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LIST OF SYMBOLS

M_0 = total moment

W = design area load l2,ln

l_n = clear span extending from face to face of columns, capitals, brackets or walls, but not less than 0.65 l1;

l_1 = length of span in the direction of M_0; and

l_2 = length of span transverse to 11.

Z = Seismic zone factor, as per IS-1893-(I) 2002

I = Importance factor as per IS-1893-(I) 2002

R = Response Reduction Factor, as per IS-1893-(I) 2002

Sa/g = Average response acceleration coefficient

A_h = Design horizontal seismic coefficient

K_c = Stiffness of column

K_s = stiffness of slab

α_c = Ratio of K_c and K_s

W_d, W_l = design dead and live loads respectively, per unit area

τ_c = Permissible shear stress

τ_v = Shear stress

CHAPTER-1

OBJECTIVE AND SCOPE

1.1 Objective

The objective of this study is to study flat plate structural system under lateral loads and compare its behavior with moment resisting frame and shear wall system.

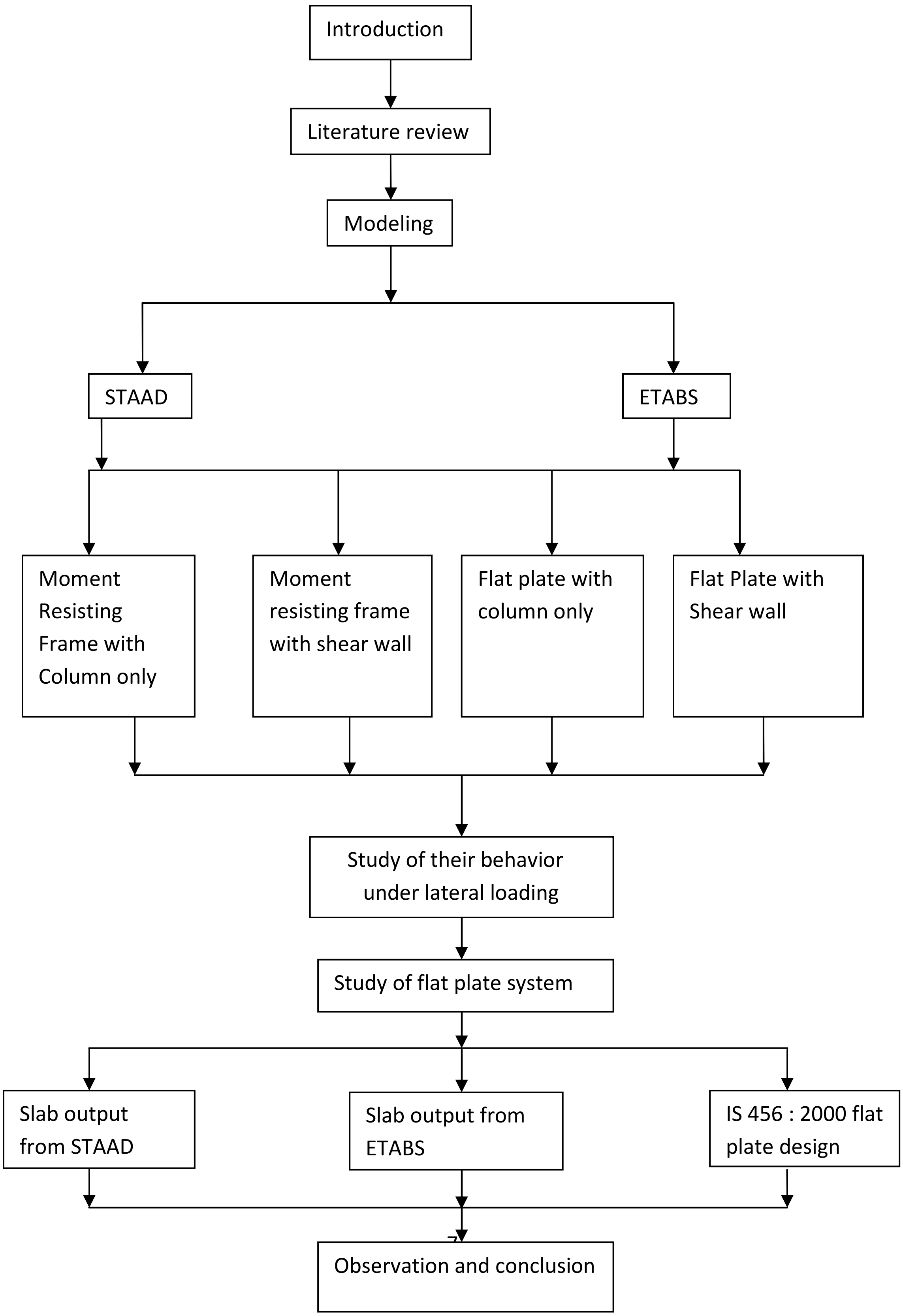
1.2 SCOPE:

Following is the scope of the present study to achieve the above objective:

1. Modeling a twenty storied structure with flat plate system with columns and shear walls simulated by finite elements in STAAD-pro software.
2. Modeling the above structure with beam elements and columns and shear walls and comparing their behavior and outputs with the above stated flat plate system.
3. Modeling a twenty storey structure with special moment resisting frame and shear walls and comparing with equivalent flat plate structure.
4. Modeling the above stated model in ETABS software and studying their behavior in ETABS.
5. Studying the problems involved in modeling of flat plate system which are deficiency of stiffness to counter lateral forces and transfer of moments and shear between columns and adjacent slabs.

CHAPTER 2

WORK FLOWCHART:



CHAPTER-3

INTRODUCTION

Real Estate industry is undergoing a revolutionary change for the past decade. Traditional masonry structures are only seen in three tier cities, sub urban areas or rural areas. There are many reasons which attribute to this dynamic change in the society. One of the reasons is growing awareness about seismic resistant structures.

This change has also resulted in adopting newer and more innovative approach over the orthodox building procedures and systems. Due to more and more demands in this industry builders and engineers have to go for much greater heights. This has resulted in newer problems for engineers as catering to wind loads and strong seismic forces along with economy and safety is turning out to be a massive challenge which engineers are facing at present.

Traditional Moment resisting frames structures is economical and cost effective for up to 15 storied structures only. Above 17- 18 stories this system does not give cost effective designs.

Shear wall structure has the answer to the above stated problem, it proves to be economical and gives a cost effective design for buildings up to the height of 30 stories. Shear walls have the ability to absorb strong lateral force and impart stiffness to the structure against lateral forces. Shear walls are used with special moment resisting frames or with ordinary moment resisting frames based on the positions, plan area of the structure or the number of shear walls that have been provided in the plan area.

Though shear wall system does have the ability to give an economic design, but due to the growing demands and competition in the market, there is a need to go for a yet another design which gives still better results as far as economy and speed of construction is concerned.

Therefore, the newest trend in the real estate industry that is being looked upon as an answer to the above stated problems is the flat plate system.

Flat plate system comprises of slab and column or shear walls as the case may be. This is similar to flat slab system but in this case there are no drop panels at junction of slab and columns which aid to shear capacity and moment transfer from columns to adjacent slabs. Use of flat plate system has a number of benefits over beam columns system or flat slab system.

1. The ease of the construction of formwork.

2. The ease of placement of flexural reinforcement.

3. The ease of casting concrete.

4. The free space for water, air pipes, etc between slab and a possible furred ceiling.

5. The free placing of walls in ground plan.

6. The use of cost effective pre-stressing methods for long spans in order to reduce Slab thickness and deflections as also the time needed to remove the formwork.

7. The reduction of building height in multi-storey structures by saving one storey height in every six story thanks to the elimination of the beam height.

These structural systems seem to attract global interest due to their advantages.

In the present paper we will be studying the response of flat plate system under lateral loads. Traditional moment resisting frames have the ability to absorb lateral forces by the rigid moment connections of beam column junction. In these structures column and beam impart their stiffness and by their combined action the lateral stiffness or in plan stiffness is enhanced. Due to this action reinforcement in beams at lower level slabs is more than that at higher levels because of the shear action at the lower levels. On the contrary, the in-plan stiffness of flat plate is very less as compared to beam column system. The thickness of slab in flat plate system is not more than 200-225mm. With this thickness there is very less lateral stiffness and all the stiffness that is present to resist lateral loads is provided by column only. Hence, there are chances of very heavy reinforcements to be provided in such structure. Also, as the slab does not have adequate thickness, they might not be capable of sustaining the unbalanced moments transferred to them. Sustaining punching shear is another important issue which needs to be catered to. It becomes a subject of paramount importance to check slabs under both these loadings i.e. shear and additional shear due to unbalanced moments. Indian code does not give a very clear analysis procedure for this, where shear and unbalanced moments can be properly analyzed at the position adjacent to column slab junction. ACI code has given clear procedure for design and check of slabs at such critical locations.

Another problem that needs the attention of engineers is the method of simulation of flat plate system. As it does not comprise of any beam member, finite element method seems only rational answer for modeling this structure. But, finite element method is not only tedious in terms simulation but the time taken by it for the analysis of the structure and for interpreting the result is very time taking which needs a lot of man hours and precision. Therefore, other alternative methods of modeling flat plate system also need to be studied as they may be able to give similar results as that of a finite element model and save lot of time and efforts.

The method which is widely used as an alternative is the equivalent frame method. In this paper the methods of modeling equivalent frame have been studied. A comparison between finite element model and equivalent frame model will also be shown to give the idea of the feasibility of such model.

Flat plate system along with columns only might not prove to be a very efficient system because of very low stiffness of the structure. This might result in very large deflections or uneconomical design of the structure. Use of shear wall along with flat plate system gives a much more efficient design which not only saves construction time but also gives a very economical result as the shear wall become the major lateral load resisting elements of this type of structure. A comparative study between flat plate and column structure and a flat plate and shear wall structure has also been done which supports the above notion.

CHAPTER-4

REVIEW OF LITERATURE

To provide a detailed review of the literature related to modeling of structures in its entirety would be difficult to address in this chapter. A brief review of previous studies on the study of flat plate system and its comparison with various other kinds of structure has been shown in brief in this section.

George E.Lelekakis, Athina T.Birda, Stergios A.Mitoulis, Theodoros A. Chrysanidis, Ioannis A.Tegos [1] of Aristotle University of Thessaloniki, Department of Civil Engineering, Greece presented a study on extended parametric investigation in order to identify the seismic response of structural systems consisting of

α) slabs-columns

b) columns-perimetric beams

c) columns shear walls-slabs

d) columns-shear walls-slabs and perimetric beams.

The mentioned systems were studied for all possible storey heights in Greece. The results of the above conditions were then matched with the compliance criteria in the Greek codes which govern the design of RCC high rise structure. Based on the compliance criteria different aspects such as torsion, capacity design and sensitivity of masonry wall were attributed to the different heights at which they could be applied.

Three characteristic cases,were examined in the present paper. These investigations have experimentally studied the response of flat-slab column connections under horizontal loading. The first experimental research, concerns the effect of variable slab loading, which produces punching shear stress in internal and external slab-column joints. The study was carried out by applying seismic loading to a model of two span flat-slab structure supported by columns. The three specimens, which included one internal and two external joints, were subjected to identical horizontal cyclic loading by increasing the target displacements, while each specimen had a different vertical slab loading. The experiments showed that the increase in the slab vertical loading leads to a dramatic reduction in the ability of carrying overturning moment and differential horizontal replacements.

The second experimental study, which concerned external joints, included 27 H-shaped specimens that were examined under seismic loading and conclusions were extracted regarding the effective width of the slab connected to the column. It was also concluded that, the ability of the specimens to develop deflections is strongly influenced by loading and more specifically higher values of dead loading Applications of flat-slab R/C structures in seismic regions 103 reduce significantly the aforementioned capability. It is noted that, the magnitude of the slab loading reflects on the value of punching shear force during an earthquake. This leads to the conclusion the structure should provide adequate resistance against punching shear in critical joints, which are overstressed during earthquake.

The third experimental investigation, which is obtained from Greek bibliography, concerns the deformability of internal slab-column joints under seismic loading. The results of the study lead to conclusions referring to the seismic behavior of these joints and particularly the inter-storey drifts of multi-storey 3D structural systems.

Analyses of structural systems have shown that fundamental period is not affected significantly neither by the density of the slab mesh nor by the use of diaphragm action.

A number of 36 models were analyzed using shell elements to model the slab and 36 models were analyzed using linear elements to model the slab. The models were the following:

Four single-storey systems with a basement (underground storey) and the rest of the models were multi-storey systems with a basement (underground storey) with heights varying from 5m to 29m. Models can be categorized according to their structural systems to:

a) Flat slab supported only by columns

b) Slab with perimetric only beams supported by columns

c) Flat slab supported by columns and shear walls

d) Slab with perimetric only beams supported by columns and shear walls.

In models in which shell elements were utilized, the slab was modeled using shell elements whilst vertical structural elements were modeled using linear beam elements.

The study came up to the following conclusions, concerning the total number of storeys which can be applied to each case:

a) Flat-slab systems with columns only can be applied under conditions buildings with a small number of storeys. However, the Greek codes' provisions, concerning the compulsory use of shear walls, lead to the conclusion that the implementation of such systems is restrained.

b) Flat-slab structural systems with perimetric beams supported only by columns, comply with both Greek codes' provisions, however in that case big cross sections for the columns is needed.

c) Flat-slab systems with shear walls can be applied for any number of storeys allowed in Greece, i.e. 9-storey buildings. The same conclusion, concerning the height of the structure, can be drawn for flat-slab systems with perimetric beams supported by columns and shear walls.

Hyun-Su Kim, Dong-Guen Lee [2] proposed an efficient analytical method in their study to obtain accurate results in significantly reduced computational time using the finite element approach. The proposed method employs super elements with fictitious beams. The stiffness degradation in the flat plate system considered in the equivalent frame method was taken into account by reducing the modulus of elasticity of floor slabs in the study. Static and dynamic analyses of example structures were performed and the efficiency and accuracy of the proposed method were verified by comparing the results with those of the refined finite element model and the equivalent frame method.

According to their study, the floor slabs are modeled using equivalent beams having effective width assuming that the equivalent beams have the same flexural stiffness as the floor slab system. The depth of the equivalent beams in the equivalent frame method was taken to be the thickness of floor slabs. The determination of the effective width for a slab is one of the most important procedures in the equivalent frame method and many researches have been performed on an effective width. The method proposed by Jacob S. Grossman for the determination of the effective width is one of the methods widely used in practical engineering. Grossman proposed an improved method to account for the degradation of the stiffness of the slabs depending on the level of the lateral drifts by introducing the stiffness degradation factor based on the tests performed at U.C. Berkeley. In the work by Grossman, it was difficult to account for the stiffness degradation in the slabs depending on the lateral drifts in the finite element method. However, it became feasible to include the stiffness degradation effect by adjusting the modulus of elasticity of the slabs in the finite element approach to have a similar effect as using the equivalent beams with the effective width in the equivalent frame method.

Grossman method for effective width determination

Various studies on the resistance capacity for the lateral loads were performed by previous researchers. Grossman concluded that the flat plate system has a good resistance capacity for the lateral loads as well as gravity loads provided a proper detailing in the joint between the column and the slab through the reviews of previous researches. And a new formula for the effective width was proposed by Grossman.

$$\alpha l_2 = K_d [0.3 l_1 + C_1 (l_2 / l_1) + (C_2 - C_1)/2](d/0.9h)(K_{FP}) \quad (1)$$

with limits: $(0.2)(K_d)(K_{FP})l_2 \leq \alpha l_2 \leq (0.5)(K_d)(K_{FP})l_2$

where, α = equivalent width factor

αl_2 = effective width of slab at center line of support

K_d = factor considering degradation of stiffness of slabs at various lateral load levels

l_1 = length of span of supports in direction parallel to lateral load

l_2 = length of span of supports in direction transverse to lateral load

C_1 = size of support in direction parallel to lateral load

C_2 = size of support in direction transverse to lateral load

d = effective depth of slab

h = slab thickness

K_{FP} = factor adjusting αl_2 at edge exterior and corner supports

(1.0 for interior supports, 0.8 for exterior and edge supports, 0.6 for corner supports)

Limitations in the equivalent frame method

Equivalent frame method can be easily applied to a flat plate structure having rectangular plan. However, it is hard to apply the equivalent frame method to flat plate structures having irregular plans. In many cases, commercial buildings using the flat plate system usually have slabs with openings to accommodate escalators or equipments. It is difficult to apply the equivalent frame method to the structures having openings in the slab.

ANALYSIS OF FLAT PLATE STRUCTURES USING THE FINITE ELEMENT APPROACH

The structures having irregular types of plans with which the equivalent frame method has limitations in analysis can be analyzed without any difficulties by the finite element method. However, the stiffness degradation in the slab could not be considered in the finite analysis method as Grossman mentioned in his study. Finite element analyses of flat plate structures were performed including the stiffness degradation in the slab by using the reduced modulus of elasticity depending on the lateral drifts to investigate the possibility of using the finite element method to overcome the shortcomings of the equivalent frame method.

The stiffness degradation in the slab depending on the lateral drift

The stiffness degradation in the slab is usually remarkable in the case of flat plate structures subjected to lateral loads. Therefore, Grossman proposed the stiffness degradation factor (KD) that can reduce the effective width of the equivalent beams depending on the lateral drift in his study based on the tests performed by Prof. Moehle at U.C. Berkeley in 1990.

It can be noticed that the lateral stiffness of the structure predicted by the finite element method (FEM) is constant regardless of the lateral drift while that from the test performed at U.C. Berkeley(UCB) is reduced as the lateral drift increases. The equivalent frame method (EFM) proposed by Grossman shows a reduction in the lateral stiffness depending on the lateral drift in a similar way to the UCB because the effective width of slabs was reduced by the stiffness degradation factor.

If the lateral stiffness of the FEM model were reduced depending on the lateral drift in a similar manner to the stiffness degradation factor proposed by Grossman, the stiffness degradation in the slab could be accounted for by the finite element method. Therefore, the stiffness reduction factor (RK) was introduced to reduce the stiffness of the FEM model depending on the lateral drift by dividing the lateral stiffness of the EFM model (KEFM) by that of the FEM model (KFEM) shown in Fig. 3 as follows:

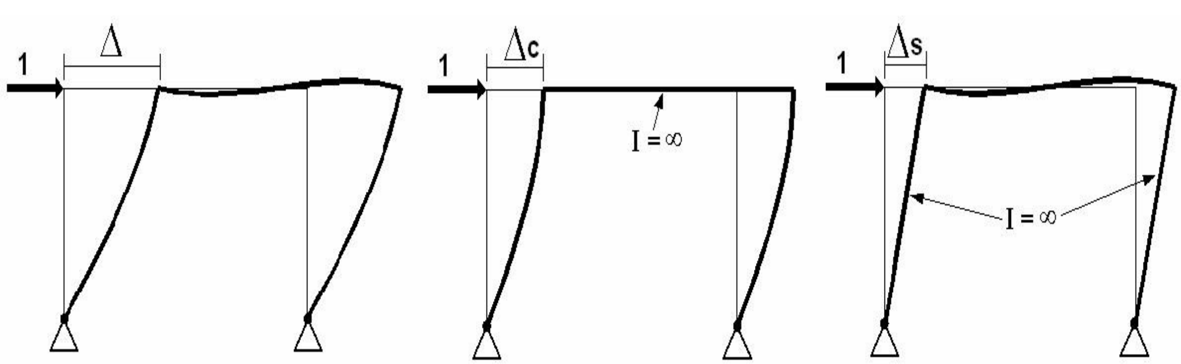
$R_K = K_{EFM}/K_{FEM}$

Stiffness reduction factor for slabs

The stiffness degradation in flat plate structures subjected to lateral loads may occur in columns as well as in slabs. However, the stiffness degradation in the column was not considered in this study on the purpose to compare with the results of the Grossman method since this study is focused on investigating an improved analytical method to overcome the limitations in the equivalent frame method by introducing the stiffness degradation in the slab in the finite element method. For the purpose of practical engineering, stiffness degradation in the column can be considered by using reduced stiffness properly. The lateral displacement(Δ) of the portal frame shown in Fig. 4 representing a simple flat plate structure can be decomposed into the displacement due to the column deformation(ΔC) and the slab deformation(ΔS) as follows:

$$\Delta = \Delta c + \Delta s \quad (3)$$

The columns are assumed to deform elastically while the slab has stiffness degradation. The lateral displacement of the structure (Δ / RK) can be decomposed into the displacement due to the column deformation (ΔC) and the slab deformation (ΔS / RKS) with stiffness degradation.



(a) Deformation of whole structure (b) Deformation of columns (c) Deformation of slabs

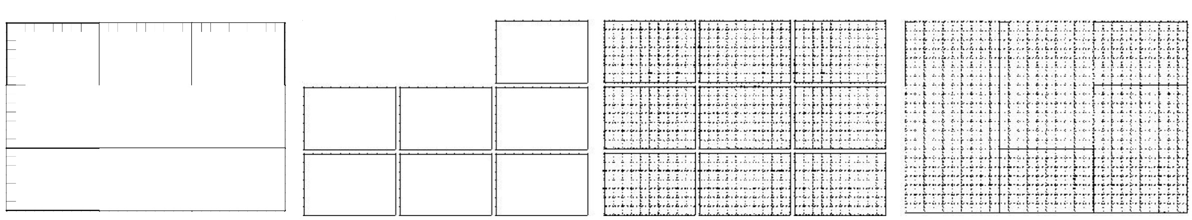
Figure 1: Showing comparison of deformation

SUPER ELEMENTS FOR ANALYSIS OF FLAT PLATE STRUCTURES

It is necessary to use a refined finite element model to represent openings in the floor slab with various shapes and sizes and represent the more accurate stress distribution in the slab. But if the entire flat plate structure were subdivided into a finer mesh with a large number of finite elements, it would cost a large amount of computational time and memory. Therefore, an efficient analytical method using super elements was proposed to save computational time and memory in this study.

Super element for flat plate structures

Most of the slabs can be divided by column lines in a rectangular subregion and the same slabs are repeatedly used in many floors in a flat plate structure. Thus it is very efficient to use super elements in the analytical model. The modeling procedure with super elements for the example structure shown in Fig. 2 is illustrated in Fig. 9. The refined mesh model of a typical flat plate system using many finite elements for the purpose of an accurate analysis is shown in Fig. 9(a). This refined mesh model can be separated into rectangular subregions of the slab having the same configuration as shown in Fig. 9(b). The node at the corners of the subregion is necessary for the connection between slabs and columns and the nodes at the boundary are to satisfy the compatibility condition at interface of subregions. Thus, all of the DOF's except those of the node at the boundary and corners can be eliminated by using the matrix condensation technique [18] for the efficiency of the analysis. And finally the super elements illustrated in Fig. 9(c) can be generated. Then the slab system in a floor is constructed by joining the active DOF`s of super elements as shown in Fig. 9(d). If the structural configurations are identical in many floors, the same assemblage of super elements can be used repeatedly in such floors for the convenience in the modeling of flat plate structures.



(a) (b) (c) (d)

Figure 2: Modeling procedure of flat slab system using super elements

The study came up to the following conclusions:
An improved analytical method that can consider the stiffness degradation effect in the slabs depending on the lateral drifts using super elements was proposed in this study for the efficient and accurate analysis of flat plate structures. The super elements and fictitious beams were used for the efficient analysis and the accuracy and the efficiency of the proposed method were investigated through the analysis of example structures. The major observations and findings could be summarized as follows:

1. The stiffness degradation in the flat plate system could be taken into account by the equivalent frame method for flat plate structures with regular plan. However, in the case of structures with irregular plan or slabs with openings, it is hard to use the equivalent frame method because of the difficulty in the determination of the effective width for the equivalent beams.

2. Structural analysis of a flat plate structure having irregular plan or slabs with openings can be performed and stress distribution of floor slabs can be easily represented using the finite element method

if the stiffness degradation in the slab could be considered properly.

3. he stiffness degradation in the flat plate system could be represented by the reduced modulus of elasticity of floor slabs in the finite element method. The modulus of elasticity was reduced based on the UCB test results in this study. However, any further research results regarding to the stiffness degradation in the slab can be used in the same manner for the proposed method.

4. The proposed method using super elements developed by introducing fictitious beams could reduce the computational time and memory significantly in the analyses. The static and dynamic analyses results by the proposed method were very similar to those of the refined mesh model in all cases of the example structures.

IIham Nurhuda*, Han Ay Lie of Universitas Diponegoro, Indonesia [3] presented a paper that showed the application of equivalent grid model to analyze flat plate structures three dimensionally. The effective grid width is analyzed empirically from experimental result. Structure analysis conducted by using both linear and nonlinear analysis.
Based on the understanding that in analysis of flat-plate we have to model plate as a unity supporting

load into two directions, the use of grid model to models flat-plate structure is studied in their work. The model of analysis shown in their work is used with the assumption that slab consists of grid formation that can distribute load to any direction. Effective distance of grid is examined by varying the distance between grid and compare the result of analysis to experiment result. Effect of number of grid to slab deflection is also observed by comparing numerical analysis to experiment.

The test slab is an idealized slab of a flat-plate floor at an intermediate level of a multistory office building. It has three bays in each direction with the center to center span are 4.6 m and 6.9 m for each direction. Slab thickness is 203 mm and story height is 3.0 m. Gravity loading consists of self-weight and live load. Design of structure subjected to lateral load is due to wind.

For experimental purpose the structure is scale to 0.4 from the original that is as follows: center to center spans in the two principal directions are 1.8 and 2.7m. Slab thickness is 81 mm, the columns extend 305 mm above the slab and 1220 mm below the slab. The column is pin supported to model inflection point of moment around column mid-height.

Modeling structure by Grid Model

For the numerical investigation, four grid models with different grid space and its dimension are made. Column is modeled as a frame element with its section properties not changes for all the four grid models. The illustration of the four grid models can be seen on fig 4 (a, b, c, d):

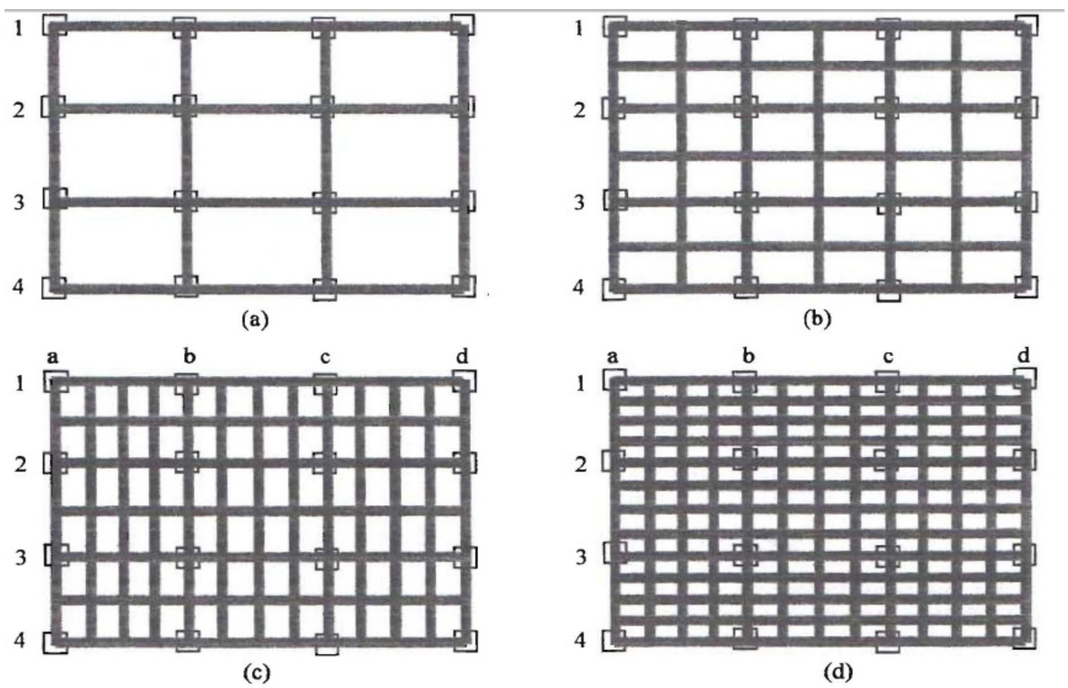


Figure 3: Showing different analysis methods for flat plate modeling

Analysis Method

Numerical analysis was governed by two methods of analysis that is linear analysis and material non-linearity analysis. Linear analysis was done to investigate the effective grid width. Nonlinear analyses that count for the material nonlinearity, was governed to observe inelastic behavior of structure after yield occur, how the models meet the experimental result. Behaviors of structure evaluated here are the relation of the load to lateral displacement, vertical deflection of slab, and shear stress distribution around column.

Conclusion

Based on the result of numerical analysis, the following conclusions are

1. The result of numerical simulation shows that the three-dimension of grid model can be used to model behavior of flat-plate structures.

2. According to the analysis result, there are relation among the effective grid width, grid space, and grid length from column to column. This analysis derives the variations of effective grid width as follows:

b =0.9.Lg.e ^(-Lg/L1)

L1 for the interior grid, and

b =0.45.Lg.e ^(-Lg/L1)

for the exterior,

Where b is the effective grid width, Lg is grid space and L 1 is grid length from column to column.

3. In this analysis, shear failure is avoided by designing the shear strength capacity almost 3 times of shear from gravitational load. It can be seen that by this way the structure perform good ductility.

5. Analysis shows that grid model able to reckon the shear force and meet the experimental result.

S. Teng, J.Z. Geng and H.K. Cheong [4] Nanyang Technological University, Singapore presented their work on strength of exterior slab-column connections.

Their work is based on the ACI 318-02 1 presentation of an eccentric shear stress model for predicting punching shear strength of slab-column connections with moment transfer. It assumes that the shear stresses due to unbalanced moment can be added directly to shear stresses due to shear force. The shear stresses due to unbalanced moment vary linearly along the critical section. The interaction between shear and moment transfer is represented by a coefficient Yv' which defines the fraction of unbalanced moment resisted by eccentric shear.

This paper begins with a summary of data obtained from numerous experiments on exterior slab column connections, including edge and corner connections. The eccentric shear stress model in the ACI 318-02 is reviewed. The predictions according to the ACI 318-02 for the collected data are analyzed and compared with the experimental results. Detailed discussions are provided and the interaction between shear and moment is studied and emphasized.

In their study, seventy-four exterior slab-column connections subjected to combined shear and moment transfer, tested by over 15 research centers around the world have been included. Of the 74 connection specimens, 46 are edge slab-column connections and 28 are corner connections.

ACI 318-02 for punching strength with moment transfer

According to the ACI 318-02, the punching shear strength of slabs without shear reinforcement can be determined from the lowest of the following expressions (in SI units).

$$v_c = 0.083 \times \left(2 + \frac{4}{\beta}\right)\sqrt{f_c'} \quad \text{(MPa)}$$

$$v_c = 0.083 \times \left(\frac{\alpha_s d}{b_o} + 2\right)\sqrt{f_c'} \quad \text{(MPa)}$$

$$v_c = 0.083 \times 4\sqrt{f_c'} \quad \text{(MPa)}$$

Where, β is the ratio of the longer side to the shorter side of the concentrated load (or columns), as is 40 for interior column, 30 for edge columns, and 20 for corner columns. bo is the length of critical shear perimeter taken at a distance of 0.5d away from the column face and has square corners for square columns and round shapes for circular columns. d is the effective depth of slabs. fc' is specific concrete cylinder strength, in MPa unit.

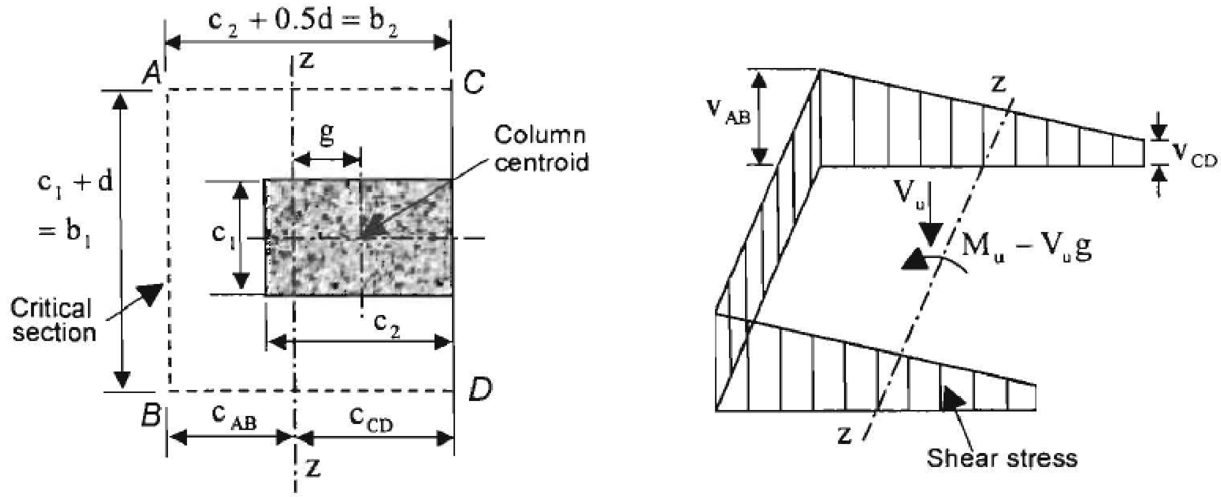


Figure 4: Showing Shear diagram for edge column

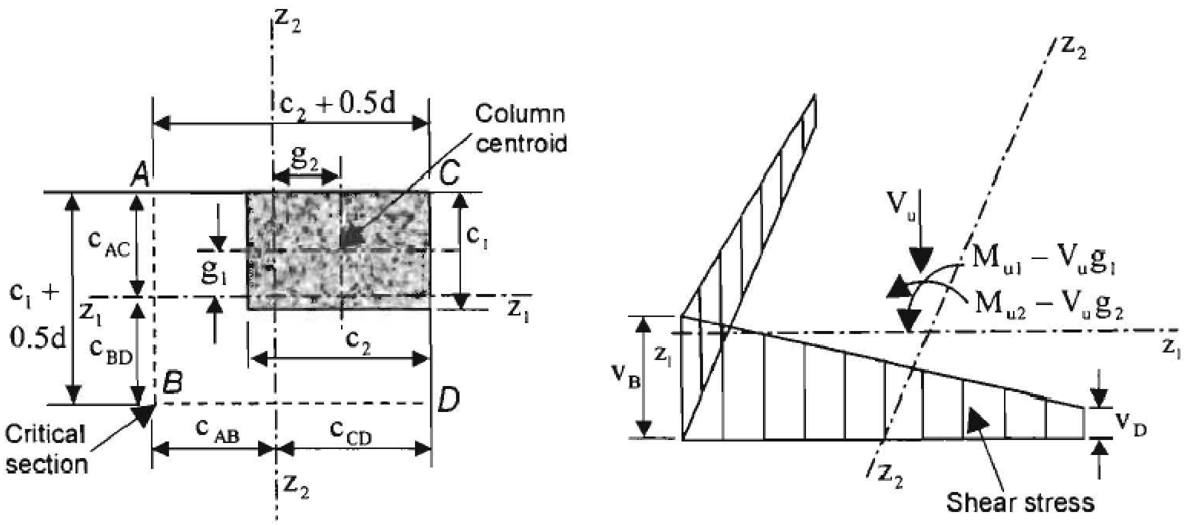


Figure 5: Showing shear diagram for Corner Column

The ACI 318-02 presents an analytical method (eccentric shear stress model) to calculate the shear stress when both shear force and unbalanced moment are transferred. It assumes that the shear stresses on the critical section due to the direct shear force can be added to the shear stresses on the same section due to moment transfer. The shear stress due to unbalanced moment is distributed linearly on the critical section. The critical ratio between measured and computed strength for edge connections is the maximum value of three ratios : V_{AB} / Vc, V_{CD} / Vc and (1- Ƴv)X(Mu- Vug}/M f ' where, V_{AB} is the shear stress along critical section AB as shown in Fig. 4; V_{CD} is the shear stress along critical section CD; Ƴv is the fraction of unbalanced moment resisted by shear; (Mu - Vug) is the ultimate unbalanced moment acting at the centroid of the slab critical section; g is the distance between centroids of the slab critical section and the column critical section; Mf is the flexural strength of slab reinforcement with a transfer width of
c1 + 3h .

The critical ratio between measured and computed strength for corner connections is the maximum value of three ratios: V_B/V_c , V_C / V_c ' and flexural strength ratio, similar to that for edge connections, where, VB is the shear stress at Point B; v c is the shear stress at Point C as shown in Fig. 5.

Edge slab-Column connection

According to ACI 318-02, analysis of the data collected reveals that calculated strength is governed by limiting shear stresses on the slab critical section rather than flexural yield for nearly all the test specimens.

Calculated strengths are almost in all cases conservative, with ratios between measured and calculated strengths ranging from 0.807 to 2.546, except four specimens, having a mean of 1.464 and a coefficient of variation of 0.286. It is interesting to note that the calculated strengths are still governed by the limiting shear stress on the critical section, not by the flexural yielding.

Their work suggested that there is no interaction between shear and moment for edge connections based on the analysis of 27 data they collected. The strong interaction between shear and moment embodied in the ACI 318-02 is the coefficient of Υv (the fraction of unbalanced moment transferred by shear).

Corner slab-Column connection

According to ACI 318-02, analysis of the data collected reveals that calculated strength is governed by limiting shear stresses on the slab critical section rather than flexural yield for all the test specimens. Calculated strengths are in all cases conservative, with ratios between measured and calculated strengths ranging from 1.067 to 3.441. Over-conservativeness and scattered trend of the data is found to occur in part because the analytical model assumes a significant interaction between shear and moment as we discussed in the previous section, which is embodied by the coefficient Υv as defined in the ACI 318-02. Analytical work has been done similar to that for edge connections to see how the predictions go by reducing this coefficient Υv step by step.

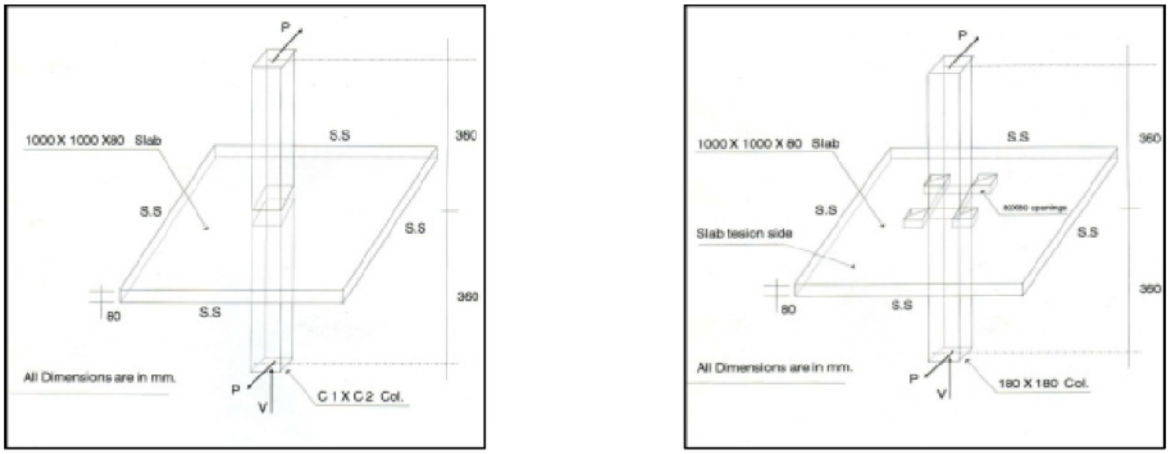
Conclusions

The conclusion of their work is as follows.

For exterior connections the interaction between shear and moment is not as strong as expected. The interaction between shear and moment is even weaker for corner connections than for edge connections. A 60 percent of ACI defined Υv value should be used for edge connections, and 10 percent of that value should be used for corner connections only. Once the reduced value of Υv is used in the ACI 318-02, the accuracy of the strength prediction for exterior slab-column connections can be improved greatly.

Omar M. Ben-Sasi [5], Civil Engineering Department, University of Misurata, Misurata, Libya presented his work on Tests of interior flat slab-column connections transferring shear force and moment.

According to his work, An experimental program was designed to study the effect on the behavior and ultimate strength of interior slab-column connections by testing to failure six specimens. The specimens consisted of two column stubs cast monolithically with the slab part that is assumed approximately bounded by the lines of contra-flexure of a prototype flat plate structure. The slab dimensions adopted for half scale of usual prototype dimensions were 100x100x8 cm (Fig. 6). Half scaling was for reasons of economy and ease of construction and handling of specimens.



(a) (b)

Figure 6: Showing column panel for Flat Plate

The concrete used for all connection specimens consisted of ordinary Portland cement, sand, and coarse aggregate of 12 mm-maximum size.

All slabs were reinforced the same with equal top and bottom mats consisted of 13ϕ10 mm steel bars both ways of 335 MPa measured yield strength. This corresponds to a steel ratio of about 0.014, a commonly used under-balanced steel ratio.

The longitudinal reinforcement for column stubs consisted of 8 bars 12mm in diameter. The ties were of 6mm-diameter bars spaced at 50 mm. The columns were made so strong in order to avoid any possible column failure during loading of specimens.

Some selected bars of slab reinforcement were cleaned enough at some locations in the vicinity of column stubs. Electrical strain gages were then carefully fixed onto these locations by applying a special epoxy and then wired for later connection to a digital strain indicator during testing. In order to avoid exposure to any moisture during contact to fresh concrete, the gages were wrapped carefully with an insulating tape. The strain indicator used for recording steel strains during specimen testing was accurate to one micro-strain.

A demec dial gauge was used for measuring concrete strains by mounting it on special stainless steel discs already pasted onto concrete surface at certain location of the slab. Demec gauge used was accurate to 0.00002 strain in a gage length of 50mm. Dial gauges of 0.01mm accuracy were used for measuring column stub axial and lateral displacement as load was increased. Lateral displacement was converted to column stub rotation upon dividing by column stub length.

Two dial gages were located very close to the column stub on opposite sides of the slab to help in detecting diagonal shear cracking by sudden increases in their deflections which were expected to occur.

GENERAL BEHAVIOR OF SPECIMENS UNDER LOADING

Upon loading of specimens, the first crack appeared for all specimens was a flexural crack at the intersection line between the slab and the column stub which lies in a direction normal to moment plane of action. This was noticed at a load level of about 20% of ultimate load for almost all specimens. The said crack turned to have the maximum width relative to other cracks that occurred later on.

As the loading was increased it widened up and turned around the nearby column stub corners and then headed in a direction making an angle of about 45º with the column side. This was clearly due to the torsion effect of the applied moment on the side slab strip when the moment transferred into the slab. This inclined torsion crack occurred at a load value of approximately 30% 0f ultimate load. As loading was further increased more flexural and torsional cracks appeared on slab tension side especially within the region close to the column stub.

Evidence of occurrence of diagonal shear cracking was noticed by a sudden increase in the deflections of the two opposite gages located near column stub, one on the slab tension side and another on the compression side. The diagonal shear cracking for most specimens was noticed to take place at a load level of about 70% of ultimate load.

Further increase in loading caused the flexural and torsion cracks to widen noticeably especially the firstly appeared ones and the tension steel crossing column face yielded. Ultimately the column stub punched through the slab secondary phenomenon to flexural steel yielding for the tested specimens.

CONCLUSIONS

1. The punching strength of an interior slab-column connection increases as the column aspect ratio c1/c2 increases whereas it decreases as the ratio of moment to shear force increases.
2. The shear stress resistance at the slab-column connection decreases as c/d ratio increases.
3. The slab region at the column corners contributes significantly to the shear capacity of a slab-column connection through the confinement it offers therein.

Fayazuddin Ahmed Syed, B. Dean Kumar, Y. Chandrasekhar, B.L.P. Swami [6] presented their work on Comparative Analysis of Flat Plate Multistoried Frames With and Without Shear Walls under Wind Loads. In the paper presented by them numerical studies for 20,40,60,80 storied for frames with normal conventional beam supported slab system, flat plate floor system, flat plate floor system with Shear walls has been conducted. A Comparison the Critical Column Axial Forces, Column moments, Lateral Drift (in mm) due to static and wind loads on the structures located at Hyderabad at a basic wind speed of 44 m/s has been observed during analysis.

The work is based on the fact that Frame action provided by a flat slab–beam and column interaction is generally insufficient to provide the required strength and stiffness for buildings taller than about 10 stories. A system consisting of shear walls, R.C. Infill Walls and flat Plate-frames may provide an appropriate lateral bracing system. Walls can be designed as plain concrete walls when there is only compression with no tension in the section. Otherwise they should be designed as reinforced concrete walls. Shear walls are specially designed structural walls incorporated in building to resist lateral forces that are produced in the plane of the wall due to wind, earthquake forces. In an earthquake, heavy wind affected prone zones these masonry infill wall panels attract large lateral forces and are damaged, or the perimeter columns, beams, and their connections fails. It is always advisable to incorporate them in buildings built in regions likely to experience earthquake of large intensity or high winds. They are usually provided between columns, in stairwells left wells, toilets, utility shafts, etc. Their thickness can as low as 150mm, or as high as 400mm in high rise buildings. Shear walls are usually provided along length and width of building. Shear walls are like vertically oriented wide beams that carry earthquake loads downwards to the foundation.

In the present investigation 20, 40, 60 and 80 storied frames were analyzed using STAAD.PRO (V8i) software package. For each of the frame under consideration, three cases were considered. Case I: Normal conventional beam supported R.C. framed structure. Case II: Flat Plate R.C framed structure. Case III: Flat Plate R.C. framed structure with Shear walls. All the three cases were considered for comparison with respect to the height of the structures. Comparisons was made with the Critical Column Axial Forces, Critical Column moments due to static and wind loads, Lateral Drift (in mm) in X, Z direction due to Wind loads. And also Strip moments (in Bending) in Flat Plates in both the Gravity and Wind loads on the structures located at Hyderabad at a basic wind speed of 44 m/s has been observed. The total height of the frames considered for the study are of 62m, 124m, 186m, 248m respectively, which represents a 20,40,60,80 storied commercial building. The Plan area of the Structure is 45 m x 30 m with columns spaced at 6 m from center to center in Z-direction, 5 m from center to center in X-direction. The height of each storey is 3.10m and all the floors are considered as Typical Floors. The location of the building is assumed to be at Hyderabad. In the present investigation 20, 40, 60 and 80 storied frames were analyzed using STAAD.PRO (V8i) software package. For each of the frame under consideration, three cases were considered. Case I: Normal conventional beam supported R.C. framed structure. Case II: Flat Plate R.C framed structure. Case III: Flat Plate R.C. framed structure with Shear walls. All the three cases were considered for comparison with respect to the height of the structures. Comparisons was made with the Critical Column Axial Forces, Critical Column moments due to static and wind loads, Lateral Drift (in mm) in X, Z direction due to Wind loads. And also Strip moments (in Bending) in Flat Plates in both the Gravity and Wind loads on the structures located at Hyderabad at a basic wind speed of 44 m/s has been observed. The total height of the frames considered for the study are of 62m, 124m, 186m, 248m respectively, which represents a 20,40,60,80 storied

commercial building. The Plan area of the Structure is 45 m x 30 m with columns spaced at 6 m from center to center in Z-direction, 5 m from center to center in X-direction. The height of each storey is 3.10m and all the floors are considered as Typical Floors.

Conclusion

1. In the case of flat plate floor system (F.P.F.S) though there are no partition walls, peripheral masonry walls are available and it doesn't take part in the interaction. As such the wall loads are directly transferred to the beams and are carried by the columns to foundation and earth mass.
2. As such the axial forces are more in flat plate floor system (F.P.F.S) compared to system with flat plate floor system with Shear walls (F.P.F.S+SW), this says that Axial forces in columns were reduced from ground floor to top floor because of the presence of shear wall in buildings.
3. It is observed that due to wind loading the column moments for flat plate floor system building are increased by 55% compared to conventional building, where as the column moments for flat plate floor system building with Shear walls has decreased by 69.17 % & 58.2 % when compared with flat floor system, conventional beam supported slab system.
4. The Shear walls with flat plates contribute towards reducing the column axial force even in the middle frame region also. In the case of other building frames there is similar reduction in column axial force when wind is acting.
5. The flat plate floor system helps in reducing the drift in the case of multi storied building compared to conventional beam slab column system. There is 38.81% reduction in the drift in the case of flat plate floor system when compared to conventional beam supported slab system.
6. The flat plate floor system can be further strengthened against the lateral loads by providing Shear walls also. The drift becomes minimum, so that there is 65.77% reduction in the drift in this case.
7. Hence it is clear that flat plate floor system helps in reducing the drift in the case of multi storied building compared to conventional beam slab column system and among these two cases the best choice is flat plate floor system with shear wall
8. It is recommended that provision of flat plate system with Shear walls (or) infilled walls is the best choice to safeguard the building against the lateral loads.

CHAPTER-5

SIMULATION AND ANALYSIS OF FLAT PLATE AND MOMENT RESISTING FRAME STRUCTURE.

Flat plate structural system is an innovative structural system which is being widely acknowledged and chosen over other conventional structural systems. This is because of a number of advantages that this system has over other conventional methods. The advantages are:

1. The ease of the construction of formwork because of the uniform thickness of slabs across the plan area, the time taken in fabrication and erection of formwork is drastically reduced.

2. The ease of placement of flexural reinforcement. This is also attributed to the uniform thickness as the reinforcement steel has to be provided within a uniform thickness only it makes it easier and fast to put the reinforcement bars in place.

3. The ease of casting concrete.

4. The free space for water, air pipes, etc between slab and a possible furred ceiling. As the thickness of slab is uniform there is no need of provisions for ducts and pipes.

5. The free placing of walls in ground plan. Absence of beam in plan makes it easier for the occupants to construct wall according to their ease.

6. The use of cost effective pre-stressing methods for long spans in order to reduce Slab thickness and deflections as also the time needed to remove the formwork.

7. The reduction of building height in multi-storey structures by saving one storey height in every six story thanks to the elimination of the beam height.

A typical flat plate system consists of columns and slabs only with no drops or beams to be provided at the junction of columns and slabs. Some key features of this kind of structures are as following:

1. In this kind of structure, the reinforcement bars for gravity loads as well as for lateral loads are placed within the slab depth only.
2. The thickness of slab considered for such structures are higher than the conventional beam column frame. The slabs are of thickness ranging from 175mm to 300mm based on the plan area, column locations and height of the structure.
3. When compared with the traditional RC moment resisting frame, these structures have larger deflections but as RC frames prove uneconomical over 15 -16 stories, use of flat plate system gives a more economical result.
4. As there are no lateral beams or additional drop panels in slabs, only slab provides lateral stiffness to the structure which is often not enough to restrain the structure from large deflections. Therefore, it is appropriate to model flat plate structures with shear wall as shear walls become the lateral force resisting elements and the slabs do not have to cater to the lateral stiffness alone. This leads to a much economical design also.
5. Due to absence of beam or drops at column slab junction, this location becomes very important to be checked under punching shear criteria, shear criteria and with regard to flexural requirements at column face.
6. Under gravity loads only, usually the slab thickness is sufficient to cater to the shear and flexural requirements of the structure.
7. But as the height of the structure increases, lateral loads become governing and detrimental for the structure. It is under these circumstances that the slab column junction becomes critical and needs a thorough check under punching shear criteria, shear criteria, flexural reinforcement and deflection check.
8. As the height of the structure increases, under wind loading and seismic loading, additional moments are induced in columns and since there are no beams or drop panels present the moments generated in the columns need to be resisted by the slabs only.
9. These moments are often unbalanced moments as they are generated from asymmetrical lateral loading.
10. Indian code for design of RCC structures i.e. IS 456 does not give a clear methodology to calculate the stress induced in the slabs due to unbalanced moments transferred from columns. ACI 318-II has given procedures by which these unbalanced moments are converted into shear stresses which are then used to design the adjacent slabs.

5.1 Methodology

In this project, we will be comparing flat plate structure with:

1. RC moment resisting frame with columns only.
2. RC moment resisting frame with shear walls and column configuration.

For the comparison of the two structural systems, a regular building of 25 storey has been considered. The storey height of the building is 3.1m. Thus, the total height of the structure is 77.5m. The bottom two stories have been taken as basements and no stilt has been considered in the building. Plan dimension of building is 25m X 15m.

Other important loading and design parameters have been considered as per the following:

Loads and forces

Loads and forces used for design shall be as defined in IS875, and is specified below.

The following type of loads and forces shall be considered.

- Dead load (DL)
- Live load (LL)
- Earthquake load (EQ)
- Wind load

Dead Load (DL)

Dead load is the load of the structure itself.

Following are the unit weight of major construction materials.

- Reinforced Cement Concrete 25.0 kN/m3
- Plain Cement Concrete 24.0 kN/ m3
- Structural Steel 78.5 kN/ m3
- Soil above water level 18.0 kN/ m3
- Masonary wall including plaster 20.0 kN/ m3

Floor finish margin of 50mm has been considered for design.

Live Load (LL)

Live load for building and structure shall be in accordance with IS875 part 2 unless otherwise specified. Live load of 300 Kg/sqm will be considered for the commercial building floors.

Seismic Load (EQ)

Seismic loads to be applied for structures shall be in accordance with the applicable provision of the IS 1893, 2002 and noted below.

- Seismic Zone Factor, Z, shall be 0.24
- Importance factor I, shall be 1.0
- Response reduction factor, shall be 5 for RCC structures (SMRF)
- Average response acceleration factor

  Ah = Z/2 x I/R x Sa/g

  E = Ah x W

  Where 'W' is seismic weight of structure with appropriate live load

Combination of loads

Concrete structural members constructed for ordinary plant installations shall be designed to have, at all sections, a calculated strength necessary to carry the following factored loads and forces.

- 1.5[DL + LL]
- 1.2[DL + LL + EQ]
- 1.5[DL + EQ]
- 0.9DL +1.5 EQ
- 1.2[DL + LL + EQ]
- 1.5[DL + EQ]
- 0.9DL +1.5 EQ

RCC GRADE: M25 for all structure except M40 for column, as per drawings in accordance with clause 6.0 (Table 5)of IS 456-2000 for all.

REINFT STEEL GRADE 415-500 N/sqmm: High yield strength deformed bars conforming to IS 1786.

5.2 Simulation in STAAD-Pro

5.2.1 Description of RC frame building Model in STAAD-Pro

The plan dimension o the building is 25m x 15m as discussed earlier.

Three column sizes have been taken which are:

600mm X 900 mm for interior columns

400 X 900 for edge columns

And 600 X 600 for corner columns

Beams of size 500 X 650 and 350 X 650 have been taken.

The building has been analysed in STAAD- Pro (V8i) software under the loading as described earlier. The building has been analysed with response spectrum analysis under dynamic earthquake loading as per IS 1893-I 2002. After the analysis of the structure the results of this structure has been compared with Flat plate structure with columns.

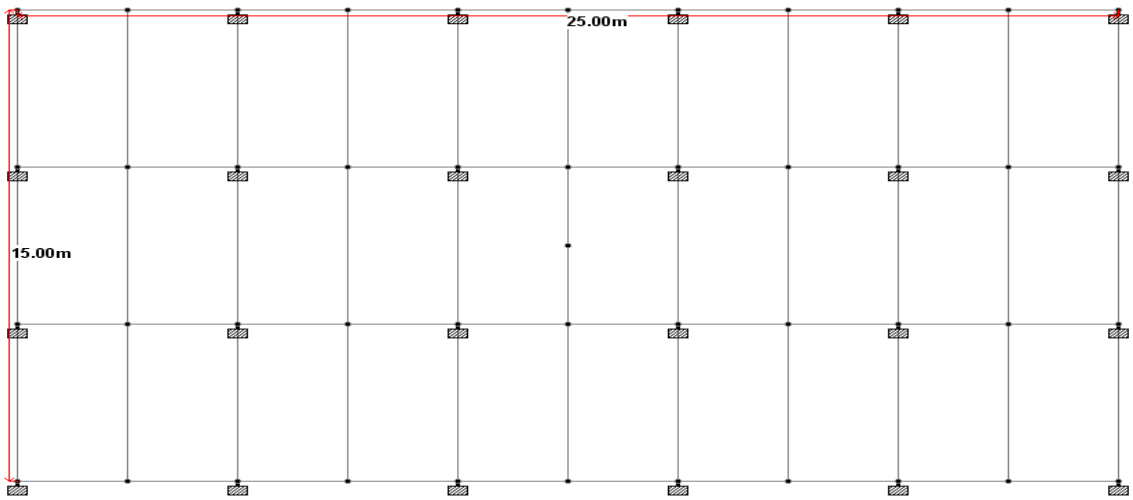


Figure 7: Plan view of RC frame STAAD Model

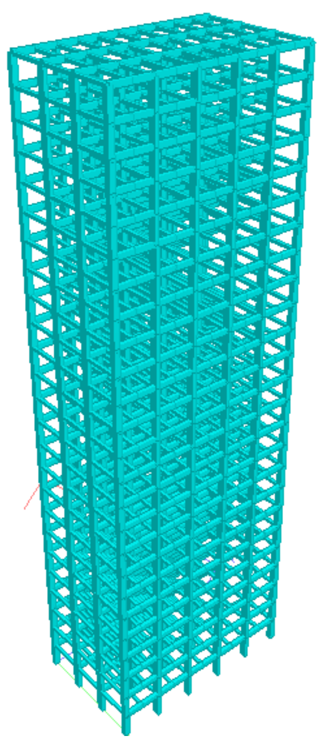


Figure 8: Orthogonal view of RC frame STAAD Model

5.2.2 Flat Plate model Corresponding to RC frame model

The above model has been compared by Flat plate structure of same height and plan dimension. For clarity, the column sizes are also the same as the above structure. The slab has been simulated as the plate element of STAAD-Pro software. Slab of thickness 200mm has been considered for analysis.

The flat plate structure is also subjected to dynamic seismic loading with response spectrum.

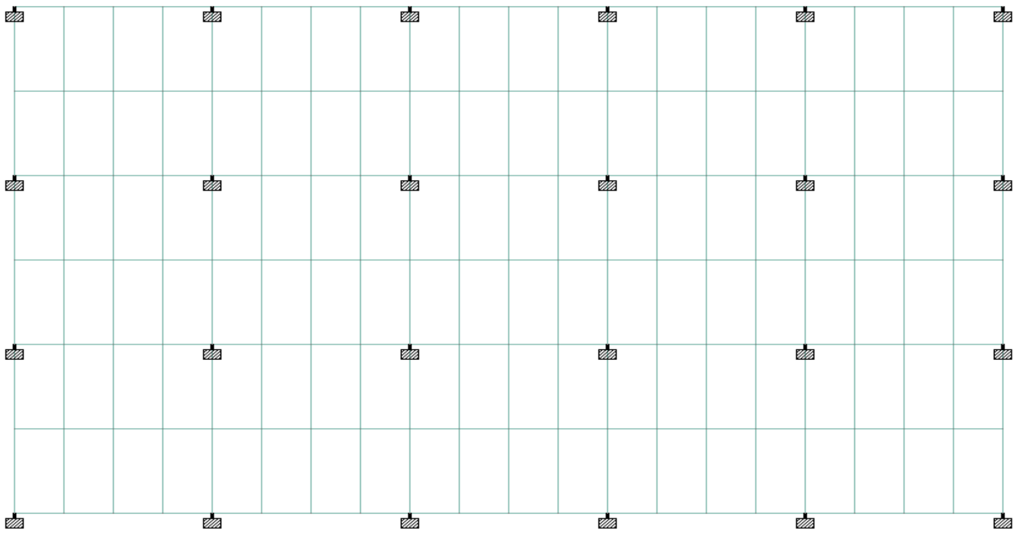


Figure 9: Plan view of flat plate model corresponding to RC frame Model in STAAD

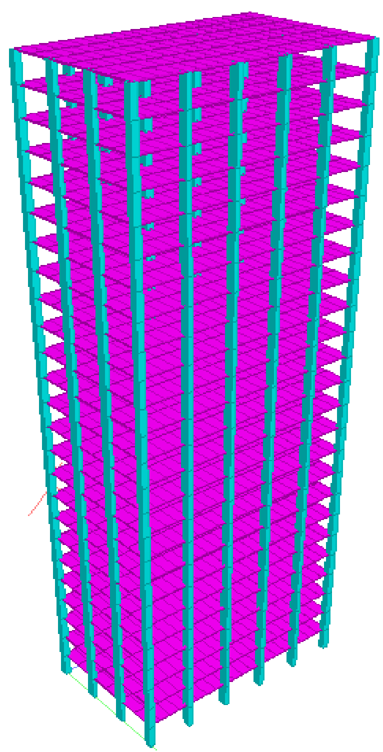


Figure 10: Orthogonal view of flat plate model corresponding to RC frame Model in STAAD

5.2.3 Description of RC frame building Model with Shear wall

The plan dimension o the building is 25m x 15m as discussed earlier.

Three column sizes have been taken which are:

600mm X 900 mm for interior columns

400 X 900 for edge columns

And 600 X 600 for corner columns

In addition to the above column sizes Shear walls have also been modelled as shown in Figure 11.

The dimensions of wall are 300mm X 5000 mm.

Beams of size 500 X 650 and 350 X 650 have been taken.

The building has been analysed in STAAD- Pro (V8i) software under the loading as described earlier. The building has been analysed with response spectrum analysis under dynamic earthquake loading as per IS 1893-I 2002. After the analysis of the structure the results of this structure has been compared with Flat plate structure with columns.

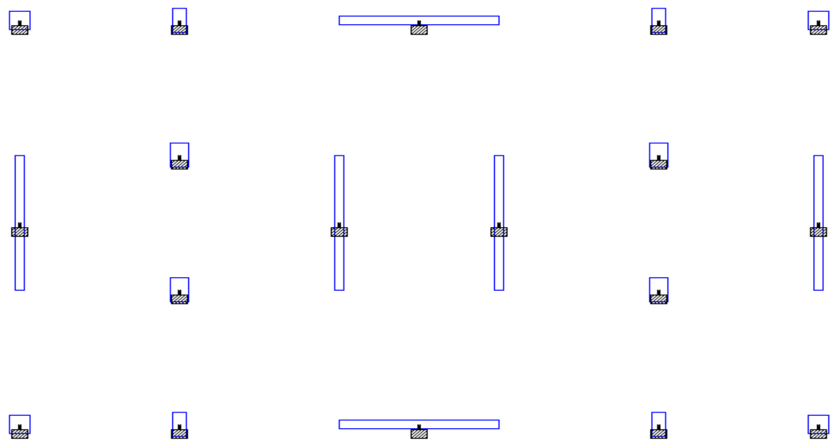


Figure 11: Plan view of RC frame model with Shear Wall in STAAD

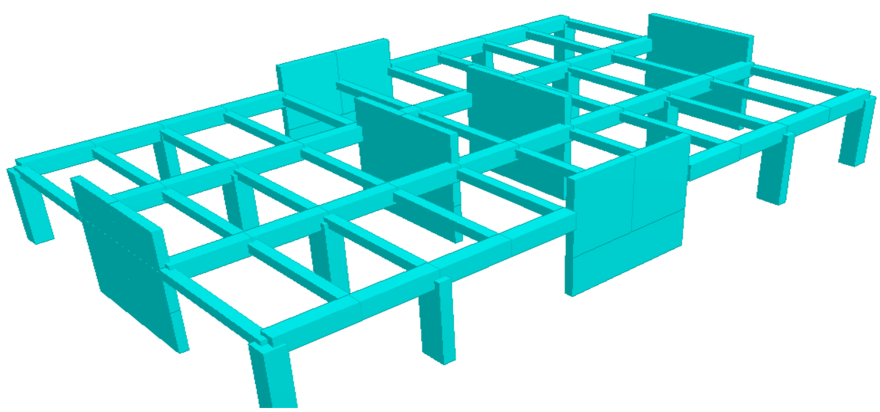


Figure 12: Orthogonal view of single floor of RC frame with Shear Wall in STAAD

5.2.4 Description of RC frame building Model with Shear wall

The above model has been compared by Flat plate structure of same height and plan dimension. For clarity, the column sizes are also the same as the above structure. Walls considered in above structure

have also been taken at same locations and with same sizes. The slab has been simulated as the plate element of STAAD-Pro software. Slab of thickness 200mm has been considered for analysis.
The flat plate structure is also subjected to dynamic seismic loading with response spectrum.

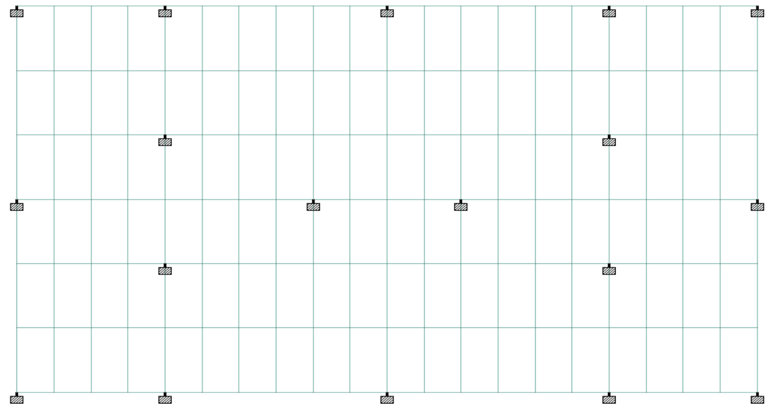


Figure 13: Plan view of Flat plate model with Shear wall in STAAD

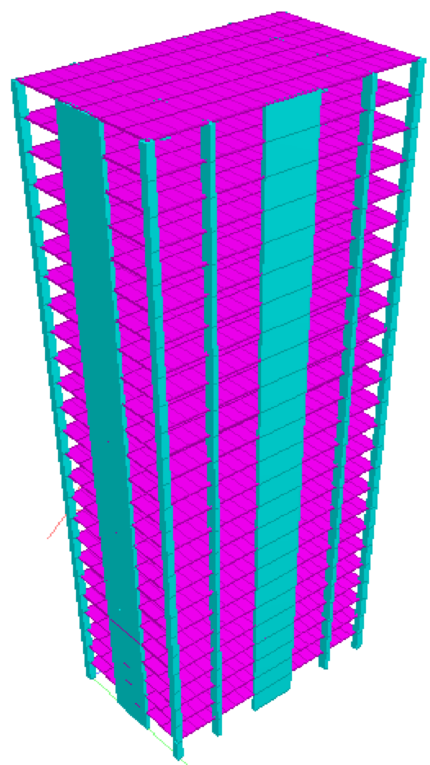


Figure 14: Orthogonal view of Flat plate with Shear wall model in STAAD

5.3 Simulation in ETABS

The models that have been simulated in STAAD pro have also been prepared in ETABS software which is new software from Computers and Structures Inc. This software is especially dedicated for the analysis of high rise buildings and is widely acclaimed to give better and more accurate results for model with shear walls and flat plate system under dynamic lateral loading.

The models have identical properties, plan dimensions, column and slab dimensions to that of STAAD models so that the comparison can be accurate.

5.3.1 Description of RC frame building Model

The plan dimension o the building is 25m x 15m as discussed earlier.

Three column sizes have been taken which are:

600mm X 900 mm for interior columns

400 X 900 for edge columns

And 600 X 600 for corner columns

Beams of size 500 X 650 and 350 X 650 have been taken.

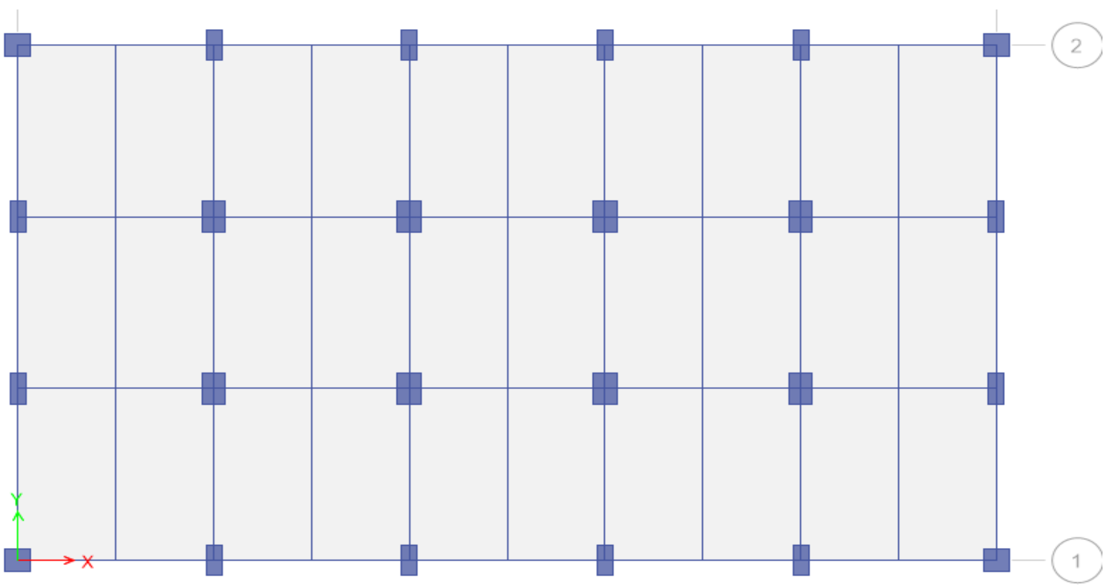


Figure 15: Plan view of RC frame ETABS Model

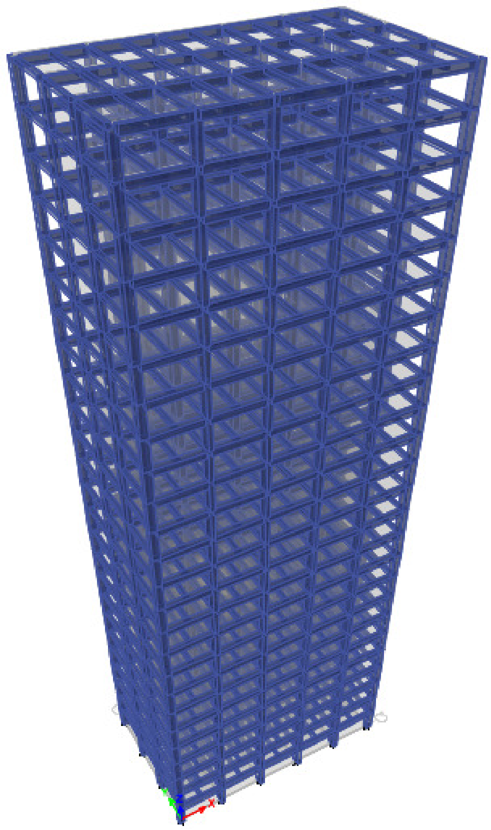


Figure 16: Orthogonal view of RC frame ETABS Model

5.3.2 Flat Plate model Corresponding to RC frame model

The above model has been compared by Flat plate structure of same height and plan dimension. For clarity, the column sizes are also the same as the above structure. The slab has been simulated as the thick shell element of ETABS software.

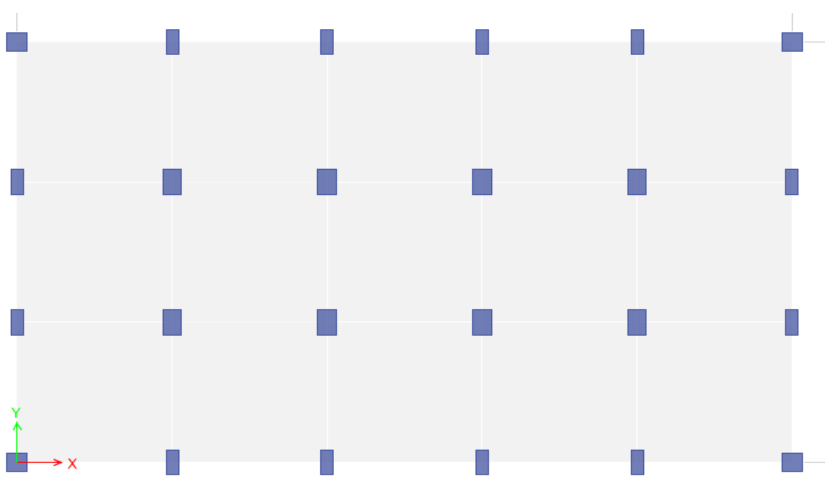


Figure 17: Plan view of flat plate model corresponding to RC frame Model in ETABS

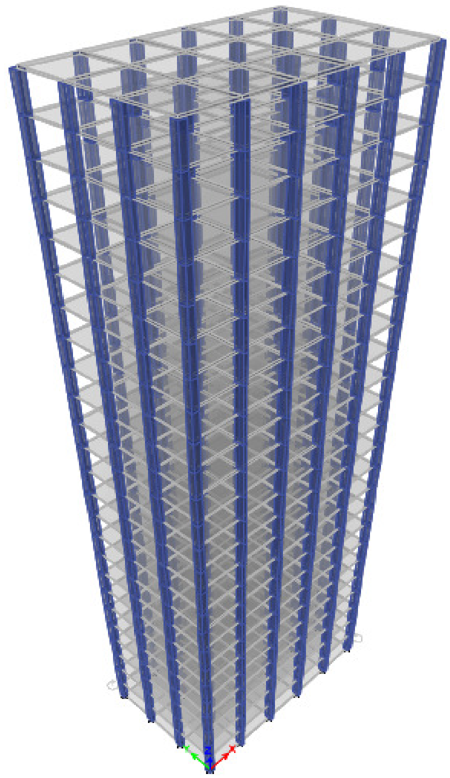


Figure 18: Orthogonal view of flat plate model corresponding to RC frame Model in ETABS

5.3.3 Description of RC frame building Model with Shear wall

The plan dimension o the building is 25m x 15m as discussed earlier.

Three column sizes have been taken which are:

600mm X 900 mm for interior columns, 400 X 900 for edge columns and 600 X 600 for corner columns

In addition to the above column sizes Shear walls have also been modelled as shown in Figure 11.

The dimensions of wall are 300mm X 5000 mm. Beams of size 500 X 650 and 350 X 650 have been taken.

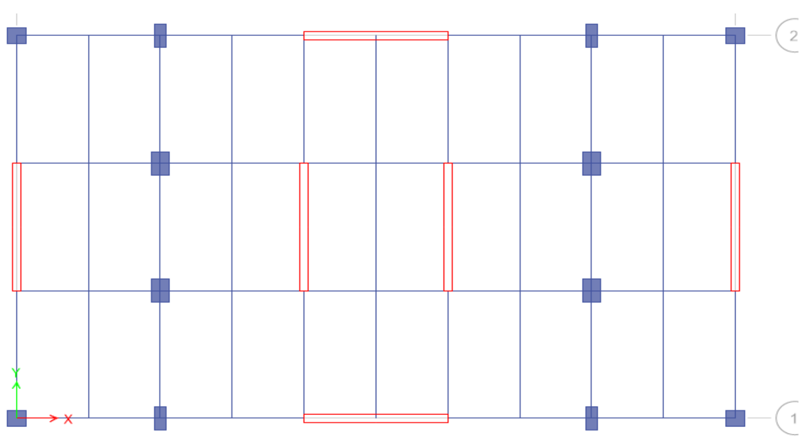


Figure 19: : Plan view of RC frame model with Shear Wall in ETABS

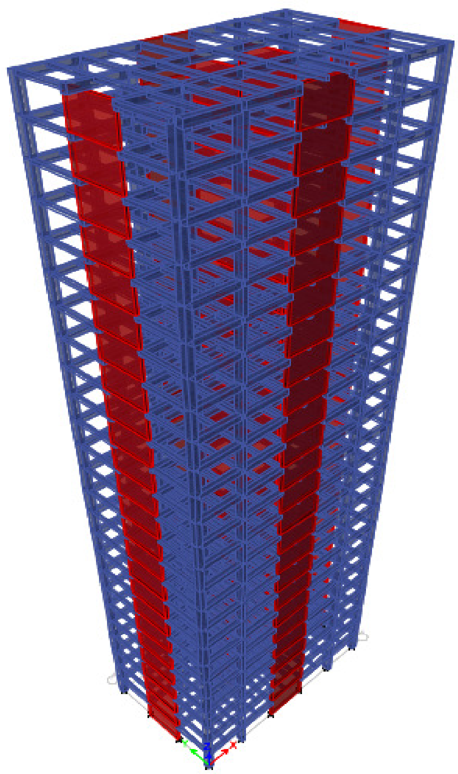


Figure 20: Orthogonal view of RC frame with Shear Wall in ETABS

5.3.4 Flat Plate model Corresponding to RC frame model with Shear wall

The above model has been compared by Flat plate structure of same height and plan dimension. For clarity, the column sizes are also the same as the above structure. Walls considered in above structure have also been taken at same locations and with same sizes. The slab has been simulated as the thick shell element of ETABS software. Slab of thickness 200mm has been considered for analysis.

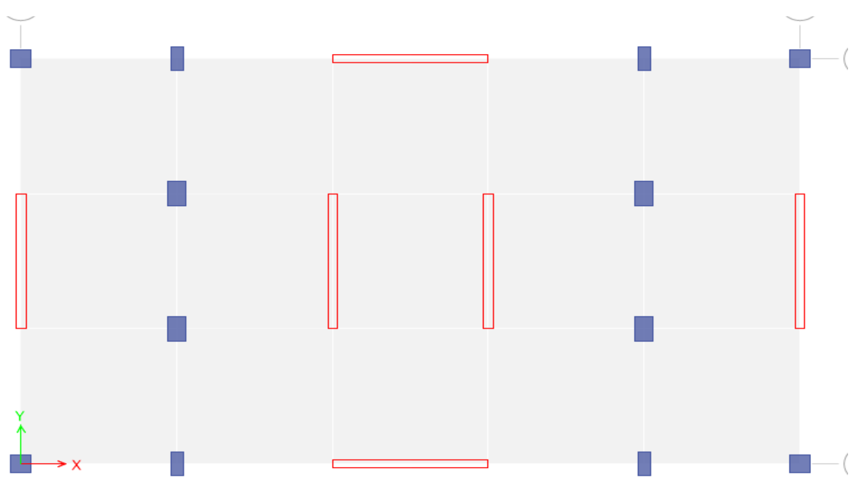


Figure 21: Plan view of Flat plate model with Shear wall in ETABS

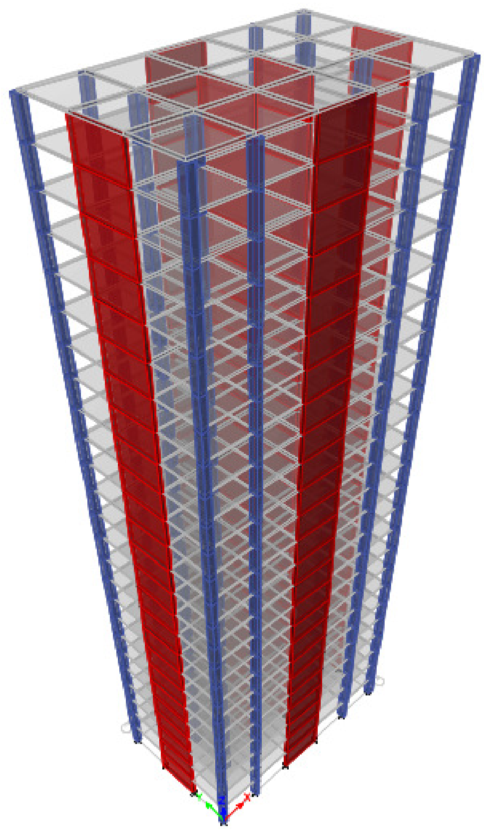


Figure 22: Orthogonal view of Flat plate with Shear wall model in ETABS

CHAPTER- 6

DESIGN OF FLAT SLAB

6.1 Design steps according to IS 456: 2000 [7]

IS 456 has given guidelines for design of flat slabs. As flat plate is a kind of flat slab which has no drops, Flat plates can also be designed for gravity loads as per IS 456 codal provisions.

Following are the design procedures as per Clause 31.

a) ***Column strip -****Column strip means a design strip having a width of 0.25 l2, but not greater than 0.25 l1, on each side of the column centre-line, where l1, is the span in the direction moments are being determined, measured centre to centre of supports and l2, is the-span transverse to l1, measured centre to centre of supports.*

Middle strip -Middle strip means a design strip bounded on each of its opposite sides by the column strip.

Panel-*Panel means that part of a slab bounded on-each of its four sides by the centre-line of a column or centre-lines of adjacent-spans.*

Drop *The drops when provided shall be rectangular in plan, and have a length in each direction not less than one-third of the panel length in that direction. For exterior panels, the width of drops at right angles to the non-continuous edge and measured from the centre-line of the columns shall be equal to one-half the width of drop for interior panels.*

Column Heads *Where column heads are provided, that portion of a column head which lies within the largest right circular cone or pyramid that has a vertex angle of 90"and can be included entirely within the outlines of the column and the column head, shall be considered for design purposes.*

Methods of Analysis and Design

It shall be permissible to design the slab system by one of the following methods:

a) The direct design method

b) The equivalent frame method

Direct Design Method

Limitations

Slab system designed by the direct design method shall fulfill the following conditions:

There shall be minimum of three continuous spans in each direction,

The panels shall be rectangular, and the ratio of the longer span to the shorter span within a panel shall not be greater than 2.0, *It shall be permissible to offset columns to a maximum of 10 percent of the span in the direction of the offset notwithstanding the provision in the above clause.*

The successive span lengths in each direction shall not differ by more than one-third of the longer span.

The end spans may be shorter but not longer than the interior spans, and

The design live load shall not exceed three times the design dead load.

Total Design Moment for a Span

In the direct design method, the total design moment for a span shall be determined for a strip bounded laterally by the centre-line of the panel on each side of the centre-line of the supports.

The absolute sum of the positive and average negative bending moments in each direction shall be taken as:

$M_0 = W*ln/8$ (1)

M_0, = *total moment*

W= design area load l2,ln

ln = clear span extending from face to face of columns, capitals, brackets or walls, but not less than 0.65 l1;

l1 = length of span in the direction of M0; and

l2 =length of span transverse to 11.

The negative design moment shall be located at the face of rectangular supports, circular supports being treated as square supports having the same area.

In an interior span, the total design moment M0, shall be distributed in the following proportions:

Negative design moment = 0.65

Positive design moment = 0.35

In an end span, the total design moment M_0 shall be distributed in the following proportions:

Interior negative design moment:

$0.75-0.1/(1+1/\alpha_c)$

Positive Design Moment

$0.63-0.28/(1+1/\alpha_c)$

Exterior negative design moment

$0.65/(1+1/\alpha_c)$

αc is the ratio of flexural stiffness of the exterior columns to the flexural stiffness of the slab at a joint taken in the direction moments are being determined and is given by

$\alpha c = \sum K_c/K_s$

It shall be permissible to modify these design moments by up to 10 percent, so long as the total design *moment, M_0, for the panel in the direction considered is not less than that required by (1)*

The negative moment section shall be designed to resist the larger of the two interior negative design moments determined for the spans framing into a common support unless an analysis is made to distribute the unbalanced moment in accordance with the stiffness of the adjoining parts.
Columns built integrally with the slab system shall be designed to-resist moments arising from loads on the slab system. (3)
At an interior support, the supporting members above and below the-slab shall be designed to resist the moment M given by the following equation, in direct proportion to their stiffnesses unless a general analysis is made:

M= 0.008((wd+0.5w1)l2 ln^2-wd'l2'ln') /(1+1/αc)* (3)

where
Wd,W1 = design dead and live loads respectively, per unit area;
l2 = length of span transverse to the direction of M0,
ln = length of the clear span in the direction of M, measured face to face of supports;
αc = ∑Kc/Ks

wd', l2', and ln', refer to the shorter span.

Calculation of Shear Stress
The critical section for shear shall be at a distance d/2 from the periphery of the column/capital/ drop panel, perpendicular to the plane of the slab where d is the effective depth of the. The shape in plan is geometrically similar to the support immediately below the slab .
The shear stress τ_v, shall be the sum of the values calculated according to following.
The nominal shear stress in flat slabs shall be taken as V/ b_0d where V is the shear force due to design load, b_0 is the periphery of the critical section and d is the effective depth.
When unbalanced gravity load, wind earthquake or other force cause transfer of bending moment between slab and column, a fraction (1-α) of the moment shall be considered transferred by eccentricity of the shear about the centroid of the critical section. Shear stress shall be take an varying linearly about the centroid of the critical section. The value of α shall be calculated from equation
. $\alpha = 1/(1+2/3*(a_1/a_2)^{0.5})$
Permissible Shear stress

When shear reinforcement is not provided, the calculated shear stress at the critical section shall not exceed K_s τ_c.
k, = (0.5 + βc) but not greater than 1, PC being the ratio of short side to long side of the column/ capital; and

$\tau_c = 0.25*(fck)^{0.5}$ *in limit state method of design.*

When the shear stress at the critical section exceeds the value given in K_s τ_c, but less than 1.5 τ_c, shear reinforcement shall be provided. If the shear stress exceeds 1.5 τ_c, the flat slab shall be redesigned. Shear stresses shall be investigated at successive sections more distant from the support and shear reinforcement shall be provided up to a section where the shear stress does not exceed 0.5 τ_c, While designing the shear reinforcement, the shear stress carried by the concrete shall be assumed to be 0.5 τ_c, and reinforcement shall carry the remaining shear.

6.2 DESIGN OF FLAT SLAB

(Under Gravity loading only)

[7] According to IS 456 : 2000

L1 (in direction of panel)	5	m
L2 (in transverse direction)	5	m
Thickness of slab	0.2	m
d, effective depth	0.175	m
Density of concrete	25	KN/sqm
Dead Load of slab	5	KN/sqm
Superimposed Dead Load	1.25	KN/sqm
Live load for residential floors	2	KN/sqm
total weight W	8.25	KN/sqm
Wul (total design load)	$W*L_2*L_1$ 206.25	KN
M_{u0l}	193.3594	KNm
For interior Panel		
Total negative moment	$(-)0.65*M_{u0l}$	
Moment	-125.68	KNm
Total positive moment	$0.35*M_{u0l}$	
Moment	67.68	KNm
For Exterior Panel		

Total storey height	4	m
Thickness of slab	0.2	m
Free column height	3.8	m
Effective column height	3.04	m
Stiffness of column Kc	I/L	
I	$BD^3/12$	
B	0.65	m
D	0.65	m
I/L	0.0030	m^3
Stiffness of slab, Ks	I/L	
I	$BD^3/12$	
B	5	m
D	0.2	m
	0.0007	
α_c	$\sum K_c/K_s$	
α_c	8.93	
L_2/L_1	1	
Live load	2	KN/sqm
Dead Load	6.25	KN/sqm
Live Load/Dead load	0.32	

Value of α_c >minimum permissible as per table-17

$1+(1/\alpha_c)$	1.11	
(-)ve Bending Moment at exterior support	$(-)0.65*M_0/(1+(1/\alpha_c))$	

-
113.0207 KNm

(+)ve span Bending Moment	$(0.63-0.28/(1+(1/\alpha_c)))*M_0$	
	73.1306	KNm
(-)ve Bending Moment at interior support	$(-)(0.75-0.1/(1+(1/\alpha_c)))*M_0$	
	-127.63	KNm

Since -127.63<-125.68, Hence moment at interior support = -127.63 KNm

.

Unbalanced columns Moments

M	$(0.08*(w_d+0.5w_l)l_2l_n^2-w_d'l_2'(l_n')^2)/(1+(1/\alpha_c))$		
w_d(dead load)	6.25	KN/sqm	
w_d'	6.25	KN/sqm	
l_2	5	m	
l_2'	5	m	
l_n	5	m	
l_n'	5	m	
w_l	2		
M	(0.08*(0.5wl*l2*(ln)^2)/(1+(1/αc))		As both spans equal

For corner column 650x650

α_c	29.36	
M	9.67	KNm

Thus, the column will be designed for additional moment = 9.67/2=4.835 KNm

For 400x900 column

b	0.4	m	d	0.9	m	I/L	0.00486
αc					47.96		
M					9.80	KNm	

Thus, the column will be designed for additional moment =9.80/2=4.9KNm

For 300x5000 wall

b	0.3	m	d	5	m	I/L	0.625
αc					3083.88		
M					10.00	KNm	

Thus, the column will be designed for additional moment =10.00/2=5.00KNm

For 600x900 wall

b	0.6	m	d	0.9	m	I/L	0.008168
αc					12.25		
M					9.25	KNm	

Thus, the column will be designed for additional moment =9.25/2=4.95KNm

check for Shear

for interior column 600x900

c_1	0.6m	c_2	0.9m
α		$1/(1+2/3*(a_1/a_2)^{0.5}$	
a1		c_1+d	
a_2		c_2+d	
a_1		0.775 m	

a_2	1.075	m
α	0.64	
τ_{ab}	$V/A+((1-\alpha)M(0.5*a_1))/J$	
A	$2(a_1+a_2)d$	
	0.6475	sqm
V	$W(l_1*l_2-a_1*a_2)$	
	199.3767	KN
J	$J_{ab}+J_{cd}+J_{bc}+J_{ad}$	
$J_{bc}=J_{ad}$	$I_{yy}+I_{zz}$ $(C_1+d)*d^3/12+(C_2+d)*d^3/12$	
$J_{ab}=J_{cd}$	$(C_2+d)*d*((C_1+d)/2)^2$	
$J_{bc}=J_{ad}$	0.0008	m^4
$J_{ab}=J_{cd}$	0.0282	m^4
J	0.0581	m^4
τ_{ab}	0.4619	N/sqmm
τ'_c	$Ks*\tau_c$	
K_s	$0.5+\beta_c$	
β_c	a_1/a_2 0.7209	
K_s	1.2209	
τ'_c	1.25	

Since $\tau_{ab}< \tau'_c$, Hence ok

For edge column 400x900

c_1	0.9m	c_2	0.4m

α $1/(1+2/3*(a_1/a_2)^{0.5}$

a_1 c_1+d

a_2 c_2+d

a_1 1.075 m

a_2 0.575 m

α 0.52

τ_{ab} $V/A+((1-\alpha)(M-Vh)X_{ab})/J$

τ_{cd} $V/A+((1-\alpha)(M-Vh)X_{cd})/J$

A $(2c_1+c_2+2d)d$

0.4463 sqm

V $W(l_1*l_2-a_1*a_2)$

201.1505 KN

J $J_{ab}+J_{bc}+J_{ad}$

X_{ab} $(c_1+d/2)*d/A$
0.3873 m

X_{cd} $(c_1+d/2)-X_{ab}$
0.6002

h $0.5*(c_1+d)-X_{ab}$
0.1502 m

$J_{bc}=J_{ad}$ $I_{yy}+I_{zz}$
$(C_1+d/2)*d^3/12+(C_1+d/2)*d^3/12+(c_1+d/2)*d*((c_1+d/2)/2-X_{ab})^2$

J_{ab} $(C_2+d)*d*((C_1+d/2)*d/A)^2$

$J_{bc}=J_{ad}$	0.0920	m^4
$J_{ab}=J_{cd}$	0.0151	m^4
J	0.1992	m^4
τ_{ab}	0.6351	N/sqmm
τ_{cd}	0.4507	
τ'_c	$K_s*\tau_c$	
K_s	$0.5+\beta_c$	
β_c	a_1/a_2 1.8696	
K_s	2.3696	
τ'_c	1.25	

Since $\tau_{ab}< \tau'_c$, Hence ok

for corner column 650x650

c_1	0.65	c_2	0.65

α	$1/(1+2/3*(a_1/a_2)^{0.5}$	
a_1	c_1+d	
a_2	c_2+d	
a_1	0.825	m
a_2	0.825	m
α	0.60	
τ_a	$V/A+((1-\alpha_x)((M_x-V_{xx}')X_{ab}))/Jx-((1-\alpha_y)((M_y-V_{uu}')y_{ad}))/J_y$	
τ_b	$V/A+((1-\alpha_x)((M_x-V_{xx}')X_{ab}))/J_x+((1-\alpha_y)((M_y-V_{uu}')y_{bc}))/J_y$	

τ_c	$V/A-((1-\alpha_x)((M_x-V_{xx'})X_{cd}))/J_x+((1-\alpha_y)((M_y-V_{uu'})y_{bc}))/J_y$	
A	$(c_1+c_2+2d)d$	
	0.2888	sqm
V	$W(l_1*l_2-a_1*a_2)$	
	200.6348	KN
J	$J_{ab}+J_{bc}+J_{ad}$	
X_{ab}	$(c_1+d/2)^2*d/2A$	
	0.1648	m
X_{cd}	$(c_1+d/2)-X_{ab}$	
	0.5727	
y_{bc}	$(c_2+d/2)^2*d/2A$	
	0.1648	
xx'	$(c_1+d)/2-c_1/2$	
	0.0875	m
uu'	$(c_2+d)/2-c_2/2$	
	0.0875	
J_x	$(C_1+d/2)*d^3/12+(C_1+d/2)*d^3/12+(c_1+d/2)*d*((c_1+d/2)/2-X_{ab})^2+(c_2+d/2)d(x_{ab})^2$	
J_y	$(C_2+d/2)*d^3/12+(C_2+d/2)*d^3/12+(c_2+d/2)*d*((c_2+d/2)/2-y_{bc})^2+(c_1+d/2)d(y_{bc})^2$	
J_x	0.0179	m^4
J_y	0.0179	m^4
J	0.0358	m^4
τ_a	1.0509	N/sqmm
τ_b	0.9818	

τ_c	1.0978
τ'_c	$K_s*\tau_c$
K_s	0.5+βc
β_c	a1/a2
	1.0000
K_s	1.5000
τ'_c	1.25

Since τab< τ'c, Hence ok

6.3 CHECK FOR UNBALANCED MOMENT TRANSFER

For edge column 400x900

Moment from STAAD		79.351	KNm
c_1	0.9 m	c2	0.4 m
α	$1/(1+2/3*(a_1/a_2)$^0.5		
a_1	c_1+d		
a_2	c_2+d		
a_1	1.075 m		
a_2	0.575 m		
α	0.52		
τ_{ab}	$V/A+((1-\alpha)(M-Vh)X_{ab})/J$		
τ_{cd}	$V/A+((1-\alpha)(M-Vh)X_{cd})/J$		
A	$(2c_1+c_2+2d)d$		
	0.4463 sqm		
V	$W(l_1*l_2-a_1*a_2)$		
	201.1505 KN		
J	$J_{ab}+J_{bc}+J_{ad}$		
X_{ab}	$(c_1+d/2)*d/A$ 0.3873 m		
X_{cd}	$(c_1+d/2)-X_{ab}$ 0.6002 m		

h	$0.5*(c_1+d)-X_{ab}$	
	0.1502	m
$J_{bc}=J_{ad}$	$I_{yy}+I_{zz}$	
	$(C_1+d/2)*d^3/12+(C_1+d/2)*d^3/12+(c_1+d/2)*d*((c_1+d/2)/2-X_{ab})^2$	
J_{ab}	$(C_2+d)*d*((C_1+d/2)*d/A)^2$	
$J_{bc}=J_{ad}$	0.0920	m^4
$J_{ab}=J_{cd}$	0.0151	m^4
J	0.1992	m^4
τ_{ab}	0.7445	N/sqmm
τ_{cd}	0.4508	N/sqmm
τ'_c	$K_s*\tau_c$	
K_s	$0.5+\beta_c$	
β_c	a_1/a_2	
	1.8696	
K_s	2.3696	
τ'_c	1.25	

Since $\tau_{ab} < \tau'_c$, Hence ok

for corner column
650x650
Moment from STAAD 52.024 KNm

c_1	0.65	c_2	0.65

α $1/(1+2/3*(a_1/a_2)^{0.5}$

a_1 c_1+d

a_2 c_2+d

a_1 0.825 m

a_2 0.825 m

α 0.60

τ_a $V/A+((1-\alpha_x)((M_x-Vxx')X_{ab}))/J_x-((1-\alpha_y)((M_y-Vuu')y_{ad}))/J_y$

τ_b $V/A+((1-\alpha_x)((M_x-Vxx')X_{ab}))/J_x+((1-\alpha_y)((M_y-Vuu')y_{bc}))/J_y$

τ_c $V/A-((1-\alpha_x)((M_x-Vxx')X_{cd}))/J_x+((1-\alpha_y)((M_y-Vuu')y_{bc}))/J_y$

A $(c_1+c_2+2d)d$

0.2888 sqm

V $W(l_1*l_2-a_1*a_2)$

200.6348 KN

J $J_{ab}+J_{bc}+J_{ad}$

X_{ab} $(c_1+d/2)^2*d/2A$
0.1648 m

X_{cd} $(c_1+d/2)-X_{ab}$
0.5727 m

y_{bc} $(c_2+d/2)^2*d/2A$
0.1648 m

xx' $(c_1+d)/2-c_1/2$
0.0875 m

uu' $(c_2+d)/2-c_2/2$
0.0875 m

J_x $(C_1+d/2)*d^3/12+(C_1+d/2)*d^3/12+(c_1+d/2)*d*((c_1+d/2)/2-X_{ab})^2+(c_2+d/2)d(x_{ab})^2$

J_y $(C_2+d/2)*d^3/12+(C_2+d/2)*d^3/12+(c_2+d/2)*d*((c_2+d/2)/2-y_{bc})^2+(c_1+d/2)d(y_{bc})^2$

J_x 0.0179 m^4

J_y	0.0179	m^4
J	0.0358	m^4
τ_a	1.0105	N/sqmm
τ_b	1.2643	N/sqmm
τ_c	1.0889	N/sqmm
τ'_c	$K_s*\tau_c$	
K_s	$0.5+\beta_c$	
β_c	a_1/a_2 1.0000	
K_s	1.5000	
τ'_c	1.25	N/sqmm

Since $\tau_a< \tau'_c$, Hence ok
Since $\tau b< \tau' c$, Hence ok
Since $\tau c< \tau' c$, Hence ok

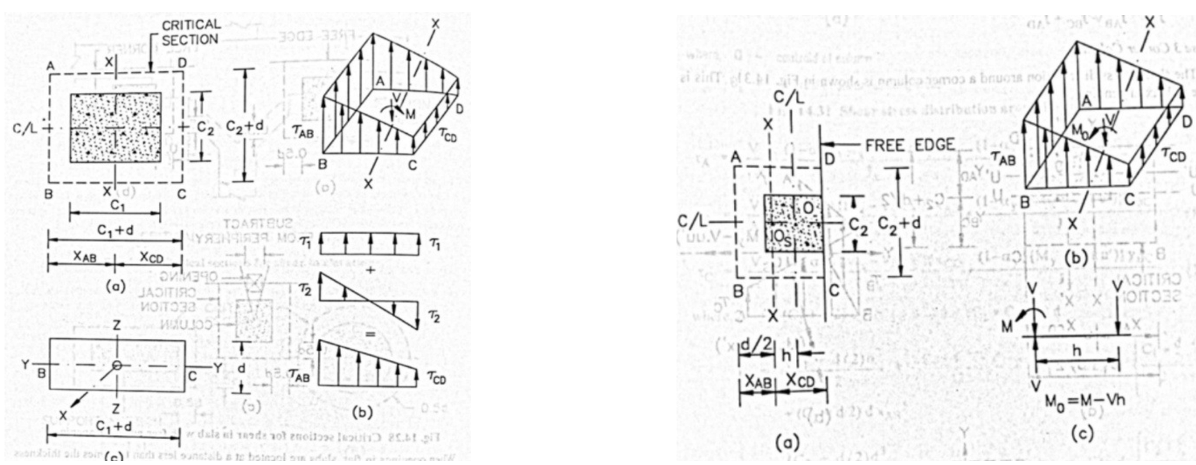


Figure 23: Shear stress distribution in an interior and Edge column

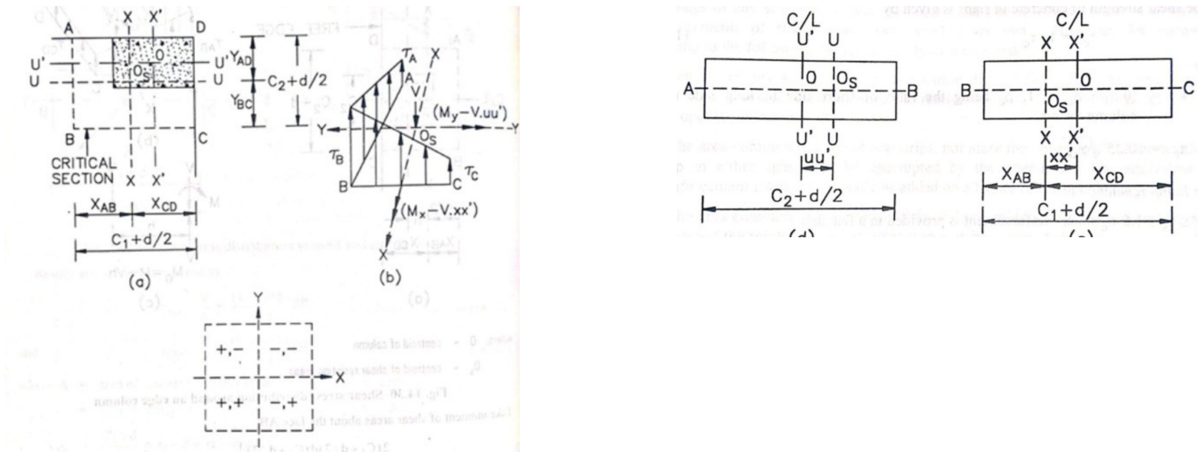


Figure 24: Shear stress distribution in a corner column

6.4 Flat Slab Design using STAAD-Pro

Moment Mx contouring

Under Gravity loading

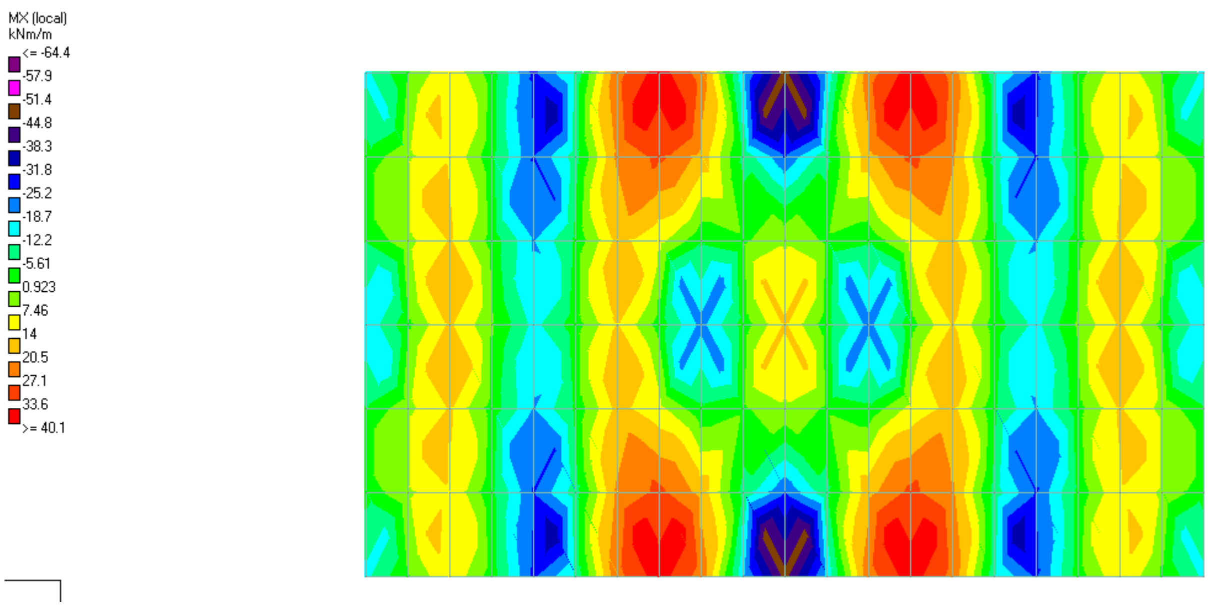


Figure 25: Diagram showing Mx contouring under Gravity Loading in STAAD

Moment My contouring

Under Gravity Loading

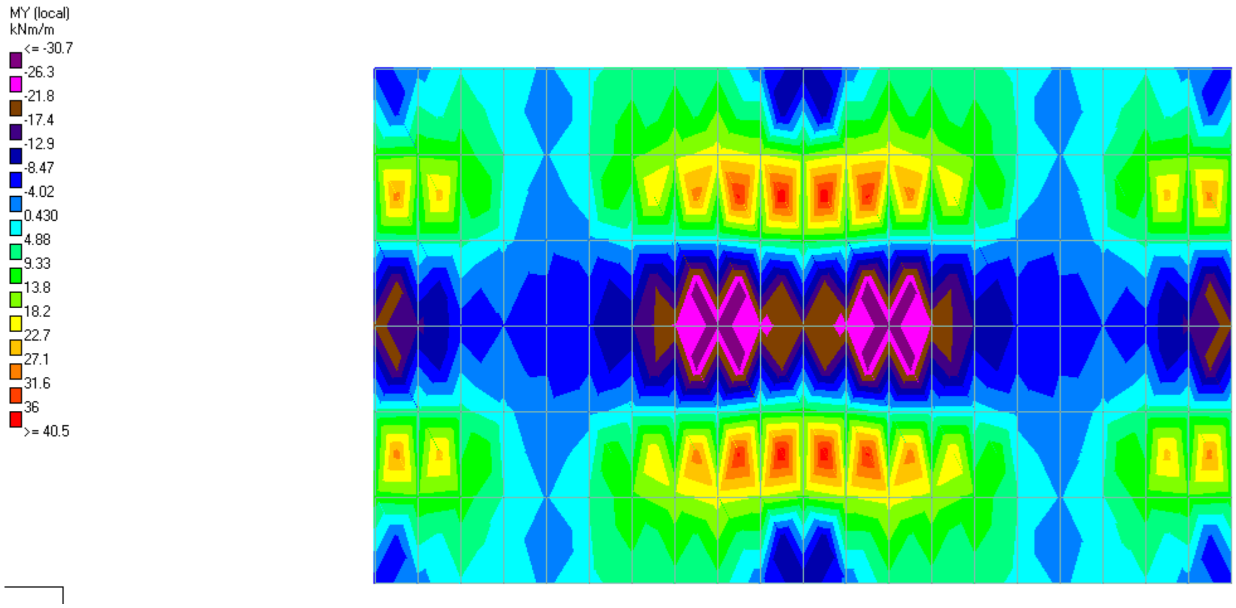


Figure 26: Diagram showing Mx contouring under Gravity Loading in STAAD

6.4 Flat Slab Design using Safe

Moment Mx diagram

Under Gravity loading

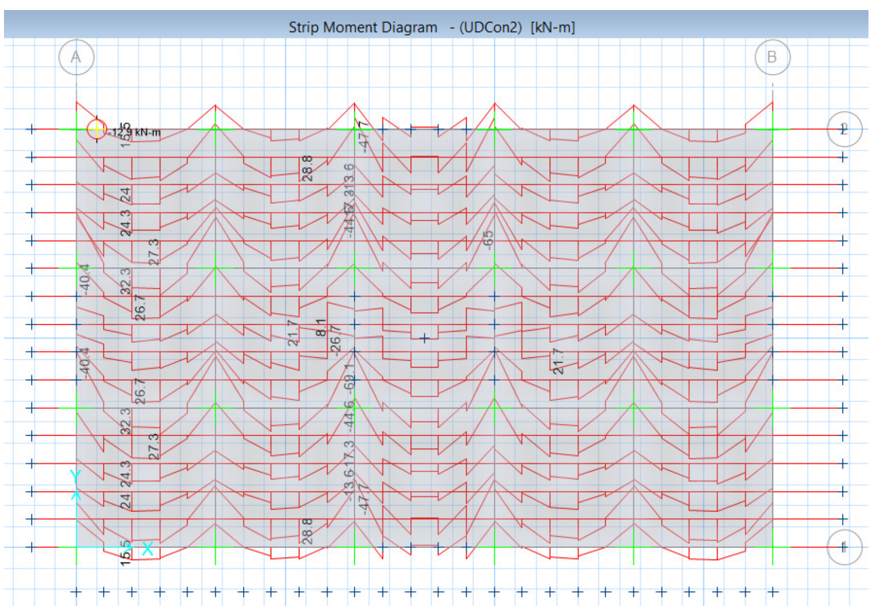


Figure 27: Diagram showing Mx contouring under Gravity Loading in Safe

Moment My diagram

Under Gravity Loading

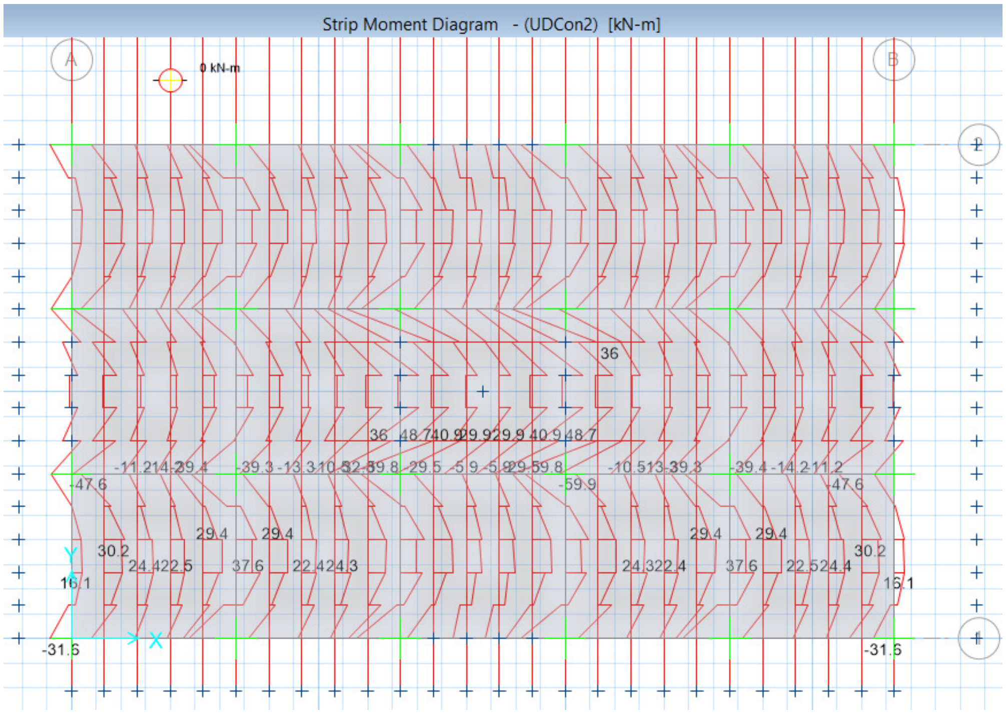


Figure 28: Diagram showing My contouring under Gravity Loading in Safe

CHAPTER-7

OBSERVATION AND CONCLUSION

Following Figures have been plotted under seismic loading in STAAD

Displacement in z (mm)

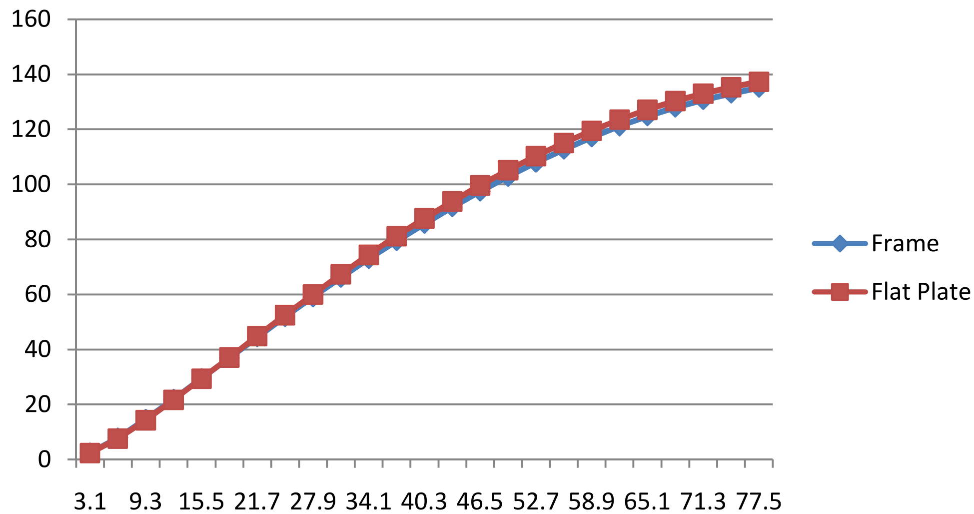


Figure 29: showing storey drift for RC frame and Flat plate with columns only in z direction

Displacement in x (mm)

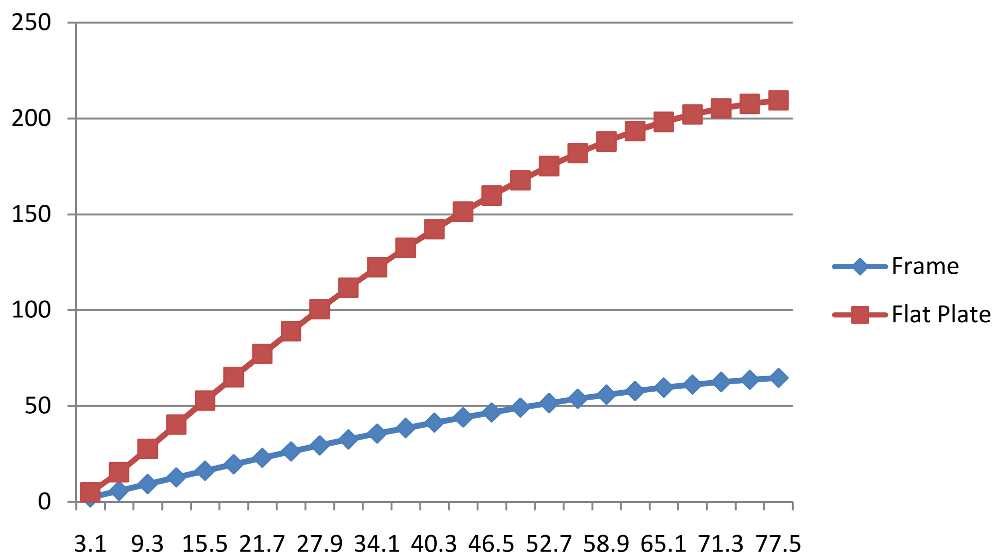


Figure 30: showing storey drift for RC frame and Flat plate with columns only in x direction

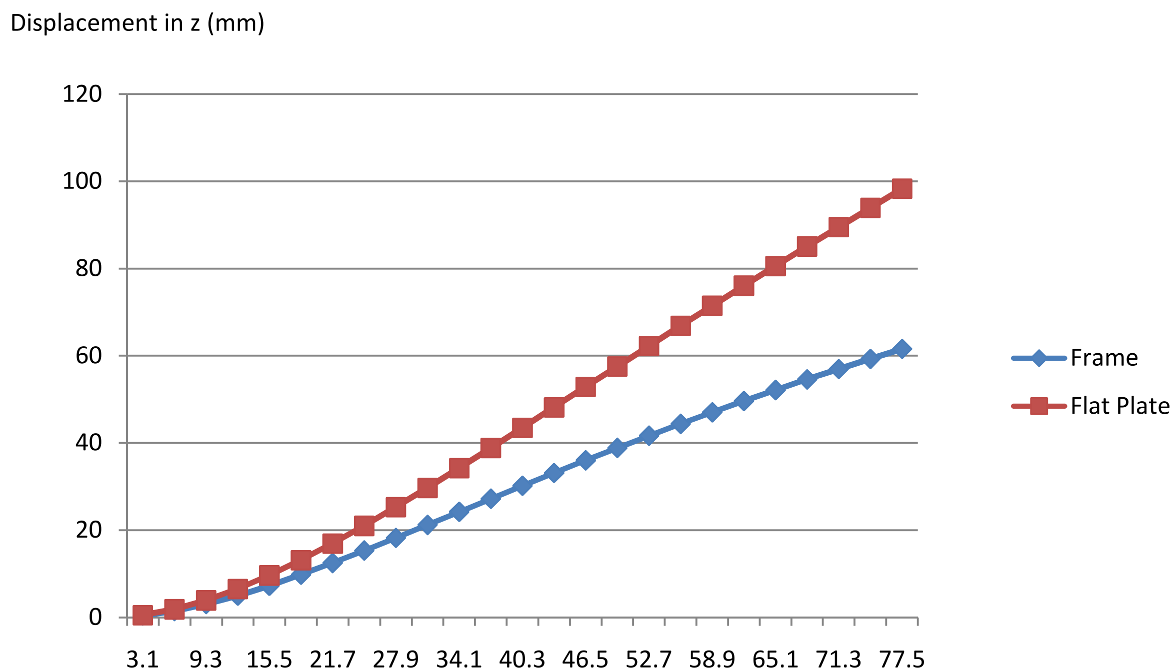


Figure 31: showing storey drift for RC frame and Flat plate with columns and shear wall in z direction

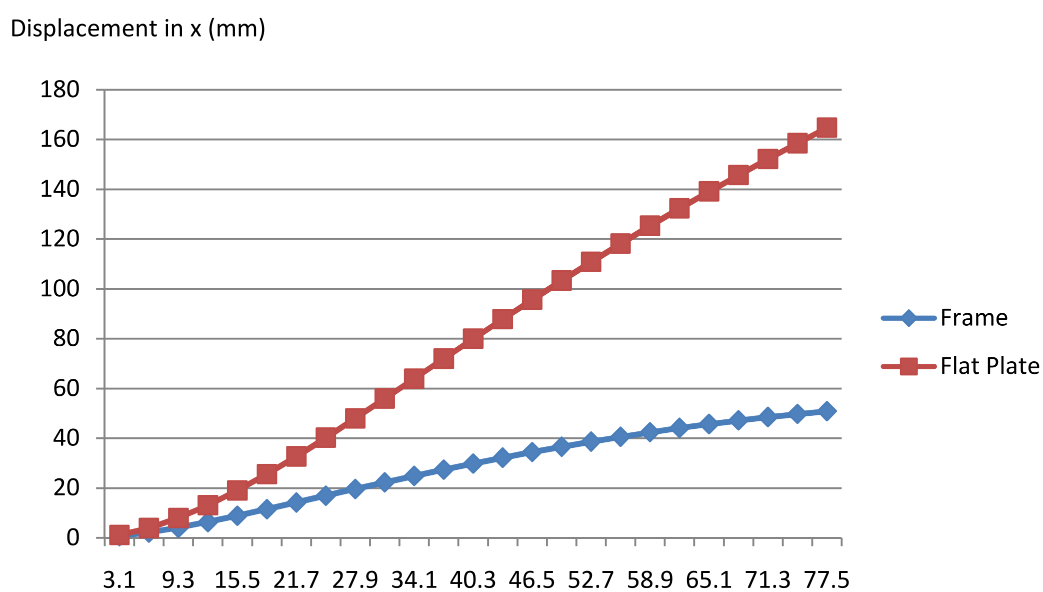


Figure 32: showing storey drift for RC frame and Flat plate with columns and shear wall in x direction

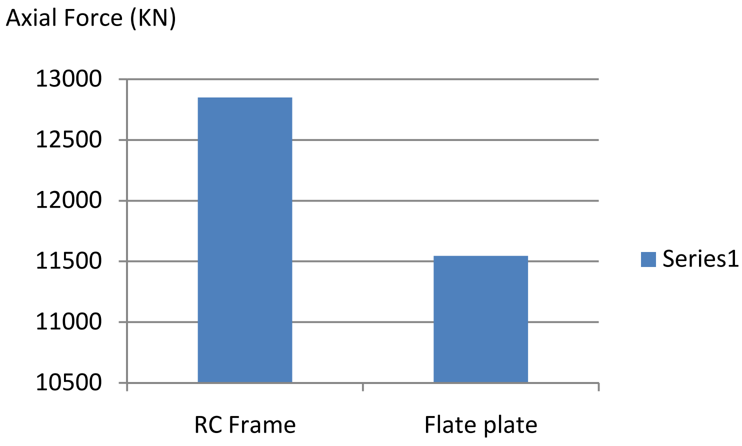


Figure 33: showing axial force in column with column configuration only

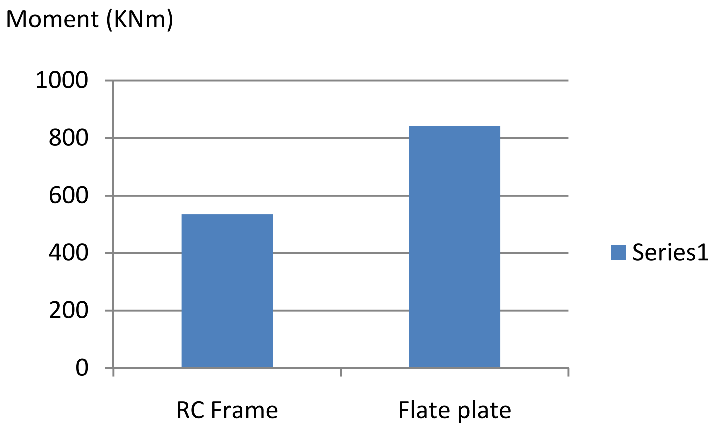


Figure 34: showing moment at base of column with column configuration only

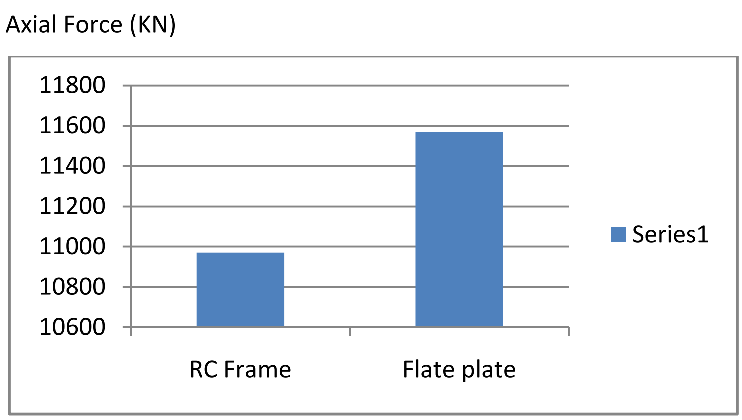


Figure 35: showing axial force in column with shear wall configuration

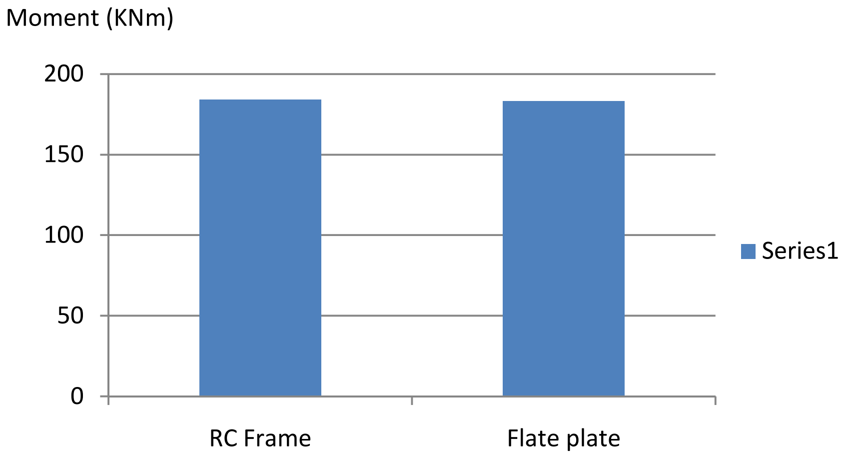


Figure 36: showing moment at base of column in column with shear wall configuration

Following Figures have been plotted under Wind load in STAAD

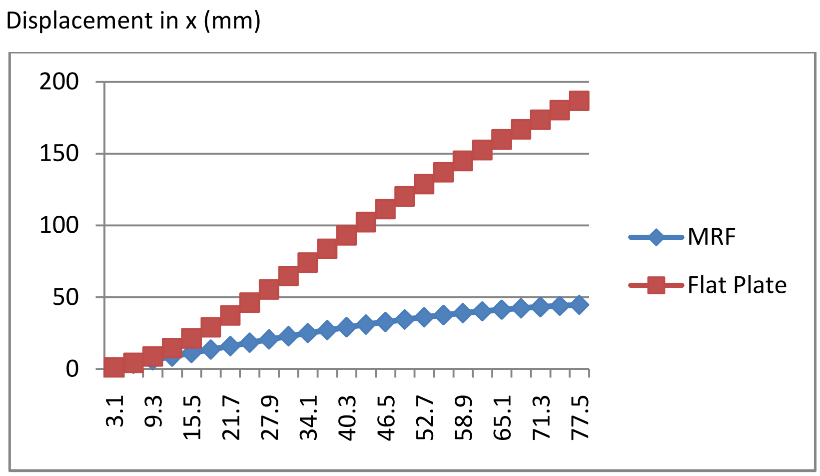


Figure 37: Graph showing deflection under wind load in Moment resisting frame and flat plate in x direction in column only configuration

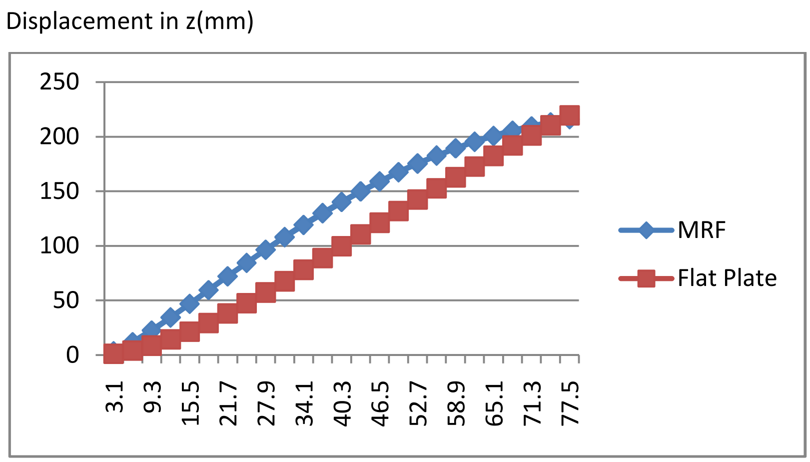


Figure 38: Graph showing deflection under wind load in Moment resisting frame and flat plate in z direction in column only configuration

Displacement in x (mm)

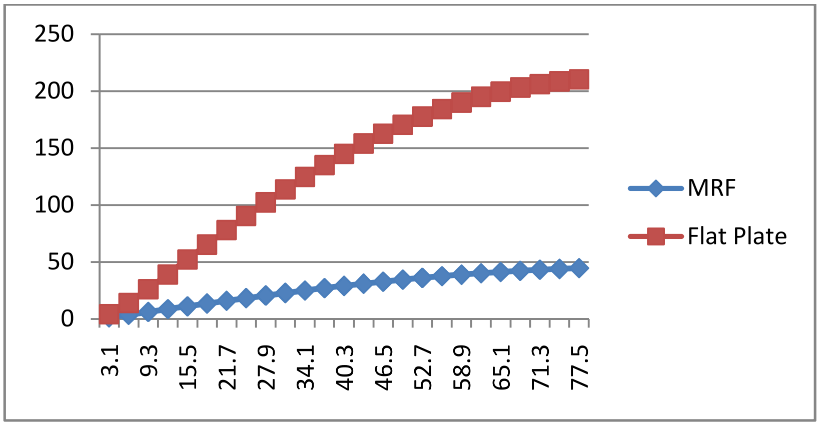


Figure 39: Graph showing deflection under wind load in Moment resisting frame and flat plate in x direction with shear wall

Displacement in z (mm)

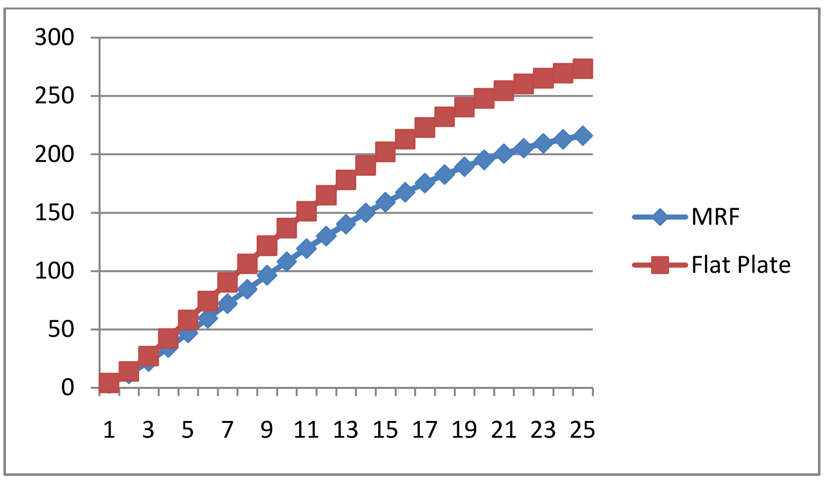


Figure 40: Graph showing deflection under wind load in Moment resisting frame and flat plate in z direction with shear wall

Displacement chart under Seismic Loading in ETABS:

Displacement in x (mm)

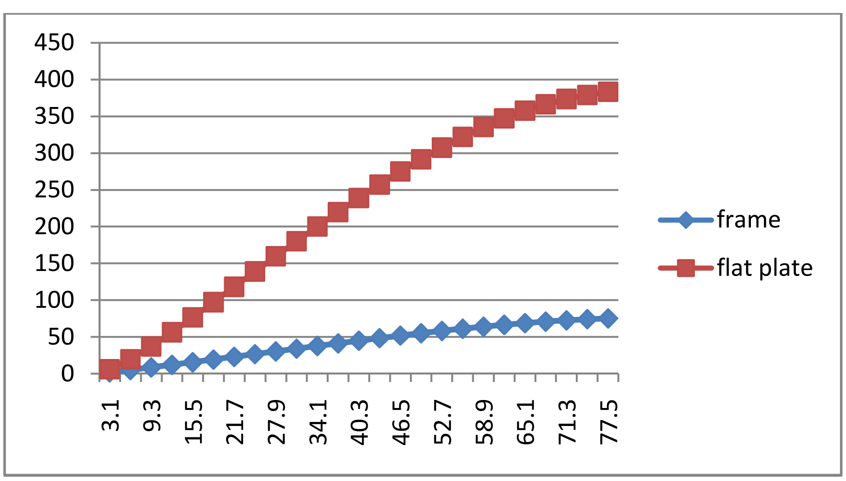


Figure 41: showing storey drift for RC frame and Flat plate with columns only in x direction

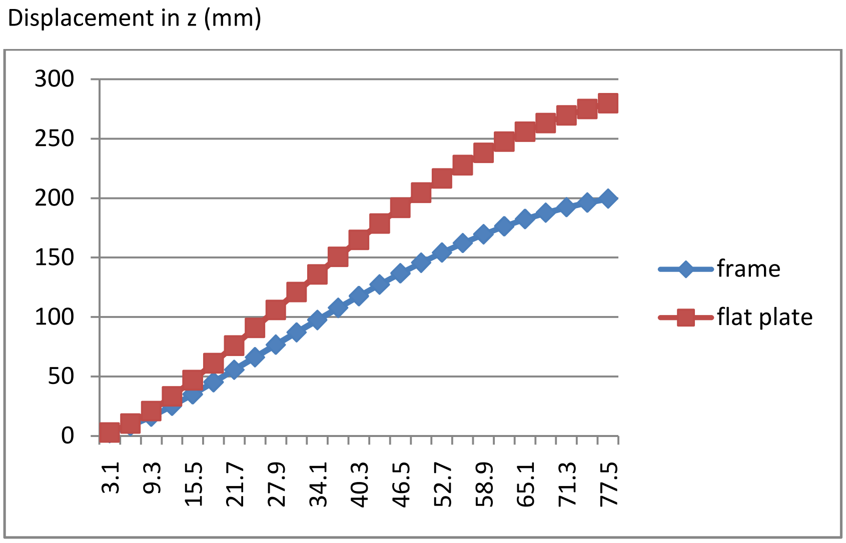


Figure 42: showing storey drift for RC frame and Flat plate with columns only in z direction

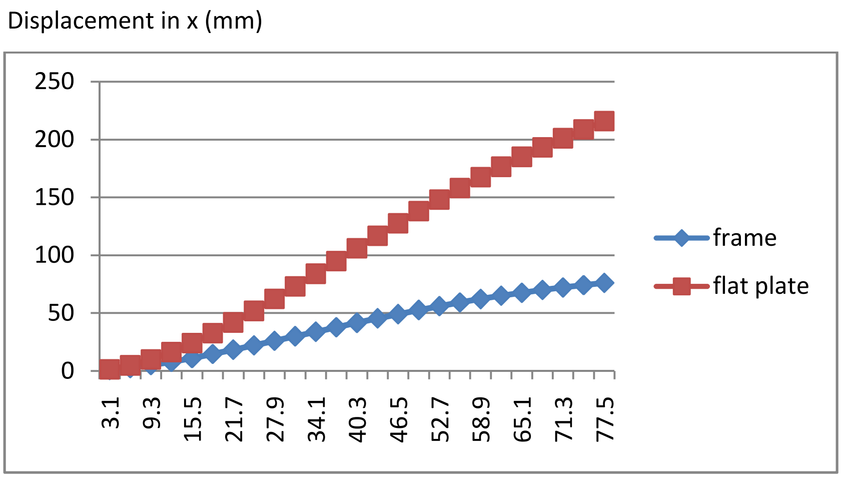


Figure 43: showing storey drift for RC frame and Flat plate with shear walls in x direction

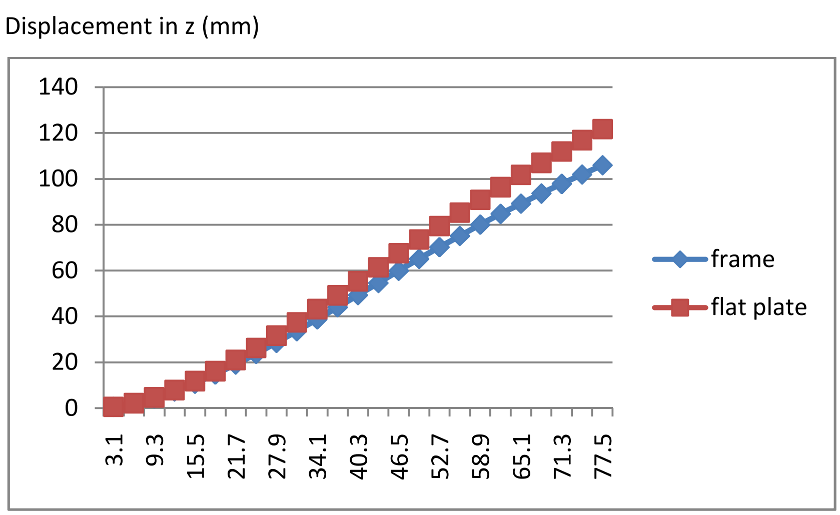


Figure 44: showing storey drift for RC frame and Flat plate with shear walls in z direction

Displacement chart under wind Load in ETABS:

Displacement in x (mm)

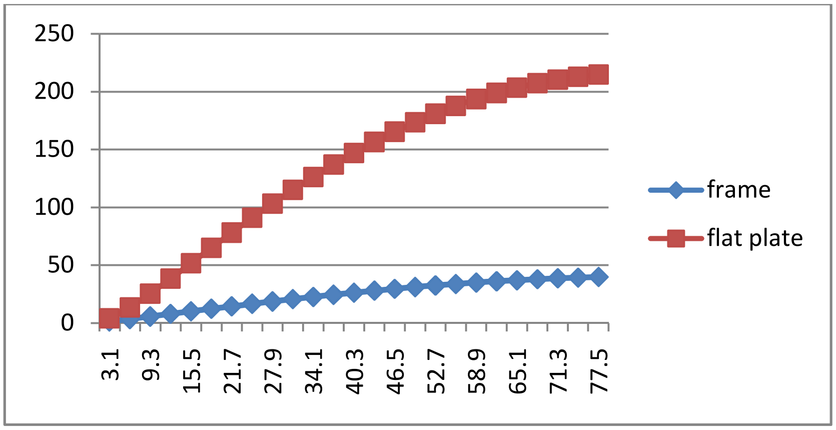


Figure 45: showing storey drift for RC frame and Flat plate with columns only in x direction

Displacement in z (mm)

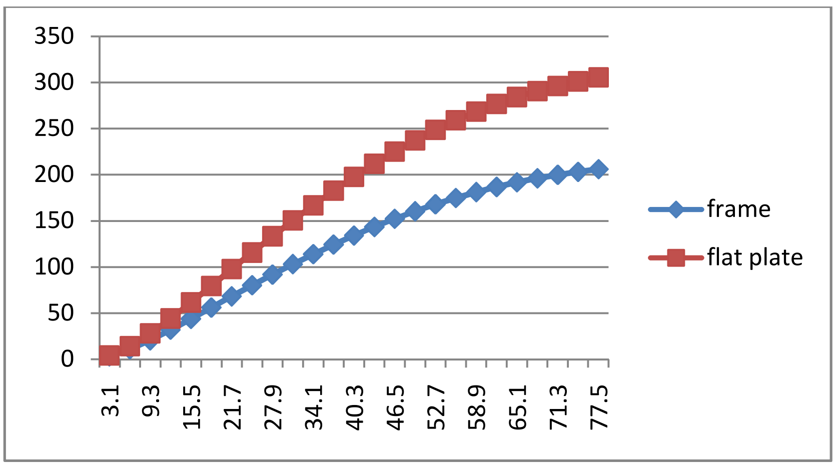


Figure 46: showing storey drift for RC frame and Flat plate with columns only in z direction

Displacement in x (mm)

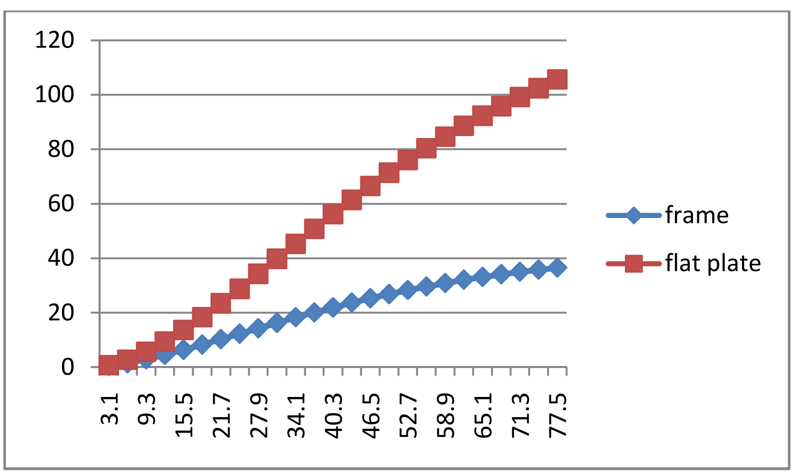


Figure 47: showing storey drift for RC frame and Flat plate with shear walls in x direction

Displacement in z (mm)

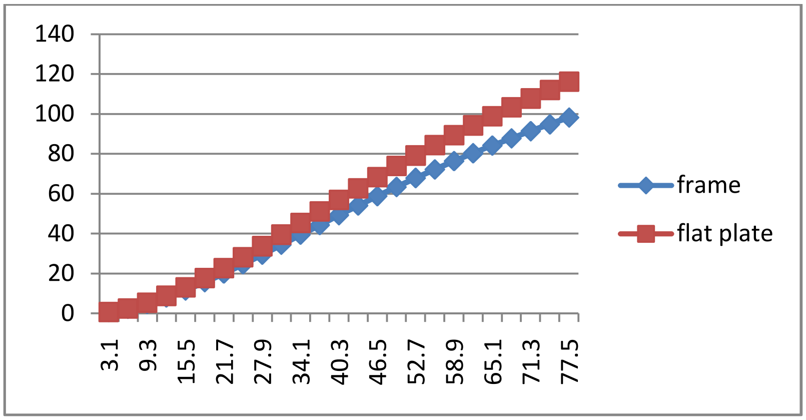


Figure 48: showing storey drift for RC frame and Flat plate with shear walls in z direction

Conclusions:

1. As it can be clearly seen from figure 15, Earthquake forces are not predominant in the shorter direction because of lower mass in that direction. Hence we find comparable deflection in case of Moment resisting frame and Flat plate.
2. But in case of x direction as shown in figure-16, where the length is greater, deflections are clearly greater in case of flat plate, which show lack of lateral load resisting elements. While Moment resisting frame show much lesser deflection.
3. With Shear walls, both the buildings i.e. RC moment resisting frame building and Flat plate structure have shown lesser deflection in shorter direction. Effect of shear wall is clearly visible as the deflections have reduced drastically. However, Flat plate model still shows larger deflection which points out lack of lateral stiffness.
4. Similarly, In longer direction also, deflections have reduced with larger deflections coming in flat plate structure only.
5. In case of moment resisting frame structure, Axial forces are higher than the flat plate structure as shown in figure 19, but the moment at base is higher (figure-20) in case of flat plate which is due to lack of stiff member at upper levels which can reduce moment induced at base.
6. But with the use of Shear wall, the problem of higher moment at base has been solved as it is clearly visible, that the moments at base are comparable in Moment Resisting frame and Flat plate structure.
7. Under wind loading, Deflections in x direction (larger direction) are very large as seen in figure22, which clearly suggests lack of lateral load resisting members in a flat plate structure.

8. In z direction, the slenderness of building plays its role and the deflections in both the case are similar because of absence of any lateral load resisting member, which is shear walls in this case.
9. The figures plotted in ETABS also depict the same trends as discussed in the above points. The only difference is that the values of displacement in case of flat plate in ETABS are higher than that of STAAD.
10. Thus the above modeling in STAAD and ETABS clearly show that there is difference in the stiffness and deflection analysis of the two softwares.
11. Moments calculated under gravity loading as per Direct Design Method of IS 456 are higher than that calculated from STAAD-Pro as evident from the calculations shown and moment diagrams of STAAD software.
12. But the moments in columns under gravity loading are lesser according to Direct Design Method as compared to values from STAAD output.
13. Moments calculated under gravity loading as per Direct Design Method of IS 456 are higher than that calculated from Safe as evident from the calculations shown and moment diagrams of Safe software.
14. But the moments in columns under gravity loading are lesser according to Direct Design Method as compared to values from ETABS output.
15. Moments induced due to lateral loads cannot be calculated from Direct Design method and hence Softwares like ETABS and STAAD are needed for analysis of such high rise buildings.

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