16TH EUROPEAN CONFERENCE ON EARTHQUAKE ENGINEERING THESSALONIKI 18 - 21 JUNE 2018

EFFECTS OF VARIOUS PARAMETERS ON NONLINEAR DYNAMIC RESPONSE OF INFILLED RC BUILDINGS WITH OPEN GROUND STORY

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ABSTRACT

In the last few decades, many numerical models have been suggested in the literature for the infill walls. There are also indirect provisions which consider infill walls and irregularities induced by these members in certain design codes. Yet these are limited and many seismic codes still do not consider the effects of infill walls. Therefore, a major number of existing buildings were designed and constructed without consideration of infill walls. In these cases, the significant contribution of infill walls by changing the lateral rigidity, strength and dynamic characteristics may lead to a structural system which may deviate from the originally designed one. Besides, the irregular distribution of infill walls in elevation and/or plan may cause significant damage or even total collapse under a significant seismic action. Such an irregularity may be caused by open ground stories. There is an architectural trend which uses open ground story as parking facility by forming a probable soft story irregularity, especially in seismically active coastal regions of Turkey. In this study, the nonlinear dynamic response of infilled RC buildings are investigated with a particular interest on the effect of open ground story. The number of story and seismic code compliance are considered as the other parameters of numerical study. The nonlinear time history analyses of three dimensional building models were conducted. The results have shown that vulnerability of the building to the influence of infill walls at the ground story increases with the descending number of stories and non-ductile character of the building.

Keywords: Soft story; Infill walls; Nonlinear response

1. INTRODUCTION

The possible effects of infill walls on the seismic response of multi-story reinforced concrete (RC) buildings have been the subject of many researches in the last few decades (CEB, 1996; Di Trapani et al., 2015). As a result of these studies, it is a well-known fact that the contribution of infill walls leads to an increase in the lateral load capacity and rigidity of RC frames. This alteration of behavior in the elastic stage not only change the lateral drift characteristics, but also increase the shear demand considerably by changing the dynamic properties. On the other hand, the interaction of infill and frame may result in detrimental effects, such as short column formations due to crushing of infills at the corners. The resulting accumulation of shear damage at the end regions of the columns and column-beam connections may become a serious problem, especially for frames with non-ductile design details (Akın et al.,2011).

When these infill walls are regarded as non-structural members, the design/assessment of buildings would be inaccurate. And it may also be stated that their irregular distribution either in elevation or plan may lead to catastrophic damage in a significant earthquake (Dolsek and Fajfar, 2001). There are many samples of residential RC buildings all over the world, which have infill walls to a considerably lesser extent at the entrance level. The main reasons are generally the use of entrance level for commercial services. One other cause is the use of these areas for parking facility. This have become an architectural trend especially in the coastal zones of Turkey with a temperate climate, which further includes active

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faults. The site survey and post-earthquake inspections show that a significant number of the buildings with open ground story (OGS) have non-ductile design details. Yet, a considerable number of buildings which are designed according to current seismic design code of Turkey (Specifications for Buildings to be Built in Seismic Zones-SBBSZ, 2007) and constructed with a control of building audit mechanism still have open ground stories. In this study, it was intended to study the possible effects of open ground story considering the parameters such as the design details of buildings and number of stories. The nonlinear time history and eigenvalue analyses of the RC building models were performed. The lateral drift characteristics, natural vibration period, base shear demand, ratio of the shear resisted by the infill walls and column end curvature demands are presented and discussed regarding the mentioned parameters of the study.

2. NUMERICAL MODELING

The nonlinear structural models of RC buildings were generated and time history analyses were conducted by using SeismoStruct (2016) software. The RC building models having 3, 5 and 8-story were designed as either non-ductile or ductile moment-resisting frame systems. All building models have four spans along both transverse directions with a span length of 5 m. between centerlines of columns (Figure 1). The story height is 3 m. A five story building model without infill walls (i.e. bare), having infill walls at all stories and with open ground story are illustrated in Figure 2. In case of infilled models, the infill walls are assumed to exist at all exterior spans and in only four interior spans along the x-direction (Figure 1). At the exterior spans, a modification was applied to reflect possible window openings, as it is explained in the following paragraphs. Rigid diaphragms were assigned at all story levels. The dead and live slab loads are assumed as 4.5 kN/m^2 and 2.0 kN/m^2, respectively. The unit weight of the concrete and infill units were taken as 24 kN/m^3 and 8 kN/m^3, respectively. The loads transferred by the slabs are defined as additional distributed mass on each beam element and the loads in the gravity direction are derived from the masses (i.e. by multiplying with gravitational acceleration, g). Both longitudinal and transverse reinforcement was deemed to have a characteristic tensile strength of 420 MPa in all models.

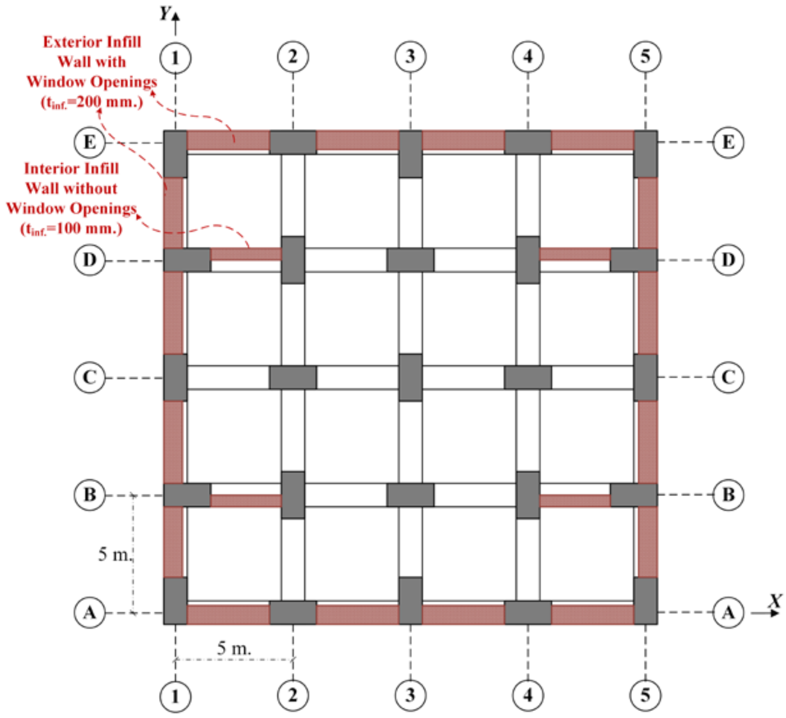

Figure 1. Plan view of the model building with infill walls

The non-ductile frames were designed considering only gravity forces and seismic design rules, such as capacity design was not applied. These models were aimed to represent the design and detailing characteristics of buildings which were constructed without a sufficient control mechanism before 1998-Seismic Design Code of Turkey (Specifications for Buildings to be Built in Disaster Areas-SBBDA, 1998). The characteristic concrete compressive strength was assumed as 16 MPa for the non-ductile building models. While considering the confinement of non-ductile members, the free ends of stirrups were assumed as anchored only to the cover concrete with 90º hooks. On the other hand the ductile frames were designed according to the current seismic design code of Turkey (SBBSZ, 2007) by considering seismic design principles (i.e. strong column-weak beam, flexure dominant behavior, etc.) and assuming high ductility level. The design was conducted with regard to the highest seismic zone in SBBSZ (2007). The concrete compressive strength of ductile buildings was taken as 30 MPa. The resulting cross-sectional dimensions and reinforcement details of all RC members in each group are provided in Table 1.

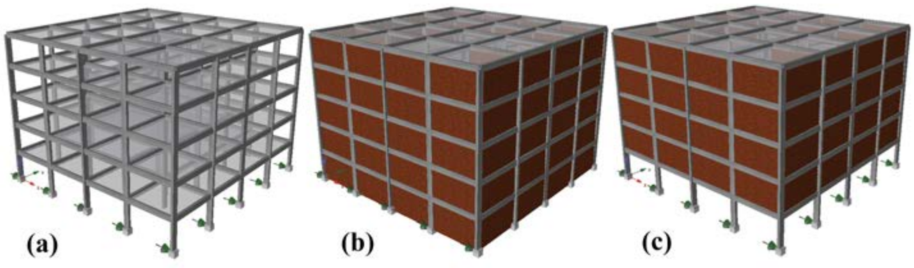

Figure 2. 5-story model buildings, (a) without infill walls (bare), (b) with infill walls at all stories, (c) with infill walls-open ground story

Table 1. The cross-sectional dimensions and longitudinal/lateral reinforcement details of RC members

Group	Beams				Columns		
	Width/ Height	Longitudinal Reinf. Span	Longitudinal Reinf. Support	Lateral Reinf.	Width/ Height	Long. Reinf.	Lateral Reinf.
3-Story Non-Ductile	250mm/ 500mm	2ϕ14 Top 3ϕ14 Bott.	4ϕ14 Top 3ϕ14 Bott.	ϕ8/200 mm.	250mm/ 350mm	3ϕ14 Top 3ϕ14 Bott.	ϕ8/200 mm.
3-Story Ductile	250mm/ 500mm	2ϕ16 Top 3ϕ16 Bott.	4ϕ16 Top 3ϕ16 Bott.	ϕ10/80 mm.	250mm/ 500mm	3ϕ16 Top 2ϕ16 Mid. 3ϕ16 Bott.	ϕ10/80 mm.
5-Story Non-Ductile	250mm/ 500mm	2ϕ14 Top 3ϕ14 Bott.	4ϕ14 Top 3ϕ14 Bott.	ϕ8/200 mm.	250mm/ 400mm	3ϕ14 Top 3ϕ14 Bott.	ϕ8/200 mm.
5-Story Ductile	250mm/ 500mm	2ϕ18 Top 3ϕ18 Bott.	4ϕ18 Top 3ϕ18 Bott.	ϕ10/80 mm.	250mm/ 550mm	3ϕ18 Top 2ϕ18 Mid. 3ϕ18 Bott.	ϕ10/80 mm.
8-Story Non-Ductile	250mm/ 600mm	2ϕ20 Top 3ϕ20 Bott.	4ϕ20 Top 3ϕ20 Bott.	ϕ8/200 mm.	300mm/ 500mm	3ϕ18 Top 3ϕ18 Bott.	ϕ8/200 mm.
8-Story Ductile	250mm/ 500mm	2ϕ20 Top 3ϕ20 Bott.	4ϕ20 Top 3ϕ20 Bott.	ϕ10/80 mm.	250mm/ 650mm	3ϕ22 Top 2ϕ22 Mid. 3ϕ22 Bott.	ϕ10/80 mm.

2.1 Modeling of RC Members

A non-linear concrete model was utilized where the confining pressure is assumed to be constant at all stages of behavior (Madas, 1993; Mander et al., 1988). The tensile capacity of concrete was neglected in the model. The modulus of elasticity values for the concrete used in non-ductile and ductile buildings were 28000 and 32000 MPa, respectively. The strain corresponding to the ultimate compressive strength of concrete was assumed as 0.002. A uniaxial steel model with a strain hardening ratio of 0.005 was utilized for both longitudinal and transverse reinforcements. "Inelastic force-based frame element" of SeismoStruct software, where the section is discretized into 50 fibres at four integration sections, was used for the non-linear modeling of beams and columns. The first and last integration sections of the beams were defined with the support reinforcements provided in Table 1. On the other hand, the second and third integration sections were defined with the span reinforcements.

2.2 Modeling of Infill Walls

The four-node infill panel element of SeismoStruct was utilized for the non-linear modeling of infill walls (Crisafulli, 1997). Along each diagonal direction, the panel element has two parallel strut members which carry axial loads and one shear spring (i.e. shear strut) which is only active in compression. The mechanical properties of the average quality (i.e. fair) infill wall that is suggested by FEMA 356 (2000) was employed in the model. The corresponding compressive strength (f_m'), modulus of elasticity (E_m) and shear strength of masonry infill walls are 4.1 MPa, 2255 MPa and 0.14 MPa, respectively. The model accounts for the diagonal capacity of the infill panel associated with four possible failure mechanisms as defined by Bertoldi et al. (1993). These four failure modes and related diagonal capacity of the infill panel are summarized in Table 2. The minimum capacity value for any failure mode is defined as the axial capacity of the struts.

Table 2. Diagonal capacity of infill panels

Failure Mode	**Diagonal Capacity, $f_{m\theta}$ (MPa)**
Diagonal Tension	$f_{m\theta} = \frac{0.6\ f_{ws}+0.3\sigma_v}{b_w/d_w}$
Sliding Shear	$f_{m\theta} = \frac{(1.2sin\theta+0.45cos\theta)f_{wu}+0.3\sigma_v}{b_w/d_w}$
Compression of corners	$f_{m\theta} = \frac{1.12f_w{\prime}\times sin\theta\times cos\theta}{K_1\times(\lambda h)^{-0.12}+K_2\times(\lambda h)^{0.88}}$
Compression at center of panel	$f_{m\theta} = \frac{1.16f_w{\prime}\times tan\theta}{K_1+K_2\times\lambda h}$

In these equations, f_{ws} and f_{wu} are the shear resistance under diagonal compression and sliding resistance of the mortar joints, respectively. These values were assumed to be equal to the shear strength of infill wall in the absence of experimental data. f_w' is the fundamental compression resistance which is taken as the compressive strength of infill (i.e. f_m'=4.1 MPa). σ_v is the vertical compressive stress due to gravity loads, which was ignored in the model. λ is defined by the following equation (FEMA 356, 2000).

$$\lambda = \sqrt[4]{\frac{E_m\times t_{inf.}\times sin2\theta}{4\times E_c\times I_c\times h_{inf.}}} \quad (1)$$

In Equation 1, $t_{inf.}$ is the infill wall thickness (200 mm. and 100 mm. for the exterior and interior infill walls, respectively) and $h_{inf.}$ is the height of infill wall. E_c and I_c are the modulus of elasticity of concrete and moment of inertia of the column about the bending direction, respectively. θ is the inclination of the diagonal of infill panel.

The strut width (b_w) which designates stiffness of the infill panel may be calculated by using the following expression.

$$b_w = 0.175(\lambda h)^{-0.4} \times d_w \quad (2)$$

where $d_{w.}$ is the diagonal infill length and h is the interstory height.

K_1 and K_2 are expressed in relation with "λh" as follows:

If $\lambda h<3.14$, then $K_1=1.3$ and $K_2=-0.178$
If $3.14<\lambda h<7.85$, then $K_1=0.707$ and $K_2=0.01$
If $\lambda h>7.85$, then $K_1=0.47$ and $K_2=0.04$

The strut area is calculated as the multiplication of the strut width (b_w) and infill wall thickness ($t_{inf.}$). The tensile strength of the infill panel was ignored in the model. The strain corresponding to the maximum stress and ultimate strain were assumed as 0.001 and 0.010, respectively. The bond strength (τ_o) for the shear spring model was defined to be the shear strength of the infill walls (i.e. 0.14 MPa). It should be noted that this value is within the range suggested for the shear bond strength in the literature (Paulay and Priestly, 1992). The friction coefficient (μ) was assumed to be 0.70 as suggested by Atkinson et al. (1989). The maximum shear resistance (τ_{max}) is calculated by using the following expression as provided in the manual of SeismoStruct.

$$\tau_{max} = \tau_o + 0.30 = 0.14 + 0.30 = 0.44\ MPa \tag{3}$$

It is reasonable to assume that there may be window openings on the infill walls which are at the outer circumference of the building. In order to account these window openings in the infill wall model, a simplified approach that is suggested by Decanini et al. (2014) was utilized. In this approach, the strength and stiffness of the infill strut is decreased to a certain percentage which is determined as a function of the opening dimensions and presence of reinforcing elements around the openings. In Turkish construction practice, generally only lintels which reinforce upper surface of the opening are provided. Therefore, "partially reinforced" case was considered in this approach. A window opening which has dimensions (i.e. length×height) of 1.5 m.×1.3 m. was presumed. The same reduction factor is suggested by Decanini et al. (2014) for both strength and stiffness reduction as follows.

$$\rho = 0.55\exp(-0.035\alpha_a) + 0.44\exp(-0.025\alpha_l) \mp 0.284\varepsilon \tag{4}$$

$$\alpha_a = \frac{l_o \times h_o}{L \times H}\ \ ;\ \ \alpha_l = \frac{l_o}{L} \tag{5}$$

In order to obtain more conservative results, ε was taken to be "1" by considering "mean+one standard deviation" for the reduction factor. l_o and h_o are the length and height of the window (i.e. l_o=1.5 m., h_o=1.3 m.), and L and H are the length and height of infill wall, respectively. The resulting reduction factor was calculated to be approximately 0.70.

2.3 Nonlinear Time History Analysis

The nonlinear analyses of the building models were performed using acceleration time history corresponding to Kocaeli-Yarımca (N-S component) record that had been obtained during 1999 Marmara Earthquake. The accelerogram used in the analyses is shown in Figure 3. The analyses were conducted only along the x-direction.

3. NUMERICAL RESULTS AND DISCUSSION

The results of analyses are summarized in terms of base shear demand, lateral drift characteristics, natural vibration period, distribution of shear demand between RC frame and infill walls and ground story column curvature demands. These results are discussed with regard to the mentioned parameters of the study. The building models are entitled for a more convenient discussion. The first letter(s) designate the type of design (ND for non-ductile, D for ductile design). Afterwards, the number of stories is mentioned (3, 5 or 8 story buildings). Lastly, the infill wall content in the model is represented (BF for bare frame with no infill walls, FI for fully infilled at all stories and OGS for open ground story with the absence of infill walls only at the ground story).

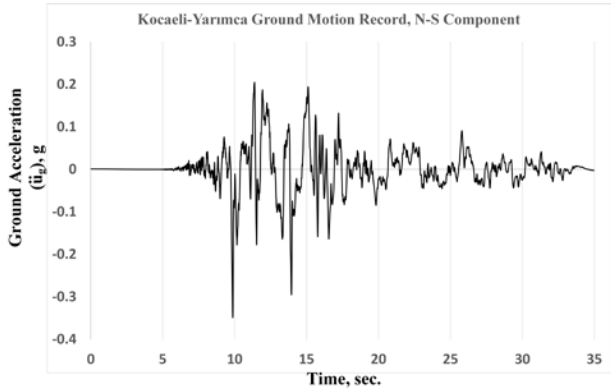

Figure 3. Horizontal ground acceleration vs. time recorded at Kocaeli-Yarımca during 1999 Marmara Earthquake

As long as the stability of solution is achieved, SeismoStruct software does not stop the time history analyses. Consequently, the behavior of structure in the highly inelastic range may be monitored. However, there is a risk that even the collapse state due to hinge formations at both ends of all ground story columns may be overlooked, unless the stability of the solution is violated. Therefore, a performance criteria should be defined for the analysis, in order to provide a meaningful comparison between the results of different models. This criteria was selected to be "0.01" for the reinforcement strain of all ground story columns. This criteria level was reached at about 12th sec. of the loading history in most of the non-ductile building models. The only two exceptions were ND-3-BF and ND-3-FI, where the entire record could be applied, similar to ductile building models.

3.1 Base Shear Demand

The base shear demand results of the building models are summarized in Table 3. The ratio of base shear demand values with respect to that of bare frame building, individually in each group are also provided in the same table.

Table 3. The base shear demand and natural vibration periods of the building models

Group	**Building Model**	**Base Shear, V_b (kN)**	**$V_b/V_{b,BF}$**	**Natural Vibration Period, Tn (sec.)**	**$(T_n/T_{n,BF})$*100 (%)**
3-Story Non-Ductile	ND-3-BF	1080.9	-	0.42	-
	ND-3-FI	2096.7	1.94	0.22	52.5
	ND-3-OGS	1143.2	1.06	0.35	82.5
3-Story Ductile	D-3-BF	1735.3	-	0.31	-
	D-3-FI	2462.0	1.42	0.19	60.5
	D-3-OGS	2164.8	1.25	0.25	79.6
5-Story Non-Ductile	ND-5-BF	1368.8	-	0.63	-
	ND-5-FI	3059.9	2.23	0.36	56.3
	ND-5-OGS	1421.4	1.04	0.46	72.1
5-Story Ductile	D-5-BF	2627.5	-	0.49	-
	D-5-FI	3418.1	1.30	0.30	60.9
	D-5-OGS	3114.5	1.19	0.35	71.8
8-Story Non-Ductile	ND-8-BF	2760.6	-	0.80	-
	ND-8-FI	4070.7	1.47	0.49	61.8
	ND-8-OGS	2888.6	1.05	0.63	79.7
8-Story Ductile	D-8-BF	4184.6	-	0.71	-
	D-8-FI	4596.5	1.10	0.43	60.7
	D-8-OGS	4276.1	1.02	0.47	65.8

The addition of infill walls at all stories (i.e. FI with respect to BF) result in a significant increase in the base shear demand by increasing the lateral rigidity of all the buildings analyzed in this study. However, these results indicate that this increase in the base shear due to infill walls is much more significant for the non-ductile buildings in comparison to their ductile counterparts (Table 3). In case of ductile buildings, the base shear increment due to infills tend to decrease as the number of stories rise and become almost inconsiderable for the model D-8-FI.

The base shear demand of the non-ductile OGS buildings is almost not altered in comparison to the bare frames in the same group. On the other hand, the base shear of the ductile OGS buildings are slightly higher when compared with the bare frames in each group, which again tend to diminish as the number of stories increase. These results may be related with the well-known fact that the highest amount of seismic shear action takes place at the ground story in ordinary moment-resisting frame type buildings. It seems to be that the effect of ground story infills is much more intensive in case of buildings which are not designed according to seismic design principles. The same conclusion may also be stated for the number of stories. The absence/presence of infill walls at the ground story become more significant in terms of base shear demand, as the number of stories decrease.

3.2 Lateral Drift Characteristics

The ultimate lateral drift ratio (DR) profiles of the buildings are shown in Figure 4. It should be noted that the values which can be experienced by the ND-3-OGS and all other 5 and 8-story non-ductile models correspond to an earlier stage of time history. In order to better comprehend the results, the relative drift ratios between neighboring stories (i.e. lower to upper story DR) are also presented in Figure 5.

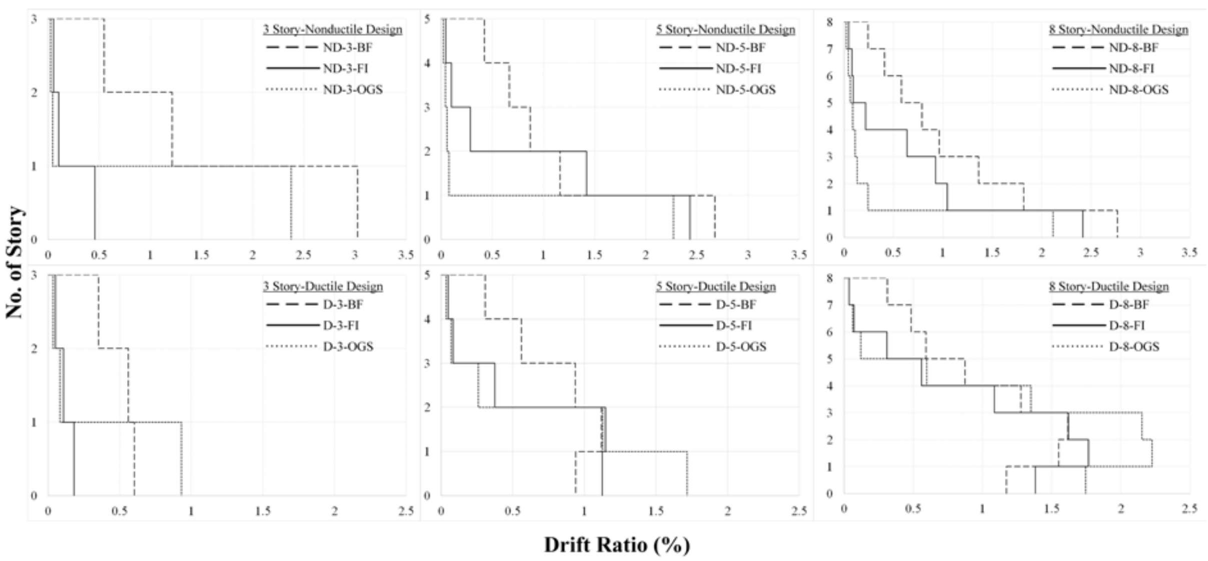

Figure 4. Lateral drift profile of the building models

In both non-ductile and ductile 3-story buildings, the presence of infill walls at the ground story result in a considerable amount of DR reduction at this story level when compared to bare frame models (Figure 4). However, this was more prominent for the non-ductile model (i.e. ND-3-FI). It is notable that this reduction in DR takes place despite a distinct increase in shear demand for fully infilled models compared to bare structures. A similar result is also observed in 5 and 8-story non-ductile models, although it is not in the same scale. On the other hand, higher DR values are shifted to the upper stories in 5 and 8-story ductile buildings. Therefore, the reduced DR values due to fully infilled frame occur at the upper levels of these buildings.

The accumulation of lateral drift at the ground story due to absence of infills at this level (i.e. OGS) becomes more significant as the number of story decreases and especially if the structure has a non-ductile design (Figure 5). Due to alteration of the drift profile in 5 and 8-story ductile buildings, such an increment of DR may be observed at the upper stories.

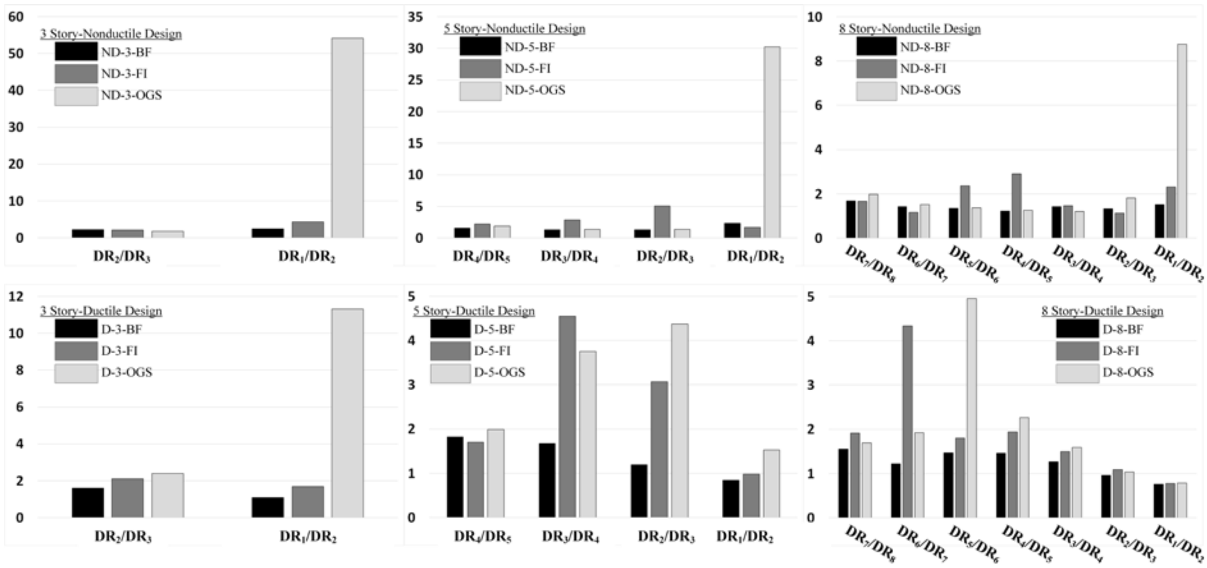

Figure 5. Relative drift ratios (DR) between neighboring stories

3.3 Natural Vibration Period

An eigenvalue analysis was conducted for the building models and corresponding natural vibration periods are summarized in Table 3. The change of natural period in FI and OGS buildings with respect to BF models in each group is also presented in this table.

As one may expect, the addition of infill walls at all stories (i.e. FI) decreased the natural period of the structure considerably by leading to higher lateral rigidity in comparison to BF models. This seems to be more sensitive to the number of stories in case of non-ductile building models. However, it is almost constant (approximately 60 percent) for ductile building models.

The lack of infill walls only at the ground story also yielded smaller natural period compared to BF models, though to a lesser extent. Among all OGS models, the reduction in the period has the least insignificant ratio in case of ND-3-OGS due to the same reason explained at the end of Section 3.1. The highest influence of infill walls on the lateral response of non-ductile low-rise buildings take place when these infills exist at the ground story. Therefore, the absence of ground story infills in these type of buildings may yield dynamic properties closer to a BF.

3.4 Distribution of Shear Demand

The distribution of shear demands at the ground story between the infill walls and RC columns is presented in Table 4 for the FI models. It is clear that a considerable amount of the shear force is carried by the infill walls at the instant of highest base shear demand. This relative shear force, $V_{b,inf}/V_b$, tend to decrease as the number of stories increase or as character of the building becomes more ductile. But even in D-8-FI, almost 42 percent of the shear force is carried by the infill walls at the peak demand.

However, this beneficial contribution of infill walls to the shear demand decreases as the infill strut models reach their ultimate strain value (i.e. 0.01). Consequently, the shear force resisted by the columns increases stepwise. This is demonstrated in Figure 6 for the ground story of ND-5-FI and D-5-FI. At the instant of peak base shear demand, the shear forces resisted by the ground story columns are only 34 and 49 percent of total shear for ND-5-FI (at 9.962 sec.) and D-5-FI (at 10.336 sec.), respectively.

However, the shear force resisted by the ground story columns attain their maximum value at a later stage. At this instant, the shear demand on the ground story columns becomes 98 and 94 percent of the total shear for ND-5-FI (at 12.41 sec.) and D-5-FI (at 15.198 sec.), respectively. This further indicates that the complication which is caused by the failure of infill walls is more significant for non-ductile buildings.

Table 4. The distribution of shear demand between columns and infill walls at the ground story

Building Model	**Total Shear Demand, V_b (kN)**	**Shear Demand on Columns, $V_{b,col.}$ (kN)**	**Shear Demand on Infill Walls, $V_{b,inf.}$ (kN)**	**$(V_{b,inf}/V_b)*100$ (%)**
ND-3-FI	2096.7	488.2	1608.5	76.7
D-3-FI	2462.0	807.0	1655.0	67.2
ND-5-FI	3059.9	1050.8	2009.1	65.7
D-5-FI	3418.1	1666.0	1752.1	51.3
ND-8-FI	4070.7	2047.4	2023.3	49.7
D-8-FI	4596.5	2675.7	1920.8	41.8

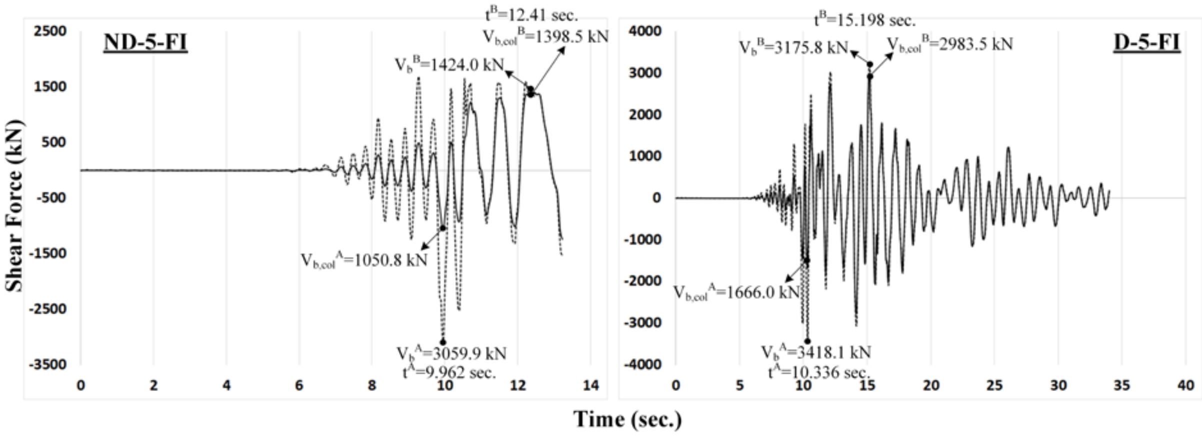

Figure 6. The shear force demand history for ND-5-FI and D-5-FI

3.5 Column Curvatures

The maximum curvature values at the bottom integration section of column C102 (at the ground story) and C202 (at the 1st story) are provided in Figure 7. In fully infilled models, the significant increase in the base shear demand also increases the flexural deformations of ground story columns. As explained in the previous part, these columns are also forced to undertake additional forces after failure of infill walls at this story. Therefore, the curvature demand of ground story columns are increased in case of fully infilled frames in comparison to bare frames. The only exception was the 3-story models, where the infill walls are effective in limiting the lateral drift even under increased shear demands. Accordingly, in 3-story fully infilled models, most particularly in ND-3-FI, the curvature demand of ground story columns are reduced compared to bare models.

The curvature demand on the ground story columns of non-ductile OGS models are lower than the bare frames since these models attain the performance criteria (i.e. failure level) at an earlier stage of time history. Besides, in all models but especially in non-ductile frames, the curvature of upper story columns are extremely small when compared with ground story columns. One may conclude that the non-ductile OGS buildings reach to a significant damage state before utilizing the flexural deformation capacity of all ground story columns. The accumulation of seismic action at the OGS may be given as the main reason of this result. It should also be added that this accumulation becomes more crucial as the number of stories decrease.

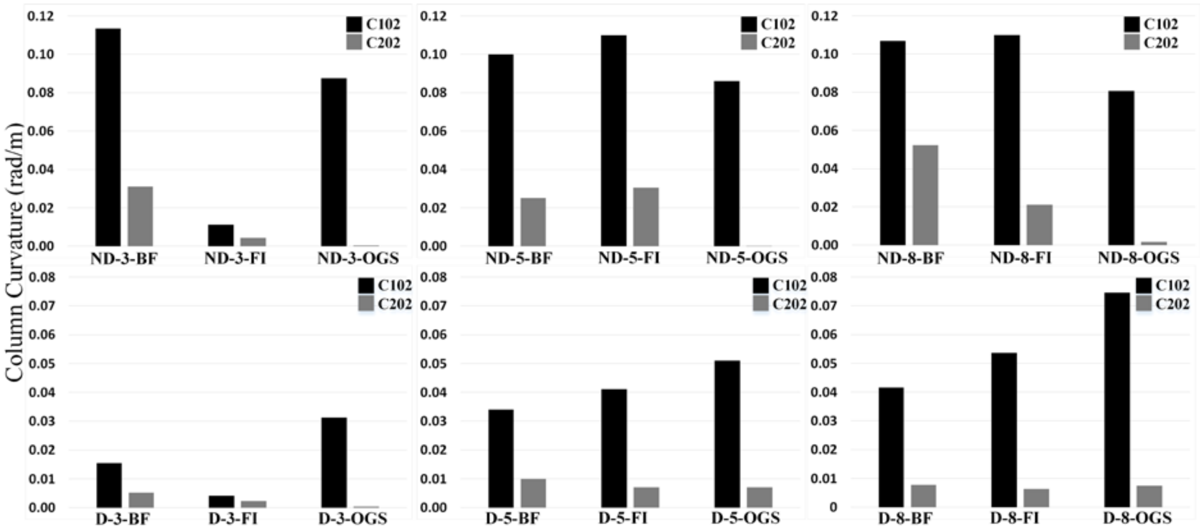

Figure 7. The maximum curvature at the bottom section of columns C102 and C202

The curvature demand of ground story columns is higher in all ductile OGS buildings compared to their bare and fully infilled counterparts. However, it should be noted that even the curvature demand of ductile OGS building columns are lower than the corresponding curvature capacities. This was demonstrated by Figure 8, where the moment-curvature diagram of the 5-story non-ductile and ductile column sections are provided. The maximum curvature values of column C102 determined from the analysis for the 5-story FI and OGS models are indicated on the figure. The normal forces that are utilized while obtaining moment-curvature diagrams are also shown in Figure 8, which are the highest axial load values for column C102 in the time history analysis results.

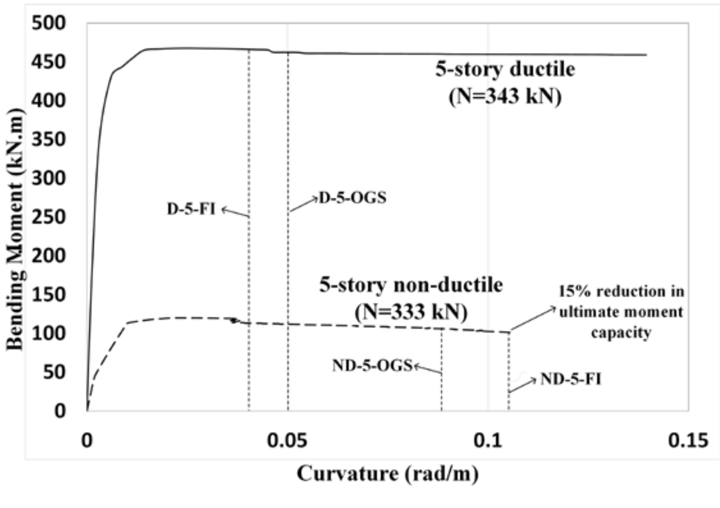

Figure 8. Moment-curvature diagram of 5-story non-ductile and ductile columns

4. CONCLUSIONS

The results of a numerical study which investigates the seismic response of OGS buildings by considering the design characteristics and number of stories is summarized here. These conclusions should not be generalized without a thorough discussion of the results.

According to the outcomes of the study, the natural vibration period is decreased and base shear demand is increased due to higher lateral rigidity and additional shear resistance provided by the infill walls, particularly those at the ground story. The intensity of this effect ascend as the number of stories decrease and/or character of building become less ductile (i.e. due to improper seismic design). Under the increased base shear demand, the lateral drifts could be limited by the infill walls only in non-ductile FI models and 3-story ductile FI building. In 5 and 8-story ductile building models, addition of infill walls altered the drift profile and forms highest drift ratio values at the upper stories in both FI and OGS cases. However, the story level where this highest drift ratio takes place seem to depend on the existence of infill walls at the ground story and number of stories. The additional shear forces undertaken by the ground story infill walls tend to decrease after the peak acceleration is attained and infill walls reached the ultimate strain. Furthermore, this causes amplification of shear demand for the ground story columns. The flexural deformations of ground story columns are also increased in case of FI models in comparison to BF models, due to increased shear demand.

When the infill walls at the ground story are removed from the models (OGS), the response of the buildings becomes closer to a bare frame response in terms of base shear demand and natural vibration period. This was especially the case in non-ductile, low-story model above all. Yet, the rigidity and additional mass provided by the upper story infill walls cause an alteration of dynamic properties, particularly in ductile buildings. On the other hand, the absence of infill walls at the ground story causes a considerable accumulation of lateral drift and related flexural deformations at this story, even under a shear demand close to bare models. Again this seems to become more intense as the number of stories decrease and seismic design rules are disregarded in design. The altered curvature demands in both OGS and FI buildings could generally be resisted and sustained by the ductile models. However, the ground story columns are pushed into high damage state and a possible failure mechanism due to these increased curvature demands.

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