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EFFECT OF INFILL WALLS ON STRUCTURAL BEHAVIOR OF RC BUILDINGS WITH VERTICAL IRREGULARITIES

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Abstract

The building structures are generally irregular and regularity is only an idealization that rarely occurs. Structural irregularity may be classified as in- plan and in-elevation. In this study, vertical irregularities are in consideration since less research has been done compared to the in-plan irregularities. Vertical irregularities designated in buildings are due to many reasons such as mass, stiffness and strength irregularity and the dynamic behavior of the structures are totally related with those three parameters. Previous researchers focused on mostly strength and stiffness properties of the models employed.

In practice, infill walls may become one of the reasons of vertical irregularity resulting in the soft story and short column. Misapplication or architectural aesthetics may result in unexpected failures at story columns due to short column effects. On the other hand, recent studies and seismic performance of RC buildings from recent earthquakes showed that infill walls have a significant effect on the strength capacity of the building and should be modeled in analysis and design.

To be able to investigate the effect of vertical irregularities due to Infill walls, i.e. soft story effect, a reliable and accurate infill wall model that simplifies modeling and decreases computational effort is needed. Thus, the most appropriate modeling approach for infill walls will be investigated to be able to get reasonably good and accurate results from the numerical models. In this study ASCE, 41 strut methodology is compared with experimental results and is modified for future studies. OpenSEES is used for modeling and comparison of the numerical models with experiments. Vertical irregularities are assigned to the regular frames based on stiffness and strength change. Computed demands are compared with the design parameters defined in IBC (International Building Code) and TEC 2007 (Turkish Earthquake Code).

Keywords: Vertical irregularity, infill wall, MRF, RC

1. Introduction

Studies on vertical irregularities are still state-of-art. Due to drawbacks of irregular structures, engineers are less confident to design these type of buildings. Vertical irregularities designated in buildings are due to many reasons such as strength, stiffness and mass irregularities and the dynamic behavior of the structures are directly related to those three parameters. A common case that leads to soft and weak stories is the use of infill walls at stories above the first one that is generally used for commercial shops. Due to their well-acknowledged contribution to stiffness and strength an unexpected failure occurs on the first floor as observed in many earthquake damages in building structures.

Due to the influence of infill walls that are considered to change the failure mechanism, their effect on the response of buildings is investigated in this study. The main focus of this study is on weak and soft story mechanisms at first story and relevant IBC and TEC2007 design parameters.

2. Literature

Recent studies on vertically irregular structures have clarified that discontinuities of mass, stiffness or strength along the height, considered by current seismic codes as irregularities in elevation, do not necessarily result in actual increases in plastic demands and, more generally, poor seismic behavior. Thus, the criteria in major international codes aimed at identifying vertical irregularities seem to penalize such discontinuities excessively and that codes are in need of improvement in order to define indicators that actually predict irregular behavior.

Costa et.al. (1988) studied a 16-story high building frame with three different horizontal layouts and five vertical configurations. They idealized buildings as a set of plane moment resisting frames connected to shear walls by rigid diaphragms. They studied 10 acceleration time series for different behavior coefficients. They found out that ductility demands in the frame and shear walls are almost same for regular buildings. For irregular ones, the ductility demand was found nearly twice the ductility demand for regular buildings. Al-Ali and Krawinkler (1998) investigated the seismic behavior of vertically irregular structures. They used mass, stiffness and strength quantities as variable parameters and obtained demands from elastic and inelastic dynamic analysis. They found that strength reduction factor of 2.0 is sufficient to cause most of the hysteretic energy demands to be dissipated. Their results showed that for the cases with combined stiffness and strength irregularities, nonlinear response yields to the strength irregularity cases. They stated that strength modifications less than 1.2 are sufficient to change the ductility distribution over height from highly nonlinear to a uniform one except the top story. The highest amplification of ductility demands occurs when the weak story is at the mid-height. Nassar and Krawinkler (1991) studied on the seismic demands for SDOF and MDOF systems. Their concern was to assess seismic design parameters. They observed that the base shear capacity depends on the failure mechanism and overturning moments in inelastic MDOF structures can be very large.

Shahrooz and Moehle, (1990a, 1990b), studied the setback structures. A one-quarter scaled six-story two-bay by two-bay ductile moment-resisting reinforced concrete test structure, which has 50% setback at mid-height, was used to determine the seismic response. They established guidelines by which to detect configurations for which concentrations of damage in tower members are likely and proposed a static lateral-load design method to improve performance.

Valmundsson et. al. (1997) studied on the mass, strength, and stiffness limits evaluation for Uniform Building code (UBC) based on two-dimensional building frames with 5, 10, 20 stories. They observed that while the ratio of the mass of one floor to the next is 1.5, ELF overestimates the base shear approximately 10% when compared with the uniform distribution of mass and the expected increase in ductility demand was not greater than 20%. Base shear can be obtained reliably based on ELF for the mass ratio up to 5.0. Das and Nau (2003) investigated the definition of irregular structures for different vertical irregularities: stiffness, strength, mass, and that due to the presence of nonstructural masonry infill. They analyzed 78 buildings with various inter-story stiffness, strength, and mass ratios for a detailed parametric study. They considered 5, 10, and 20-story special moment resisting frame (SMRF). They observed and proposed that, if stiffness or strength irregularity exists at the first-story level, a higher over-strength ratio (presently 1.2) can be used for the first story columns. Also, they suggested that design shear should be based on the maximum probable strengths of the captive columns of the buildings with nonstructural infill walls. They observed that ELF (UBC) method has an acceptable accuracy for the design of vertically irregular structures.

Chintanapakdee and Chopra (2004) investigated vertical irregularities by using the results of modal pushover (MPA) and response history analysis (RHA). They designed forty-eight irregular frames, all 12-story with strong columns and weak beams. They observed three irregularity cases; stiffness, strength and combined stiffness and strength at eight different locations along the height using two modification factors. They calculated bias and dispersion of MPA estimate and observed that: the bias in MPA procedure did not increase, MPA procedure is less accurate relative to the regular frames in estimating the seismic response of frames with irregularities, in spite of the larger bias in estimating drift demands, and MPA procedure identifies stories with the largest drifts with sufficient accuracy.

Ko and Lee (2006) investigate a 17-storey high rise, 1/12 scaled mock-up, tested under shaking table test, with a high degree of torsional eccentricity and soft-story irregularities in the bottom two stories. They observed coupled torsion and transitional modes together. Lee and Koo (2007) tested three 1:12 scaled 17-story RC wall building models having different types of irregularity (frame system, the shear wall at the interior frame, and shear wall at exterior frame) at the lower two stories at shaking table. Their results showed that building with frame systems and shear wall at exterior frame system lead to almost same lateral displacement. The maximum values of the base shear and overturning moments appear to be similar. The amount of the total absorbed energy was observed as almost similar to three models.

Fragiadakis et. al. (2006) four types of vertical irregularities such as stiffness, strength, combined stiffness and strength, and mass by using a methodology based on the incremental dynamic analysis (IDA). They observed that: combined stiffness and strength and only strength irregularities are more effective than mass and only stiffness irregularities; mass irregularities' effects are reciprocal; the effects of irregularities are highly dependent on the record selection. Karavasilis, et. al. (2008) studied on an extensive parametric study on the inelastic seismic response of steel moment-resisting-frames with vertical mass irregularity. They designed 135 frames according to European seismic and structural codes and these frames were subjected to 30 earthquake motions and their scaled versions to be able to get different limit states, excluding near fault effect. They offered a formula which estimates inelastic deformation demands. Athanassiadou C.J. (2008) studied the effects of the ductility class in EC8 for multistory RC concrete frame buildings, irregular in elevation. He studied with six ten-story frame buildings for high and medium ductility classes under the same peak ground acceleration (PGA) of 0.25g. Seismic performance of all irregular frames observed as equally satisfactory. Interstory drifts of the irregular frames are not exceeding 0.40% for the design earthquake and 1.0% for the collapse prevention one (adapted failure values are 2% - 3%).The over the strength of the irregular frames are found same as the regular ones and pushover analysis underestimates the response quantities at the upper stories of the irregular frames.

Sadashiva et. al. (2012) studied coupled vertical stiffness-strength irregularities of 3, 5, 9, and 15-story steel building frames with a constant mass at each floor level. They observed and concluded that the ELF method is not allowed to be used for the design of the irregular structures due to the codes do not have a systematic quantitative justification for irregularity. They developed simple equations to rapidly estimate the likely increase in median peak ISDR (inter-story drift ratio) due to coupled stiffness–strength irregularity.

In practice, infill walls may also become one of the reasons of vertical irregularity. Misapplication or architectural aesthetics may result in unexpected failures at story columns due to short column effects. On the other hand, recent studies (Hashemi and Mossallam, 2006; Das and Nau, 2003) and seismic performance of RC buildings from recent earthquakes showed that infill walls have a significant effect on the strength capacity of the building and should be modeled in analysis and design. For further investigation, infill walls are also considered in the models for the assessment.

Mehrabi et.al (1996) tested infilled RC frames monotonically and reversed cyclically. They investigated the failure mechanism of the infill walls and gave a damage index for failure mechanisms. Crisafulli et.al (1997,2005) studied analytical modeling of the infilled frames and compared the results with experiments. They studied the similarities and differences of the infill wall models used in literature. Dolsek and Fajfar (2002) developed an analytical model for infilled reinforced concrete frames based on the dynamic test results. They stated in their research that the most uncertain part of their model is the contact region of infill and reinforced concrete frame. The authors also emphasized that the results may change dramatically even for previously damaged frames and it is thought hard to estimate. El-Dakhakhni vd. (2003) developed a three strut model, which captures the failure mechanism, for infilled steel frames. The proposed model gives the opportunity to make a nonlinear analysis of the infill walls. This three strut model based on the contact region of the infill wall to the frame. Öztürk (2005) studied performance assessment of the infilled frames based on FEMA 273 and Smith and Carter's methods. The only linear assessment was considered in his study. Shing P. vd. (2009) studied performance assessment of infilled RC frame shaking table experiments and quasi-static tests. They used micro modeling with finite element methodology to capture the experimental results. Fenerci (2013), Redmond et.al. (2015) and Ezzatfar et.al. (2014) studied pseudo-dynamic experimental set-up with micro and macro modeling approaches and made an assessment based on the criteria's on ASCE 41. Tabeshpour et.al.,(2012), summarized the preceding researchers' studies for the parameters used in infill wall modeling and the limitations.

3. Modeling and Verification

OpenSees (2015) software was used for the modeling of frames. Force-based element type; nonlinear-beam-column element was used for the columns and beams with fiber section definitions. Concrete02, linear tension softening material model was used. Confined material properties are determined according to the modified Kent and Park material model. Steel02 material, Giuffré-Menegotto-Pinto model with isotropic strain hardening was used for rebar material model. The modeling of the infills is quite challenging. The simplest modeling approach with enough accuracy with the experimental studies was preferred not to lose the scope of the study. ASCE 41 strut model was used at the very first step to understanding the behavior and then was compared to infill walled frame test results. Concrete07 was used for the infill wall material and the only compression was modeled. For infills, three strut models are preferred for the further study of the short column effects (Fig. 1). ASCE41 modeling parameters were preferred (Eq. (4-6)).

$$a = 0.175 \cdot (\lambda_1 \cdot h_{col})^{-0.4} \cdot r_{inf} \tag{4}$$

$$\lambda_1 = \left[\frac{E_{inf} \cdot t_{inf} \cdot \sin(2\theta)}{4 \cdot E_{fr} \cdot I_{col} \cdot h_{inf}}\right]^{1/4} \tag{5}$$

$$z = \frac{\pi}{2 \cdot \lambda_1} \tag{6}$$

where;
h_{col} : Column height between centerlines of beam
h_{inf} : Height of infill panel
E_{fr} : Expected modulus of elasticity of frame material
E_{inf} : Expected modulus of elasticity of infill materials
I_{col} : Moment of inertia of column
r_{inf} : Diagonal length of infill panel
t_{inf} : Thickness of infill panel and equivalent strut
θ: Angle whose tangent is the infill height-to-length aspect ratio, in radians.

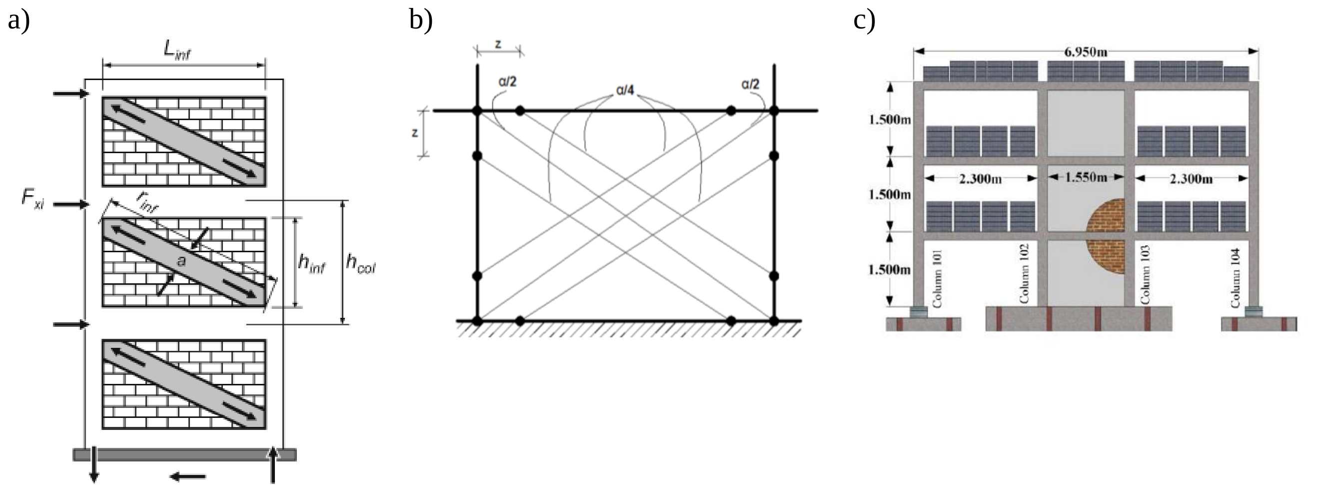


Fig. 1 – a) ASCE41 Concentric compression strut model b) three strut model c) METU Specimen (Fenerci 2013)

Infill wall parameters are defined according to the formulas given in Eq. (7).

$$f_s = \frac{f_m f_t}{1.5(f_m + f_t)} \qquad G = \frac{E}{2(1+\vartheta)} \qquad f_t = \frac{f_m}{10} \qquad f_m > 5 \qquad k = \frac{E w_e t}{2(1+\vartheta)h}$$

$$N_y = \frac{f_s t d}{1000} \qquad \Delta_y = \frac{N_y}{k} \tag{7}$$

Contact length in the models was taken as z/3 based on literature and experimental results and the sensitivity analysis results of Akansel and Moehle (2017). Compression strength of the infill, f_c', is calculated based on the infilled walls axial load capacity calculated from the shear diagonal failure mechanism. Yielding strain has been chosen as 0.003 based on the experimental results from the Mehrabi et.al (1996), Bal et. al. (2008) and (Fenerci (2013), Redmond et.al. (2015), and Ezzatfar et.al. (2014)).

Akansel and Moehle (2017) made some sensitivity analysis for the infill wall parameters and obtained the best-matched values for the METU test specimen (Ezzatfar et.al. (2014)). The parameters, determined based on results of the sensitivity analysis, are given in Table 1. This frame was chosen for sensitivity analysis for being much more realistic than one bay frames. In Table 1, "* f_m" means that the parameter is chosen a coefficient times of f_m. For example, f_t was chosen 0.07*f_m. f_m for this study is 5 MPa.

Table 1 – Sensitivity Analysis Results

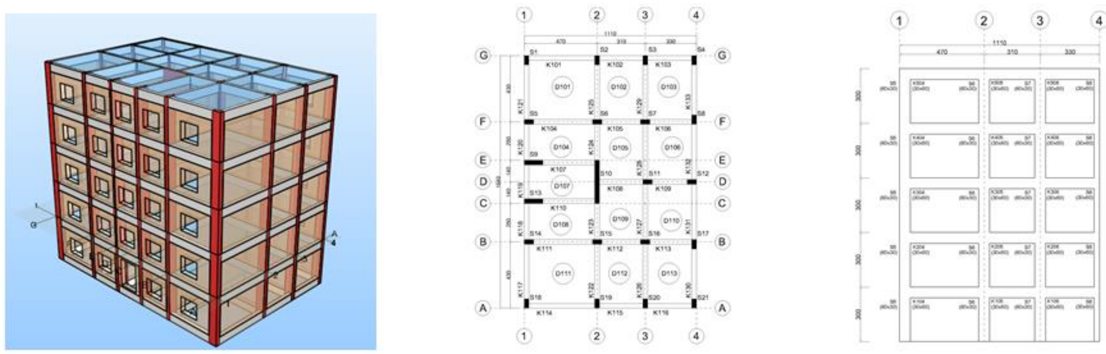
CASE	**f_t (*f_m)**	**E_m (*f_m)**	**Z**	**ε_m**	**x_n**
Decided	0.07	550	z/3	0.003	2

4. Modeling of Building Frame

In this study, a 5 story residence building, which is designed according to TEC 2007 (Turkish Seismic Code), TS498 (Turkish Load Standards) and TS500 (Turkish Design Code), is employed (Fig. 2). This building is modified from an existing one. Dimensions of the building are selected from the range given in previous studies (Bal et.al (2008)). All story heights are 3.0 m and the building plan area is 182 m^2. The compressive strength of concrete is taken as 25 MPa, the yield strength of steel is 420 MPa. The loads used in the design are given in Table 2. The strength and stiffness of the columns were designed to be the same in each story. The stiffness of all beams is taken as the same with different strength range. The columns are designed to be stronger than the beams.

Table 2 – Loads used in design

Concrete density (kN/m^3)	25.0
Internal wall load - 20 cm hollow brick (kN/m^2)	3.3
Live load (at rooms) (kN/m^2)	2.0
Live load (at halls and stairs) (kN/m^2)	3.5



a) b) F-F c)

Fig. 2 – 5-story, a) 3D view, b) plan, c) FF axes frame

The building is designed using PROBINA v.18 software considering that it was in seismic zone 1(highest seismic zone in Turkey). In Figure 6, the elastic and design spectra used are shown. High ductility systems were chosen (R=8) as specified in TEC2007. The first period of the building is computed as 0.461 s.

5. IBC 2012 and TEC 2007 Limits for Vertical Irregularities

Vertical irregularities are specified in IBC2012 (1705.11). These irregularity conditions are designated according to ASCE 7. In this study, only 1a and 5a, soft and weak story limit parameters are considered. "*Stiffness-soft story irregularity is defined to exist where there is a story in which the lateral stiffness is less than 70% of that in the story above or less than 80% of the average stiffness of the three stories above*. *Discontinuity in lateral strength–weak story irregularity is defined to exist where the story lateral strength is less than 80% of that in the story above. The story lateral strength is the total lateral strength of all seismic-resisting elements sharing the story shear for the direction under consideration.*" The IBC soft story parameter is taken as 1/0.7=1.43 because TEC 2007 soft story parameter is the reverse ratio of the drift ratios.

In TEC 2007, irregularities are divided into two parts: irregularities in plan and vertical irregularities.

B1 – Strength Irregularities between neighboring stories (Weak Story):

In RC structures, the ratio of the effective shear area of any floor to the one upstairs' is called as strength irregularity coefficient, ηci. If this coefficient is smaller than 0.8 as defined in Eq 1, it is stated that there exists weak story. The effective shear area at any floor is defined in Eq 2. If the total infill wall area of the i'th floor is greater than the (i+1)'th floor, the infill wall areas are not taken into account when calculating the η_{ci}. If the ηci is within the range given in Eq 3, $(\eta_{ci})_{min}$ is multiplied by structural behavior factor of 1.25 and will be applied to the whole building at all earthquake directions. $(\eta ci)_{min}$ cannot be smaller than 0.6.

$$[\eta_{ci} = (\Sigma A_e)_i / (\Sigma A_e)_{i+1} < 0.80] \quad (1)$$

$$\Sigma A_e = \Sigma A_w + \Sigma A_g + 0.15\, \Sigma A_k \quad (2)$$

$$0.60 \leq (\eta ci)_{min} < 0.80 \quad (3)$$

ΣA_e= Effective shear area at any floor and at the earthquake direction under consideration.
ΣA_g= Cross section area of the shear walls parallel to the earthquake direction under consideration at any floor
ΣA_k= Cross section area of the infill walls (except door and windows holes) parallel to the earthquake direction under consideration at any floor
A_w= Effective cross section area of columns (except the overhangs of columns at the perpendicular to the earthquake direction under consideration)

B2 – Stiffness Irregularities between neighboring stories (Soft Story):

Soft story is defined in TEC2007 as; the case when the ratio of the i'th floor average inter story drift ratio to the (i+1)th or (i-1)'th floor is greater than 2 (Eq 4). η_{ki} is the stiffness irregularity coefficient. Soft story calculations must be done under the 5% eccentricity consideration. Δ, is for inter-story drift and h, is for height.

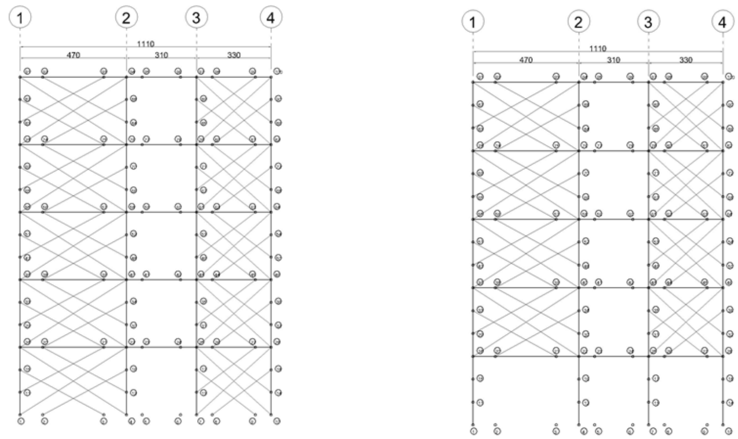
$$\eta_{ki} = (\Delta_i / h_i)ort / (\Delta_i+1 / h_i+1)ort > 2.0 \quad (4)$$

6. Analysis and Results

Linear Static and Modal Analysis were performed to compare and investigate the TEC (2007) and IBC (2012) design parameters. The nonlinear push over analysis was done to determine the inelastic deformations. The frames are generated by changing story heights, the stiffness of columns (CDM) and reinforcements (RM) as given in Table 3. The parameter values are decided based on engineering judgment and code values. The inner frame on FF axis was chosen (Fig.2). The infills at first and the last bay is modeled (Fig.3). To exemplify the cases in Fig 3, "5Story-Infill-H2.4-CDM0.8-RM1.0" means that it is a 5 story building frame with infill walls, 2.4 m height, 0.8 for column dimensions multiplication and 1.0 for reinforcement multiplier for columns. To study the effect of infill walls on the soft story and weak story mechanisms the infill walls on the first story were removed. In Fig. 3, the infill wall arrangements are given for the study cases in Table 3. In plots where analysis results are presented (Fig.4-7), "Infill" means there are infill walls at each story and "No1Infill" means that there are no infill walls at first story bays (Fig.3).

Table 3 – Study Cases for Frames

	Range
First Story Height (m) (H)	2.5, 3.0, 5.0
Column dimension multiplier coefficient (CDM)	0.8, 1
Column reinforcement area multiplier coefficient (RM)	1, 1.5



a) b)

Fig. 3 –a) Infills at each story (“Infill” in Figures), b) Infills at each story except first story (“No1Infill” in Figures)

Linear Static Procedure was applied based on TEC (2007). The weak story identifier parameter, n_{ci} was found below the 0.8 limit when the first story infill walls have been removed (Fig. 4). Soft story mechanism identifier, n_{ki} is given for different height frames in Fig. 4. The frames with H=2.5 m and 3.0 m are under both IBC (2012) and TEC (2007) limits. However, when we change the first story height to 5.0 m, frames without first story infill walls passes both IBC (2012) limits and frames with CDM=0.8 (less stiff) passes TEC (2007) limit.

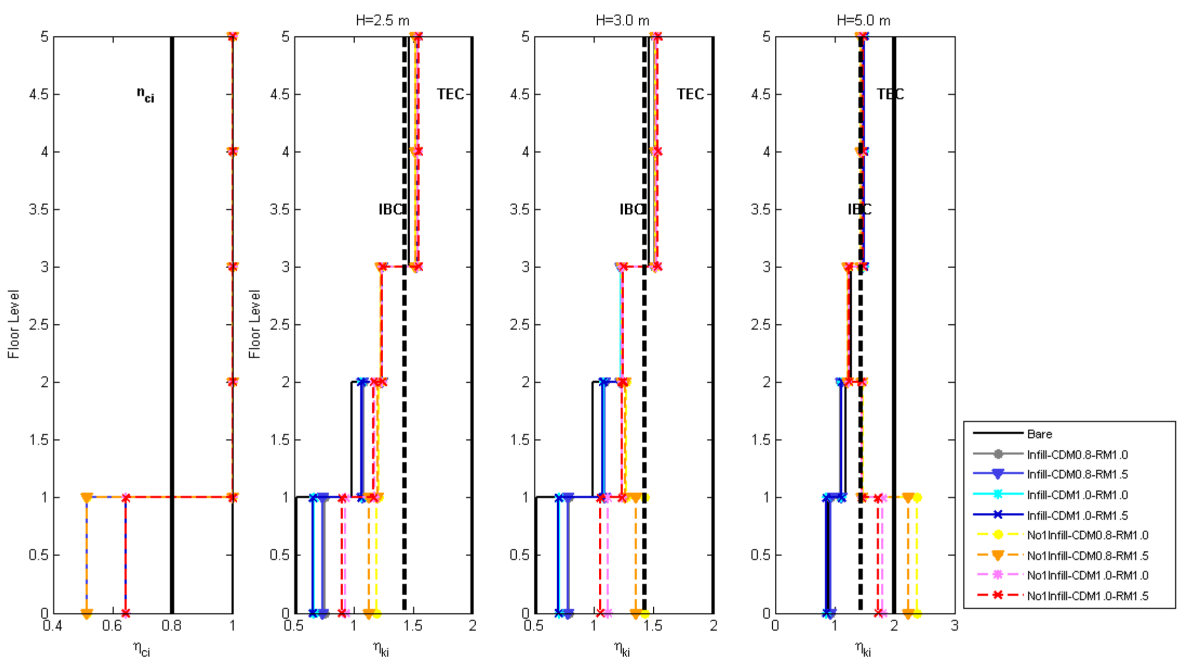


Fig. 4 – Linear Procedure Results for Frames with Different First Story Heights and with Table 3 variables.

Mod Superposition Analysis was done based on TEC (2007). CQC method was applied for the summation of the modes. In Fig. 5, the results of mod participation analysis are very close to the linear procedure. Fully infilled frames with 5.0 m height in Fig. 5 are even under the limit for IBC (2012).

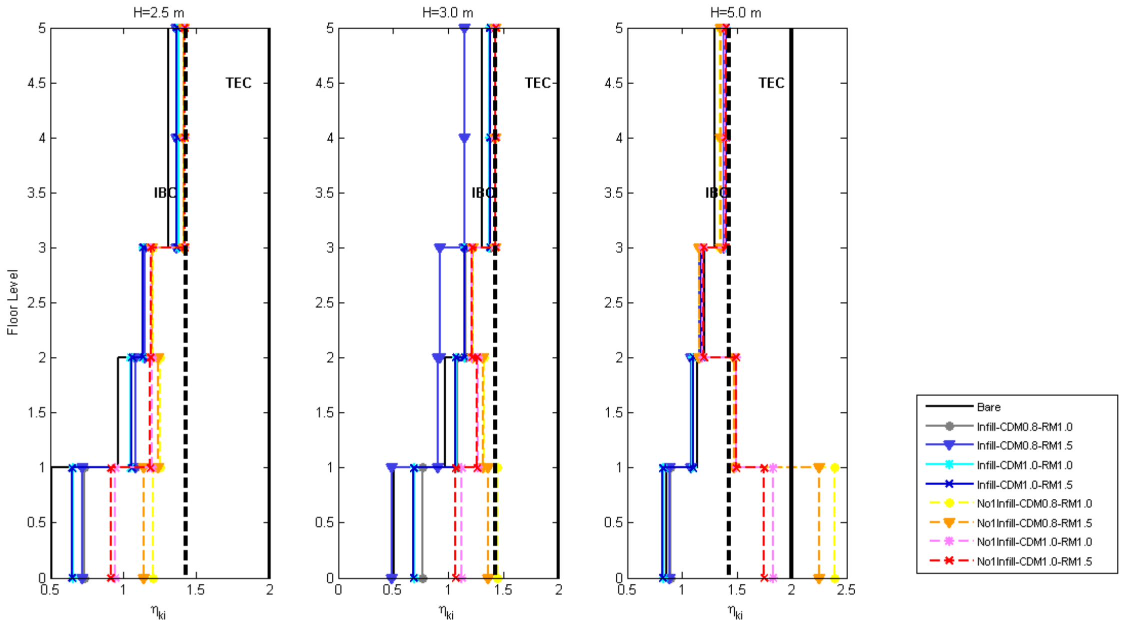


Fig. 5 – Mod Superposition Analysis Results for Frames with Different First Story Heights and with Table 3 variables.

Nonlinear Pushover Analysis is done with inverted triangular load pattern. Nonlinear pushover analysis results are plotted at 0.01 story drift ratio. This number is relevant with the linear procedure and mod superposition analysis results. Drift ratios are under 0.008 for the linear and mod superposition analysis. In Fig. 6, the frames infilled at each story are under both IBC (2012) and TEC (2007) limits. When we remove the first story infill walls from the model, the nonlinear response is approximately four times of the mod participation and linear static procedure results (Fig. 6). Higher first stories without infill walls yield to larger n_{ki} values.

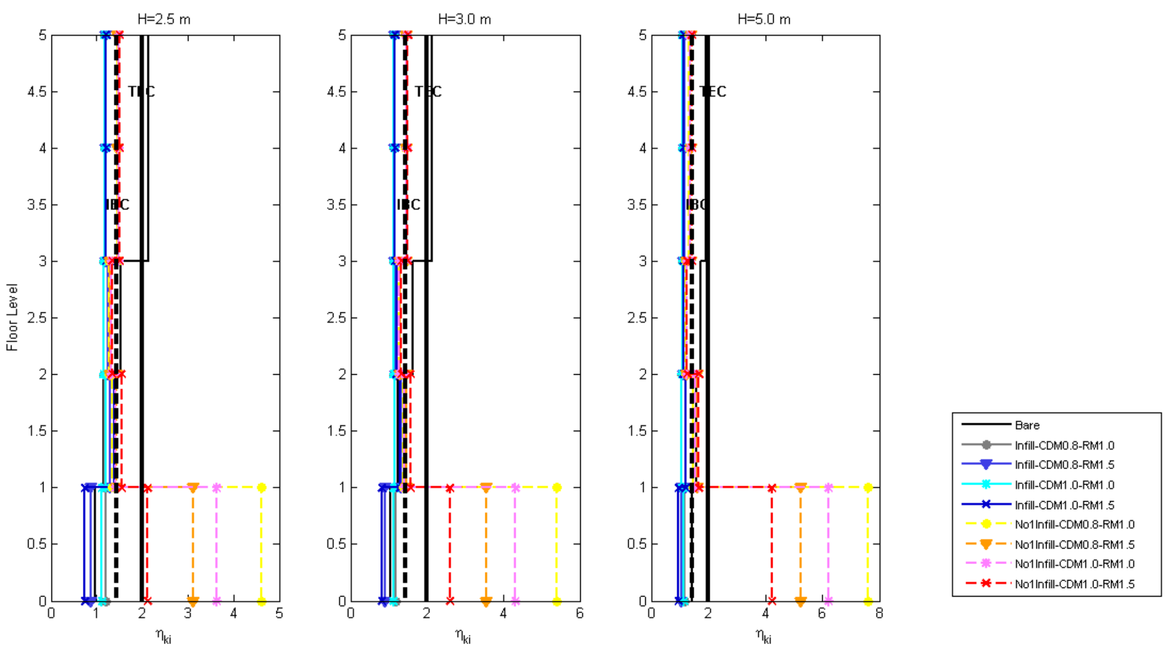


Fig. 6 – Nonlinear Pushover Analysis Results for Frames with Different First Story Heights and with Table 3 variables.

In Fig. 7, normalized pushover curves are plotted for each case. It can be observed from Fig. 7 that, the infill walls are effective at increasing the strength, however, the displacement capacity decrease drastically, especially for the fully infilled frames. First story height increase results in a decrease in the strength. The 5.0 m first story height frame without infill walls at first story has the lower strength than the bare frame. The increase in the first story height makes frames much more vulnerable to strength loss. In Table 4, strength ratios calculated from maximum normalized base shears for each frame are given. When we increase the first story height, the reinforced concrete frame becomes weaker and infill walls affect the strength highly. For fully infilled frames, an increase in height has a rising trend and the cases with no infill walls at first story, the trend in strength ratio is declining.

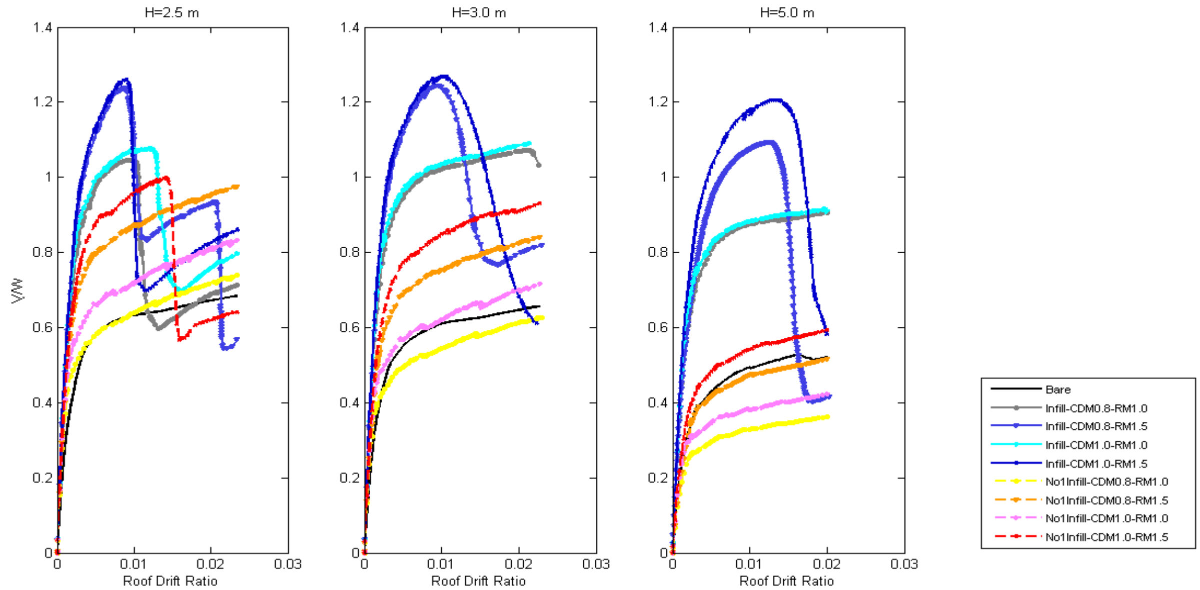


Fig. 7 – Normalized pushover curves for each case.

Table 4 – Strength ratios obtained from normalized base shears

Height/Frames	H=2.5 m	H=3.0 m	H=5.0 m
Infill-CDM0.8-RM1.0 / Bare	1.53	1.63	1.71
Infill-CDM0.8-RM1.5 / Bare	1.81	1.89	2.07
Infill-CDM1.0-RM1.0 / Bare	1.57	1.66	1.73
Infill-CDM1.0-RM1.5 / Bare	1.84	1.93	2.28
No1Infill-CDM0.8-RM1.0 / Bare	1.08	0.96	0.68
No1Infill-CDM0.8-RM1.5 / Bare	1.43	1.28	0.97
No1Infill-CDM1.0-RM1.0 / Bare	1.22	1.09	0.79
No1Infill-CDM1.0-RM1.5 / Bare	1.46	1.42	1.12

7. Comments, Conclusions, and Future Studies

The Linear procedure is a code specified method which considers fundamental period for the earthquake design loads. Mod superposition method is an extended linear procedure including higher mode effects in many codes and specification to calculate the earthquake forces for design. Linear procedure and mod superposition analysis results seem similar to each other according to obtained results. Non-linear pushover analysis is a good way to see the capacity and can give an idea about the dynamic behavior. Nonlinear pushover analysis results show that the inelastic deformations are greater than linear procedure and this means that displacements are underestimated. Infill wall arrangements in elevation seem to create soft story mechanisms and results in strength loss in the whole system. The reinforced concrete frame becomes weaker and infill walls affect the strength highly when we increase the first story height. Fully infilled frames affect the strength ratio positively even though the first story height increased. On the other hand, removal of the first story infills affect the strength ratio negatively. Frames may even become weaker than the bare correspondingly. That is the reason that vertical irregularity penalty coefficients in design should be studied in detail. Nonlinear pushover analysis results give almost the same story drift ratio values with linear and mod superposition analysis results which makes the comparison much more meaningful. For a better comparison, the failure mechanisms will be investigated as a future study such as done by Akansel and Moehle (2017). Dynamic analysis results will be done and other alternative parameters for n_{ki} will be investigated.

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