INFLUENCE OF INFILLS ON THE SEISMIC BEHAVIOUR OF A SIX STOREY RC FRAME AT DIFFERENT LEVELS OF DEMAND

Maria FAVVATA

ABSTRACT

The effect of infills on the global and local performances of a 6-storey reinforced concrete (RC) frame structure is investigated at two different seismic demand levels, using dynamic step by step analyses procedures. Three different seismic excitations extracted from PEER's database are considered and are scaled to fit the Eurocode 8 (EC8) elastic spectrum for low zone of seismic hazard and ground type B. The examined seismic demands are the Significant Damage and the Near Collapse limit state (EC8-part3). Two types of infilled RC frames are studied: (a) fully infilled and (b) pilotis type frame (without infills at the base floor). The case of bare frame is also included for comparison reasons. Results in terms of failure modes, interstory drifts, top displacements, base shear requirements, ductility requirements and infills local inelastic responses are presented. In this study, the influence of infills on the seismic response of the 6-storey frame is beneficial at the limit states of significant damage and near collapse, as long as infills have not exceeded their maximum capacity strength. Nevertheless, at the near collapse level, the presence of infills is not enough to eliminate the maximum inelastic requirements for plastic rotation of the columns at the base of pilotis frame. Furthermore, at both levels of seismic demand, the base shear requirements of the pilotis type frame are greater than the corresponding demands of the other two frames. Thus, in the case of pilotis additional measurements have to be taken in order to ensure columns from shear failure.

Keywords: limit states; masonry infills; pilotis; seismic requirements; reinforced concrete frame; nonlinear time history analyses

1. INTRODUCTION

The most common type of existing reinforced concrete (RC) structural system is the masonry infilled frame. However, it is well known that the interaction between infills and bare frame can lead to unexpected effects on the seismic response of the structure such as soft-story mechanisms, shear failure in columns and damages inside the RC joint region (Karayannis et al. 2011). Nevertheless, analyzing the results of past earthquakes it has been observed that several regular infilled frame structures with symmetrical distribution of infill in both elevation and plan performed reasonably well, preventing the collapse of several buildings (1985 Mexico City earthquake). Moreover, a completely infilled low rise RC frame at Bhuj with uniform strength and stiffness properties sustained nearly uniform damage along its height (Murty et al. 2002). On the contrary, the absence of infills at the base floor of the structures has been responsible for severe damages and even collapses with a common type of structural failure to be that of soft/or weak story. For example, Kirac et al. (2011) reported that 59.67% of the damaged buildings at Kocaeli-Izmit earthquake (1999 Turkey) were due to base soft-story. Also, during 2001 Bhuj (India) earthquake most of the 130 buildings that collapsed in Ahmedabad were of open ground story configuration while among those that did not collapse the damage was concentrated in the columns of the open ground story (Murty et al. 2002). Although this behaviour is considered as typical for the pilotis type RC frames, soft story mechanisms have been observed in vertically regular infilled structures as well (e.g. Dolšek and Fajfar 2008, Dolšek and Fajfar 2001, Favvata et al. 2012). Of course, over the last five decades many analytical works have been performed for the study of infills effect on the seismic response of the RC structures, while for the evaluation of the overall seismic performance of these
structures, nonlinear static analysis procedures are usually applied (Capacity Spectrum Method, Coefficient Method, N2 Method). Different limit states of the seismic codes have also been incorporated in the assessment of infills effect on the overall performances of the RC frame structures (Dolšek and Fajfar 2008, Favvata et al. 2013, Celarec et al. 2012, Repapis et al. 2006, e.t.c.). The aim of this study is to investigate the effect of infills on the global and local requirements of RC frames at different seismic demand levels, using dynamic step by step analyses procedures. The dynamic step by step analyses have been performed using seismic excitations that have been properly scaled to fit the EC8 elastic spectrum at both seismic demands. Three types of RC frames are studied: (a) fully infilled, (b) pilotis type frame and (c) bare frame for comparison reasons. For the needs of this investigation a 6-storey RC frame is considered and special purpose inelastic elements-models are adopted for the simulation of the columns, beams and infills. The analyses are performed using the program Drain-2Dx. Results in terms of failure modes, interstory drifts, top displacements, base shear requirements, ductility requirements and infills local inelastic response are presented. The maximum plastic rotation demands of beams and columns are estimated and the attained limit state is identified.

2. DESIGN OF THE SIX STOREY RC FRAME

The examined RC structure is a 6-storey frame building structure designed according to the Greek codes that are very close to Eurocodes 2 & 8. The mass of the structure is taken equal to \( M = (G + 0.3Q) \) (where, \( G \) gravity loads and \( Q \) live loads). The design base shear force of the examined 6-story structure was equal to \( V = (0.3g/q)M = 594.69\text{kN} \) where, \( q \) is the behaviour factor of the structure equal to 3.5. Reduced values of member moments of inertia \( I_{ef} \) were considered in the design to account for the cracking; for beams \( I_{ef} = 0.5I_g \) and for the columns \( I_{ef} = 0.9I_g \) (where \( I_g \) the moment of inertia of the gross section). Critical for the dimensioning of the columns proved to be in most of the cases the code provision regarding the axial load ratio limitation \( v_c \leq 0.65 \) and in a few cases the code requirements for minimum dimensions. Structural geometry and reinforcement of the columns of the 6-storey frame are shown in Figure 1. It is noted that for the examined RC frame structure a strict code design procedure is followed. For this reason, as it can be observed in Figure 1 some columns have different requirements for reinforcement in the top section compared to the ones of the bottom section.

![Figure 1: Structural system and column reinforcements of the 6-storey RC frame.](image-url)
3. STRUCTURAL MODELLING AND SEISMIC DEMANDS

3.1 Simulation of beams and columns

The structural system consists of beams and columns. The structure is modeled as a 2D assemblage of non-linear elements connected at nodes. The mass is lumped at the nodes and each node has three degrees of freedom. The finite element mesh utilizes an one-dimensional element for each structural member. Two types of one-dimensional beam-column elements were used. The first one is the common lumped plasticity beam-column element and it was used for the modelling of the beams. With this element-model the inelastic behaviour is concentrated in zero-length “plastic hinges” at the element’s ends. For the modelling of the columns a different type of element is adopted. That was the “distributed plasticity” special purpose element. This type of element is accounting for the spread of inelastic behaviour both over the cross-sections and along the deformable region of the member length. Moreover, this element performs numerical integration of the virtual work along the length of the member using data deduced from cross-section analysis at pre-selected locations. Thus, the deformable part of the element is divided into a number of segments and the behaviour of each segment is monitored at the centre cross-section (control section) of it. The cross-section analysis that is performed at the control sections is based on the fibre model. This fibre model accounts rationally for axial – moment (P-M) interaction.

3.2 Simulation of infills

For the simulation of the local response of the masonry infill panel the equivalent diagonal strut model is used. A special purpose element is used for the modelling of the infills (Karayannis et al. 2005). This element accounts for more accurate definition of the response properties of infilled masonry since it includes degrading branch (Figure 2). Special attention has been given to the implementation of this element for the simulation of the infill panel in order to exhibit axial response only and not flexural one. An important issue in modelling the infill panel is the determination of the response characteristics of the diagonal strut model, taking into account the actual conditions of the effective lateral confinement of the masonry by the reinforced concrete frame. The actual properties of the infill panel have been approached using the experimental results by Karayannis et al. (2005). The mechanical properties of the infill panel are presented in Figure 2.

3.3 Examined levels of seismic demand

Two seismic demands according to the EC8-part3 (2004) are taken into account: (a) demand for Significant Damage (SD) limit state that corresponds to ground motions with return periods of 475 years and (b) demand for Near Collapse (NC) limit state that corresponds to ground motions with return periods of 2475 years. For this purpose, three different seismic excitations extracted from the PEER’s database are considered. The selected ground motions are scaled to fit the Eurocode 8 (EC8) elastic spectrum for low zone of seismic hazard and ground type B. The characteristics of the seismic excitations are presented in Table 1.

4. RESULTS

The influence of the infills on the maximum interstory drifts of the 6-storey frame at the significant damage and at the near collapse seismic demand levels is presented in Figures 3a and 3b, respectively. In these figures, at each floor level the depicted value of interstory drift is the maximum of the three corresponding requirements as resulted from the seismic analyses. It can be observed that at the seismic demand of significant damage (SD) the global interstory drifts of the infilled frames are less than the corresponding values of the bare frame at all the story levels (Figure 3a). The small differences between drifts of the infilled frames indicate similar damage distribution, with an exception the first floor level where the corresponding requirements are a bit greater in the pilotis type frame.
a. Characteristics of the equivalent diagonal strut

\[
w = 0.175 \left( \lambda_1 \cdot h_{inf} \right)^{0.4} f_{m}^{'\prime} \\
\lambda_1 = \frac{E_m \cdot h_{ref}}{4t_{inf}^2}\sin 2\theta
\]

\(E_m\): modulus of elasticity of masonry
\(EI\): flexural stiffness of columns

**Thickness of masonry infill**

- \(t_{inf} = 10\) cm

**Floor level**

| Floor level | ref (m) | w (mm) | maximum axial compressive strength
<table>
<thead>
<tr>
<th></th>
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</tr>
</thead>
<tbody>
<tr>
<td>1st</td>
<td>6.128</td>
<td>731</td>
<td>369.15</td>
</tr>
<tr>
<td>2nd</td>
<td>6.019</td>
<td>689</td>
<td>348.03</td>
</tr>
<tr>
<td>3rd</td>
<td>6.087</td>
<td>655</td>
<td>330.77</td>
</tr>
<tr>
<td>4th</td>
<td>6.133</td>
<td>655</td>
<td>330.77</td>
</tr>
<tr>
<td>5th</td>
<td>6.197</td>
<td>630</td>
<td>318.15</td>
</tr>
<tr>
<td>6th</td>
<td>6.242</td>
<td>600</td>
<td>303.00</td>
</tr>
</tbody>
</table>

Maximum compression strength

\(f_{m}^{'\prime} \approx 5.05\) MPa

b. Response of the infill element

Figure 2. Simulation of the infill panel based on the diagonal strut model and mechanical parameters

Table 1. Main characteristics of the seismic excitations and scale factors that fit the EC8 spectrum at two different levels of seismic demand.

<table>
<thead>
<tr>
<th>Event (seismic record)</th>
<th>Year</th>
<th>PGA(g)</th>
<th>Duration</th>
<th>Mag</th>
<th>Significant Damage</th>
<th>Near Collapse</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hector Mine ((FN 1830))</td>
<td>1999</td>
<td>0.043</td>
<td>38s</td>
<td>7.13</td>
<td>5.34</td>
<td>9.25</td>
</tr>
<tr>
<td>Chi-Chi-Taiwan-06 ((FN 3495))</td>
<td>1999</td>
<td>0.089</td>
<td>75s</td>
<td>6.30</td>
<td>2.51</td>
<td>4.35</td>
</tr>
<tr>
<td>Northridge-01 ((FN 1008))</td>
<td>1994</td>
<td>0.153</td>
<td>40s</td>
<td>6.69</td>
<td>1.70</td>
<td>2.95</td>
</tr>
</tbody>
</table>

At the near collapse seismic (NC) demand level, the interstory drifts are much smaller for the infilled frames in the upper floor levels (4th - 6th) than the corresponding requirements of the bare frame (Figure 3b). Further, Figure 3b shows that the absence of infills in the base floor of the frame (pilotis type) led to an increase of the maximum interstory drifts in the first and the second floor level of the frame in
comparison to the corresponding values of the other two frames (fully infilled and bare frame). Thus, at the NC level of seismic demand the damage distribution between the three frames is expected to be quite different. This observation is clearly depicted in Figure 4.

Seismic demand: Significant Damage

Seismic demand: Near Collapse

Figure 3. Maximum interstory drifts of the 6-storey frame structures (a) at significant damage and at (b) at near collapse seismic demand - maximum requirements from the three seismic excitations

Seismic performance of RC beams

Seismic performance of RC columns

Acceptable limits of $\theta_p$ (in rad) according to ATC-40:

**Beams**
- SS (structural stability): $\theta_p = 0.02$
- LS (life safety): $\theta_p = 0.01$
- IO (immediate occupancy): $\theta_p = 0.005$

**Columns**
- SS (structural stability): $\theta_p = 0.015$
- LS (life safety): $\theta_p = 0.0075$
- IO (immediate occupancy): $\theta_p = 0.0025$

Figure 4. Maximum plastic rotations $\theta_p$ and corresponding limit states of beams and columns of the 6-storey frame structures at the near collapse seismic demand, (a) and (b) maximum requirements from the three seismic excitations (c) failure mode in case of FN 1008 seismic excitation

In Figure 4 the maximum plastic rotations $\theta_p$ that are developed in the structural members of the RC
frames at the seismic demand of NC are presented and are also compared with the acceptable plastic rotation levels of three different limit states. For this purpose, the local limit states of structural stability (SS), life safety (LS) and immediate occupancy (IO) according to ATC-40 [14], are used. The final failure mode of the three frames (infilled and bare) is also presented in Figure 4c for the case of Northridge earthquake (FN1008).

The results show that in the case of bare frame the damage is mainly concentrated in the beams of the structure with the maximum requirements to be observed at the 5th floor level (Figure 4a). With the exception of the beams in the top floor of the bare frame all the other beams have exceeded the limit state of structural stability (SS). Critical could be characterized the inelastic response of the columns of the 6th story level of the bare frame since a soft-story mechanism has been developed (Figures 4b and c). Nevertheless, the developing plastic rotations in these columns are less than the local limit of \( \theta_p \) at IO limit state.

The presence of infills has drastically changed the damage distribution throughout the frame. As it can be observed in Figure 4 for the case of the fully infilled frame all the columns remain in the elastic range, while from the 3rd to the 6th floor level the beams’ local inelastic requirements have been greatly decreased in comparison to the corresponding local performances that are developed in the case of the bare frame. The seismic performances of these beams do not exceed the local limit state of SS, indicate this way the beneficial effect of infills. However, the maximum plastic rotations that are developed in the beams of the first two floor levels of the infilled frame are greater than the corresponding demands of the same beams in the case of the bare frame. Similar observations are obtained when the maximum local inelastic requirements of beams between pilotis type frame and bare frame, are compared (Figs 4a and c).

Nevertheless, the absence of infills at the base floor (pilotis frame) has as a result:
(a) the maximum plastic requirements of beams at the 1st floor level to be much higher than the corresponding demands in the other two frames (bare and infilled),
(b) the damage distribution at the columns has been significantly altered and has been concentrated at the base of the 1st floor (Figure 4b), while in case of the bare frame the inelastic response of the columns was observed at the 6th floor and in the case of the infilled frame all columns were behaved totally elastic,
(c) the maximum inelastic requirements for plastic rotation that are developed in the columns at the base of the pilotis type frame have not been reduced due to the presence of infills in comparison to the maximum demands that are developed on the critical columns of the bare frame at the top floor.

The influence of the infills on the top displacement \( \delta_{top} \) and on the base shear \( V \) requirements that are developed at the seismic demand levels of SD and NC are presented for the seismic excitation of Chi-Chi- Taiwan-06 (FN3495) in Figures 5a and b, respectively. The maximum seismic performances of the 6-storey frames in terms of top displacement \( \delta_{top} \) and base shear \( V \) requirements for all the examined cases of this study are presented in Table 2. Based on these results it can be observed that in all the examined cases of this study infills reduce the maximum requirements for top displacement \( \delta_{top} \) of the 6-storey frame. This reduction is less intense in the SD seismic demand level and that is attributed to the fact that the level of damages throughout the frame at this seismic demand is low. However, the base shear \( V \) requirements of the 6-storey frame in the case of pilotis type are greater than the corresponding demands of the other two frames (bare frame and infilled frame) at both levels of seismic demand (Figure 5 and Table 2).

An approximate value of the response reduction factor \( R \) of the frames can be obtained as a ratio of \( V_u/V_d \), where \( V_u \) is the maximum base shear requirements and \( V_d \) is the design base shear of the 6-storey frame that is equal to 594.69kN. Thus, the relative values of \( R \) for the examined frames at each seismic demand level are:

\[
\text{Seismic demand of Significant Damage:} \\
R_{bare} = 1.72 \quad R_{infilled} = 1.51 \quad R_{pilotis} = 2.76
\]

\[
\text{Seismic demand of Near Collapse:} \\
R_{bare} = 2.31 \quad R_{infilled} = 2.10 \quad R_{pilotis} = 3.29
\]
Figure 5. Time history requirements of the 6-storey frames in terms of top displacement $\delta_{\text{top}}$ and base shear $V$ at the seismic demands of significant damage (a) and near collapse (b).

Table 2. Maximum seismic performances of the 6-storey frames in terms of top displacement $\delta_{\text{top}}$ and base shear $V$ requirements.

<table>
<thead>
<tr>
<th>Seismic demand of Significant Damage</th>
<th>Seismic excitation</th>
<th>FN1008</th>
<th>FN1830</th>
<th>FN 3495</th>
</tr>
</thead>
<tbody>
<tr>
<td>bare frame</td>
<td>$\delta_{\text{top}}$ (m)</td>
<td>0.0506</td>
<td>0.0602</td>
<td>0.0605</td>
</tr>
<tr>
<td>fully infilled frame</td>
<td>$V$ (kN)</td>
<td>1025.0</td>
<td>933.8</td>
<td>944.3</td>
</tr>
<tr>
<td>pilotis type frame</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>bare frame</td>
<td>$\delta_{\text{top}}$ (m)</td>
<td>0.0335</td>
<td>0.0415</td>
<td>0.0363</td>
</tr>
<tr>
<td>fully infilled frame</td>
<td>$V$ (kN)</td>
<td>893.4</td>
<td>897.3</td>
<td>882.1</td>
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<tr>
<td>pilotis type frame</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>bare frame</td>
<td>$\delta_{\text{top}}$ (m)</td>
<td>0.0283</td>
<td>0.0357</td>
<td>0.0325</td>
</tr>
<tr>
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<td>$V$ (kN)</td>
<td>1547.5</td>
<td>1640.6</td>
<td>1533.9</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Seismic demand of Near Collapse</th>
<th>Seismic excitation</th>
<th>FN1008</th>
<th>FN1830</th>
<th>FN 3495</th>
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</thead>
<tbody>
<tr>
<td>bare frame</td>
<td>$\delta_{\text{top}}$ (m)</td>
<td>0.1507</td>
<td>0.0894</td>
<td>0.1186</td>
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<tr>
<td>fully infilled frame</td>
<td>$V$ (kN)</td>
<td>1371.9</td>
<td>1115.4</td>
<td>1214.1</td>
</tr>
<tr>
<td>pilotis type frame</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>bare frame</td>
<td>$\delta_{\text{top}}$ (m)</td>
<td>0.04438</td>
<td>0.0608</td>
<td>0.051</td>
</tr>
<tr>
<td>fully infilled frame</td>
<td>$V$ (kN)</td>
<td>1248.6</td>
<td>1192.1</td>
<td>927.6</td>
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<tr>
<td>pilotis type frame</td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>bare frame</td>
<td>$\delta_{\text{top}}$ (m)</td>
<td>0.0807</td>
<td>0.0671</td>
<td>0.0515</td>
</tr>
<tr>
<td>fully infilled frame</td>
<td>$V$ (kN)</td>
<td>1956.8</td>
<td>1931.6</td>
<td>1814.3</td>
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</tbody>
</table>

In all the cases the response reduction factors $R$ are less than the design behaviour factor $q$ of the structure that is equal to 3.5. It can also be observed that at both seismic demands levels the smaller
values of R are obtained in the cases of fully infilled frames, while the pilotis type frames yielded the higher values of R due to the shear effects in the columns. The local inelastic responses of infills are presented in Figures 6a and b, in terms of compressive axial deformations vs. top displacement of the structure at the seismic demand of NC for the seismic excitation of Northridge (FN1008). In the same figure the three different response levels of the infills are also shown (see Figure 2b): (a) deformation level (‰) at maximum strength (RL3 – blue line), (b) deformation level (‰) with strength and stiffness degradation (RL4 – green line) and (c) ultimate deformation level (‰) – infill collapse (RL5 – red line).

Figure 6. Local inelastic responses of infills vs. top displacement of frames in case of FN 1008 seismic excitation at near collapse limit state (a), (b). Failure mode and corresponding limit state of infills at significant damage (c) and at near collapse seismic demand (d) - maximum requirements from the three seismic excitations.
It can be observed that in the case of fully infilled frame the infill panels of the 1st floor exceed the deformation response level of RL3 (1.3%) at a top drift equal to 3.8 cm with the maximum requirement for axial deformation to be equal to 1.53%. In the case of pilotis frame, the infills of 2nd floor have totally collapsed since they have exceeded the deformation response level of RL5 (3.9%) at a top drift equal to 7.96 cm with the maximum requirement for axial deformation to be equal to 4.19%.

The final failure modes of the infills at the seismic demand levels of significant damage and near collapse are presented in Figures 6c and d, respectively. In these figures the blue hatching indicates that the developing demands for deformation of infills have reached the deformation level at ultimate stress strength (initiation of strength degradation), while the red hatching denotes infills collapse. In Table 3 the top drifts (hstr) that correspond to the first time that each deformation level of the infill is reached during the seismic analyses are also given for both seismic demand levels.

In the examined cases of this study, infills collapse has occurred only in case of pilotis type frame at the 2nd floor and for the case of NC demand (Figure 6 and Table 3). In all the other cases, significant strength and stiffness degradation in the infills (response level – RL4) has not been observed. Cracking of infills have been occurred at the low (1st to 3rd) floors of the infilled frames (fully infilled and pilotis) both at SD and NC seismic demands.

<table>
<thead>
<tr>
<th>Seismic demand of Significant Damage</th>
<th>first cracking (RL1)</th>
<th>RL3</th>
<th>RL4</th>
<th>RL5</th>
</tr>
</thead>
<tbody>
<tr>
<td>fully infilled frame</td>
<td>0.0082</td>
<td>0.036</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>pilotis type frame</td>
<td>0.0091</td>
<td>0.036</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Seismic demand of Near Collapse</th>
<th>first cracking (RL1)</th>
<th>RL3</th>
<th>RL4</th>
<th>RL5</th>
</tr>
</thead>
<tbody>
<tr>
<td>fully infilled frame</td>
<td>0.0100</td>
<td>0.034</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>pilotis type frame</td>
<td>0.0097</td>
<td>0.036</td>
<td>0.065</td>
<td>0.081</td>
</tr>
</tbody>
</table>

*level reached for the first time during the seismic analyses (three seismic excitations)

5. CONCLUSIONS

In this study, the influence of the infills on the global and local seismic performances of a 6-storey reinforced concrete (RC) frame structure is investigated at the seismic demand levels of significant damage and near collapse. For this purpose, dynamic step by step analyses have been performed using three different seismic excitations that have been properly scaled to fit the EC8 elastic spectrum at both seismic demands. Three types of RC frames are studied: (a) fully infilled, (b) pilotis type frame and (c) bare frame for comparison reasons. Results in terms of failure modes, interstory drifts, top displacements, base shear requirements, ductility requirements and infills local inelastic responses are presented. The main conclusions of this study are:

- The influence of infills on the seismic response of the 6-storey frame is beneficial both at the significant damage and at near collapse seismic demand levels as long as the infills have not exceeded their maximum capacity strength.
- The damage distribution over the structure is completely changed due to the presence of infills.
- Nevertheless, the absence of infills at the base floor (pilotis frame) causes additional modification on the damage distribution throughout the frame.
- Infills total collapse has been observed only in the case of pilotis type frame at the seismic demand level of near collapse.
- However, at both levels of seismic demand, the base shear requirements in the case of pilotis type frame are greater than the corresponding demands of the other two frames. Thus, in the case of pilotis special measurements have to be taken in order to ensure the columns of the 6-storey RC frame from shear failure.
7. REFERENCES


