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Authors: Necati Mert, Alexis Nzapfakumunsi

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Seismic design evaluation of T shaped irregular RC building plans by using pushover analysis

Alexis Nzapfakumunsi*[1], Necati Mert [2]
*[1]Department of Civil Engineering, Sakarya University/Institute of Natural Sciences, Turkey, MSc. Student
[2]Faculty of Engineering, Department of Civil Engineering Sakarya University, Turkey

Abstract

This study has been conducted to find out how Turkish Seismic Code 2007 is effective in providing the Life Safety (LS) and Performance Level (PL) of T shaped plan irregularities reinforced concrete multi-story buildings; with special lateral loads bearing moment frame systems. For this purpose, a set of 12 multi-story residential buildings: 3-rectangular plans as reference and their respective 9- different T shaped plans of 3-, 6- and 8- stories were considered in the North Anatolian Fault Zone (NAFZ) Turkey and were designed based on the code. Then, a modal and a static non-linear pushover analysis were conducted for all buildings. Base shear forces and roofs displacement for each building were computed. Besides, the formation trend of plastic hinges and their respective distribution in the entire buildings were inspected for evaluating the seismic performances. Results show that for some buildings the expected performance level which is LS has been overreached and even in some cases buildings reach the collapse prevention level. Based on this, it seems that the code provisions still require further upgrading to create enough confidence in the civil engineering community.

Keywords: Pushover Analysis, RC Building, T Shaped Plans, Life Safety (LS), Performance Level (PL)

1. INTRODUCTION

TSC 2007 and many other modern seismic design codes for building systems declare; explicitly or implicitly that the design of buildings based on their provisions and by using different analysis methodologies like *(simplified static analysis, modal analysis, nonlinear pushover analysis and nonlinear time-history analysis)* leads to Life Safety (LS), as their minimum Performance Level (PL). Regardless of that, there are some buildings designed based on the current existing seismic codes and constructed by using high standard materials under a very good supervision either, that have shown inadequate performance levels, and even collapsed in some recent earthquakes. In almost all cases the extent of damage has been so high for the buildings subjected to the earthquake, that the demolishing and reconstruction of the building have become inevitable. All this bring the idea that code provisions and regulations still require further upgrading to create enough confidence in the civil engineering community.

Up to now, regarding the adequacy evaluation of seismic design codes' provisions and their requirements numerous studies have been conducted. In the year 2000, Fajfar presented a relatively simple nonlinear method for the seismic analysis of structures (the N2 method) [1]. It brings together the response spectrum analysis of an equivalent single-degree-of-freedom system and the pushover analysis of a multi-degree-of-freedom model. Same year, Hosseini and

Yaghoobi Vayeghan conducted a study on the design verification of an existing 8-story irregular steel building by both pushover and three-dimensional dynamic analyses [2]. In 2007, Goulet and colleagues presented a state-of-the-art seismic performance assessment through application to a reinforced-concrete moment-frame building designed per 2003 building code provisions [3]. 2008, Virote assessed the seismic performances of reinforced-concrete buildings by nonlinear static analysis (pushover analysis and modal pushover analysis and nonlinear time history analysis [4]. 2012, Epackachi illustrated in his study the linear and nonlinear behavior of one of the tallest RC buildings, a 56-storey structure, located in a high seismic zone in Iran [5]. 2014, Phaiboon Panyakapo studied the performance of 9 story RC building by cyclic pushover procedure [6]. Same year, Thwin carried out computer aided analysis of twelve storied reinforced-concrete rectangular shape residential building for static and dynamic approach by using ETABS software [7]. With regards to multistory buildings, in 2014 Moniri evaluated the results of illustrious characteristics of near-fault ground motions on the seismic response of three reinforced concrete structures (6-Story, 10-Story and 15-Story) [8]. 2016, M.R. Falamarz and Sheikhabasi A. Zerva illustrate the effect of numerical – soil foundation structure modeling on the seismic response of a tall bridge pier via pushover analysis [9]. 2016, Alessandra Fiore and Girolamo Spagnoletti evaluated in their study the prediction of shear brittle collapse mechanisms due to the infilled reinforced concrete building by using pushover analysis [10]. 2017, Shuang Li, Changhai Zhai Lili Xie and Zhanxuan Zuo compared static pushover and dynamic analysis using RC building shaking table experiment [11]. 2017, Mahmood Hosseini and Banafshehalsadat Hashemi evaluated the seismic design of RC buildings for near source earthquakes by using nonlinear time history analyses for the codes IBC 2009 and ACI 318-2014 [12].

The present study has been carried out to find out how Turkish seismic design code 2007 is proficient in providing LS PL in reinforced concrete multi-story buildings of T shaped irregularity with special moment frame lateral load bearing system. For this purpose, a set of 12 multi-story residential buildings: 3-rectangular plans as reference and their respective 9-different T shaped plans of 3-, 6-, and 8- stories were considered in the highest seismic hazard zone of Turkey (NAFZ), assuming site soil classification of Z1 to TSC 2007 [13]. For being more consistent and realistic same materials were used for all the buildings.

2. GEOMETRY AND MODELING OF THE CONSIDERED RECTANGULAR REFERENCE AND T SHAPED BUILDINGS

3-, 6-, and 8-story rectangular concrete moment resisting frame buildings with RC floors and their corresponding various T shaped buildings obtained by removing the corner columns of the front axis ones after ones, were considered. For all the plans 5 by 5 bays, spanning 3 to 4 meters in Y and X directions respectively and an inter-storey height equal to 3 m for each level were used. The dimensions of the columns'sections remain constant for the whole height of the building and are of types 30x60 and 30x30 for model1, 40x60 and 50x50 for model2, 40x80 and 60x60 for model3. All the buildings have the same beams of section dimensions of 30x60 with different span lengths and same slab depth of 12. (units are centimeters, cm).
First, the considered buildings were analyzed and designed based on the regulations of TS 500 - 2000 by using linear elastic approaches as provided by Etabs 2016 integrated building design software. These buildings were assumed to be the existing buildings in Turkey; and then by keeping materials, section properties and loads parameters the same, a full-scale 3D models for all the buildings have been done by means of analytical structural analyzing computer program Etabs 2016. During modeling the weight of slabs was transferred to beams as dead loads accordingly and the columns were assigned a fixed support at the bottom. after that, modal and nonlinear static modal pushover analysis were carried out. Fig.1,2,3 and 4 show the plans and 3D views of the considered buildings, and Tables 1 and 2 present their specifications. Whereas Table 3,4 and 5 give the un-damped natural periods of the first three modes of the designed buildings. The response values which were used for evaluating seismic performance of the buildings included roof displacement and base shear forces, all in both positive and negative main directions. Also, the formation trend of plastic hinges, their corresponding PL as well as their distribution in the entire building were investigated for evaluating the achieved seismic performance.

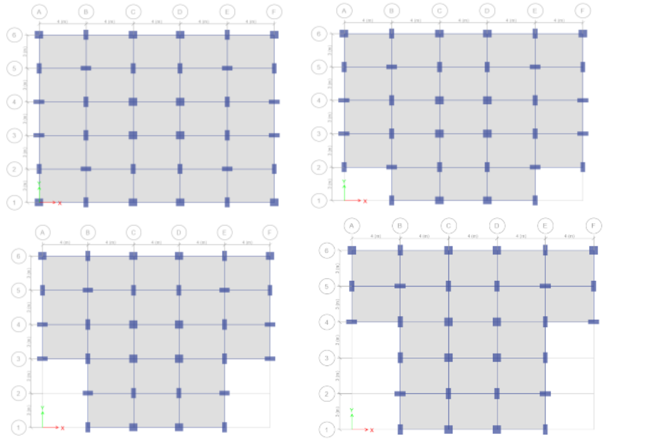


Figure1. Plan views of rectangular reference and T shaped models

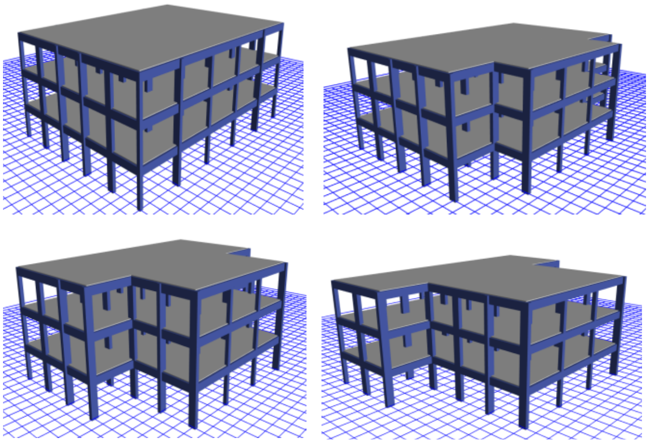


Figure2. 3D views of model1_ref, model1_T1, model1_T2 and model1_T3 (from top left to bottom right)

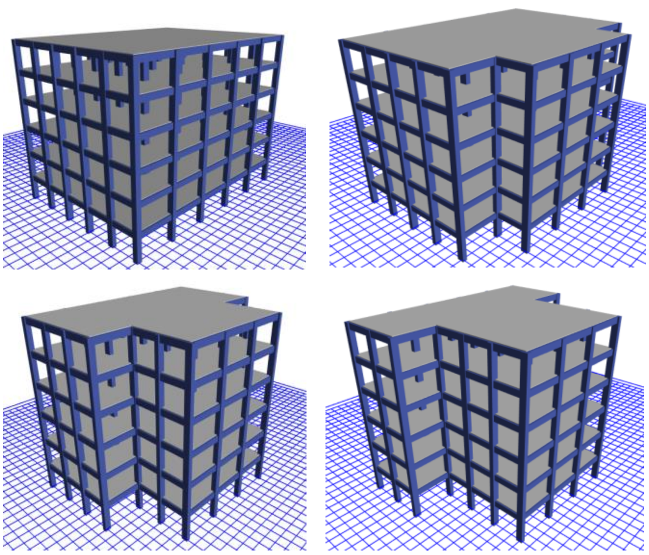


Figure3. 3D views of model2_ref, model2_T1, model2_T2 and model2_T3 (from top left to bottom right)

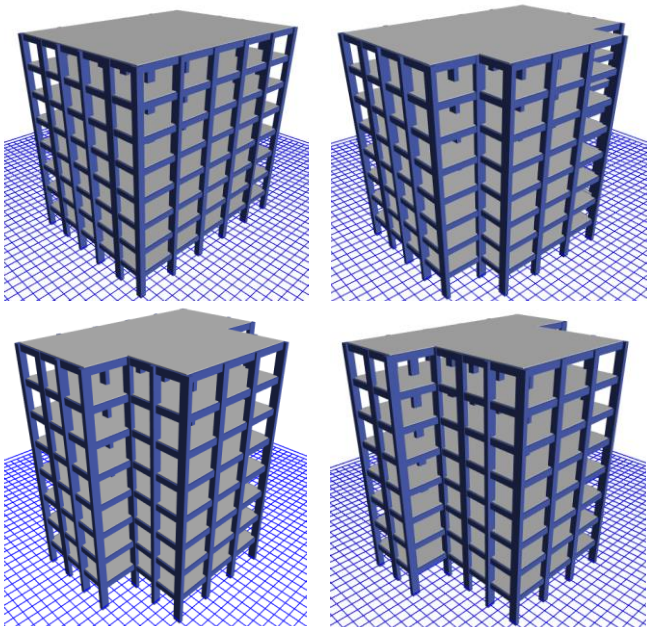


Figure 4. 3D views of model3_ref, model3_T1, model3_T2 and model3_T3 (from top left to bottom right)

Table 1. Material properties of the buildings

	Symbol	Longitudinal & confinement steel	Concrete
Modulus of elasticity	E	200000 MPa	25 743 MPa
Weight per unit volume	W	76.9729 kN/m³	23.536 kN/m³
Specified concrete compressive strength	f'_{ck}	-	30 MPa
Minimum yield stress	f_{yk}	420 MPa	-
Minimum tensile stress	F_{uk}	550 MPa	-
Expected yield stress	f_{ye}	420 MPa	-
Expected tensile stress	f_{ue}	550 MPa	-
Poisson ratio	V	-	0.2

Table 2. Specification of the considered buildings

Items	Quantities
Story height	3m
Dead load on floors due to finishing	1 kN/m²
Live load on floors	2 kN/m²
Partition and exterior walls' load	5.25 kN/m

Table 3. Un-damped natural periods in sec of the first three modes of Model1

	Model1_ref	Model1_T1	Model1_T2	Model1_T3
Mode 1	0.489	0.488	0.493	0.503
Mode 2	0.403	0.404	0.406	0.4
Mode 3	0.395	0.396	0.403	0.399

Table 4. Un-damped natural periods in sec of the first three modes of Model2

	Model2_ref	Model2_T1	Model2_T2	Model2_T3
Mode 1	0.781	0.784	0.787	0.789
Mode 2	0.661	0.666	0.666	0.662
Mode 3	0.644	0.655	0.662	0.662

Table 5. Un-damped natural periods in sec of the first three modes of model3

	Model3_ref	Model3_T1	Model3_T2	Model3_T3
Mode 1	0.973	0.978	0.981	0.987
Mode 2	0.785	0.8	0.81	0.808
Mode 3	0.784	0.793	0.793	0.785

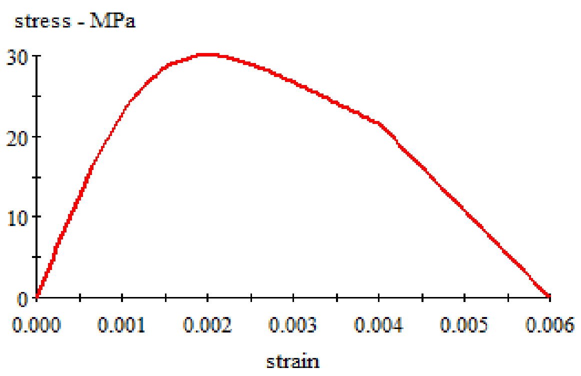
It is clearly observed from Table 3, table 4 and table 5 that the first and second periods which are related to lateral modes in the two main directions (X and Y respectively) are close to one another, as expected. It's also obvious that the reference and the T shaped models' modes 'periods are not much different from each other as the T shaped plans have a reduced mass accompanied by decrease in rigidity due to the removal of some elements. Besides, the third mode of all designed models which is the torsional mode, and its corresponding period in not much different from the lateral modes' periods and in all cases the reference model has a smaller value than all its corresponding T shaped plans. The achievable PL of both rectangular reference and the T shaped designed buildings can be found by static pushover analysis as detailed in the next section.

3. SEISMIC ASSESSMENT OF THE CONSIDERED BUILDING BY USING STATIC PUSHOVER ANALYSIS

During the analysis unconfined and confined Mander models for concrete and the uniaxial steel model with strain-hardening were used [14] [15]. (see fig. 5). Effective rigidity of frame members was taken into account by decreasing 0.7 and 0.3 for column and beam respectively based on TSC 2007 [13]. Torsion constant was reduced to 0.01 to prevent members to undergo torsion and P-delta non-linearity geometry effects were neglected.

Plastic hinges for flexural behavior were defined by a bi-linear moment-rotation relationship. The ultimate moment value was determined for each element using moment–curvature analysis. To simulate the structure sections nonlinearities plastic hinges were defined and applied to both ends of each frame elements. Interacting behavior P-M2-M3 hinges type have been used for Columns and moment M3 hinges type have been used for beams. In fig.6 XTRACT [16] was used to derive the moment–curvature relationships of beams about the X- axis in both positive and negative directions where values were found to be 151.6 kN.m and 79.96 kN.m respectively. Also, the effective yield curvature of $5.468*10^{-3}$ 1/m and $5.312*10^{-3}$ 1/m were computed with respect to the same order.

(*Because of lack of space only few diagrams were presented*)



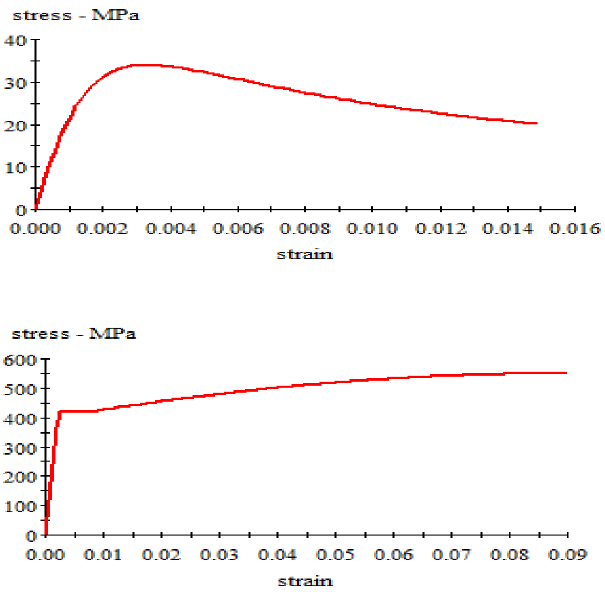


Figure 5. Mander models for unconfined and confined concrete and the uniaxial strain hardening steel model (from top to bottom)

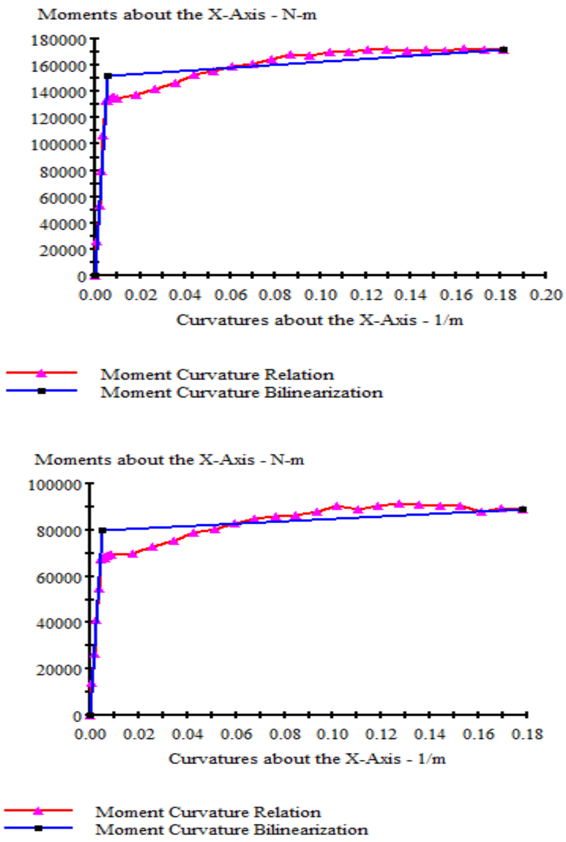


Figure 6. Curvature-Moment relation and idealization for Beams (top positive and bottom negative)

During the pushover analysis, the roof displacement value was at first generated by default by the program but assuring that is kept constant for all the buildings in the same categories; and then the pushover analysis took place Fig.9,10&11. The generated pushover curves and their bi-linear approximation were used to figure out the performance points of all of the reference buildings (model1_ref, model2_ref and model3_ref). (see section 3.1). Secondly, to capture the observed damage in the elements of the considered buildings the pushover analysis was carried out again up to the targeted displacement. The pushover results and the number of formed plastic hinges for x direction can be found in Table 7. Demand response spectrum and capacity diagram in positive X and Y directions are shown in Fig. 8. *(because of lack of space only some results have been shown)*

3.1. Theory and calculation

According to Turkish Seismic Code 2007 [13], by considering the zone (1) and soil types (Z1*: TA = 0.1, TB = 0.3*), where: *A (T) = Ao. I. S (T), Sae (T) = A (T). g*. The Response Spectrum of the selected earthquake is defined as follows:

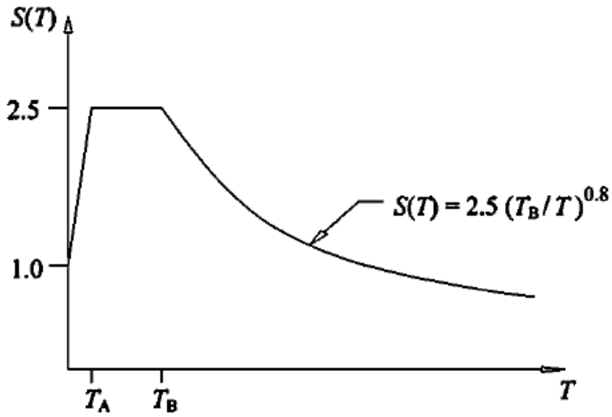


Figure 7. Response spectrum curve

Demand Response Spectrum can be computed by using the formula Sd (T) = Sa (T) / ω^2, where $\omega = 2\pi/T$. In the determination of the Capacity Diagram of the buildings the following formulas have been used: $d_1^{(i)} = u^{(i)}_{xN1} / \Phi_{xN1}\, \Gamma_{x1}$ for spectral displacement and $a_1^{(i)} = V^{(i)}_{x1} / M_{x1}$ for spectral acceleration where:

$$\sum_{n=1}^{Y} M_{xn} = \sum_{n=1}^{Y} L_{xn}^2 / M_n \geq 0.9 \sum_{i=1}^{N} m_i,$$

$\Gamma_{x1} = L_{x1} / M_1$ and

$$L_{xn} = \sum_{i=1}^{n} m_i\, \Phi_{xin}$$

$$M_n = \sum_{i=1}^{N} \left(m_i \Phi_{xin}^2 + m_i \Phi_{yin}^2 + m_{\theta i} \Phi_{\theta in}^2 \right)$$

Table 6. Capacity calculations of model1_ref for X and Y directions

Story	m	Φ_{xN1}	m* Φ_{xN1}	m* Φ^2_{xN1}	L_{x1}	M_1	M_{x1}	Γ_{x1}
3	379.35	0.037	14.03	0.5193				
2	379.35	0.028	10.62	0.2974	29.59	0.88	993.96	33.59
1	379.35	0.013	4.93	0.0641				

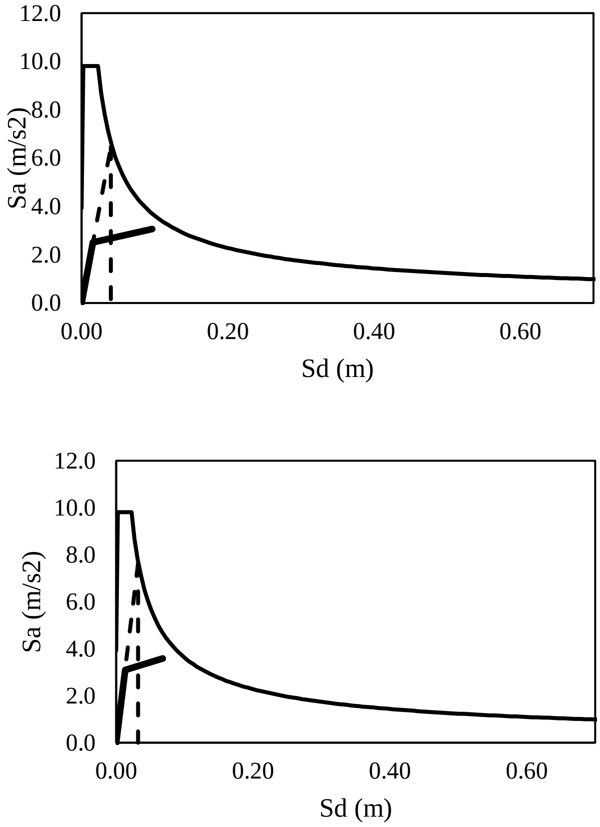


Figure 8. Demand Response Spectrum and Capacity diagram in Positive X and Y directions (from top to bottom)

As it can be observed from Fig. 9, 10 & 11, for the buildings in the same category, in all directions the base shear forces decrease; from model_ref. to model_T3 as the T irregularity increases and in terms of elevations, it decreases from high rise (model2 & 3) to low rise (model1). With regards to the authors knowledge, this should bring the idea that the low-rise buildings can be built more economically by means of performance-based designs rather than following the codes' provisions blindly.

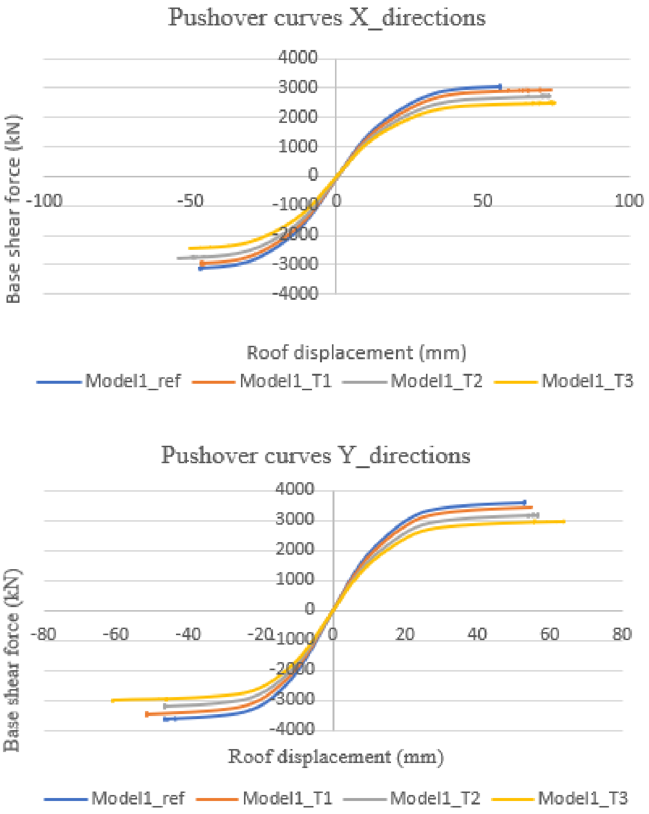


Figure 9. Model1 pushover curves in X and Y positive and negative directions

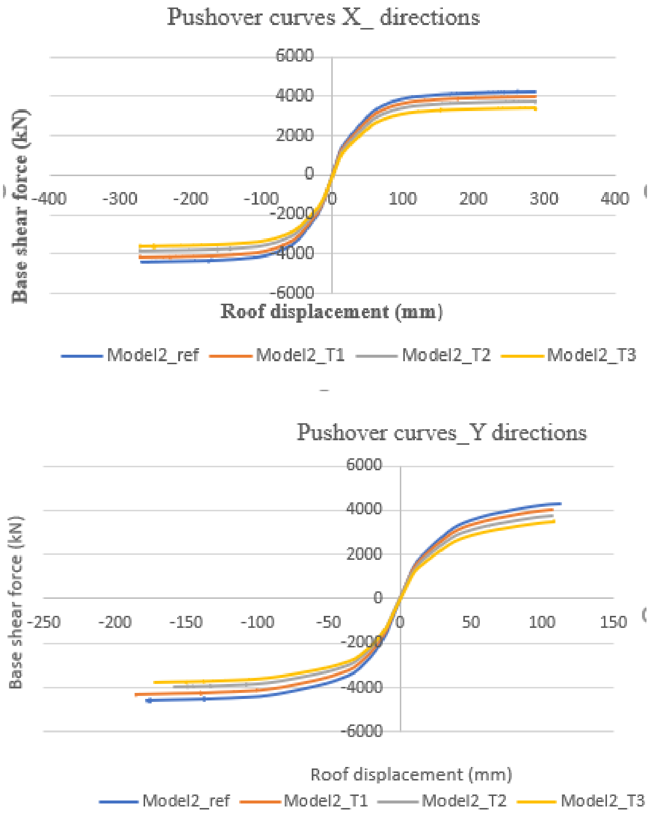


Figure 10. Model2 pushover curves in X and Y positive and negative directions

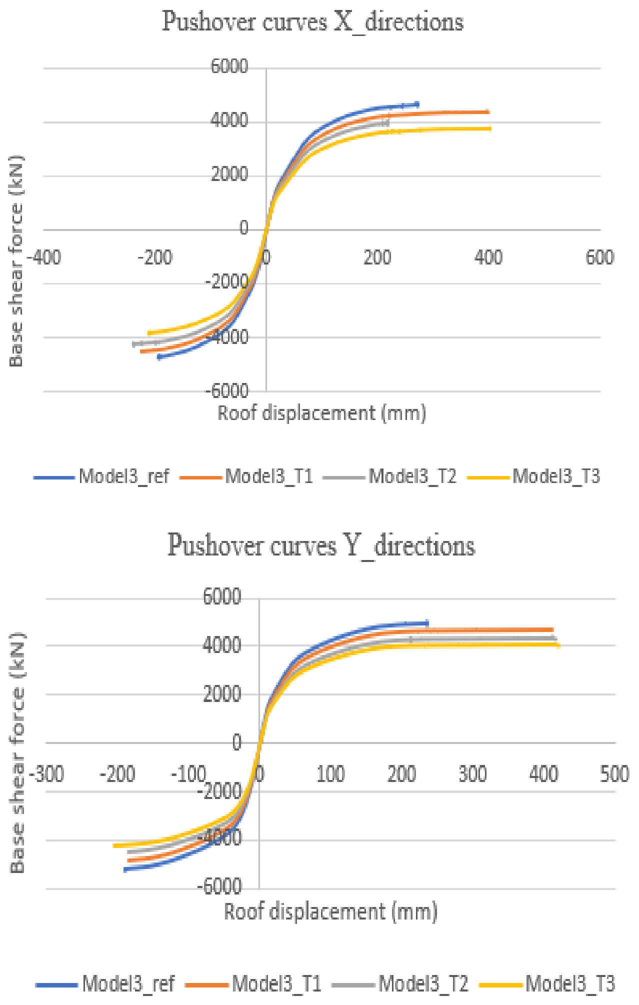
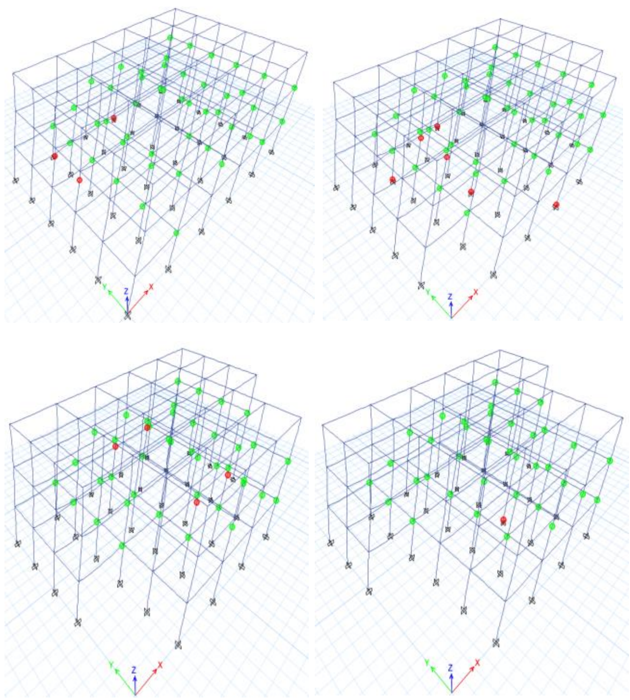


Figure 11. Model3 pushover curves in X and Y positive and negative directions



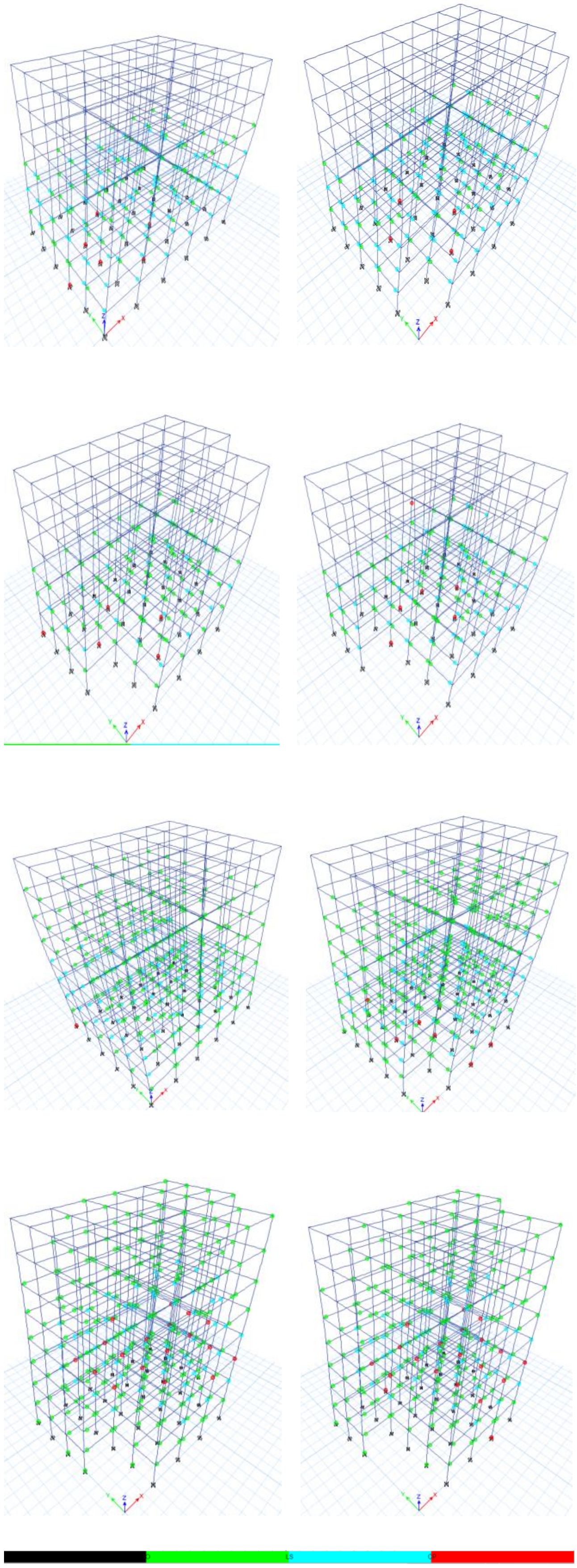


Figure 12. Hinge formations in the considered buildings

	Model1_ref			Model1_T1			Model1_T2			Model1_T3		
Number of Steps	Displacement(mm)	Base shear Force(kN)	No of plastic hinges > CP	Displacement(mm)	Base shear Force(kN)	No of plastic hinges > CP	Displacement(mm)	Base shear Force(kN)	No of plastic hinges > CP	Displacement(mm)	Base shear Force(kN)	No of plastic hinges > CP
0	0.2	0.0	0	0.2	0.0	0	0.2	0.0	0	0.2	0.0	0
1	0.6	59.0	0	0.6	56.1	0	0.6	51.2	0	0.6	46.0	0
2	1.0	117.9	0	1.0	112.1	0	1.0	102.3	0	1.0	91.9	0
3	1.4	176.9	0	1.4	168.2	0	1.4	153.5	0	1.4	137.9	0
4	1.8	235.9	0	1.8	224.3	0	1.8	204.6	0	1.8	183.8	0
5	2.2	294.9	0	2.2	280.4	0	2.2	255.8	0	2.2	229.8	0
75	32.4	2780.1	0	32.8	2642.2	0	32.0	2410.3	0	31.7	2168.6	0
76	32.8	2792.2	0	33.2	2653.8	1	32.4	2421.5	0	32.4	2189.0	0
77	33.2	2804.0	0	33.6	2665.3	1	32.8	2431.9	0	32.8	2198.8	0
78	33.6	2815.7	0	34.0	2676.7	1	33.2	2442.4	0	33.2	2208.3	0
79	34.0	2827.3	0	34.4	2688.0	1	33.6	2452.6	1	33.6	2217.8	0
80	34.4	2838.1	0	34.8	2698.6	1	34.0	2462.8	1	34.0	2226.9	0
81	34.8	2848.9	1	35.2	2709.0	1	34.4	2472.9	1	34.4	2235.7	0
82	35.2	2859.4	3	35.6	2719.2	1	34.8	2482.6	1	34.8	2244.0	0
83	35.6	2869.7	3	36.0	2729.1	1	35.2	2492.0	1	35.2	2251.8	0
84	36.0	2878.1	3	36.4	2737.0	1	35.6	2501.0	1	35.6	2259.5	0
85	36.4	2886.2	3	36.8	2744.4	1	36.0	2509.5	1	36.0	2266.8	0
86	36.8	2893.4	3	37.2	2750.9	1	36.4	2517.3	1	36.4	2273.7	0
87	37.2	2900.1	3	37.6	2757.3	1	36.8	2524.8	1	36.8	2280.2	0
88	37.6	2906.3	3	38.0	2763.7	2	37.2	2531.7	2	37.2	2286.4	0
89	38.0	2912.4	3	38.4	2770.1	2	37.6	2538.0	2	37.6	2292.0	0
90	38.4	2918.5	3	38.8	2776.1	2	38.0	2544.3	2	38.0	2297.4	1
91	38.8	2924.5	3	39.2	2782.1	3	38.4	2550.2	2	38.4	2302.3	1
92	39.2	2930.0	3	39.6	2787.8	4	38.8	2555.9	3	38.8	2307.2	1
93	39.6	2935.3	3	40.0	2793.2	5	39.2	2561.3	4	39.2	2312.0	1
94	40.0	2940.7	3	40.2	2795.4	6	39.6	2566.6	4	39.6	2316.8	1
95	40.2	2942.6	3	.	.	.	40.0	2571.8	4	40.0	2321.5	1
96	.	.	.	.	.	.	40.2	2574.2	4	40.2	2323.1	1

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4. CONCLUSIONS

Results show that for both rectangular and T shaped residential buildings the expected performance level which is LS has been overreached and even in some cases the buildings reach the collapse prevention level. This overreach can be mainly due to the fact that the models used in this study neglect foundation flexibility (*and many other elements that contribute to their strength like infilled walls*) as many of the observed points in the collapse prevention level can be found on the fixed bottom end of ground stories' columns. Based on this, code provisions still need improvement, especially on the flexibility of numbers of elements and their locations that can reach collapse prevention level without preventing the performance of the entire building from being in the Life Safety zone. Other studies can be done by taking into account the foundation flexibility or using other methodologies.

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