Comparison of the IBC 2006 Equivalent Lateral Force Procedure to FEMA 356/ASCE 41 Life Safety Acceptance Criteria for Timber Structures

D.P. Dodge[1] and C.B. Chadwell[2]

[1] *Staff Structural Engineer, URS Corporation, San Francisco, CA, USA*
[2] *Asst. Professor, Dept. of Civil and Environmental Engineering, California Polytechnic State University, San Luis Obispo, CA, USA*
Email: darius_dodge@urscorp.com, chadwell@calpoly.edu

ABSTRACT :

In order to better understand the impact of the 2006 International Building Code (IBC 2006) relative to other performance-based design criteria - FEMA 356/ASCE 41 (ASCE 2000, ASCE 2007), a study was conducted to compare acceptance criterion between the two documents in the context of light frame wood sheathed shear wall buildings. The comparison is made between the life-safety requirements of FEMA 356/ASCE 41 and the IBC 2006 drift limit states using results from test data. Data taken from pseudo-static testing of light-frame structural panel shear walls was calibrated to a pinched-hysteretic force-displacement articulation model. Nonlinear time-history analyses were conducted for a range of modeled timber structures using the analytical wall models. Three probability of exceedence ground motion suites, obtained from the SAC Steel Study (SAC, 1997), were used as analytical input. Demands were collected in the form of either drifts (for use in IBC 2006 acceptance evaluation) or ductility demands (for use in FEMA356/ASCE 41 acceptance criterion). Results suggest that evaluation using the IBC 2006 produces (in the context of light frame wood sheathed walls) a generally a more conservative estimate to the overall performance of a structure when compared to the life-safety acceptance criterion of FEMA 356/ASCE 41.

KEYWORDS:
FEMA Acceptance Criteria, Timber Shear Walls

1. Introduction

In recent years, the shift in earthquake engineering is more toward performance-based design. In particular, performance-based design of timber structures under the demands of seismic loading has been of significant interest. In the newly adopted California Building Code (International Building Code, 2006 – IBC 2006) non-bearing light frame structures sheathed with structural paneling have been given specific design coefficients for response modification (R), overstrength (Ω_0), and a life safety event displacement limit state for a performance-based static design procedure. However, FEMA 356/ASCE 41 outlines specific methodology for performance based design of this particular structural system with displacement ductility demand limit states for immediate occupancy (IO) life safety (LS) and collapse prevention (CP). The research presented herein compares code compliance with the life safety requirements of FEMA 356/ASCE 41 with the IBC life safe allowable drift limit state using results from test data.

The experimental phase of the investigation consisted of pseudo-static testing of full-scale, standard construction timber shear walls (8ft by 8ft) subject to the CUREE-Caltech loading protocol. It should be noted that three of the four tested walls were constructed with a spray-applied polyurethane foam (SPF) infill as part of a concurrent study (Dodge and Chadwell, 2008). Data used in this investigation consists of that collected from both bare and SPF infill walls. The difference in hysteretic behavior between the different walls is small and is conjectured not to influence the findings presented herein. Furthermore, the SPF foam considered is not currently a structural product and the differences in hysteretic behavior with, versus without the foam infill can be taken simply as material variability due to minor inconsequential contributions of non structural components. Using the collected data, phenomenological hysteresis backbone curves were developed and used for calibration of a single degree-of-freedom pinched hysteretic force-displacement articulation model for nonlinear time history analysis.

The analytical investigation consisted of two parts: one, completion of a series of nonlinear time-history analyses using the single degree-of-freedom pinched hysteretic force-displacement articulation model calibrated from experimental testing; and two, a simple reliability analysis to identify the probability of failure between the IBC 2006 drift limit state and FEMA 356/ASCE 41 ductility demand limit states.

As part of this study, both one- and two-story structures were modeled with a range of reasonable building weights. Using the fitted backbone curves (from data collected from each of the four walls tested), the force displacement behavior was scaled to represent the minimum required code design specific to the building weight of interest. Input for the time-history analyses included varying probability of exceedence ground motion records from the SAC Steel Investigation (SAC, 1997). These included 10%, 2%, and 50% probability of exceedence in a 50 year period ground motion suites. However, in an effort to make an equitable comparison between the FEMA 356/ASCE 41 and the IBC 2006 acceptance criterion, only the results from analyses with the ground motion suite having a 10% chance of exceeding in a 50 year period are presented herein. Lastly, reliability analyses were conducted to establish a probably of failure for each of the acceptance criterion considered.

2. Pseudo-Static Testing of Timber Shear Walls and Model Development

2.1. Pseudo-Static Testing of Timber Shear Walls

A total of four 8ft by 8ft walls were tested with the CUREE-Caltech pseudo-static loading protocol (Krawinkler et al., 2001): one control wall (No Foam) and three walls with the spray applied polyurethane foam infill (Foam_1, Foam_2, and Foam_3). The No Foam wall was a bare wood sheathed wall built using standard United States construction practices. Each wall contained typical wood framed wall components including as: a 2x4 double top plate (drag strut), 3x4 bottom sill, Simpson Strong Tie® shear wall hold downs, ½in thick oriented strand board panel (OSB), nailing at 12in on center (o.c.) in the field and 4in o.c. on the panel edge, and 5/8in diameter anchor bolts. The ½in thick panel was used as the basis for design of remaining components. Component strength was designed to well exceed the resultant forces from application of shear panel capacity. Components that were designed by this method include boundary elements, bottom sill, collector, anchor bolts, and boundary element anchors.

Walls were tested using an 80 kip large displacement hydraulic actuator against a 150 kip capacity strong wall. A high-stiffness out-of-plane restraint system was added to provide representative boundary conditions as expected in standard construction for a light-frame residential structure. The experimental set up is shown in Figure 2.1.

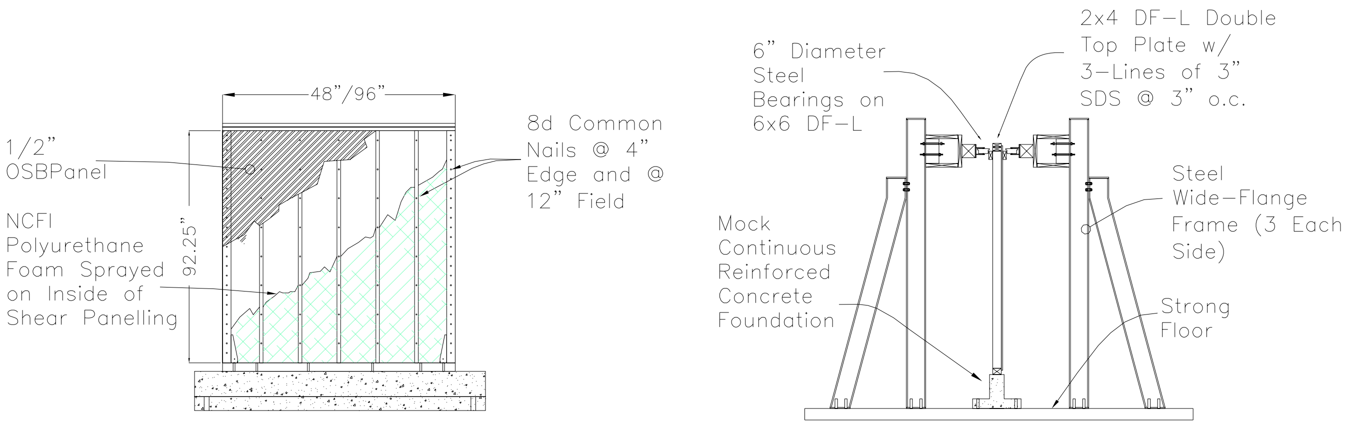

Figure 2.1: Experimental Testing Setup

All of the tested walls exhibited normal hysteretic response during cyclic loading both with and without SPF infill application. Consistent with the findings of ASCE/SEI 41, Supplement No. 1 (Elwood, 2007), provided as recommendations for addendum to ASCE41, bounding backbone curves were used in this study to describe the recorded force displacement data from testing (Figure 2.2).

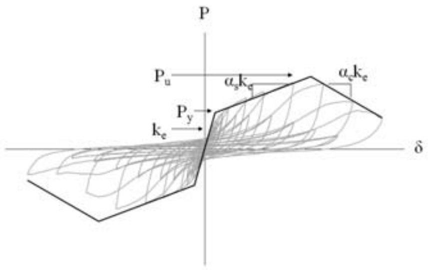

Figure 2.2: Bounding Backbone Curve Used for Model Calibration

Each curve was a construction of intersecting line segments fitting the peak values of each primary drift cycle to represent elastic, post-yield and post-peak slopes. Each wall configuration was entered into the model and run according to the CUREE-Caltech protocol, to match results from testing. Figure 2.3 presents the best fit for each wall configuration. Calibration was conducted using a visual iterative process, as well as a final numerical verification of major drift cycles between actual and model data. Table 2.1 provides the force and drift values for each peak drift cycle of walls tested.

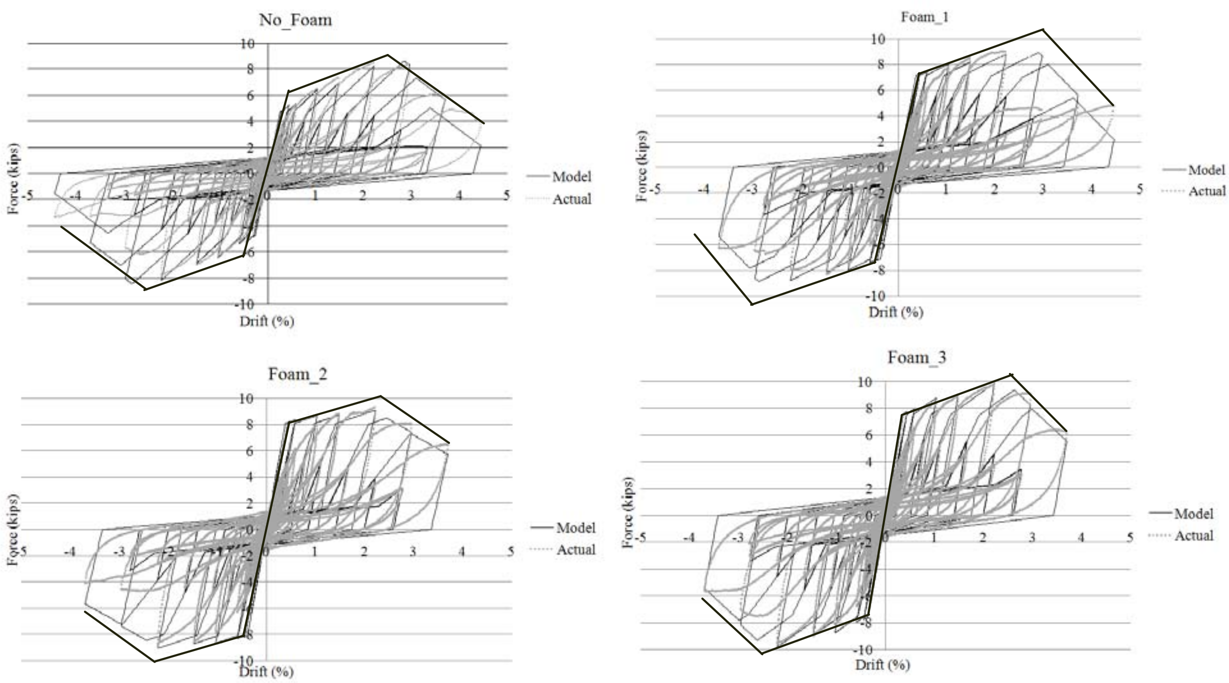

Figure 2.3: Model Calibration with Testing Results and Hysteresis Fit

Table 2.1: Summary of 8' by 8' Wall Hysteretic Backbone Curves

No Foam		Foam_1		Foam_2		Foam_3	
Drift (%)	Force (kips)	Drift (%)	Force (kips)	Drift (%)	Force (kips)	Drift (%)	Force (kips)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
0.25	5.18	0.26	5.10	0.25	5.07	0.27	5.63
2.22	6.97	1.48	7.68	1.48	7.78	1.48	8.01
2.96	5.52	2.96	3.60	2.96	5.53	2.96	5.87
3.70	5.52	3.70	3.60	3.70	5.53	3.70	5.87

2.2. Force-Displacement Articulation Model Development from Associated Testing Results

Simulations incorporated a suite of light frame residential structures developed from a range of reasonable one- and two-story building weights (one story buildings weighing 30 kips to 110 kips in increments of 5 kips and two story buildings weighing 60 kips to 220 kips in increments of 10 kips). Then, assuming two-lines of lateral resistance, appropriate wall length was derived from IBC 2006 equivalent lateral force procedure and wall properties were scaled using original 8ft by 8ft wall backbone properties from testing. By using the IBC 2006/2003 NEHRP provisions for construction of the earthquake design spectra and using an assumed Site Soil Class D (according to IBC 2006), seismic base shear and associated minimum required wall length were derived. The response modification coefficient (R) used for calculation of base shear according to IBC 2006 provisions is 6.5 for non-bearing light frame structural wood panel shear walls.

The scaling procedure used to modify the tested 8ft by 8ft shear walls to the minimum required length by IBC 2006 incorporated scaling equations for modifying strength, stiffness, and equivalent hysteretic energy properties. The specifics of the scaling processes used are not provided here for brevity, but can be found in Dodge, 2008. Figure 2.4 provides select examples of resulting backbones from the scaling procedure for three select modeled walls (8ft, 12ft, and 16ft).

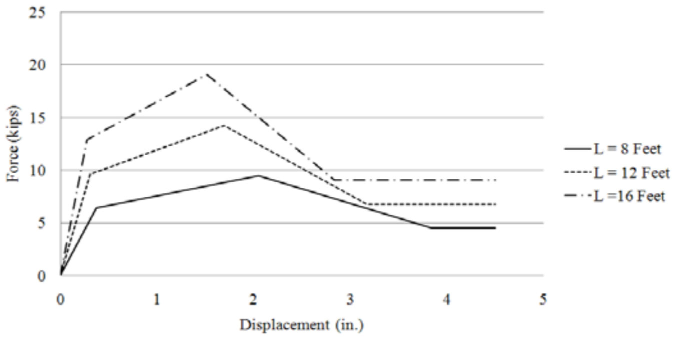

Figure 2.4: Select Wall Backbones Following Scaling Procedure

The 8ft by 8ft model incorporated strength and stiffness degradation that is inherent in cyclic loading of structures (Ibarra et al., 2005). Normalized hysteretic energy (NHE) was collected for each wall configuration and for each individual direction of loading. Hysteretic energy collected for positive and negative directions of loading (E_i), defined by the sign of the resisting force change that occurred. The sum of all hysteretic energy (E_t) is considered the system hysteretic energy capacity and is the basis for a singular degradation parameter. The sum of all absorbed energy (E_j) represents accumulated damage during cyclic excitation. Each wall property uses the parameter β, expressed in Equation 2.1 (Ibarra et al., 2005), to quantify the amount of wall deterioration.

$$\beta_i = \left(\frac{E_i}{E_t - \sum_{j=1}^{i} E_j} \right)^C \tag{2.1}$$

Where: E_i = *Hysteretic Energy Per Excursion, i*

$E_t = \gamma F_y \delta_y$ = *System Hysteretic Energy Capacity*

ΣE_j = *Sum of Hysteretic Energy in All Previous Excursions*

c = *factor for rate of deterioration*

F_y = *System Yield Strength*

δ_y = *Displacement at Yield*

From shear wall testing, it was found that NHE for each direction of loading was unique. This provided the basis for calibration of each wall model. Table 2.2 presents the values of summed NHE, E_t (unit-less).

Table 2.2: Summary of Summed NHE for Each 8ft by 8ft Wall

No Foam	Foam_1	Foam_2	Foam_3
126.08	95.20	115.53	98.57

3. Analytical Simulations and Results

3.1 Input for Analytical Simulations

Three suites of ground motions, with varying probabilities of exceedence, were input to the various models,. For each building simulated and ground motion used, peak displacements and ductility demands were collected. FEMA 356/ASCE 41 ductility demand limit states and IBC drift limit states were selected to represent acceptance criteria. The input ground motions were obtained from the SAC Steel Project (SAC, 1997) and included 10% (10 in 50, or design level earthquake), 2% (2 in 50, or maximum considered earthquake) and 50% (50 in 50, or service level earthquake) probability of exceedence in 50 year period ground motions.

3.2 Results from Computer Based Earthquake Simulations

Performance-based acceptance criterion, according to IBC 2006, is prescribed as a drift limit of 2.5% for any building-type structure considered. FEMA 356 provisions; however, rely upon specific values from the tested nonlinear hysteretic backbone curves (FEMA 356, Section 2.4.4.3.1) to define IO, LS, and CP limit states. FEMA 356/ASCE 41 limit states found from the tested and fitted backbone curves are provided in Table 3.1.

Table 3.1: Summary of Ductility Demand Limit States According to FEMA 356/ASCE 41

	No Foam	Foam_1	Foam_2	Foam_3
Immediate Occupancy	3.81	4.29	3.05	3.69
Life Safety	5.69	6.41	4.55	5.50
Collapse Prevention	12.09	11.44	12.69	11.41

Simulation results are presented as a ratio of the demands normalized by the acceptance criterion: 2.5% drift in the context of IBC 2006 or the life safety limit state in the context of FEMA 356/ASCE 41. Comparison between the differing acceptance criterion show that a particular design may be adequate when considering the IBC 2006 and not adequate when considering FEMA 356/ASCE 41 or vice-versa. This result is not unexpected as the acceptance criterion as per FEMA 356/ASCE 41 is a relative dimensionless quantity that depends on the behavior of the system and the IBC 2006 acceptance criteria is an absolute value for all short period systems (2.5%).

For the purposes of this paper, focus will be paid to the design level earthquake (DLE) simulations (the 10% probability of exceedence in 50 year event) so as to make a meaningful comparison. The intent of the IBC 2006 provisions, for a typical building of this type and occupancy is to provide life safety for the DLE. As can be seen in Figures 3.1 to 3.3, the comparison between results using IBC 2006 criterion versus FEMA 356/ASCE41 criterion show that there are noticeable differences. Both magnitudes of normalized demand as well as the trend with respect to increasing building weight are observed to be different between the codes. It should be noted that tabularized values plotted in Figures 3.1 to 3.3 greater than 1.0 represent a particular limit state not being satisfied. Furthermore, the following figures represent the average normalized demand over the 20 ground motions considered within the DLE suite.

Figure 3.4 depicts the percentage difference in normalized demand between the two measures of performance for both one- and two-story structures considered herein. The figure suggests that by using either of the two documents as an acceptance criterion, the designer may not satisfy the performance requirement of the other document by as much as approximately 40%.

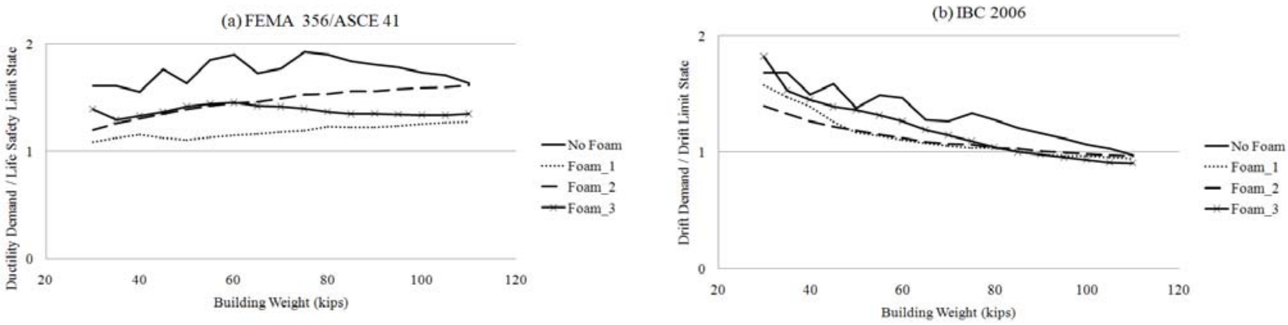

Figure 3.1: One-Story Results for Design Level Earthquake Using FEMA 356/ASCE 41(a) and IBC 2006(b)

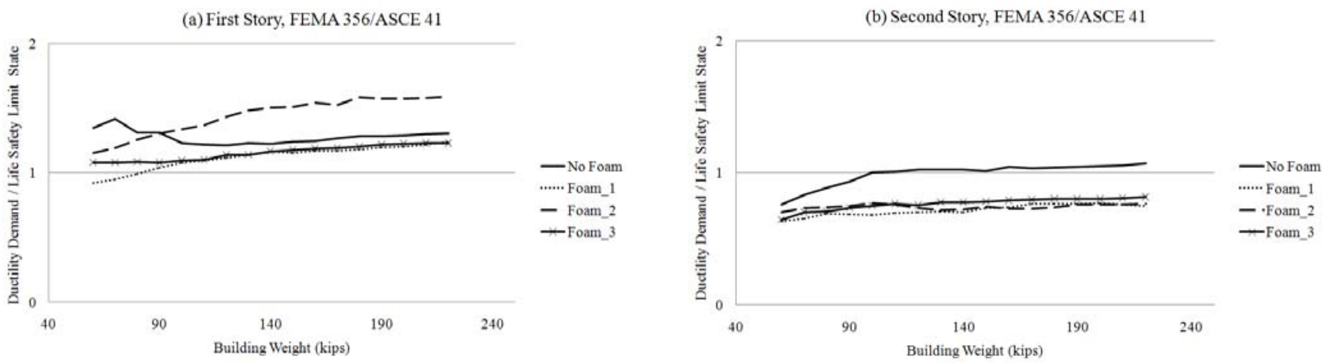

Figure 3.2: Two-Story Results for Design Level Earthquake Using FEMA 356/ASCE 41

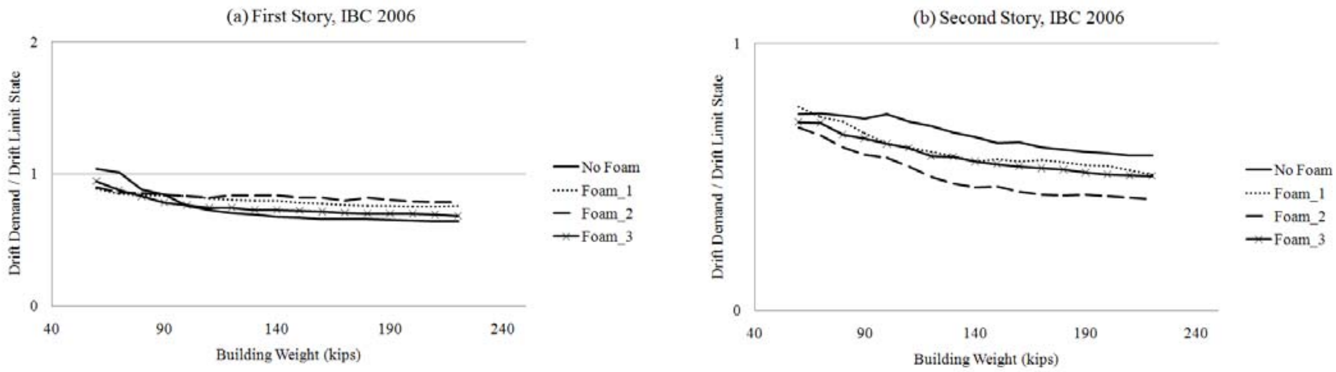

Figure 3.3: Two-Story Results for Design Level Earthquake Using IBC 2006

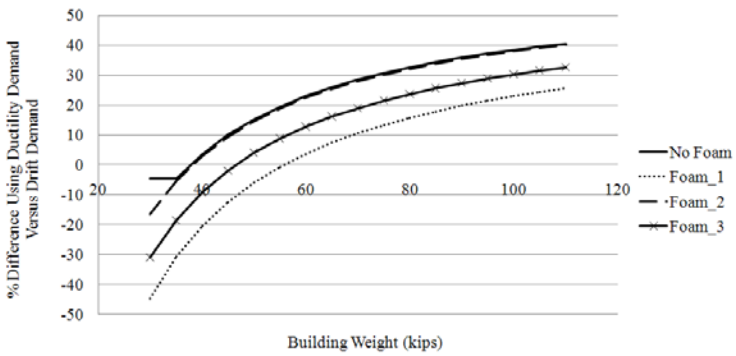

Figure 3.4: Percentage Difference Between Normalized Ductility Demand and Normalized Drift Demand

3.3 Results from Reliability Analysis

Simplified reliability analyses were conducted in order to establish a probability of failure for each of the FEMA 356/ASCE 41 and IBC 2006 acceptance criteria. For these analyses, a simple margin of safety formulation was used to compare demand to available capacity normalized to the standard deviation based upon a reliability index (Equation 3.1).

$$\beta = \frac{\mu_M}{\sigma_M} \qquad (3.1)$$

Where: μ_M = *Difference in mean resistance and mean demand*

σ_M = *Difference in standard deviation between resistance and demand*

By using this approach, a more representative description of structural performance is provided by incorporating the effect of inherent variation throughout the data. The results confirm, for all cases considered, the use of IBC 2006 provisions yield significantly different results as compared to the FEMA 356/ASCE 41 results (Figures 3.5 to 3.7).

The findings using reliability analyses are consistent with the findings provided in Figures 3.1 to 3.3. In all cases, a lower probability of failure is given for both a one- or two-story structure under the demands of a DLE, using drift demand as compared to the results using ductility demand (Figures 3.5 to 3.7).

Table 3.2 shows a summary of both the data from all DLE simulations conducted as well as the associated reliability analyses results. Values provided in this table are an average of all analyses conducted for each building type (one- or two-story). In nearly all cases, use of drift demand and the IBC 2006 drift acceptance criterion provides a lower prediction of probability of failure.

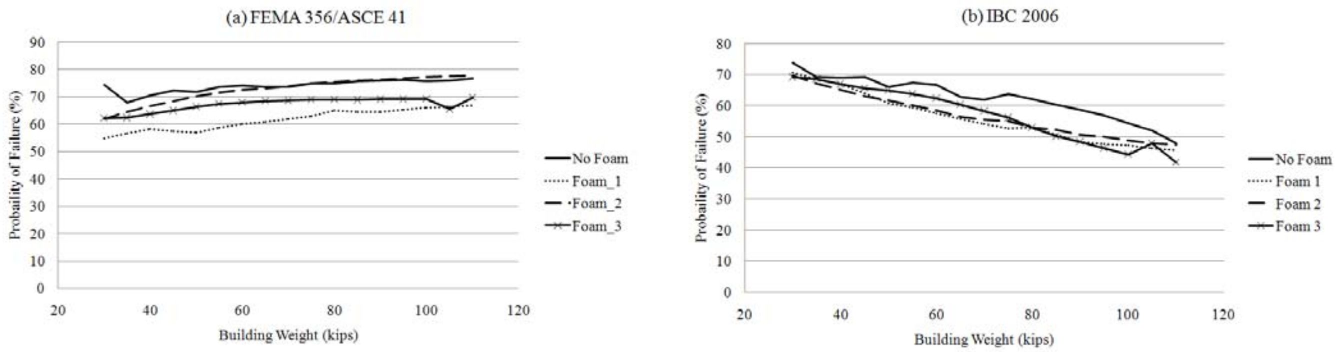

Figure 3.5: One-Story Probability of Failure Results for Design Level Earthquake for FEMA 356/ASCE 41(a) and IBC 2006(b)

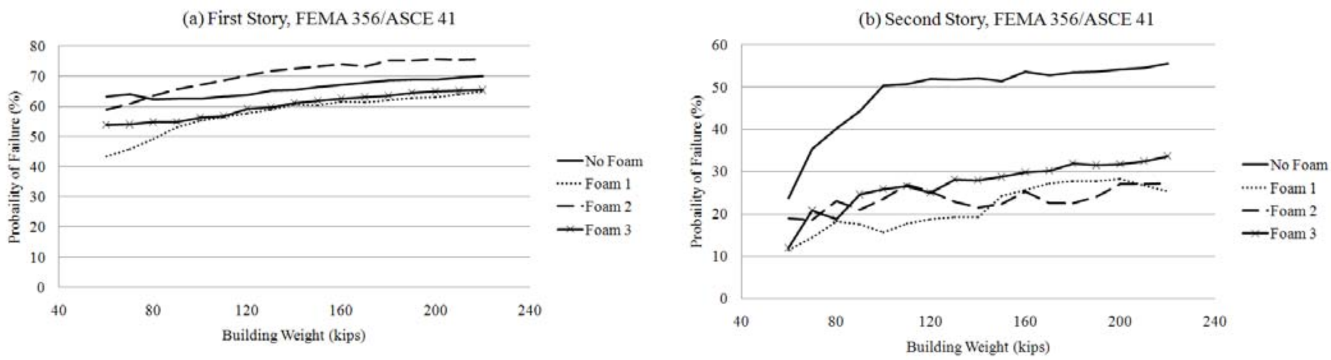

Figure 3.6: Two-Story Probability of Failure Results for Design Level Earthquake Using FEMA 356/ASCE 41

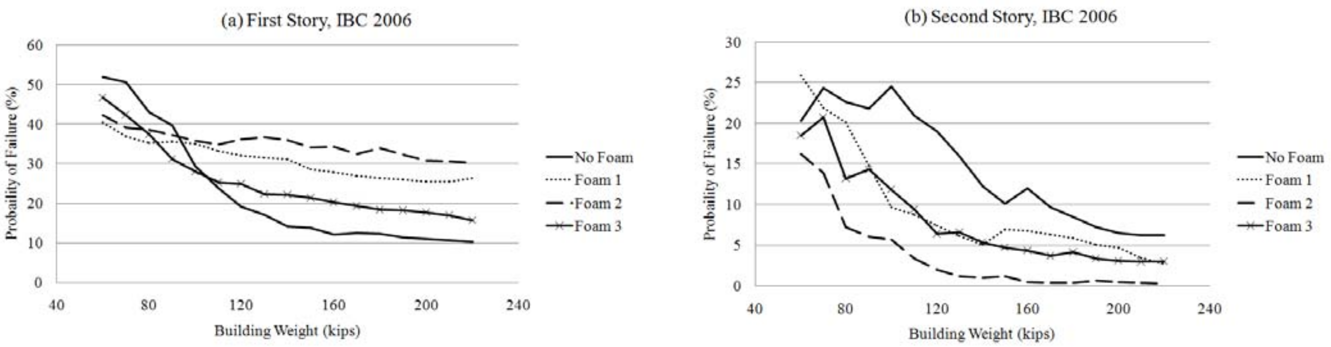

Figure 3.7: Two-Story Probability of Failure Results for Design Level Earthquake Using IBC 2006

In addition, in consideration of the one story building when using either of the acceptance criterion, results from the reliability analysis show that the probably of failure is consistently greater than 50%. This, in turn, suggests that either using a response modification coefficient (R) equal to 6.5 that is currently dictated for this type of wall by IBC 2006, may not be sufficient, or the 10% in 50 ground motion suite used in this analysis is not representative of the spectral ordinate used in the prototype buildings considered. Research is ongoing to rectify this discrepancy.

Table 3.2: Summary of All Simulation and Reliability Analysis Results

			No Foam (No Units)	Foam, Mean (No Units)	Prob. Of Failure, No Foam (%)	Prob. Of Failure, Foam, Mean (%)
Ductility Demand	One Story	10 in 50	1.7	1.3	73.9	67.1
	First of Two Story	10 in 50	1.3	1.2	65.9	62.8
	Second of Two Story	10 in 50	1.0	0.7	48.7	24.2
Drift Demand	One Story	10 in 50	1.3	1.1	62.5	56.4
	First of Two Story	10 in 50	0.7	0.8	22.5	30.4
	Second of Two Story	10 in 50	0.7	0.6	14.6	7.0

4. Conclusions

As is not unexpected, the results of this study suggest that the determination of structural acceptance criterion, under the demands of seismic loading, will depend upon the design criteria chosen. Specifically, research presented herein shows that the determination of performance of non-bearing light frame structures will show a different level of performance depending on the use of either provisions of IBC 2006 or FEMA 356/ASCE 41. The following represent the main results and observations from this study:

- IBC 2006 may underestimate the demand (normalized) relative to FEMA 356/ASCE 41 for non-bearing light frame structural panels under consideration of a life safety condition for the majority of building weights considered herein
- IBC 2006 may underestimate probability of failure relative to FEMA 356/ASCE 41 for non-bearing light frame structural panels under consideration of a life safety condition for the majority of building weights considered herein
- The current R factor used in non-bearing light frame timber structures may result in an underestimation of demands when using the IBC 2006 equivalent lateral force procedure

REFERENCES

American Society of Civil Engineers (2000) Prestandard and Commentary for The Seismic Rehabilitation of Buildings : FEMA 356, Washington Federal Emergency Management Agency

American Society of Civil Engineers (2007), Seismic Rehabilitation of Existing Buildings: ASCE 41, American Society of Civil Engineers

Dodge, D.P., Chadwell, C.B. (2008), *Experimental and Analytical Investigation of Spray-Applied Polyurethane Foam Infill Timber Shear Walls*, 2008 SEI Structures Congress

Dodge, D.P. (2008), *Investigation of Seismic Performance of Timber Shear Walls Using Spray-Applied Polyurethane Foam Infill* Graduate Research Thesis, California Polytechnic State University, San Luis Obispo

Elwood, K.J. (2007), *Improved Predictions of Seismic Performance of Existing Concrete Buildings – ASCE/SEI 41, Supplement No. 1*, 2007 SEI Structures Congress

Krawinkler, H., Parisi, F., Ibarra, L., Ayoub, A., Medina, R., (2001) *Development of a Testing Protocol for CUREE-Caltech Woodframe Project*, The CUREE Caltech Woodframe Project, 2001

Ibarra, L.F., Medina R.A., Krawinkler H. (2005), *Hysteretic Models that Incorporate Strength and Stiffness Deterioration,* Earthquake Engineering and Structural Dynamics, 34, 1489-1511

International Code Council (2006) International Building Code 2006: IBC 2006 International Code Council

National Association of Home Builders Research Center (1996), *Light Gauge Steel Framing with Sprayable Polyurethane Foam,* NAHB

Sac Joint Venture Steel Project Phase 2 (1997), *Develop Suites of Time Histories: Project Task 5.4.1, Draft Report,* SAC