Evaluation of Seismic Behavior of RC Moment Resisting Frame with Masonry Infill Walls

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Abstract
Masonry infill walls are frequently used as interior partitions or exterior walls for low-rise RC buildings. These infill walls are usually considered to be non-structural elements, and thus they are ignored in analytical models, because they are assumed to be beneficial to the structural behavior. In order to test this hypothesis, structural analyses were performed for a low-rise RC moment-resisting frame with and without masonry infill walls. From the analytical results, it has been shown that masonry infill walls can increase the strength and stiffness of a building structure, resulting in a decreased inter-story drift ratio. However, seismic forces applied to the structure are increased, because natural periods of the structure are shortened by the increase of stiffness. It should be noted that partial damage of infill walls between floor slabs can cause vertical irregularity of the strength and stiffness of the structure. It has also been shown that the inelastic deformation of a RC moment-resisting frame with a soft story is concentrated on the first story columns, and this partial damage may cause collapse of the entire system. To solve this problem, a structural design method has been proposed in this study.

Keywords: masonry infill wall; RC moment resisting frame; soft story; seismic responses; nonlinear behavior; plastic hinge rotation

1. Introduction
Unreinforced masonry infill walls, which are commonly used in developing countries with regions of high seismicity, will somewhat affect the seismic behavior of RC frame buildings. In recent earthquakes, numerous buildings were severely damaged or collapsed, due to the non-structural masonry walls, which were not considered in structural design. Fig.1 shows damage to masonry infill walls in RC frames and a building in Turkey with masonry infill walls having collapsed lower stories, due to the Izmit earthquake (1999).

In the case of residential buildings constructed in Korea, a reinforced concrete moment resisting frame with masonry infill walls is adopted as the structural system for many buildings, and they usually have pilotis in lower stories to meet architectural requirements as shown in Fig.2. In this structural system, the infill walls strengthen the upper stories, resulting in a soft first story of the structure, which is known to be a very weak system against seismic load. In the design of buildings, infill walls in upper stories are usually considered as non-structural elements, and are not included in the analytical model.

In this study, the effect of masonry infill walls on the seismic behaviors of RC buildings is investigated and a novel structural design method for a RC moment-resisting frame with a soft story has been proposed. In previous studies (Stafford, 1962; Klingner and Bertero, 1976), it has been found that masonry infill walls can provide considerable lateral resistance, and tend to be partially separated from the boundary frames, showing

Fig.1. Damage of Building Frame with Masonry Infill Walls

Fig.2. Villa-Style House Buildings with Pilotis in Lower Story
a compression strut mechanism under earthquake excitations. Therefore, an equivalent diagonal strut is used for the analytical model of the infill masonry wall in this study. Two historical earthquakes and an artificial earthquake created based on the design response spectrum, are used for dynamic analyses of the example structures. The computer code DRAIN-2DX (Prakash et al., 1993) is employed for nonlinear analysis of the example structures subjected to seismic loads.

2. Effect of Masonry Infill Walls

2.1 Previous Studies

Previous research on the structural behavior of RC frames with masonry infill walls subject to static and dynamic lateral cyclic loads has shown that infill walls lead to significant increases in strength and stiffness, compared to bare RC frames. Fiorato et al. (1970) tested a 1/8-scale reinforced concrete frame infilled with brick masonry, by using monotonically increasing load and cyclic lateral load. This research was followed by the studies of Klingner and Bertero (1976), Bertero and Brokken (1983), Zarnic and Tomazevic (1985), and Schmidt (1989). More recently, single-story reinforced concrete frames with masonry infills were studied by Mehrabi et al. (1994, 1996, 2002). The latter study also examined the behavior of masonry infill under out-of-plane loads. Angel (1994) performed analytical and experimental studies to predict the out-of-plane behavior and strength of masonry panels with and without previous in-plane damage. Based on the analytical model and on the supporting experimental results, this research resulted in the development of a seismic evaluation and rehabilitation procedure for unreinforced masonry infill panels loaded normal to their plane. Khalid et al. (1997) studied fragility analyses focusing on low-rise Lightly Reinforced Concrete (LRC) frame buildings with and without infill walls. Based on the obtained fragility curves, it was concluded that adding masonry infill walls to low-rise LRC frame buildings significantly reduces the likelihood of seismic damage. Madan et al. (1997) proposed an analytical macro model based on an equivalent strut approach integrated with a smooth hysteretic model for representing masonry infill panels in nonlinear analysis of frame structures. Simulations of experimental force-deformation behavior of prototype infill frame subassemblies were performed to validate the proposed model. Masonry infills in reinforced concrete buildings cause several undesirable effects under seismic loading: short-column effect, soft-story effect, torsion, and out-of-plane collapse. Hence, seismic codes tend to discourage such constructions in high seismicity regions. However, in several moderate earthquakes, such buildings have shown excellent performance even though many such buildings were not designed and detailed for earthquake forces. Murty et al. (2000) presented some experimental results showing that the masonry infills contribute significant lateral stiffness, strength, overall ductility and energy dissipation capacity. In their study, it is concluded that there is a need to develop robust seismic design procedures for such masonry infill RC frames considering that such buildings are the most common type of structures used for multi-story constructions in developing countries. In most instances, the lateral resistance of an infilled frame cannot be easily determined by a simple summation of those of the infill walls and the bounding frame.

2.2 Analytical Model of Masonry Infill Walls

Analytical models for masonry infill walls can be classified into micro and macro models. In the micro model, infill walls are modeled in detail at components level, such as mortar, bricks, and interface elements, to represent the behavior of infill walls more accurately. When the micro model is used for modeling a whole building structure, computational efforts may be required that are unacceptable in practice. Therefore, the micro model is generally used for numerical simulation and parametric studies, in comparison with the results of an experimental investigation. The macro model allows representation of the global behavior of infill walls, and its influence on the structural response. Stafford (1962) has provided an equivalent width of infill walls, using an elastic theory to represent the stiffness of infill walls. The equivalent width is a function of the stiffness of the infill walls, relative to that of the boundary frame. By analogy between a beam and an elastic foundation, Stafford Smith has defined a dimensionless relative stiffness parameter, to determine the degree of frame–infill interaction, and thereby the effective width of the strut. The most commonly used macro model is the bi-equivalent diagonal strut model. Therefore, in this study, the masonry infill walls were modeled by using the bi-equivalent diagonal strut model.

3. Example Structures and Seismic Excitations

3.1 Design of Example Structures

Five-story reinforced concrete framed structures are used for example structures. They are designed for three levels (low, moderate and high seismicity regions) of seismic hazard. The plans and elevations of example structures are shown in Fig.3. The boundary conditions of all columns of the first story are the fixed support. Dead loads of 5.5kN/m² and live loads of 2.5kN/m² are assumed to be applied to the example structures. Wind loads and seismic loads are determined according to the Uniform Building Code (UBC) 97. A basic wind speed of 70 mph is selected to determine wind loads. The soil profile type was assumed to be S₂, and the importance factor of 1.0 is used to determine seismic loads.

In UBC-97, the use of the ordinary moment resisting frame (OMRF) is restricted in seismic zones 2A, 2B, 3 and 4, and the intermediate moment resisting
frame (IMRF) is not allowed in seismic zones 3 and 4. Therefore, three example structures with special moment resisting frame (SMRF), using the modification factors of 8.5, are designed for seismic zones 1, 2B and 4, respectively, to investigate the inelastic behaviors of building structures in low (LSR), moderate (MSR) and high (HSR) seismicity regions. The calculated design earthquake loads of each case are shown in Table 1. Section lists of all members are also presented in Tables 2–4.

Table 1. Design Lateral Loads

|        | LSR  | MSR  | HSR  |
|--------|------|------|------|
| SMRF (R=8.5) | 307.61 | 769.03 | 1845.67 |

Table 2. Section Lists of Example Structure in LSR (Unit: mm)

| Column | Story | Section | Girder | Story | Section |
|--------|-------|---------|--------|-------|---------|
| C1     | 1–2   | 500x500 | G1     | All   | 300x600 |
|        | 3–5   | 400x400 |        |       |         |
| C2     | 1–2   | 400x400 | G2     | All   | 300x600 |
|        | 3–5   | 400x400 |        |       |         |
| C3     | 1–2   | 500x500 | G3     | All   | 300x600 |
|        | 3–5   | 400x400 |        |       |         |
| C4     | 1–2   | 400x400 | G4     | All   | 300x600 |
|        | 3–5   | 400x400 |        |       |         |

Table 3. Section Lists of Example Structure in MSR (Unit: mm)

| Column | Story | Section | Girder | Story | Section |
|--------|-------|---------|--------|-------|---------|
| C1     | 1–2   | 600x600 | G1     | All   | 300x600 |
|        | 3–5   | 500x500 |        |       |         |
| C2     | 1–2   | 500x500 | G2     | All   | 300x600 |
|        | 3–5   | 500x500 |        |       |         |
| C3     | 1–2   | 600x600 | G3     | All   | 300x700 |
|        | 3–5   | 500x500 |        |       |         |
| C4     | 1–2   | 500x500 | G4     | All   | 300x700 |
|        | 3–5   | 500x500 |        |       |         |

Table 4. Section Lists of Example Structure in HSR (Unit: mm)

| Column | Story | Section | Girder | Story | Section |
|--------|-------|---------|--------|-------|---------|
| C1     | 1–2   | 600x600 | G1     | All   | 300x600 |
|        | 3–5   | 500x500 |        |       |         |
| C2     | 1–2   | 500x500 | G2     | All   | 300x600 |
|        | 3–5   | 500x500 |        |       |         |
| C3     | 1–2   | 600x600 | G3     | All   | 300x700 |
|        | 3–5   | 500x500 |        |       |         |
| C4     | 1–2   | 500x500 | G4     | All   | 300x700 |
|        | 3–5   | 500x500 |        |       |         |

3.2 Analytical Model of Example Structures

Three types of example structures are used to evaluate the influence of masonry infill walls as show in Fig.4. Model F does not have any infill walls in a RC moment resisting frame. Model S has infill walls in all but the first story; and Model W has infill walls in all stories. The computer code DRAIN-2DX (Prakash et al., 1993) is employed for 2-dimensional nonlinear analysis. Because the plan of the example building structure shown in Fig.3.(a) is symmetric about the X-axis, the 3-dimensional example structure can be modeled by equivalent 2-dimensional frames as shown in Fig.4. The equivalent 2-dimensional models include only in-plane three DOFs. The force-displacement relationships of structural elements used in this study are shown in Fig.5. A bi-linear hysteresis relationship with 2% strain hardening ratio is assumed for the plastic hinges at both ends of the column and beam elements, respectively. For modeling of masonry infill walls, an equivalent diagonal compression strut...
is used, as shown in Fig.5(b). In this study, the out-of-plane failure of masonry infill walls is neglected. Because infill walls are fully added to each story without openings, the seismic response of slender RC structural elements is dominated by not shear behavior but flexural behavior. Therefore, the inelastic shear behavior of columns and beams is neglected.

For estimation of the initial stiffness and maximum strength of the infill walls, the effective width ($W_{ef}$), initial stiffness ($K_{in}$) and maximum strength ($F_{max}$) of the equivalent diagonal compression strut were proposed by Dolšek and Fajfar (2002), and are used in this study as follows.

$$W_{ef} = 0.175(\lambda H)^{4.5} \sqrt{H^2 + L^2}, \quad \lambda = \frac{E_s t_s \sin(2\theta)}{4E_s I_H}$$  \hspace{1cm} (1)

where, $E_s$ and $E_c$ are the modulus of elasticity of the infill wall and the concrete (i.e. the boundary frame material), respectively. $\theta = \arctan(H/L)$ is the inclination of the diagonal, $t_s$ is the thickness of the infill wall, and $I_H$ is the moment of inertia of the column of the boundary frame, whereas $H_{in}, H$ and $L$ are the net height of the infill wall, the story height, and the bay length of the frame respectively. The initial lateral stiffness $K_s$ is equal to

$$K_s = \frac{E_w t_s}{\sqrt{H^2 + L^2}}$$  \hspace{1cm} (2)

A simplified form of the maximum strength of the infill walls (Dolšek and Fajfar, 2002) is

$$F_{max} = 0.818 \frac{L_{in} f_{tp} f_{m}}{C_i} \left(1 + \sqrt{C_i^2 + 1}\right), \quad C_i = 1.925 \frac{L_{in}}{H_{in}}$$  \hspace{1cm} (3)

where $f_{tp}$ is the cracking strength of the infill, obtained from a diagonal compression test, and $L_{in}$ and $H_{in}$ are the length and height of the infill, respectively.

In UBC-97, the elastic modulus of the masonry infill wall is proposed to be $E_w = 750 f_{m}$, by using $f_{m}$, that is the compressive strength of a masonry prism. This equation is suitable for the elastic modulus of the reinforced block masonry infill walls widely used in the USA. However, unreinforced brick masonry infill walls are generally used in Korea. Kim et al. (2001) proposed the equation of $E_w = 100 f_{m}$ for the elastic modulus of the unreinforced brick masonry walls, and this is employed in this paper. The cracking strength of the infill wall ($f_{tp}$) is selected to be 0.36MPa, based on the tensile strength of a masonry prism experiment. The crack ($d_{cr}$) and maximum displacement ($d_{max}$) of infill walls in Fig.5(b) are assumed to be 0.1% and 0.6% of inter-story displacements, respectively, which means the immediate occupancy and collapse prevention performance level from Table C1-3 in the FEMA-356. Using above equations, the properties of the diagonal strut are calculated as shown in Table 5.

### 3.3 Earthquake Ground Motions

Nonlinear time history analyses of example structures were performed using two historical earthquakes, i.e. El Centro (1940) and Taft (1952) earthquakes, and an artificial earthquake created based on the design spectrum. The effective peak ground acceleration (EPA) of each ground motion data is scaled to 0.08g, 0.2g and 0.4g. Fig.6. shows the spectral acceleration data of three earthquake loads with the EPA of 0.4g. The natural periods of three example structures are also shown in Fig.6. Because of the additional stiffness of infill walls, the natural period of Model S is shorter than that of Model F without infill walls, and the natural period of Model W having infill walls in all the stories is the shortest. Accordingly, the seismic loads expected for Model W will be larger than that for the other models.

### 4. Evaluation of Seismic Behavior

#### 4.1 Force-Displacement Relationship

In ATC-40, the roof displacement and base shear are used to define the force-displacement relationship for multi degree of freedom (MDOF) structures. However, it is difficult to define the force-displacement relationship for MDOF structures using the roof displacement and base shear, because the roof displacement cannot represent the deformed shape of the MDOF structure. Lee et al. (1996) proposed an analytical method that can take the deformed shape of structures into account. In the conventional equivalent single-degree of freedom (ESDOF) system methods, it is required to evaluate the properties of the ESDOF system for the MDOF system prior to the inelastic dynamic response analysis of the ESDOF system. In the study performed by Lee et al. (1996), the representative responses for MDOF systems

#### Table 5. Properties of Diagonal Strut

| Property | $t_s$ (mm) | $W_{ef}$ (mm) | $K_s$ (kN/mm²) | $F_{max}$ (kN) |
|----------|------------|---------------|----------------|----------------|
| Value    | 200        | 907.6         | 21.3           | 848.1          |

#### Fig.6. Response Spectrum of Earthquake Loads
are directly evaluated from the inelastic dynamic responses of the MDOF systems. In general, dynamic responses of most of the multi-story frames with regular configurations are governed by the first mode shape of the structures. Therefore, the representative displacement of a multi-story building structure is assumed to be the first modal displacement for the MDOF response obtained from inelastic dynamic analysis of the MDOF system. This method proposed by Lee et al. (1996) is employed in this study. The analytical models in regions of low, moderate, and high seismicity have been designed by using the same gravity load and response modification factor, but they were designed for different seismic hazard levels. Because the force-displacement relationship of each analytical model for the artificial earthquake is very similar to those for the other ground motions, the force-displacement relationships only of the artificial ground motion are shown in Figs.7. - 9.

Since the building structures in low seismicity regions are usually designed not by earthquake loads, but by gravity loads, they have a larger overstrength factor, compared to the structures in high seismicity regions. Models F, S and W of the building structures in low seismicity regions do not clearly show ductile or inelastic behavior against ground motions of the design level, because they have additional strength and stiffness due to the masonry infill walls, resulting in considerable overstrength. In the case of Models S and W of the structures in moderate and high seismicity regions, the slope of the force-displacement hysteresis curve is decreased during a loading-unloading cycle, because of the stiffness reduction caused by the cracking of infill walls, as shown in Figs.8. and 9. Especially in the case of Model S in HSR, inelastic deformation behavior mainly occurs in the first story columns, because of the effect of infill walls in the upper stories of Model S, resulting in a soft first story. Accordingly, the force-displacement relationship of Model S is similar to that of Model F.

4.2 Inter-Story Drift Ratio

Based on analytical results, the average inter-story drift ratios are presented in Fig.10., and these average values of inter-story drift ratios for each Model under three seismic loads are calculated, as shown in Table 6. It can be seen from the figure that the inter-story drift ratio of Model W is smaller than that of Model F, due to increased system stiffness, caused by masonry infill walls. This tendency is apparent in low seismic hazard regions, because the seismic load is too small to induce inelastic deformation behavior, and thus the stiffness and strength of infill walls, as well as of boundary frames, are retained. Small seismic loads in the design of structures may result in a larger overstrength in low seismicity regions. The inter-story drift ratio of Model S is larger than that of Model F at the soft first story, though inter-story drift ratios of Model S in the upper stories are smaller than those of Model F. The inter-story drift ratios of Model S are similar to those of Model W, except the first story. Therefore, if structures with infill walls in all stories except the first story, are designed without proper consideration of the effects of their infill walls, earthquake damage may be concentrated on the first story columns; and partial damage may lead to the possibility of collapse of the system. Irregular distributions of masonry infill walls in elevation can result in unacceptably elastic displacement in the soft story frame, or soft stories can be formed, due to premature failure of masonry infill walls at any level of framed structures. This can unintentionally weaken the seismic resistance capacity of the structures.
4.3 Distribution of Plastic Hinges

Distributions of the maximum rotation angle of the plastic hinges on each structural member are shown in Fig.11. The size of circle indicates the degree of rotation angle of the plastic hinges at each member's ends. Therefore, a larger circle means a larger inelastic deformation. In Fig.11., the character 'C' indicates that the infill walls are cracked, and the character 'Y' is used when the infill walls have yielded. If one of two diagonal compression struts has yielded, the infill wall is assumed to lose its load resistance capacity in this study. In the case of Models S and W, all of the infill walls in lower stories yield, but the infill walls in upper stories still have seismic resistance capacity, with some cracks. When analytical results of Models F and W are compared, it can be seen that the inelastic deformation behavior of main frame members in upper stories, and enhance the seismic resistance capacity of the structures subjected to design level earthquake loads. In lower stories, however, the inelastic deformation of main frame members of Model W is similar to that of model F. In the case of Model S, the inelastic deformation behaviors are concentrated on the first story columns and beams, and they are larger than those of Model F. Therefore, if earthquake loads bigger than the design level are applied to Model S, local damage may occur in the soft first story, and this local damage may cause collapse of the entire structural system.

5. Novel Structural Design Method for a Structure with Soft Story

If the infill walls are distributed discontinuously in the building, and there is the possibility of thereby weakening the seismic resistance capacity of the structure, structural design methods that can solve this problem are required. In this study, additional shear force ($V_1$), that is determined in proportion to the seismic design base shear force ($V_b$) when seismic design forces are calculated by the equivalent static load method, is used for redesign of the main structural members in the soft first story, to enhance its seismic resistance capacity, as shown in Fig.12.

![Fig.12. Design Method Applying Additional Shear Force](image)

The seismic design base shear ($V_b$) is easily calculated by a seismic design code without earthquake response analyses. For example, in the Uniform Building Code (UBC)-97, $V_b$ is determined in accordance with the following equation:

$$V_b = \frac{C_f I W}{RT}$$  \hspace{1cm} (4)

where, $C_f$ is the seismic coefficient determined by soil profile type and seismic zone factor, $I$ is the occupancy importance factor, $R$ is the response modification factor, $T$ is the fundamental period of vibration. When the weight ($W$) of the structure under consideration is determined, the design base shear ($V_b$) is readily determined because the other coefficients are easily determined from the seismic design code in accordance with the building use, building height, construction site, and structural system. In addition to UBC-97, most seismic design codes provide similar simple equations for the design base shear. When $V_b$ is calculated by a seismic design code without structural analyses, $V_1$ can be simply determined by using the proportional factors (1.0 and 1.5 proposed in this
study). The $V_f$ values evaluated in this paper can be applied only to the building in this study. Therefore, the $V_f$ value for another building structure should be recalculated. However, as stated above, $V_f$ is simply determined based on $V_b$ that is easily calculated by a seismic design code without seismic response analyses. Inter-story drift ratios of Model F and Model S in high seismicity regions, when subjected to design level earthquake load (0.4g EPA) and 3/2 of design level earthquake load (0.6g EPA), are presented in Fig.13. When an earthquake load greater than the design level is applied, not only the inter-story drift ratio of the first story, but also that of the second story of Model S becomes larger, compared to that of Model F, as shown in the figure. This problem can be solved by using the design method mentioned previously. After redesign of the main frame members in the soft first story, by applying additional shear force ($V_1$), the inter-story drift ratios of Model F and Model S are recalculated, as shown in Fig.13. Here, additional shear force ($V_1$) is calculated to be 1.0 times and 1.5 times as large as the base shear force ($V_b$), based on the column shear force ratios shown in Table 7.

### Table 7. Story Shear Force of Members in 1st Story (Unit: kN)

|       | El Centro | Taft | Artificial | Average |
|-------|-----------|------|------------|---------|
|       | Col.      | Infill. | Col. | Infill. | Col. | Infill. | Col. | Infill. |
| LSR   | Model F   | 892   | 707     | 706     | 768   | -      | -    | -      |
|       | Model S   | 1614  | 1555    | 1220    | 229   | 1323   | 242  | 1203   |
|       | Model W   | 182   | 1066    | 246     | 229   | 1323   | 242  | 1203   |
| MSR   | Model F   | 1607  | 1277    | 1342    | 1490  | -      | -    | -      |
|       | Model S   | 2164  | 2458    | 2277    | 2300  | -      | -    | -      |
|       | Model W   | 1870  | 1328    | 1177    | 1323  | 1135   | 1326 | 1326   |
| HSR   | Model F   | 2503  | 2453    | 2476    | 2478  | -      | -    | -      |
|       | Model S   | 3074  | 3122    | 2883    | 3026  | -      | -    | -      |
|       | Model W   | 2707  | 1303    | 2649    | 1329  | 2591   | 2649 | 1320   |

In spite of the additional shear force ($V_1$), there is no increase of section size of column and beam members. However, the yield moments of column and beam members are increased by 1.3 and 1.2 times, respectively, by adding reinforcing bars, when additional shear force ($V_1$) is equal to the base shear force ($V_b$). In the case of $V_1=1.5V_b$, the yield moments of column and beam members are increased by 1.42 and 1.28 times, respectively. It can be seen from Fig.13. that Model S can show similar inter-story drift ratios to Model F, when the main frame members in the soft first story of Model S are redesign, by using additional shear force ($V_1=V_b$). If additional shear force ($V_1$) that is 1.5 times bigger than the base shear force ($V_b$) is used for the redesign of Model S, the structure with a soft first story is expected to show considerably enhanced seismic resistance capacity, when subjected to earthquake loads greater than the design level. Fig.14. shows the distributions of the maximum rotation angle of plastic hinges on each structural member, after redesign of Model S. As can be seen in the figure, the distribution of plastic hinge rotation angle of Model S is similar to that of Model F, and the seismic energy is dissipated throughout the structural system of the redesigned Model S. Accordingly, if the proposed structural design method using additional shear force ($V_1$) is used for redesign of the main frame members in the soft story, the building structure having discontinuously distributed infill walls is expected to have enhanced seismic resistance capacity after redesign. In this study, the value of additional shear force ($V_1$) is selected to be 1.0 or 1.5 times larger than the base shear force ($V_b$). If more various structures and earthquake loads are used for analytical studies, a proper scale factor for additional shear force ($V_1$) can be proposed in further research.

![Fig.13. Average of Inter-Story Drift Ratios (HSR)](image)

(a) 0.4g EPA  
(b) 0.6g EPA  
(c) Model S (Re-design)

![Fig.14. Plastic Hinge Rotation Angle (HSR, El Centro EQ.)](image)
6. Conclusions
In this study, the seismic behaviors of RC moment-resisting frames with and without masonry infill walls have been investigated. To this end, example structures were designed for three levels of seismic hazard, i.e. low, moderate, and high seismicity regions. Nonlinear time history analyses were performed, using two historical and one artificial earthquake load. Based on analysis and evaluation results, the main features of this study are summarized below:

1. In general, infill walls are considered to be non-structural elements, and thus their strength and stiffness are ignored in structural design, because they are assumed to be beneficial to seismic resistance capacity. However, because infill walls can possibly decrease seismic resistance capacity, they should be considered in structural design.

2. Since building structures with masonry infill walls in low seismicity regions have additional strength and stiffness due to infill walls, resulting in considerable overstrength, they do not clearly show inelastic behavior against ground motions of design level. When an earthquake load greater than the design level is applied to a structure with infill walls, the infill walls in lower stories are damaged. After the destruction of these damaged infill walls, earthquake loads are resisted only by boundary frames in several stories, even if the structure has infill walls evenly distributed over the stories, i.e. there is no initial soft story. Consequently, the seismic behaviors of the stories having destroyed infill walls are similar to those of soft stories. This feature should be considered in the seismic design of structures with masonry infill walls.

3. Irregular distributions of masonry infill walls in elevation may unintentionally cause soft stories in building structures. When a structure with soft story is subjected to seismic excitation, significant elastic displacement occurs in the soft story frame, and this can weaken the seismic resistance capacity of the structure. The partial damage of the soft story frame may lead to the possibility of collapse of the whole structure. In order to consider this effect of soft story, it is recommended to take the strength and stiffness of the infill walls into account in the structural analysis model.

4. A structural design method has been proposed in this study in order to enhance the seismic resistance capacity of structures with soft stories. After redesign of the main frame members in the soft story, by applying additional shear force ($V_i$), the yield moments of column and beam members are increased, without section size increase of the column and beam members. Additional shear force is simply determined based on the seismic design base shear ($V_b$) that is directly obtained by a seismic design code. This $V_i$ value for redesign can be easily recalculated for each building structure with soft stories without the complicated seismic analysis. However, the $V_i$ values evaluated in this paper can be applied only to the example building in this study. Therefore, the $V_i$ value for another building structure should be recalculated.

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