ASCE 7 R-Values in Soft-Story Building Retrofits

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Abstract

This paper presents the SEAONC Existing Buildings Committee (EBC) efforts to determine whether retrofits using the most stringent (lowest) $R$-value in parallel lateral force resisting systems (LFRS) is necessary to protect life safety. Case study buildings were analyzed representative of buildings in the City of San Francisco’s soft-story program. It was found that retrofits designed with $R$-values associated with wood structural panels (WSP) and steel moment-resisting frames (MF) on independent lines-of-resistance have performance at least equal to that of a code-conforming benchmark retrofit consisting entirely of WSP. Hence, retrofits using the most stringent $R$-value may be considered over-conservative in the context of minimum code requirements. The findings support allowing retrofits using parallel mixed systems each designed to their respective code-specified $R$-value. At the request of San Francisco, the EBC provided language allowing this design approach in San Francisco’s Administrative Bulletin AB-107.

Introduction

Programs to retrofit wood-frame buildings having so-called soft-stories\(^1\) are underway in California. A popular design approach is based on the IEBC Chapter A4, *Earthquake Risk Reduction in Wood-Frame Residential Buildings with Soft, Weak or Open-Front Walls* (ICC 2012). The design force is based on ASCE 7-10 that includes a response modification coefficient ($R$-value) which depends on the seismic force-resisting system (ASCE 2010).

For many buildings, a practical retrofit uses a combination of wood structural panel shear walls (WSP) and steel moment-resisting frames (MF) in the transverse direction at the first story. The MF allows automobile access and parking space that would otherwise be blocked by WSP. WSP and MF have different $R$-values,\(^2\) and for buildings greater than two stories, ASCE 7-10 requires using the most stringent (lowest) $R$-value in parallel systems (Section 12.2.3). A retrofit consisting of both WSP and OMF is required to have the WSP nearly twice as strong as that in a retrofit consisting entirely of WSP. Many engineers question the necessity of this restriction, which often creates problems with the design and placement of stronger WSP, holdowns, and foundations within the building envelope. This paper presents the SEAONC Existing Buildings Committee (EBC) efforts in determining whether using the lowest $R$-value in parallel systems is necessary to protect life safety.

San Francisco Buildings

Many San Francisco neighborhoods constructed in circa 1920 have a signature architectural style with many relatively narrow midblock buildings (Figure 1a). They are typically residential buildings having large open areas on the ground floor for automobile parking and common areas (Figure 1b). To provide access for automobiles, there are large openings on the transverse wall facing the adjacent street. Since the residential units have numerous transverse partition walls compared to the parking level, the ground floors are usually weaker than the upper stories, and hence, many such buildings are soft-story candidates in their transverse directions. In the longitudinal direction, they have long exterior walls with few openings thus providing adequate length for any WSP retrofit that might be necessary.

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\(^1\)The term “soft-story” as defined in the building code indicates a stiffness discontinuity, while the term “weak-story” denotes a strength discontinuity (Table 12.3-2 Vertical Structural Irregularities in ASCE 7-10 (2010)). In practice, buildings with either or both of these irregularities are often referred to as “soft-story” buildings, and that is how the term is used herein.

\(^2\)ASCE 7-10 Table 12.2-1 defines three types of steel moment-resisting frames: Special Moment Frames (SMF, $R = 8$), Intermediate Moment Frames (IMF, 4.5), and Ordinary Moment Frames (OMF, 3.5). WSP has $R = 6.5$. 
The EBC performed two parallel analytical studies. Retrofits consisted of strengthening independent lines-of-resistance (line) in the first story transverse direction of a three-story building. Each study had different analytical approaches. The results from both studies were in reasonable agreement thus providing a sound basis for EBC recommendations.

Numerous incremental dynamic analyses (IDA) using suites of ground motions were performed to estimate median spectral accelerations for various damage states. Short period spectral acceleration was used as the performance measure denoted here as spectral capacity. Spectral capacities of various alternative retrofits were compared to that for a benchmark consisting of an all-WSP base case retrofit designed with \( R = 6.5 \). The benchmark was a code-conforming retrofit meeting IEBC requirement. Alternatives having similar or larger spectral capacities than the benchmark were deemed acceptable.

**First Case Study**

The archetype building retrofit consisted of strengthening three lines-of-resistance in the transverse direction as shown in Figure 2. WSP shear walls were located at the rear and middle, and a steel moment-resisting frame (MF) at the front (except for the all-WSP cases discussed below). The existing longitudinal walls were assumed to have adequate strength and stiffness and therefore were not retrofitted. It was also assumed that the upper stories had stiffness and strength greater than the retrofitted first story such that the building behavior was governed by the first story.

Eight different retrofits were considered. Retrofit design assumed a rigid floor diaphragm. Design force for each line \( (V_d) \) was proportioned according to the relative stiffness of the line as follows.

\[
V_d = \frac{0.75 \times C_i \times W \times \alpha}{R} \\
C_i = 0.67 \times S_{MS} \\
\alpha = \frac{K_i}{K_T}
\]

The parameters are defined in ASCE 7-10. The floor flat seismic weight was 40 psf and the roof flat seismic weight was 30 psf for a total building weight \( W \) of 210 kips (two floors and roof). This load accounted for the floor framing, interior partitions and exterior walls. \( S_{MS} \) is taken as 1.5; typical for central San Francisco. The 0.75 factor is per IEBC Chapter A4. \( K_i \) is the stiffness of the LFRS on line \( i \), and \( K_T \) is the building total lateral stiffness. The design force for each line was about one-third of the total force.

In all retrofit cases, the WSP design was governed by strength and the MF design was governed by ASCE 7-10 drift requirements. The MF were stronger than the minimum ASCE 7-10 force requirements. A drift limit of 2.5% of the story height was typically used. However, this limit was relaxed to 2.8% for the IMF design for this study. In fact, if the limit of 2.5% was used for the IMF design, it would have the exact same design as the OMF. This is because of the similarity of the ratio of \( C_d/R \) for the OMF and IMF which are 0.86 and 0.89, respectively.

Two all-WSP retrofits used WSP on all three lines-of-resistance; each was designed using different \( R \)-values. They were used for comparison purposes since they were not practical due to precluding automobile access through the building front.

WSP shear walls were stud walls with one layer of 15/32” Struct I sheathing and 10d nails (Table 1). The base case
BC1P was designed per code using $R = 6.5$, and BC2P used $R = 3.5$. BC1P was a code-conforming benchmark retrofit meeting IEBC requirements, and BC2P was significantly stronger and would probably never be used in practice. WSP design was based on AWC (2015) provisions.

<table>
<thead>
<tr>
<th>Retrofit Mark</th>
<th>Design Description</th>
<th>Edge Nailing</th>
<th>Governed Design</th>
</tr>
</thead>
<tbody>
<tr>
<td>BC1P</td>
<td>All WSP ($R = 6.5$)</td>
<td>10d @ 6”</td>
<td>Strength</td>
</tr>
<tr>
<td>BC2P</td>
<td>All WSP ($R = 3.5$)</td>
<td>10d @ 4”</td>
<td>Strength</td>
</tr>
</tbody>
</table>

Six different retrofits had combinations of MF and WSP (Table 2). Three different MF types were used: ordinary moment frames (OMF), intermediate moment frames (IMF) and special moment frames (SMF). Each MF type had two designs. The first used a code-conforming $R$-value (most stringent (lowest) value for the LFRS in the first story). The second used $R$-values designated for the individual LFRS on each line-of-resistance. MF were single-bay frames designed according to AISC Specification (2010) and AISC Seismic Provisions (2010).

WSP shear walls were modeled with shear-spring elements in which the lateral force is dependent only on the interstory drift. The elements used the Modified Ibarra-Medina-Krawinkler Deterioration Model with Pinched Hysteretic response (OpenSees ModIMKPinching Material). This model is capable of capturing stiffness and strength deterioration based on the magnitude and number of cycles the element undergoes. The model determines the rate of deterioration based on the hysteretic energy dissipated. The model incorporates a relatively large number of parameters which requires calibration to component tests. One such calibration for WSP is shown in Figure 4 and shows that the model reasonably captures the yield load, peak load and the deterioration.

The computer model (Figure 3) was developed using the OpenSees program (McKenna 1997). The stiff and strong upper stories were assumed rigid relative to the retrofitted first story. Therefore, only the first story was modeled with the total building mass lumped at the second floor.
Figure 4. WSP ModIMKPinching Calibration to Test Data

Single-bay MF were modeled with linear-elastic beam elements and nonlinear rotational springs (Figure 3). The columns were pinned at their base. The rotational springs used the Modified Ibarra-Medina-Krawinkler Deterioration Model with Bilinear Hysteretic Response (OpenSees Bilinear Material). This model captures strength and stiffness deterioration similar to the ModIMKPinching Material, but pinching is not modeled because steel beam elements do not typically display this type of behavior. The model parameters were based on parameters for AISC wide flange shapes developed by Lignos and Krawinkler (2011). Figure 5a shows the cyclic moment-rotation of the W12x30 beam used in the OMF analyses based on the loading protocol in Figure 5b. The backbone is shown as a reference to highlight the cyclic deterioration due to damage.

Figure 5. W12x30 Moment-rotation Behavior under cyclic loading protocol

The longitudinal walls were modeled by four linear-elastic shear-spring elements. Each spring had stiffness of 40 k/in resulting with a building period of 0.37 sec in the longitudinal direction. Rigid diaphragm behavior was modeled by an in-plane floor truss using very stiff truss elements. Two P-delta elements were attached to the in-plane truss at the quarter points of the building. P-delta elements were assigned negative stiffness \((k)\) by the usual technique \((k = -W/2h)\), where \(h\) = story height = 108 in = 9 feet). The mass was lumped at five discrete points on the in-plane truss and were proportioned according to tributary area.

The ultimate strength and fundamental period of each retrofit are presented in Table 3. The ultimate strength ratio is the ratio of the peak strengths at all lines-of-resistance divided by the building weight (210 kips). The strength ratios varied from 0.25 (all-WSP, \(R = 6.5\)) to 0.46 (WSP and IMF, \(R = 4.5\)). The fundamental periods ranged from 0.25 sec (all-WSP, \(R = 3.5\)) to 0.53 sec (WSP and SMF).
The backbone curves for the WSP and each MF type are shown in Figure 6. The MF were designed using their respective R-values as specified by ASCE 7-10. MF strengths varied only by about 15% that was much less than the variation in R-values (greater than a factor of two when going from 3.5 to 8). The relative uniformity in strength was due to the frame members being governed by drift rather than strength requirements. MF were considerably stronger than the WSP in part because drift governed the MF design.

Due to the lack of lateral bracing of the OMF, hardening was not modeled and the post-peak negative slope was set to occur at a smaller drift than those of the WSP, IMF and SMF. This is consistent with the recommendations of ATC-114 (2016). Complete loss of strength of the WSP, IMF and SMF was set to occur at 20% story drift based on judgment.

Figure 7 shows a representative drift response of the retrofit having WSP (designed with $R = 6.5$) and OMF ($R = 3.5$) at shaking intensity causing incipient extensive damage (story drift > 4% in two lines). It can be seen that there is twisting of the building. The WSP at the building rear deflects more than the OMF at front. This is due to the weaker WSP yielding more than the stronger OMF.

Performance was assessed through the use of Incremental Dynamic Analysis (Vamvatsikos and Cornell 2002). In this method, a series of nonlinear dynamic analyses are performed under a suite of ground motions with each ground motion scaled to higher intensity levels until collapse occurs. The analyses start at a low ground motion intensity that does not cause any inelasticity thus producing a set of data points that capture all significant damage states through collapse. Intensity measures (IM) and damage measures (DM) are tracked for each analysis and plotted as an IDA graph set. The IM and DM chosen were the spectral acceleration of the particular ground motion at the fundamental period of the building, and the first story peak transient displacement, respectively. Collapse was defined as the point of dynamic instability at which there is a loss of lateral stiffness such that the building cannot resist the dynamic loading in addition to the P-delta effects. This was signaled in an analysis by a very large displacement.

Equation 1

$$ R = 6.5 \text{ for WSP} \quad R = 4.5 \text{ for IMF} \quad R = 8 \text{ for SMF} \quad R = 3.5 \text{ for OMF} $$

**Table 3. Retrofit Periods and Ultimate Strengths**

<table>
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<tr>
<th>System</th>
<th>Period (sec)</th>
<th>Ultimate Strength Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>WSP (Conservative; $R = 3.5$)</td>
<td>0.38</td>
<td>0.37</td>
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<tr>
<td>WSP (Code; $R = 6.5$)</td>
<td>0.43</td>
<td>0.25</td>
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<td>OMF (Code $R = 3.5$ All Lines)</td>
<td>0.45</td>
<td>0.44</td>
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<tr>
<td>OMF (WSP $R = 6.5$; OMF $R = 3.5$)</td>
<td>0.48</td>
<td>0.35</td>
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<tr>
<td>IMF (Code $R = 4.5$ All Lines)</td>
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<td>0.46</td>
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<tr>
<td>IMF (WSP $R = 6.5$; IMF $R = 4.5$)</td>
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<td>0.33</td>
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<tr>
<td>SMF (Code $R = 6.5$ All Lines)</td>
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<td>0.3</td>
</tr>
<tr>
<td>SMF (WSP $R = 6.5$; SMF $R = 8$)</td>
<td>0.53</td>
<td>0.3</td>
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</table>

Displacement (inch)

<table>
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<th>Displacement (inch)</th>
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<tr>
<td>9</td>
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<td>5.9</td>
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</table>

Figure 6. Element Backbone Curves for different LFRS (excluding P-Delta effects)

Earthquake ground shaking was represented by a suite of 36 far-field earthquake record pairs compiled by Haselton (http://www.csuchico.edu/structural/researchdatabases/gound_motion_sets.shtml). One record from each earthquake record pair was oriented in the building transverse direction. Among other criteria, the records had magnitudes greater than 6.5, peak ground acceleration greater than 0.2g, site classes C and D, and were all either strike-slip and thrust faults. The set included the ground motion pairs used in FEMA P695 (2009). The suite median spectrum shape has reasonable agreement with the ASCE 7-10 design spectrum for a rock site (class B) in San Francisco.

Figure 7. Example drift response of retrofit having WSP and OMF at an incipient extensive damage state

Figure 8 shows the hysteretic response for the retrofits at the building front (OMF) and rear (WSP). It can be seen that prior to yielding of the WSP and OMF, the stiffness of each element is nearly the same. After WSP yielding and prior to OMF yielding, there is a huge disparity in stiffness that causes the observed torsion. At this damage state, the
building front has peak displacement of about 3.2 in indicating limited yielding in the OMF (3% drift).

Figure 8. Example LFRS response of retrofit having WSP and OMF at an incipient extensive damage state

Figure 9 shows a representative drift response of the retrofit having WSP ($R = 6.5$) and OMF (3.5) at shaking intensity incipient to collapse. If the record were scaled slightly larger, building instability would occur and indicated by very large displacements. Similar to the extensive damage example (Figure 7), there is significant torsion for the reasons cited above.

Figure 9. Example drift response of retrofit having WSP and OMF at an incipient collapse condition

The spectral accelerations (IM) and peak transient story drifts (DM) were tracked for each analysis for each ground motion up through collapse. Examples IDA graphs for the retrofit having WSP ($R = 6.5$) and OMF (3.5) are shown in Figure 11. IDA graphs shown are for all 36 ground motions in the suite (one record from each earthquake record pair). Spectral acceleration is that at 0.48 sec period for each record. Story drift is taken as the displacement at line 2 divided by story height of 108 in (9 feet). The point at which an IDA graph becomes horizontal ("flatlines") is defined as collapse. There is a considerable scatter in spectral accelerations causing flatlines across the earthquake suite. The spectral accelerations vary from about 0.25g to 4.6g (see the right side of Figure 11). Hence, a best estimate was used here, and taken as the median (the value in the middle). The best estimate of the spectral acceleration causing collapse is 1.35g, and denoted here as spectral capacity.
Retrofit alternatives designed using mixed $R$-values were designed with $R$-values associated with the respective WSP and MF. For instance, the WSP & OMF alternative had the WSP designed using $R = 6.5$ and the OMF designed using $R = 3.5$. Accordingly, the OMF is stronger than the WSP in this case.

Retrofit alternatives designed using the most stringent $R$-values were those designed with lowest $R$-value associated with WSP and MF per ASCE 7-10 rules. For instance, the WSP & OMF alternative used $R = 3.5$ in the design of both the WSP and OMF. However, in this case the OMF was stronger than the WSP because stronger members were needed to satisfy drift requirements.

The all-WSP benchmark had a spectral capacity of 1.11g, and alternatives having greater spectral capacities were deemed acceptable. All alternatives were acceptable since their spectral capacities were greater than that for the benchmark (spectral capacities range from 1.35g to 1.94g).

Retrofits alternatives using the most stringent $R$-value generally had larger spectral capacities that the same alternative designed using mixed $R$-values. For example, the spectral capacity of the WSP & OMF alternative using the most stringent $R$-value (3.5) was 24% greater than that from the same alternative designed using mixed $R$-values (6.5 and 3.5). Both spectral capacities were greater than that from the base case all-WSP benchmark. Hence, retrofits using the most stringent $R$-value may be considered over-conservative in the context of minimum code requirements.

Also shown in Figure 12 are the results from a methodology (termed SPO2-IDA) that converts a static pushover curve to an IDA curve. SPO2-IDA is based on statistics from numerous analyses of single-degree-of-freedom systems with a wide range of periods and backbone curves ranging from simple to complex (Vanvatsikos and Cornell 2006). The trend from SPO2-IDA agrees favorably with the spectral capacities derived here from numerous IDA analyses. This agreement suggests that SPO2-IDA may be used in lieu of the considerable effort associated with running huge numbers of time history analyses to generate IDA graphs (Figure 11).

Figure 12. Spectral capacities for all eight retrofit cases (collapse damage state)

Second Case Study

Figures 13 and 14 respectively show the archetype building and computer model. The lateral force resisting system (LFRS) was modeled by nonlinear shear-spring elements. WSP elements had realistic generalized lateral-force versus lateral-displacement behavior including pinching hysteretic with cycling (Figure 15). MF elements also had realistic behavior including full hysteresis with cycling (Figure 15). Element ultimate strengths ($V_u$) were computed as follows.

$$V_u = 0.75 \times C_i \times W_0 \times \Omega_0$$

$$C_i = \frac{0.67 \times S_{MS}}{R}$$

The parameters are defined in ASCE 7-10. $W_0$ is the tributary seismic weight taken by the element. $S_{MS}$ is taken as 1.5; typical for central San Francisco. The 0.75 factor is per IEBC Chapter A4. The MF strengths do not account for additional strength often resulting from meeting ASCE 7-10 drift checks. In the First Case Study above, note how meeting the drift checks resulted with the OMF, IMF, and SMF having about the same strength (Figure 6). Hence, the MF strengths used in the Second Case Study might be on the low side when compared to those resulting from situations where drift design governs.
Figure 13. Archetype building of Second Case Study

Figure 14. Computer model idealization

Figure 15. WSP and MF computer models

Figure 16. Element backbone curves for different LFRS including P-delta element effect (for Line A)

ASCE 7-10 assigns different R-values to various LFRS to make the systems have equivalent performance. That is, lower ductility OMF requires greater strength (smaller R) than higher ductility SMF (larger R). To account for this aspect, element ductility was set such that each system would have the same static collapse displacement. That is, the same lateral displacement of 24 inch when zero force occurs under P-delta effects (Figure 16). The 24 inch was based on judgment as the point where the building would collapse under its own weight from P-delta effects.

The upper story elements were set twice as stiff and strong as the all-WSP first story elements to account for the numerous lath-and-plaster cross-walls in residential areas. Governing inelastic actions occur in the first story as a result.

P-delta elements were placed in parallel with each WSP and MF element. They were assigned negative stiffness (k) by the usual technique (k = -W/h, where h = story height = 108 in = 9 feet).

Floor diaphragms were assumed semi-rigid, and modeled by linear-elastic beams continuous over the four lines-of-resistance. They were assumed to be a single layer of straight lumber sheathing, and equivalent beam stiffness properties were estimated using lab tests as a basis (AWC 2015, Bott...
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2005). Sample analyses using stiffer diaphragms did not appreciably change the results (also note the First Case Study above assumed rigid diaphragms). Hence, the semi-rigid lumber diaphragm behaved closer to rigid rather than completely flexible.

The model represents the building in its transverse direction and assumes the LFRS in longitudinal direction has negligible effect on twisting of the building. It turns out that this is a conservative assumption as shown below.

Two damage states were assigned: extensive damage and collapse. A damage state was attained when two or more lines-of-resistance suffered peak transient inter-story drifts > 4% for extensive damage, and > 25% for collapse.

Earthquake ground shaking was represented by a suite of twenty earthquake records. The suite was that from the SAC project formulated to have a 10% probability of exceedance in 50 years at a generic site in Los Angeles (Somerville et al. 1997). The suite median spectrum shape had reasonable agreement with ASCE 7-10 earthquakes for a stiff soil site (class D) in San Francisco.

A best estimate for spectral capacity was taken as the median spectral value at 0.3 sec period of the suite when scaled such that one-half of the records (10 of 20) in the suite caused a particular damage state. This approach is like that used by the FEMA P-695 project for quantification of building seismic performance factors (FEMA 2009).

The all-WSP benchmark retrofit had a building fundamental period of 0.3 sec. Alternative retrofit having combinations of WSP and MF had periods that varied slightly from 0.3 sec.

OMF Retrofit. The retrofit consisted of an OMF on line A and WSP on lines B to D (Figure 13). Two alternatives were considered.

- Retrofit A1 used $R = 6.5$ and 3.5 for the WSP and OMF, respectively (strength = 0.40W). Use of differing $R$-values is not allowed by ASCE 7-10.
- Retrofit A2 used an $R = 3.5$ for both the WSP and OMF (0.65W). This is consistent with ASCE 7-10 (design based on the most stringent $R$-value).

Figure 17 shows example displacement time history responses for the benchmark all-WSP base case ($R = 6.5$) and retrofit A1 (one OMF with $R = 3.5$). The earthquake record was scaled so that the building was in an incipient extensive damage state. The base case has all lines moving in-phase since the retrofits are completely symmetric about the building centerline. In contrast, retrofit A1 has twisting about the front of the building (line A) having the relatively strong OMF that did not yield. Note how the base case displacement pattern is very similar to line C of retrofit A1. In essence, line C moved in about the same way in both retrofits, but retrofit A1 had twisting with line D having the most movement.

![Figure 17. Example time history response of 2nd floor at an incipient extensive damage state](image)

OMF on Line A
(a)
Strong OMF does not yield
(b)
WSP pinched hysteresis

Figure 18 shows the OMF (line A) and WSP (line D) response for retrofit A1. The strong OMF has no yielding whereas the weaker WSP has moderate inelastic action (5.5 inch peak deformation).

![Figure 18. Example OMF and WSP hysteretic response for retrofit A1 at an incipient extensive damage state](image)

The base case and retrofit A1 have virtually the same spectral capacities for both the extensive damage and collapse states. It was therefore deemed acceptable to use the WSP and OMF combination each designed to their respective code-specified $R$-value. Retrofit A2 has spectral capacities more than 50% greater than that for the base case. While the larger capacities

Figure 19 compares the spectral capacities. For instance, the first column in Figure 19a is the spectral capacity (1.36g) of the benchmark all-WSP base case for extensive damage taken as the median value of the earthquake suite when scaled so that one-half of the records (10 of 20) cause the building to have drifts greater than 4% in two or more lines-of-resistance. The all-WSP ultimate lateral strength was 0.35W, where $W =$ total building weight = 212 kips.

The base case and retrofit A1 have virtually the same spectral capacities for both the extensive damage and collapse states. It was therefore deemed acceptable to use the WSP and OMF combination each designed to their respective code-specified $R$-value. Retrofit A2 has spectral capacities more than 50% greater than that for the base case. While the larger capacities
indicate retrofit A2 is more rugged, it represents an unnecessary conservatism within the context of minimum building code requirements.

Two-MF Retrofits. The retrofit consists of MF on lines A and B, and WSP on lines C and D. Two different alternatives were considered.

- Retrofit C1 used WSP (R = 6.5) and SMF (8).
- Retrofit C2 used WSP (6.5) and OMF (3.5).

Both C1 and C2 have larger spectral capacities than the base case (Figure 21). It seems paradoxical that the OMF retrofit (C2) has slightly larger spectral capacities than the SMF retrofit (C1) indicating that the OMF retrofit is more rugged. However, the strength of OMF retrofit (0.50W) is nearly 60% greater than the SMF retrofit (0.32W).

Longitudinal LFRS. The analyses above ignored the possible effect of the longitudinal LFRS in resisting twisting of the building. Considered here was the case assuming the longitudinal LFRS was very stiff and strong so the building cannot twist. No twisting is an upper bound on behavior, whereas ignoring the longitudinal LFRS as in the prior analyses is a lower bound.

- Retrofit A3 used R = 6.5 and 3.5 respectively for the WSP and OMF (same as A1 above), but with the building restrained from twisting.

Figure 22 compares the spectral capacities from the all-WSP base case (BC) and retrofits A1 (free to twist) and A3 (no twisting). The building with a very stiff and strong longitudinal LFRS (A3) has larger spectral capacities compared to that for the building having a very flexible longitudinal LFRS (A1). Hence, ignoring the longitudinal LFRS was a conservative assumption. Note that the First Case Study assumed the longitudinal LFRS as linear-elastic (no yielding) and thus providing considerable resistance to building twisting. This is the likely the main reason the First Case Study showed less twisting than the Second Case Study analyses that assumed no restraint by the longitudinal LFRS (compare Figures 7 and 17).
Figure 22. Effect of building twisting on spectral capacities

OMF Displacements. OMF are best suited to applications requiring limited ductility, and its peak deformations provide indications of ductility demands. This section presents the peak drifts at the spectral capacities for extensive damage (1.36 g) and collapse (1.92 g) in the all-WSP base case (from Figure 19). Recall that retrofit performance at the base case spectral capacities was deemed acceptable. Three alternatives were considered.

- Retrofit A1 above (WSP and OMF).
- Retrofit C2 above (WSP and 2 OMF).
- Retrofit A3 above (same as A1 but with no twisting).

Figure 23 show the median peak drifts in the OMF on line A. For instance, the first column in Figure 23a is the retrofit A1 median peak drift (0.8 inch) from the suite of the earthquake runs when the suite was scaled to the median spectral acceleration for extensive damage in the all-WSP base case (drift > 4% = 4.3 inch).

Retrofit A1 and C1 have twisting of the building and the OMF peak drifts were less than 3 inches (2.3% drift). The yielding of the WSP governs the building performance by effectively limiting the OMF ductility demands when the building twists (Figure 17). Retrofit A3 having no twisting has a peak drift of about 6 inches (5.6% drift) at the spectral acceleration causing collapse (> 25% drift) of the all-WSP base case. The peak drifts are not excessive and should be readily attainable by well-designed OMF meeting the recommendations of AISC Seismic section E1 (AISC 2016). Hence, use of OMF in soft-story retrofits appears viable.

Figure 23. Peak drifts in OMF at spectral capacities from the all-WSP base case

Comparison of Case Studies

The previous sections demonstrated via two parallel studies that retrofits designed with R-values associated with wood structural panels (WSP) and steel moment-resisting frames on independent lines-of-resistance have performance at least equal to that of a code-conforming benchmark retrofit consisting entirely of WSP.

Each study had reasonable modeling assumptions and analysis methods that somewhat differed. As a consequence, there were differences in the spectral capacities between the studies. The key point is that the conclusions from the two case studies were the same even with different approaches.

Differences in modeling assumptions include:

- Building fundamental period
- Influence of the longitudinal LFRS in resisting building twisting
- Component ultimate strengths
- Component ductility as expressed by the rate of decline in strength beyond the peak force (post-peak slope)
- Component cyclic strength degradation (the Second Case Study ignored).

Differences in analysis methods include:

- Use of different earthquake record suites
- The way that spectral capacity was computed (the First Case Study took the spectral value at the building period for each record whereas the Second Case Study took the median of spectral accelerations from the scaled suite).

Additional analyses from the First Case Study suggest that the differences in spectral capacities between the studies were likely affected more from the assumed building fundamental periods than the assumed component ultimate strengths, ductility (post-peak force slopes), and cyclic degradation.
Building ruggedness (shaking intensity causing damage) typically increases with building ultimate strength, and this trend generally held for both case studies. Figure 24 shows the collapse capacities of each retrofit from each study as a function of the retrofit ultimate strengths. The Second Case Study had higher spectral capacities than the First Case Study at a given lateral ultimate strength.

Figure 24. Relationship between collapse capacities and ultimate strength

Figure 25 shows the influence of initial building period on the spectral capacity for collapse for five all-WSP retrofits (based on the computer model in Figure 3). All models had the same yield and ultimate strengths but the initial stiffness was varied to adjust the period. The SPO2-IDA methodology was used to estimate spectral capacities.

The spectral capacities are clearly inversely proportional to the building periods. The period of all the retrofits used in the First Case Study were based on eigenvalue analysis of each retrofitted building with the as-designed member stiffness. These periods ranged from 0.43 sec to 0.53 sec (Table 3). The Second Case Study set the periods a priori to about 0.3 sec. The spectral capacity for collapse for a model with 0.3 sec period was 2.5g while that for a model with 0.5 sec period was 1.5g indicating a strong effect of period on collapse capacity, based on SPO2-IDA methodology.

Figure 25. Relationship between spectral capacities for collapse and building period using SPO2-IDA methodology

Figure 26 illustrates the effect of component modeling on spectral capacity for collapse. Figure 26a shows the hysteretic curve of an all-WSP retrofit without cyclic degradation under a drift loading protocol like that in Figure 5b. This model had a median spectral capacity of 1.49g using IDA with 36 ground motions.

Figure 26b shows the same model with calibrated cyclic degradation, and it had a spectral capacity of 1.36g which is only 9% less than the model without degradation.

The effect of component ductility (post-peak slope) on spectral capacity can be seen by comparing Figures 26a and 26c. The post-peak slope in Figure 26c is steeper (meaning less ductility) than that in Figure 26a, and it had a spectral capacity only 9% less. In fact, the spectral capacities in both Figures 26b and 26c were virtually the same. Their backbone curves degrade to zero strength at about 12% drift. The backbone in Figure 26a degrades to zero strength at 20% drift.
The EBC analyzed case study buildings for a type representative of the majority in San Francisco’s retrofit program. Various alternative retrofits having combinations of WSP and MF were compared to that for a benchmark consisting of an IEBC code-conforming all-WSP base case retrofit ($R = 6.5$). Alternatives having the same or better performance (i.e., larger spectral capacities) than the benchmark were deemed acceptable. Two parallel studies were performed as a check. Key points:

- Retrofits using combinations of WSP and MF performed equal or better than the all-WSP benchmark. For the mixed-$R$ value retrofits, all of the MF were designed using code-specified $R$-values (OMF, IMF, and SMF designed with $R = 3.5$, $4.5$, and $8$, respectively). For the code-conforming retrofits, the MF were designed using the most stringent (lowest) $R$-value of the systems used in the first story (WSP or MF). The First Case Study had MF design controlled by drift requirements whereas the Second Case Study had MF design based entirely on strength requirements.
- Retrofits having MF designed to their respective code-specified $R$-values had relatively small variations in performance. It did not matter much whether OMF, IMF or SMF were used (Figures 12, 19, 20 and 21).
- Differences in the spectral capacities between the two case studies were due to the somewhat different analytical approaches used. However, both studies led to the same conclusions.

**Conclusion**

The EBC study showed that retrofits using the $R$-values associated with WSP and MF on independent lines-of-resistance have performance at least equal to that of a code-conforming retrofit consisting entirely of WSP. Retrofits using the most stringent $R$-value can be over-conservative in the context of minimum code requirements.

The findings support allowing retrofits using parallel mixed systems each designed to their respective code-specified $R$-value. The EBC provided San Francisco with language allowing this design approach under San Francisco Department of Building Inspection’s Administrative Bulletin AB-107.

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References


