Influence of Masonry Panels on the Life-Cycle Expected Performance Functions of Multi-Story RC Frames

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SUMMARY:  
A study is presented about the influence of infill masonry panels on the life-cycle expected performance functions of multi-story reinforced concrete frames exposed to severe seismic hazard conditions. It is done, comparing the present values of expected damage functions of a sample including the same reinforced concrete rigid-frame system: in one case there is not any infill-wall; in the second case, there are masonry infill-walls at all stories, except the first one, at the two exterior frames in planes parallel to the direction of ground motion, while in the third case infill-walls are inserted at all stories of the exterior frames of the building. The hysteretic behavior and damage model used here for concrete elements and for masonry walls take into account the strength and stiffness degradation as functions of damage accumulation.

Keywords: Masonry walls, life-cycle optimization, seismic performance, seismic risk, structural reliability.

1. INTRODUCTION

The influence on the structural performance of infilled-frames subjected to lateral loads has been recognized at least since the late 1930s [Rathbun, 1938]; it has been the subject of many experimental studies around the world, considering different lateral and vertical load conditions and taking into account particular characteristics of masonry components, construction methods, block types that vary from one region to another (e.g. Polyakov, 1956; Holmes, 1961; Stafford-Smith, 1962; Liao, 1964; Esteva, 1966; among others), as summarized by Crisafulli (1997).

In addition, experimental studies done by Tomaževič et al. (1996) on 32 half-scale masonry walls subjected to different lateral load patterns (statically and dynamically) at two levels of vertical loads, showed significant differences in lateral resistance and initial stiffness. As a general rule, they measured higher resistance and larger ultimate displacement for monotonically loading than for the other load patterns.

This study is oriented to presenting some information about the expected life-cycle benefits that can be obtained from the use of non reinforced and horizontally-reinforced confined masonry panels as structural infilling elements in rigid frame multistory buildings built at sites exposed to significant seismic hazard conditions. For this purpose, the nonlinear behavior of confined masonry panels subjected to cyclic loads is represented by the model proposed by Pérez-Martínez and Esteva (2011), based on experimental information provided by cyclic load tests performed at the National Center for Disaster Prevention of Mexico (CENAPRED) on full-scale confined masonry panels [Aguilar et al., 1996; Alcocer and Zepeda, 1999], where reinforcement confinement details and joint mortar properties were defined in compliance with the requirements established by the 1993 issue of the Mexico City Building Code [MCBC, 1993].
2. SEISMIC RELIABILITY ANALYSIS

The life-cycle framework adopted here for the evaluation of expected performance functions is taken from Esteva et al. (2002). Let $Q$ designate an intensity measure of a seismic ground motion acting on a structural system and $Z$ a quantitative indicator of the performance of that system when subjected to that action. For many applications, $Z$ is a safety factor equal to $\ln(C/D)$, where $C$ is the (drift) capacity of the system and $D$ is the corresponding seismic drift demand. For a given combination of structural system and seismic excitation, $C$ and $D$ are determined by means of a pushover analysis and a step-by-step analysis, respectively.

Following Cornell (1969), the seismic reliability level of a system is expressed in terms of the safety index $\beta$, equal to $\frac{Z}{\sigma}$, where $Z$ is the expected value of $Z$ (defined in the foregoing paragraph) and $\sigma$ is its standard deviation. In addition, the probability of failure $p_F$ is assumed to be approximately equal to $\Phi(-\beta)$, where $\Phi(.)$ is the normal standard cumulative probability distribution function.

2.1. Constitutive functions

Constitutive functions for reinforced concrete members are taken from Campos and Esteva (1997) and the hysteretic behavior and damage model for masonry walls from Pérez-Martínez and Esteva (2011), presented in Fig. 2.1. Both models are based on the concept of damage accumulation proposed by Wang and Shah (1987); they take into account the strength and stiffness degradation and are incorporated in program Drain 2D [Powell, 1973].

![Hysteretic models of a) RC element ends and b) Confined masonry wall](image)

Figure 2.1 Hysteretic models of a) RC element ends and b) Confined masonry wall

2.2. Capacity of the system and seismic demand

In order to estimate the lateral deformation capacity of each system, a pushover analysis with constant displacement configuration was performed on the corresponding detailed model. The displacement configuration used for this purpose was taken equal to that determined from conventional (square root of sum of squares) superposition of linear modal responses for the specified seismic-design spectrum; the shape of the resulting configuration of lateral displacements was kept constant while the frame was forced into the inelastic range. For this analysis, the excitation was a base acceleration growing slowly as a linear function of time; all gravitational loads and mechanical properties were taken equal to their expected values. The lateral deformation capacity was taken as equal to that corresponding to a reduction of 20% in the maximum base shear force reached by the RC frame during the pushover analysis.

Two measures of the seismic demand were considered:

a) The global lateral distortion, equal to the ratio of the peak absolute value of the amplitude of the roof displacement relative to that of the base of the structure, or
b) The local lateral distortion, equal to the ratio of the peak absolute value of the amplitude of the first story displacement relative to that of the base of the structure.

Case (b) will be used when the soft-story mechanism is produced. Both cases are obtained by dynamic nonlinear analyses of MDOF systems subjected to the ground motion accelerations time histories considered. For each system studied, a sample of dynamic response time histories was obtained, where both the system properties and the ground motion time histories were taken as random: the former were generated by means of Monte Carlo simulation, and the latter are either actual records or scaled versions of them.

2.3. Normalized intensity measure

A normalized intensity measure is defined as

\[ Q = m \cdot S_a / V_y \]

where \( S_a \) is the linear pseudo-acceleration response ordinate for the fundamental period of the system of interest, \( m \) is the mass of that system, and \( V_y \) is the yield value of the base shear force obtained by least squares fitting to the base-shear vs. lateral displacement function resulting from a pushover analysis.

2.4. Seismic hazard function

The seismic hazard function \( \nu(y) \) is defined here as the annual rate of occurrence of earthquakes with intensities \( Y \) greater than any given value, \( y \), at the site; it can be represented as follows:

\[
\nu(y) = K \cdot y^{-r} \left[ 1 - \left( \frac{y}{y_1} \right)^\varepsilon \right] \text{ if } y < y_1 \quad ; \quad \nu(y) = 0, \text{ otherwise.} \tag{2.1} \]

Here, \( y \) is a value of the earthquake intensity, measured by the ordinate of the linear pseudo-acceleration response spectrum for the fundamental period of the system of interest; \( r, y_1, \) and \( \varepsilon \) are parameters determined by site-specific seismic hazard studies.

2.5. Seismic vulnerability functions

According to Esteva \textit{et al.}, (2002), the vulnerability function \( \delta(y) \) can be expressed as the sum of the contributions of the expected damage costs in the cases of failure and survival of the system:

\[
\delta(y) = \delta_F \cdot p_F(y) + \bar{\delta}(y|S) \left[ 1 - p_F(y) \right] \tag{2.2} \]

where

\[
\bar{\delta}(y|S) = (\lambda + \eta_i) \sum_i r_{oi} \cdot \hat{g}(\Psi_i) \tag{2.3} \]

In these equations, \( p_F(y) \) is the failure probability under the action of an earthquake with intensity \( y \), \( \delta_F \) is the expected cost of failure in case it occurs, and \( \bar{\delta}(y|S) \) is the expected cost of damage, conditional to survival. Both \( \delta_F \) and \( \bar{\delta}(y|S) \) are normalized with respect to the initial construction cost, \( C_0 \).

In order to obtain the expected damage cost conditional to survival, \( \bar{\delta}(y|S) \), it is necessary to segregate the initial total cost \( C_0 \) into the contributions of the different system segments, such as: structural frame, partitions and similar elements, installations, and floor systems and it is done according to Sierra (2002), Ismael (2003), Pérez-Martínez and Esteva (2012).

3. LIFE-CYCLE OBJECTIVE FUNCTION

For purposes of life-cycle utility-based decision making, an objective function must be defined. Following Esteva (1969) and Rosenblueth (1976), the utility function for a system which is repaired or rebuilt immediately after each damaging earthquake, using the same specifications as the original
system, can be expressed by Eq. 3.1, where $\alpha$ is a vector formed by the values of the parameters that determine the relevant mechanical properties of the system to be designed:

$$U = C_0(\alpha) + \frac{D_0(\alpha)}{\gamma}$$  \hspace{1cm} (3.1)

Here, $C_0(\alpha)$ is the initial cost function, $D_0$ is the expected cost of damage and failure per unit time (year), and $\gamma$ is an adequate discount (interest) rate. $D_0$ depends on both the seismic activity at the site of interest and the damage function $\delta(y)$, which expresses the expected cost of damage as a fraction of $C_0$ for an earthquake with intensity equal to $y$. $D_0(\alpha)/\gamma$ is the present value of the expected cost of damage, which is estimated as the sum of the contributions of the conditions of survival and failure, identified in Eqs. 3.2 – 3.4 by subscripts $S$ and $F$, respectively:

$$D_0(\alpha) = [\Delta_S + \Delta_F] \cdot C_0(\alpha)$$  \hspace{1cm} (3.2)

where

$$\Delta_S = \int \left[ \frac{du_S(y)}{dy} \right] \cdot \delta_S(y) \cdot \left[ 1 - p_F(y) \right] \, dy$$  \hspace{1cm} (3.3)

$$\Delta_F = \int \left[ \frac{du_F(y)}{dy} \right] \cdot \delta_F \cdot p_F(y) \, dy$$  \hspace{1cm} (3.4)

4. ILLUSTRATIVE EXAMPLE

The illustrative example compares the present values of expected damage functions of a sample including the same reinforced concrete rigid-frame system: in one case there is not any infill-wall; in the second case, there are masonry infill-walls at all stories, except the first one, at the two exterior frames in planes parallel to the direction of ground motion, while in the third case infill-walls are inserted at all stories of the exterior frames of the building. The resulting configurations are shown in Fig. 4.1. The structural systems are located at a firm ground and were designed in accordance with the applicable seismic design regulations established for seismic Zone D by the Civil Works Design Handbook (Manual de Diseño de Obras Civiles, MDOC, 1993).

The building is regular both in plan and in elevation (Fig. 4.1) located at a firm ground site, base-shear coefficient of 0.1, and a seismic force reduction factor of 4.0. They are defined in terms of nominal values of both the gravitational loads and the mechanical properties of the structural members. The geometrical properties of the structural members are summarized in Table 4.1.

![Figure 4.1](image-url) General dimensions of cases studied; $L=6.0$ m, $h=3.0$ m, $h_1=4.0$ m.
Table 4.1. Geometry of columns and beams of cases studied: \( d = \text{depth}, b = \text{width}. \)

<table>
<thead>
<tr>
<th>Element</th>
<th>Story</th>
<th>Cross section ( d \times b ) (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Columns</td>
<td>1-2</td>
<td>500 x 500</td>
</tr>
<tr>
<td></td>
<td>3-5</td>
<td>450 x 450</td>
</tr>
<tr>
<td>Beams</td>
<td>1-2</td>
<td>250 x 500</td>
</tr>
<tr>
<td></td>
<td>3-5</td>
<td>200 x 500</td>
</tr>
</tbody>
</table>

In order to evaluate influence of masonry panels on the life-cycle expected performance functions of multi-story RC frames, three cases were selected: (1) bare frame system; (2) soft-story system; and (3) RC frame with masonry-infill in all panels. Additionally, two options of masonry panels are considered: (a) without horizontal reinforcement (M2); and (b) with reinforcement bars at the horizontal joints, a reinforcement ratio of 0.182% of the vertical cross section of the panel (M4) (See Table 4.2). The infill panels are built with traditional hand-made solid clay bricks [Aguilar et al., 1996], which are modeled taking into account the strength and stiffness degradation as functions of damage accumulation (Sec. 2.1).

Table 4.2 Cases studied: bare frame, soft-story and RC frame with masonry-infill in all panels

<table>
<thead>
<tr>
<th>Case</th>
<th>Horizontal reinforcement ratio of the masonry wall</th>
<th>ID</th>
<th>( T_0 ) (s) using expected values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bare frame</td>
<td>--</td>
<td>BF</td>
<td>0.88</td>
</tr>
<tr>
<td>Soft-story</td>
<td>--</td>
<td>SS-M2</td>
<td>0.55</td>
</tr>
<tr>
<td>Masonry-infill in all panels</td>
<td>--</td>
<td>FI-M2</td>
<td>0.28</td>
</tr>
<tr>
<td></td>
<td>Maximum</td>
<td>SS-M4</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Maximum</td>
<td>FI-M4</td>
<td></td>
</tr>
</tbody>
</table>

4.1. Selection of seismic excitations

Fig. 4.2 shows three typical ground motion acceleration time histories recorded at hard-rock sites in the Pacific coast of Mexico (Acapulco area) and the response spectra for a larger sample, for 5 percent damping ratio [Pérez-Martínez and Esteva, 2012].

**Figure 4.2** Typical ground motion acceleration time histories recorded at hard-rock sites in the Pacific coast of Mexico (Acapulco area) and their response spectra for 5 percent damping ratio. \( T \) is the natural period of vibration of the system considered.
4.2. Seismic reliability analysis

Lateral deformation capacity is obtained from a pushover analysis (C, Sec. 2) and it is presented in Fig. 4.3 where it can be observed, among other things, that the horizontal reinforcement in the masonry panel produces an increase of about 60 percent in the base-shear strength of the building; however, a drastic decrease in the base-shear is observed for a deformation much smaller than the deformation capacity of all the other specimens.

![Figure 4.3 Pushover analysis and its corresponding elasto-plastic fitting for cases studied (C, Sec. 2).](image)

Peak values of the global distortions (seismic demand $D$, Sec. 2) of the different systems are presented in Fig. 4.6. As expected, the largest global seismic demands are experienced by system BF, which has a deformation capacity not much larger than the other systems.

![Figure 4.4 Peak values of global distortions $\psi$ of detailed models of the systems ($D$, Sec. 2) vs. linear pseudo-acceleration response ordinates for the fundamental period of the system of interest, $S_a$.](image)

The reliability functions $\beta(Q)$ for the systems considered were determined for all the systems, according to the definition presented in Section 2; the results are shown in Figure 4.5.

4.3. Life-cycle analysis

The seismic hazard function is defined here as the annual rate of occurrence of intensities $Y$ greater than any given value, $y$, at the site. Intensities are measured by the ordinate of the linear pseudo-acceleration response spectrum, for 5 percent damping, for the natural period, $T$, of the system considered. Figure 4.6 shows the seismic hazard functions for the three basic cases presented in Table 4.2; they were determined using computer program PSM [Ordaz et al., 1996].
Figure 4.5 Reliability index $\beta$ for cases studied as function of the normalized intensity measure $Q$.

Figure 4.6 Annual rates of occurrence of earthquakes with intensities greater than $y$ at the site. $T$ is determined using expected values of the mechanical properties of the system.

Figure 4.7 shows the expected damage functions conditional to survival for RC frame, partition wall and masonry wall (types M2 and M4, Table 4.2), determined in accordance with Eqns. 3.2 – 3.4. In this study, it is considered that the bare frame system BF is infilled with partition walls at all stories at the two exterior frames in planes parallel to the direction of ground motion. Partitions walls do not contribute to the strength and stiffness of the structural systems, but they contribute to the expected damage costs.

<table>
<thead>
<tr>
<th>Element</th>
<th>Damage</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>RC Frame (RCf)</td>
<td>Initial cracking, $\psi_0$</td>
<td>0.005</td>
</tr>
<tr>
<td></td>
<td>Impending collapse, $\psi_u$</td>
<td>0.040</td>
</tr>
<tr>
<td>Partition wall (PW)</td>
<td>0.004</td>
<td>0.008</td>
</tr>
<tr>
<td></td>
<td>Reyes (1999)</td>
<td></td>
</tr>
<tr>
<td>Masonry walls</td>
<td>M2</td>
<td>0.002</td>
</tr>
<tr>
<td></td>
<td>M4</td>
<td>0.010</td>
</tr>
<tr>
<td></td>
<td>Pérez-Martínez &amp; Esteva (2011)</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.7 Expected damage functions of the systems studied, conditional to survival
In Table 4.3, the additional investment needed to replace the partition walls of the BF system by masonry walls (M2 or M4, Table 4.2) is presented for the three cases studied (Fig. 4.1).

<table>
<thead>
<tr>
<th>System</th>
<th>Masonry walls (A)</th>
<th>Partition wall (B)</th>
<th>Cost (C) = (A) + (B)</th>
<th>Additional investment $\Delta i$ = $C_1 - C_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>BF</td>
<td>0</td>
<td>2.40</td>
<td>2.40</td>
<td>0.00</td>
</tr>
<tr>
<td>SS-M2</td>
<td>3.60</td>
<td>0.48</td>
<td>4.08</td>
<td>1.68</td>
</tr>
<tr>
<td>SS-M4</td>
<td>3.78</td>
<td></td>
<td>3.78</td>
<td>1.38</td>
</tr>
<tr>
<td>FI-M2</td>
<td>4.50</td>
<td>0.00</td>
<td>4.50</td>
<td>2.10</td>
</tr>
<tr>
<td>FI-M4</td>
<td>4.73</td>
<td></td>
<td>4.73</td>
<td>2.33</td>
</tr>
</tbody>
</table>

Figure 4.8 shows present values of expected damage costs, determined in accordance with Section 3: (a) conditional to survival, $\Delta S C_0 / \gamma$ and (b) failure, $\Delta F C_0 / \gamma$. Finally, Figure 4.9 shows the life-cycle present values of structural systems: (a) without the additional investment $\Delta i$ (Table 4.3) and (b) including $\Delta i$.

5. CONCLUDING REMARKS

A study is presented about the influence of infill masonry panels on the life-cycle expected performance functions of multi-story reinforced concrete frames exposed to severe seismic hazard conditions. For the cases studied, this influence is summarized in Fig. 4.9b. Similar studies are recommended considering different types of masonry elements, for systems located at sites with different seismic hazard conditions and types of soil.
Quantitative results have been presented about the influence of infill masonry panels on the following concepts:

a) Lateral deformation capacity (Pushover analysis, Sec. 4.2, Fig. 4.3)

b) Peak values of global distortions $\psi$ of detailed models of the systems (Sec. 4.2, Fig. 4.4)

c) Reliability index $\beta$ as function of the normalized intensity measure $Q$ (Sec. 4.2, Fig. 4.5)

d) The natural period of vibration of the system considered and consequently the annual rates of occurrence of earthquakes with intensities greater than $y$ at the site (Sec. 4.3, Fig. 4.6).

Additional results and details about the methodology and the coefficients used in this study are presented by Pérez-Martínez (2010).

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REFERENCES


Liao, H. M. (1964). Studies on Reinforced Concrete Shear Walls and Framed Masonry Shear Walls, Report, Engineering Research Institute, University of Tokyo, 89 pp.


