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DIRECT DISPLACEMENT PROCEDURE FOR PERFORMANCE-BASED SEISMIC DESIGN OF MULTISTORY WOODFRAME STRUCTURES

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ABSTRACT

Performance-based seismic design (PBSD) is not a new concept. The design philosophy has been advanced for steel and concrete structures for some years. By contrast, however, PBSD concepts for woodframe structures have only been recently started to be explored. An important development in PBSD, which couples specified drift limit states and seismic hazard levels, has been the introduction of direct displacement design (DDD) procedures. In the 1990's, Priestly first suggested displacement-based design using equivalent linearization of SDOF system as an alternative to full nonlinear time-history analysis of engineered structures. Filiatrault and Folz later adapted this approach and proposed a direct displacement design procedure for wood structures modeled using a nonlinear SDOF system. This paper presents a procedure that can be used for PBSD of multistory woodframe structures by modifying the design acceleration response spectrum into inter-story drift spectra which can be used to design and limit the drift level of each story. The proposed procedure does not require nonlinear time history analysis of the complete structure. Instead, only simple modal analysis and approximations of the backbone curves of the participating shearwall segments are needed. These backbone curves are easily obtained either analytically or experimentally. The structure is assumed to have symmetric plan and rigid diaphragms, assumptions that greatly simplify the analysis (no torsion effects), and yet are reasonable for most wood structures of regular plan. A structure designed using the proposed design methodology will simultaneously meet multiple performance levels (e.g., safety and damage limitation requirements) for each story. An example design for a two-story woodframe structure is presented.

Introduction

Approximately 90% of the residential structures in the United States are light-frame wood construction. These residential structures represent the single largest investment for most families or individuals. Current design procedures for wood-frame construction, including for the case of seismic design, are largely prescriptive in nature with one target objective, i.e., to prevent structural collapse. Recent earthquakes (e.g., Loma Prieta 1989 and Northridge 1994) have revealed that even when casualties were limited, the economic losses and social disruption could be enormous and unmanageable. More than half of the approximately $16B loss due to the 1994 Northridge earthquake was caused by damage to wood structures and more than 100,000 individuals were displaced from their homes.

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The vast majority of woodframe structures, even those built in high hazard regions, are engineered to meet strength requirements using deemed-to-comply or other pre-engineered approaches. The implementation of a performance-based seismic design philosophy will require engineers to perform more advanced analyses of complete structures, often comprising a large number of complex and interconnected assemblies. Advances are being made in the area of “whole structure” modelling of wood structures, however it is likely that the majority of these structures will be designed by engineers without access to such modelling capability. Still, techniques will be needed to allow the engineer to evaluate expected structural performance under seismic loading to compare this to target performance requirements (drift limits) without having to resort to fully dynamic modelling and analysis of the structure.

Priestly (1998) first suggested direct displacement design as an alternative to full nonlinear time-history analysis of concrete structures. Filiatrault and Folz (2002) later adapted this approach and proposed a direct displacement design procedure for wood structures that could be adequately modeled using a nonlinear SDOF system. This paper presents a procedure that can be used for multistory wood structures with some advantages over the approach proposed by Folz and Filiatrault (2002). Among them, the proposed procedure does not require the backbone curve of the complete structure (which requires a nonlinear pushover analysis of the complete structure). Instead, only the backbone curves of the participating shearwall segments are needed. These can be easily obtained either analytically or experimentally.

Direct Displacement Design of Multistory Woodframe Structures

The structural model that forms the basis for the proposed displacement-based design procedure for multistory wood structures is derived from the widely used modal analysis approach. The model is based on equivalent linearization of a nonlinear MDOF system in which the story stiffness of the MDOF linear elastic system is estimated with the lower secant stiffness at the target maximum inter-story drift and an equivalent viscous damping ratio that is higher than that of the actual nonlinear system. This design procedure can be used to determine a design configuration (e.g., nailing pattern) that will meet multiple performance requirements. In displacement-based seismic design of multistory wood structures, a performance level is met when inter-story drift of each floor is maintained below a specified target drift limit under a given seismic hazard level. The following sections will describe the formulation of the structural model and the determination of design points to assist with the selection of shearwalls or story backbone curves.

Natural Vibration Frequencies and Modes

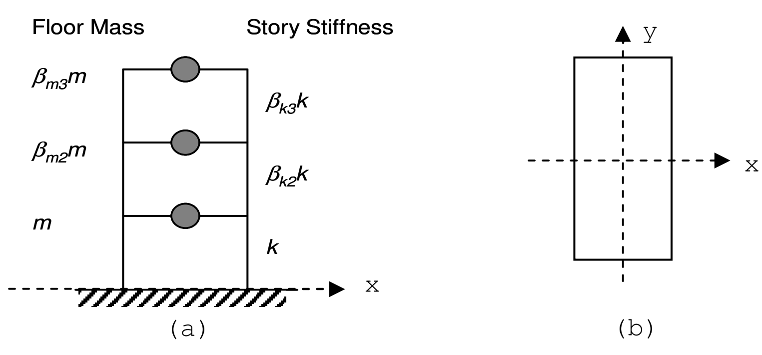

Figure 1. Multistory structure (a) elevation view parallel to x axis, (b) plan view.

Consider a multistory structure as shown in Fig. 1 with symmetric-plan about the x and y axes and having rigid floor diaphragms. This type of symmetric-plan structures can be analyzed independently in the two lateral directions. The natural vibration frequencies, ω_n, and modes, ϕ_{jn}, can be determined by solving the following eigenvalue problem,

$$\left[K-\omega_n^2 M\right]\phi_{jn}=0 \tag{1}$$

where K and M are the stiffness and mass matrices. The mass matrix is a diagonal matrix,

$$M=m\begin{bmatrix}1 & 0 & 0\\ 0 & \beta_{m2} & 0\\ 0 & 0 & \beta_{m3}\end{bmatrix} \tag{2}$$

where m is the total lumped mass for 1st floor diaphragm and β_{mj} is the j^{th} floor mass ratio (relative to the 1st floor). Accordingly, the stiffness matrix is given by

$$K=k\begin{bmatrix}1+\beta_{k2} & -\beta_{k2} & 0\\ -\beta_{k2} & \beta_{k2}+\beta_{k3} & -\beta_{k3}\\ 0 & -\beta_{k3} & \beta_{k3}\end{bmatrix} \tag{3}$$

where k is the 1st-story secant stiffness and β_{kj} is the j^{th} floor secant stiffness ratio (relative to the 1st floor). In design of a new structure, floor masses are usually known values. The lateral stiffness required for each floor is unknown. However, designers can specify or estimate the relative floor stiffness, β_{kj}. Thus, the 1st-floor stiffness, k, is the only design parameter to be determined.

Let k and m equal to unity and solve for the eigenvalue problem, the natural frequencies and period are

$$\omega_n=\alpha_n\sqrt{k/m} \qquad \mathrm{T_n}=\frac{2\pi}{\alpha_n\sqrt{k/m}} \tag{4}$$

Natural frequency parameter, α_n, in eq. 4 is a useful design parameter as it can be used to generate inter-story drift spectra. The natural vibration mode corresponding to each frequency is a vector given by ϕ_{jn}, where subscripts *n* and *j* are mode and floor numbers, respectively.

The extent to which the n^{th} mode is excited by the ground motion is determined by the modal participation factor, Γ_n

$$\Gamma_n=\frac{\sum_{j=1}^{Nfloor}\beta_{mj}\phi_{jn}}{\sum_{j=1}^{Nfloor}\beta_{mj}\left(\phi_{jn}\right)^2} \tag{5}$$

where $Nfloor$ is the total number of stories of the structure. Since, limiting inter-story drift is the main objective for a direct displacement design procedure, a more useful measurement of the contribution of each mode to the total inter-story drift is defined here as inter-story drift factor, γ_{jn}:

$$\gamma_{jn}=\Gamma_n\left(\phi_{jn}-\phi_{j-1,n}\right) \tag{6}$$

Once α_n and γ_{jn} are determined, the design inter-story drift spectra can be generated. The process of generating inter-story drift spectra will be discussed later.

Design Performance Levels

The traditional force-based seismic design procedure for woodframe construction is largely prescriptive in nature with one main target objective, i.e., to prevent structural collapse. The recent major earthquakes (e.g., 1989 Loma Prieta and 1994 Northridge) demonstrated that woodframe structures built per prescriptive force-based design code, such as the 1988 Uniform Building Code (ICBO, 1988), were relatively effective in preventing structural collapse but much less effective in reducing economic losses.

In order to overcome the drawbacks of prescriptive force-based design procedures, performance-based engineering concepts were introduced and have gained momentum in North America in recent years. Performance-based design procedures are formulated such that the resulting design will simultaneously meet multiple objectives or performance levels. In the case of seismic design, seismic hazard levels are coupled with target drift limits to form these performance levels.

Two performance limit states, immediate occupancy (IO) and life safety (LS), defined in the FEMA 356 (2000), are adopted as the design objectives to illustrate the proposed direct displacement design procedure. Of course there is no limit to the number of hazard level / drift limit pairs that can be included. Table 1 shows the FEMA 356 IO and LS performance requirements for wood shearwalls.

Table 1. Immediate occupancy and life safety performance levels for wood shearwalls.

Limit State	**Seismic Hazard**	**Drift Limits**
Immediate Occupancy	50%/50yr	1%
Life Safety	10%/50yr	2%

The objective of the IO performance level is to minimize significant economic loss when subjected to a moderate earthquake event having a 50% probability of exceedence in 50 years. Damage to the structure or the resisting shearwalls is expected to be minor if the transient drift is kept below 1% of the story height. In general, structural repair is not required prior to re-occupancy. For a more severe seismic event (10% probability of exceedence in 50 years), the objective changes from loss prevention / minimization to reduction of risk of life-threatening injury to the occupants. It is assumed that with a maximum of 2% transient drift permitted, significant damage to the structure has occurred and the stiffness and strength have degraded but still provide a margin of safety against partial or total collapse.

Equivalent Viscous Damping Ratio

The seismic hazard associated with the performance level must be defined in terms of the design response spectrum with an equivalent viscous damping ratio, ζ_{eq}. One possible empirical equation for ζ_{eq} was developed in this study for the hysteretic damping experienced by a full nonlinear MDOF system subjects to actual ground motions.

$$\zeta_{eq} = \begin{cases} 8\Delta & for\ \Delta \le 2.5\% \\ 20 & for\ \Delta > 2.5\% \end{cases} \tag{7}$$

where ζ_{eq} is in percentage of critical damping and Δ is the target inter-story drift in percentage of story height. This empirical equation was determined by matching the median peak drift responses of the full nonlinear time-history analyses to the drift responses determined using the equivalent linear elastic MDOF system. The SAWS (Seismic Analysis of Wood Structures) program (Folz and Filiatrault, 2004) along with unscaled ground motion records obtained from the PEER Strong Motion Database was used for the nonlinear time-history analyses. For the equivalent linear elastic MDOF system, design response spectra generated at various damping ratios were used to determine the peak inter-story drifts.

Fig. 2 shows selected results of the first-floor peak inter-story drift distribution predicted using SAWS for a two-story woodframe structure. Contour lines for the first-floor drift determined using the equivalent linear system for damping ratios ranging from 5% to 20% are also shown. The damping ζ_{eq} increases about 8% per every 1% increment of story drift and reaches a conservative maximum of 20% for story drift larger than 2.5%. Other studies of equivalent viscous damping based on the results of cyclic pushover analyses of woodframe structures showed that the maximum equivalent viscous damping ratio is about 18% of critical (Filiatrault et al. 2004).

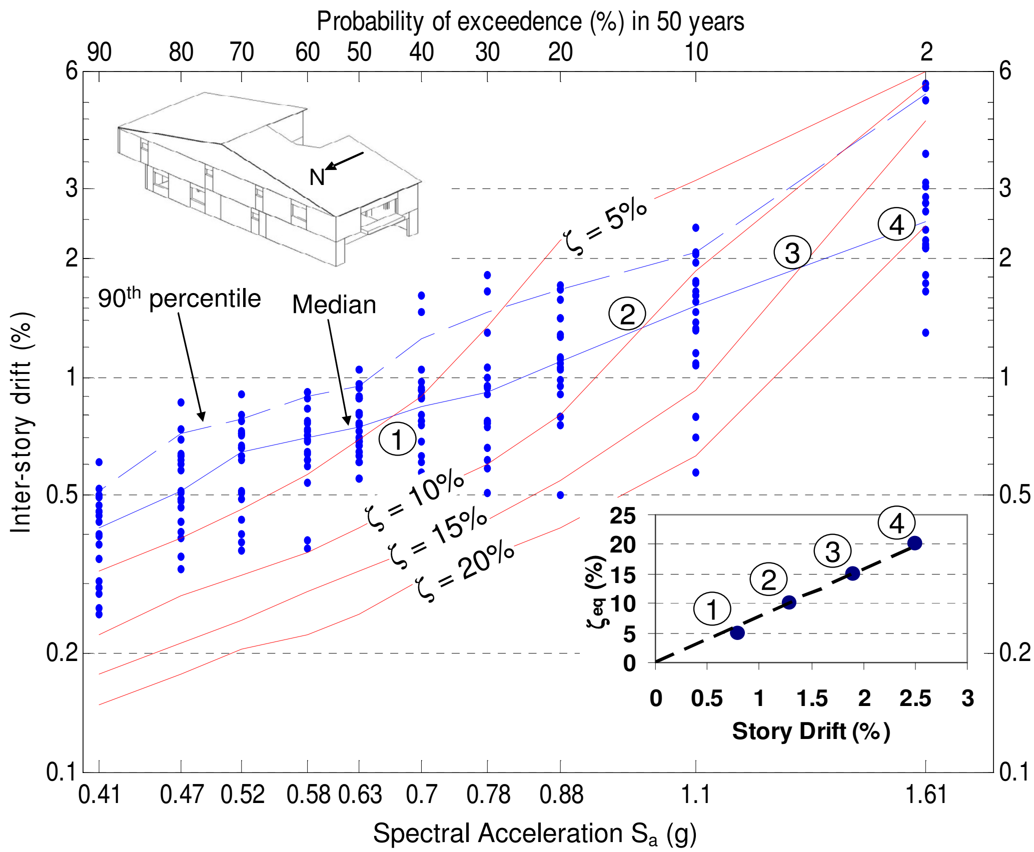

Figure 2. Peak first-story drift distribution of a two-story woodframe structure.

Design Inter-story Drift Spectra

The United States Geological Survey (USGS) had developed a set of national earthquake hazard maps. These maps can be used to generate horizontal acceleration design spectra using the procedure described in section 1.6.1.5 of the FEMA 356 (2000) for different seismic hazard levels. Based on Eq. 7, values of ζ_{eq} of 8% and 16% can be used for the IO and LS performance levels, accordingly. Fig. 3(a) shows an example LS performance level (10%/50yr) design acceleration response spectrum for Los Angeles, California, assuming soil type D and an equivalent viscous damping ratio of 16%. The acceleration response spectrum can be converted into a displacement response spectrum using eq. 8:

$$S_d = \left(\frac{T}{2\pi}\right)^2 S_a \tag{8}$$

where T is the elastic period of the complete structure and S_a and S_d are the spectral acceleration and spectral displacement, respectively.

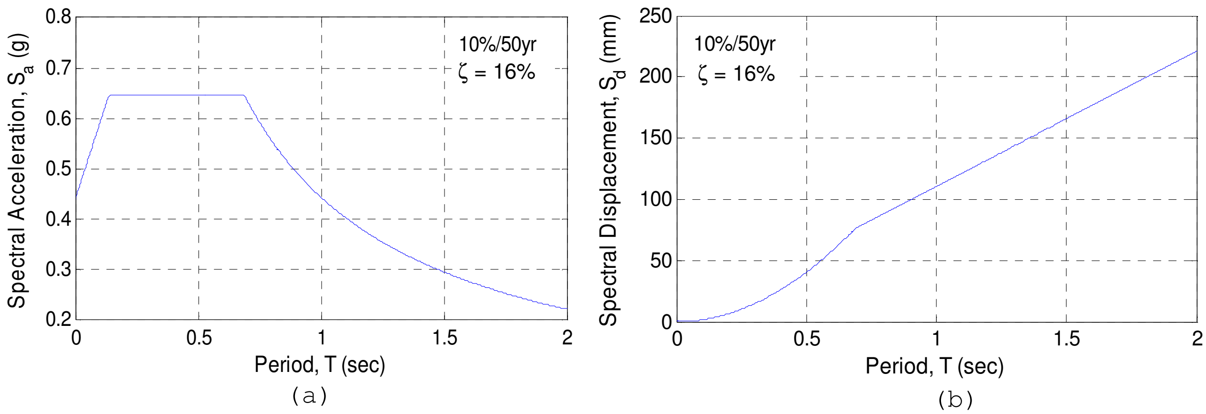

Figure 3. (a) Acceleration and (b) displacement design spectra for life safety performance level.

Fig. 3(b) represents the displacement response of the overall structure modeled as an elastic SDOF system. In performance-based seismic design, inter-story drift is used as a metric to quantify the damage state or performance of the structures. Therefore, the displacement-based design procedure is herein formulated in terms of the inter-story drift response.

The previously defined natural frequency parameter, α_n , and the inter-story drift factor, γ_{jn} can be used to create the inter-story drift spectra. The first step to generating an inter-story drift spectrum is to plot the contribution of each mode to the total drift of the floor considered. Fig. 4 illustrates the process of constructing an inter-story drift spectrum normalized to the first-floor period, $\bar{T}$. Multiplying the elastic period (x-axis) and the spectral displacement (y-axis) in Fig. 3(b) by α_n and γ_{jn} , respectively, produces the contribution of each mode to the story-drift. The square-root-of-sum-of-squares (SRSS) rule is used to obtained the total inter-story drift, Δ_j , for each value of $\bar{T}$.

$$\Delta_j = \sqrt{\sum_n \left(\gamma_{jn} S_d\right)^2} \tag{9}$$

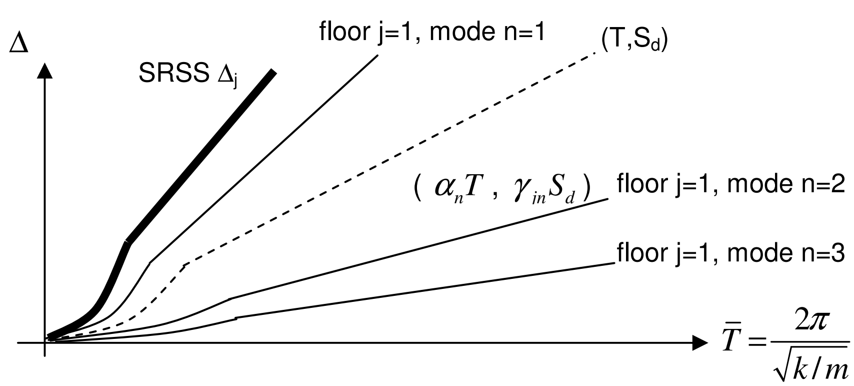

Figure 4. Construction of an inter-story drift spectrum.

Required Equivalent 1st-floor Period and Stiffness

The governing or the 'weakest' floor in a building can be determined by combining the inter-story drift spectrum for each floor into one figure. Consider, as an example, the inter-story drift spectra for a three-story building shown in Fig. 5. The steepest drift response curve corresponds to the floor in a building that will reach the design drift limit first. For the LS performance level, the drift limit is 2%. A required equivalent 1st-floor period, $\bar{T}_{eq}$,can be obtained by locating the point on the inter-story drift for the governing floor that corresponds to 2% drift.

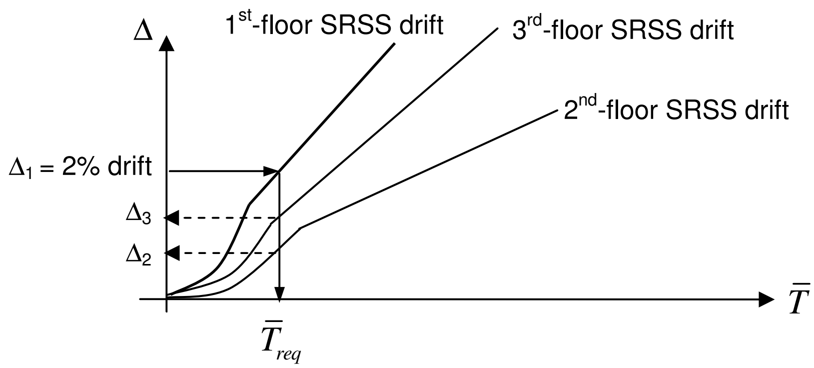

Figure 5. Inter-story drift spectra of a 3-story building.

Knowing the required equivalent period, $\bar{T}_{eq}$, the story drifts for the remaining floors (Δ_2 and Δ_3) at the design limit can be obtained directly from Fig. 5. The required 1st-floor secant stiffness, k_{eq}, can be calculated using Eq. 10 once the required equivalent period is determined.

$$k_{eq} = \left(\frac{2\pi}{\bar{T}_{eq}}\right)^2 m \tag{10}$$

Using the story stiffness ratios of Eq. 3, the required secant stiffness for 2nd and 3rd floors are $\beta_{k2}k_{eq}$ and $\beta_{k3}k_{eq}$. Accordingly, the required story backbone force, $F_{req\,j}$, for the jth floor is:

$$F_{req\,j} = k_{req}\beta_{kj}\Delta_j \tag{11}$$

Design Points for Story Backbone

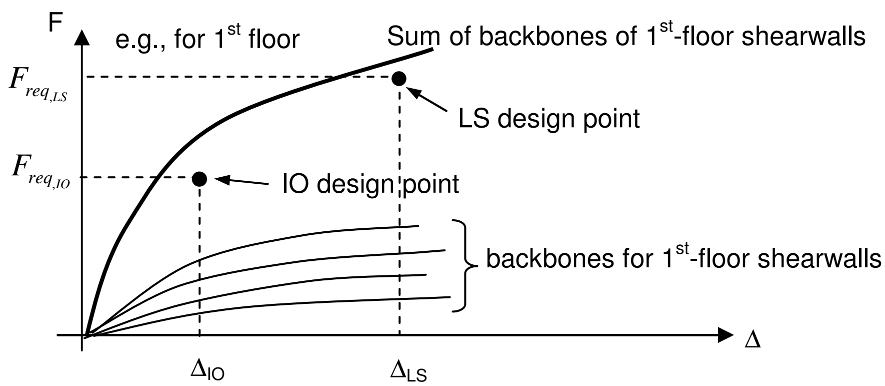

Figure 6. Design points for story backbone curve.

Plotting the inter-story drift and the required story backbone force at the drift level forms a design backbone point that is required to achieve the specified performance level. Fig. 6 shows an example LS design point for the 1st-floor backbone curve. Based on Fig. 5, the 1st-story controls, e.g. will reach the LS design drift limit of 2% first. The required 1st-story backbone force for LS, $F_{req,LS}$, at the design drift limit of 2% is then calculated using Eq. 11. A design point for the story backbone curve can be determined similarly for each floor. To meet additional performance requirements, e.g., IO, the design process is repeated using the design inter-story drift spectra for (e.g.) the 50%/50yr hazard level and a design point for IO, as shown in Fig. 6, can be obtained.

In order to meet these two performance levels simultaneously, the story backbone curve should pass through or exceed the minimum IO and LS design points. As previously mentioned, the structure is assumed to have symmetric plan and rigid diaphragms (no torsional effects). As a result, the story backbone is simply the sum of the backbone curves of the participating shearwall segments. The backbone curves of shearwall segments can be obtained either analytically or experimentally.

Example

Description of Woodframe Structure

Fig. 7 shows a two-story woodframe townhouse structure built for the first shake table testing program of the NSF/NEES funded NEESWood project. The height of the structure from the base to the eaves of the roof is 5.49 m, with a story height of 2.74 m, and the structure has approximately 150 m^2 of living space. The exterior walls of the structure are sheathed with 11 mm thick oriented strand board (OSB) connected to the frames using 8d common nails (63.5 mm long x 3.3 mm in diameter). All framing materials for the shearwalls are nominal 51 mm x 102 mm dimension lumber except those shearwalls located on the west side of the 1st floor (garage wall), where 51 mm x 152 mm dimension lumber was used. The effective seismic weights for the 1st and 2nd floors are 159 kN and 166 kN, respectively. In this example, only the weak direction (North-South) of the structure is analyzed.

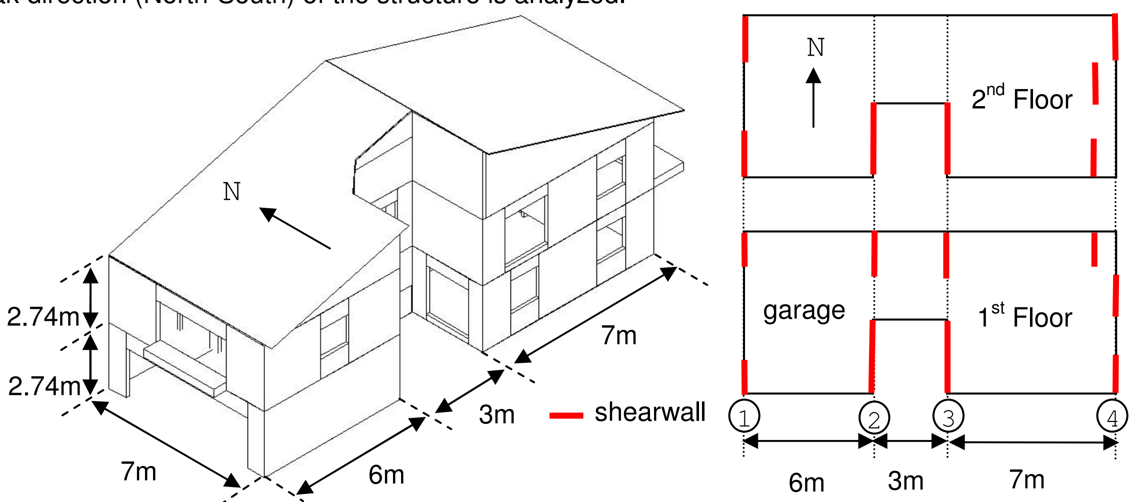

Figure 7. Full-scale two-story woodframe structure.

Nailing Patterns Considered

The nail spacing for the second floor shearwalls was kept constant with panel interior and exterior nail spacing equal to 300 mm and 150 mm, respectively. The interior nail spacing for the first floor shearwalls was also kept constant at 300 mm. Thus the only design variable was the exterior nail spacing for the first floor shearwalls. Four possible exterior nailing patterns (Table 2) for the first floor shearwalls were considered. The story backbone of each design shown in Table 2 was obtained analytically using the CASHEW (Cyclic Analysis of Shearwall) program (Folz and Filiatrault, 2001). The story backbone curves for all four nailing patterns are shown in Fig 8. Note that the peak force of the 1st floor backbone curves reduces from design 1 to 4 as the interior nail spacing was increased.

Table 2. Four possible nailing patterns for the first-floor shearwalls.

Design	First Floor Shearwall Panel Perimeter Nail Spacing (mm)			
	Line 1	Line 2	Line 3	Line 4
1	100	150	100	75
2	100	150	100	150
3	100	150	150	150
4	150	150	150	150

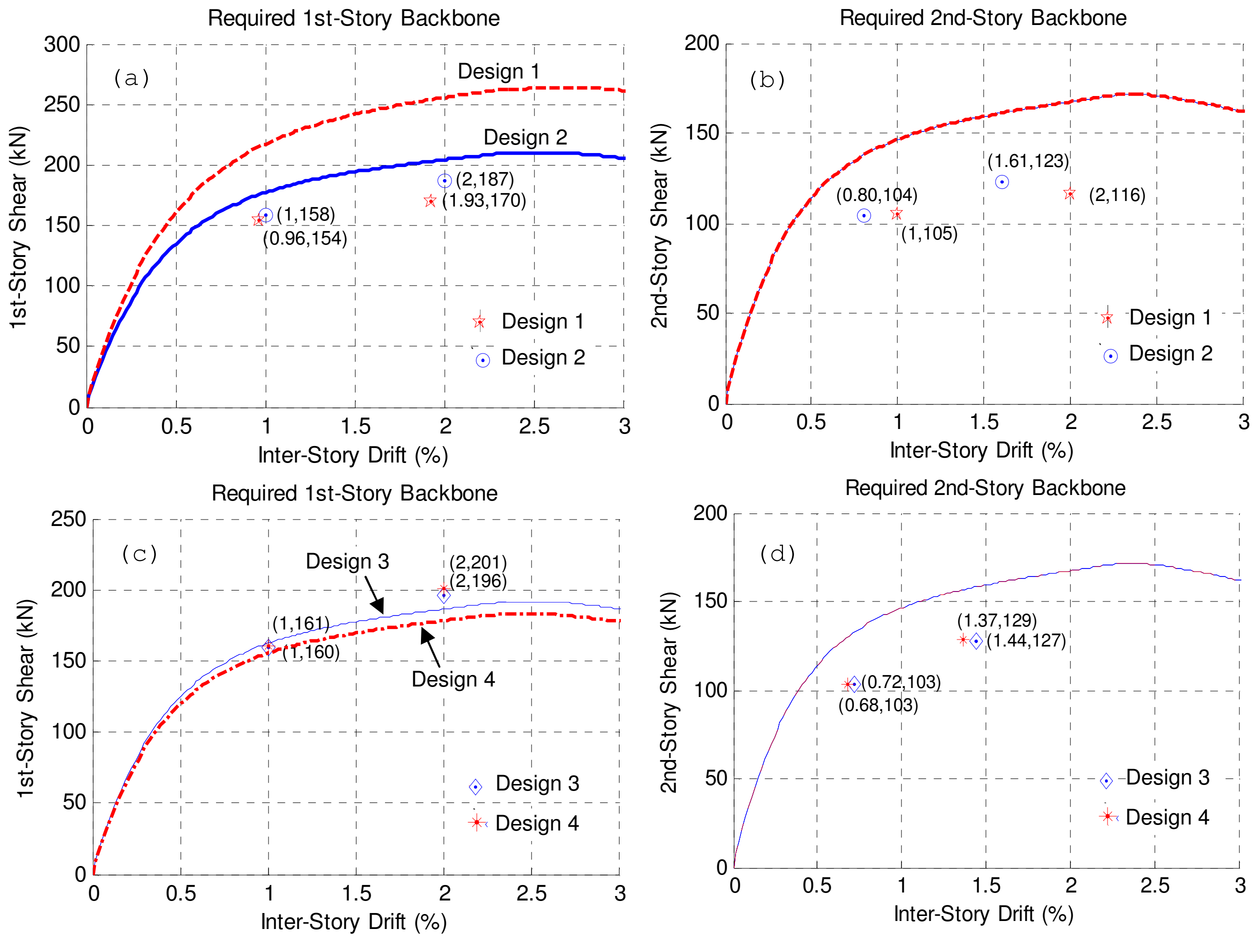

Figure 8. Design backbone points for the four nailing patterns.

Using the previously discussed direct displacement procedure, the design points for IO and LS performance levels were determined. The value for each design point is given in Table 3 and also is shown graphically in Fig. 8. Design 1 was determined according to the seismic provisions of the 1998 Uniform Building Code (ICBO, 1988). Examine Fig. 8 shows that both designs 1 and 2 meet the IO and LS performance requirements, since the design points are below the backbone curves. In this example, the force-based design meets both IO and LS performance levels. However, design 2 is closer to both target requirements and therefore can be considered a more economical design. Fig. 8(c) shows that design 3 meets IO performance level but it fails the LS requirement. Design 4 with wider nail spacings fails both the IO and LS performance requirements at the first floor (Fig. 8(c)) since both design points are above the story backbone curve provided by nailing pattern 4.

Table 3. Example direct displacement design for a two-story woodframe structure.

Design	β_{k2}	m	β_{m2}	α_1	α_2	Γ_1	Γ_2
	unitless	kN/g	unitless	unitless	unitless	unitless	unitless
1	0.66	159/g	1.0446	0.5661	1.4040	-1.3549	-0.4569
2	0.82			0.5897	1.5024	-1.3760	-0.3890
3	0.90			0.5988	1.5502	-1.3835	-0.3613
4	0.94			0.6028	1.5737	-1.3867	-0.3488

Design	Immediate Occupancy						Life Safety					
	T_{eq}	k_{eq}	Δ_1	Δ_2	F_{req1}	F_{req2}	T_{eq}	k_{eq}	Δ_1	Δ_2	F_{req1}	F_{req2}
	s	kN/mm	mm	mm	kN	kN	s	kN/mm	mm	mm	kN	kN
1	0.332	5.82	26.5	27.4	154	105	0.446	3.22	52.8	54.9	170	116
2	0.333	5.77	27.4	22.0	158	104	0.434	3.41	54.9	44.1	187	123
3	0.331	5.83	27.4	19.7	160	103	0.423	3.58	54.9	39.5	196	127
4	0.331	5.86	27.4	18.7	161	103	0.418	3.66	54.9	37.5	201	129

Conclusions

This paper presented a direct displacement design procedure for the seismic design of multistory woodframe structures of regular plan. The proposed approach does not require a nonlinear pushover analysis of the complete structure. Instead, only the backbone curves of the shearwall segments are needed. These backbone curves can be obtained through experimental testing of shearwalls. Alternatively, analytical model can also be used to determine the backbone curves of the participating shearwalls. This design procedure is suitable for performance-based seismic design since it can be used to determine a design configuration that will meet multiple performance requirements simultaneously. As an example, the design of a two-story woodframe structure is presented. The direct displacement design procedure was used to identify nailing patterns that will meet both the immediate occupancy and the life safety performance requirements. It was shown that the proposed procedure is able to determine the location or the floor that fails the performance requirement, and in turn can assist designers in selecting appropriate replacement shearwalls at this design-critical location. The procedure can, of course, be used for any number of hazard level / drift limit pairs specified in a multi-level PBSD framework.

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