A Comparative Assessment of the Collapse Performance of Soft, Weak or Open-Front Wall Woodframe Buildings Retrofitted using Alternative Procedures

Henry V. Burton, PhD., S.E. and Morolake Omoya
University of California
Los Angeles, CA

Aryan Rezaei Rad
Swiss Federal Institute of Technology (EPFL)
Lausanne, Switzerland

Jonathan Buckalew
Nabih Youssef Structural Engineers
Los Angeles, CA

Abstract

The main objective of this study is to conduct a performance-based assessment of a multi-unit residential woodframe buildings with open-front wall construction, to compare the incremental increase in collapse safety that results from applying three alternative retrofit procedures: (1) the basic structural guidelines of the LA Ordinance, (2) the FEMA P-807 guidelines and (3) ASCE 41-13. Three-dimensional structural models of the existing and retrofitted building cases are constructed in OpenSees and subjected to nonlinear static and response history analyses including incremental dynamic analysis to collapse. The building cases are evaluated using the FEMA P-695 guidelines using median collapse capacity as the measure of collapse safety.

The median collapse capacity of the building case retrofitted using the ASCE 41-13 guidelines is 36% higher than the existing building case. 61% of the collapse cases occurred in the transverse direction in the existing building and all in the 1st story. In contrast, the collapse cases occurred equally in the longitudinal and transverse directions and all in the 2nd story for the ASCE 41-13 retrofitted building case. For the structural model that incorporated the LA Ordinance retrofit, the median collapse capacity increased by 14%. The FEMA P807 retrofit had the lowest impact on collapse safety with an 11% increase in the median collapse capacity.

Introduction

Housing plays a primary role in establishing earthquake resilient communities, since schools, businesses, neighborhood districts and cultural establishments all rely on residents having healthy living conditions and remaining in the affected region (Poland, 2009). As such, ensuring adequate seismic performance of residential buildings is an ongoing challenge for many California cities. In Los Angeles, residential construction primarily consists of woodframe buildings (Reitherman et al., 2002). A large number of these buildings have open ground floors to facilitate parking, which creates the presence of a soft and weak first story. Past events such as the 1989 Lomo Prieta and 1994 Northridge earthquakes have underscored the vulnerability of soft-story woodframe buildings to collapse. 49,000 woodframe apartment buildings were damaged during the Northridge earthquake alone, 2/3 of which were soft-story buildings (Los Angeles Times, 2016).

In December 2014, the City of Los Angeles established the Resilience by Design initiative with the goal of enhancing the city's resilience by strengthening its social and economic functions. As part of the initiative, the Mayor's Office worked with various experts to develop tools and strategies that are needed to adapt to and recover from major disruptive events including storms, earthquake and economic recessions (LADBS, 2015). One of those initiatives, which was signed into law on October 9, 2015, mandated the seismic retrofit of for Soft, Weak and Open Front (SWOF) woodframe buildings.

The Los Angeles Department of Building and Safety (LADBS) estimates that there are 13,500 *SWOF* buildings through the city (Los Angeles Times, 2016).

Los Angeles Ordinance Nos. 183893 and 184081 have established mandatory standards for retrofitting existing woodframe buildings with soft, weak or open-front walls. The ordinance defines a soft wall line as an exterior wall line in which the lateral stiffness in a story is less than 70% of the stiffness of the wall above in the direction under consideration. A weak wall line is defined as one in which the strength of the first story wall is less than 80% of the strength of the second story in the direction under consideration. An open-front wall line is an exterior wall line without vertical lateral force resisting elements in the first story such that the diaphragm above cantilevers more than 25% of the distance between lines of lateral resistance from which the cantilever extends. The ordinance applies to existing woodframe buildings in which (1) the permit application for new construction was submitted prior to January 1, 1978 and (2) a soft, weak or open-front wall line exists in the first story. Residential buildings containing three dwelling units or less are exempt. Figure 1 shows some of the more common woodframe apartment building layouts that comprise the inventory of SWOF buildings in LA City, which meet the criteria for retrofit under the ordinance.

LADBS developed a set of structural guidelines that provide the basic engineering requirements for retrofitting SWOF buildings in accordance with the ordinance. In lieu of the requirements provided in the Structural Design Guidelines (LADBS, 2016), alternative retrofit methods that enhance the performance of the entire first story and are at least equivalent to the ordinance requirements are permitted. These alternative retrofit procedures must be based on one of the following documents: ASCE 41-13 Seismic Evaluation and Retrofit of Existing Buildings, FEMA P807 Seismic Evaluation and Retrofit of Multi-Unit Woodframe Buildings with Weak First Stories and Appendix Chapter A4, 2012 International Existing Building Code (IEBC). If either of these three alternative guidelines are used, the entire weak first story must be analyzed and designed.

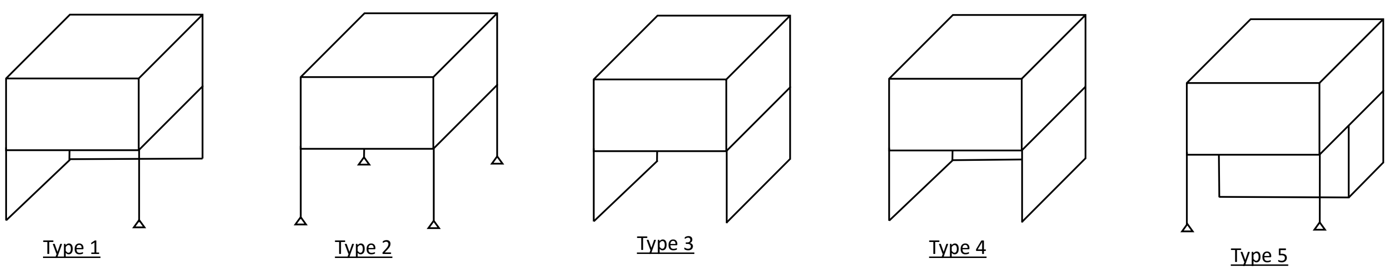


Figure 1 Common SWOF Building Layouts

Currently, there is no available quantitative information on how much these alternative retrofit methods will improve the seismic performance of SWOF buildings. The main objective of this study is to conduct a performance-based assessment of a SWOF woodframe building to quantify the incremental increase in collapse safety that results from applying two of the three alternative retrofit procedures: ASCE 41-13 and FEMA P807.

Engineering Requirements for Retrofitting Soft, Weak or Open-Front Wall Woodframe Buildings

LADBS Structural Design Guidelines

The basic engineering requirements for performing the retrofit of *SWOF* buildings in accordance with the ordinance are outlined in the Structural Design Guidelines prepared by *LADBS* (2016). The design forces used to retrofit the wall line are required to be based on 75% of the design base shear obtained from the ASCE 7-10 standard. The design forces can be obtained from a two-dimensional analysis of the *SWOF* wall line using the equivalent lateral force procedure in ASCE 7-10, Section 12.8. The tributary design base shear for the wall line is determined based on a flexible diaphragm assumption. Steel moment frames, wood structural panels and cantilevered columns are permitted to be used as strengthening elements. In accordance with Table 12.2-1 of ASCE 7-10, (OMFs) are generally not permitted in seismic categories D, E and F and intermediate moment frames are not permitted in seismic design categories D or E. However, in cases where (a) the building height does not exceed 35 feet, (b) the roof and floor dead loads do not exceed 35 psf and (c) the wall dead loads do not exceed 20 psf, steel *OMFs* can be used in seismic design categories D and E and intermediate moment frames can be used in seismic design category F. Concrete and masonry walls and steel braced frames cannot be used. The R-Factor used to design the strengthening elements must be less than or equal to that of the existing lateral force resisting elements in the story above but does not need to be less than 3.5. The story drift limit is based on the smaller of the allowable deformation compatible with all vertical load- resisting elements and 2.5%.

ASCE 41-13

The ordinance requires that the ASCE 41-13 retrofit be based on achieving the Life Safety performance level in the BSE-1E event (20% in 50-year hazard level), which is intended to result in a spectral acceleration level that is approximately 75% the value obtained from ASCE 7-10. However, the strength of the retrofitted story should not exceed 1.3 times the strength of the story above. Four analysis procedures are included in the ASCE 41-13 guidelines: linear static, linear dynamic and nonlinear dynamic. The linear static procedure is consistent with the analysis approach permitted by the ordinance. However, in accordance with Section 7.3.1.2 of ASCE 41-13, the linear static procedure is not permitted to be used for buildings with a vertical stiffness irregularity defined by having an average drift in any story that is more that 150% of that of the story above or below. It is currently unclear whether or not LADBS will permit the linear static procedure for ASCE 41-13 retrofits.

FEMA P-807

The FEMA P-807 retrofit guidelines are based on the statistical evaluation of hundreds of surrogate models that were analyzed using nonlinear response history analysis. It considers the consequence associated with providing "too much" strength in the first story which could led to excessive damage in the upper stories. The performance of the existing and retrofitted buildings is defined in a probabilistic manner (e.g. the probability of exceeding the drift demands associated with the onset of strength loss). According to the *LADBS* Structural Design Guidelines, the spectral acceleration corresponding to $0.5S_{MS}$ per ASCE 7-10 must be used for a FEMA P807 retrofit with the exception of buildings located in site class E, where the value of F_a must be taken as 1.3. The target performance objective must be based on achieving a 20% maximum probability of exceeding drift demands corresponding to the onset of strength loss in the seismic force-resisting wood-frame elements.

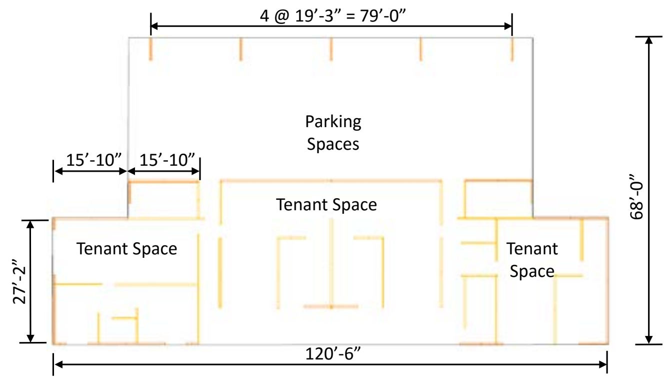
International Existing Building Code (IEBC)

The retrofit procedures in *IEBC* A4 are also based on using 75% of the design base shear determined using the ASCE 7-10 standard. The requirements for the strength reduction factor are also similar to the ordinance requirements. However, the R-Factor based on the strengthening element is permitted provided that the retrofitted structure does not have an extreme weak structure irregularity as defined Table 12.3-2. *IEBC* A4 does not permit vertical elements that are not sheathed structural panels to be considered as providing structural resistance.

Description of Case Study Building and Retrofit Designs

Description of Case Study Building

Three of the four retrofit procedures (*LADBS Structural Design Guideline*, *ASCE 41-13* and *FEMA P-807*), are applied to a pre-1978 3-story apartment building with woodframe construction to compare the increase in collapse safety. The building is located near downtown Los Angeles and has a partially open 1st story to accommodate parking, an irregular plan including re-entrant corners and is closest to the Type 5 configuration shown in Figure 1. Floor plans with the overall dimensions are shown in Figure 2. Each floor, including the roof, has an area of approximately 6,400 square feet. The typical story height is 9'-3". The existing perimeter walls are constructed with stucco on the exterior and gypsum wall board on the interior. All existing interior partitions are constructed with gypsum wall board on both sides. There is an open wall line located along Line A, which serves as the entrance to the parking area on the ground floor. This line consists of perforated (with window openings) walls in the second and third stories and is completely open in the first story, with the exception of four steel pipes supporting the gravity loads in the above floors.



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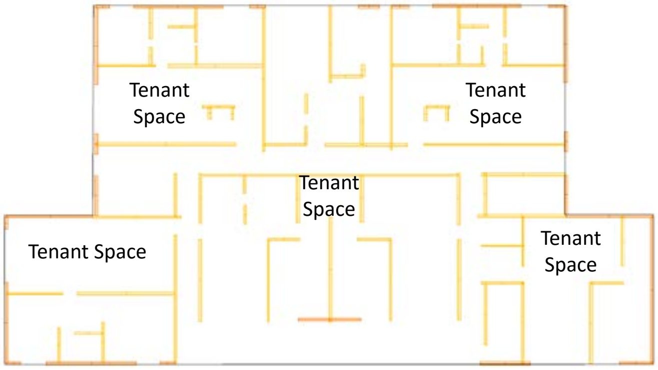


(b)

Figure 2 Plan layout of (a) 1st (ground), (b) 2nd and 3rd floor levels for existing building

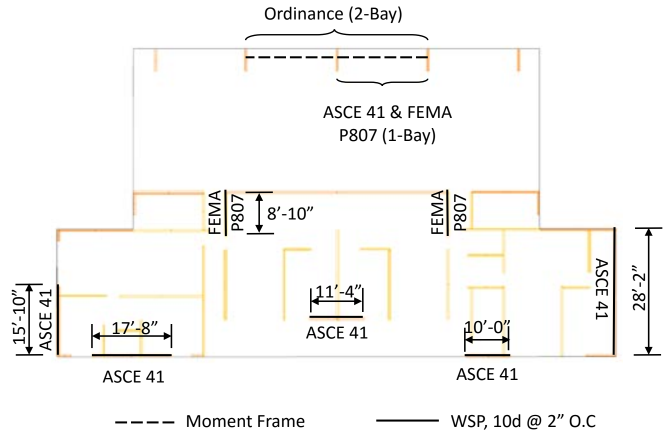


Figure 3 Plan layout of new elements for alternative retrofit schemes

Retrofit Design Per LADBS Structural Design Guidelines

The procedures outlined in the *LADBS Structural Design Guidelines* are applied to retrofit the open wall line along the entrance to the parking spaces. The wall line is retrofitted by adding a 2-bay, steel *OMF* as shown in Figure 3. Based on the location of the building, the mapped spectral acceleration parameters at the short and one-second periods are $S_S = 2.18\ g$ and $S_1 = 0.77\ g$ respectively (ASCE 7-10 Section 11.4.3). The empirical period is computed as 0.34 seconds using Equation 12.8-7 of ASCE 7-10. The building is assumed to be in Risk Category II with an importance factor $I = 1.0$. Assuming soil site class D, the site coefficients F_a and F_v are 1.0 and 1.5 respectively. With the one-second spectral acceleration parameter being greater than $0.75\ g$, the building is assigned to Seismic Design Category E. The seismic response modification coefficient and deflection amplification factors are taken to be $R = 3.5$ and $C_D = 3.0$, which is consistent with stucco and gypsum wall board panels serving as the lateral force resisting elements in the upper stories (LADBS, 2016). Using Equation 12.8-1 of ASCE 7-10, the seismic response (base shear) coefficient is computed to be 0.42 (before the 75% reduction). The seismic weight is computed using 15 psf as the dead load at the floors and roof, 10 psf for the weight of the interior partitions and 15 psf for the exterior wall weight per square foot of wall (LADBS, 2016).

The tributary area for the open wall line is 1,422 ft^2 which corresponds to a seismic weight of 129 kips. The design base shear, after the 75% reduction per the ordinance, is computed to be $V = 40.6\ kips$. A structural model of the 2-bay frame is constructed using ETABS. In addition to the tributary gravity loads, the design base shear is applied to the frame as a point load on the frame. The design forces in the framing elements are based on the load combinations for strength design provided in Section 12.14.2 of ASCE 7-10. *W12 X 96* steel sections are used for the two beams (maximum demand to capacity ratio, DCR_{max} = 0.19) and three columns (maximum demand to capacity ratio, DCR_{max} = 0.29). The maximum frame deflection is computed to be 1.54 inches, which is less than the 2.78 inches allowable deflection corresponding to the 2.5% drift limit.

Retrofit Design Per ASCE 41-13

The *ASCE 41-13* guidelines are used to design a full-story retrofit of the case study building. The retrofit is designed to achieve the Life Safety performance level in the BSE-1E event (20% in 50-year earthquake). For the downtown Los Angeles site, the 20% in 50-year hazard level corresponds to mapped spectral acceleration parameters at the short and one-second periods $S_S = 1.0\ g$ and $S_1 = 0.30\ g$ respectively. The site coefficients F_a and F_v are (from ASCE 41-13 Tables 2-3 and 2-4) are the same as was obtained from ASCE 7-10. From Table 2-5 of ASCE 41-13, the level of seismicity was determined to be "high". The design spectral acceleration obtained from the general spectrum (ASCE 41-13 Section 2.4.1.7) is $Sa_{(BSE,\ IE)} = 1.0\ g$.

The linear static procedure (*LSP*) is used to for the retrofit. In order to compute the pseudo seismic force (ASCE 41-13 Equation 7-21), the modification factor used to relate expected inelastic displacements to elastic displacements is computed to be $C_1 = 1.1$. The modification factor to account for hysteresis shape is calculated using Equation 7-23 to be $C_1 = 1.0$. The effective mass factor to account for higher modal mass participation effects (Table 7-4) $C_m = 1.0$. Given these factors and the seismic weight of the building, $W = 450$ *kips*, the pseudo seismic base shear is computed to be $W = 495$ *kips.*

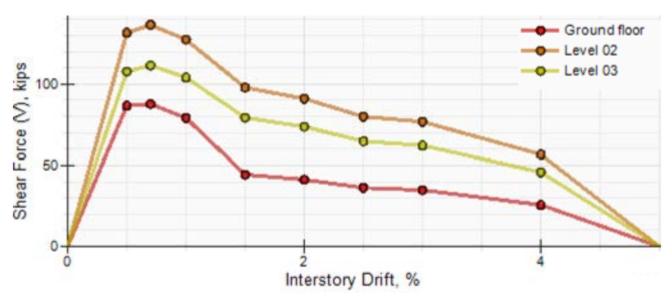
A structural model of the entire building is constructed in ETABS. The wood panels are modeled using elastic link elements. The stiffness of each panel is computed as the yield strength (ASCE 41-13 Table 12-1) divided by the yield displacement (ASCE 41-13, Equation 12-2). The vertical distribution of lateral forces in the building is derived from Equations 7-24 and 7-25 of ASCE 41-13. The lateral forces are applied at each floor level at the center of mass and the gravity loads are applied as area loads. The diaphragm at each of the three floor levels is modeled as rigid. For the existing building, the stucco (exterior) and gypsum board wood panels serve as the primary elements. The component-demand modification factor (*m-factor)* for stucco and gypsum board wood panels are 3.6 and 4.7 respectively for the life-safety limit state. The expected strength (Q_{CE}) is taken to be 350 plf for the perimeter (stucco) panels and 100 plf for the interior (gypsum wall board) panels (ASCE 41-13 Table 12-1). A knowledge factor $\kappa = 0.75$ is used in light of the fact that the default strength and stiffness values are used.

After applying the equivalent lateral forces and gravity loads to the structure, the elastic force demands in each of the wood panels is extracted. As expected, none of the panels in the entire building met the acceptance criteria defined in Equation 7-36 of ASCE 41-13. The ratio of the force demands caused by gravity loads, Q_{UD}, to the expected strength of the wood panels adjusted for ductility and the knowledge factor, $m\kappa Q_{CE}$, ranged from 1.45 to 6.97 in the 1st story. The final retrofit scheme is shown in Figure 3. In the longitudinal direction, a 1-bay *OMF* with a *W12x96* beam and columns is placed along the open wall line at the entrance to the parking spaces and three perimeter walls are replaced with wood structural panels (WSP) with 10d common nails at 2 inches on center. The maximum ratio of Q_{UD} to $m\kappa Q_{CE}$, is 0.4 for the moment frame and 0.93 for the wood structural panels. In the transverse direction, two of the exterior panels are replaced with *WSP* with 10d common nails at 2 inches on center and the maximum ratio of Q_{UD} to $m\kappa Q_{CE}$, is 0.99 for the wood structural panels. To achieve this retrofit scheme, the existing panels were removed from the structural model and their strength was not considered.

Retrofit Design Per FEMA-P807

The FEMA P807 guideline is also used to design a full first-story retrofit of the existing building. A target performance objective of a 20% maximum probability of exceeding drift demands corresponding to the onset of strength loss (*OSL*) for a spectral acceleration corresponding to $0.5S_{MS}$ is used in accordance with the Structural Guidelines (LADBS, 2016). First, the performance of the existing structure is evaluated. The plan layout and seismic weight of the existing building is imported into the FEMA P807 electronic tool including the location of each of the interior and exterior panels. The short period spectral acceleration demand corresponding to $0.5S_{MS}$ is also entered. The nonlinear load-deflection curves for the stucco and gypsum wall board panels are obtained from Table 4-1 of FEMA P807.

The FEMA P807 electronic tool provides the pushover response for individual stories. Note that this is different from doing a building-level pushover response and generating force-displacement curves at each story. The response provided by FEMA P807 is obtained by doing a single-story nonlinear static analysis using the relevant panel layout for that story. The lateral load distribution is based on a rigid diaphragm assumption. The story-shear versus story drift response for the existing building is shown in Figure 4. The terms X- and Z-Direction will be used to describe the longitudinal and transverse directions respectively. It shows that, in the X-Direction, the second story is the strongest of the three with a peak strength of approximately 139 kips. The first story is the weakest of the three with a lateral strength of approximately 88 kips. The second story is also the strongest in the Z-Direction with a lateral strength of approximately 121 kips. The 1st story has a strength of approximately 83 kips in the Z-Direction. The overall ductility of the three stories is the same with a complete loss of lateral strength occurring at 4.5% story drift. The spectral capacity of the existing first story corresponding to a 20% probability of exceeding (*POE*) at the *OSL* limit state is about *0.31 g* in the longitudinal direction and *0.35* in the transverse direction, which is less than $0.5Sms$ (1.1 g) indicating that the existing first story does not meet the FEMA P-807 criterion.



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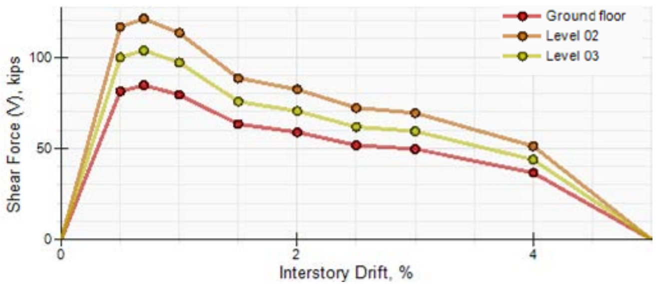


(b)

Figure 4 Story shear force versus interstory drift for (a) X- and (b) Z-Directions

The upper limit on the strength of the retrofitted first story is computed to be 171 kips in the X-Direction and 151 kips in the Z-Direction. However, these strength values are less than what is needed to achieve the 20% *POE* at *OSL*. In light of this result, the "optimized" retrofit solution is adopted for both directions. The resulting retrofit scheme is shown in Figure 3. It includes of a 1-bay *OMF* with W12x53 beams and column placed along the open wall line at the entrance to the parking spaces in the X-Direction and two interior walls are replaced with wood structural panels (WSP) with 10d nails at 2 inches on center. The spectral capacity of the retrofitted first story corresponding to 20% *POE* of the *OSL* limit state is 0.74g and 0.97g for the X- and Z-Directions respectively.

Collapse Performance Assessment of Existing and Retrofitted Buildings

A collapse assessment of the four building cases (existing, Ordinance retrofit, ASCE 41 retrofit, FEMA P-807 retrofit) is performed to estimate the relative improvement in collapse safety of the different retrofit procedures. Collapse vulnerability is quantified using the median collapse capacity. Other metrics such as the conditional probability of collapse and the collapse margin ratio are not considered.

Structural Modeling

Three-dimensional numerical models of the existing and retrofitted buildings cases are developed in *OpenSees*. The wood panels are idealized using a two-node link element with a horizontal spring that captures the force-deformation behavior of the panel (Figure 5). The two nodes are located at the top and bottom of each panel at the mid-span. The SAWS material (Foltz and Filiatrault, 2004) is used to model the nonlinear response of the panels. The hysteretic model is defined by 10 parameters, which are summarized in Table 1 for stucco, gypsum wall-board and wood sheathing panels (WSP) with 10d nails at 2 inches on-center. The *WSP* values are obtained from the CASHEW computer program for cyclic analysis of wood shear walls (Filiatrault (2001) and the gypsum and stucco parameters are from Folz and Filiatrault (2003). The *OMF* beams and columns are modeled using elastic elements with concentrated plastic hinges, which incorporate the Modified Ibarra-Krawinkler deterioration model (Ibarra et al. 2005). The model parameters for the hinges are obtained from the empirical equations developed by Lignos and Krawinkler (2013). A leaning column is placed at the center of mass to incorporate P-Δ effects. A rigid diaphragm constraint was added to each of the three suspended floor levels.

Table 1 SAWS model parameters for wood panels

Material	K_0 (kips/in/ft)	F_0 (kips/ft)	F_1 (kips/ft)	D_u(inches)	r_1	r_2	r_3	r_4	α	β
Stucco	3.62	0.2	0.04	0.67	0.058	-0.050	1.00	0.020	0.6	1.1
Gypsum Wallboard	1.88	0.1	0.02	1.11	0.029	-0.017	1.00	0.005	0.8	1.1
WSP w/ 10d @ 2" O.C.	4.23	2.0	0.25	1.97	0.023	-0.04	1.01	0.01	0.8	1.1

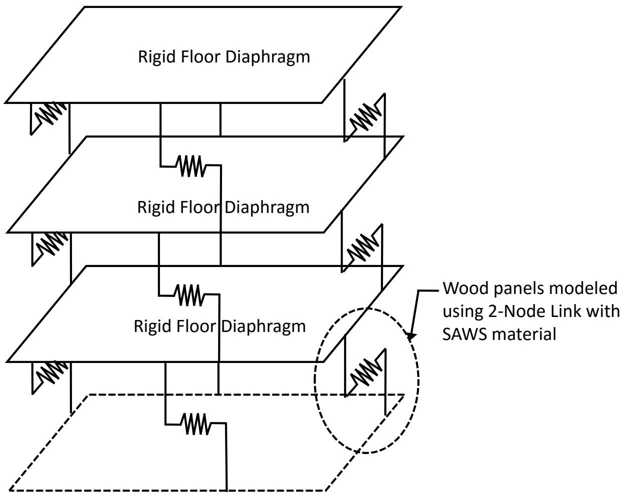
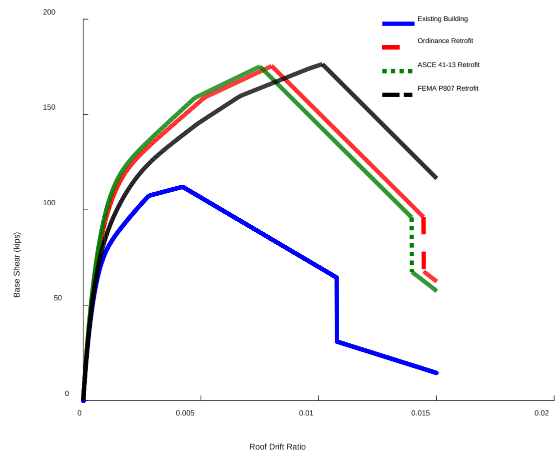


Figure 5 Schematic illustration of structural modeling approach

Nonlinear Static Analysis

Nonlinear static (pushover) analyses are performed on the numerical models of the existing and retrofitted building cases to investigate the general load deflection relationship. The pushover analyses are conducted using the load distribution from ASCE 7-10, Section 12.8-3. The overall base shear versus roof drift for the four building cases are compared in Figure 6. As shown in Figure 6a, the lateral strength in the X-Direction is approximately 112 kips which occurs at a roof drift of approximately 0.4%. At a roof drift of 1.5%, the base shear degrades to about 10% the peak strength. All three retrofit methods increase the lateral strength to approximately 176 kips. For the Ordinance and ASCE 41-13 retrofits, the peak strength occurs at 0.75% drift and for the FEMA P807 retrofit, the peak strength occurs at 1% and degrades at a slower rate than the other two retrofit cases. For example, at 1.5% roof drift, the FEMA P807 retrofit case degrades to 66% of the peak strength whereas the Ordinance and ASCE 41-13 retrofit cases degrade to approximately 30% of the peak strength. Recall that, in the X-Direction, the Ordinance retrofit consists of adding a 2-bay moment frame with *W12 X 96* beams and columns, a 1-bay moment frame with *W12 X 96* beams and columns plus *WSP* panels are used for the ASCE 41-13 and the FEMA P807 retrofit only uses the 1-bay moment frame with *W12 X 53* beams and columns. This suggests that strength provided by the Ordinance and ASCE 41 retrofits are greater than that of FEMA P807. The equal lateral strength of the three cases stems from the fact that strength degradation of the retrofitted cases is controlled by the second story. However, from Figure 7, it can be observed that the FEMA P807 retrofit produces the lowest concentration of drift in the 2nd story and as a result, the most ductile response.

The pushover response for the Z-Direction is shown in Figure 6a. The peak strength for the ASCE 41 and FEMA P807 retrofit cases is approximately 45% higher than the existing case. The peak strength occurs at 0.4% in the existing case and 0.7% in the both the retrofitted cases. Recall that the Z-Direction was not changed by the Ordinance retrofit. Like the longitudinal direction, strength degradation is controlled by the 2nd story for the ASCE 41 retrofit (Figure 8b). However, for the FEMA P807 retrofit (Figure 8c), strength degradation is still controlled by the first story.



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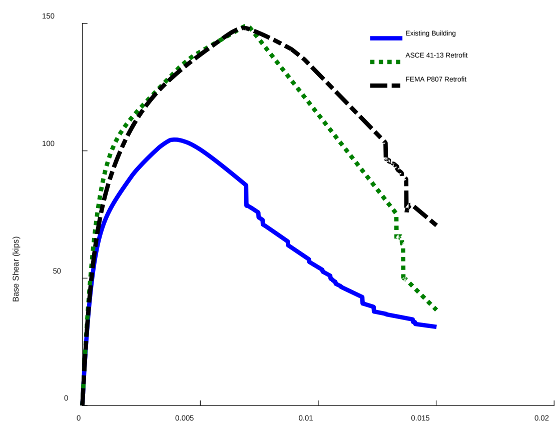
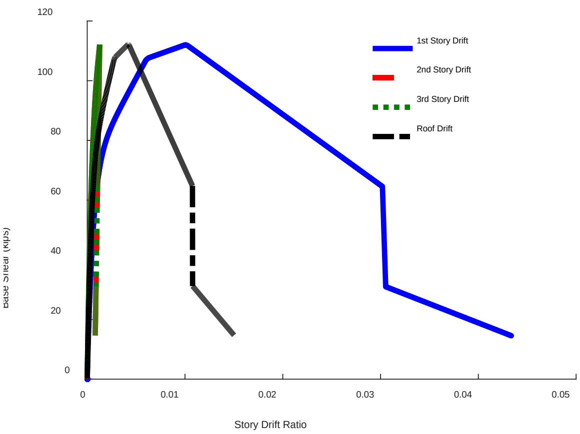


(b)

Figure 6 Pushover response showing base shear versus roof drift for (a) X- and (b) Z-Directions



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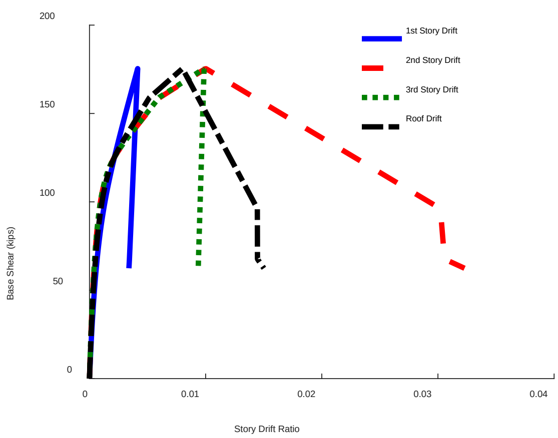

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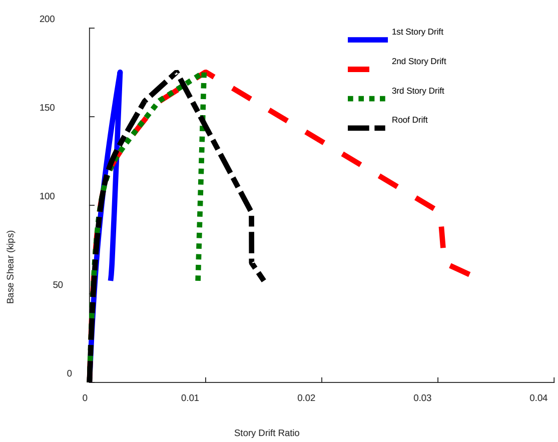

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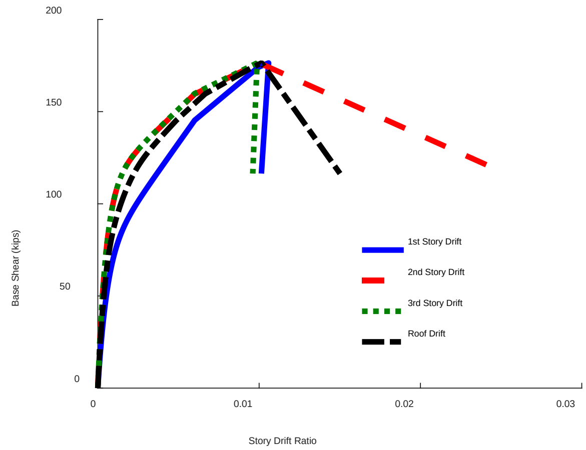
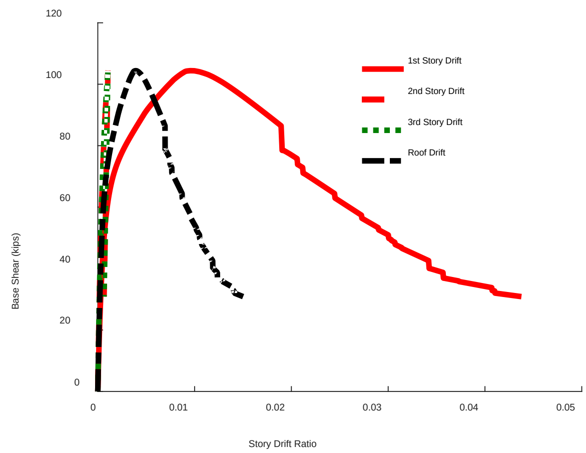


(d)

Figure 7 Pushover response in X-Direction showing base shear versus drift for all stories (a) Existing, (b) Ordinance Retrofit, (c) ASCE 41-13 Retrofit, (d) FEMA P807 Retrofit



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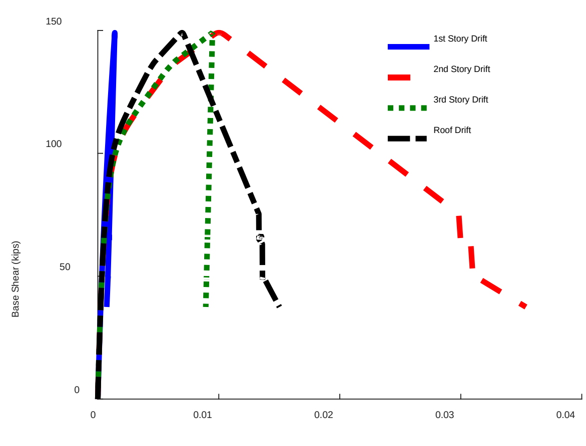

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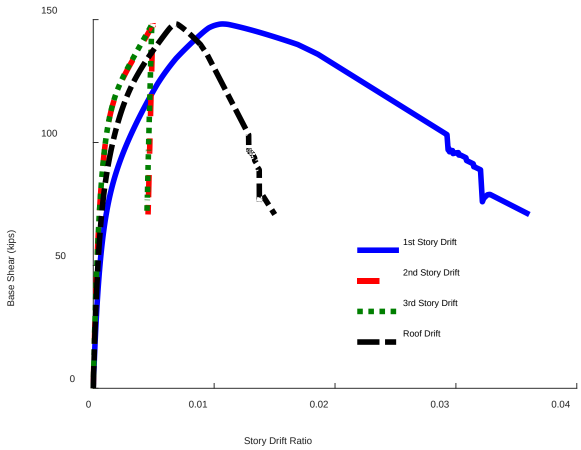


(c)

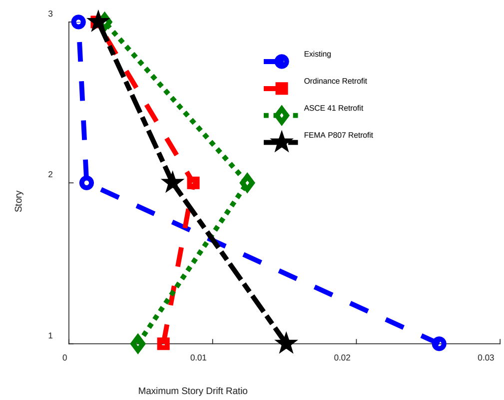
Figure 8 Pushover response in Z-Direction showing base shear versus drift for all stories (a) Existing, (b) ASCE 41-13 Retrofit, (c) FEMA P807 Retrofit

Nonlinear Response History (Pre-Collapse) Analyses

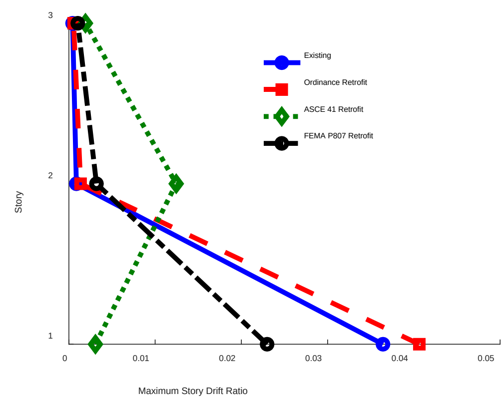
Nonlinear response history analyses were run using the far-field record set of 22 component pairs of the ground motions specified in the FEMA P695 (FEMA 2009) guidelines using bi-directional loading. The ground motions are scaled such that the geometric mean of each ground motion pair matches the target spectral acceleration level. Two analyses are conducted for each ground motion pair by switching the orthogonal direction of each of the motions. As such, a total of 44 analysis cases are performed for each building case. The median maximum story drift profile for the 22 ground motion pairs scaled to 75% of the DBE hazard level is shown in Figure 9.

The median of the maximum drift profile for the X-Direction shows a high concentration of drifts in the 1st story of the existing building. This observation is consistent with the results of the pushover analyses. The median peak drift at the 1st story is approximately 2.5% compared to less than 0.2% for the 2nd and 3rd stories. For all three retrofit cases, the story drift is significantly reduced in the 1st stories and is increased in the 2nd and 3rd stories. For the ASCE 41-13 and Ordinance retrofits, the peak drift occurs in the 2nd story, which is also consistent with the observations in the pushover response. For the FEMA P807 retrofit, the peak drift occurs in the 1st story. However, it is significantly reduced having a median value of 1.75% compared to 2.5% in the existing building.

In the Z-Direction, the median maximum drift at the 1st story for the existing building is 3.6% suggesting that it is the more vulnerable of the two orthogonal directions. The ASCE 41-13 retrofit reduces the 1st story median peak drift to 0.3% and the 2nd story median peak drift is increased from 0.1% to 1.2%. The FEMA P807 retrofit reduces the median peak drift in the 1st story to 2.3%



(a)



(b)

Figure 9 Median story drift profile from nonlinear response history analyses using bi-directional inertial loading at 75% DBE hazard level for (a) X- and (b) Z-Directions

Collapse Performance Assessment

The collapse safety of the four building cases is assessed using incremental dynamic analyses, where each ground motion pair is scaled until the collapse point is reached. The collapse analysis is performed using bi-directional loading where the 22 pairs of ground motions are scaled such that their geometric mean match the target intensity.

Figure 10 shows the collapse fragility curve for the existing building subjecting to bi-directional loading including the effects of the spectral shape factor (*SSF*) and modeling uncertainty (*MU*). After accounting for *SSF* and *MU*, the median collapse capacity is computed to be 0.97g. The collapse fragility curves for all four building cases is shown in Figure 11 and Table 2 shows a summary of the collapse results. Recall that two analysis cases were performed for each ground motion pair using bi-directional loading. For the existing building, 27 of the 44 (61%) analysis cases collapsed in the Z-Direction. Collapse occurred in the 1st story for all 44 of the analysis cases. These two observations are consistent with the results of the nonlinear static and pre-collapse analyses in the previous sections which showed a lower peak strength and the highest drift demands occurring in the Z-Direction and always in the 1st story for the existing building.

Retrofitting the building per the LA Ordinance increases the median collapse capacity by 14%. For this retrofit case, 41 of the 44 (93%) of the analyses cases collapsed in the Z-Direction and all 44 collapses occurred in the 1st story. This is consistent with the fact that the ordinance retrofit only addresses the X-Direction. The FEMA P807 retrofit increases the median collapse capacity by 11% where 31 of the 44 analysis cases (70%) collapsed in the Z-Direction and 40 of the 44 (90%) collapsed in the 1sts story. This is consistent with the results of the pushover and pre-collapse analyses which showed that, for this particular retrofit case, drifts were still concentrated in the 1st story.

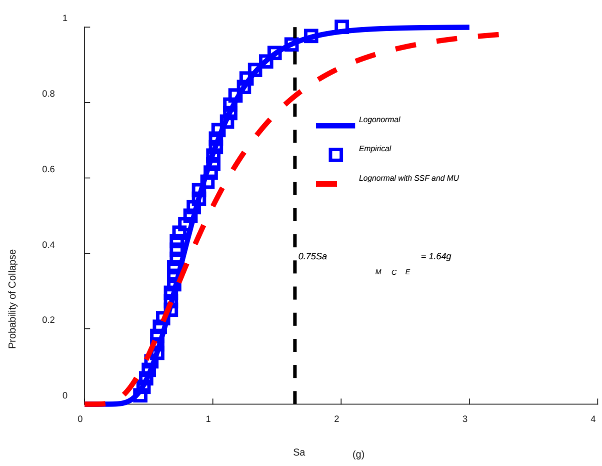


Figure 10 Collapse fragility curve for existing building subjected to bi-directional loading including the effect of the spectral shape factor (SSF) and modeling uncertainty (MU)

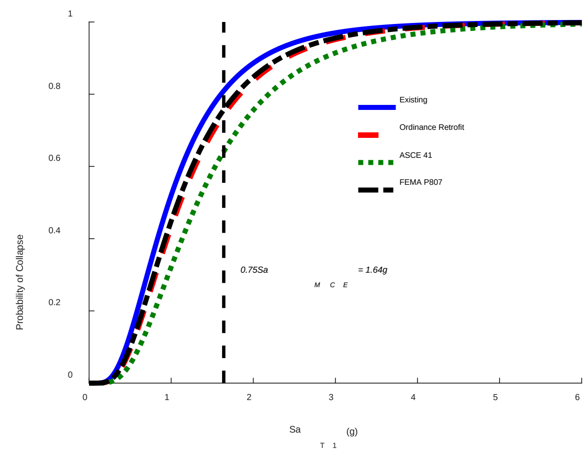


Figure 11 Collapse fragility curve for existing and retrofitted building cases subjected to bi-directional loading including the effect of the *SSF* and *MU*

Table 2 Summary of collapse results for all buildings using bi-directional loading

Building Case	**$Sa_{col,med}$ (g)**	**Collapse Direction**		**Collapsed Story**	
		X	**Z**	**1st**	**2nd**
Existing	0.97	17	27	44	0
Ordinance Retrofit	1.11	3	41	44	0
ASCE 41 Retrofit	1.32	22	22	1	43
FEMA P807 Retrofit	1.08	13	31	40	4

Conclusion

A comparative assessment of the increase in collapse safety of an existing pre-1978 multi-family residential woodframe building retrofitted using three alternative procedures is presented in this study. The building is retrofitted using the Structural Guidelines that provide the basic engineering requirements for the LA Ordinance (wall-line retrofit), the ASCE 41-13 guidelines for Seismic Evaluation and Retrofit of Existing Buildings and the FEMA P807 guidelines for Seismic Evaluation and Retrofit of Multi-Unit Woodframe Buildings with Weak First Stories. The three sets of procedures are permitted to be used for retrofitting *SWOF* buildings per the LA Ordinance.

The LA Ordinance retrofit comprised of adding a 2-bay *OMF* in the longitudinal direction. No modifications were made in the transverse direction for the LA Ordinance retrofit. For the ASCE 41-13 retrofit in the longitudinal direction, a 1-bay *OMF* was added and three perimeter walls using *WSP* with 10d common nails at 2” O.C. were replaced. In the transverse direction, two exterior walls were replaced with *WSP* with 10d common nails at 2” O.C. For the FEMA P807 retrofit, a 1-bay

OMF was added in the longitudinal direction with no modifications to the wall panels. In the transverse direction, two interior walls were replaced with *WSP* with 10d nails at 2" O.C.

Three-dimensional structural models of the four building cases (existing plus three retrofit cases) were constructed in OpenSees. The wall panels were modeled using a 2-node link element with a horizontal spring containing the SAWS material model to capture the nonlinear behavior of the wall panels. A rigid diaphragm assumption was used at the suspended floor levels and a leaning column was placed at the center of mass to include P-Δ effects. The structural models were subjected to nonlinear static and dynamic analyses. The nonlinear dynamic analyses were performed by applying bi-directional inertial loading using the 22 pairs of far field ground motions from the FEMA P695 guidelines.

The nonlinear response history analysis case in which the ground motions were scaled to 75% of the DBE hazard level produced a median maximum story drift demand of 3.6% in the transverse direction and 2.5% in the longitudinal direction for the existing building. In both cases, the story drifts were concentrated in the 1st story. In the longitudinal direction, the LA Ordinance and ASCE 41-13 retrofits significantly reduced the 1st story drifts (by a factor of about 5) to the extent that the peak drifts occurred in the 2nd story. This also occurred in the transverse direction for the ASCE 41-13 retrofit. For the FEMA P807 retrofit, the 1st story drift was also reduced but the peak drift of the retrofitted building still occurred in the first story.

The ASCE 41-13 retrofit resulted in the most significant improvement in collapse safety, resulting in a 36% increase in the median collapse capacity compared to 14% and 11% for the LA Ordinance and FEMA P-807 retrofit methods respectively. For the existing building 61% of the collapse cases occurred in the transverse direction and all in the 1st story. In contrast, for the ASCE 41 retrofit, 50% of the collapse cases occur in the transverse direction and over 90% in the 2nd story.

Acknowledgements

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