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SEISMIC PERFORMANCE ASSESSMENT OF WOOD-FRAME SHEAR WALL STRUCTURES SUBJECTED TO GROUND MOTION VARIATIONS

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Abstract

Wood-frame shear wall buildings are commonly used for residential and non-residential buildings. Reliable prediction of the seismic behavior of this structural system requires an understanding of the variability in the seismic performance from the inherent randomness associated with the prediction of seismic demand from ground motions. This paper assesses the influence of certain characteristic features of selected earthquake ground motions including the source-to-site distance (far-field and near-field ground motions), scaling methods (amplitude scaling and spectrum matching) and ground motion response history amplitude features (peak ground acceleration and velocity) on the computed seismic behavior, up to collapse, of wood-frame shear wall buildings. To capture the influence of these ground motion variations in the seismic performance of wood-frame shear wall buildings, a sample set of four building models with different design features were selected from those that were originally studied in the FEMA P-695 report. Incremental nonlinear dynamic analysis is performed on the selected building models using the FEMA P-695 ground motion record sets. The results from dynamic analysis is used to compute the median drift and base shear demands at design spectral intensities and collapse margin ratio (CMR) at collapse level spectral intensities. These metrics are used to compare the performance of the selected building models.

Keywords: Wood-frame shear wall buildings, Ground motion variations, Seismic performance assessment

1. Introduction

Wood-frame shear wall structures are widely used for commercial and residential building applications and have traditionally performed well during earthquakes. These buildings have a structural framing system to carry gravity loads, and wood shear walls to resist lateral loads. Horizontal or sloped diaphragms distribute the lateral loads to the shear walls. Wood-frame shear walls consist primarily of wall framing studs designed to carry the gravity loads and sheathing (oriented strand board, OSB or plywood) that is nailed to the wall framing studs and plates to resist lateral loads including earthquake and wind loads. The walls are connected to the foundation anchor plates using mechanical hold down devices and anchor bolts. When subjected to an earthquake ground motion, the lateral load resisting shear wall system dissipates energy through inelastic behavior in the nailed sheathing to framing connections, other connections to wall framing members such as provided by anchor bolts and hold down devices and wood compression deformation.

The seismic design of wood-frame shear wall buildings is guided by building codes, which provide requirements for the selection of the structural components based on forces and displacements that are computed from a linear equivalent load analysis or from a linear dynamic analysis (response history analysis or response spectrum analysis) of the building models. Where linear or nonlinear response history analysis is used (which is rare in design-based analysis) the seismic demand on any typical structure is largely dependent on the ground motion characteristics and the various methods employed in the selection and scaling of these ground motions. The seismic demand from a typical ground motion varies based on the magnitude of the earthquake event; the fault mechanism; the strike, dip and rake angle; the source type; site conditions; source-to-site distances; duration; pulse characteristics; shear wave velocity of the supporting soil. The methods employed to adjust the seismic demand of a specific ground motion to the design demand also contributes to the variability in the structural response. This inherent variability should be considered in prediction of the structural performance, and if necessary, inform future adjustments in the analysis and design process.

The main objective of the study presented in this paper is to evaluate the sensitivity of the seismic response of wood-frame shear wall buildings to a variety of ground motion characteristics. The seismic performance is assessed at two levels: maximum considered earthquake (MCE) level analysis and collapse level analysis. The MCE level analysis evaluates the building performance based on the median drift and base shear demands from response history analysis of wood-frame shear wall buildings with ground motions scaled to the MCE adjusted target spectrum from Chapter 11 of ASCE 7-05 [1]. The collapse level analysis evaluates the building response based on the FEMA P-695 seismic response evaluation metrics [2]. These metrics include median collapse spectral acceleration ($\hat{S}_{CT}$), collapse margin ratio (CMR) and standard deviation of the individual collapse spectral intensities (β_{RTR}) based on the results from incremental dynamic analysis. The evaluation metrics $\hat{S}_{CT}$ and β_{RTR} is also used to develop fragility curves which can give the probability of collapse at a specific level of spectral acceleration.

The ground motions used for the seismic performance evaluation are the same as those specified for the FEMA P-695 Methodology. Among the various characteristic features of the ground motions, this study is specifically focused on evaluating the seismic performance based on ground motion source-to-site distances (far-field and near-field ground motions) and the correlations of the seismic collapse spectral accelerations to amplitude features (peak ground acceleration and velocity) of the recorded ground motions. Another topic of research in this paper is the evaluation of seismic performance with ground motion adjustment methods (amplitude scaling and spectrum matching).

For analytical evaluation of the seismic response at different levels of spectral acceleration, a set of building designs were selected and analytical models were developed in OpenSees [3]. The selected models include three story-models with low and high aspect ratio shear walls designed for seismic design category (SDC) D_{max} and D_{min}. These buildings are a subset from a group of 16 models that were originally used as examples for demonstrating the FEMA P-695 methodology. Performance evaluation metrics are computed by nonlinear dynamic analysis using a single scale factor at MCE level and incremental dynamic analysis (IDA) at collapse level of spectral accelerations for ground motions.

The results from the analysis using amplitude scaled and spectrum matched ground motions indicated a conservative bias in the collapse capacity and seismic demand using spectrum matched ground motions. Based on source-to-site distance characteristics of the ground motions, the collapse capacity of near-field ground motions is lower than far-field ground motions. A correlation study of the collapse capacity to the peak amplitude features indicated the significance of including peak ground acceleration and velocity factors in the selection of the ground motions for performance assessment.

2. Overview of Building Models

To assess the seismic performance of wood-frame shear wall structures to various ground motion characteristics, a set of building models are selected. These building models (also referred as index models (IM)) are originally designed as part of the ATC 63 project for the development of FEMA P-695 methodology and determination of seismic design coefficients. The building designs are unchanged in this study and within FEMA P-695 are described as complaint with the requirements of ASCE 7-05 [1] and SDPWS-2008 [4].

2.1 Building Model Design Features

The selected models are three-story buildings with a story height of 10 ft. with low and high aspect ratio shear walls. The aspect ratio of the shear wall is the ratio of the height to the width of the shear wall within one story. As per the design provisions of SDPWS-2008, high aspect ratio models are designed by applying a strength capacity reduction factor to account for reduced stiffness. The building designs used oriented strand board (OSB) and plywood as the sheathing material. Panel-to-stud fasteners are 8d and 10d common nails with a nail spacing ranging between 2″ to 6″. Table 1 summarizes the typical design features of the selected models. Figure 1 shows the elevation view of the selected building models including the details about the configuration and position of wood shear walls.

Table 1 – Design features of selected building models

Index Model	No. of Stories	Building Type	No. of Shear Walls per Story	Shear Wall Aspect Ratio	Shear Wall Length (ft.)	Seismic Design Category	Period (C_uT_a) (sec/cycle)	MCE Level Spectral Acceleration (S_{MT} , g)
IM 9	3	Commercial	3	Low (1.11)	10	D_{max}	0.36	1.5
IM 10	3	Residential	5	High (3.33)	3	D_{max}	0.36	1.5
IM 11	3	Commercial	2	Low (1.71)	7	D_{min}	0.41	0.75
IM 12	3	Residential	3	High (3.33)	3	D_{min}	0.41	0.75

2.2 Analytical Modeling of Wood-Frame Shear Wall Building Designs

To perform nonlinear dynamic analysis, the baseline analytical model of the building designs was developed in OpenSees to replicate as much as possible the assumptions utilized in the FEMA P-695 wood-frame example. The hysteresis behavior of the shear walls is modeled using the wood shear wall model CASHEW, developed as part of CUREE Caltech Woodframe Project [5, 6]. Figure 1 (left) shows a typical wood shear wall assembly and the different components that contribute to the lateral strength and stiffness. The seismic energy is dissipated mainly through the deformations in the nailing connectors that are produced by the displacement of the sheathing panels relative to the wall framing (e.g. studs and plates). The in-plane individual shear deformations in the sheathing panel and axial deformations in the framing members are assumed to be small [5]. This CASHEW model accurately represents the pinching, stiffness and strength degradation properties of the hysteresis behavior of wood shear wall as a function of the sheathing panel geometry, sheathing panel material and thickness, nail size and nail spacing.

At the building level, all the floor and roof diaphragms are assumed to have infinite in-plane stiffness and thus the response of the building can be defined with three degrees of freedom (two translational and one in-plane rotation) per floor level [6]. As the selected building model designs in this study are symmetric and regular, the 3D building model is simplified to a 2D planar model and thus the number of degrees of freedom per floor level can be reduced to just the lateral deflection of the shear wall as shown in Figure 2 (right).

In OpenSees the vertical shear walls are modeled using zero-length shear spring elements assigned with the CASHEW material model (named as SAWS in OpenSees material database) connected using two double nodes at each floor level. The total mass of each of the story level is assigned at the top node of the floor level. To be consistent with the FEMA P-695 wood-frame example, the nonlinear dynamic analysis is based on Rayleigh damping model proportional to mass and initial stiffness of the structure. The proportionality constants for the damping model are evaluated by assigning 1% critical damping to first two modes of the structure. A study on effects of these damping assumptions on seismic behavior of wood-frame shear wall buildings can be found in [7].

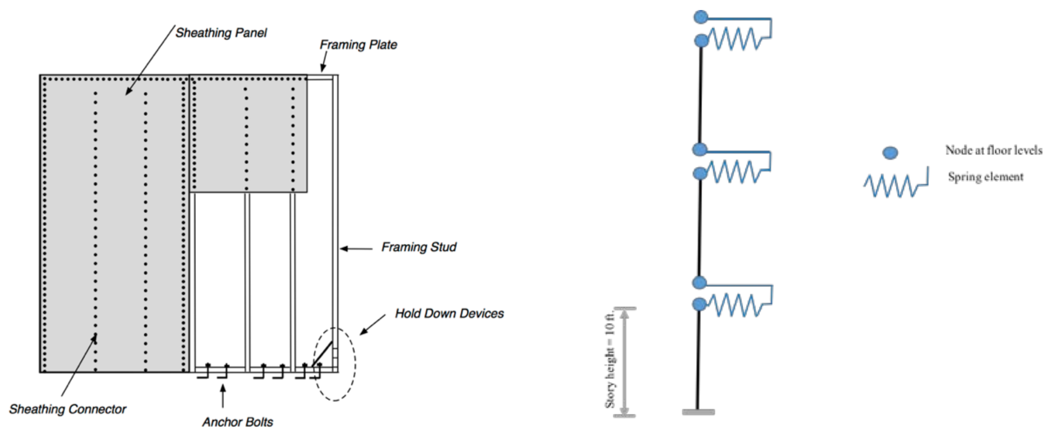


Fig. 1 – Components of a typical shear wall (left) and analytical representation in OpenSees (right)

3. Seismic Performance Evaluation Methods

The seismic performance of the wood-frame shear wall building structures to different ground motion characteristics are evaluated at the maximum considered earthquake (MCE) level and at the collapse level of spectral intensities using ground motions selected from FEMA P-695. The design spectrum is developed based on ASCE 7-05 for seismic design category (SDC) D_{max} (for IM 9 and IM 10 using with S_{MS} = 1.5g and S_{M1} = 0.9g) and D_{min} (for IM 11 and IM 12 using S_{MS} = 0.75g and S_{M1} = 0.3g).

At the MCE level of analysis, the selected set of ground motions from the FEMA P-695 wood-frame example was first normalized based on the peak ground velocity of the record set and then the normalized records are collectively anchored to the design response spectrum at the fundamental period ($T = C_u T_a$) computed based on ASCE 7-05. These ground motions are later used for performing nonlinear dynamic analysis for the building models. For each ground motion the maximum story drift and base shear was recorded and the median of these results for the entire ground motion set is reported.

For collapse level of analysis, the normalized and anchored ground motions are further used in incremental dynamic analysis to study the building response up to collapse level spectral intensities. To be consistent with the FEMA P-695 wood-frame example, collapse in wood shear wall building is defined when the maximum interstory drift exceeds 7% (non-simulated collapse). This limit was originally selected based on previous experimental studies and considered to be mostly independent of the building and shear wall configurations [8]. The spectral intensity that causes collapse in the building model is recorded for each ground motion and the median collapse spectral intensity ($\hat{S}_{CT}$) where half of the ground motions produced collapse is recorded. Using this information, the CMR value, which represents the factor that the design spectral acceleration is multiplied by to cause collapse, is evaluated. This is a convenient measure to compare the collapse capacities of different building models. The probability of collapse is represented using fragility curves.

These curves show the lognormal distribution of the collapse spectral intensity based on the median collapse spectral intensity and the standard deviation of the individual collapse spectral intensities (β_{RTR}) from the median value. Adjustment of the fragility curve for uncertainties related to the quality of analytical assumptions, design procedures, and testing is not included in the fragility curves presented herein.

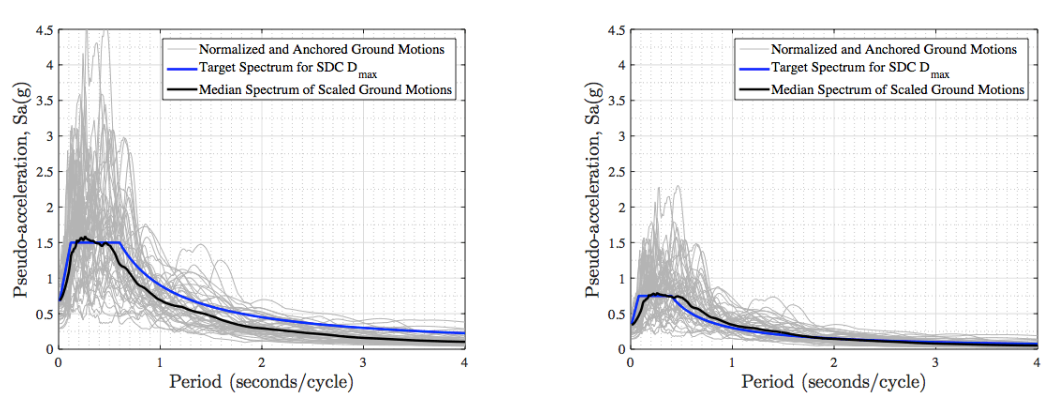
4. Seismic Performance Assessment with Ground Motion Variations

As explained in the introduction, variability of input ground motion characteristics contributes greatly to the variations in the seismic performance assessment of structural systems. The following sections describes the variations in seismic response to three main attribute of input ground motions: modification-to-target procedures, source-to-site distance of records and peak ground acceleration and velocity.

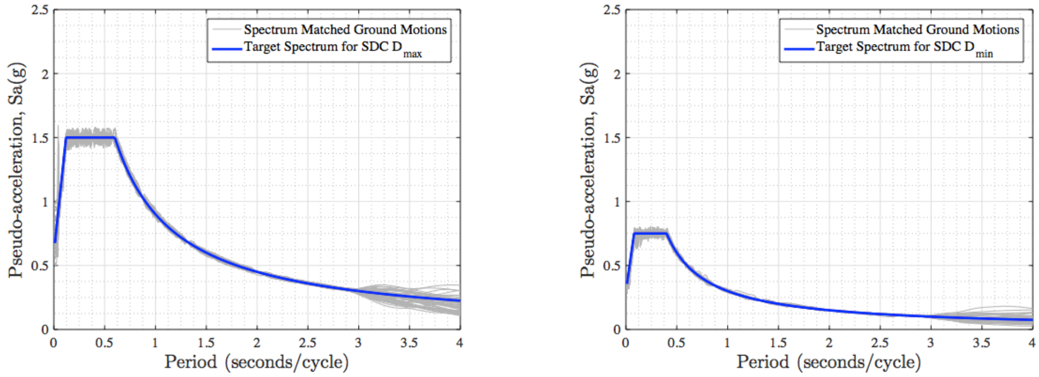
4.1 Ground Motion Modification Procedures

Appropriate selection and scaling of ground motions for response history analysis is critical to predict the expected seismic response of building structures. The current state of practice of nonlinear analysis (ASCE 7-16 [8], ASCE 41 [9], NEHRP recommended provisions [10]) requires the selection of appropriate ground motions that best represents the characteristics of an expected earthquake event including magnitude, fault distance and source mechanism. These selected ground motions should be suitably modified to match the spectral demands based on a target design response spectrum. This modification can be either done by amplitude scaling or by spectrum matching [11]. In amplitude scaling, the ground motion history is scaled by a constant factor so that at the fundamental period the 5% damped pseudo spectral acceleration of the ground motion matches with the design spectral acceleration [1]. Alternatively, in spectrum matching the frequency content of the ground motion is varied so that the response spectrum of the ground motion is tightly matched to the target design spectrum at a range of periods. Spectrum matching reduces the variability of the selected set of ground motions, which results in reduced variability in computed response (relative to analysis performed with amplitude scaled records) [12].

To study the seismic performance variations with ground motion modification procedures, nonlinear dynamic analysis is performed on the selected building models using amplitude scaled and spectrum matched far-field ground motion records from FEMA P-695. The ground motions are spectrum matched using Spectrum Match Toolkit [13] which is based on RspMatch09 program [14]. The RspMatch program alters the original time series by adding wavelet functions so that the new modified time series generates a response spectrum that is compatible with the design spectrum. Figure 3 shows the amplitude scaled and spectrum matched response spectra of the FEMA P-695 far-field set of ground motions, scaled and matched to target spectrum for SDC D_{max} and D_{min}. For amplitude scaling, the median response spectrum of the normalized ground motions is matched to the design spectrum at the fundamental period ($T = C_u T_a$). In spectrum matching, the suite of ground motions is matched between T = 0.01 sec to 3 sec allowing a tolerance limit of 5% using a maximum of 500 iterations to reach the tolerance levels. The 3 second upper limit on the matching period accommodates softening of the system due to inelastic behavior.



(a) Amplitude matched to SDC D_{max} (b) Amplitude matched to SDC D_{min}



(c) Spectrum matched to SDC D_{max} (d) Spectrum matched to SDC D_{min}

Fig. 2 – Response spectrum of amplitude and spectrum matched ground motions matched to SDC D_{max} and D_{min}

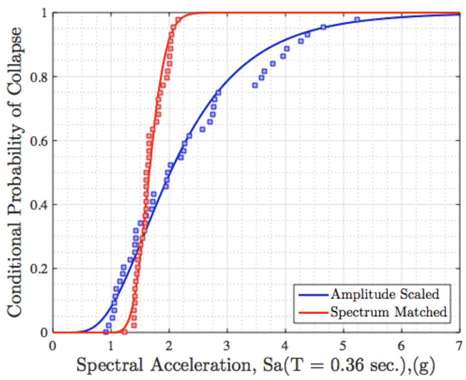
The results from nonlinear response history analysis of building models to amplitude scaled and spectrum matched ground motions are presented in Table 2. At collapse level, the spectrum matched ground motions produced a conservative bias in the seismic response and reduced the predicted collapse capacity of the building model as compared with amplitude scaled records except for the index model IM 12. The dispersion of the results for collapse spectral intensities is considerably reduced with spectrum matched records. A similar trend is also observed with the results from MCE level analysis where the median of maximum interstory drift and base shear using spectrum matched ground motions are lower compared to the results with amplitude scaled records. It is also observed that the number of iterations and tolerance level has significance in the collapse analysis results.

Another interesting point to notice here is the influence of the fundamental period of the structure and the range of matching periods on the predicted response. It is perceived from Figure 2 that there is less variability in the response spectrum of amplitude scaled ground motions at periods greater than 2 sec. and a greater variability is observed at short period (less than 1 sec). Thus spectral matching predominantly alters the frequency content at the lower periods than at the higher periods. As the fundamental period of the selected models is 0.36 sec. and 0.41 sec., this greater variability in the ground motion spectral intensities is reflected in the computed collapse spectral intensities and eventually CMR also. Additionally, it should be noticed that the results for amplitude scaled ground motions presented in Table 2 is evaluated by scaling the ground motions to the fundamental period derived using FEMA P-695 ($T = C_u T_a$). The collapse analysis results can vary significantly if the ground motions are scaled with respect to the fundamental period derived from modal analysis.

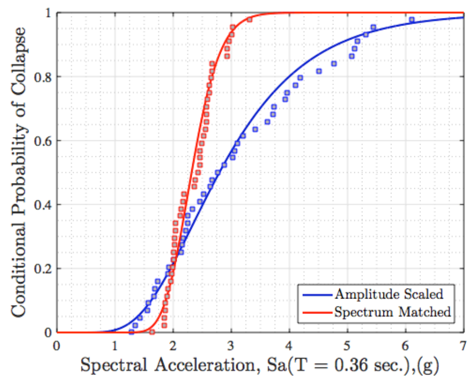
Table 2 – Collapse and MCE level analysis results using amplitude scaled and response spectrum matched ground motions

Index Model	Collapse Level Analysis						MCE Level Analysis			
	Amplitude Scale			Spectrum Match			Amplitude Scale		Spectrum Match	
	$\hat{S}_{CT}$ (g)	CMR	β_{RTR}	$\hat{S}_{CT}$ (g)	CMR	β_{RTR}	Median Drift	Median Base Shear (kips)	Median Drift	Median Base Shear (kips)
IM 9	2.02	1.35	0.50	1.66	1.11	0.14	0.0353	65.4	0.0281	42.9
IM 10	2.80	1.86	0.43	2.32	1.55	0.17	0.0182	35.4	0.0147	24.9
IM 11	1.78	2.37	0.55	1.76	2.35	0.23	0.0162	25.5	0.009	18.0
IM 12	2.26	3.02	0.56	2.47	3.30	0.28	0.0110	13.8	0.0067	9.91

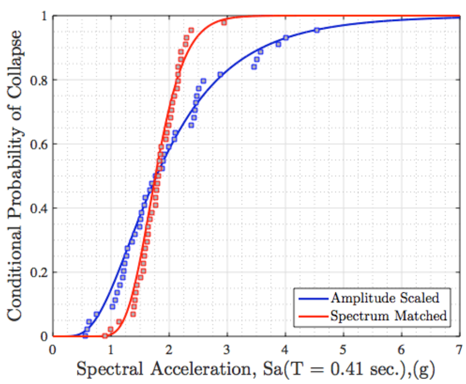
Figure 3 gives the fragility curves of index models developed based on the collapse spectral accelerations. These fragility curves helps to understand the probability of collapse at different levels of spectral intensities and the distribution of these around the median collapse spectral intensities. From the fragility plots, it is inferred that the variability of the individual ground motion collapse spectral intensities from the median collapse spectral intensity is less for spectrum matched ground motions as compared to the amplitude scaled records.



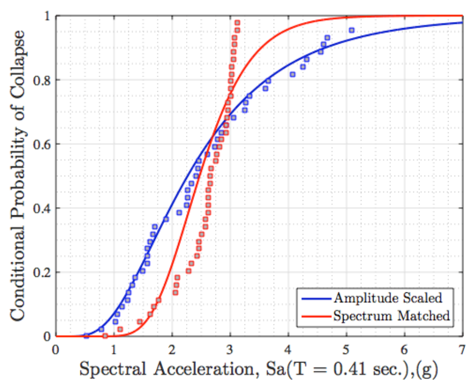
(a) IM 9



(b) IM 10



(c) IM 11



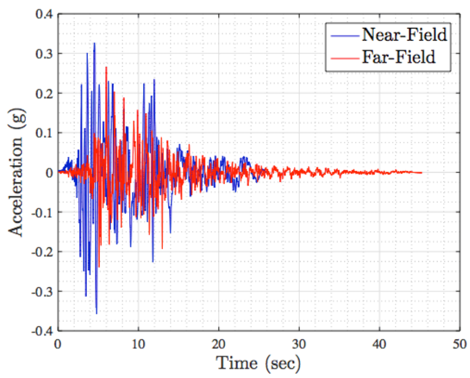
(d) IM 12

Fig. 3 – Fragility plots for index models with amplitude scaled and response spectrum matched ground motions

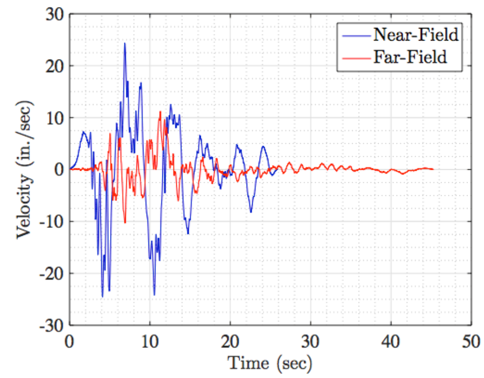
4.2 Ground Motion Source to Site Characteristics

One of the important characteristics of ground motion records considered in this study is the effect of record source-to-site distance in the collapse spectral intensities, specifically the implications of near-fault ground motions. These ground motions are recorded at a relatively short distance (normally less than 10 km) from a fault rupture line. The near-fault ground motions can sometimes have a large pulse with a large concentration of seismic energy near the beginning of the velocity time history and when subjected to a building structure it can significantly reduce the collapse capacity of a structure [15, 16]. This pulse type motion occurs when a fault rupture propagates towards a site at a velocity close to the shear wave velocity of the soil type and thus the major share of seismic energy reaches the site in a short time [17].

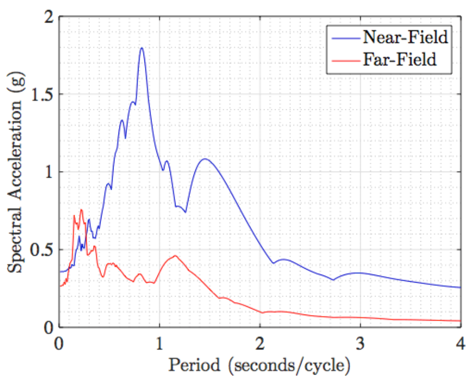
Figure 4 gives a comparison of near-field and far-field ground motion records from earthquake events of same magnitude. The near-field record is taken from Duzce Turkey earthquake of 1999 (PEER NGA record sequence number - 1605) and the far-field record is from Hector Mine earthquake 1999 (PEER NGA record sequence number - 1787), both of magnitude 7.1 and site class D. The near-field ground motion has greater PGA, PGV, 5% damped spectral displacement and pseudo-acceleration than the far-field record of same magnitude earthquake event. Thus for two earthquakes of same magnitude and site class, the near-field record poses more risk to building systems. By modeling and analyzing different single and multi-degree of freedom systems, it was concluded from several researchers that the elastic and inelastic displacement demands posed by near-fault records can be larger than those demands from far-fault records. Thus it is important to consider near-fault ground motion shaking effects in design and assessment of structural systems.



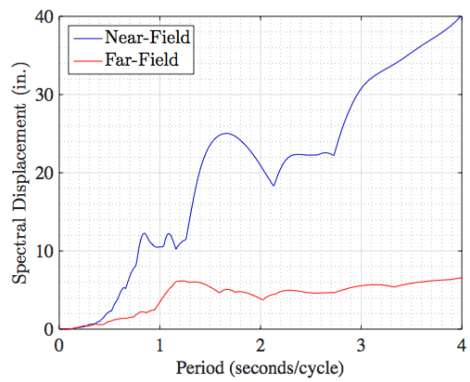
(a) Acceleration time history



(b) Velocity time history



(c) Pseudo spectral acceleration



(d) Spectral displacement

Fig. 4 – Comparison of far-field (RSN: 1605) and near-field (RSN: 1787) ground motion records from earthquake events of magnitude 7.1

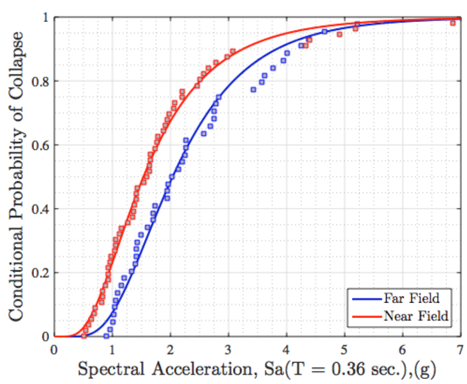
To study this source distance variability, the building models were subjected to suites of far-field records (22 ground motions pairs from sites located more than 10 km from the fault rupture line) and near-field records (28 ground motion pairs from sites located with 10 km from fault rupture line) from FEMA P-695. Out of the 28 selected near-field record pairs, 14 pairs have strong velocity pulses (Pulse subset) caused by near-fault directivity and the remaining pairs do not have velocity pulses (No-Pulse subset). Normalization is completed separately for these two subsets due to the distinctive large peak ground velocity values for the Pulse subset.

Table 3 summarizes the results from nonlinear analysis on the building models using far-field and near-field ground motions. For all the index models, collapse capacity ($\hat{S}_{CT}$ and CMR) that are based on near-field set of ground motions are lower than the equivalent set of far-field ground motions. Similar trends are observed in the MCE level analysis where the median drift and base shear demands were greater for near-field records. Study of the collapse spectral intensities of the near-field records indicated that the Pulse subset produced lower collapse spectral intensities as compared to the No-Pulse subset, but greater dispersion for the record-to-record variability was observed for the No-Pulse subset collapse spectral intensities.

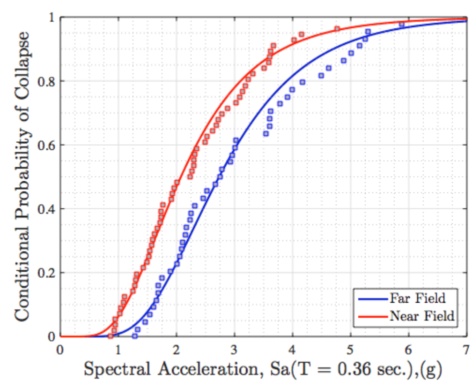
Figure 5 presents the fragility curves of the index models developed based on the collapse spectral accelerations. These fragility curves helps to understand the probability of collapse at different levels of spectral intensities and the distribution of these around the median collapse spectral intensities. It is clear from the fragility curves that the relative variation of individual ground motion collapse spectral intensities from the median collapse spectral intensity is fairly close between the far field and the near field ground motion records.

Table 3 – Collapse and MCE level analysis results using FEMA P-695 far-field and near-field ground motions

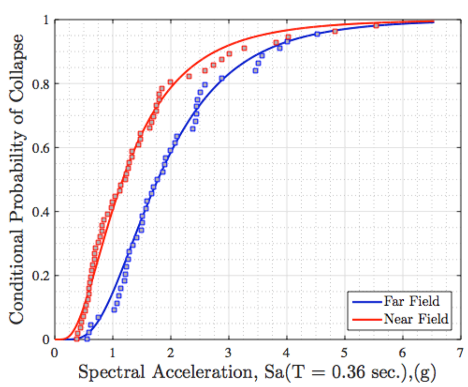
Index Model	Collapse Level Analysis						MCE Level Analysis			
	Far Field Records			Near-Field Records			Far-Field Records		Near-Field Records	
	$\hat{S}_{CT}$ (g)	CMR	β_{RTR}	$\hat{S}_{CT}$ (g)	CMR	β_{RTR}	Median Drift	Median Base Shear (kips)	Median Drift	Median Base Shear (kips)
IM 9	2.02	1.35	0.50	1.53	1.02	0.59	0.0353	65.4	0.0730	67.1
IM 10	2.80	1.86	0.43	2.08	1.38	0.48	0.0182	35.4	0.0332	37.8
IM 11	1.78	2.37	0.55	1.17	1.56	0.67	0.0162	25.5	0.0321	27.8
IM 12	2.26	3.02	0.56	1.57	2.09	0.60	0.0110	13.8	0.0165	15.6



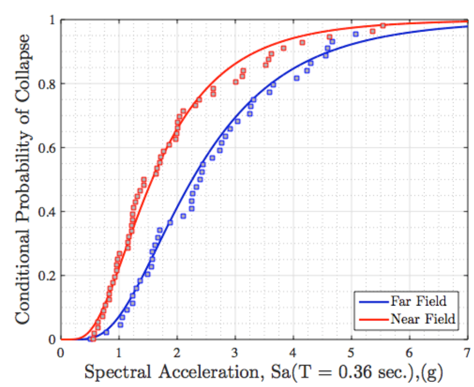
(a) IM 9



(b) IM 10



(c) IM 11



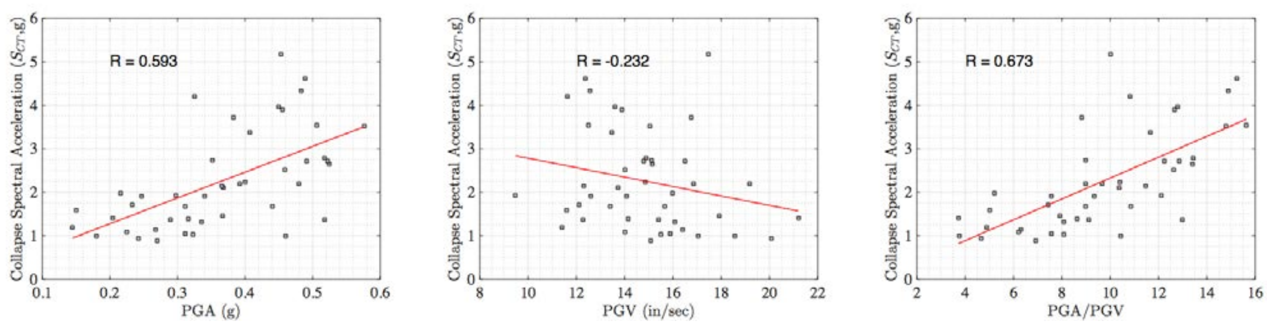
(d) IM 12

Fig. 5 – Fragility plots for building models with far-field and near-field ground motions

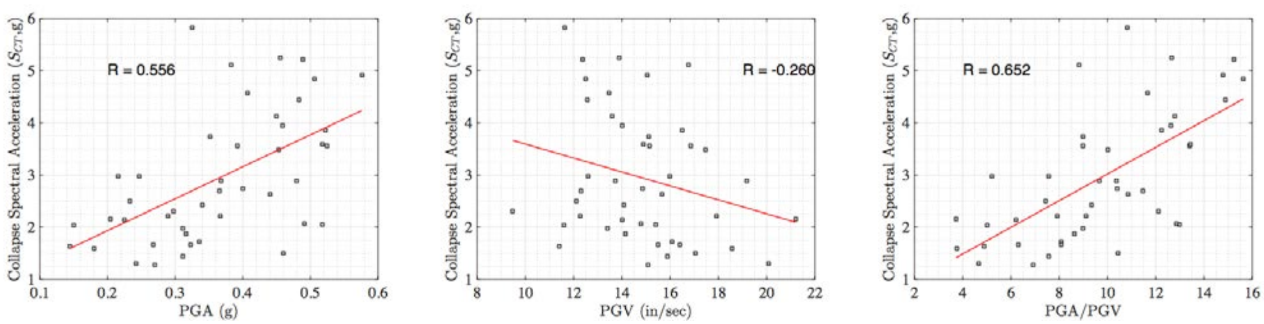
4.3 Influence of PGA, PGV and Ratio of PGA to PGV

Though intensity of a design ground motion can be specified with peak ground acceleration (PGA), velocity (PGV) and displacement (PGD), PGA is generally used as a metric for record selection based on ground motion intensity [18]. By analyzing the typical ground motion record data of an earthquake, it can be observed that PGA and PGV are recorded at different time instances and are caused by seismic waves of different frequencies. PGA is related with high frequency seismic waves, where as PGV is related with low frequency seismic waves. Since PGV is less sensitive to higher-frequency components of any typical ground motion, it characterizes the ground motion amplitude at intermediate level of frequencies. Previous research has also established a greater attenuation of PGA with distance than the PGV [19]. This results in larger PGA to PGV ratio for the ground motions recorded near the earthquake source and lower for a greater distance from the source. Thus the ratio of PGA to PGV (A/V) is a relative measure of frequency content and strong ground motion duration of ground motions. The research presented in [19] also concluded that the ratio of PGA to PGV is correlated to the earthquake magnitude, distance from epicenter and frequency content of the acceleration records. By virtue of this, ratio of PGA to PGV can be used as a potential ground motion selection criteria.

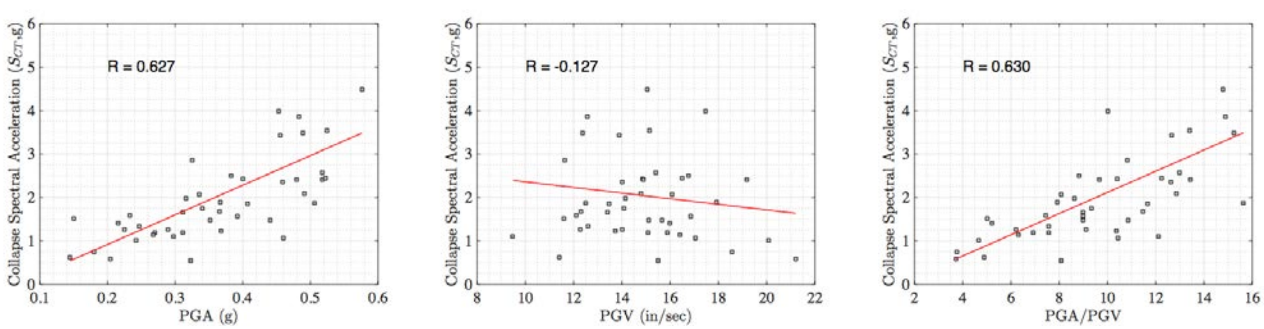
Figure 6 shows the distribution of the collapse spectral intensities of the normalized far-field set of ground motions to their respective peak amplitude values including PGA, PGV and ratio of PGA to PGV. The PGA of the normalized set of far-field ground motions vary from 0.15 g to 0.82 g with an average of PGA of 0.39 g. For the selected normalized record set, the PGV velocity between 9.48 in/sec. to 21.2 in/sec. with a mean value of 14.8 in/sec. To measure the linear dependence between the collapse spectral accelerations to the peak amplitude values, Pearson correlation coefficient (R) is derived and this is also included in the plots in Figure 6.



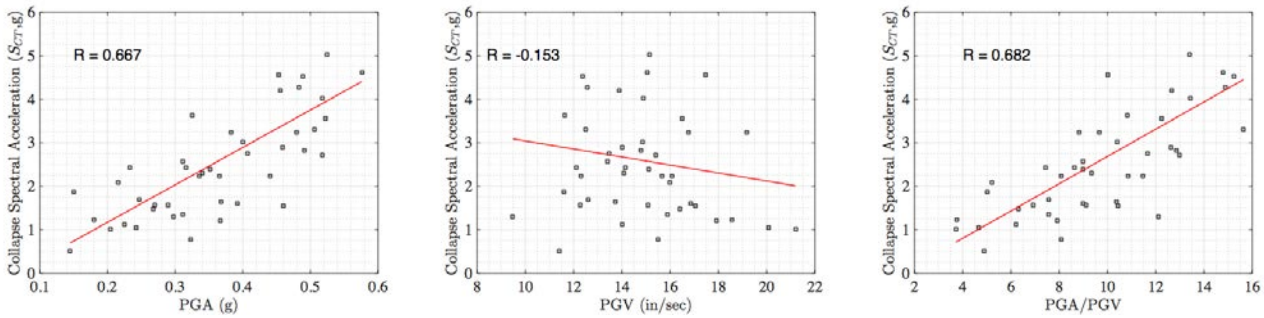
(a) IM 9



(b) IM 10



(c) IM 11



(d) IM 12

Fig. 6 – Correlations of collapse spectral intensities to PGA, PGV and ratio of PGA to PGV for building models

For the set of far-field ground motions, the collapse spectral accelerations have a positive correlation with the PGA and negative correlation with the PGV. The collapse spectral intensities also showed a positive correlation with the ratio of PGA to PGV. This same trend in observed for all the four models selected in this study (with varying shear wall aspect ratio and seismic design category) and thus the correlation coefficients are independent to the design features. Due to its good correlation with the collapse characteristics, ratio of PGA to PGV should be considered in the selection of input ground motions for seismic performance assessment.

5. Summary and Conclusions

The research presented in this paper evaluates the variability in the seismic performance of wood-frame shear wall building models with variations in the ground motion characteristics. Four building models with variations in the shear wall aspect ratio and seismic design forces were selected to study the seismic performance with variations in ground motion matching procedures, source-to-site distance characteristics and peak amplitude response at the collapse level and design level of spectral accelerations.

To study the influence of ground motion scaling procedure, the selected building models are analyzed with amplitude scaled and spectrum matched far-field ground motions. Spectrum matched ground motions produced lower CMR results as compared to the amplitude scaled records at the collapse level for models IM 9, IM 10 and IM 11 and larger CMR results for model IM 12. Alternatively, at the MCE level, spectrum matched records generated less median drift and base shear demands as compared with amplitude scaled motions. Thus spectrum matched motions should be cautiously used in seismic performance assessment as it may lead to unconservative results. As expected, matching also reduced the dispersion in the collapse spectral acceleration.

Ground motion source-to-site distance on the seismic behavior is evaluated using the set of far-field and near-field record set from FEMA P-695. The set of near-field records have more seismic collapse risk than the far-field records. Also the presence of velocity pulses further reduces the collapse capacity of the building structures. At the MCE level of ground motions, the median drift and base shear demands of the near-field records larger than the corresponding set of far-field records.

The significance of peak amplitude values of the ground motion as a ground motion selection criterion is assessed by studying the correlation of the collapse spectral intensities with PGA, PGV and ratio of PGA to PGV of the far-field set of records. It is observed that the ratio of PGA to PGV has a strong correlation to the ground motion characteristics including the strong motion duration and frequency content. This ratio also has good positive correlation with the collapse spectral intensities of the ground motions and hence it should be considered in the selection of ground motions for seismic performance assessment.

It should be noted here that the seismic performance variations studied in this paper is only based on the ground motion records used in FEMA P-695. From a statistical point of view, increasing the number of ground motions in the input bin is expected to improve the accuracy of the seismic response predictions. Though not included in this study, the seismic performance of building models can also have correlations to the spectral shape, strong motion duration, magnitude of earthquake and site conditions of the selected ground motions.

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