ADDIS ABABA UNIVERISTY

SCHOOL OF GRADUATE STUDIES

HOW HIGH COULD BUILDINGS MADE OF RIBBED OR FLAT SLAB CONSTRUCTION WITH OUT SHEAR WALLS BE BUILT? (CASE STUDY)

By Mohammed Arega Ali

December 2006

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A thesis submitted to the school of graduate studies of Addis Ababa University in partial fulfillment of the Requirements for the Degree of Masters in Civil Engineering

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DESCRIPTION OF SYMBOLS

Symbols	Descriptions
bo	The column width in the direction of interest and
C	Torsional stiffness of the torsional restraint element
C2	Dimension of the specific support perpendicular to the equivalent frame
Ec	Modules of elasticity of concrete
E_s	The modules of elasticity of steel
Ft	Concentrated force at top
GFRS	gravity force- resisting system
h	The slab depth
H	The total horizontal reaction at the bottom of the story
h	The story height;
H	Height of the building
hs	The slab depth.
H_u	The total factored lateral force acting within the story
I	Importance factor
I_c,	The moments of inertia of the concrete sections.
I_s	The moments of inertia of reinforcement.
Kc	The gross support stiffness
KD	Ductility class
KR	Regularity in elevation
Kt	The stiffness of the torsional restraint elements
KW	Prevailing failure mode
L	The story height
l2	Transverse dimension of the panel adjacent to the specific support
LFRS	lateral-force –resisting system
N	The total vertical reaction at the bottom of the story.

N_{cr}	critical value for failure in a sway mode
N_{sd}	The design value of the total vertical load
P_X	Total dead, floor live
T1	Fundamental period of vibration of the structure for translational motion
V_u	Shear in the story due to wind or earth quake load acting on the frame above the story in question
V_x	the story shears;
W	Seismic dead load computed
αo	Bed rock acceleration
γ	Behavior factor
γo	Basic value of behavior factor
Δ	The story drift;
σ	The horizontal displacement at the top of the story; relative to the bottom of the story
Δo	First order deflection due to the story shear.
Δ_u	The elastically -computed 1st order lateral deflection due to H_u (neglecting P-Δ effect) at the top of story relative to bottom of the story.
βd	Creep, $\frac{Factored\ dead\ loadin\ the\ story}{total\ \ factored\ load\ in\ the\ story}$
SPu	the total axial load in all the columns in the story
θ	Moment ratio
Ig	Gross moment of inertia of a section
ω	Ratio of $\sum\left(\frac{EI}{lc}\right)$ of compression member to $\sum\left(\frac{EI}{lc}\right)$ of flexural members in a plane of one end of a compression member.

ABSTRACT

The main objective of this thesis was to study the stability of slab-column, specifically ribbed slab-column, for high rising building frame systems with out lateral load resisting structural element. The study also tried to answer the question how the stability of ribbed slab column frames without lateral load resisting structural element determines the limit of the story number which can be built using ribbed slab column frame system in Addis Ababa and other seismic areas in Ethiopia.

Computer aided analysis based on 3-dimensional model consisting reduced column and ribbed beam stiffness for case of ACI and unreduced stiffness for case of EBCS was used in the determination of stability index of a story. This can be applied for various categories of building frame and structural element sizes to carry out the study of stability.

The selection of a building configuration is one of the most important aspects of the overall design; it is influenced primarily by the intended function, architectural considerations, internal traffic flow, building height, aspect ratio and to a lesser extent the intensity of loading. In this study, buildings are categorized as commercial (shops and market complex), office and apartment buildings. The arrangement or grid spacing for structural element is compatible with the international standards given from Time-Savers standards for building types and Neufert (architects' data).

The study showed that the stability index or moment ratio of a frame system should not be greater than 0.25 according to EBCS-8 1995 and 0.3 for ACI building code standard, but the building systems which were described in this thesis work have a value which is much greater than this value for eight story and above .

The serviceability limit of a story is not satisfied in most analyzed ribbed slab frame systems. Therefore, the ribbed slab column frame systems are designed for gravity load, and for deformation compatibility lateral force resisting structural element should be

used to ensure that the system can maintain the required serviceability and ultimate limit state specified on EBCS and ACI code of practice.

1. INTRODUCTION

1.1 Background and identification of the problem

Nowadays, one of the most common floor systems for mid-rise to high-rise building construction in Addis Ababa and other regional cities is ribbed slab system. Ribbed slab construction possesses some advantages over conventional slab-beam-column construction. This type of construction has architectural flexibility, more clear space, less building height, lack of sharp corners and affords simple construction by using standard modular and reusable formwork.

However, the transfer of the shear and bending forces between the slab and the columns could result in a punching shear failure and partial collapse. As the slab cracks, excessive story drift may result in instability of the frame due to the p-Δ effects [16]. The conventional available hollow concrete block size results its limitation on the concrete slab thickness. This slab column frame system is also susceptible to significant reduction in stiffness as a consequence of slab cracking that arises from construction loads, service gravity load, lateral loads, etc. Even the punching failure is prevented by introducing adequate reinforcement at slab column connection.

The serious problem in this slab-column frame system is its low lateral stiffness that results in a higher lateral drift, which is defined as the relative magnitude of the lateral displacement of the top of a building with respect to its height [15]. This can make the system vulnerable to severe damage during earth quake of moderate intensity. The instability problem that arises in such type of frame system can be prevented by providing adequate lateral load resisting structural elements.

> Macgregor (1997) has pointed out that *"A frame consisting of columns either in flat plates or slab with drop panels, which does not have shear walls or other bracing elements, is inefficient in resisting lateral loads and is subjected to significant lateral drift deflections. These deflections are amplified by the p-delta moment resulting from gravity loads. As a result, flat plat structures generally are braced by shear walls.* [15]*"*

In general, as the height of a building increases, a point is reached beyond which the lateral sways under lateral load, and hence consideration of stiffness not strength, will govern the design of structural system [20]. The point at which this condition is reached depends on the type of structural system. Ideally, if a system does not require significant change in the sizes of the structural members, we need to have lateral load resisting element in order to support the design's vertical and lateral loads.
Nowadays design engineers use this type of slab-column frame system with out additional lateral load resisting system for mid-rise to high-rise buildings.
In this thesis work the researcher would like to show the clear boundary of the story height number that can be built without lateral load resisting structural element.
The ranges of applicability shown may vary some what depending upon the use of the building, the story heights, the design live and earth quake zoning.

1.2 Aim and scope

There are different types of flat slab and ribbed slab structural systems. The researcher's intention in this thesis work is to carry out a detailed study on the stability of column for different stories which can support reinforced flat or ribbed slab building system. In addition to this, the buckling analysis of braced and unbraced columns supporting flat or ribbed slab building may be generalized to a similar framing system in which a portion of flat slab stiffness is replaced by the stiffness of horizontal equivalent beam. It requires analytical analysis in order to decide which portion of the flat slab can be counted to substitute the stiffness of replaced beam grid (equivalent slab). But in this thesis work, the effective slab width can be determined from the different code recommendations and the already designed case ribbed slab building structures. The designed buildings are located in moderate earth quake area (zone 2) to severe earth quake area (zone 4) according to EBCS-8 categorizations. The regularity of the buildings in terms of mass and stiffness of members in both plan and elevation in this study enables the utilization of equivalent frame analysis in the assessment of seismic response.

1.3 Different types of flat slab and ribbed slab system

Slabs are structures that transmit loads normal to their plane. Concrete slabs are widely in use as floors not only in industrial and residential buildings but also as decks in bridges. The big advantage is flexibility in method of manufacturing. They can be made in-situ as well as prefabricated and brought to construction site.

There are four basic stages in the behavior of a slab loaded to failure [15]:

1) Before cracking, the slab acts as elastic plate and, for short time load the deformation, stresses and strains can be predicted for an elastic analysis.
2) After cracking and before yielding of the reinforcement, the slab is no longer of constant stiffness, since the cracked regions and the slab are no longer isotropic and the crack pattern may differ in the two directions.
3) Yielding of the reinforcement eventually starts in one or more regions of high moment and spreads through the slab as moments are redistributed from yielded regions to areas that are still elastic.
4) Although the yield lines divide the plate to form a plastic mechanism, the hinges jam with increased deflection and the slab forms a very flat compression arch. This assumes that the surrounding structure is stiff enough to provide reactions for the arch. This stage of behavior is not counted on in design at present.

Some of the flat and ribbed slab systems are

1.3.1 Solid flat slabs with and with out drops[2]

Solid flat slabs are supported only by columns and are applied mostly in large industrial and public buildings. For lower loads they can have constant thickness and they are called flat slab or flat plates. To increase punching resistance, flat slabs with drops and column heads are applied. The advantage of flat slab is its relatively simple structure and short time of construction even with an in-situ method.

The provision of drop panels at the column supports avoids the need for shear reinforcement and increases the stiffness of the slab and the economical span range. Alternatively, a structural steel shear head can be incorporated to maintain a flush soffit to allow for easy construction and efficient use of large forming systems.

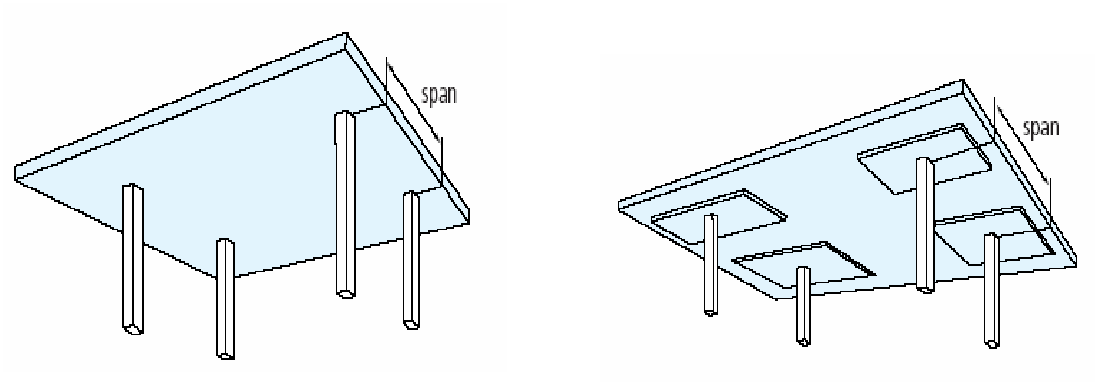


Figure 1.1 Flat slab systems without drop and with drop [16]

1.3.2 Waffle slabs

Waffle slabs have the shape of a set of crossing joists with a thin plate on the upper side. The meaning of this system is to increase the effective depth keeping a relatively low self weight of the structure. Such slabs can be designed to work either as flat slab or two way slabs and are applied when long span, up to 10m, are needed.

Waffle slab floors are commonly used when buildings are subjected to heavy imposed loading. They are very efficient in the use of materials and provide very economical long spans. But the additional complexity of formwork can often slow the construction. For large spans, the thickness required to transmit the vertical loads to the column exceeds the thickness required for bending. As a result the concrete at the middle of the panel is not efficiently used.

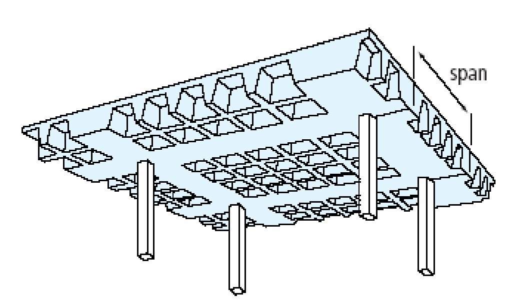


Figure 1.2 Waffle slab system [16]

1.3.3 Ribbed slab with and with out HCB

Ribbed slabs are in-situ concrete ribs with hollow blocks or voids. These type of structural plate systems can minimize formwork complexity by using standard modular, reusable formwork. When flying form panels are used, the ribs should be positioned away from the column lines. Ribbed slab floors are very adaptable for accommodating a range of service openings. This type of ribbed slab constructions is frequently built in Addis Ababa and other regions. According to EBCS, ribs shall not be less than 70mm in width; and shall have a depth, excluding any topping of not more than 4 times the minimum width of ribs. The rib spacing shall not exceed 1.0m. If the rib spacing exceeds 1.0m, the topping shall be designed as slab resting on the ribs considering load concentration, if any. The thickness of topping shall not be less than 40mm, nor less than $\frac{1}{10}$ of the clear distance between the ribs. If the span of the ribbed slab exceeds 6.0m, transverse ribs shall be provided. When these ribs are provided, the center to center distance shall not exceed 20 times the over all depth of the ribbed slab. The transverse ribs shall be designed for at least half of the value of maximum moments and shear force in the longitudinal ribs.

In general, ribbed slab structural systems can categorize the slab as [2]

1) Structural topping contributes to structural strength
2) Non-structural topping, where topping does not contribute to the structural strength. In addition to this, the slabs can be divided as slabs with permanent blocks and those with out permanent blocks for the purpose of thickness of topping.

Ribbed slab system can be classified as [2]:

a. **Slabs with permanent blocks**

i. **Structural topping and structural -type blocks**

The clear distance between ribs should not be more than 500mm. The width of the rib will be determined by consideration of cover, bar spacing and fire requirements. But the depth of the rib excluding the topping should not exceed four times the width. If the blocks are suitably manufactured and have adequate strength they can be considered to contribute to the strength of the slab in the design calculations, but in many designs no such allowance is made. These permanent blocks which are capable of contributing to the structural strength if it can be jointed with cement -sand mortar. During construction the hollow tiles should be well soaked in water prior to placing the concrete, otherwise shrinkage cracking of the top concrete flange is liable to occur. This probably develops strength for topping.

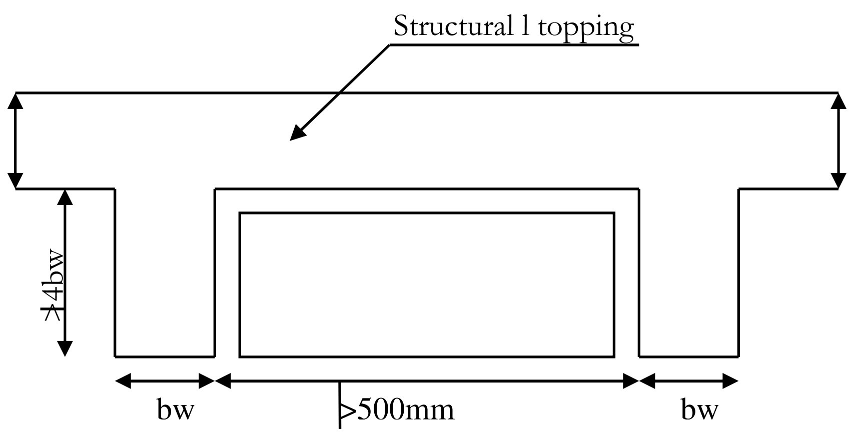


Figure 1.3 permanent blocks contributing to structural strength [2]

ii. Structural topping and non structural type blocks

In this case the spacing of the ribs can be increased above 500mm clear but the center of the ribs must not exceed 1500mm. As in (i) the depth of the rib should not exceed four times its width. The minimum thickness of topping should be the greater of 40mm or one –tenth of the clear distance between the ribs.

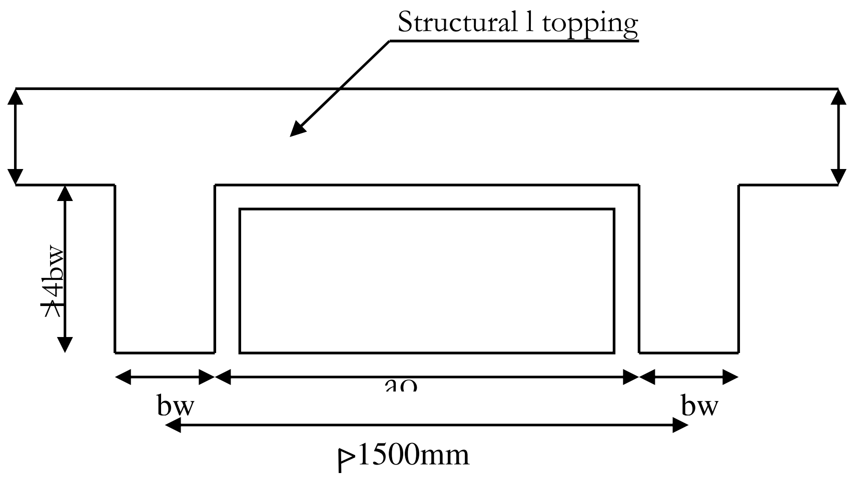


Figure 1.4 permanent blocks not contributing to structural strength [2]

iii. Non-structural topping and structural –type blocks

If the topping does not contribute to the strength, then the blocks must be structural type. In this case the thickness of the block is a basic requirement. The rib and the hollow concrete block should have good bondage and the block and the ribs act as one unit.

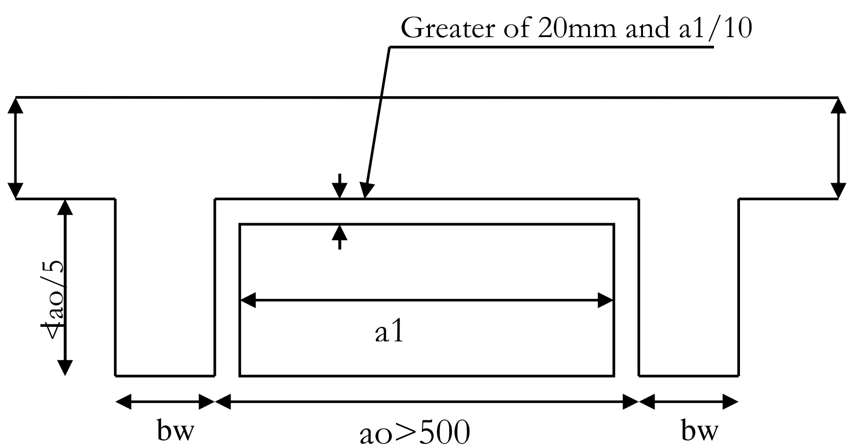


Figure 1.5 Blocks contributing to structural strength but topping does not [2]

b. Slabs without permanent blocks

For this type of slab the minimum thickness of topping is the greater of 50mm and one-tenth of the clear distance between ribs.

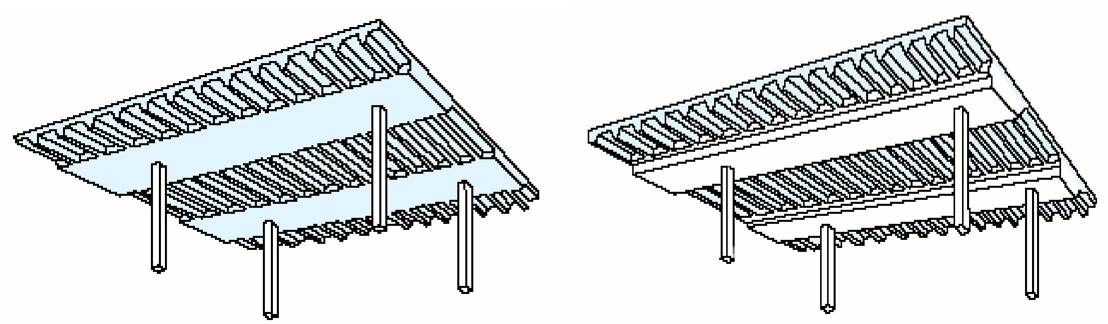


Figure 1.6 Ribbed slab systems with out HCB block

2. STABILITY OF COLUMNS IN RIBBED OR FLAT SLAB FRAME SYSTEM

2.1 INTRODUCTION

The design of high rise buildings essentially involves a conceptual design (by architects) approximation analysis, primary design and optimization, to safely carry gravity and lateral loads. The design criteria are strength, serviceability, stability and human comfort. The strength can be satisfied by limit stresses, while the serviceability is satisfied by drift limit and stability is satisfied by sufficient factor of safety against buckling and p-Δ effect [3]. The absence of beams in site cast concrete flat slab or ribbed slab construction may limit the structural performance of these systems. The relatively shallow depth of the joint between the columns and slabs can restrict their capacity to carry heavy loads on the slab and can limit their resistance to lateral forces [3]. The stability of individual column member and the entire structure as a whole is influenced by the geometry, material, loading effect and framing systems.

In this thesis work the researcher's intention is to see the effect of framing system on the stability of multistory slab-column building system as moment resistance frame system designed in Addis Ababa and other regions in Ethiopia.

The aim of Structural Engineer is to arrive at suitable structural schemes to satisfy the above mentioned criteria.

The most commonly used structural systems have been classified as [12]

i. Moment Resisting Frames

Beam-column system in which the beams and columns are rigidly connected to provide moment resistance at joints, are placed in two orthogonal directions to resist lateral loads in each direction. Each frame is required to resist its proportion of the lateral shear, which is determined on the basis of its relative stiffness compared to the total. The efficiency of development of lateral stiffness is dependent on bay span, number of bays in the frame, number of frames and the available depth in the floors for the frame girders. The design of these frames is controlled, therefore, by the bending stiffness of individual members .The deeper the member, the more efficiently the bending stiffness can be developed. In current practice, buildings with pure shear frames are generally restricted to only a few stories in height, since other more efficient forms are available.

ii. **Shear Wall-Frame Systems**

Structural walls or shear walls refer to structures in which the resistance to horizontal forces is principally provided by walls. These walls are usually constructed of reinforced concrete or masonry. The great advantage of structural walls is the protection their natural stiffness offers to non structure through limiting inter story deflections.

iii. **Shear Truss-Outrigger Braced Systems**

iv. **Framed-Tubes**

The framed tube system is a special case of the moment resisting frame, which usually consists of closely spaced wide steel columns combined with relatively deep beams. These frames are usually, but not only, located on the perimeter of the structure, to introduce more stiffness to overcome the problem of excessive horizontal deflection of orthodox moment resisting frames, as the expense of reduction in ductility.

v. **Tube-in-Tube Systems with interior columns**

vi. **Bundled Tubes**

vii. **Truss Tubes without interior columns**

viii. **Modular Tubes**

The structural system should be able to carry different types of loads such as gravity, lateral, temperature, blast and impact loads.

Optimizing the different type of structural systems for multi story buildings, Khan, Lyengar and Colaco prepare the graph below which shows number of story versus structural system.

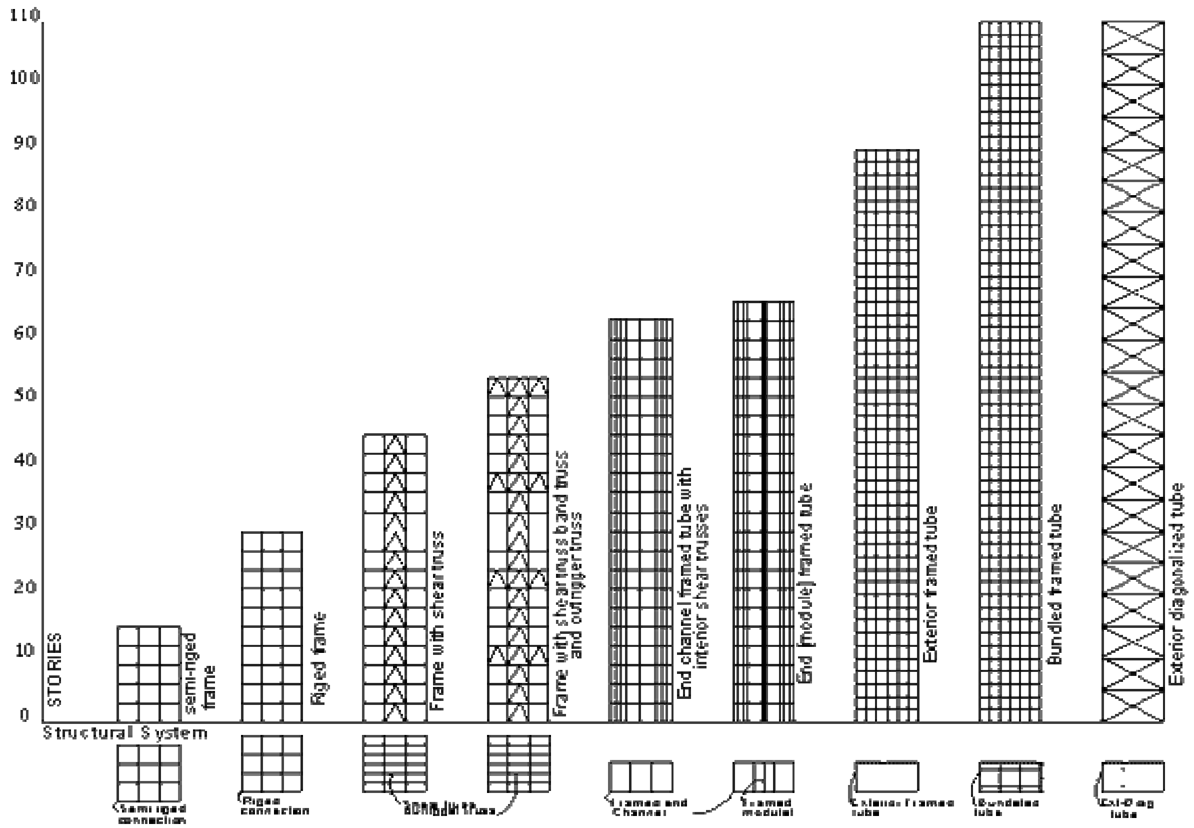


Figure 2.1 Number of story Vs Structural system [12]

From the graph we can see that the moment resisting frames (deep beam-column, rigid frame system) can be stable until 15 stories. When we are supposed to increase the story above it, we can use other structural systems.

The use of rigid frame generally restricts columns' placements too regular, orthogonal lay outs, often requires deeper beams and more closely spaced larger columns. These frame systems are normally not well suited for structures with usually long spans or tall columns. Rigid frames depend on extra stiff connections in the structural frame work to resist the effect of lateral forces. However, rigid frame is the most structurally inefficient of lateral load resisting systems. It is the most suitable for low or broad structures requiring relatively modest resistance or in taller buildings and it may be used in combination with other systems. In addition, rigid frames also place greater stress on the structural frame work, and as a result, columns and beams may be heavier, column spacing may be less than those in a structure relaying on some other lateral force resisting systems.

2.1.1 Effective strip width of flat and ribbed slab for frame analysis

The design of flat slab building is typically carried out in a similar manner to conventional beam column frame system. The buckling analysis of braced and unbraced column supporting flat or ribbed slab buildings may be generalized to a similar framing building in which the portion of flat-slab stiffness is replaced by the stiffness of horizontal beam. It requires analytical analysis in order to decide which portion of flat slab can be counted to substitute for the stiffness of replaced beam grid. An effective slab width model is commonly used to model the lateral load stiffness of the slab column frame. In the effective slab width method, the three dimensional system is modeled as a two dimensional frame using an effective slab width $\alpha l2$ and a conventional column [1].

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 $\alpha-$ value for the effective slab width model depends primarily on the column and slab aspect ratios, where as the influence of cracking on stiffness is accounted for by using an additional factor, commonly referred to as β-factor (cracking) [1].

In equivalent frame method, a flat-slab building is modeled as a set of parallel plane frames in which the columns are modeled as equivalent columns whose flexibility of the columns above and below the slab and the flexibility of torsion members, and the slabs are modeled as equivalent beams. The equivalent beam has a depth equal to that of the original slab and an effective width that targets both the strength and the stiffness of the slab [15].

For the modeling of the slabs, the portion of the slab that will contribute to the frame analysis should be determined as the width of the concealed beam within this slab portion. 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 [3]. In this case 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. Further more, the increase inertia of the shallow beam, although related linearly with width, may lead to 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 imposed demand [3].

For the determination of the width of the concealed beam, or the slab beam, there are several considerations.

According to ACI, slabs can be designed using the direct design method and equivalent frame design method. The direct design method is easier to use than the equivalent frame method but it can have limitations on using this method [15]. Where as the equivalent frame method is accomplished by elastic frame analysis. This method is also a good indication in defining the effective width of the slab frame for determination of the system stability in

unbraced system. Equivalent frame method analysis of slab system stiffness can be different for only vertical load carrying system and vertical and lateral load carrying frame system.

I. **Equivalent frame analysis of slab systems for vertical loads [15]**

The slab is divided into a series of equivalent frames running in two directions of the building, these frames consist of the slab and the columns above and below the slab. For gravity load analysis ACI code allows analysis of the entire equivalent frame extending over the height of the building, or each floor can be considered separately with the far ends of the column and middle strip. This method is used for moment distribution for full stiffness of the section.

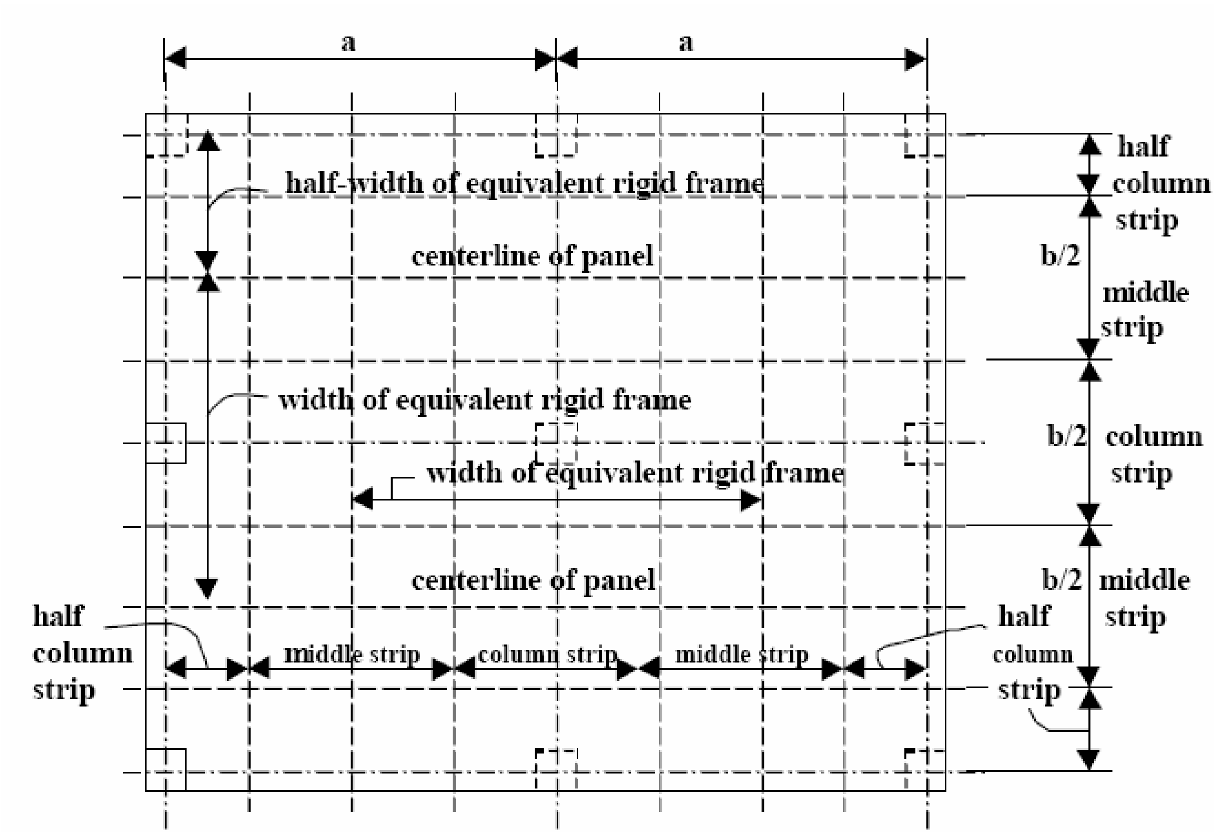


Figure 2.2 Plan of flat slab without beam grid for vertical loading frame analysis only

II. **Equivalent frame analysis of laterally loaded unbraced frame [15]**

Unbraced slab-column frames are sometimes used for low buildings or the top few floors in tall building where the story to story drift in the upper stories may be reduced by terminating the shear wall before the top of the building. For such cases it is necessary to analyze equivalent frame structures for both gravity and lateral loads and add the result. For the gravity load analysis, the equivalent frame analysis can be used with out modification. For lateral load analysis, however, this equivalent frame underestimates the lateral deflection and hence the p-Δ effects, because it is based on uncracked EI values.

For both gravity and lateral load analysis the slab beam strip should be attached to columns by torsional member. For laterally loaded frames the effect of cracking and reinforcement is taken in to account when computing the stiffness of the frame members.

Safely provision of ACI code and similar design specifications ensures that, under loads up to the full service load, stresses in both steel and concrete remain with in the elastic ranges. Consequently deflections that occur at once upon application of load, the so called immediate deflections can calculated based on the properties either of the uncracked elastic member, or the cracked elastic member or some combination of these.

The procedure to analyze an unbraced slab and column structure according to the ACI code is as follows.

A. **Equivalent frame method using ACI code**

For vertical and horizontal loads, these slabs may be analyzed by studying the equivalent frames in each direction. The internal forces in the slab and supports can be determined by calculating the resulting equivalent frames for all load hypotheses, taking into consideration the most unfavorable combinations.

Stiffness specifications of beams and supports in equivalent frames [15]

The following criteria should be followed for vertical loads.

- When defining the moment of inertia of the beams that represent the slab, the gross moment of inertia corresponding to the total equivalent frame width should be employed by taking into consideration the variation in stiffness that exists along the length of the slab.

- When defining the moment of inertia of the supports by taking into account the effect produced by the torsional restraint transversely conferred by the slab, an equivalent stiffness *Keq* should be employed in accordance with the following expression:

$$\frac{1}{Keq} = \frac{1}{Kc} + \frac{1}{Kt} \qquad [2.1.1]$$

Where: Kc*: The* gross support stiffness

Kt*:* the stiffness of the torsional restraint elements. A torsional support restraint element may be defined as the portion of slab with a width that is equal to the dimension c1 of the support or capital, and a length that is equal to the width of the equivalent frame.

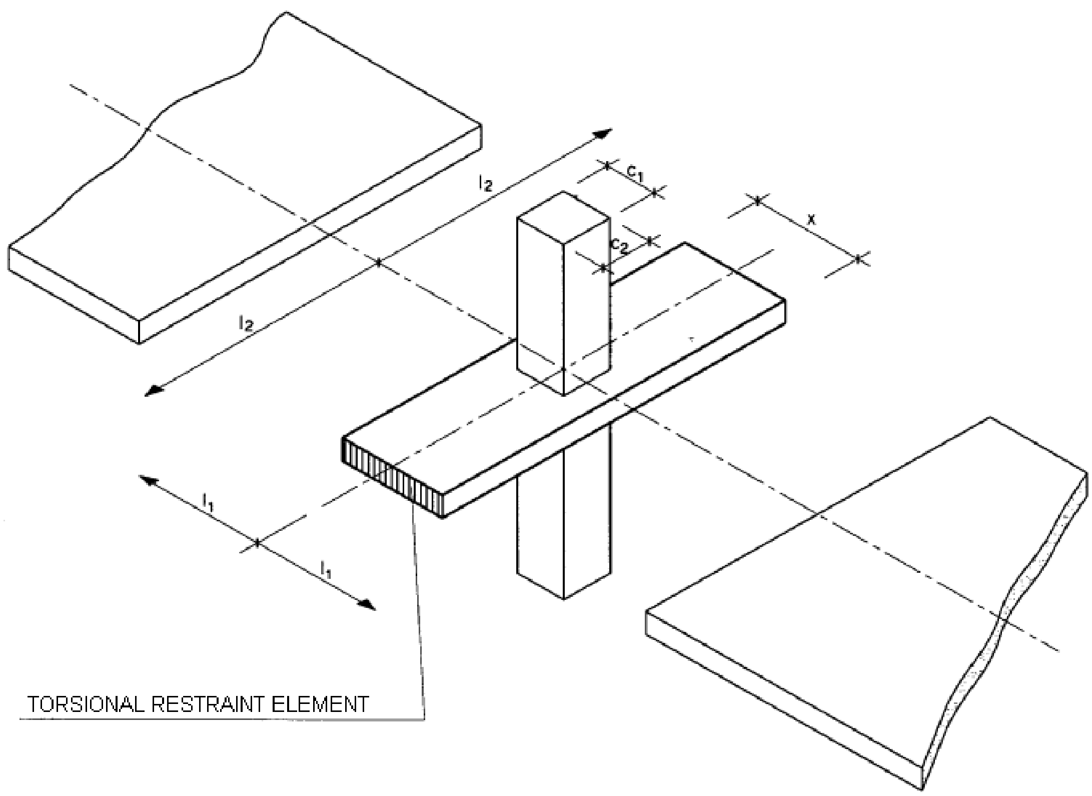
$$K_t = \sum \left(\frac{9\ E_c C}{l_2 \left(1 - \frac{c_2}{l_2}\right)^3} \right) \qquad [2.1.2]$$

Where: Ec: Modulus of elasticity of concrete.

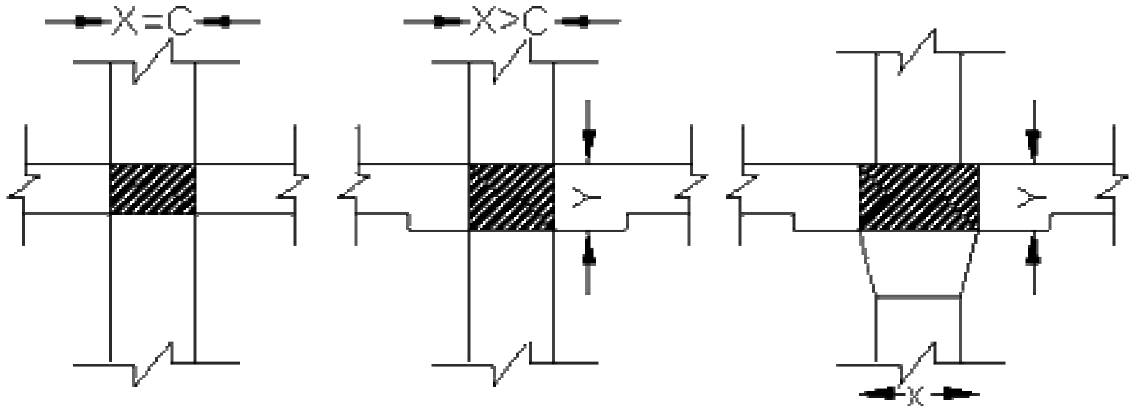
C*: Torsional* stiffness of the torsional restraint element.

l 2: Transverse dimension of the panel adjacent to the specific support.

C2: Dimension of the specific support perpendicular to the equivalent frame.



a)



b)

Figure 2.3 Stiffness of torsional restraint elements

For interior frames, *Kt* is the sum of the torsional stiffness of the torsional restraint elements that exist on both sides of the specific support. For exterior frames, *Kt* is the torsional stiffness of the single torsional restraint element adjacent to the specific support. The following expression can be employed as a definition of *C,*

$$C = \left(1 - 0.63\,\frac{X}{Y}\right) X\;3\,\frac{Y}{3} \qquad X<Y \qquad [2.1.3]$$

According to ACI code, equation 2.1 can determine the effective width of the slab-beam for frame analysis.

ACI states “the effective beam 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……” [1].Hence ACI defines a region of the slab which has special flexural reinforcement and which is like abeam concealed within the slab thickness with a width of [3]

b_w = C+2(1.5**h**) [2.1.4]

Moments determined from the equivalent columns in the frame analysis are used in the design of the columns above and below the slab beams.

According to the Mexico building code (1987), the equivalent slab-beam element for frame analysis is the same as [3]

b_{eff}= C+3**h** [2.1.5]

Where

C: is the column dimension in the direction perpendicular to the analysis

h: is the slab depth.

The Greek code for Reinforced Concrete Structures (1999) states that in the presence of lateral loads, the slab beam width is taken as [3]

$b_{eff} = b_o + 2\mathbf{h}_s$ [2.1.6]

Where

b_o: is the column width in the direction of interest and

$\mathbf{h}_s$: is the slab depth.

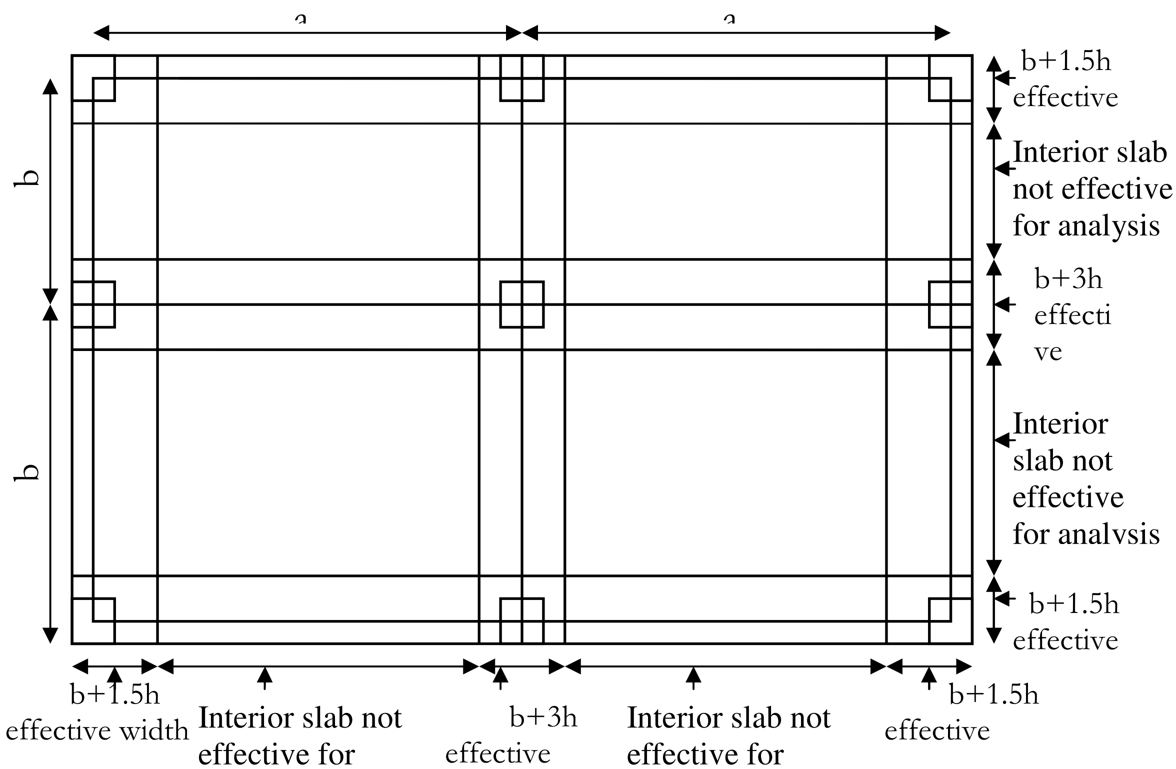


Figure 2.4 Plan of flat slab for effective slab width for lateral and vertical load carrying frame analysis.

In general the studies of flat plate systems have focused on developing and refining a model that describes the stiffness of the slab column frame and the strength of the slab column connection.

According to EBCS-2, an equivalent beam shall be taken as having the width and thickness of the slab forming the column strip. Our code lacks the recommendation with respect to the effective width of the slab beams for the case of lateral loading supporting, future improvement in this regard is required [9].

Considering all the above recommendations and the available case data in this study (i.e. the ribbed slab frame system) the given full width of ribbed beam is effective in frame analysis.

2.1.2 Determination of effective buckling length of columns in a frame

The fundamental equations for the design of slender compression member were derived from hinged ends and must be modified to account for the effect of end restraints. This is done by using an "effective length" Kl_u, in the computation of slenderness effect. The primary design aid available for designers to estimate the effective length factor K is the alignment chart [20]. This allows a graphical determination of K for a column of constant cross section in a multy bay frame because of the difference in behavior between a braced and an unbraced frame system. It is necessary to have one set of effective length factors for completely braced or completely unbraced frames. In an actual structure, however, there is a rare occurrence of completely braced frame or completely unbraced frame. The buckling length of columns depends on column location, size, dimension, and base support condition in addition to flat slab or ribbed slab thickness in the case of slab-column framing system. The columns on the edge of slab-column frame system have relatively smaller buckling strength than that of columns in the center of slab column frame building. The corner columns have even lower buckling strength than that of the edge column of the same dimension.

The effective length is a function of the relative stiffness at each end of the compression member, the effect of widely varying beam and column reinforcement percentage. Beam cracking should be considered in determing the relative end stiffness. In determing the effective length factor K, the rigidity of the flexural members may be calculated on the basis of the moment of inertia of the cracked transformed section and the rigidity of the compression members on the basis of EI with $\beta_d=0$

ACI developed the following simplified equation for computing the effective length factors for braced and unbraced members [3].

For braced compression members, an upper bound to the effective length factor may be taken as the smaller of the following two expressions.

$K=0.7+00.5(\omega_A+\omega_B) \leq 1.0$ [2.1.2.1]

$K=0.85+00.5\omega_{min} \leq 1.0$ [2.1.2.2]

Where ω_A and ω_B are the values of w at the two ends of the column and ω_{min} is the smaller of the two values

For unbraced compression member restrained at both ends, the effective length may be taken as

For $\omega_m<2$

$$K = \frac{(20 - Wm)}{20 \times \sqrt{(1 + Wm)}}. \quad [2.1.2.3]$$

For $\omega_m \geq 2$

$$K = 0.9\sqrt{(1 + Wm)}$$

Where, ω_m is the average of the ω values at the two ends of the compression member.

For unbraced compression member.

For unbraced compression member hinged at one end, the effective length factor may be taken as

$K=2.0+0.3\omega$ [2..1.2.4]

Where: ω is the value at the restraint end.

ω Ratio of $\sum \frac{EI}{lc}$ of compression member to $\sum \frac{EI}{lc}$ of flexural members in a plane of one end of a compression member.

Effective buckling length of compression members according to EBCS-2 [9]

The effective buckling length Le of a column in a given plane may be obtained from the following approximate equations

a) Non-sway mode

$$\frac{Le}{L} = \frac{\alpha m + 0.4}{\alpha m + 0.8} \geq 0.7 \quad [2.1.2.5]$$

b) Sway mode

$$\frac{Le}{L} = \sqrt{\frac{7.5 + 4(\alpha 1 + \alpha 2) + 1.6\alpha 1\alpha 2}{7.5 + \alpha 1 + \alpha 2}} \geq 1.15 \qquad [2.1.2.6]$$

Or conservatively

$$\frac{Le}{L} = \sqrt{(1 + 0.8\alpha m)} \qquad [2.1.2.7]$$

The stiffness coefficients α_1 and α_2 from the model shown

$$\alpha 1 = \frac{K1 + Kc}{K11 + K12} \qquad [2.1.2.8]$$

$$\alpha 1 = \frac{K2 + Kc}{K21 + K22} \qquad [2.1.2.9]$$

$$\alpha m = \frac{\alpha 1 + \alpha 2}{2} \qquad [2.1.2.10]$$

Where: K_1 and K_2 are column stiffness coefficients $\frac{EI}{L}$

K_c is the stiffness coefficient $\frac{EI}{L}$ of the column being designed

K_{ij} is the effective beam stiffness coefficient $\frac{EI}{L}$

=1.0 opposite end elastically or rigidly restrained

=0.5 opposite end free to rotate

= 0 for a cantilever beam

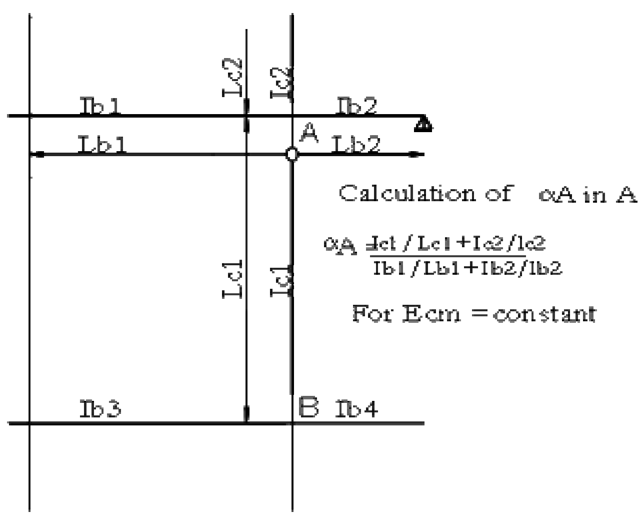


Figure 2.5 Model for computation of stiffness coefficient

The above approximate equations for effective length calculation are applicable for values of $\alpha 1$ or $\alpha 2$ not exceeding 10. For calculating α only members properly framed into the end of the column in the appropriate plan of bending shall be considered. The stiffness of each shall be obtained by dividing the second moment of area of its concrete section by its actual length. When the connection between a column and its base is not designed to resist other than nominal moment α at such positions, α shall be taken as 10, if a base is designed to resist the column moment, it may be taken as 1.0.

2.1.3 Classification of frames as braced or unbraced and sway or non sway

2.1.3.1 Classification According to EBCS

Structures or structural members may be classified as braced or unbraced depending on the provision of bracing elements and as sway or non sway depending on their sensitivity to second order effect due to lateral displacements.

A frame may be classified as non sway if its response in plane horizontal forces is sufficiently stiff for it to be acceptably accurate to neglect any additional internal force or moments arising from horizontal nodes. A frame may be classified as non sway for a given load case if the critical load ratio $\frac{Nsd}{Ncr}$ for that load case satisfies the criterion [9]:

$$\frac{Nsd}{Ncr} \leq 0.1 \qquad [2.1.3.1]$$

Where N_{sd}: is the design value of the total vertical load

N_{cr} *:* is its critical value for failure in a sway mode

Beam-and-column type plane frames in building structures with beams connecting each column at each story level may be classified as a non sway frame for a given load case. If the horizontal displacements in each story due to the design loads (both horizontal and vertical), plus the initial sway imperfection based on the 1st order theory satisfy these equation [9]

$$\frac{N\delta}{HL} \leq 0.1 \qquad [2.1.3.2]$$

Where:

δ: is the horizontal displacement at the top of the story relative to the bottom of the story

L: is the story height

H: the total horizontal reaction at the bottom of the story

N: is the total vertical reaction at the bottom of the story.

The displacement δ above shall be determined using stiffness values for beams and columns corresponding to the ultimate limit state. As an approximation, displacements calculated using moment of inertia of the gross section may be multiplied by the ratio of the gross column stiffness to the effective column stiffness to obtain δ .

2.1.3.2 Classification According to ACI

According to ACI, when the stability index [19]

$Q = \frac{\sum Pu\Delta u}{Huhs}$ For a story is not greater than 0.04 or the p-Δ moment does not exceed 5% of the 1st order moment, then the structure can be considered to be braced.

Where:

H_u : is the total factored lateral force acting within the story and

Δ_u : is the elastically -computed 1st order lateral deflection due to H_u (neglecting P-Δ effect) at the top of story relative to bottom of the story.

A story to be considered non sway when the stability index.

$Q = \frac{\sum Pu\Delta u}{VuLu}$ For a story is not greater than 0.05

Where Σ P_U and V_U are the total factored vertical load and story shear, respectively Δo is the 1st order relative deflection between the top and the bottom story due to V_U; and L_c is the length of compressive member measured center to center of the joints in the frame.

In accordance with ACI code 10.11.1 the section properties of the frame members used to calculate Q must be taken in to account the effect of axial loads, cracked regions along the length of the member, and the duration of the loads. Alternatively the section properties may be represented using the modules of elasticity Ec and the following section properties [19].

Moment of inertia

Beams 0.35Ig

Columns 0.7Ig

Walls-uncracked 0.7Ig

Walls- cracked 0.35Ig

Flat plates and flat slabs 0.25Ig

The moment of inertial must be divided by $(1+\beta_d)$ when the sustained lateral loads acting for stability check.

In frame system all frame elements shall have adequate resistance to failure in a sway mode. However, where the frame is shown to be a non sway frame no further sway mode verification is required. All frames including sway frames shall also be checked for adequate resistance to failure in non sway modes.

2.1.4 Determination of frame element stiffness EI for analysis

Frame member stiffness **EI** and **GJ** should reflect the degree of cracking and inelastic action which has occurred along each member before yielding. Non-linearity arises from the stress strain relation of the materials. The development of cracks in the concrete and secondary load-deflection (slenderness) effects lead to a situation in which the maximum load and bending moment induced in a column cannot readily be related to the loads acting on the structure. The column rigidity depends on the axial load of the moment variation along the length, the percentage reinforcement, the steel placement, the strength of steel and concrete, and their critical strain values.

In flat slab structural system, the appropriate stiffness to be used in strength calculations must be chosen to estimate the lateral deflection accurately at the factored load level.

However, in braced frames the relative values of stiffness are important. Two usual assumptions are to use gross EI values for all members or, to use half the gross EI of the beam stem for beams and the gross EI for columns.

For frames that are free to sway, a realistic estimate of EI is desirable and should be used if 2nd order analysis are carried out.

Two conditions determine whether if it is necessary to consider torsional stiffness in the analysis of a given structure [15];

1) The relative magnitude of the torsional and flexural stiffness

2) Whether torsion is required for equilibrium of the structure (equilibrium or primary torsion) or is due to members twisting to maintain deformation (compatibility or indeterminate torsion). In the case of compatibility torsion the torsion stiffness may be neglected. For cases involving equilibrium torsion, torsional stiffness should be considered.

The effective stiffness of a column EI_e is [5]

$$EI_e=0.2E_cI_c+E_sI_s \qquad [2.1.4.1]$$

Where E_c= 1100fcd

E_s is the modules of elasticity of steel

I_c, I_s are the moments of inertia of the concrete and reinforcement sections.

Alternatively

$$EI_e \geq 0.4E_cI_c \qquad [2.1.4.2]$$

In the case of sway frame creep may be incorporated by dividing the effective stiffness by $(1+\beta_d)$. Where β_d is defined as:

$$\beta = \frac{factored \quad dead \quad load \quad in \ the \ story}{total \quad factored \quad load \quad in \ the \ story}$$

Then the stiffness EI of a column is

$$EI = \frac{0.4EcIg}{1+\beta d} \qquad [2.1.4.3]$$

For computer analysis according to ACI code of practice, the stiffness values for the frame elements for column and equivalent slab beam member the equation below is used as an input for the structural frame [19].

$$EIbeam = \frac{0.35EcIg}{1+\beta d} \qquad [2.1.4.5]$$

And for columns

$$EIcolumn = \frac{0.7EcIg}{1+\beta d} \qquad [2.1.4.6]$$

Based on studies Mac-Grigor (1997) recommends the above two equations when carrying out second order analysis. The loads causing appreciable side sway are generally short duration loads, such as wind or earth quake; as a result it should not cause creep deflections so the β_d value is taken as zero.

For our computer analysis the reduction in stiffness is taken as the above two equations according to ACI code of practice. EBCS-2 1995 recommends using any reasonable assumption may be adopted for computing the relative flexural and torsional stiffness of member. In this research work no reduction of stiffness for analysis instead the researcher uses the displacement behavior factor γ =0.44 for low ductility class building. The assumption made shall be consistent throughout the analysis.

2.1.5 Limitation of lateral drift for Building frames according to EBCS and ACI

Lateral drift is likely to be a significant design issue for flat –slab frames. In many cases, lateral drift consideration rather than strength considerations will control the frame proportions. The limit states for the case study of flat slab or ribbed slab building in global sense, particularly in terms of inter story drift ratio and the behavior of the failure modes of the structural system are governed by deformation. Large flexibility of flat slab or ribbed slab frames causes concern in seismic design for several reasons including possible damage to structural and non structural components and over all stability of the frame under excessive drifts.

2.1.5.1 Serviceability limit state

According to EBCS 8-95 limits the inter story drift [10],

a. For buildings having non structural elements of brittle material attached to the structure:

$d_r, \leq 0.01h$ [2.1.5.1]

b. For buildings having non structural elements fixed in a way as not to interfere with structural deformations:

$d_r, \leq 0.015h$ [2.1.5.2]

Where d_r : design interstory drift

h: story height

According to ACI code of practice considering column between the two floors i-1 and i in the frame shown in figure2.6.The maximum lateral displacement or drift at the upper end of the top column in the frame is Xmax and the total height of the building is hs. A large drift or lateral displacement of the building upper floors results in cracking of the masonry and interior finishes. Unless precautions are taken to permit movement of interior partitions with out damage, the maximum lateral deflection limitation should be $\frac{hs}{500}$ [18].

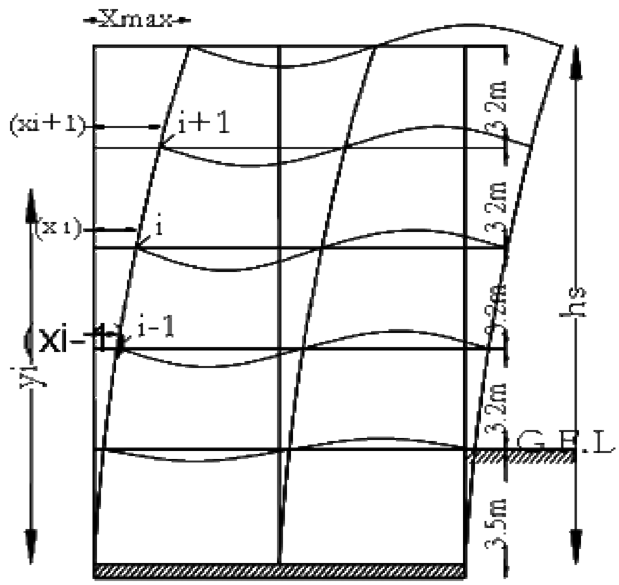


Figure 2.6 Second order frame parameters
Δ-P Drift of frame element [18].

Hence a good assumption is to choose Xmax in the range of $\frac{hs}{350}$ *to* $\frac{hs}{500}$*, considering that a full braced structural system has normally a ratio of maximum drift Xmax to frame height hs less than* $\frac{1}{1500}$.If Xi is the drift at floor level i, and hi is the height of the column between floors i-1 and i in the figure below, it can be assumed that the proportional horizontal drift for a particular floor is directly proportional to the square of the ratio of the height hi of the floor and the total height hs of the entire frame [18].

Hence $Xi = X\max\sqrt{\frac{hi}{hs}}$

2.1.5.2 Ultimate limit state induced by lateral displacement

It is required that the p-Δ effect be considered in the determination of member forces and story displacements, if this is found significant. The p-Δ effect need not be considered if the ratio, θ, of the secondary to the primary moment for any story does not exceed 0.10, or in seismic zone 3 0r 4. According to EBCS-8 1995 the p-Δ effect on the frame influence by the moment ratio (stability index) θ defined as [10]

$$\theta = \left(\frac{Ptotdr}{Vtoth}\right) \qquad [2.1.5.3]$$

Where P_{tot}= total gravity load at and above the story considered

d_r= design inter story drift, evaluated as the difference of the average lateral displacement at the top and bottom of the story under consideration. The displacement induced by the design seismic action shall be calculated on the basis of the structural system by means of the following simplified expression:

$$ds = \frac{de}{\gamma d}$$

Where ds=dr displacement of a point induced by the design seismic action

γ_d =displacement behavior factor, assumed equal to γ=0.44 for low ductility class building frames.

d_e =displacement of the same point of the structural system, as determined by a linear analysis based on the design response spectrum.

V_{tot} =total seismic story shear

h= inter story height;

θ is not allowed to exceed 0.25.

According to ACI building code the stability index

$Q = \frac{\sum Pu\,\Delta u}{VuLu}$ greater than 0.30 is not allowed for moderate earthquake regions.

Where Σ P_U and V_U are the total factored vertical load and story shear, respectively

Δo is the 1st order relative deflection between the top and the bottom story due to V_U.

L_c is the length of compressive member measured center to center of the joints in the frame.

In accordance with ACI code 10.11.1 the section properties of the frame members used to calculate Q must be taken in to account the effect of axial loads, cracked regions along the length of the member, and the duration of the loads.

2.1.6 Method of Calculating Lateral (seismic) Loads and Combinations According to EBCS-1

The earth quake effect of a structure can be analyzed either, using response spectrum analysis or equivalent static analysis procedure.

The response spectrum method or the mode superposition method is generally applicable to analysis of dynamic response of complex structures in their linear range of behavior. This particular applies to analysis of forces and deformation in multistory buildings due to medium intensity ground shaking causing moderately large but essentially linear response of the structure. The method is based on the fact that for certain forms of damping which are reasonable model for many buildings the response in each natural mode of vibration can be computed independently of the others and the modal responses can be combined to determine the total response. The equivalent lateral load analysis procedure requires less effort because, except for the fundamental period, the periods and shapes of the higher natural modes are not needed.

The magnitude of lateral forces is based on an estimate of the fundamental period of vibration and their distribution on simple formula appropriate for buildings with regular distribution of mass and stiffness over height.

Both procedures are based on the same basic assumptions and are applicable to buildings whose dynamic response behavior is in reasonable conformity with the implication of the assumptions that forces and deformation can be determined by combining the result of independent analysis of a planer idealization of the building for each horizontal component of ground motion, and including torsional moment determined on an indirect empirical bases. In particular, both methods may be inadequate if the lateral motions in two orthogonal directions and the torsional motions are strongly coupled.

The main difference between the two procedures lies in the magnitude of the base shear and distribution of the lateral forces. Where as in the response spectrum method the force calculations are based on computed periods and mode shapes of several modes of vibration. In the equivalent lateral load they are based on an estimate of the fundamental period and simple formula for distribution of forces which are appropriate for buildings with regular distribution of mass and stiffness over height. Generally, it is adequate to use the equivalent lateral force procedure for buildings with the following properties: seismic force resisting system has the same configuration in all stories and in all floors; floor masses do not differ by more than, say, 30% in adjacent floor; and cross sectional area and moments of inertia of structural members do not differ by more than about 30% in adjacent stories.

The seismcity of the area and the potential hazard due to failure of the building should also be considered in deciding whether the equivalent lateral force procedure is adequate, For example, even irregular buildings, that may require response spectrum analysis according to the criteria described, may be analyzed by the equivalent lateral force procedure if they are not located in highly seismic area.

For the purpose of seismic design, building structures are distinguished as regular and non – regular.

According to EBCS-8, we can decide which method of structural model, structural analysis, the value of behavior factor γ geometric non- regularity limit, non-regularity distribution of over strength in elevation exceeding the limits can be most appropriate for our structure system based on the given table [10].

Table2.1 Consequences of structural regularity on seismic design

Regularity		Allowed simplification		Behavior factor
Plan	Elevation	Model	Analysis	
Yes	Yes	planer	Static*	Basic
Yes	No	planer	Static*	Increased
No	Yes	spatial	Static*	Basic
No	No	spatial	Dynamic	Increased

*For T1 < 2 s

2.1.6.1 **Equivalent Static Analysis**

This type of analysis can be applied to buildings whose response is not significantly affected by contribution from higher modes of vibration [10].

The seismic Base shear force F_b for each main direction is determined from

$F_b = S_d\,(T1)\,W$ [2.1.6.1]

Where: S_d (T1) ordinate of the design spectrum at period T1

$S_d\,(T1) = \alpha\beta\gamma$

$\alpha = \alpha o I$

αo= Bed rock acceleration

I= Importance factor

$\beta = \frac{1.2s}{T^{2/3}}$ Design response factor, $\beta \leq 2.5$

γ=Behavior factor= $\gamma_o K_D K_R K_W$

γ_o =Basic value of behavior factor

K_D= Ductility class

K_R= Regularity in elevation

K_W= Prevailing failure mode

T1 Fundamental period of vibration of the structure for translational motion in the direction considered,

$T1=C1H^{3/4}$ [2.1.6.2]

H= Height of the building

W Seismic dead load computed

$Ft=0.07T1F_b$ = Concentrated force at top [2.1.6.3]

The base shear distribution over the height of the building can be calculated using the equation

$$Fi = \frac{(Fb - Ft)Wihi}{\sum Wihj}$$ [2.1.6.4]

For this thesis work the researcher uses a total of 3 combinations, the combinations are as follows [7]:

Combo 1 = 1.3DL + 1.6LL

Combo 2 = 0.75(1.3DL+1.6LL)+EQalongX+ve+Accidental torsion

Combo 3 = 0.75(1.3DL + 1.6LL)+EQalong Y +ve + Accidental torsion

These three combinations are sufficient for symmetric in plan and elevation buildings considered.

2.1.7 Seismic vulnerability of ribbed or flat slab frame structures [19]

Building systems constructed to resist seismic forces are often designed with lateral force resisting system to resist the seismic forces and non participating system (gravity force resisting system) to support vertical loads.

Slab-column connections are susceptible to punching shear failures when subjected to inelastic load reversal. The methods for strengthening the slab column connections against punching-shear failures include the use of drop panels or slab shear reinforcement or the use of high strength concrete in the vicinity of the slab column. Among these, drop panels (or shear capitals) tend to increase the strength of the perimeter of the punching shear critical section [3]. Drop panels with small plan dimension are not effective in an earth quake when lateral force can produce reversal of relatively high unbalanced moments in slab column vicinity. In such cases uninverted punching failure can occur for which the additional slab depth provided by the shear capital is not effective in punching shear capacity.

Nowadays structural engineers design a building with lateral -force –resisting system (LFRS) and a non participating system or gravity force- resisting system (GFRS).

The elements designated to be part of the LFRS are proportioned to resist the entire design seismic forces and provide sufficient stiffness to limit the lateral drift to acceptable levels. The elements to be part of the GFRS are proportioned assuming that they do not contribute to the seismic resistance, that is, these elements are designed to resist gravity force only.

2.1.8 Method of designing slender concrete columns in sway frame

Slender column design for sway frame may be accomplished by one of the three methods. These methods are [18]:

- Second order computer analysis
- Direct p-Δ analysis

- Sway moment magnifier method

The point in the analysis of slender column is to determine the magnified moments acting on the column due to the applied factored loads and slenderness effect.

A second – order analysis is a frame analysis that includes the internal force effects resulting from lateral displacement (deflection) of a column. When such analysis is performed in order to evaluate $\delta_s M_s$ in a non braced frame, the deflections must be computed on the basis of fully cracked sections with reduced EI stiffness values. The analysis should verify that the predict strength of the compression members of a structural frame are in a good agreement with in a15% range with results of frame analysis for columns indeterminate reinforced structures. The structure being analyzed should result in geometry of members similar to the geometry of the section to be built. If the members in the final structure have cross sectional dimension differing by more than 10% from those assumed in the analysis, a new computation cycle has to be performed.

A second – order analysis is an iterative procedure of the p-Δ effects on the slender column, including shear deformation. To do this analysis each necessary load combination which includes lateral loads must be used. From the computer analysis the load combination which causes the worst end moments will be used for design. But in case of our code recommendation, we should include additional accidental moment. Since the computer is doing a second-order analysis the moments calculated will already be magnified. Hence, the values of P_u, Mu1 and Mu2 can be read directly from the computer out put.

The P-Δ procedure can be performed in computer analysis using the following procedure [18].

1. Chose geometrical sections of the frame and its columns and their stiffness EI.
2. Calculate the drift, that is , the lateral deflections Δi, and the corresponding ultimate loads P_{ui} at joints i=1,…….,n
3. Find the equivalent horizontal forces Hi from $Hi = \frac{P\Delta}{hi}$
4. Add the values obtained in step 3 to the actual lateral loads acting on the frame.
5. Perform computer analysis using the appropriate computer program.

6. The iterative computer program, using stiffness EI chosen for the input data gives ΔI results that have to be compared with the xi values allowed.
7. If all Δi values are $\leq$ all the Xi values, accept the solution and the design as a second order analysis solution. If not, run additional computer cycles with modified stiffness until the desired are achieved.

Direct p-Δ analysis

The iterative calculation can be described mathematically by using an infinite series. The sum of this series gives the second order deflection, Δ

$$\Delta = \frac{\Delta c}{\left(1 - \frac{\gamma \sum Pu \, \Delta u}{Vulc}\right)} \qquad [2.1.8.1]$$

Where

V_u=Shear in the story due to wind or earth quake loads acting on the frame above the story in question

l_c= story height

ΣP_u= the total axial load in all the columns in the story

γ=1.5

Δ_o=first order deflection due to the story shear, v

Δ=Second order- deflection

Both Δ_o and Δ refer to the lateral deflection of the top of the story relative to the bottom of the story.

Since the moments in the frame are directly proportional to the deflections, the second order moments are

$$M = \delta s Ms = \frac{Mo}{\left(1 - \frac{\gamma \sum Pu \, \Delta o}{Vulc}\right)} \qquad [2.1.8.2]$$

Where M_o and M are the first and second order moments respectively.

ACI defines the stability index for a story as

$$Q = \frac{\sum Pu\Delta c}{Vclc} \qquad [2.1.8.3]$$

Substituting this and omitting the flexibility factor γ gives

$$\delta sMs = \frac{1}{1-Q} \geq Ms \qquad [2.1.8.4]$$

In a braced structure, the relative deflections of the top and the bottom of a story are largely controlled by the slope of the shear walls or bracing element in that story. In such a case the relative deflection of a given story is not independent of the adjacent stories. In this case the moments and deflections of the entire frame can be magnified using the frame magnifier, δf, given by

$$\delta f = \frac{1}{1 - \sum \frac{(\sum \gamma Pu)}{lc} \Delta oi \, 2\left[\sum \left(\sum Vu\right) i \Delta oi\right]} \qquad [2.1.8.5]$$

The procedure for calculating the second order moments and deflections is the same as the procedure for the unbraced frame, except that the same magnifier δf is used in all braced stories.

This procedure is applicable to braced frames and to the braced portions of partially braced structures.

Design procedure for slender columns in sway frame consists of the following five basic steps [18]

1. The unmagnified moments, Mns, due to loads not causing appreciable sway are computed. This is done using a regular 1st order elastic frame analysis using the uncracked member stiffness and which includes the combination of lateral load.
2. The magnified sway moments, δsMs, are computed.
3. The magnified sway moments δsMs are added to the unmagnified non sway moments, Mns. This is done at each end of each column.
4. Check whether the maximum moments occur between the ends of the column. Normally the maximum moments in the column will be at one end and the column is designed for this moments. If the axial loads on the column are high and the slenderness exceeds the limit, it is necessary to check if the moments at some section between the ends of the column exceeds the maximum end moment.
5. Check whether side sways buckling occur under gravity loads alone.

In this literature only second order computer analysis method and direct p-Δ analysis is considered since these are easily handled using computer.

3. CATEGORIZING STRUCTURAL CONFIGURATION ACCORDING TO THEIR FUNCTION

3.1 INTRODUCTION

In most cases, buildings principally for human occupancy are designed conceptually by architects. That is to say that architects are the ones principally responsible for the configuration of buildings for human occupancy.

Configuration has to do with the shape and size of the building. Inevitably shape and size to a large extent determine or greatly influence the type, shape, arrangement, size location and most other aspect of the structural concept. Also the architectural configuration determines the location and nature of non- structural elements of the building.

Configuration, therefore, encompasses:

- Architectural shape and size
- Type, size and location of structural elements.
- Type, size and location of non- structural elements.

Non linear analysis of reinforced concrete building structure is computationally intensive; the analysis will be conducted for the different categories of buildings by increasing the story height. The researcher chose to begin from the mid rise (6 story) buildings since it is the basis of applicability of the ribbed slab structural system. The number of the story increases until it can satisfy the drift demand which is recommended by the EBCS and ACI building codes. The given data for the analysis of the categorized structural systems are compatible with the different building codes place limitation on building heights, and areas in relation to the type of construction employed and the use or uses to which the building will be put.

3.1.1 Existing Grid System for Most Commercial Buildings Compatible with International Standard Grid Arrangement

In the previous discussion, we reviewed the background, basic information and input data for analysis and determination of the different parameters. The selected dimensions of buildings in design are shown below. For simplicity, the building is symmetric in plan with six bays in x- direction and five bays in y direction. This enables the use of 2D as well as 3D model and static frame analysis for lateral load determination. Each story contains 30 rectangular ribbed slab panels with dimensions 6.0x5.0m and primarily the columns and beam dimensions are taken from the different cases selected and column space are compatible with the international standards given from Time-Savers standards for building types and Neufert (architects' data).

For 4-6 story (G+2 - G+4) buildings columns size 500x500mm ribbed beam size main 600x300mm secondary 400x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 7-8 story (G+5-G+6) buildings columns size 600x600mm ribbed beam size main 700x300mm secondary 500x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 9-10 story (G+7-G+8) buildings columns size 700x700mm ribbed beam size main 800x300mm secondary 600x300 he depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 11-12 story (G+9-G+10) buildings columns size 800x800mm ribbed beam size main 900x300mm secondary 700x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 13-14 story (G+11-G+12) buildings columns size 900x900mm ribbed beam size main 1000x300mm secondary 800x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

The story height used for ground floor 4m, for all story above is as 3.4m & foundation column 3.5m.It should be noted that the story height parameter has a significant effect on the flexibility of structures. The columns' size decreases every 3 story by 50mm in either direction and the specified dimensions are used for the 1^{st} 3 stories beginning from foundation.

Further more, equal span lengths in both direction are frequently preferred in flat and ribbed slabs, thus maximize the efficiency of two way slab reinforcement. The loadings can be calculated according to EBCS 1 for shops a live load of $\frac{4.0Kn}{M2}$, uniformly distributed assumed partition load $\frac{1.5Kn}{M2}$.

The above considered span lengths, column sizes and ribbed beam sizes represent most case commercial structures built in Addis Ababa (zone 2) and areas in (zone 4) according to EBCS-8 categorizations.

For the case of buildings which are located in zone 4, the frame sizes were the same as frames located in zone 2 described above.

The figure below shows a typical ribbed beam plan and elevation for this category.

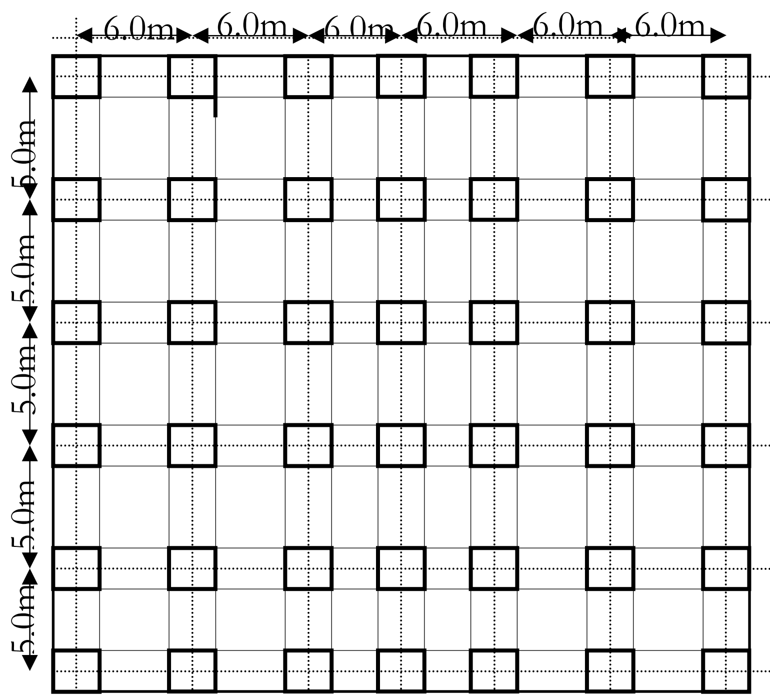


Figure 3.1 Typical ribbed beam lay out plan.

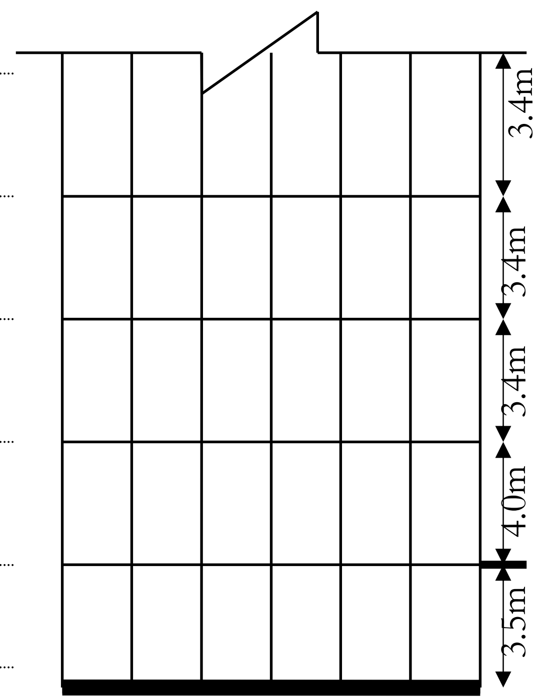


Figure 3.2 Typical ribbed beam elevation.

3.1.2 Existing Grid System for Most Apartment Buildings Compatible with International Standard Grid Arrangements

The selected dimensions of buildings in design are shown below. For simplicity, the building is symmetric in plan with five bays in x- direction and four bays in y direction. This enables the use of 2D as well as 3D model and static frame analysis for lateral load determination. Each story contains 25 rectangular ribbed slab panels with dimensions 5.0x5.0m and primarily the columns and beam dimensions are taken from the different cases selected and column spacing are compatible with the international standards given from Time-Savers standards for building types and Neufert (architects' data).

For 4-6 story (G+2 - G+4) buildings columns size 400x400mm ribbed beam size main 500x300mm secondary 500x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 7-8 story (G+5-G+6) buildings columns size 500x500mm ribbed beam size main 600x300mm secondary 500x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 9-10 story (G+7-G+8) buildings columns size 600x600mm ribbed beam size main 700x300mm secondary 500x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 11-12 story (G+9-G+10) buildings columns size 700x700mm ribbed beam size main 800x300mm secondary 600x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 13-14 story (G+11-G+12) buildings columns size 800x800mm ribbed beam size main 900x300mm secondary 600x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

The story height used for ground floor 3m, for all story above it as 3.0m and foundation column 3.5m.It should be noted that the story height parameter has a significant effect on the flexibility of structures. The columns' size decreases every 3 story by 50mm in either direction and the specified dimensions are used for the 1st 3 stories beginning from foundation.

The loadings can be calculated according to EBCS 1 for apartment of $\frac{2.5\,Kn}{M\ 2}$, uniformly distributed assumed partition load of $\frac{1.0\,Kn}{M\ 2}$.

The above considered span lengths, column sizes and ribbed beam sizes represent most case apartment structures built in Addis Ababa (zone 2) and areas in (zone 4) according to EBCS-8 categorizations.

For the case of buildings which are located in zone 4, the frame sizes were the same as frames located in zone 2 described above.

The figure below shows a typical ribbed beam plan and elevation for this category.

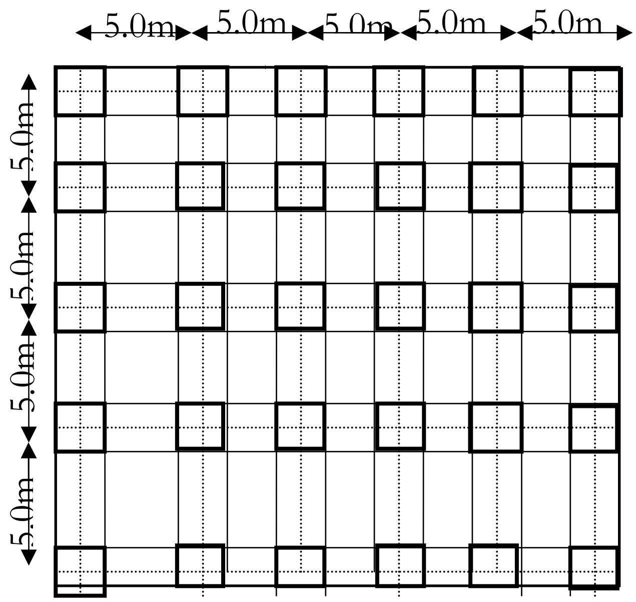


Figure 3.3 Typical ribbed beam lay out plan.

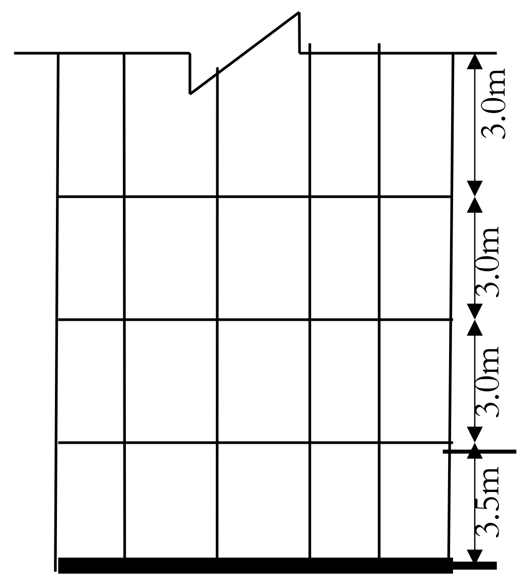


Figure 3.4 Typical ribbed beam elevation.

3.1.3 Existing Grid System for Most Office Buildings Compatible with International Standard Grid Arrangements.

The selected dimensions of buildings in design are shown below. For simplicity, the building is symmetric in plan with four bays in x- direction and three bays in y direction. This enables the use of 2D as well as 3D model and static frame analysis for lateral load determination. Each story contains 30.25 square ribbed slab panels with dimensions 5.5x5.5m and primarily the columns and beam dimensions are taken from the different cases selected and column spacing are compatible with the international standards given from Time-Savers standards for building types and Neufert (architects' data).

For 4-6 story (G+2 - G+4) buildings columns size 450x450mm ribbed beam size main 600x300mm secondary 450x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 7-8 story (G+5-G+6) buildings columns size 550x550mm ribbed beam size main 700x300mm secondary 500x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 9-10 story (G+7-G+8) buildings columns size 650x650mm ribbed beam size main 800x300mm secondary 600x300 he depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 11-12 story (G+9-G+10) buildings columns size 750x750mm ribbed beam size main 900x300mm secondary 600x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

For 13-14 story (G+11-G+12) buildings columns size 850x850mm ribbed beam size main 1000x300mm secondary 700x300 the depth of the beam is already limited by the maximum available hollow concrete block size plus topping.

The story height used for ground floor 3.2m, for all story above it as 3.2m and foundation column 3.5m.It should be noted that the story height parameter has a significant effect on the flexibility of structures. The columns size decreases every 3 story by 50mm in either direction and the specified dimensions are used for the 1st 3 stories beginning from foundation.

The loadings can be calculated according to EBCS 1 for office building of $\frac{3\ Kn}{M\ \ 2}$ uniformly distributed assumed partition load of $\frac{1.25\ Kn}{M\ 2}$.

The above considered span lengths, column sizes and ribbed beam sizes represent most case office building structures built in Addis Ababa (zone 2) and areas in (zone 4) according to EBCS-8 categorizations.

For the case of buildings which are located in zone 4, the frame sizes were increased by 50mm more from the frames located in zone 2 described above.

The figure below shows atypical ribbed beam plan and elevation for this category.

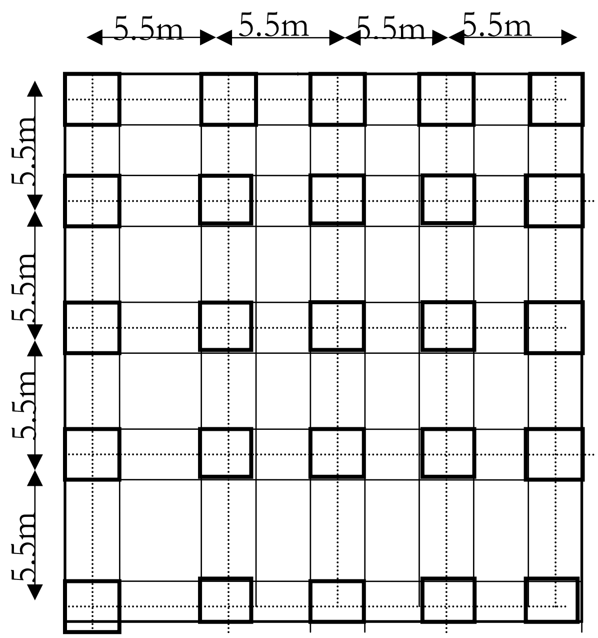


Figure 3.5 Typical ribbed beam lay out plan.

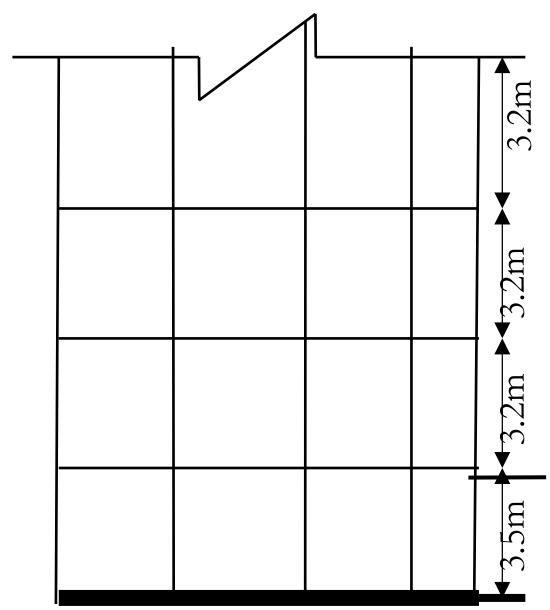


Figure 3.6 Typical ribbed beam elevation.

4. SUMMARY OF THE ANALYSIS

4.1 SUMMARY

This study is conducted to determine the boundary of number of stories in which ribbed slab building frame structure without lateral load resisting system can be built in Addis Ababa and other seismic areas in Ethiopia.

Three categories of buildings, namely commercial (market complex), office and apartment buildings were chosen for the purpose of investigation. The frame systems for all categories were routinely analyzed for six to fourteen stories and the results are summarized in the table 4.1-4.6.

The preliminary evaluation of seismic response indicates that the model structure is very flexible due to the absence of deep beams and shear walls. The performance of the structure was investigated through equivalent static analysis. The ultimate limit state and serviceability limit state of the frame structural system were the major determinant analysis out put. Compressive strength of concrete, yield strength of steel, structural member size, story height and spacing, were the key parameters that were taken from the case data.

The structures were located within zone 2 and zone 4 according to Ethiopian Building Code Standard 8-1995.

The following parameters were assumed for all building structures investigated in the present study [6].

1- The structure is regular in plan as well as in elevation
2- Bed rock acceleration α_o=0.05 for zone 2 and α_o=0.1 for zone 4
3- Importance factor I=1.2

   This is for buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural, institutions etc.

4 - Site coefficient from sub soil condition S= 1.2

   This is for deep deposits medium dense sand, gravel or medium stiff clays with thickness from several tens to many hundreds of meter, characterized by shear wave velocity vs-values of at least $\frac{200\ m}{S}$ at a depth of 10m ; increases at least $\frac{350\ m}{S}$ at a depth of 50m

5- Basic value of behavior factor γ_o=0.2

6- Ductility class DC"L" K_D=2.0

7- Regularity in elevation K_R= 1.1

8- Prevailing failure mode K_W=1.0

Table 4.1 Typical story information for commercial building located in zone 2

Table 4.2 Typical story information for apartment building located in zone 2

Table 4.3Typical story information for office building located in zone 2

Table 4.4 Typical story information for commercial building located in zone 4

Table 4.5 Typical story information for apartment building located in zone 4

Table 4.6 Typical story information for office building located in zone 4

5.0 CONCLUSIONS AND RECOMMENDATIONS

The following conclusions and recommendations can be made based on the results of the present investigation.

1. Commercial buildings which are located in zones 2 and 4 having story height of 3.5m, column size of 600x600mm (the column size is reduced by 50mm for every three stories in either direction) and column spacing of 6.0m by 5.0m should have additional lateral load resisting structural member for seven and above stories in order to stabilize the frame system.
2. Apartment buildings which are located in zones 2 and 4 having story height of 3.0m, column size of 500x500mm (the column size is reduced by 50mm for every three stories in either direction) and column spacing of 5.0m by 5.0m should have additional lateral load resisting structural member for eight and above stories in order to stabilize the frame system.
3. Office buildings which are located in zones 2 and 4 having story height of 3.2m, column size of 550x550mm (the column size is reduced by 50mm for every three stories in either direction) and column spacing of 5.50m by 5.50m should have additional lateral load resisting structural member for eight and above stories in order to stabilize the frame system.
4. The serviceability limit of a story is not satisfied in most of the analyzed ribbed slab frame systems.
5. Ribbed slab frame system buildings without additional lateral load resisting structural elements are too flexible this can be avoided by introducing deeper beam on the top and middle height of the story for mid rise (below eight story) building frames.
6. Architects and structural engineers should discuss alternative structural configurations at the earliest stage of design concept development to ensure that undesirable geometry is not locked into the system before structural design begins.

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DECLARATION

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

Name: Mohammed Arega Ali

Signature: -------------------------------------

Date: December-2006

This thesis has been submitted for examination on my approval as a university advisor

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Dr.-Ing. Girma Zerayohannes