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SIMPLIFIED PERFORMANCE-BASED DESIGN OF NEESWOOD CAPSTONE BUILDING AND PRE-TEST PERFORMANCE EVALUATION

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ABSTRACT

This paper presents a simplified direct displacement design (DDD) procedure which was used in the design of the shear walls for a six-story wood frame test building for a Network for Earthquake Engineering Simulation project (NEESWood). The test building was designed to meet four performance expectations (damage limitation, life-safety, far-field collapse prevention, and near-fault collapse prevention). The NEESWood Capstone Building was recently tested in-full-scale on the E-defense (Miki) shake table in July 2009. As part of the pre-test performance assessment process, a series of nonlinear time-history analyses were performed to verify that the inter-story drift requirements were met. Additionally, collapse analysis in accordance with the methodology presented in the ATC-63 90% draft report was also carried out. The results of incremental dynamic analyses confirmed that the Capstone Building designed using the DDD procedure has adequate capacity or margin against collapse.

Introduction

The design provisions in the current United States (US) building codes limit the story height of wood frame buildings to five stories (ICC 2006) in general, and even four stories in some jurisdictions. The height limitation reflects the lack of knowledge of the dynamic response of taller wood buildings under lateral loadings, fire safety considerations and other local district land use regulations. Such height restrictions have limited the use of wood for multi-story construction in the US. Recent trends in building construction show an increasing demand for multi-story wood frame buildings in the US. The increased demand for tall wood frame buildings highlights the need for new design methodologies for building taller wood frame structures with confidence, including those in seismic. One such effort is the NEESWood project which focuses on the development of a performance-based seismic design (PBSD) procedure for mid-rise wood frame construction in regions of moderate to high seismicity (van de Lindt et al. 2008). Two full-scale shake table test programs were conducted as part of the NEESWood project: (1) a two-story Benchmark Wood Frame Building was tested at the University at Buffalo (UB) Network for Earthquake Engineering Simulation (NEES) site (Christovasilis et al. 2007), and (2) a full-scale six-story Capstone Building was tested on the E-defense (Miki) shake table in July 2009. The Benchmark Building was representative of a typical townhouse structure built in the Western US in the 1980's. Using the results and research findings obtained from the Benchmark test, numerical tools for nonlinear time-history analysis (NLTHA) and a preliminary version of a

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new direct displacement design (DDD) procedure for PBSD of multi-story wood frame buildings were developed (Pang and Rosowsky 2009). The proposed DDD procedure was later simplified and was used to design the shear walls of the six-story NEESWood Capstone building.

Description of Capstone Building

The elevation and first floor plan views of the NEESWood Capstone Building are shown in Figure 1. The plan dimensions of the building are approximately 18.1 m in the longitudinal direction and 12.1 m in the transverse direction. The height of the building from the base to the top of the roof parapet is approximately 17.8 m, with a story clear height of 2.74 m for 1st and 6th stories and a story clear height of 2.44m for 2nd to 5th stories. The shear walls are built with nominal 51 mm × 152 mm Douglas Fir and Spruce Pine Fir studs spaced at 406 mm on-center and 10d common nails (3.76 mm in diameter) are used to fasten the 11.9 mm thick Oriented Strand Board (OSB) to the framing members. The total living space of the test building is approximately 1350 m^2. There are 23 living units with four apartment units on each floor except for the 6th floor which contains a large luxury penthouse and two regular apartment units. The total seismic weight of the as-designed building was estimated to be 2749 kN.

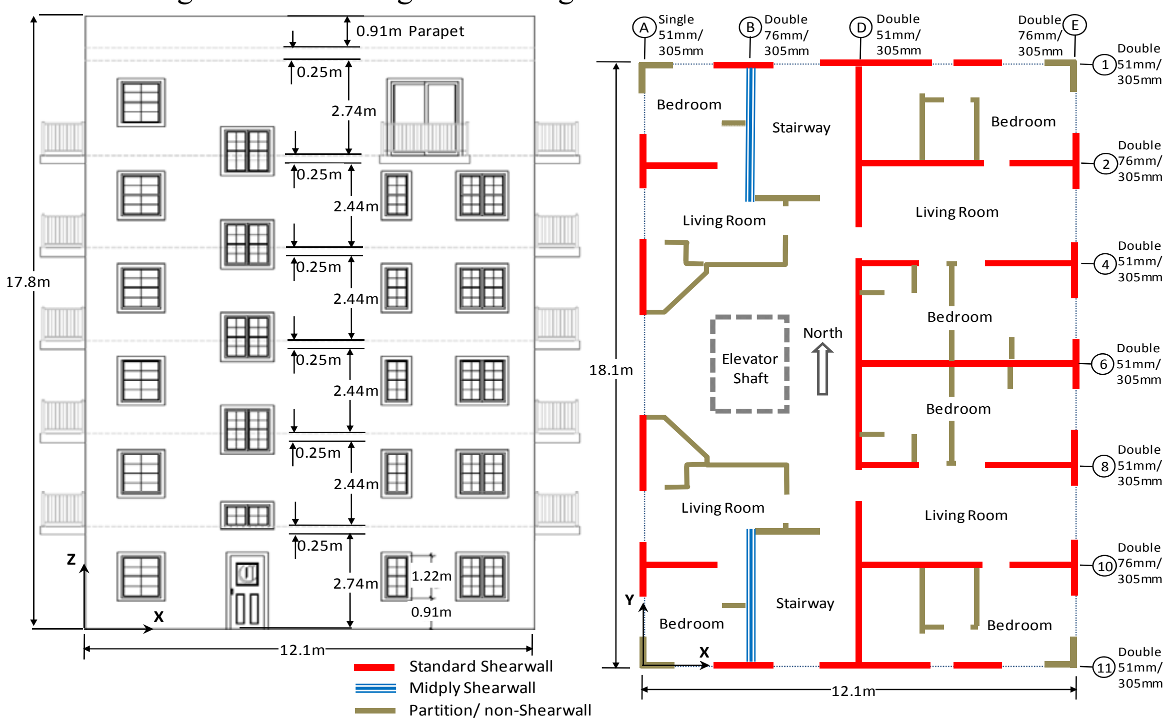

Figure 1: NEESWood Capstone Building, (a) elevation view, and (b) first-floor plan view.

Performance Expectations and Design Spectra

In DDD, each performance requirement is specified by a probability of non-exceedance (NE) of an inter-story drift limit at a specified level of seismic hazard:

$$P_{NE}(\theta < \theta_{\lim} \mid H) \geq NE_t \tag{1}$$

where θ and θ_{lim} are the inter-story drift and target drift limit, respectively. The term $P_{NE}(.)$ is the NE probability of the inter-story drift at a prescribed hazard level (seismic intensity, H) and NE_t is the target NE probability. The target performance expectations for the six-story Capstone Building are listed in Table 1. The drift limits for Levels 1 to 2 were adopted from the design guidelines in the ASCE-41(2006) for immediate occupancy and life safety performance levels, respectively. The Level 3 drift limit was selected based on observations made during the NEESWood Benchmark test (Christovasilis et al. 2007) while the drift limit for Level 4 (7%) was based on the collapse drift limit used in the ATC-63 project to evaluate the collapse probability of wood buildings (ATC 2008). The seismic hazards for performance levels 1 to 3 are associated with earthquakes having 50%, 10% and 2% exceedance probabilities in 50 years, respectively. The Capstone Building is assumed to be located in Southern California and founded on stiff soil (Site Class D). The 5%-damped horizontal acceleration design spectrum parameters for seismic hazard Levels 1 to 3, determined in accordance with ASCE-41, are listed in Table 2. These far-field response spectra were used in the simplified DDD procedure to design the Capstone Building. Since Level 4 is associated with the near-fault ground motions, the response spectrum was not specifically determined or used in the design process. However, a suite of un-scaled near-fault ground motions (Krawinkler et al. 2003) were used in the NLTHA to verify the design of the Capstone Building at Level 4.

Table 1: Performance expectations.

Performance Level	*Seismic Hazard*	*Performance Expectations*	
		Inter-story Drift Limit	NE Probability
Level 1	50%/50yr	1%	50%
Level 2	10%/50yr	2%	50%
Level 3	2%/50yr	4%	80%
Level 4	Near-Fault	7%	50%

Table 2: Design spectral acceleration values for 5% damping.

Hazard Level	*Intensity (% of DBE)*	*Exceedance Probability*	*Spectral Acceleration*	
			Short-period S_S(g)	1-second $S_1^{(a)}$ (g)
Short Return Period Earthquake	44%	50%/50yr	0.44	0.26
Design Basis Earthquake (DBE)	100%	10%/50yr	1.00	0.60
Maximum Credible Earthquake	150%	2%/50yr	1.50	0.90

Shear Wall Database

The Capstone Building is built almost entirely using conventional North American style stud wall systems (referred as *standard* walls in this paper), except for an interior wall line parallel to the longitudinal direction (Figure 1) in which very high shear capacity is required along which a new system known as *midply* construction is used (Varoglu et al. 2007). In the simplified DDD procedure, the complete shear wall backbone curve (force versus displacement) is required. The shear wall backbone curves for the six-story Capstone Building were determined from backbone curves obtained using the M-CASHEW program, a Matlab version of the CASHEW (Cyclic Analysis of Wood SHEar Walls) program (Folz and Filiatrault 2001a). The shear wall backbone curve is defined by the following five-parameter equation used in the modified Stewart hysteretic model:

Table 3: Shear wall database (per meter of wall width).

Wall Type/ Panel Layer	Edge Nail Spacing (mm)	K_o (kN/mm)	F_u (kN)	Backbone Force (kN)				
				Drift				
				0.5%	1.0%	2.0%	3.0%	4.0%
Wall Height = 2.74 m								
Standard[(a)]	51	2.269	31.68	19.42	26.68	31.6	27.36	22.92
	76	1.861	21.37	14.41	18.75	21.22	18.05	14.88
	102	1.586	16.40	11.49	14.53	16.13	13.69	11.24
	152	1.138	11.20	8.12	10.13	11.01	9.44	7.87
Midply[(b)]	51	2.890	61.53	29.82	46.39	61.52	53.09	44.66
	76	2.514	41.81	23.83	34.75	40.95	35.5	30.05
	102	2.208	31.83	19.76	27.69	30.79	26.77	22.75
	152	1.813	21.70	14.85	19.69	20.93	18.27	15.60
GWB[(c)]	406	0.743	2.03	1.95	1.85	1.37	0.88	0.39
Wall Height = 2.74 m								
Standard[(a)]	51	2.432	32.2	19.15	26.82	32.05	28.13	23.82
	76	2.176	21.94	14.8	19.17	21.87	18.7	15.52
	102	1.740	16.75	11.64	14.91	16.58	14.18	11.79
	152	1.356	11.41	8.34	10.3	11.27	9.65	8.03
Midply[(b)]	51	2.971	63.47	28.28	45.33	62.69	55.8	47.52
	76	2.633	42.67	22.94	34.33	41.95	36.58	31.21
	102	2.396	32.26	19.42	27.56	31.5	27.54	23.57
	152	1.988	22.11	14.79	19.87	21.38	18.76	16.14
GWB[(c)]	406	1.231	2.11	2.04	1.88	1.30	0.73	0.16

(a) Standard wall model is built with 11.9 mm thick OSB connected to framing members by 10d common nails (3.76 mm dia.) in single-shear.
(b) Midply wall model is built with 11.9 mm thick OSB connected to framing members by 10d common nails (3.76mm dia.) in double-shear
(c) Gypsum wall board model is built with 12.7 mm thick GWB connected to framing members by #6 bugle head drywall screws (3.61 mm dia.) in single-shear.

$$F_b(\Delta) = \begin{cases} (1 - \exp(-K_0\Delta / F_0))\left(r_1K_0\Delta + F_0\right) & \text{for} \quad \Delta \le \Delta_u \\ F_u + r_2K_0(\Delta - \Delta_u) & \text{for} \quad \Delta > \Delta_u \end{cases} \tag{2}$$

More details on the modified Stewart hysteretic model can be found in Folz and Filiatrault (2001a). Using the M-CASHEW program, a shear wall database contains the backbone parameters for 2.74 m and 2.44 m tall standard and midply shear walls with field nail spacing of 305 mm and edge nail spacings of 51, 76, 102 and 152 mm was generated (Table 3). The hysteretic parameters for the sheathing nails and dry wall screws used to generate the shear wall database can be found in Pang et al. (2009).

Simplified Direct Displacement Design (DDD) Procedure

The DDD procedure used to design the shear walls of the six-story NEESWood Capstone Building is a simplified version of the recently proposed DDD procedure (Pang and Rosowsky 2009). Table 4 summarizes the design forces for Performance Level 3 obtained using the simplified DDD procedure. The design procedure is briefly described in the following sections and more details on the simplified DDD procedure can be found elsewhere (Pang et al. 2009).

Table 4: Summary of DDD calculations.

Story	h_s (m)	h_o (m)	Δ_{it} (%)	W (kN)	Δ_{it} (mm)	Δ_o (mm)	$W*\Delta_o$ (kN-mm)	C_v	Δ_v	C_v*h_o (m)	$W*\Delta_o^2$ x10^3 (kN-mm^2)	V_s (kN)	K_s (kN/mm)	F (kN)	$F*h_o$ (kN-m)
1	3.05	3.05	2.13	491	65	65	31862	0.057	1.000	0.17	2066	2167	33.41	123	375.7
2	2.74	5.79	2.13	474	58	123	58401	0.104	0.943	0.60	7196	2043	35.01	226	1308.5
3	2.74	8.53	2.13	474	58	182	86064	0.154	0.839	1.31	15629	1817	31.14	333	2841.7
4	2.74	11.28	2.13	474	58	240	113727	0.203	0.685	2.29	27290	1484	25.43	440	4962.1
5	2.74	14.02	2.13	518	58	298	154391	0.276	0.482	3.87	46060	1044	17.89	597	8374.9
6	3.05	17.07	2.13	318	65	363	115563	0.206	0.206	3.52	41971	447	6.89	447	7631.4
Σ				2749	Δ_{eff} =	250	560008	1.000	h_{eff} =	11.77	140212			2167	25494.3

Step 1: The design response spectrum, $\overline{S}_X$**,** is computed as the product of the code specified acceleration response spectrum, S_x, and the **non-exceedance probability adjustment factor, C_{NE}** (i.e., = $C_{NE}S_X$). Since the code specified median spectral value is assumed to be unbiased, the C_{NE} factor is modeled using a lognormal distribution with a median value of 1.0 and a logarithmic standard deviation, β_R.

$$C_{NE} = \exp[\Phi^{-1}(NE_t)\beta_R] \tag{3}$$

where Φ^{-1}(.) is the inverse CDF of the standard normal distribution. The logarithmic standard deviation accounts for the uncertainty of the ground motions, β_{EQ}, as well as the uncertainty associated with the design procedure (i.e., simplified DDD procedure), β_{DS} and is computed as $\beta_R = \sqrt{(\beta_{EQ}^2 + \beta_{DS}^2)}$. Following the ATC-63 study, a fixed value of 0.4 was assumed for the β_{EQ}. Since the simplified DDD procedure does not explicitly account for a number of factors that might affect the actual inter-story drift response (e.g., torsion, higher mode effects or flexible diaphragms) the uncertainties introduced into the

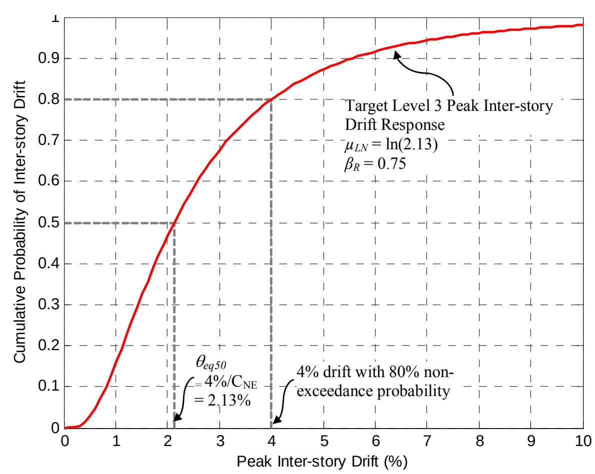

Figure 2: Target story drift distribution curve.

analysis arising from the design assumptions, β_{DS}, was assumed to be 0.6 and the total uncertainty β_R, was determined to be 0.75. At Level 3, the target NE probability of inter-story drift limit is 80%. The C_{NE} for 80% NE probability is computed as $\exp[\Phi^{-1}(0.8)\times 0.75] = 1.88$.

Step 2: The design inter-story drift limit for seismic hazard Level 3 is 4% with an 80% NE probability. **The equivalent 50% NE drift limit θ_{eq50}** can be computed as $4\%/C_{NE} = 2.13\%$ (Figure 2). This equivalent 50% NE inter-story drift limit was used in the displacement-based design of the six-story building (Table 4).

Step 3: The vertical distribution factors for base shear, C_v, were assumed to be propotional to the effective floor weight, W, and the target floor displacement relative to the ground, Δ_o

$$C_{v_j} = \frac{W_j \Delta_{oj}}{\sum_i W_i \Delta_{oi}} \tag{4}$$

where subscript i is the floor number, W is the lumped seismic weight of the floor/roof diaphragm and Δ_o is the target floor displacement relative to the ground (Figure 3). The seismic weights listed in Table 4 were estimated based on the tributary area of the shear walls (i.e., half of the wall weight was assigned to the floor above and half to the floor below).

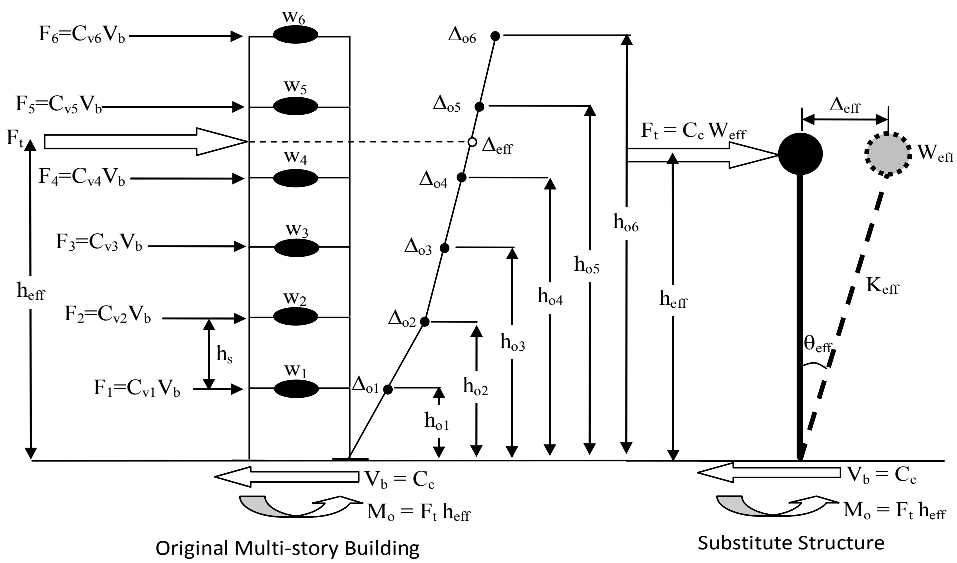

Figure 3: Six-story building and substitute structure.

Step 4: Effective height, h_{eff}, of the *substitute structure* is located at the centroid of the assumed lateral force distribution and is calculated as:

$$h_{eff} = \frac{\sum_i C_{v_i} h_{oi}}{\sum_i C_{v_i} = 1} = \sum_i C_{v_i} h_{oi} \tag{5}$$

where β_{vi} is the story shear factor computed as the sum of the vertical distribution factors, c_{vi}, on and above the i^{th} floor and h_o is the floor height with respect to the ground. The effective height for the six-story Capstone Building was determined to be 11.77 m (Table 4).

Step 5: Target displacement at the effective height, Δ_{eff}

The effective height (11.77 m) for the NEESWood Capstone Building is located between levels 4 and 5 (Table 4). Using interpolation, the effective displacement with respect to the ground level is 250 mm.

Step 6: The effective seismic weight, W_{eff} , of the six-story Capstone Building, computed using the following equation, was 2237 kN which is about 81% of its total weight.

$$W_{eff} = \frac{\left(\sum_i W_i \Delta_{oi}\right)^2}{\sum_i W_i \Delta_{oi}^{\ 2}} \tag{6}$$

The $\sum_i W_i \Delta_{oi}$ and $\sum_i W_i \Delta_{oi}^{\ 2}$ terms are shown in last row of Table 4.

Step 7: Damping reduction factor, B_ξ is calculated using the equation in the ASCE-41 (2006) section 1.6.1.5, B_ξ=4/[5.6-ln(100ξ_{eff})], where ξ_{eff} is the effective viscous damping as a fraction of the critical damping. The ξ_{eff} is computed as the sum of the hysteretic damping, ξ_{hyst}, and the intrinsic damping, ξ_{int}. In the design of the six-story Capstone Building, ξ_{int}= 5% was assumed and the hysteretic damping was estimated using the following equation (Pang et al. 2009):

$$\xi_{hyst} = 0.32e^{-1.38\frac{K_s}{K_o}} \tag{7}$$

From the shear wall database, the secant stiffness (at 2.13% drift)-to-initial stiffness ratio is about 0.30. Substituting K_s/K_o of 0.30 into the equivalent hysteretic damping equation (7) gives an estimated hysteretic damping of 0.21 and the damping reduction factor therefore is 1.71.

Step 8. Design base shear coefficient, C_c

$$C_c = \min\left\{\frac{C_{NE}S_{XS}}{B_\xi};\quad \frac{g}{4\pi^2\Delta_{eff}}\left(\frac{C_{NE}S_{X1}}{B_\xi}\right)^2\right. \tag{8}$$

The capacity spectrum method was used to determine the design base shear coefficient. Equation (8) is the solution for the intersection between the demand and the capacity spectra (Figure 4). For seismic hazard Level 3, the spectral design values for short-period, S_{MS}, and 1-second period, S_{M1}, are 0.9 and 1.5 g, respectively (Table 2). Using equation (8), the base shear coefficient for seismic hazard level 3 therefore is 0.969.

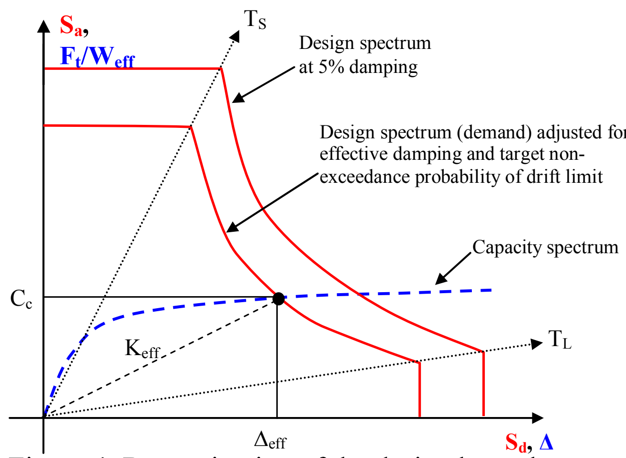

Figure 4: Determination of the design base shear coefficient using capacity spectrum approach.

Step 9. Design forces

Once the base shear coefficient is obtained, the base shear is calculated as $V_b = C_C W_{eff}$.

Equivalent static lateral forces, F_i

$$F_i = C_{v_i} C_c W_{eff} = C_{v_i} V_b \tag{9}$$

Story shears, V_{s_i}

$$V_{s_i} = \beta_{v_i} V_b \tag{10}$$

Overturning moment, M_{o_i}

$$M_{o_i} = \sum_{j=i}^{N_s} F_j \left(h_j - h_i\right) \tag{11}$$

where N_s is the total number of stories (i.e., six for the Capstone Building).
Effective secant stiffness (SDOF), K_{eff}

$$K_{eff} = \frac{C_c W_{eff}}{\theta_{eff} h_{eff}} = \frac{C_c W_{eff}}{\Delta_{eff}} \tag{12}$$

Required secant stiffness for each story,

$$K_{s_i} = \frac{V_{s_i}}{\Delta_{it_i}} \tag{13}$$

From Table 4, the design base shear and overturning moment are approximately 2167 kN and 25494 kN-m, respectively. The required effective secant stiffness of the building at the target drift limit, computed using equation (12), is 8.65 kN/mm. The effective secant period, computed as $2\pi/\sqrt{(g\times K_{eff}/W_{eff})}$, therefore is 1.02 s. Recall that the secant-to-initial stiffness ratio of 0.30 was assumed when determining the hysteretic damping, the minimum initial design stiffness therefore is $K_{eff}/0.30$ = 28.85 kN/mm and the associated initial period is 0.56 s.

Step 10. Select shear walls

The design points, or expected design inter-story drift and required story shear pairs (θ_{it} and V_s), are shown in Table 4. Shear wall nailing schedules were selected from the shear wall database (Table 3). For Level 3, Shear wall backbone forces were taken from the "2% drift" column since the adjusted design inter-story drift was determined to be 2.13%. The design story shears were distributed to wall lines according to their tributary areas. Direct summation of the equivalent stiffness of shear wall segments was used to generate the story backbone curves. The nailing patterns for the shear walls for each floor were determined such that the story backbone curve was above the design points associated with that floor (Figure 5). The complete shear wall nail schedules for each story are given by Pang et al. (2009).

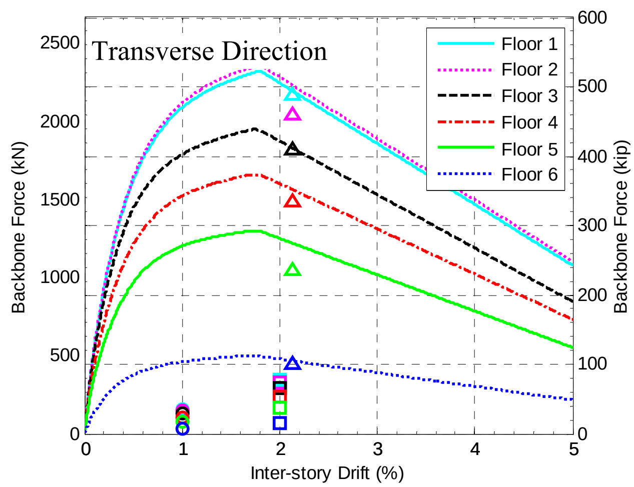

Figure 5: Design points for seismic hazard Level 3 and inter-story backbone curves.

Nonlinear Time-history Analyses

To verify the design requirements were met, a series of nonlinear time-history analyses (NLTHA) were performed using (1): the M-SAWS program (Pang et al. 2009), a Matlab version of the SAWS program (Folz and Filiatrault, 2001b), which considers only the pure-shear deformation of the shear walls, and (2) the SAPWood program (Pei and van de Lindt 2009), which considers the effect of overturning and uplift as well as the vertical ground motion excitation. Since the M-SAWS model does not consider the out-of-plane behavior of diaphragms but the SAPWood model does, the M-SAWS and SAPWood models are here in referred to as the 2D and 3D models, respectively. It was determined that most of the hysteretic damping is accounted for in the nonlinear hysteresis model itself, thus only low level of damping values (2% and 5% of critical dampings) were used in the nonlinear time-history analysis. Two sets of ground motion ensembles were considered in the NLTHA: (1) 22 bi-axial ATC-63 far-field ground motions scaled according to the ATC-63 methodology (ATC 2008) for seismic hazard Levels 1-3, and (2) six bi-axial CUREE unscaled near-fault ground motions (Krawinkler et al. 2003) for seismic hazard Level 4. In NLTHA, the bi-axial far-field ground motions were rotated by 90-degrees and thus, at each performance level, the building was analyzed twice for each of the 22 record pairs for a total of 44 analyses. Similarly, the building was also analyzed using the six pairs of near-fault ground motions rotated at 0 and 90 degrees for seismic hazard Level 4.

Expected Peak Inter-story Drift Distributions

The peak inter-story drift distributions based on results from the 3D and 2D NLTHA are shown in Figure 6. Including the vertical effect generally results in slightly higher peak inter-story drifts than those obtained from the 2D NLTHA (i.e., moving the peak inter-story drift curves to the right). However, for the six-story Capstone Building, the differences in the inter-story drifts between the 2D and 3D models are not felt to be significant. This result is not unexpected because of the aspect ratio (lateral dimension/height approximately equal to one) of the building that makes the dynamic behavior shear-dominant, which is commonly seen in most wood frame building of regular floor plans. In summary, both the 2D and 3D NLTHA indicate that the Capstone Building designed using the simplified DDD procedure satisfies all four design objectives. The median peak drifts at the Levels 1 and 2 were considerably lower than the 1% and 2% drift limits, while the median peak drift at the Level 3 was 1.41% with a 97% probability of not exceeding the 4% drift limit. At Level 4, the probability of exceeding the 7% drift limit was approximately 13% which satisfied the near-fault ground motion performance requirement. The drift profiles (relative to the ground) of two selected earthquake records at the MCE level (2%/50yr) also are shown in Figure 6. It can be seen that the drift profiles are relatively uniform which means the seismic demand was distributed evenly among the stories. In other words, the Capstone Building does not have a significant "weak-story".

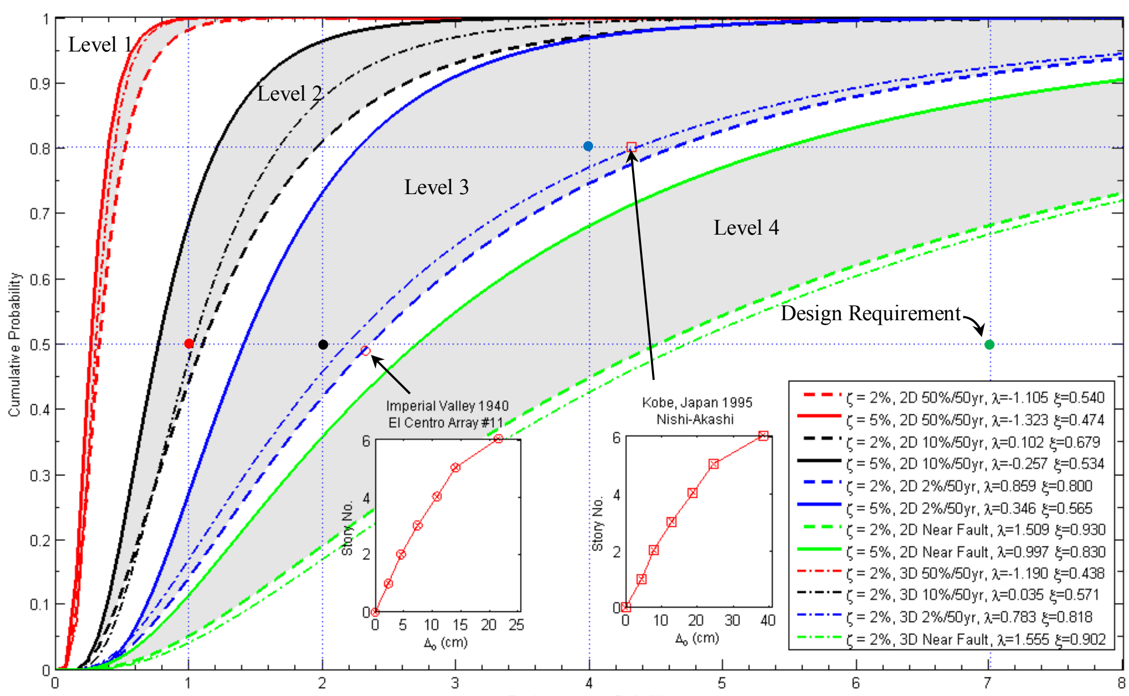

Figure 6: Peak inter-story drift distributions of the NEESWood Capstone Building.

ATC-63 Collapse Margin Ratio

In addition to the NLTHA, collapse margin ratio analysis per the ATC-63 methodology (ATC 2008) also was performed. The ATC-63 methodology was developed for evaluating the collapse risk of structures designed using the current code specified force-based procedures under Maximum Considered Earthquake (MCE) ground motions. To compute the collapse capacity, incremental dynamics analysis was performed using the ATC-63 far-field ground motions. Based on the IDA results (Figure 7a), the unadjusted collapse margin ratio (CMR) is 2.82/1.50 = 1.88. According to the ATC-63 methodology, the raw CMR must be adjusted for the spectral shape before the acceptance criterion can be determined. The spectral shape factor (SSF) is a function of the seismic design category (SDC), ductility of the structure and the upper limit of the code-defined fundamental period of the structure (ATC 2008). The Capstone Building is designed for SDC D_{max} (Southern California regions) and the code-defined period is 0.57 s (per ASCE-07). From the elastic-plastic curve (Figure 7b), the ductility factor, μ_c, is 3.32. Using Table B-4 in the ATC-63 90% draft report, the SSF is 1.22 (ATC 2008). Therefore, the adjusted collapse margin ratio (ACMR), computed as CMR×SSF, is 2.29. The acceptable ACMR value for an individual system (i.e., < 20% collapse probability) depends on the uncertainties of the model and the design procedure. Using the same assumptions as the ATC-63 wood building design examples, the uncertainty in ground motion records is 0.40, design requirement uncertainty (B-Good) is 0.30, test data quality (B-Good) is 0.30, and modeling uncertainty (C-Fair) is 0.45. Thus the composite/total uncertainty, β_{TOT}, is 0.75 (Table 7-2c, ATC 2008). The Capstone Building satisfies the ATC-63 collapse margin requirement, since the ACMR of the (2.09) is higher than the acceptable ACMR for individual building (1.88) determined from Table 7-3 in the ATC-63 90% draft report. Based on the adjusted collapse fragility curve, the collapse probability of the Capstone Building at MCE Level is approximately 16%.

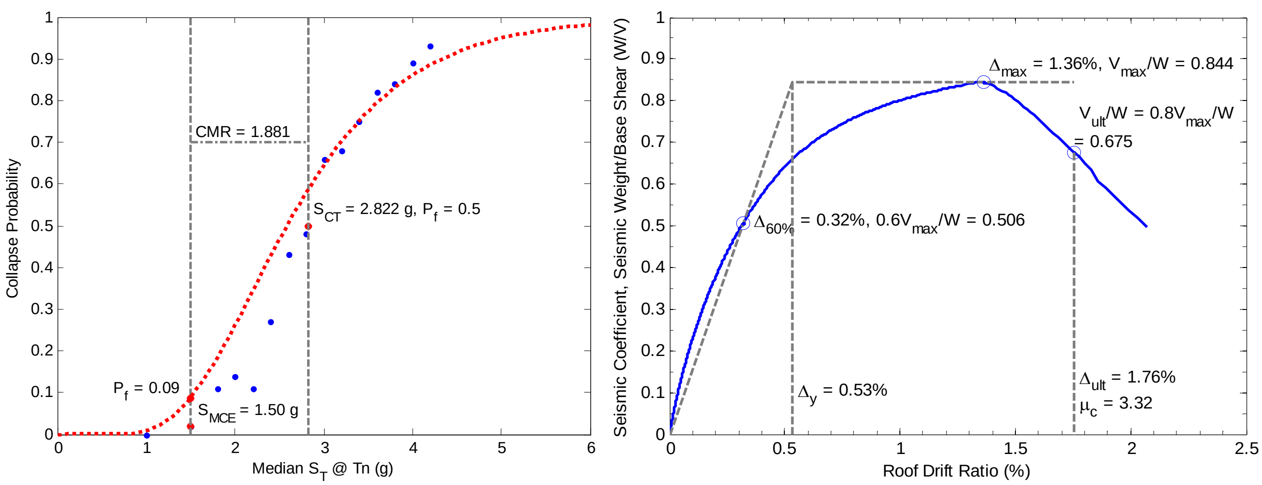

Figure 7: (a) collapse fragility curve, and (b) pushover backbone curve of Capstone Building.

Summary and Conclusion

A simplified direct displacement design (DDD) procedure, which can be used to consider drift limit non-exceedance probabilities other than 50%, was used to design the shear walls of the six-story NEESWood Capstone Building. The proposed design procedure is relatively simple and the shear wall design process can be performed using a spreadsheet. To validate the design procedure, two numerical models (2D and 3D models) were constructed and nonlinear time-history analyses (NLTHA) were performed using the ATC-63 far-field ground motions and a set of near-fault ground motions. The results of the NLTHA confirmed that the Capstone Building designed using the simplified DDD procedure satisfies all four design performance requirements. Additionally, the results of the NLTHA show that the seismic demand was distributed evenly among the stories (uniform drift profiles). Finally, the collapse margin ratio of the Capstone Building under MCE ground motions was determined to be acceptable per the ATC-63 methodology.

References

ASCE 2006. *Seismic Rehabilitation of Existing Buildings* (ASCE/SEI 41-06), American Society of Civil Engineers, Reston, VA.

ATC 2008. *Quantification of Building Seismic Performance Factors*, ATC-63 90%-draft Project Report, Applied Technology Council, Redwood City, CA.

Christovasilis, I.P., Filiatrault, A., and Wanitkorkul, A. 2007. *Seismic Testing of a Full-Scale Two-Story Wood Light-frame Building: NEESWood Benchmark Test*, NEESWood Report NW-01, State University of New York at Buffalo, NY.

ICC 2006. *International Building Code*, Building Officials and Code, International Code Council, Country Club Hills, IL.

Folz, B., and Filiatrault, A. 2001a. "Cyclic analysis of wood shear walls," *ASCE J. Struct. Eng.*, 127(4): 433-441.

Folz, B., and Filiatrault, A. 2001b. *A Computer Program for Seismic Analysis of Woodframe Structures*, CUREE Report W-21, Task 1.5.1, Consortium of Universities for Research in Earthquake Engineering, Richmond, CA.

Krawinkler, H., Zareian, F., Ibarra, L., Medina, R., and Lee, S. 2003. *Seismic Demands for Single- and Multi-story Woodframe Buildings*, CUREE Report W-26, Consortium of Universities for Research in Earthquake Engineering, Richmond, C.A.

Pang, W.C., and Rosowsky, D.V. 2009. "Direct Displacement Procedure for Performance-based Seismic Design of Mid-rise Woodframe Structures," *Earthquake Spectra*, 25(3), 583-605.

Pang, W.C., Rosowsky, D.V., van de Lindt, J.W., and Pei, S. 2009. "Simplified Direct Displacement Design of Six-story NEESWood Capstone Building and Pre-test Seismic Performance Assessment," *NEESWood Report NW-05*, Clemson University, Clemson, SC.

Pei, S. and van de Lindt, J.W. 2009. "Coupled Shear-bending Formulation for Seismic Analysis of Stacked Wood Shear Wall Systems," *J. of Earthquake Engineering and Structural Dynamics*, 38: 1631-1647.

van de Lindt, J.W., Rosowsky, D.V., Filiatrault, A., Symans, M., and Davidson, R. 2006. "Development of a Performance-Based Seismic Design Philosophy for Mid-Rise Woodframe Construction: Progress on the NEESWood Project," *9th World Conference on Timber Engineering*, Portland, OR.

Varoglu, E., Karacabeyli, E., Stiemer, S., Ni, C., Buitelaar, M., and Lungu, D. 2007. "Midply Wood Shear Wall System: Performance in Dynamic Testing," *ASCE Journal of Structural Engineering* 33(7):1035-1042.