α-value for the effective slab width model depends primarily on the, whereas the influence of cracking on stiffness is accounted for by using an additional factor, commonly referred to as a β-factor. Analytical and experimental studies were conducted to assess appropriate modification factors to use for effective slab width (α) and cracking (β) for lateral load analysis of slab-column frames. Using an effective slab with model α-values of 0.75, and β-values of 1/3, resulted in generally good correspondence between experimental and analytical results for periods and top displacements in the elastic range. [29]

Hueste and et al (2007), Suggest the recommendation by Hwang and Moehle (2000) which may be used to establish the effective stiffness of the slab and include the impact of cracking. Hwang and Moehle (2000) recommended that the uncracked effective stiffness for a model with rigid joints, for ratios of c_2/c_1 from ½ to 2 and a slab aspect ratio l_2/l_1 greater than 2/3, be determined using an effective beam width represented as

$$b_{int} = 2C_1 + l_1/3 \qquad (2.15)$$

Where b_{int} is the effective width for interior frame connections; C_1 and l_1 are the column dimension and slab span parallel to the direction of load being considered; and c_2 and l_2 correspond to orthogonal directions. For exterior connections half the width defined for the interior connection will be used. Effects of cross section changes, such as slab opening, are to be considered. One way to accomplish this is to vary the value of the effective beam along the span. [8]

To account for cracking, a stiffness reduction factor β has been proposed by Hwang and Moehle 2000 for non pre-stressed slabs and is given by;

$$\beta = 4c/l > 1/3 \qquad (2.16)$$

Where c and *l* are the column dimension and slab span parallel to the load direction. For the determination of the width of the concealed beam, or unbalanced moment transfer width or the slab-beam, there are several different considerations.

2.2.3. UNBALANCED MOMENT TRANSFER WIDTH

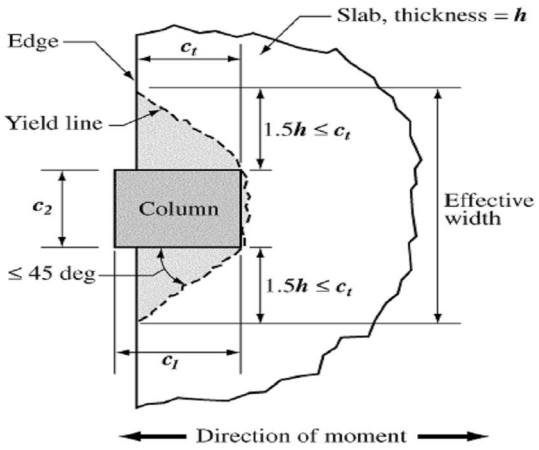
An expression for the width of the concealed beam by the Mexico Building Code (1987) is given as

$$b_{eff} = c + 3h \qquad (2.17)$$

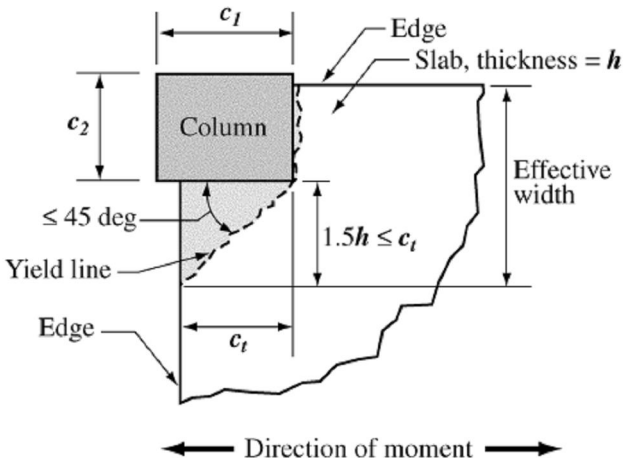
Where c is the column dimension in the direction perpendicular to the analysis and h is the slab depth. And the Greek Code for Reinforced Concrete Structures (1999) states that in the presence of lateral loads, the slab beam width is taken as

$$b_{eff} = b_o + 2h_s \qquad (2.18)$$

Where b_o is the column width in the direction of interest and h_s is the slab depth. Finally in ACI 318-99, Clause 13.5.3 states "*This section is concerned primarily with slab systems without beams……all reinforcement resisting the part of the moment to be transferred to the column by flexure should be placed between lines that are one and one-half the slab or drop panel thickness, 1.5h, on each side of the column…..*". Hence ACI defines a region of the slab which has special flexural reinforcement and which is like a beam concealed within the slab thickness with a width of b_w = c+2*(1.5h) which is similar to EBSC 2 and the Mexican Building Code (1987).



a) Edge connection



b) Corner connection

Fig.2.9. Effective width for reinforcement placement in edge and corner connections. [2]

The main issues in lateral load analysis and design of flat-slab frames can be listed as ***lateral load stiffness***, ***lateral load strength, ductility and resistance to progressive collapse*** (Moehle, 1986). These issues are discussed briefly in the following paragraphs.

A-Lateral Load Stiffness

Lateral drift is likely to be a significant design issue for flat-slab frames. In many cases, lateral drift considerations rather than strength considerations will control the frame proportions. Large flexibility of flat-slab frames causes concern in seismic design for several reasons including possible damage to structural and non-structural components and overall of the frame under excessive drifts.

B-Lateral load strength and ductility

Both shear and unbalanced moment result in slab shear stresses in the vicinity of the column and shear stresses due to the combination of both may control strength. Under combined shear and moment transfer, a linear variation of shear stresses is presumed to develop according to the ACI Building Code (ACI 318). The model predicts a linear interaction between shear and unbalanced moment transfer strengths. Shear and unbalanced moment strength of edge and corner slab-column connections are generally affected by the same parameters that influence the behavior of interior connections, when design and analysis procedures are similar. However, for frames located in regions of high seismic risk, most building codes require the use of perimeter frames to comprise conventional beam-column framing.

C-Resistance to progressive collapse

Slab-column connections are susceptible to punching shear failures when subjected to inelastic load reversals. The methods for strengthening the slab-column connections against punch-shear failures include the use of drop panels or slab shear reinforcement or the use of high strength concrete in the slab at the vicinity of the columns. Among these, drop panels (or shear capitals) tend to increase the length of the perimeter of the punching shear critical section. Drop panels with small plan dimensions are not effective in an earthquake when lateral forces can produce reversals of relatively high unbalanced moments in slab in column vicinity. In such cases, an inverted punching failure can occur for which the additional slab depth provided by the shear capital is not effective in increasing punching shear capacity. An alternative to preventing the punching shear failure is the use of slab shear reinforcement generally in the form of vertical legs of stirrups. Anchorage is provided by means of hooks, bends and the longitudinal slab flexural reinforcing bars lodged at the corners of the stirrup. With stud shear reinforcement, anchorage is provided mechanically by means of a forged head at one end and a steel strip at the other end.

Megally and Ghali (2000) stated that providing stud shear reinforcement increases the punching resistance and prevents brittle failure even in a severe earthquake. Finally, the use of high strength concrete instead of normal strength concrete delays the punching failure until the yield strength of more flexural reinforcing bars is developed. Punching strength is directly proportional to the tensile strength of concrete different than other types of failures in ordinary structures that are dominated by steel; therefore in flat-slab structures, concrete quality is extremely important. [6]

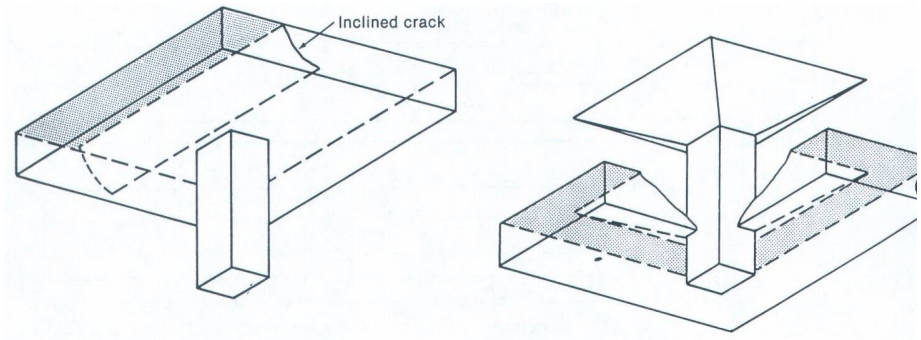
Although all the methods discussed above were successful in increasing the punching strength, the effects of the three different strengthening methods on ductility are substantially different. In addition, when a single connection fails, loads originally supported at the connection will be transferred to surrounding connections if no provisions were made to suspend the slab at the punched connection. These may, in turn, fail under increased loads, possibly resulting in a progressive collapse. To avoid a progressive collapse, either original punching failure must be prevented or progressive collapse must be arrested by suspending the slab from the columns after punching of the individual connections. The first method of avoiding the original punching shear failure can be achieved either by ensuring very low shear stresses under gravity loads or by providing slab stirrup reinforcement. Both of these methods are often considered unsatisfactory because of apparent economic reasons. An alternative method of preventing progressive collapse by suspending the slab from the columns is, in many cases, the more acceptable solution. To achieve this goal, the bottom longitudinal reinforcement should be continuous over the columns at slab-column connections since it is capable of significant post-punching resistance.

2.3. FORCE AND MOMENT ANALYSIS

2.3.1. SHEAR IN FLAT SLAB BUILDINGS

When two-way slabs are supported directly by columns, as in flat slabs and flat plates, or when slabs carry concentrated loads, as in footings, shear near the columns is of critical importance. Tests of flat plate structures indicate that, in most practical cases, the capacity is governed by shear.

Two kinds of shear may be critical in the design of flat slabs, flat plates, or footings as shown in fig 2.10. The first is the familiar beam-type shear leading to diagonal tension failure. Applicable particularly to long narrow slabs, this analysis considers the slab to act as a wide beam, spanning between supports provided by the perpendicular column strips. A potential diagonal crack extends in a plane across the entire width 1_2 of the slab. According to ACI 318, the critical section is taken a distance *d* from the face of the column or capital. As for beams, the design shear strength (φV, must be at least equal to the required strength V_u at factored loads, i.e. $V_u \leq \varphi V_c$ *or* else shear reinforcement is required.



(a) One - way shear (b) Two - way shear

Fig. 2.10 Shear failure in slab. [10]

The nominal shear strength V_c for one-way shear can be calculated by

$$V_c = \frac{1}{6}\sqrt{f'_c}\, b_w d \quad \text{KN} \tag{2.19}$$

Where;

b_w -is the span between two supports in the direction being considered.

d - Is the effective depth.

f'_c – Concrete compressive strength.

Alternatively, ACI states that failure may occur by *punching shear*, with the potential diagonal crack following the surface of a truncated cone or pyramid around the column, capital, or drop panel, as shown in Fig. *2.11*. The failure surface extends from the bottom of the slab, at the support, diagonally upward to the top surface. The angle of inclination with the horizontal, θ (see Figure 2.11), depends upon the nature and amount of reinforcement in the slab. It may range between about 20 and 45°. The critical section for shear is taken perpendicular to the plane of the slab and a distance *d*/2 from the periphery of the support, as shown. The shear force V_u to be resisted can be calculated as the total factored load on the area bounded by panel centerlines around the column less the load applied within the area defined by the critical shear perimeter.

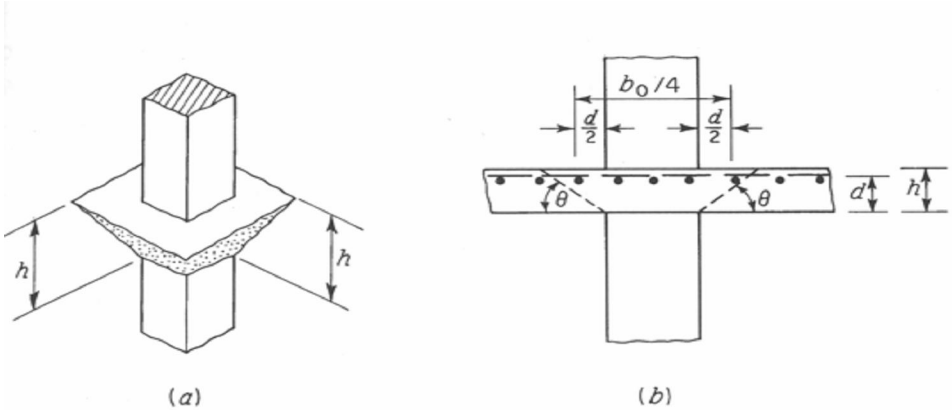


Figure 2.11. Failure surface defined by punching shear.[30]

At such a section, in addition to the shearing stresses and horizontal compressive stresses due to negative bending moment, vertical or somewhat inclined compressive stress is present, owing to the reaction of the column. The simultaneous presence of vertical and horizontal compression increases the shear strength of the concrete.

Many studies have been performed to address the issues of flat plate punching resistance. In June 2000, the International Workshop on Punching Shear of Reinforced Concrete Slabs was held at the Royal Institute of Technology, Stockholm, Sweden. A total of 54 new papers were submitted into the proceedings, most of them addressing the direct punching resistance of slabs. This research activity has, over the years, helped change building codes around the world to prevent this type of failure from occurring.

For example, in 1971, the American Concrete Institute adopted a change to their code (ACI 318-71), to require continuous bottom reinforcement through the column at slab-column connections. This change was intended to prevent progressive collapse once a punching shear failure occurred.

Punching failure can result from two different loading conditions; excessive gravity loading around a slab-column connection (pure shear) or a combined gravity loading and lateral loading due to wind or earthquake (shear and flexure interaction). In the first case, the gravity load being transferred from the slab to the column causes direct shear stresses, which are greater than the shear capacity of the slab concrete. In the second case, the gravity load causes similar shear stresses and the lateral deflections cause an unbalanced moment that contributes additional shearing stress. Punching occurs when the total shear stress exceeds the concrete shear capacity. The punching caused by shear and flexure is the primary failure mode that should be investigated carefully.

For slabs supported by columns having a ratio of long to short sides not greater than *2,* tests indicate that the nominal shear strength may be taken equal to

$$V_c = 4\sqrt{f'_c}\,b_o d \quad KN \qquad (2.20)$$

Where

b_o - the perimeter along the critical section.

d- is the effective depth.

f'_c – Concrete compressive strength.

However, for slabs supported by very rectangular columns, the shear strength predicted by Equation (2-20) has been found to be unconservative. The value of V_c approaches $2\sqrt{f'_c}\,b_o d$ *as* β_c, the ratio of long to short sides of the column, becomes very large.

Based on this, ACI Code 11.12.2 states further that V_c in punching shear shall not be taken greater than

$$V_c = \left(2 + \frac{4}{\beta_c}\right)\frac{\sqrt{f'_c}}{12} b_o d \quad KN \qquad (2.21)$$

Further tests have shown that the shear strength V_c decreases as the ratio of critical perimeter to slab depth, b_o/d, increases. Accordingly, ACI Code 11.12.2 states that V_c in punching shear must not be taken greater than

$$V_c = \left(\frac{\alpha_s * d}{b_o} + 2\right) * \left(\sqrt{f'_c}\,b_o d / 12\right) \quad KN \qquad (2.22)$$

Where a_s *is 40* for interior columns, *30* for edge columns, and *20* for corner columns, i.e., columns having critical sections with *4, 3,* or *2* sides, respectively. The shear design strength is the smallest of three equations given above.

2.3.2. MOMENT TRANSFRER AT SLAB COLUMN CONNECTIONS

Eccentric shear stress model

The eccentric shear stress model is the basis of the general design procedure embodied in ACI 318 for determining the shear strength of slab-column connections transferring shear and moment. The model is adopted by the 1971 version of the ACI 318 and only minor modifications have been included in subsequent version. Recently, ACI 318-05 has incorporated special provisions related to the lateral - load capacity of slab-column connections in structures located in regions of high seismic risk. [18]

The analysis for punching shear in flat plates and flat slabs presented in the previous procedure assumed that the shear force V_u was resisted by shearing stresses uniformly distributed around the perimeter b_o of the critical section, at distance $d/2$ from the face of the supporting column, and the nominal shear strength V_c was given by Eqs. (2-20, 2-21, 2-22).

If significant moments are to be transferred from the slab to the columns, as would result from unbalanced gravity loads on either side of a column and/or from horizontal loading due to wind or seismic effects, the shear stress on the critical section is no longer uniformly distributed. *i.e*. slab-column connections experience very complex behavior when subjected to lateral displacement or unbalanced gravity loads. This involves transfer of flexure, shear, and torsion in the portion of slab around the column.

Relatively simple design equations have been derived by considering the critical section to be at d/2 away from the face of the column and by assuming that shear stress on the critical perimeter varies linearly with distance from the centroidal axis.

For a slab -column connection transferring shear and moment, The ACI 318-05 design equations for limiting the shear stress v_u are given by

$$v_u \leq \varphi V_n \qquad (2.23)$$

$$v_u = \frac{V_u}{b_o * d} \pm \frac{\gamma_v M_u C}{Jc} \qquad (2.24)$$

Where,

v_u- is the factored shear stress.

φ - is the stress reduction factor for shear.

v_n- is the nominal shear stress.

V_u- is the factored shear force acting at the centroid of the critical section.

M_u - is the factored unbalanced bending moment acting about the centroid of critical section.

d - is the distance from the extreme compression fiber to the centroid of the longitudinal tension reinforcement.

b_o - is the length of the perimeter of the critical section.

c- is the distance from the centroidal axis of the critical section to the point where shear stress is being computed.

J- is a property of the critical section analogues to the polar moment of inertia.

γ_v – is the fraction of the unbalanced moment considered to be transferred by eccentricity of shear, defined by

$$\gamma_v = 1 - \frac{1}{1 + \left(\frac{2}{3}\right)\sqrt{b_1 / b_1}} \qquad (2.25)$$

b_1 and b_2 - are the widths of the critical section measured in the direction of the span for which M_u is determined (Direction 1) and in the perpendicular direction (Direction 2).

For an interior column and critical section of rectangular shape, b_o and J are determined by

$$b_o = 2(b_1 + b_2) \qquad (2.26)$$

$$J = \frac{2db_1^3}{12} + \frac{2b_1d^3}{12} + \frac{db_2b_1^2}{2} \qquad (2.27)$$

The first term of Eq. (2.24), the shear stress due to direct shear, is assumed uniformly distributed on the critical section, and the fraction γ_v M_u is assumed to be resisted by linear variation of shear stress on the critical section.

The situation can also be modeled as shown in Figure 2.15 and 2.16. Here V_u represents the total vertical reaction to be transferred to the column, and M_{ub} represents the unbalanced moment to be transferred, both as factored loads. The vertical force V_u causes shear stress distributed more or less uniformly around the perimeter of the critical section as assumed earlier, represented by the inner pair of vertical arrows, acting downward. The unbalanced moment M_u causes additional loading on the joint, represented by the outer pair of vertical arrows, which add to the shear stresses otherwise present on the right side, in the sketch, and subtract on the left side.

Tests indicate that for square columns about 60 percent of the unbalanced moment is transferred by flexure (forces T and C in *Figure 2.15*) and about 40 percent by shear stresses on the faces of the critical section. For rectangular columns, it is reasonable to suppose that the portion transferred by flexure increases as the width

of the critical section that resists the moment increases, i.e., as $c_2 + d$ becomes larger relative to $c_1 + d$ in *Figure 2.13.*

According to ACI Code 13.5.3, the moment considered to be transferred by flexure is

$$M_{uf} = \gamma_f M_u \qquad (2.28)$$

Where-

$$\gamma_f = \frac{1}{1 + \frac{2}{3}\sqrt{b_1 / b_2}} \qquad (2.29).$$

While that assumed to be transferred by shear, According to ACI Code 11.12.6, is

$$M_{uv} = \left[1 - \frac{1}{1 + \frac{2}{3}\sqrt{b_1/b_2}}\right] M_u . \qquad (2.30)$$

Where:

$b_1=c_1+d$, and $b_2=c_2+d$, …… for Interior Column

$b_1=c_1+d/2$, and $b_2=c_2+d$, ……..for Edge Column

$b_1=c_1+d/2$, and $b_2=c_2+d/2$,……..For Corner Column.

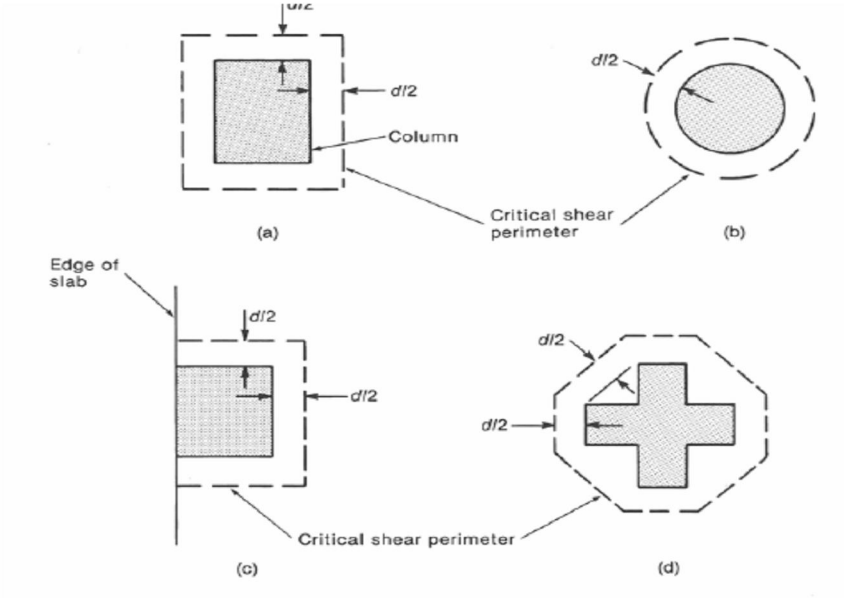
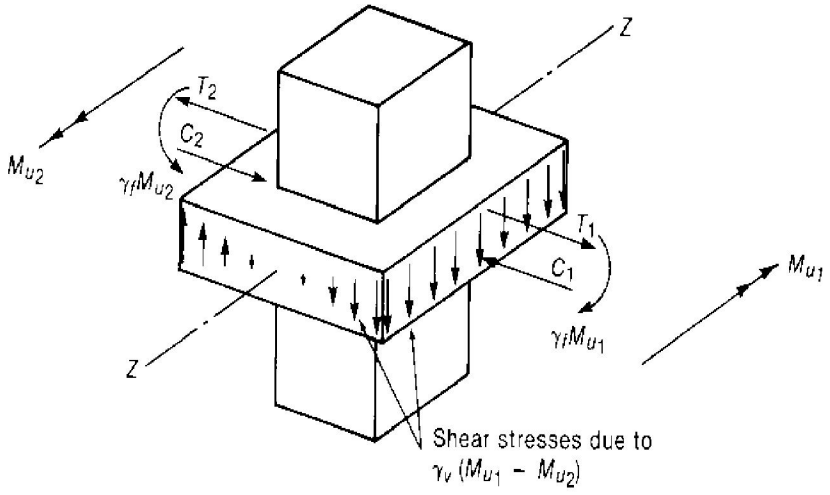
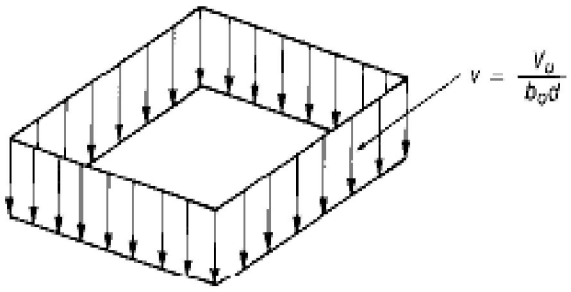


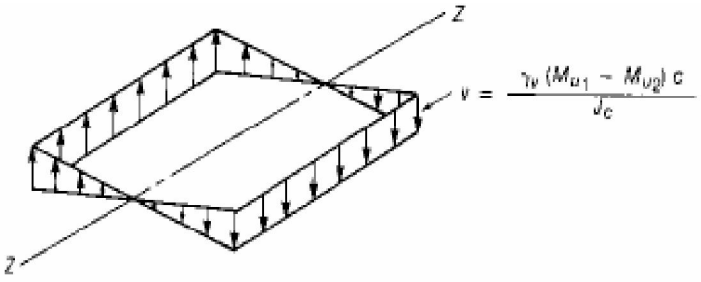
Figure 2.12. Failure surface defined by punching shear. [10]



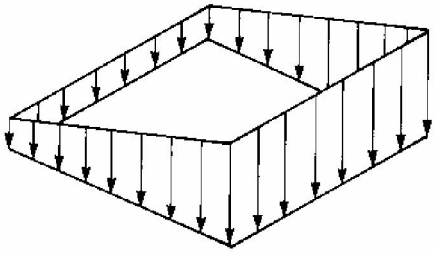
A) Transfer of Unbalanced moments to column.



B) Shear stress due to Vu.

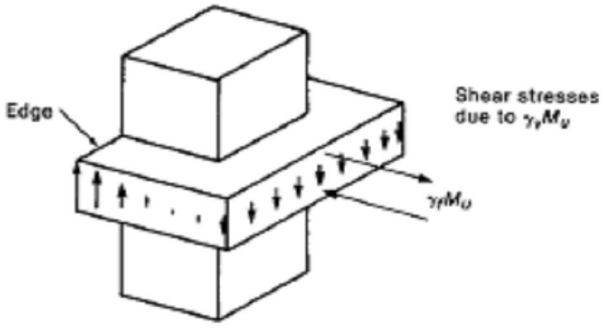


C) Shear Stress due to unbalanced moment.

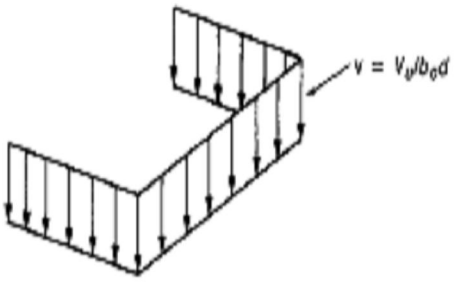


D) Total Shear Stress

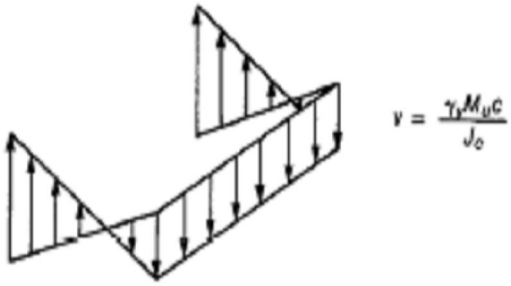
Figure2.13.Shear stress due to unbalanced moments at interior columns [10]



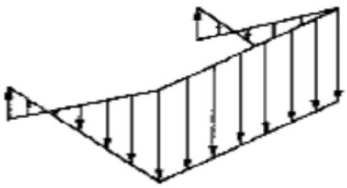
A) Transfer of moment at edge column.



B) Shear Stress due to V_u.



C) Shear Stress due to M_u.



D) Total shear stress

Figure 2.14.Shear stress due to unbalanced moments at edge columns.[10]

The portion of the moment not carried by eccentric shear is to be carried by slab flexural reinforcement placed within lines 1.5h on either side of the column. Where h is the slab thickness. This flexural reinforcement is also used to resist slab design moments within the column strip. [19]

2.4. CONNECTION DEFORMATION CAPACITY

Even if other structural components, such as shear walls or perimeter moment frames, are used as the primary system to resist lateral loads, slab-column connections should have sufficient rotational capacity to avoid a punching failure so that the gravity load-carrying capacity can be maintained under seismic excitations. Therefore, deformation capacity is of particular concern for slab-column connections subjected to lateral loads.

To ensure minimal level of ductility ACI 352.1R-89 recommends that for all slab-column connection subjected to seismic loads, without shear reinforcement, the direct factored shear V_u acting on the connection, for which inelastic moment transfer is anticipated ,must satisfy

$$V_u \leq 0.4V_c = 0.4v_c b_o d \qquad (2.31)$$

Where,

v_c - is the nominal shear stress resistance as defined in equations (2.20-2.22)

V_u- is the direct factored gravity shear force.

V_c- is the nominal shear force resistance at the critical section.

The limitation defined by equation 2.31 above was based on a review of test data that revealed that the deformation capacity of interior connection without shear reinforcement is inversely related to the direct shear on the connection. Connections not complying with Eq. 2.31 exhibit virtually no post-yield deformation capacity under lateral loading. Pan and mahole (1989) found that when the stress due to direct shear approaches $0.4v_c$, the connection experiences a brittle failure for story drift ratio of approximately 1.5%. However equation 2.31 may be waived if calculations demonstrate that the imposed displacement will not induce yield in the slab system. For example, use of structural wall may

adequately limit the imposed drifts on the slab-column frames such that yield at the slab-column connection may not occur. [18]

Test data strongly indicated the trend of connection deformation capacity being reduced by the increased gravity load. The connection deformation capacity, generally expressed as the inter-story drift ratio, was empirically formulated by Pan and Moehle (1989), Megally and Ghali (2000), Durrani, Du, and Luo (1995), Hueste and Wight (1999), and Robertson and Johnson (2006). It is noted that the ratio of V_u / V_c has been hitherto formulated as the only variable affecting the connection deformation capacity.

Instead of calculating the induced effects under the design displacement, ACI318-05 describes a perspective approach. The connection is evaluated based on a simple relationship between the design story drift ratio (DR) and the shear stress due to factored gravity loads. The design DR (story drift divided by storey height) should be taken as the largest value for the adjacent stories above and below the connection. [18] The maximum DR (in percent) that the slab-column connection can tolerate, in the absence of shear reinforcement, is given by the following relationship and illustrated in fig 2.16

$$DR = \begin{cases} 3.5 - 5.0VR \ (\ forVR \ < 0.6) \\ \\ 0.5(\ forVR \ \geq 0.6) \end{cases} \qquad (2.28)$$

Where VR is the shear ratio, defined as

$$VR = \frac{V_u}{\phi v_c b_o d} \qquad (2.29)$$

The factored shear force V_u on the slab critical section for two-way action is determined for the load combination 1.2D + 1.0L where D and L are the dead and live loads respectively.

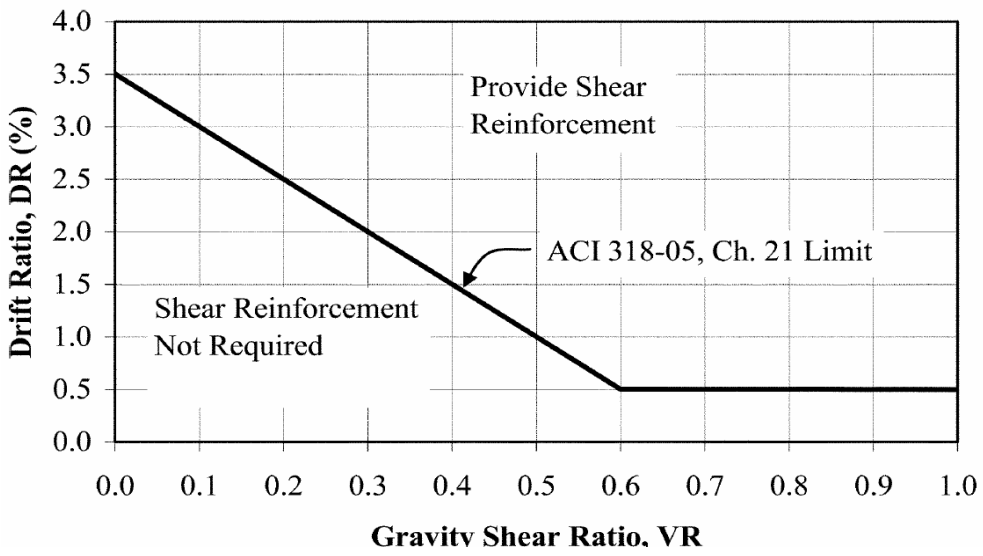


Fig 2.15. *Relationship for determining adequacy of slab-column connections in seismic regions.* [19]

3- PROJECT METHODOLOGY

3.1. Introduction

This chapter presents the proposed methodology to evaluate the effectiveness of the flat slab buildings with moment-resisting framing systems for a residential apartment buildings located in seismic zone 2. The methodology includes procedure followed in the analysis and design of these systems against vertical and lateral loads followed by evaluation against storey drift requirement. The chapter gives detailed account of variables, constraints and properties of structural members considered in creating the framing systems and a comprehensive account of the design analysis procedure of flat slab buildings. SAP 2000 and Microsoft Excel spreadsheets are software packages used in the analysis and design. The steps involved in the methodology are: basic model specifications, modeling of the flat slab and moment resisting framing systems, force analysis, structural design for strength, designing and evaluating these structures against story drift requirement and practical slab depth attainment respectively.

Basic model specifications, are first outlined as they outlines the study variables and constants. For the purpose of simplicity, building type, floor system, floor area, bay size and column layout and heights remain constant throughout the study. However, number of stories and member sizes are varied. The numbers of stories are varied to reflect a difference in aspect ratio of the buildings which is directly related to the magnitude of seismic force acting on the structure while member sizes are varied to assess design requirements. The next step examines modeling of the framing systems and material specifications. Analysis and design for ultimate failure using critical load combinations follows. Factored slab moments related to earth quake should be determined for load combination given in Eq.3.1 and 3.2. [2]

$$U=1.2D+E+L \qquad (3.1)$$

$$U=0.9D+E+1.6H \qquad (3.2)$$

Where;

U-is total factored load,

D is service dead load,

L-is service live load,

E-Earth quake loading and

H-is lateral load due to pressure of soil, water in soil or other material.

The final section in the methodology outlines the evaluation criterion which is the lateral drift and attainment of reasonable slab depth. Lateral drift from the framing systems are checked against code requirements. The framing system must meet drift requirements as well as be reasonable slab depth, as larger slab depth may result in increase dead load and cost. Typical slab span-depth ratios for RC flat plate construction are roughly 20 to 30. [29] And these values are taken to be a governing parameter during the overall slab depth determinations.

3.2. Study Variables and Constants

Flat slab structures have the advantages of simple construction and formwork, as well as a flat ceiling. Flat ceiling typically reduces ceiling finishing costs since architectural finishes are applied directly to the underside of the slab. And these structures are generally used for relatively light loads and for short to medium spans ranging from 4m to 6m. Hence, an apartment building is selected in the study. Bay size is often dictated by the architectural and functional requirements of the building. Large bay sizes are required to achieve maximum unobstructed floor space however, the economical use of a flat plate floor system usually requires columns that are spaced closer than those supporting one way joist floor systems. Since flat plates are economical for short to medium spans, a 5m × 5m bay size is selected. The square bay size has aspect ratio of one, and is the most economical as discussed in the previous chapter. Preliminary slab thickness and column sizes for the 5m × 5m floor bay sizes are chosen based serviceability requirement and data taken from existing building projects to conform to practical

member sizes. However, these member sizes are checked against code requirements to ensure adequacy. Columns are sized not only for adequate strength, but also for constructability and practicality to ensure that reinforcements are not congested. Square columns are selected in order to simplify design by eliminating the effects of orientation. Slab thickness for the moment-resisting framing system is determined depending on punching shear requirements as per section 2.3.2 of this document which is referred from Section 3.3.2 of ACI 318-02. It should also be noted that the story height parameter has a significant effect on the flexibility of structures, thus it is beneficial in design to retain it as short as practically possible. A story height of 3m is selected in this study.

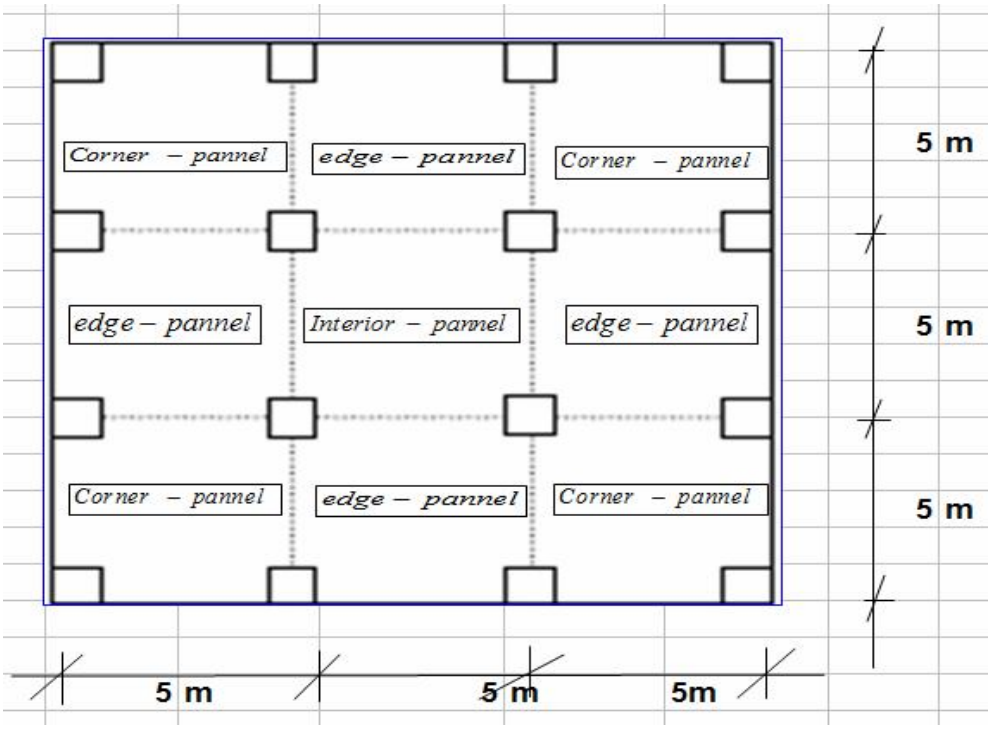


Fig 3.1 Typical floors lay out of the flat slab used through out the study.

Normal strength concrete C-25 with elastic modulus (E_c) 29,000 KN/m^2, Density of 23.5KN/ m^2, Poisson's ratio of 0.2 and partial safety factor of 1.5 is used as the concrete material (for column design) while deformed bars having yield strength

(f_{yk}) of 300MPa (S-300) and elastic modulus (E_s) of 200MPa and Partial safety factor of 1.15 are used for the reinforcing steel. The material properties remain constant throughout the study. The implementation of high-strength concrete is outside the scope of this work.

3.3. Methods

Direct design method (DDM), Equivalent Frame Method (EFM) or the Finite Element Method (FEM), are commonly used for determining moments or shearing forces for a flat slab floor system. In practically all instances, reinforcement location and orientation are specified by the design engineer, independent of the analysis method. This decision is made on the basis of support layout, ease of construction, and code stipulations. The analysis determines the amount of reinforcement. In this research the finite element method is used to calculate the shear force and bending Moment. Principal steps in analysis and design using FEM include selection of elements, meshing and idealization of the concrete structure features. Load combinations are taken in accordance with sec 9.2.1.of ACI318-2005.

Modeling for gravity Loading
Element Modeling

Flat slab structures are modeled using 4-node shell elements because shell elements are more suited than plate elements because shell elements represent both bending and in-plane (membrane) actions in a slab. According to Concrete international “software for the design of concrete building” a well distributed meshing, about eight elements per span length, will yield acceptable designs. Hence, the 5 m wide slabs were modeled by 6 shell elements in the y-direction and x direction and without stiffness modification as Mac Gregor (2005) recommend using elastic analysis method without stiffness modification in the case of gravity loading.

Concrete material properties are assumed as summarized below.

Table 3.1.Material properties assumed for concrete.

Material	Density (KN/m^3)	Modulus of elasticity KN/m^2	Poisson ratio
Concrete	23.5	29,000	0.2

Loading

Applied loads as surface pressure over the top face of shell elements are.

Dead Load

*Self weight of the RC Slab = h*23.5 KN/M^2*

*Weight of 3cm thick cement screed applied on the floor=0.03*23KN/M^2*

*Weight from light weight floor finish=0.003*16KN/M^2*

Weight from Partition wall =1.5KN/m^2

Live load = 2 kN/m^2

After running every model it is always a good practice to check equilibrium and examine the deflected shape prior to analyzing results to detect any possible errors, especially in support conditions and loading directions. However, the magnitude of deflection at any point represents the elastic properties of the materials which need to be adjusted in order to account for cracking and time-dependent effects such as creep.

Modeling for Seismic Loading

Having a complete three dimensional model for the entire building may not be practical if the buildings are symmetric both in plan and elevation. However, dividing the building into small models should not be done without ensuring that the overall behavior is not changed significantly.

The regularity of the buildings in terms of mass and stiffness in both plan and elevation enabled the utilization of two-dimensional analysis in the assessment of seismic response. In this study, the three dimensional flat-slab building is modeled as 2-D planar frame. In such a way that an exterior and an interior frame were linked with rigid links as shown, so that only lateral forces and displacements are transmitted between the two frames.

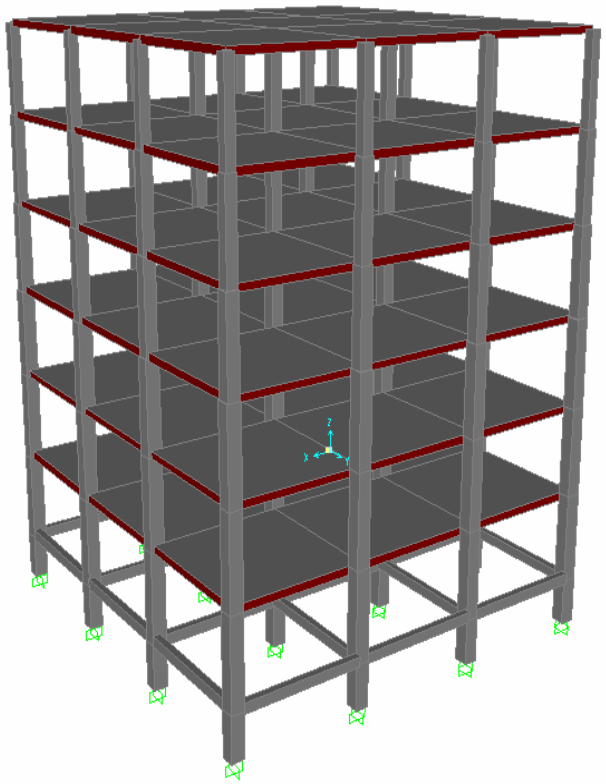


Fig 3.2 Three dimensional flat slab Building.

Exterior frame Interior frame

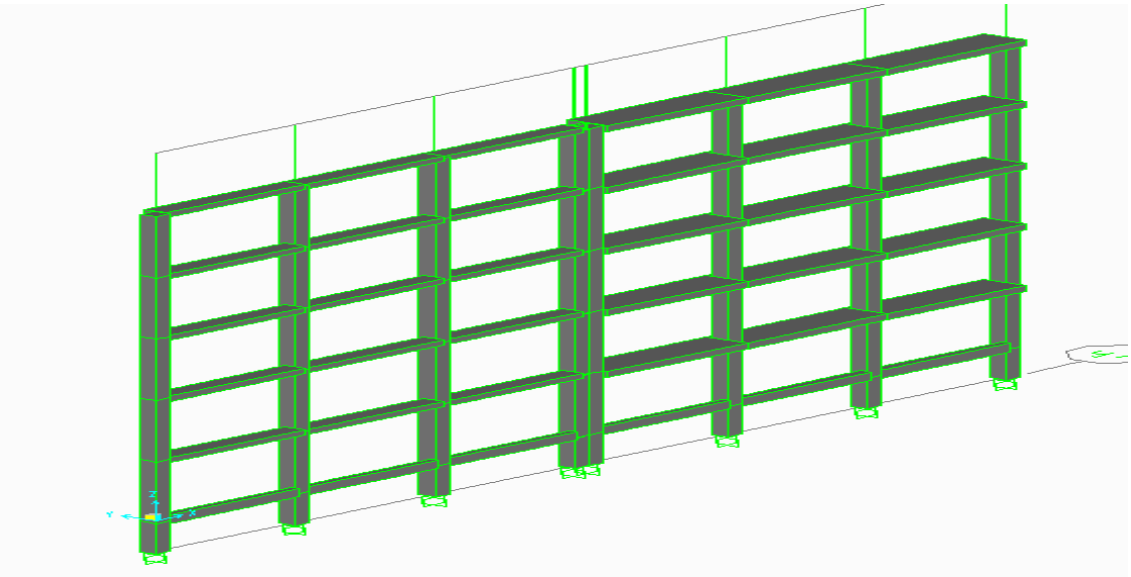


Fig 3.3 Two dimensional equivalent frame in 3d View

For the modeling of the 2-D planar frame, the portion of the slab that will contribute to the frame analysis should be determined as well as the width of the concealed beam within this slab portion. Two simplified methods exist for the former, namely the Effective Beam Width and the Equivalent Column Method, in which the effect of the slab is accommodated by appropriate modification of the beam width or the column stiffness, respectively. In this thesis the effective beam width models is used, where the actual slab is replaced with a beam element which rotates uniformly across its transverse width. And the effective beam width is defined as the slab width for which uniform rotation across its width gives the same column displacement as the original slab. In this method, the columns of the model structure remain the same, but floor slabs are replaced with equivalent slab-beams. Determination of an accurate effective width is an issue of paramount importance with respect to modeling of the structures.

It is discussed in the previous chapters that many analytical and experimental studies for obtaining the effective beam width have been performed. Some studies focused on defining the elastic widths of the effective beam according to

connection geometries. Others concerned the stiffness reduction phenomenon due to cracking and vertical member rectangularity in connection geometries and the combined effects of creep and shrinkage in stiffness reduction in the analysis model. Each study was based on the elastic and non-elastic behavior of the slab.
In this study Effective Beam Width is used and the three-dimensional system is modeled as a two-dimensional frame using an effective slab width $\boldsymbol{\alpha l_2}$ and a conventional column, where the α-factor is derived using elastic plate theory to result in an equivalent slab width with uniform rotation across the effective slab width that yields the same rotation stiffness as the original system with non-uniform rotation. The α-value for the effective slab width model depends primarily on the column and slab aspect ratios, whereas the influence of cracking on stiffness is accounted for by using an additional factor, commonly referred to as a β-factor. Following the recommendation by Kang and Wallace (2005) an effective slab with model with α-values of 0.75 and β-values of 1/3 is used throughout the study. The same author explains that these values resulted in generally good correspondence between experimental and analytical results for periods and top displacements in the elastic range.

A width for the concealed beam, a region of the slab which has special flexural reinforcement is determined by *Eq.2.17* of this document.

$$b_w = c+2*(1.5h) \qquad (2.17)$$

Where c is the column dimension in the direction perpendicular to the analysis and h is the slab depth. This expression is identical to the Ethiopian Building Code Standard (EBCS-2), ACI and the Mexican Code.

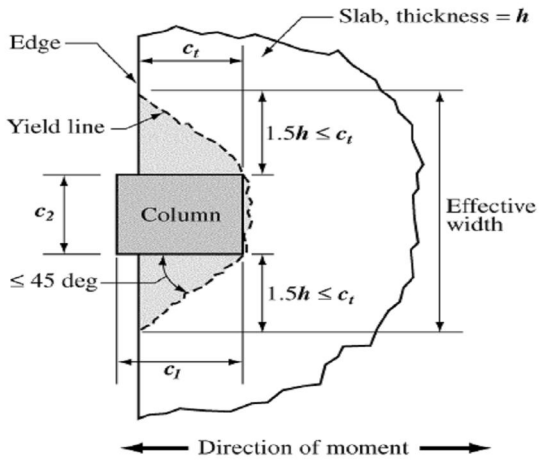


Fig.3.4 Effective width for reinforcement placement in edge connections.[2]

3.4. FLAT SLAB FLOOR SYSTEM.

A flat plate floor system is a concrete slab supported directly on columns with reinforcement in two orthogonal directions. The flat plate floor system has advantages in providing flexibility in layout of columns, partitions, small openings, etc.

According to Table 9.5(c) of ACI 318-05, the effective depth of flat plate is required to be greater than or equal the values shown on table 3.2

	Without drop panels‡			With drop panels‡		
	Exterior panels		Interior panels	Exterior panels		Interior panels
f_y, MPa†	Without edge beams	With edge beams§		Without edge beams	With edge beams§	
280	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{40}$	$\frac{\ell_n}{40}$
420	$\frac{\ell_n}{30}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{33}$	$\frac{\ell_n}{36}$	$\frac{\ell_n}{36}$
520	$\frac{\ell_n}{28}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{31}$	$\frac{\ell_n}{34}$	$\frac{\ell_n}{34}$

* For two-way construction, ℓ_n is the length of clear span in the long direction, measured face-to-face of supports in slabs without beams and face-to-face of beams or other supports in other cases.
† For f_y between the values given in the table, minimum thickness shall be determined by linear interpolation.
‡ Drop panels as defined in 13.2.5.
§ Slabs with beams between columns along exterior edges. The value of α_f for the edge beam shall not be less than 0.8.

Table 3.2: Minimum thickness for flat slabs [10]

Shear in flat plate floor system.

One-way and two-way shears are considered in the design of flat plates supported directly on columns. One-way or beam-type shear is critical in long narrow slabs. Analysis for beam shear considers the slab to act as a wide beam spanning between columns. The critical section is taken a distance *d* from the face of the column.

Two-way or punching shear is generally the critical of the two types of shear in square flat slab systems supported directly on columns. Punching shear considers failure along the surface of a truncated cone or inverted pyramid around a column. The critical section is taken perpendicular to the slab at a distance *d*/2 from the perimeter of a column. In the absence of a significant moment transfer the shear force V_u to be resisted is calculated as the total factored load on the area bounded by panel centerlines around the column, less the load applied within the area defined by the critical shear perimeter. However, significant moments are transferred to the edge and corner columns due to gravity loads and significant moments to all interior and exterior columns as a result of horizontal loading due Seismic actions. Transfer of unbalanced moment between a slab and a column takes place by a combination of flexure and eccentricity of shear. Shear due to moment transfer is assumed to act on a critical section at a distance *d*/2 from the face of the column. In this research the traditional ACI method for calculating the strengths of slab column joints transferring both shear and moments which is given by Eq. 2.21 of this document.

$$v_u = \frac{V_u}{b_o * d} \pm \frac{\gamma_v M_u C}{J_c} \qquad (2.21)$$

Where

V_u-is the shear being transferred. It acts through the centroid of the critical section for shear; γ_v is the fraction of moment to be transferred by shear stress on the critical section.

J_c-is a section property similar to the polar moment of inertia of the critical shear section about the centroidal axes of the critical section.

C-is distance from centroid of critical section to the face of section where stress is being computed.

The portion of the moment transferred by flexure is assumed to be transferred over a width of slab equal to the concealed beam with transverse column width c_2+1.5h or two times (1.5*h*) on each side of the column. The moment on this effective slab width is resisted by concentrating negative reinforcement in the slab width.

The portions of the total moment to be transferred by flexure is taken to be

$$M = \gamma_f M_u$$

Where γ_f is the fraction of the moment that is transferred by flexural stress on the critical section.

And it is given by

$$\gamma_f = \frac{1}{1+\left(\frac{2}{3}\right)\sqrt{b1/b1}}$$

Where -b_1 and b_2 are column dimensions. For square interior or corner columns, 40% of the transfer moment is considered transferred by eccentricity of the shear and 60% by flexure. The flat slab buildings analyzed and designed in this study are assumed to be located in seismic hazard zone 2.

4-RESULTS AND DISCUSSIONS

The frames system flat slab structures are modeled and designed by chancing the number of stories or the Building height. The Ethiopian building standards are used to determine seismic actions, design the columns and determine the required amount of reinforcements the results are summarized in tables below.

Table 4.1 Interior Column sections and reinforcements

Building Type	Location	Column Dimension	Main Reinforcement
G+1	Ground floor	30cm x 30cm	8ϕ16
	1st Floor	30cm x 30cm	8ϕ16
G+2	Ground floor	30cm x 30cm	12ϕ20
	1st Floor	30cm x 30cm	8ϕ16
	2nd Floor	30cm x 30cm	8ϕ16
G+3	Ground floor	40cm x 40cm	12ϕ20
	1st Floor	30cm x 30cm	12ϕ20
	2nd Floor	30cm x 30cm	8ϕ16
	3rd Floor	30cm x 30cm	8ϕ16
G+4	Ground floor	45cm x 45cm	12ϕ24
	1st Floor	40cm x 40cm	12ϕ20
	2nd Floor	30cm x 30cm	12ϕ20
	3rd Floor	30cm x 30cm	8ϕ16
	4th Floor	30cm x 30cm	8ϕ16
G+5	Ground floor	50cm x 50cm	12ϕ24
	1st Floor	45cm x 45cm	12ϕ24
	2nd Floor	40cm x 40cm	12ϕ20
	3rd Floor	30cm x 30cm	12ϕ20
	4th Floor	30cm x 30cm	8ϕ16
	5th Floor	30cm x 30cm	8ϕ16
G+6	Ground floor	50cm x 50cm	12ϕ24
	1st Floor	50cm x 50cm	12ϕ24
	2nd Floor	45cm x 45cm	12ϕ24
	3rd Floor	40cm x 40cm	12ϕ20
	4th Floor	30cm x 30cm	12ϕ20
	5th Floor	30cm x 30cm	8ϕ16
	6th Floor	30cm x 30cm	8ϕ16

Table 4.2 Edge Columns section and reinforcement

Building Type	Location	Column Dimension	Main Reinforcement
G+1	Ground floor	30cm x 30cm	8ϕ16
	1st Floor	30cm x 30cm	8ϕ16
G+2	Ground floor	40cm x 40cm	8ϕ16
	1st Floor	30cm x 30cm	8ϕ16
	2nd Floor	30cm x 30cm	8ϕ16
G+3	Ground floor	40cm x 40cm	8ϕ16
	1st Floor	40cm x 40cm	8ϕ16
	2nd Floor	30cm x 30cm	8ϕ16
	3rd Floor	30cm x 30cm	8ϕ16
G+4	Ground floor	40cm x 40cm	8ϕ20
	1st Floor	40cm x 40cm	8ϕ16
	2nd Floor	40cm x 40cm	8ϕ16
	3rd Floor	30cm x 30cm	8ϕ16
	4th Floor	30cm x 30cm	8ϕ16
G+5	Ground floor	50cm x 50cm	8ϕ24
	1st Floor	40cm x 40cm	8ϕ20
	2nd Floor	40cm x 40cm	8ϕ16
	3rd Floor	40cm x 40cm	8ϕ16
	4th Floor	30cm x 30cm	8ϕ16
	5th Floor	30cm x 30cm	8ϕ16
G+6	Ground floor	50cm x 50cm	8ϕ24
	1st Floor	50cm x 50cm	8ϕ24
	2nd Floor	40cm x 40cm	8ϕ20
	3rd Floor	40cm x 40cm	8ϕ16
	4th Floor	40cm x 40cm	8ϕ16
	5th Floor	30cm x 30cm	8ϕ16
	6th Floor	30cm x 30cm	8ϕ16

Table 4.3 Corner Columns section and reinforcement

Building Type	Location	Column Dimension	Main Reinforcement
G+1	Ground floor	30cm x 30cm	8ϕ20
	1st Floor	30cm x 30cm	8ϕ20
G+2	Ground floor	30cm x 30cm	8ϕ20
	1st Floor	30cm x 30cm	8ϕ20
	2nd Floor	30cm x 30cm	8ϕ16
G+3	Ground floor	40cm x 40cm	8ϕ20
	1st Floor	30cm x 30cm	8ϕ20
	2nd Floor	30cm x 30cm	8ϕ20
	3rd Floor	30cm x 30cm	8ϕ16
G+4	Ground floor	40cm x 40cm	8ϕ20
	1st Floor	40cm x 40cm	8ϕ20
	2nd Floor	30cm x 30cm	8ϕ20
	3rd Floor	30cm x 30cm	8ϕ20
	4th Floor	30cm x 30cm	8ϕ16
G+5	Ground floor	50cm x 50cm	8ϕ24
	1st Floor	40cm x 40cm	8ϕ20
	2nd Floor	40cm x 40cm	8ϕ20
	3rd Floor	30cm x 30cm	8ϕ20
	4th Floor	30cm x 30cm	8ϕ20
	5th Floor	30cm x 30cm	8ϕ16
G+6	Ground floor	50cm x 50cm	8ϕ24
	1st Floor	50cm x 50cm	8ϕ24
	2nd Floor	40cm x 40cm	8ϕ20
	3rd Floor	40cm x 40cm	8ϕ20
	4th Floor	30cm x 30cm	8ϕ20
	5th Floor	30cm x 30cm	8ϕ20
	6th Floor	30cm x 30cm	8ϕ16

The Moment out put from the FEM analysis shows a very high moment values around the support and which exponentially decrease at points further from the support. However, average values are taken over a design strip as The ACI Code design procedures consider the average moments over the width of the middle and column strips. In this study average values are taken over the whole strip by taking average values of moments along a line passing through the face of the column for negative moments and values along a mid span for positive moments.

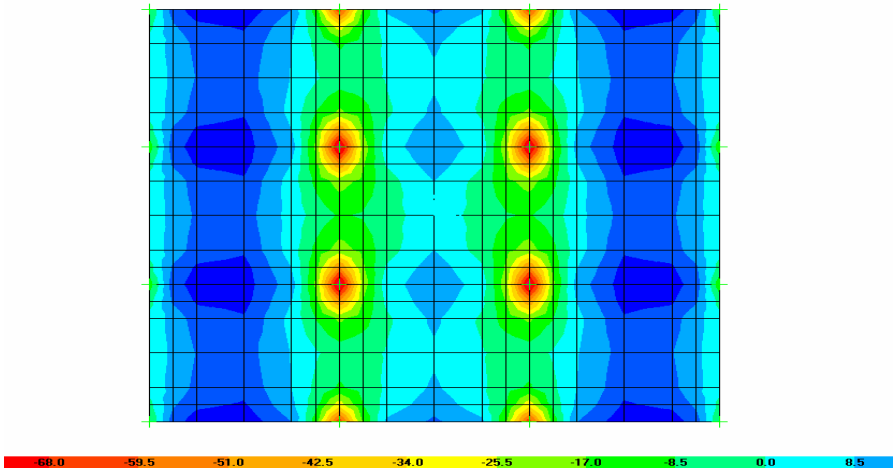


Fig 4.1 Moment in the y-direction ($M_{y\text{-}y}$) for gravity loading

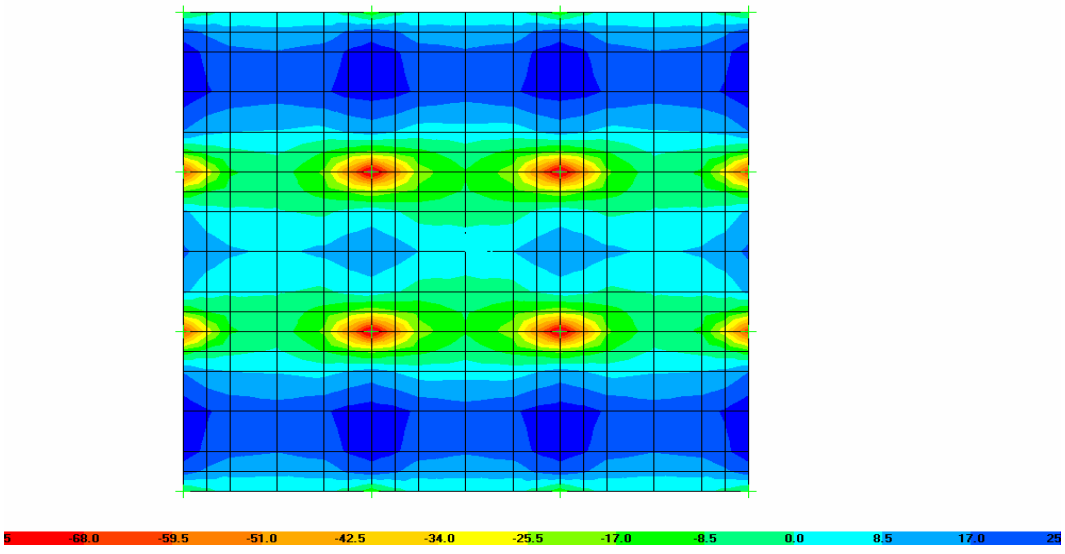


Fig 4.2 Moment in the y-direction ($M_{x\text{-}x}$) for gravity loading

Table 4.10: Slab Depth, D design output for strength requirements

	Location	D	L/D	V_g	ϕV_c	$\frac{V_g}{\phi V_c}$	Check for L/D	Check for Ductility
G+1	**Int. Connection**	17	29.4	285.17	589.98	0.483	√	x x
	Edge. Connection (Ext. Frame)	17	29.4	103.12	365.55	0.282	√	x x
	Edge. Connection (Ext. Frame)	17	29.4	103.12	365.55	0.282	√	x x
	Corner Connection	17	29.4	42.3	118.94	0.356	√	x x
G+2	**Int. Connection**	17	29.4	285.17	589.98	0.483	√	x x
	Edge. Connection (Ext. Frame)	17	29.4	103.12	365.55	0.282	√	x x
	Edge. Connection (Ext. Frame)	17	29.4	103.12	365.55	0.282	√	x x
	Corner Connection	17	29.4	42.3	118.94	0.356	√	x x
G+3	**Int. Connection**	17	29.4	285.17	589.98	0.483	√	x x
	Edge. Connection (Ext. Frame)	17	29.4	103.12	365.55	0.282	√	x x
	Edge. Connection (Ext. Frame)	17	29.4	103.12	365.55	0.282	√	x x
	Corner Connection	17	29.4	42.3	118.94	0.356	√	x x
G+4	**Int. Connection**	22	22.7	327.59	741.0	0.442	√	x x
	Edge. Connection (Ext. Frame)	22	22.7	118.34	429.32	0.276	√	x x
	Edge. Connection (Ext. Frame)	22	22.7	118.34	429.32	0.276	√	x x
	Corner Connection	22	22.7	48.59	139.14	0.349	√	x x

	Location	D	L/D	V_g	ϕV_c	$\frac{V}{\phi V_c}$	Check for L/D	Check for Ductility
G+5	***Int. Connection***	23	21.7	335.96	799.53	0.42	√	x x
	Edge. Connection (Ext. Frame)	23	21.7	121.36	455.21	0.27	√	x x
	Edge. Connection (Ext. Frame)	23	21.7	121.36	455.21	0.27	√	x x
	Corner Connection	23	21.7	49.82	147.37	0.34	√	x x
G+6	***Int. Connection***	25	20	353.26	896.07	0.399	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.72	508.5	0.251	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.62	508.5	0.251	√	√ √
	Corner Connection	25	20	52.1	164.3	0.317	√	√ √
G+7	***Int. Connection***	27	18.5	385.65	996.63	0.387	x	
	Edge. Connection (Ext. Frame)	27	18.5	140.0	563.8	0.248	x	
	Edge. Connection (Ext. Frame)	27	18.5	140.0	563.8	0.248	x	
	Corner Connection	27	18.5	57.0	181.85	0.313	x	

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Table 4.11. Slab Depth, D design output for Ductility requirements.

	Location	D	L/D	V_g	ϕV_c	$\frac{V_g}{\phi V_c}$	Check for L/D	Check for Ductility
G+1	***Int. Connection***	25	20	353.26	884.76	0.399	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.62	508.5	0.251	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.62	508.5	0.251	√	√ √
	Corner Connection	25	20	52.1	164.3	0.317	√	√ √
G+2	***Int. Connection***	25	20	353.26	884.76	0.399	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.62	508.5	0.251	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.62	508.5	0.251	√	√ √
	Corner Connection	25	20	52.1	164.3	0.317	√	√ √
G+3	***Int. Connection***	25	20	353.26	884.76	0.399	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.62	508.5	0.251	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.62	508.5	0.251	√	√ √
	Corner Connection	25	20	52.1	164.3	0.317	√	√ √
G+4	***Int. Connection***	25	20	353.26	884.76	0.399	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.62	508.5	0.251	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.62	508.5	0.251	√	√ √
	Corner Connection	25	20	52.1	164.3	0.317	√	√ √

	Location	D	L/D	V_g	V_c	$\frac{V}{\phi V_c}$	Check for L/D	Check for Ductility
G+5 and G+6	***Int. Connection***	25	20	353.26	884.76	0.399	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.72	508.5	0.251	√	√ √
	Edge. Connection (Ext. Frame)	25	20	127.62	508.5	0.251	√	√ √
	Corner Connection	25	20	52.1	164.3	0.317	√	√ √

Note: "√" Stands for 20 < L/D <30, "√ √" for $\frac{V_g}{\phi V_c} \leq 0.4$

"x" Stands for L/D < 20 and L/D > 30 , "x x " for $\frac{V_g}{\phi V_c} \geq 0.4$

Results shown on Table 4.10 and 4.11 express that the low rise (G+1 – G+3), medium span buildings designed for strength requirement required approximately 50% Additional overall slab depth increment for ductility requirement. How ever medium -rise (G + 4 – G + 6) flat slab Buildings resulted in almost equal overall depth requirement for strength and ductility.

Buildings with number of stories greater than six came up with a higher over all slab depth which brought the span to depth ratio less than 20, and this violates the ACI recommendation stated in section 3.1 of this paper.

5- CONCLUSION AND RECOMMENDATION

Regardless of their architectural flexibility flat slab structures without shear walls exhibit a joint rotation which has to be counteracted by increasing the slab depth in the absence of shear wall or other stiffening mechanism. So, the increment in slab depth tends to increase the stiffness at slab column connection and reduce the bent action. However the increment in slab depth will result in higher dead load and increased overall building height which will result in higher seismic force on the structure.

According to this thesis FEM gives accurate results as compared to DDM and EFM, how ever, flat slab frames should be properly modeled along with appropriate stiffness reductions due to cracking, creep and shrinkage.

Medium Spanned flat slab buildings analyzed and designed in this study resulted in somehow reasonable overall depth for low rise (G+1 – G+3) buildings and reasonable overall depth for medium-rise (G + 4 – G + 6) Buildings. How ever buildings with number of stories greater than six came up with a higher over all slab depth which brought the span to depth ratio less than 20, and violate the ACI recommendation stated in section 3.1 of this paper. The other observation obtained from this study is that the low rise, medium span buildings designed for strength requirement required approximately 50% slab depth increment for ductility requirement. And since drift requirement for a flat slab design is a function of direct gravity load, short span low rise flat slab buildings will be more effective in terms of economy.

Finally, Low to Medium rise flat slab structures (G+1 - G+6) , medium spanned and located in seismic zone 2, which are designed and analyzed in this study found to effectively resist seismic hazards. How ever, high-rise buildings (buildings with

number of stories greater than six) required larger depth, which will significantly increase building height, weight and seismic effect on the structure. So, high-rise, medium spanned flat slab buildings, constructed in seismic zone II need to be stiffened with shear walls or other stiffening mechanisms.

6-RECOMMENDATION FOR FUTURE RESEARCH

The work of this thesis has been limited to reinforced concrete flat-slab structural frame systems without out shear wall or any joint stiffening mechanism. However flat-slab structural frame systems can be constructed with stiffening mechanisms such as drop-panels, perimeter beams and with provisions of shear reinforcement. So, some of the future research can be listed as follows,

1- Assessment of flat slab construction with out shear walls with respect to the effectiveness against lateral load resisting capacity by taking column dimension as the a governing parameter for joint stability.

2- Assessment of ribbed slab construction with out shear walls with respect to the effectiveness against lateral load resisting capacity.

3- Assessment of flat plate construction with shear reinforcement but lacking shear walls with respect to the effectiveness against lateral load resisting capacity.

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DECLARATION

I, the undersigned, declare that this thesis is my original work and has not been presented for a degree in any other university. All sources of materials used for the thesis have been duly acknowledged.

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DEPARTMENT OF CIVIL ENGINEERING

DATE: AUGUST, 2009

Flat Slab Desigh Summery

f_{yk} = 300,000 f_{yd} = 260,869
d= 0.241 f_{cd} = 11,333

$$\rho = \frac{1}{m}\left[1 - \sqrt{1 - \frac{2mR_n}{f_{yd}}}\right]$$

$$m = \frac{f_{yd}}{f_{cd}} \qquad R_n = \frac{Mu}{bd^2}$$

ρ_{min} 0.001667 [EBCS 2 7.2.2.2]

Table 4.4 Slab reinforcement Summery for G+1 Building

Corner Pannel	Column Strip															Middle Strip		
	Int.frame							Ext. frame										
	Exterior negative			Positive	Interior negative			Exterior negative			Positive	Interior negative			Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L	L	L
Width	0.625	0	0.68	1.25	0.63	0	0.68	0.63	0	0.68	1.25	0.63	0.05	0.68	1.00	1.00	1.00
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667	0.001667	0.00167	0.00166667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.01853
Mu	19.6887	1.324296	1.41075	42.8	19.698786	1.011503	3.17475	25.7409	1.88047	1.00575	26.83	29.33022	2.062017	1.59975	1.00	22.60	10.76
Rn	542.379	456.0168	35.98423	589.52153	542.65694	348.3077	80.9789	709.103	647.533	25.6538	369.55287	807.9811	710.0489	40.80508	17.21733	389.112	185.258518
ρ	0.002131	0.001785	0.000138	0.0023219	0.0021325	0.001356	0.00031	0.00281	0.00256	9.8E-05	0.0014405	0.003216	0.002813	0.000157	6.61E-05	0.00152	0.00071606
As,calc	321.0435	21.50597	271.125	699.46805	321.21233	20.08333	271.125	423.113	30.8178	271.125	502.08333	484.4594	33.89578	271.125	401.6667	401.667	401.666667
dia	14	12	12	12	14	12	12	14	12	12	12	14	12	12	10	10	10
# of bars	2	0	2	6	2	0	2	3	0	2	4	3	0	2	5	5	5
Spacing	250	-	250	200	250	-	250	200	-	250	250	150	-	250	150	150	150

Edge Pannel	Column Strip											Middle Strip		
	Int.frame							Ext. frame						
	Exterior negative			Positive	Interior negative			Negative			Positive	Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L	L	L
Width	0.63	0	0.68	1.25	0.63	0	0.68	0.63	0	0.68	1.25	2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853
Mu	18.27536	1.296029	0.93375	12.90	20.30612	1.02365	3.37973	0.66667	0.01333	0.225	10.51	2.85	23.13	18.09
Rn	503.4448	446.2833	23.81731	177.68289	559.3876	352.4903	86.2072	18.3652	4.59129	5.73911	144.76335	19.62776	159.2948	124.5846
ρ	0.001975	0.001746	9.14E-05	0.0006865	0.0022	0.001373	0.00033	7E-05	1.8E-05	2.2E-05	0.0005585	7.53E-05	0.000615	0.00048
As,calc	297.448	21.03732	271.125	502.08333	331.37954	20.08333	271.125	251.042	20.0833	271.125	502.08333	1004.167	1004.167	1004.167
dia	12	12	12	12	12	12	12	12	12	12	12	12	12	12
# of bars	3	0	2	4	3	0	2	2	0	2	4	9	9	9
Spacing	200	-	250	250	200	-	250	250	-	250	250	250	250	250

Interior Pannel	Column Strip				Middle Strip	
	Negative			Positive	Negative	Positive
Span	L/2	L' - L/2	L-L'	L	L	L
Width	0.625	0	0.68	1.25	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853
Mu	22.75879	1.072703	4.2075	10.61	17.96	5.50
Rn	626.953	369.3817	107.3214	146.14073	123.68933	37.87814
ρ	0.002474	0.00144	0.000413	0.0005639	0.0004768	0.000145
As,calc	372.6094	17.3499	67.24434	169.86484	287.24761	87.62959
dia	12	12	12	12	10	10
# of bars	3	0	1	2	4	1
Spacing	150	-	350	350	350	350

Note *L* - **Strip Width**
L' - **Flexural moment transfer width**

Table 4.5 Slab reinforcement Summery for G+2 Building

Corner Pannel	Column Strip															Middle Strip		
	Int.frame							Ext. frame										
	Exterior negative			Positive	Interior negative			Exterior negative			Positive	Interior negative				Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'		L	L	L
Width	0.625	0	0.68	1.25	0.63	0	0.68	0.63	0	0.68	1.25	0.63	0.05	0.68		2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667		0.001667	0.00167	0.00166667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853		23.01853	23.0185	23.01853
Mu	19.6887	1.324296	1.41075	42.8	19.698786	1.011503	3.17475	25.7409	1.88047	1.00575	26.83	29.33022	2.062017	1.59975		1.00	22.60	10.76
Rn	542.379	456.0168	35.98423	589.52153	542.65694	348.3077	80.9789	709.103	647.533	25.6538	369.55287	807.9811	710.0489	40.80508		6.886934	155.645	74.1034073
ρ	0.002131	0.001785	0.000138	0.0023219	0.0021325	0.001356	0.00031	0.00281	0.00256	9.8E-05	0.0014405	0.003216	0.002813	0.000157		2.64E-05	0.0006	0.000285
As,calc	321.0435	21.50597	271.125	699.46805	321.21233	20.08333	271.125	423.113	30.8178	271.125	502.08333	484.4594	33.89578	271.125		1004.167	1004.17	1004.16667
dia	14	12	12	12	14	12	12	14	12	12	12	14	12	12		10	10	10
# of bars	2	0	2	6	2	0	2	3	0	2	4	3	0	2		13	13	13
Spacing	250	-	250	200	250	-	250	200	-	250	250	150	-	250		150	150	150

Edge Pannel	Column Strip											Middle Strip		
	Int.frame							Ext. frame						
	Exterior negative			Positive	Interior negative			Negative			Positive	Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L	L	L
Width	0.63	0	0.68	1.25	0.63	0	0.68	0.63	0	0.68	1.25	2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853
Mu	18.27536	1.296029	0.93375	12.90	20.30612	1.02365	3.37973	0.66667	0.01333	0.225	10.51	2.85	23.13	18.09
Rn	503.4448	446.2833	23.81731	177.68289	559.3876	352.4903	86.2072	18.3652	4.59129	5.73911	144.76335	19.62776	159.2948	124.5846
ρ	0.001975	0.001746	9.14E-05	0.0006865	0.0022	0.001373	0.00033	7E-05	1.8E-05	2.2E-05	0.0005585	7.53E-05	0.000615	0.00048
As,calc	297.448	21.03732	271.125	502.08333	331.37954	20.08333	271.125	251.042	20.0833	271.125	502.08333	1004.167	1004.167	1004.167
dia	12	12	12	12	12	12	12	12	12	12	12	12	12	12
# of bars	3	0	2	4	3	0	2	2	0	2	4	9	9	9
Spacing	200	-	250	250	200	-	250	250	-	250	250	250	250	250

Interior Pannel	Column Strip				Middle Strip	
	Negative			Positive	Negative	Positive
Span	L/2	L' - L/2	L-L'	L	L	L
Width	0.625	0	0.68	1.25	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853
Mu	22.75879	1.072703	4.2075	10.61	17.96	5.50
Rn	626.953	369.3817	107.3214	146.14073	123.68933	37.87814
ρ	0.002474	0.00144	0.000413	0.0005639	0.0004768	0.000145
As,calc	372.6094	17.3499	67.24434	169.86484	287.24761	87.62959
dia	12	12	12	12	10	10
# of bars	3	0	1	2	4	1
Spacing	150	-	350	350	350	350

Note *L* - **Strip Width**

L' - **Flexural moment transfer width**

Table 4.6 Slab reinforcement Summery for G+3 Building

Corner Pannel	Column Strip															Middle Strip		
	Int.frame							Ext. frame										
	Exterior negative			Positive	Interior negative			Exterior negative			Positive	Interior negative				Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'		L	L	L
Width (m)	0.625	0.14	0.76	1.25	0.63	0.14	0.76	0.63	0.14	0.76	1.25	0.63	0.14	0.76		2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667		0.001667	0.00167	0.00166667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853		23.01853	23.0185	23.01853
Mu	34.06846	6.805362	1.593625	54.775	30.123013	5.074963	3.58629	42.6368	10.5045	1.17338	26.83	21.73054	4.725641	1.866375		1.00	22.60	10.76
Rn	1978.137	1796.108	75.84555	1590.2163	1749.05	1339.411	170.683	2475.64	2772.39	55.8446	778.92292	1261.753	1247.216	88.82657		14.51589	328.059	156.191029
ρ	0.008394	0.007539	0.000292	0.0065967	0.0073217	0.00548	0.00066	0.01084	0.0124	0.00021	0.0030962	0.005141	0.005078	0.000342		5.57E-05	0.00128	0.00060292
As,calc	870.8528	172.0843	210.9583	1368.8115	759.62469	125.0823	210.958	1124.98	282.94	210.958	642.46389	533.3692	115.8998	210.9583		691.6667	691.667	691.666667
dia	16	10	12	12	12	10	12	14	12	12	12	12	10	12		12	12	12
# of bars	4	0	2	12	7	0	2	7	3	2	6	5	0	2		6	6	6
Spacing (mm)	100	-	350	100	50	-	350	50	-	350	200	100	-	350		350	350	350

Edge Pannel	Column Strip											Middle Strip		
	Int.frame							Ext. frame						
	Exterior negative			Positive	Interior negative			Negative			Positive	Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L	L	L
Width (m)	0.63	0.14	0.76	1.25	0.63	0.14	0.76	0.63	0.14	0.76	1.25	2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853
Mu	32.65513	6.727628	1.054792	12.90	30.730346	5.108366	3.81784	0.66667	0.04333	0.2625	10.51	2.85	23.13	18.09
Rn	1896.074	1775.592	50.2008	374.51009	1784.3139	1348.227	181.703	38.7091	11.4368	12.4932	305.12411	41.3703	335.7526	262.5925
ρ	0.008006	0.007444	0.000193	0.0014602	0.0074846	0.005519	0.0007	0.00015	4.4E-05	4.8E-05	0.0011858	0.000159	0.001307	0.001019
As,calc	830.622	169.9152	210.9583	345.83333	776.53039	125.9654	210.958	172.917	38.0417	210.958	345.83333	691.6667	691.6667	691.6667
dia	16	10	20	12	12	10	12	12	10	12	12	10	10	10
# of bars	4	0	1	3	7	0	2	2	0	2	3	9	9	9
Spacing (mm)	150	-	350	350	50	-	350	350	-	350	350	250	250	250

Interior Pannel	Column Strip				Middle Strip	
	Negative			Positive	Negative	Positive
Span	L/2	L' - L/2	L-L'	L	L	L
Width (m)	0.625	0.14	0.76	1.25	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853
Mu	33.18301	-5.24326	4.752917	10.61	17.96	5.50
Rn	3082.759	-10064.2	296.6636	246.42183	104.28219	31.93497
ρ	0.014108	-0.02894	0.001153	0.0009551	0.0004016	0.000123
As,calc	1463.709	38.04167	210.9583	345.83333	691.66667	691.6667
dia	16	16	12	12	10	10
# of bars	12	0	2	3	9	9
Spacing (mm)	50	-	350	350	250	250

Note ***L*** **- Strip Width**
L' **- Flexural moment transfer width**

Table 4.7 Slab reinforcement Summery for G+4 Building

Corner Pannel	Column Strip															Middle Strip		
	Int.frame							Ext. frame										
	Exterior negative			Positive	Interior negative			Exterior negative			Positive	Interior negative				Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'		L	L	L
Width	0.625	0.03	0.65	1.25	0.63	0.03	0.65	0.63	0.03	0.65	1.25	0.63	0.03	0.65		2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667		0.001667	0.00167	0.00166667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853		23.01853	23.0185	23.01853
Mu	33.20208	1.216083	1.213333	63.07	48.042135	1.112085	8.77067	38.5235	2.90308	1.00575	26.83	34.03348	1.652278	6.021		1.00	22.60	6.24
Rn	1463.428	201.8396	7.745505	209.361	318.95194	184.5785	55.9889	255.758	481.839	6.42036	89.062241	225.9484	274.2371	38.436		1.659751	37.5104	10.3568465
ρ	0.006028	0.000781	2.97E-05	0.0008101	0.0012404	0.000713	0.00022	0.00099	0.00189	2.5E-05	0.0003428	0.000875	0.001064	0.000148		6.36E-06	0.00014	3.9719E-05
$As_{,calc}$	907.9723	10.04167	261.0833	502.08333	251.04167	10.04167	261.083	251.042	11.3757	261.083	502.08333	251.0417	10.04167	261.0833		1004.167	1004.17	1004.16667
dia	14	12	12	12	12	12	12	12	12	12	12	12	12	12		12	12	12
# of bars	6	0	2	4	2	0	2	2	0	2	4	2	0	2		9	9	9
Spacing	100	-	250	250	250	-	250	250	-	250	250	250	-	250		250	250	250

Edge Pannel	Column Strip											Middle Strip		
	Int.frame							Ext. frame						
	Exterior negative			Positive	Interior negative			Negative			Positive	Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L	L	L
Width	0.63	0.03	0.65	1.25	0.63	0.03	0.65	0.63	0.03	0.65	1.25	2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853
Mu	37.20208	1.256083	2.513333	8.00	48.668802	1.118352	8.97433	0.66667	0.01333	0.225	12.91	1.00	23.13	14.00
Rn	1024.833	865.0563	66.57369	110.19094	1340.7153	770.2017	237.714	18.3652	9.18258	5.95985	177.82063	6.886934	159.2948	96.41707
ρ	0.004124	0.003453	0.000256	0.0004245	0.0054858	0.00306	0.00092	7E-05	3.5E-05	2.3E-05	0.0006871	2.64E-05	0.000615	0.000371
$As_{,calc}$	621.2238	20.80618	261.0833	502.08333	826.29509	18.43789	261.083	251.042	10.0417	261.083	502.08333	1004.167	1004.167	1004.167
dia	14	12	12	12	14	12	12	12	12	12	12	12	12	12
# of bars	4	0	2	4	5	0	2	2	0	2	4	9	9	9
Spacing	150	-	250	250	100	-	250	250	-	250	250	250	250	250

Interior Pannel	Column Strip				Middle Strip	
	Negative			Positive	Negative	Positive
Span	L/2	L' - L/2	L-L'	L	L	L
Width(m)	0.625	0.03	0.65	1.25	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853
Mu	48.6688	1.118352	8.974333	9.50	14.96	5.50
Rn	1340.715	770.2017	237.714	130.85174	103.02853	37.87814
ρ	0.005486	0.00306	0.000921	0.0005045	0.0003968	0.000145
$As_{,calc}$	826.2951	18.43789	261.0833	502.08333	1004.1667	1004.167
dia	14	12	12	12	12	12
# of bars	5	0	2	4	9	9
Spacing (mm)	100	-	250	250	250	250

Note ***L*** **- Strip Width**

L' **- Flexural moment transfer width**

Table 4.8 Slab reinforcement Summery for G+5 Building

Corner Pannel	Column Strip															Middle Strip		
	Int.frame							Ext. frame										
	Exterior negative			Positive	Interior negative			Exterior negative			Positive	Interior negative				Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'		L	L	L
Width (m)	0.625	0.03	0.66	1.25	0.63	0.03	0.66	0.63	0.03	0.66	1.25	0.63	0.03	0.66		2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667		0.001667	0.00167	0.00166667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853		23.01853	23.0185	23.01853
Mu	34.93945	1.526614	1.36895	64.725	33.544534	1.271498	3.08068	51.324	6.2834	1.05045	26.83	33.47739	3.830065	1.67085		1.00	22.60	10.76
Rn	1831.351	230.2585	9.457014	234.29864	242.85636	191.7794	21.2821	371.577	947.722	7.25674	97.122172	242.3702	577.6871	11.54261		1.809955	40.905	19.4751131
ρ	0.007703	0.000892	3.63E-05	0.0009076	0.0009411	0.000741	8.2E-05	0.00145	0.0038	2.8E-05	0.0003739	0.000939	0.002274	4.43E-05		6.94E-06	0.00016	7.4719E-05
$As_{, calc}$	1063.995	11.05	241.2583	460.41667	230.20833	11.05	241.258	230.208	25.1877	241.258	460.41667	230.2083	15.07653	241.2583		920.8333	920.833	920.833333
dia	16	12	12	12	12	12	12	12	12	12	12	12	12	12		12	12	12
# of bars	5	0	2	4	2	0	2	2	0	2	4	2	0	2		8	8	8
Spacing (mm)	100	-	300	300	300	-	300	300	-	300	300	300	-	300		300	300	300

Edge Pannel	Column Strip											Middle Strip		
	Int.frame							Ext. frame						
	Exterior negative			Positive	Interior negative			Negative			Positive	Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L	L	L
Width (m)	0.63	0.03	0.66	1.25	0.63	0.03	0.66	0.63	0.03	0.66	1.25	2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853
Mu	33.53	1.509654	0.906083	12.90	34.151868	1.278786	3.27959	0.66667	0.02133	0.235	10.51	2.85	23.13	18.09
Rn	1098.294	1030.319	28.3232	211.29788	1118.7934	872.7542	102.516	21.8396	14.5597	7.34585	172.15045	23.34105	189.431	148.1542
ρ	0.004437	0.004148	0.000109	0.0008177	0.0045243	0.003485	0.00039	8.4E-05	5.6E-05	2.8E-05	0.000665	8.96E-05	0.000732	0.000572
$As_{, calc}$	612.8175	27.49824	241.2583	460.41667	624.91952	23.10806	241.258	230.208	11.05	241.258	460.41667	920.8333	920.8333	920.8333
dia	14	12	12	12	14	14	14	14	14	14	14	12	12	12
# of bars	4	0	2	4	4	0	2	1	0	2	3	8	8	8
Spacing (mm)	150	-	300	300	150	-	350	350	-	350	350	300	300	300

Interior Pannel	Column Strip				Middle Strip	
	Negative			Positive	Negative	Positive
Span	L/2	L' - L/2	L-L'	L	L	L
Width (m)	0.625	0.03	0.66	1.25	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853
Mu	36.60453	1.308218	4.082833	10.61	17.96	5.50
Rn	1199.141	892.8412	127.625	173.78842	147.08954	45.04412
ρ	0.00487	0.003569	0.000492	0.0006714	0.0005676	0.000173
$As_{, calc}$	672.6193	23.66368	241.2583	460.41667	920.83333	920.8333
dia	14	12	12	12	12	12
# of bars	4.369416	0	2.133192	4.0709771	8.1419543	8.141954
Spacing (mm)	100	-	300	300	300	300

Note ***L*** **- Strip Width**

L' **- Flexural moment transfer width**

Table 4.9 Slab reinforcement Summery for G+6 Building

Corner Pannel	Column Strip															Middle Strip		
	Int.frame							Ext. frame										
	Exterior negative			Positive	Interior negative			Exterior negative			Positive	Interior negative				Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'		L	L	L
Width (m)	0.625	0.00	0.63	1.25	0.63	0.00	0.63	0.63	0.00	0.63	1.25	0.63	0.00	0.63		2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667		0.001667	0.00167	0.00166667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853		23.01853	23.0185	23.01853
Mu	36.322	0	1.30625	72.59	40.350917	0	2.93958	38.3148	2.88638	1.00575	26.83	42.27261	3.097409	1.59975		1.00	22.60	10.76
Rn	1000.589	0	35.98423	999.84504	1111.5764	0	80.9789	1055.49	0	27.7061	369.55287	1164.515	0	44.06949		6.886934	155.645	74.1034073
ρ	0.004022	0	0.000138	0.0040186	0.0044934	0	0.00031	0.00425	0	0.00011	0.0014405	0.00472	0	0.000169		2.64E-05	0.0006	0.000285
$As_{, calc}$	605.7769	0	251.0417	1210.6074	676.82365	0	251.042	640.811	0	251.042	502.08333	711.016	0	251.0417		1004.167	1004.17	1004.16667
dia	14	14	14	14	14	14	14	14	14	14	14	14	14	14		14	14	14
# of bars	4	0	2	8	4	0	2	4	0	2	3	5	0	2		7	7	7
Spacing (mm)	150	-	350	150	100	-	350	150	-	350	350	100	-	350		350	350	350

Edge Pannel	Column Strip											Middle Strip		
	Int.frame							Ext. frame						
	Exterior negative			Positive	Interior negative			Negative			Positive	Exterior negative	Positive	Interior negative
Span	L/2	L' - L/2	L-L'	L	L/2	L' - L/2	L-L'	L/2	L' - L/2	L-L'	L	L	L	L
Width (m)	0.63	0.00	0.63	1.25	0.63	0.00	0.63	0.63	0.00	0.63	1.25	2.50	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667	0.00167	0.00167	0.00167	0.00167	0.0016667	0.001667	0.001667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853	23.0185	23.0185	23.0185	23.0185	23.01853	23.01853	23.01853	23.01853
Mu	34.90867	0	0.864583	12.90	40.95825	0	3.12938	0.66667	0.01333	0.225	10.51	2.85	23.13	18.09
Rn	1538.648	0	38.1077	142.14631	1805.2912	0	137.932	29.3843	0	9.91718	115.81068	7.851104	63.71791	49.83385
ρ	0.006364	0	0.000146	0.0005484	0.0075819	0	0.00053	0.00011	0	3.8E-05	0.0004462	3.01E-05	0.000245	0.000191
$As_{, calc}$	958.6291	0	251.0417	502.08333	1142.0256	0	251.042	251.042	0	251.042	502.08333	1004.167	1004.167	1004.167
dia	16	16	16	16	20	16	16	16	16	16	16	16	16	16
# of bars	6	1	2	3	5	1	2	2	1	2	3	6	6	6
Spacing (mm)	100	-	250	350	100	-	250	250	-	250	350	350	350	350

Interior Pannel	Column Strip				Middle Strip	
	Negative			Positive	Negative	Positive
Span	L/2	L' - L/2	L-L'	L	L	L
Width (m)	0.625	0.00	0.625	1.25	2.50	2.50
ρ_{min}	0.001667	0.001667	0.001667	0.0016667	0.0016667	0.001667
m	23.01853	23.01853	23.01853	23.01853	23.01853	23.01853
Mu	43.41	0	3.895833	10.61	17.96	5.50
Rn	1913.396	0	171.7142	116.91259	49.475732	15.15125
ρ	0.008087	0	0.000663	0.0004505	0.0001901	5.81E-05
$As_{, calc}$	1218.179	0	99.91001	135.71364	114.5191	35.01658
dia	20	20	12	12	12	8
# of bars	4	0	1	1	1	1
Spacing (mm)	150	-	350	350	350	350

Note ***L*** **- Strip Width**

L' **- Flexural moment transfer width**