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Study of RC Frame Structures with Plan Irregularities using Response Spectrum Method

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Abstract: One of the main causes of seismic damage amplification is structural irregularities. Indeed, past earthquakes have shown that structures with irregular configuration or asymmetric distribution of structural characteristics are subject to increased demand for seismic damage. The sources of irregularity in a construction setup can be multiple and of distinct types and are generally categorized into two main classifications: plan and elevation irregularities. Both these types of irregularities often involve the development of fragile collapse mechanisms owing to a local rise in seismic demand in particular components that are not always sufficiently supplied with structure. Among the two kinds of structural irregularities mentioned above, in-plan irregularities appear to have the most adverse effects on the applicability of classical nonlinear static processes (NSPs), exactly because such techniques have been created for the seismic evaluation of buildings whose behavior is mainly translational. This is why experts in this sector have extensively researched the expansion of NSPs to plan uneven construction structures in latest years. This dissertation is therefore aimed at studying and understanding the critical behavior of irregularity in plans constructions subject to seismic excitement. The main parameters for determining the performance point of all 10 designs, modelled in Etab 9.6.2, were lateral displacement, storey drift, base shear, storey displacement. The findings of all 10 models are provided from software consisting of Response Spectrum curve and hinge formation, which generates consciousness of planning easy scheduled constructions to minimize the impact of the earthquake.

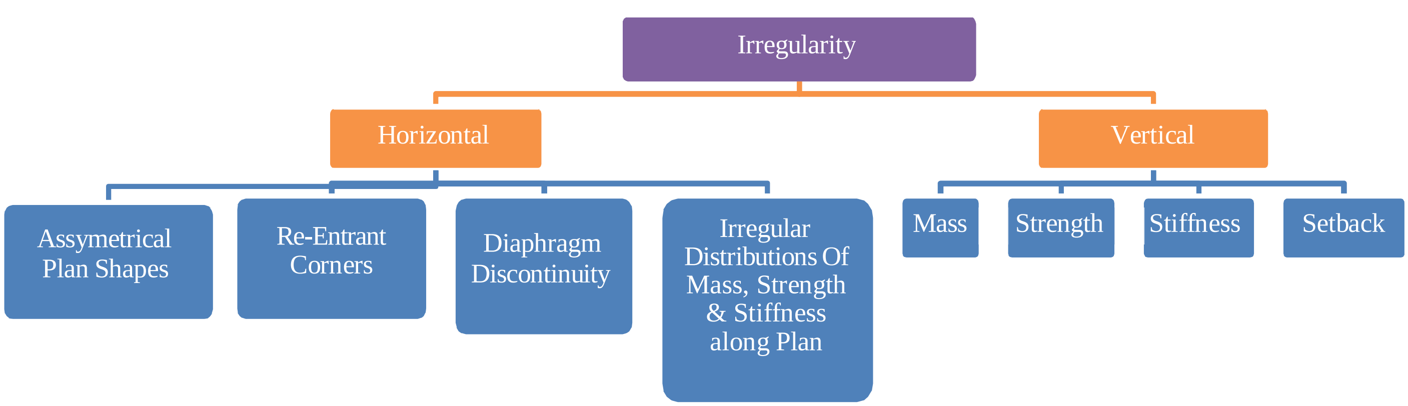
Key words: analysis of seismic excitation, irregularity of plan, curve of response spectrum, point of success.

I. INTRODUCTION

The building component resisting seismic forces is known as the lateral force resisting system (L.F.R.S). The building's L.F.R.S may be of different types. The most common forms of these systems in a structure are special moment-resistant frames, shear walls and dual frame-shear wall systems. The damage in a structure generally starts at the location of the weak structural planes present in the building systems. These weaknesses trigger further structural deterioration leading to structural collapse. These weaknesses often occur due to the presence of stiffness, strength and mass structural irregularities in a building system. Structural irregularities can be broadly classified as plan and vertical irregularities.

In this present work two types of structures considered are reinforced concrete regular and irregular multistory buildings. Here 15 storey buildings are analyzed by above methods by using IS 1893-2002 (part1).

Fig -1: Classification of various types of structural irregularities



A. Organization of the Dissertation work

The dissertation work presented here has been divided into six chapters

Chapter 1 The first chapter provides a fundamental introduction to the various kinds of plan and vertical irregularities in building systems.

Chapter 2 The second chapter deals with reviews of the literature available on previous research conducted by multiple other writers.

Chapter 3 The third chapter deals with the structural modelling of the 10 models used in the dissertation work with different irregularity in plan.

Chapter 4 The fourth chapter deals with the methodology used for the analysis using ETABs software.

Chapter 5 The fifth chapter deals with the results and discussions of the present study.

Chapter 6 The sixth chapter deals with the conclusions drawn from the study. The scope of work for further study has also been identified in this chapter.

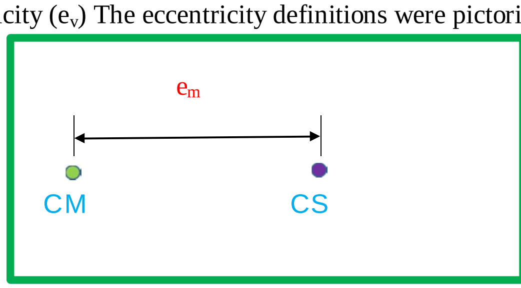
II. LITERATURE REVIEW

A. Review Of Research Work On Plan Irregularities

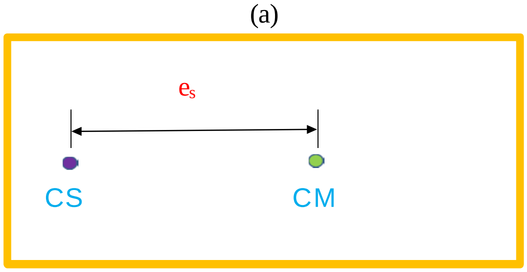
Building structure performance assessment during past earthquakes suggests that plan irregularities are one of the major causes of earthquake damage. Due to uneven mass distribution, stiffness, and strength along the plan, irregularities may occur. There has been extensive research effort in recent years to examine the behaviour, during seismic excitement, of the plan of asymmetric buildings (*Tso and Myslimaj 2003; Tso and Bozorgnia 1986; Tso & Sadek 1985 and Tso & Sadek 1989*).

B. Single-story construction models

Earlier studies investigated the torsional effects of single-story construction models on irregular construction systems. Their simplicity was one of the main reasons for adopting single-storey models. These models have been used to determine the influence of torsion on parameters of seismic response and these results have also been used to formulate design methodologies for irregular construction systems. Multistory building models, however, have been used in recent years to determine the realistic inelastic torsional response of irregular building systems in the plan. But the use of multi- story building models is limited due to complexities, and it is one of the main reasons why many researchers still prefer single-story building models *(Ladinovic 2008; Lignos and Gantes 2005; Luchinni et al. 2011)*. Previous researchers on plan irregularities using single-story models focused mainly on variation of CM (Mass Center) or CS (Stiffness Center) positions in relation to each other to create eccentricity. Due to eccentricity, the main objective was to determine the torsional response of building systems. The eccentricity generated in this case was called as stiffness eccentricity (e_s) to create eccentricity some researchers varied position of CS or CR keeping position of CM constant. Some researchers varied the position of CM holding CS as constant, and the eccentricity generated in this case was referred to as mass eccentricity (e_m) [*Tso and Myslimaj 2003*]. In contrast to earlier approaches, some researchers have created differences in strengths of resistant elements to vary the center of strength (CV) position with respect to CM, and the eccentricity generated was known as strength eccentricity (e_v) The eccentricity definitions were pictorially described in Figure 2.1.



(a)



(b)

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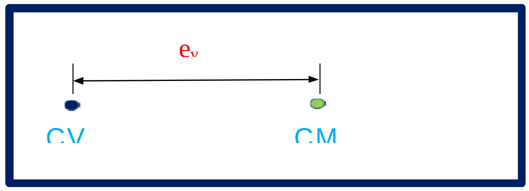

(c)

Figure 2.1 Types of eccentricity:
(a) Mass eccentricity,
(b) Stiffness eccentricity,
(c) Strength eccentricity

Research work on irregular construction systems began in the early 1980s with *Tso and Sadek (1985)*, who determined the variation in ductility demand by performing inelastic seismic response of a simple one-story mass eccentric model with degradation of stiffness using Clough's stiffness degradation model and bi-linear hysteric model. Analytical study results showed that after the elastic range the time period had a predominant effect on the ductility demand. The results comparison showed a 20 percent difference between Clough's and the bilinear model in the results obtained. Irregular strength and stiffness distributions are one of the major causes of earthquake failure. Both of these irregularities are interdependent and, in order to study the effect of these irregularities on seismic response, researchers such as *Tso and Bozorgnia (1986)* identified the inelastic seismic response of plan asymmetric building models (as described in Table 1.3) using curves proposed by *Tso and Dempsey (1990)* with strength and rigidity eccentricity. Analytical study results showed the effectiveness of the curves proposed by *Tso and Dempsey (1990)* with the exception of torsionally stiff, low yield strength structures.

*Sadek and Tso (1989) c*onducted inelastic analysis of mono-symmetric building systems with strength eccentricity. The strength center was defined in terms of the resistant elements yield strength. From analytical studies it was found that it was useful to predict the elastic seismic response that the code defined eccentricities based on rigidity criteria. However, eccentricity was found to be useful in determining seismic response in the inelastic range parameter of strength.

Pekau and Guimond (1990) checked the adequacy of accidental eccentricity to account for the torsion induced by the variation in strength and rigidity of the resistant elements achieved using the relationship between elasto-plastic force and deformation. Analytical study results showed that torsional amplification occurred due to variation in strength and stiffness. Lastly, it was found that the code prescribed 5 percent provision for accidental eccentricity was inadequate.

S. No	**Model Name**	**Description**
1	Me	Mass eccentric model with all three resistant elements having equal yield deformation
2	S_{e1}	Stiffness eccentric Model with identical yield strength.
3	S_{e2}	Stiffness eccentric Model with identical yield deformation.

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Table 2.1 Model descriptions adopted by *Tso and Sadek (1989)*

Based on their analytical studies of irregular plan buildings, *Duan and Chandler (1991)* recommended a change in design eccentricity in Mexico code 87 as $1.5e_S + b$ and $0.5e_S - 0.1b$ compared to the previous value of $e_S - 0.1b$ and $e_S - 0.05b$.
Chandler and Hutchinson (1992) identified the effects of torsional coupling on eccentric building systems with one story stiffness. A strong dependency of torsional coupling effects on the structure's natural time period was observed from the results of analytical studies. In addition, the effectiveness of torsional design provisions as prescribed by various codes of practice (ATC 3-06, NEHRP, NBCC 90, and EC8:1989) was determined by conducting elastic and inelastic analyzes of eccentric building systems with one story. Table 2.2 and Table 2.3 showed the results of the code evaluation obtained for asymmetric building system as per different codes. Analytical study results showed more flexible edge displacement compared to stiff edge.

S. No	**Code**	**Results**
1	NEHRP	Inadequate for building systems with small and moderate eccentricity. Satisfactory results for building systems with large eccentricity.
2	ATC	Same as NEHRP.
3	NBCC	Inadequate for buildings with low time periods ($T < 0.5S$) Over-conservative for higher time periods at all eccentricities.
4	EC8	Conservative for small eccentricity. Over conservative for medium to large eccentric buildings system with higher time periods.

Table 2.2 Code Results of *Chandler and Hutchinson's (1992)* evaluation

S. No	Code Name	Results
1	NZS	Conservative Estimate of displacement
2	UBC	Conservative Estimate of displacement for DAF / FRF =1
3	NBCC	Conservative Estimate of displacement for DAF / FRF =0.6 -1.0

Table 2.3 Results of *Chandler and Hutchinson (1992)*

Chandler et al. (1995) checked the torsional provisions prescribed by various practice codes. Two types of construction models were considered for analytical study, namely torsionally balanced (TB) and torsionally unbalanced (TU). The torsional imbalance in the building model was created by varying center position of stiffness inducing eccentricity of stiffness equal to 0.05*b*. The torsionally unbalanced construction models were further divided into two types with moderate and low torsional rigidity

C. Results of Chandler and Hutchinson (1992)

Tnamely A1 and A2. Results of analytical studies showed the variation in seismic response in A1 and A2 models with more deformation of the flexible edge compared to the rigid edge. The stiff edge of small-time (T < 1 Sec) building systems designed in accordance with NZS 4203 and EC8:1989 had the least additional demand for ductility. However, the additional ductility demand was found to be largest for building systems with T > 1 Sec. In case of TU systems designed according to EC 8 -1989 the ductility demand exceeded by 2.5 percent as compared to the TB system.

Ferhi and Truman (1996) determined seismic response of building systems with the presence of stiffness and strength eccentricity. Both elastic and inelastic seismic behavior were studied. It was observed from the analytical study of the building systems that the seismic response showed greater dependence in elastic range on stiffness eccentricity. However, it was observed that the effect of force eccentricity on seismic response was in the inelastic range.

Chandler and Duan (1997) developed an optimized process to determine torsionally balanced and unbalanced structure's seismic response. The proposed optimization procedure included parameters such as eccentricity (e), standardized gyration radius (Pk), force reduction factor (R) and uncoupled lateral period (Ty). The authors proposed expressions of eccentricity and strength factor design and compared them with defined expressions of code.

UBC 94, EC8-94 and NBCC-95 were the codes used in the study. In both torsionally balanced (TB) and torsionally unbalanced (TU) models, the analytical study was conducted. Analytical results showed that the over- resistance factor was found to be significantly lower compared to UBC-94 and NBCC- 95, but higher than EC8 for the entire PK range. However, the results of the proposed procedure are comparable with code-defined torsionally unbalanced structures (TU) procedures. In the design procedure, parameters e, pk, R, Ty were found to influence the seismic response. Finally, the procedure was found to apply to torsionally unbalanced single-storey and multi-storey structures.

In view of the earthquake components in two perpendicular directions, *De-La- Colina (1999)* studied the effects of torsion on simple torsionally unbalanced building systems. The effects of the following parameters have been studied: (a) factor for seismic force reduction, (b) design eccentricity, (c) natural time period. Figure 2.2 shows the structural model used for the analytical study. Based on the results of the analytical study, it was concluded that the ductility demand for flexible element decreased with an increase in the force reduction factor. With regard to the effect of the time period, it was found that the demand for ductility increased over time for torsionally unbalanced stiff elements and vice versa for flexible elements that were torsionally unbalanced. The increase in the value of eccentricity stiffness reduced the standardized demand for ductility. Based on these results, eccentricity of strength was concluded to have a greater effect on seismic response compared to eccentricity of stiffness

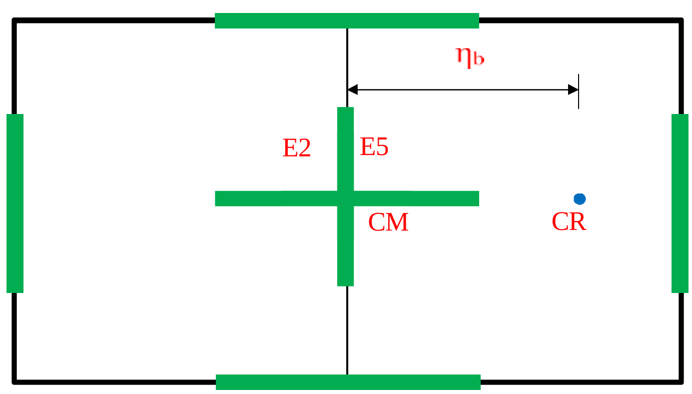

Figure 2.2 De-La Colina Structural Model 1999)

Ghersi and Rossi (2001) used elastic and inelastic analysis to determine the influence of bi-directional seismic excitation on the seismic response of eccentric stiffness of one-story building systems. The inelastic analysis seismic response was compared with the elastic analysis results. Analytical results showed that consideration of bi-directional seismic excitation effects resulted in minor seismic

De Stefano and Pintuchhi (2002) considered the phenomenon of inelastic interaction between axial forces and horizontal forces when modeling asymmetric building systems with irregular rigidity. Based on analytical study result. it was concluded that taking into account the interaction phenomenon between axial force and horizontal force resulted in a 20 percent reduction in floor rotation, *Dutta and Das (2002)* studied the seismic response of asymmetric structures under bi-directional seismic excitation in the single-story plan. The authors proposed two models of hysteresis as shown in Figure 2.3 (a, b) from analytical study. These models accounted for the deterioration of strength and stiffness of the RC structural elements under cyclic loading. From the analytical study results, it was found that the demands of local deformation at both stiff and flexible edges showed variation when considering the deterioration of strength. Considering unidirectional seismic excitation resulted in lower levels of demand for deformation at both flexible and rigid edge. Similar to *Tso and Myslimaj (2002)*, *Myslimaj (2002)*, these results were found.

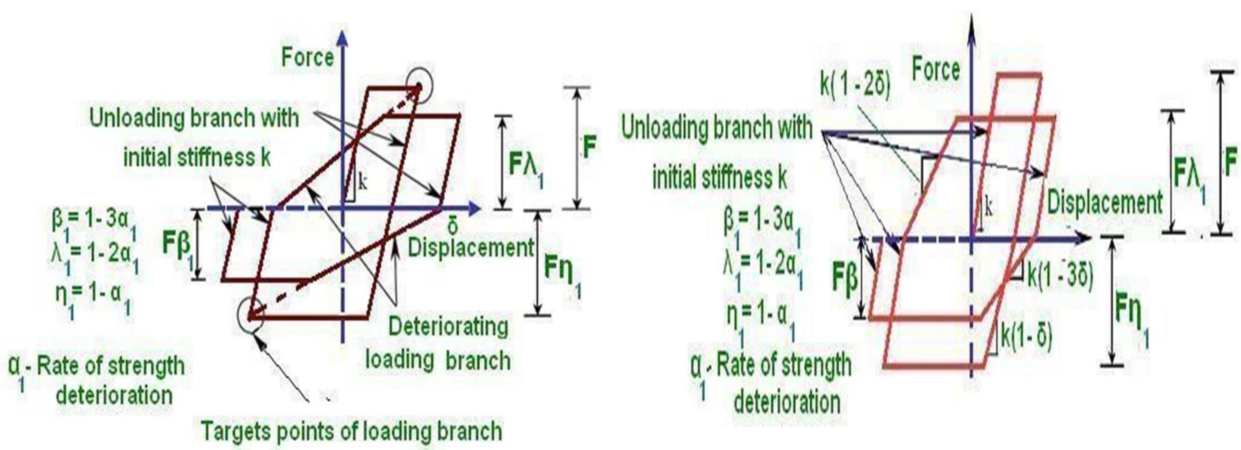

Figure 2.3 (a, b) Dutta and Das hysteresis model (2002)

Tso and Myslimaj (2003) proposed a new Approach For strength and stiffness distribution called a yield-based approach.
The authors modeled a single-story structure with a rigid rectangular deck supported by two resistant elements in X and five resistant elements in Y direction for analytical study The resistant elements were modeled for force-deformation relationship using elasto-plastic, the bilinear and Clough's hysteresis models. The authors proposed a design parameterr β depending on the location of themass center (CM), rigidity (CR), strength (CV) and displacement of yield (CV). Dynamic analysis of the models was performed to determine the balanced CV-CR location. It was found from the results of the analytical study that the structure satisfied a balanced CV- CR location and had low torsional response when the value of ηb is between zero and unity.
Fujii et al. (2004*)* suggested a simplified non-linear analysis procedure for asymmetric structures with eccentricity stiffness modeled as SDOF's and MDOF's. Analytical study results showed that, compared to torsionally flexible building systems, the torsionally rigid building systems experienced greater oscillations in first mode. Comparing the responses of MDOF and SDOF models for TS and TF building systems, it was found that only torsionally rigid building systems applied to SDOF models. Finally, it was found that the proposed analytical procedure is effective in determining the seismic response of TS building systems.
Moghadam and Aziminejad (2005) performed asymmetric structures PBD (Performance-based design). The researchers evaluated the seismic response of single- story structures (code designed) with irregular configuration to optimize configurations of mass, rigidity and strength centers corresponding to different plastic hinge formations.
The authors adopted the balanced CV— CR location concept proposed by *Tso and Myslimaj (2003)* to assess the structure's best performance level. Based on the analytical study, it was concluded that the best location of CV–CR (Stiffness Center and Stiffness Center) depended on the structure's required level of performance and also on the indices of damage as shown in Table 2.4.

F2	Building Model with torsional stiffness less than Model S and F1.

Table 2.4 Different positions of centers of mass, stiffness, strength and displacement for different values of

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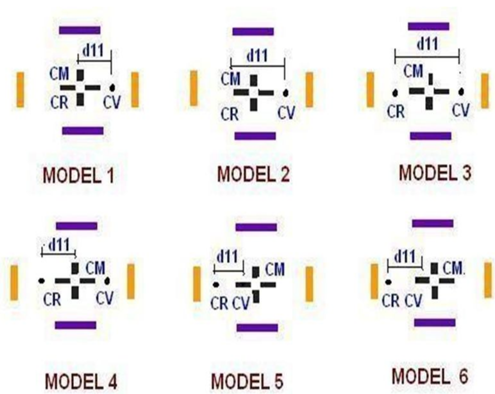
Shakib and Ghasemi (2007 have determine
The effect on seismic response of various typ
es of plan asymmetric structures with stiffn
es asymmetry of consideration of near-fault
and far-fault excitations. The authors suggest
ed a new approach to minimize rotational de
formation following Tso and Myslimaj (2003)
who suggested balanced CV- CR location to m
-inimize rotational deformation. In the propos
- ed approach whereby the pattern of strength
distribution is made equal to the distribution
of yield of displacement modified by a param
-eter β. From the results of the analytical study
it was found that the displacement demand on
stiff edges is higher compared to the flexible ed
-ges in the case of near-fault motions when $\beta > 0$
. In the case of motions with far-reaching faults wh
en $\beta < 0$, the demand for displacement on flexible
edges was higher than on rigid edges
Jarernprasert et al. (2008) determined the stiffness
-eccentricity inelastic torsional response of asymmetric single-story plan systems designed in accordance with IBC 2006 and Mexico City Building Code 2004. The method of modal analysis was adopted for the analysis of this building model. The effect of seismic excitation on the following parameters has been studied, (a) ratio of uncoupled torsional to transitional frequencies, (b) target ductility design, (c) natural elastic time period and normalized static eccentricity. New reduction and amplification factor for these parameters was also proposed by the researchers (a, b, c). From analytical study results, it was found that these parameters (a, b, c) had a significant influence on the building system's inelastic

De Stefano and Pintuchhi (2002*)* considered the phenomenon of inelastic interaction between axial forces and horizontal forces when modeling asymmetric building systems with irregular rigidity. Based on analytical study result. it was concluded that taking into account the interaction phenomenon between axial force and horizontal force resulted in a 20 percent reduction in floor rotation,

Aziminezad and Moghadam (2010) determined the effects of strength distribution and strength, rigidity and mass configuration on the seismic response of a one-story plan asymmetric construction system that was subjected to near-field and far-field ground movements. As shown in Figure.2.4, models with different yield displacement values, strength and stiffness eccentricity were considered. Dynamic nonlinear analysis analyzed the models and from the results of the analytical study it was found that for torsionally flexible building systems, the strength distribution had a minor effect for both near-field and far-field excitations. But torsionally rigid building systems ' seismic response was largely influenced by the distribution of strength. With regard to the modal periods, it was

Figure. 2.4 Aziminejad and Moghadam models (2010)



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found that the maximum value of modal periods along the X-axis was compared to the other two modal periods and that the ratio of lateral to torsional frequency was found to be higher in y direction. It was also concluded that the torsionally rigid building systems with balanced CV-CR location performed better than other building models in both near- field and far-field excitation. Luchinni et al. (2011) identified the nonlinear seismic response of single-story building models with eccentricity in both directions using the shear torque procedure and verified this approach using IDA analysis. Four types of building models were modeled for analytical study, namely S1, S2, R1, and R2. The S1 model was an asymmetric one- way system with $e_S = 0.1\ bw$. The S2 model was a two-way asymmetric system in both directions with $es = 0.05\ bw$. Model R1 contained only x-direction uniform distribution of strength, whereas model R2 contained uniform distributions of strength in both directions. The analytical study results showed that the base shear torque surface was effective in predicting the stiffness center location. The predicted seismic response was comparable to the IDA analysis response.

D. A Plan asymmetric Multistorey structures

Because of their simplicity and ability to clearly depict the effect of different seismic response parameters, single-story models were widely used in previous analytical studies on irregular plane structures. Most design criteria have been formulated based on results obtained in single-storey models. But several researchers have shown that single- story models have resulted in inaccurate torsional response prediction. Modeling and analysis of multi-storey building models has been made much simpler by developing powerful software tools. Multi-storey construction models provide realistic torsional response prediction. Modeling and analysis of multi-storey building models has been made much simpler by developing powerful software tools. Multi-storey construction models provide realistic torsional response prediction. Although studies of irregular plan building models began in the 1990s *[Killar and Fajfar 1997, 2002; Moghadam 1998; but, Fajfar et al. (2002)]* were one of the leading researchers in this field who proposed a new method that was an extension of the N2 method. The method proposed applied to the realistic 3D construction models. An eight-story R.C. for analytical study. The construction was modeled on structural walls. Comparison of the results of the proposed procedure with the results of non-linear dynamic analysis. The ability of the proposed method to predict the seismic response of torsionally rigid structure was justified from the comparison of results. The method did not, however, include the effects of lateral torsional coupling and was found to be unconservative in comparison with the method of N2.

De-la-Colina (2003) conducted evaluations of several code-specified procedures for analyzing procedures for multi-story building systems with weight and rigidity irregularities subject to bi-directional seismic excitation (EI Centro earthquake). Analytical studies have been performed on several 5 floor buildings with eccentricity of mass and stiffness. Researchers used shear beam models to represent resistant elements. The authors had found the optimal values of storey eccentricity based on the code-defined procedures

Chopra and Goel (2004*)* have proposed a new method based on their earlier method being extended *(Chopra and Goel 2002)*.

The torsional amplification of the structure was accounted for in the proposed method by applying the lateral forces in combination with the torsional moments at each structure floor. From the structure's modal analysis, lateral forces and torsional moments were obtained. For building systems with different uncoupled lateral to torsional vibration periods, a comparison was made between the results of the proposed method and the non-linear dynamic analysis. The accuracy of the proposed procedure for symmetric structures was verified from the analytical study results. However, the accuracy of the proposed procedure decreases with the increase in torsional coupling magnitude due to the use for modal combination of full quadratic combination (CQC) rule.

Fajfar et al. (2005) again proposed a new method based on the N2 method in correlation with his earlier studies. With the results obtained from linear dynamic analysis, modal responses obtained from pushover analysis of 3D structures were made in the proposed method. The displacement and deformation distributions along height were controlled by N2 method in the proposed procedure, and linear dynamic analysis defined the magnitude of torsional amplification.

Stathopoulos and Anagnostopoulos (2005) were one of the few researchers who attempted to evaluate nonlinear analysis of the torsional response of realistic 3D structures (both according to EC8:2004 and UBC 97). The authors conducted analyticalstudies with bi-directional excitations on realistic 3 storeyed and 5 storeyed RC framed buildings (with flexible and rigid edges). From the results obtained (multistory structures) it was found that the flexible side of the inelastic displacement was higher than the stiff side. The results obtained for single-story structures, however, were contradictory to the results obtained for multi-story structures with mass irregularity under bi-directional seismic excitation action. The torsionally rigid building systems were observed to undergo less plastic deformation compared to the torsionally flexible building systems. The results obtained from single-storey models contradict these findings.

Penelis and Kappos (2005) proposed a method for determining the inelastic torsional response of single-story and multi-story plan asymmetric structures. Single degree of freedom (SDOF) systems were the models used for analytical studies and incorporated the effects of torsional and translation modes. The spectral load vectors were obtained from the elastic spectral analysis in the proposed method and these load vectors were applied to carry out 3D pushover analysis on the structure. Comparison of the results of the proposed procedure with the results of non-linear dynamic analysis. It was found that in the case of single-story structures the inelastic seismic response obtained by both methods varies by 10 percent and in the case of multi-story structures by 20 percent.

The elastic and inelastic seismic response of five-story steel framed structure with mass eccentricity was determined by *Marusic and Fajfar (2005).* The eccentricity was taken as 5%, 10% and 15% of the dimensions of the plan. Three types of building models have been adopted for analytical study as described in Table 2.5. The first-floor height was maintained for the building model as 4 m and other floor heights as 3.5 m. The bi- directional seismic excitation was subjected to the multistory structure. The results were almost comparable with *Perus and Fajfar (2005)* at flexible edges. However, in the case of rigid edges of torsionally rigid and flexible building systems, the results of both papers did not correlate.

Table 2.5 *Marusic and Fajfar model description (2005)*

Model Name	Description
S	Torsionally stiff building model with moment resistant beam column connections (All beam-column connections).
F1	Building Model with torsional stiffness equal to Model S with moment resistant beam column connections (Corner beams only).

Stefano et al. (2006) identified the difference between the asymmetric structures of the one-story inelastic seismic response and the multi-story plan. For analytical study, the building model created a single story and a six-story steel frame with mass applied at 0.15 *b* of the geometric structure that causes mass eccentricity. The effect of resisting elements' over-strength has also been evaluated. Analytical studies demonstrated the influence of over-strength on the building system’s ductility demand, and this influence showed variation for single and multi-story building systems. Finally, it was found that the seismic response from the single-storey model was different from the response from multi-storey models. From analytical study results it was found that for an eccentricity ratio of less than 0.5, the number of resistant planes in seismic response direction had no influence on seismic response and lateral displacements decreased with increased demand for ductility. Parameters such as degree of torsional coupling, uncoupled lateral time and eccentricity had a greater impact on seismic response.

Ghersi et al. (2006) determined the efficacy of the modal analysis procedure in assessing the asymmetric structure of the inelastic seismic response of the multistory plan. Asymmetry was introduced by variation of application load at 0.15*L* away from the geometric center causing mass eccentricity in a six-story asymmetric steel framed building. Modal analysis results were compared with static analysis results and Chandler's procedure to check the accuracy of the latter. Compared to other methods of analysis, the proposed method yielded good seismic performance of buildings. However, the distribution of strength along the plan given by the proposed method is comparable to the method suggested by Ghersi and Rossi, but it was easier to apply than the latter method.

Aziminejad and Moghadam (2009) have determined the seismic performance of eight asymmetric (Stiffness and strength) building systems with different distributions of strength. The eight different building systems were considered in the stiffness and strength center position (Table 2.6). Using OPENSEES software, these construction models were analyzed using nonlinear dynamic analysis. It was concluded from the results of the analytical study that building systems with strength eccentricity equal to a fourth of the distance between strength and stiffness positions performed better on the criteria of rotation and drift. The effectiveness of accidental eccentricity provisions was evaluated by *Stahopoulos and Anangnopoulos (2010)*. Four types of building models were created by the authors for analytical study. One storey shear beam with stiffness eccentricity and one storey frame models with mass eccentricity respectively were the first and second models.

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The third model and fourth model consisted of three-story building with five- story frame type combining mass and asymmetry of stiffness along plan. Taking into account a bilinear force-displacement behavior with magnitude of strain hardening taken equal to 0.05, the shear beam models were modeled. In the creation of the plastic hinge model, plastic hinge and moment-rotation relationships of Takeda were used to idealize frame members. The one-story and three-story building models were subjected to accidental 0 to $0.05L$ eccentricities, whereas the five-story building model was subjected to an additional $0.1L$ eccentricity in addition to the eccentricities mentioned above. Results of the analytical study suggested that the consideration of accidental design eccentricity (ADE) in the case of one-story shear beam models resulted in a reduction in ductility requirements of edge elements in the case of building systems with a longer time period (Ty). For $Ty > 0.5s$, the demand for ductility for $ADE = 0.05L$ decreases by 10% and for $ADE = 0.10L$ by 10 – 20 percent.

Table 2.6 Configurations of different models considered

S. No.	Model Name	Ratio of Stiffness to yield displacement eccentricity
1	Symmetric	*0*
2	Stiffness Symmetric	*1*
3	Balance (0.75 CV - CR)	*0.75*
4	Balance (0.50 CV - CR)	*0.50*
5	Balance (0.25 CV - CR)	*0.25*
6	Strength Symmetric	*0*
7	De-Stefano (0.25 CM - CR)	*-0.33*
8	De-Stefano (0.50 CM - CR)	*-1*

III. STRUCTURAL MODELLING

The model shapes specified are as follows, shown in Table 3.1

Regular Rectangular shape (A1)	E-Shape (A2)	H-Shape (A3)
T-Shape (A4)	L-Shape (A5)	C-Shape (A6)
I-Shape (A7)	Plus (+) -Shape (A8)	Square with Core (A9)
	Rectangle with Core (A10)	

Table 3.1

The specifications for all the structural models mentioned above are the same and are given as follows, shown in Table (3.2,3.3,3.4)

Live Load	$3\ kN/m^2$
Roof Live Load	$1.5\ kN/m^2$
Finish Load	$1\ kN/m^2$

Table 3.2 Load Data

Earthquake Zone	III
Damping Ratio	5 %
Importance Factor	1.0
Type of Soil	Medium Soil
Type of Structure	All general RC frame
Response Reduction Factor	5 [SMRF]
Time Period	Program calculated
Foundation Depth	2.5 m
Poisson's Ratio	.015

Table 3.3 Seismic Definition

Plan Dimension	$16m \times 20m$
Bay width along X Direction	$4\ m$
Bay width along Y Direction	$4\ m$
Grade of Steel	Fe 415
Grade of Concrete	M25
Size of Beams	$350mm \times 450mm$
Size of Columns	$450mm \times 450mm$
Thickness of Slab	$175\ mm$
Density of Concrete	$25k\frac{N}{m^3}$
Floor Finishes	$1k\frac{N}{m^3}$
Live Load	$3\ k\frac{N}{m^3}$
Roof Live Load	$1.5\ k\frac{N}{m^3}$
Thickness of Outer Wall	$230\ mm$
Thickness of Inner Wall	$115\ mm$
Height of each floor	3.5 m
Zone Factor, Z	0.36
Soil Type	II-Medium

Table 3.4 Structure Data

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The plan is shown below for each building considered in this thesis report,

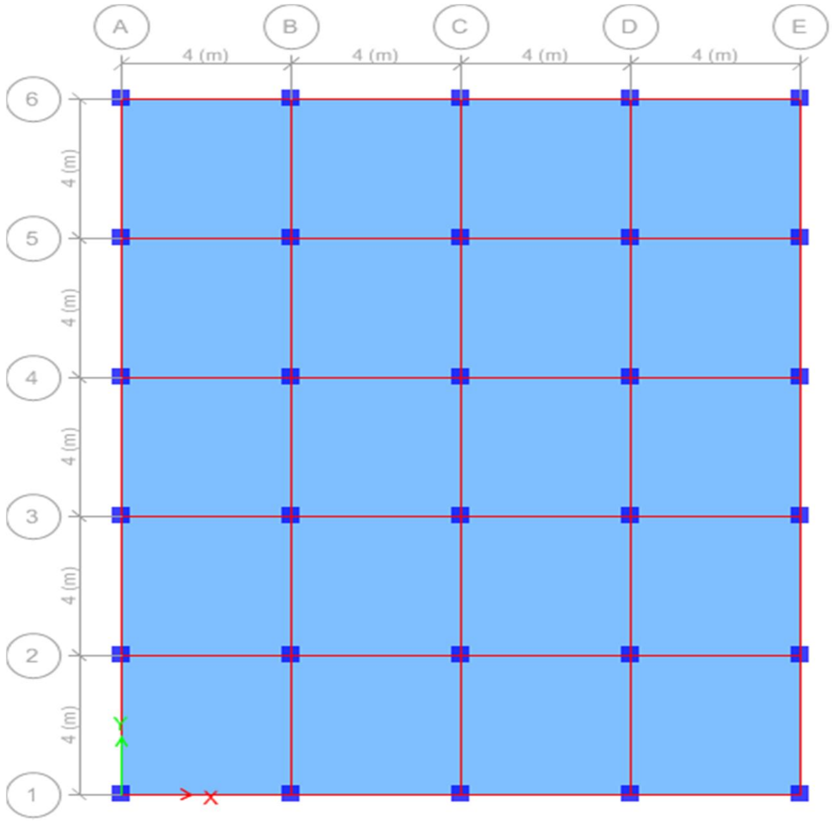

Figure 3.1: Regular Rectangular Shape (A-1) Plan View

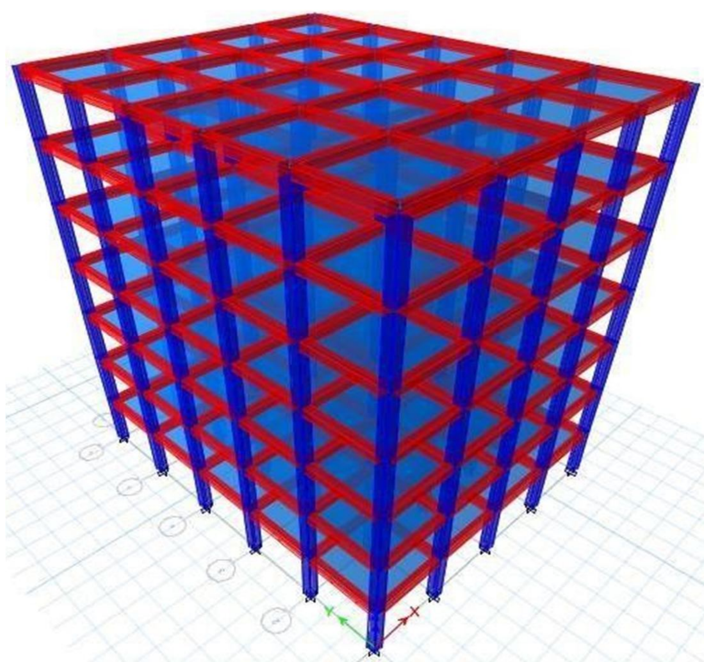

Figure 3.2: Regular Rectangular Shape (A-1) 3-D View

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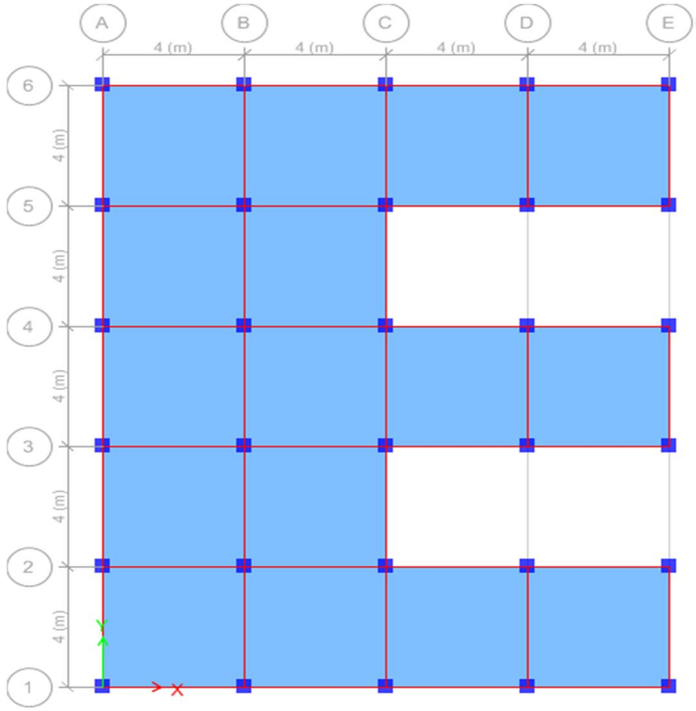

Figure 3.3: E Shape (A-2) Plan View

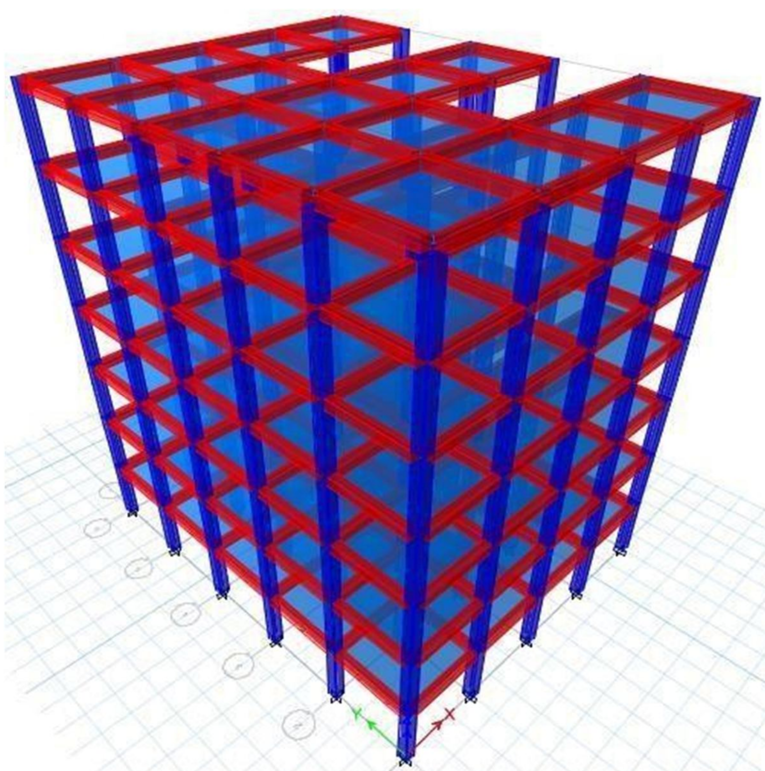

Figure 3.4: E Shape (A-2) 3-D View

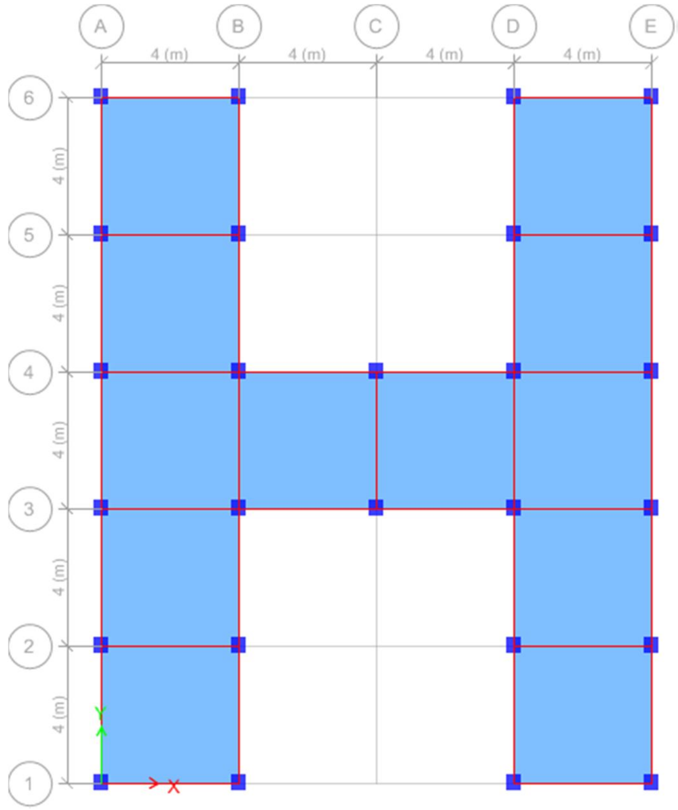

Figure 3.5: H Shape (A-3) Plan View

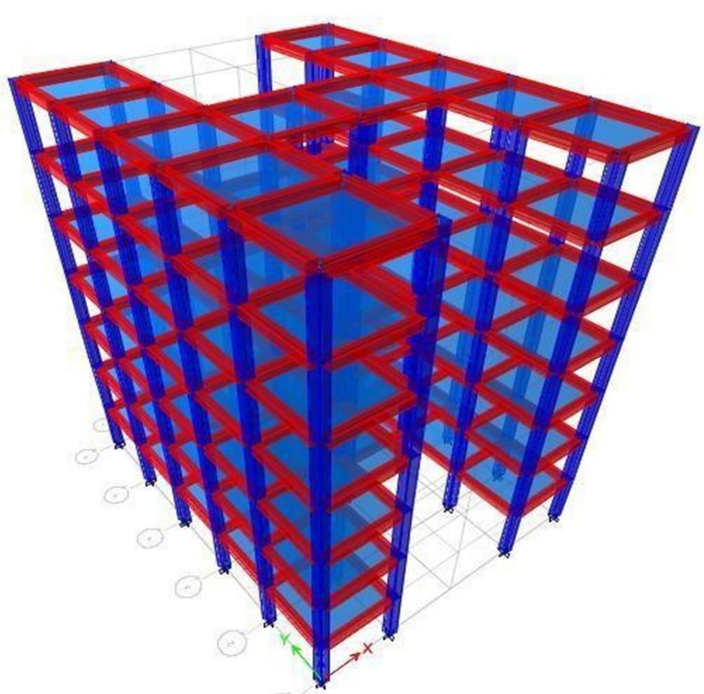

Figure 3.6: H Shape (A-3) 3-D View

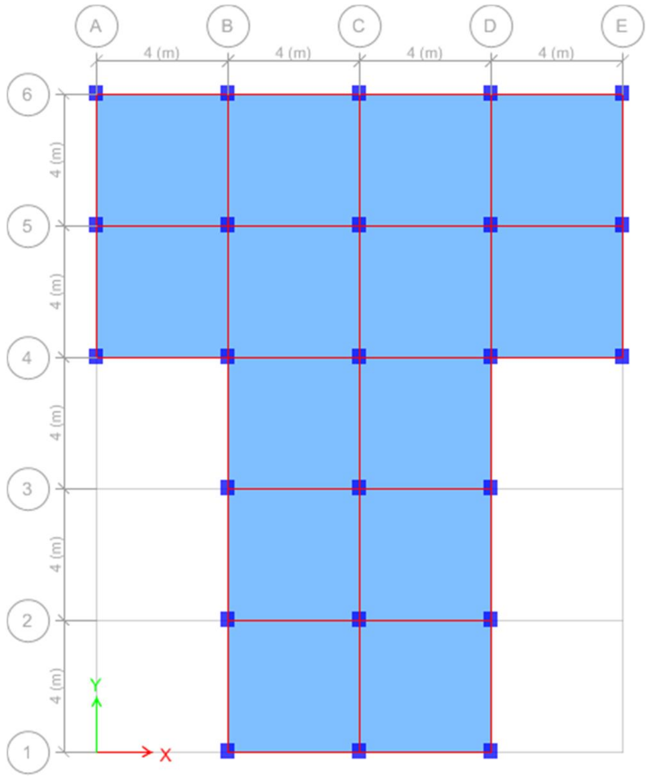

Figure 3.7: T Shape (A-4) Plan View

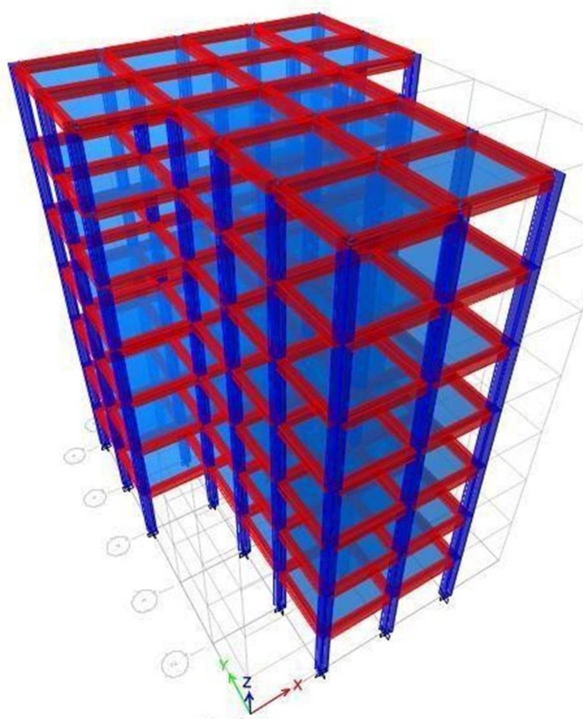

Figure 3.8: T Shape (A-4) 3-D View

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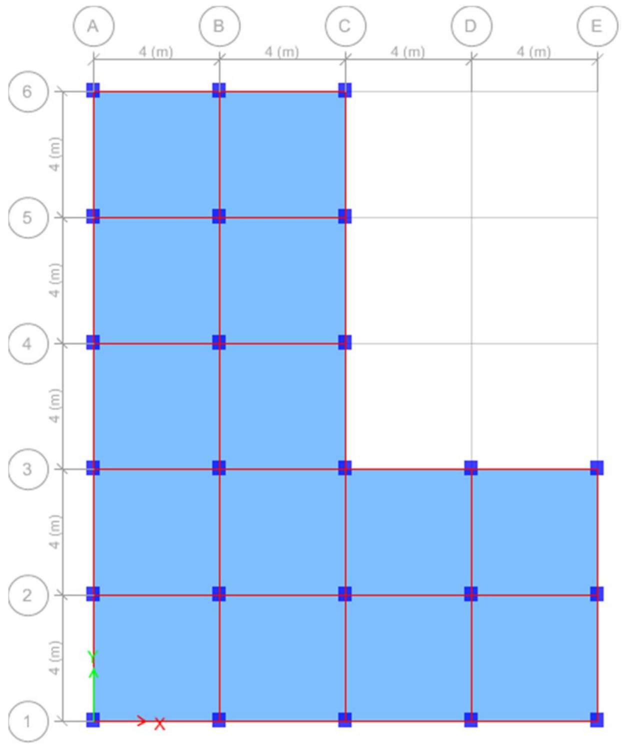


Figure 3.9: L Shape (A-5) Plan View

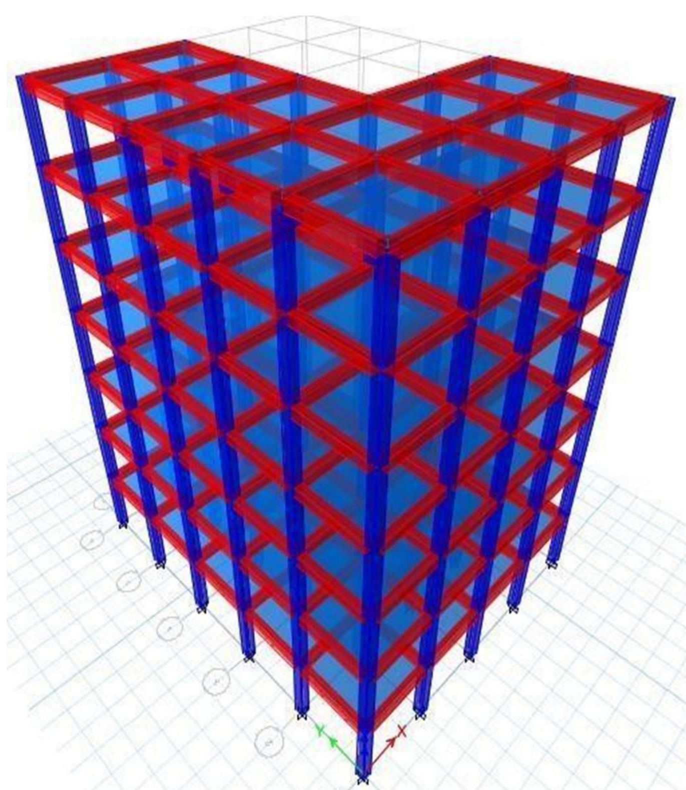
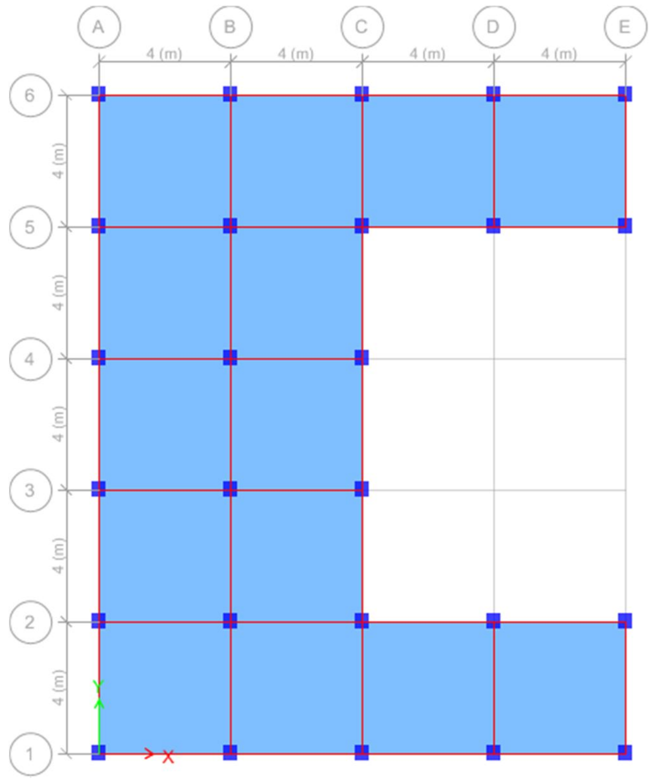


Figure 3.10: L Shape (A-5) 3-D View



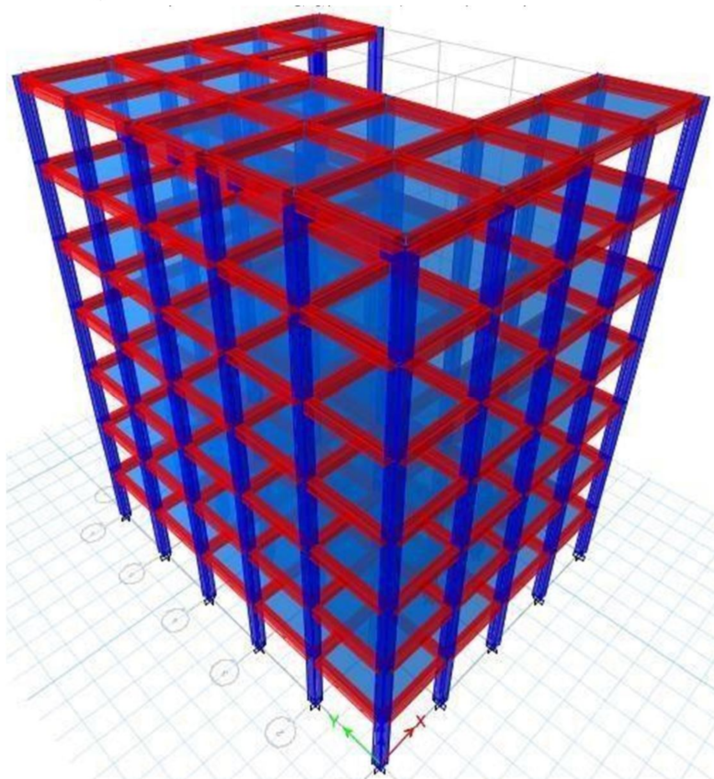


Figure 3.12: C Shape (A-6) 3-D View

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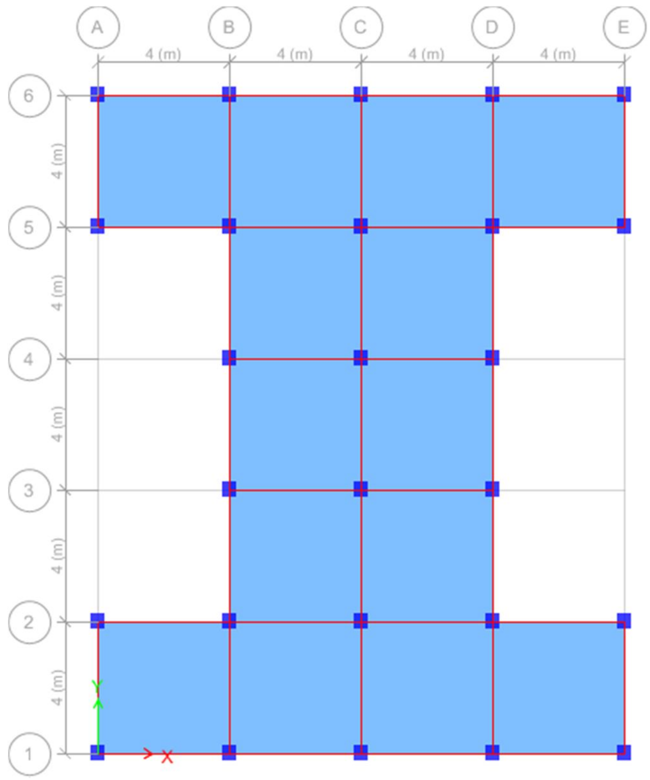

Figure 3.13: I Shape (A-7) Plan View

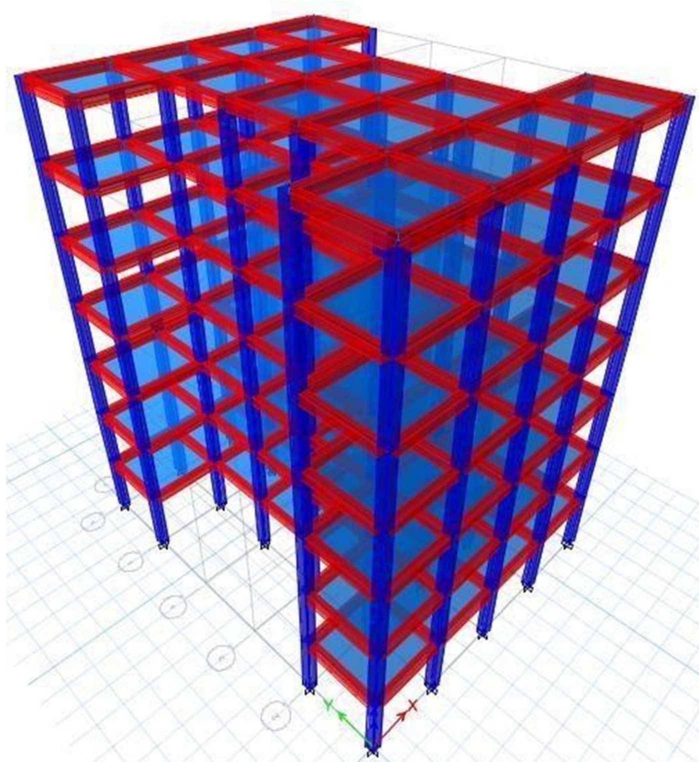

Figure 3.14: I Shape (A-7) 3-D View

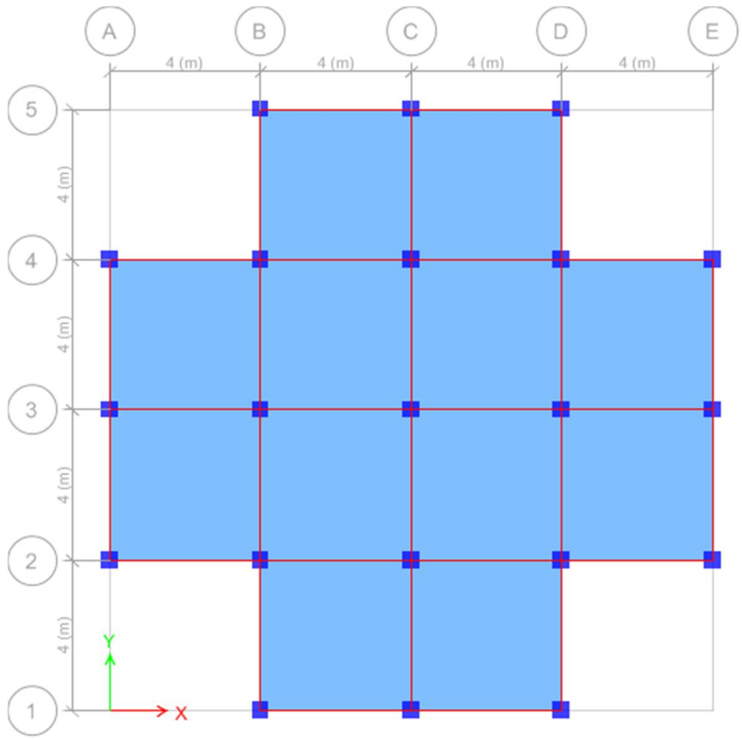

Figure 3.15: Plus (+) Shape (A-8) Plan View

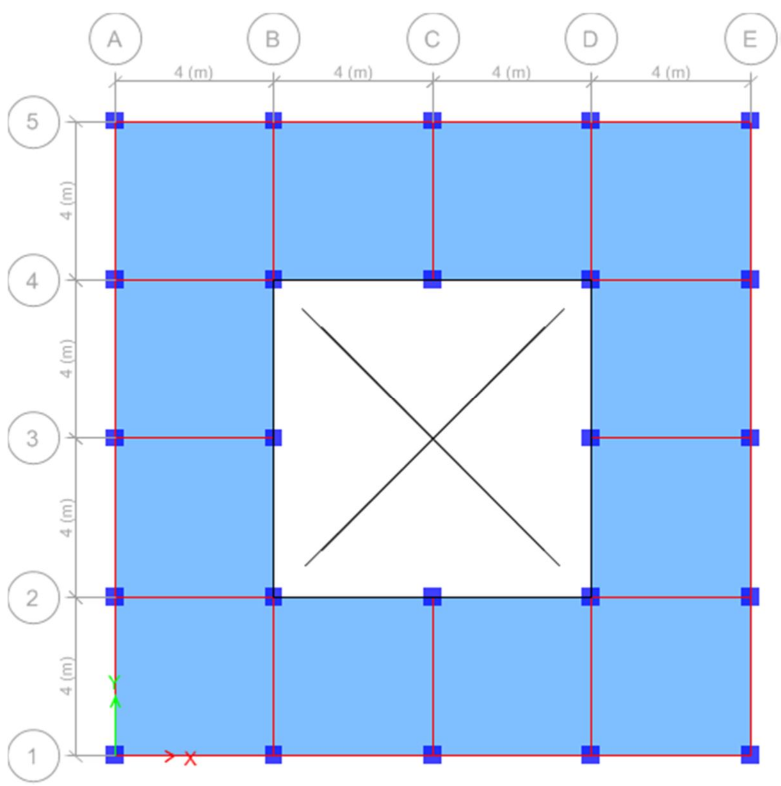

Figure 3.16: Plus (+) Shape (A-8) 3-D View

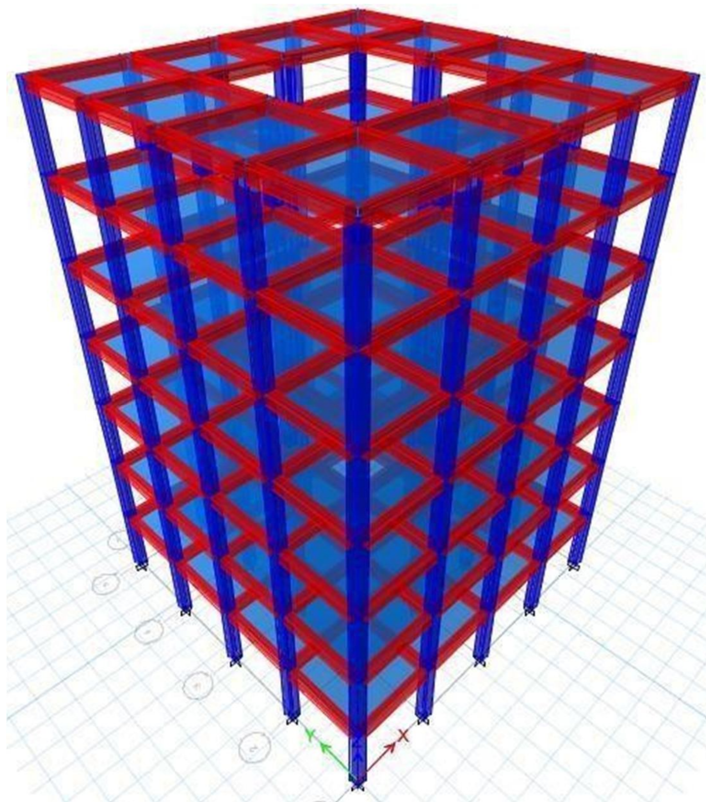

Figure 3.17: Square with core (A-9) Plan View

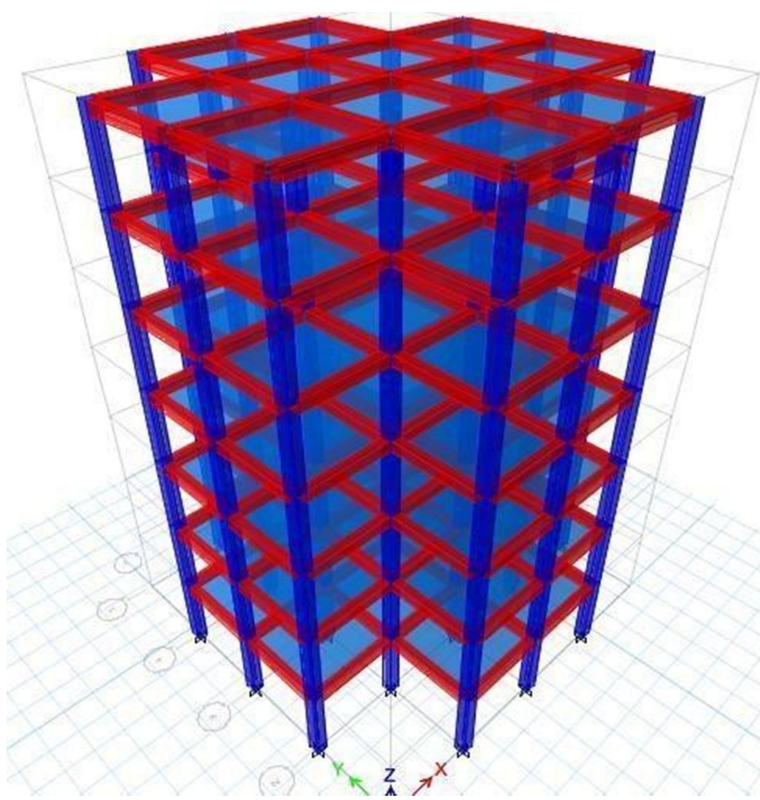

Figure 3.18: Square with core (A-9) 3-D View

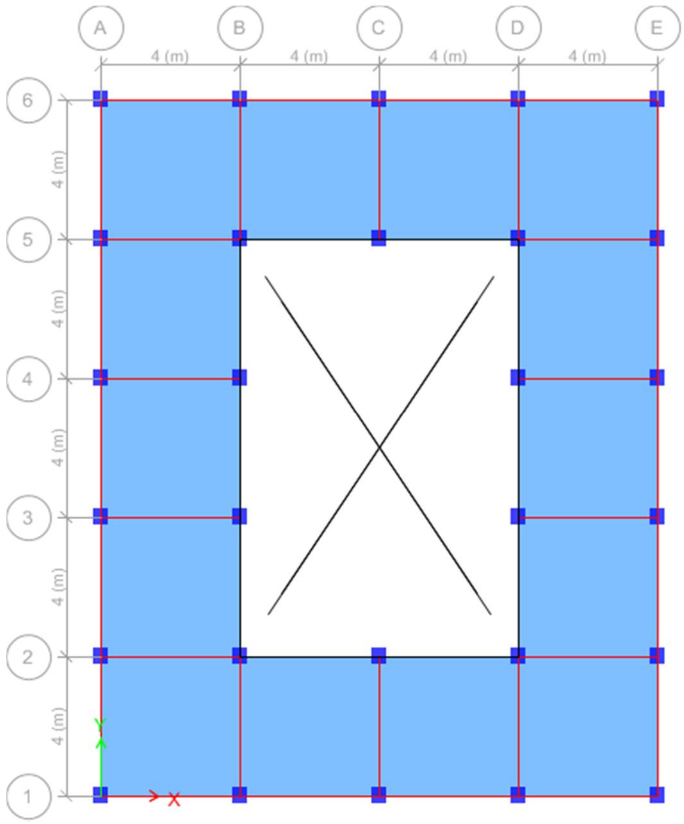

Figure 3.19: Rectangle with core (A-10) Plan View

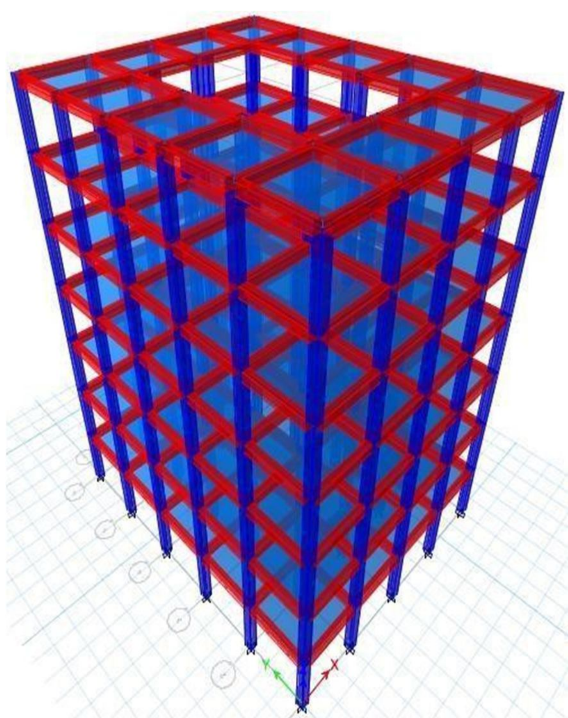
Figure 3.20: Rectangle with core (A-10) 3-D View

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IV. ANALYSIS AND DESIGN RESULTS

A portion of the example investigation and configuration comes after the analysis done in E-Tabs about have been appeared underneath.

ETABS 2016 Concrete Frame Design

IS 456:2000 Beam Section Design (Envelope) Beam Element Details

Section Properties

Level	Element	Unique Name	Section ID	Length (ft)	LLRF
GF	B11	128	B1	14.2	0.894

b (ft)	h (ft)	b_f (ft)	d_s (ft)	d_{ct} (ft)	d_{cb} (ft)
1.5	1.5	1.5	0	0.1	0.1

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt. Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
22360.68	20	1	415	415

Design Code Parameters

Flexural Reinforcement for Major Axis Moment, M_{u3}

γ_C	γ_S
1.5	1.15

	End-I Rebar Area mm²	End-I Rebar %	Middle Rebar Area mm²	Middle Rebar %	End-J Rebar Area mm²	End-J Rebar %
Top (+2 Axis)	1164	0.56	541	0.26	1040	0.5
Bot (-2 Axis)	582	0.28	850	0.41	541	0.26

Flexural Design Moment, M_{u3}

	End-I Design Mu kN-m	End-I Station Loc ft	Middle Design Mu kN-m	Middle Station Loc ft	End-J Design Mu kN-m	End-J Station Loc ft
Top (+2 Axis)	-158.3348	0.8	0	9.4	-157.4616	13.4
Combo	DCon8		DCon14		DCon3	
Bot (-2 Axis)	0	0.8	120.3867	7.1	0	13.4
Combo	DCon8		DCon2		DCon14	

Shear Reinforcement for Major Shear, V_{u2}

End-I Rebar A_{sv} /s mm²/m	Middle Rebar A_{sv} /s mm²/m	End-J Rebar A_{sv} /s mm²/m
1024.85	506.78	1143.98

Design Shear Force for Major Shear, V_{u2}

End-I Design V_u kN	End-I Station Loc ft	Middle Design V_u kN	Middle Station Loc ft	End-J Design V_u kN	End-J Station Loc ft
230.9276	2.1	0.1101	9.4	249.0925	12
DCon3		DCon14		DCon4	

Torsion Reinforcement

Shear Rebar A_{svt} /s mm²/m
0

Design Torsion Force

Design T_u kN-m	Station Loc ft	Design T_u kN-m	Station Loc ft
0.042	3.5	0.042	3.5
DCon5		DCon5	

ETABS 2016 Concrete Frame Design
IS 456:2000 Column Section Design (Envelope)

Column Element Details

Level	Element	Unique Name	Section ID	Length (ft)	LLRF
GF	C9	47	C1	15	0.531

Section Properties

b (ft)	h (ft)	dc (ft)	Cover (Torsion) (ft)
1.5	1.5	0.2	0.1

Material Properties

E_c (MPa)	f_{ck} (MPa)	Lt. Wt Factor (Unitless)	f_y (MPa)	f_{ys} (MPa)
22360.68	20	1	415	415

Design Code Parameters

γ_c	γ_s
1.5	1.15

Longitudinal Reinforcement Design for P_u - M_{u2} - M_{u3} Interaction

Column End	Rebar Area mm²	Rebar %
Top	3258	1.56
Bottom	4549	2.18