Displacement-Based Seismic Design of Light-Frame Wood Buildings

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1. Summary

This paper discusses recent developments in the direct displacement-based seismic design for light-frame wood buildings. The underlying design philosophy and methodology of direct displacement-based seismic design are first presented. In this design approach, the wood structure is modeled as an equivalent single-degree-of-freedom system with equivalent secant lateral stiffness and equivalent viscous damping ratio representative of the global behavior of the structure at the design displacement. To evaluate these properties, specialized numerical models to predict the seismic response of wood shear walls and complete three-dimensional light-frame wood buildings are discussed. Finally, an example of the direct displacement-based design of a light-frame wood building is presented.

2. Introduction

Performance-based seismic design of structures is based on coupling multiple performance limit states with specified seismic hazard levels. This design philosophy overcomes several of the shortcomings of the traditional force-based seismic design procedure, which has been the cornerstone of building code requirements to date. Although the performance-based seismic design approach has advanced for some types of structures, its application to light-frame wood buildings has only recently been formulated [1].

Since inter-story drift is a key parameter for the control of damage in wood buildings, it is rational to examine a performance-based seismic design procedure wherein displacements are at the core of the design process. In this regard, the direct displacement-based approach, originally proposed by Priestley [2], [3] for reinforced concrete structures, is an appropriate, straightforward seismic design procedure to adopt for wood buildings. Direct displacement-based seismic design assumes that the structural system can be represented by a single-degree-of-freedom (SDOF) model with equivalent elastic lateral stiffness and viscous damping ratio representative of the characteristics of the original structure at a target lateral displacement.

The main objective of this paper is to discuss recent developments towards the development of direct displacement-based seismic design for light-frame wood buildings and provide an example of its application.

3. Overview of Direct Displacement-Based Seismic Design

The basic elements of the direct displacement-based seismic design procedure for light-frame wood buildings are briefly summarized in this section. A more detailed presentation of this design approach has been provided previously [1].

The central concept of the direct displacement-based approach, as originally proposed by Priestley [2] [3], is that the seismic design of a structure is based on a specified target displacement for a given seismic hazard level. For this purpose, the structure is modeled as a SDOF system with equivalent elastic lateral stiffness and viscous damping properties representative of the global behavior of the actual structure at the target displacement. The first step in this design procedure is the definition of the target displacement Δ_t that the building should not exceed under a given seismic hazard level. The seismic hazard associated with the target displacement must then be defined in terms of a design relative displacement response spectrum corresponding to the equivalent viscous damping exhibited by the structure at the target displacement.

In order to capture the energy dissipation characteristics of the structure at the target displacement, an equivalent viscous damping ratio must be determined. Based on the results of cyclic pushover analyses conducted on four different full-scale index building models [4], it was observed that the equivalent viscous damping ratio remains fairly constant with building drift ratio. Consequently, the variation of equivalent viscous damping ratio ζ_{eq} with building drift ratio δ can be conservatively represented by the following empirical formula:

$$\zeta_{eq} = \begin{cases} 0.5\delta & for \;\; \delta \le 0.35\% \\ 0.18 & for \;\; \delta > 0.35\% \end{cases} \qquad [1]$$

Knowing the target displacement and the equivalent viscous damping of the building at the target displacement, the equivalent elastic period of the building T_{eq} can be obtained directly from the design displacement response spectrum. With the building represented as an equivalent linear SDOF system, the required equivalent lateral stiffness k_{eq}^r is given by:

$$k_{eq}^r = \frac{4\pi^2 W_{eff}}{g\,T_{eq}^2} \qquad [2]$$

where W_{eff} is the effective seismic weight acting on the building and g is the acceleration of gravity.

The actual equivalent lateral stiffness k_{eq}^a of the building at the target displacement Δ_t can be determined from the results of a static pushover analysis. The actual equivalent lateral stiffness of the building must be compared to the required equivalent lateral stiffness. If these two stiffness values differ substantially, the lateral-load resisting system of the building must be modified. If the actual lateral stiffness of the building is nearly equal to the required lateral stiffness, the design process is completed by computing the required base shear capacity V_b of the building:

$$V_b = k_{eq}^a \, \Delta_t \qquad [3]$$

This base shear can then be used to design the other structural elements of the structure.

4. Design Example

4.1 Building Configuration

As an illustrative example of the use of the direct displacement-based seismic design procedure, one of the index buildings developed under the CUREE-Caltech Woodframe Project in California for use in loss estimation and benefit-to-cost ratio analysis is considered [5]. This index building represents a two-story townhouse containing three units, each having approximately 150 m^2 of living space with an attached two-car garage, as shown in Fig. 1. It is on a level lot with a slab-on-grade and spread foundations. This building is assumed to have been built as a “production house” in either the 1980’s or 1990’s, located in either Northern or Southern California. The design is based on engineered construction using the 1988 edition of the Uniform Building Code [6]. The height of the townhouse building from the first floor slab to the roof eaves is 5.49 m and its weight is 840 kN.

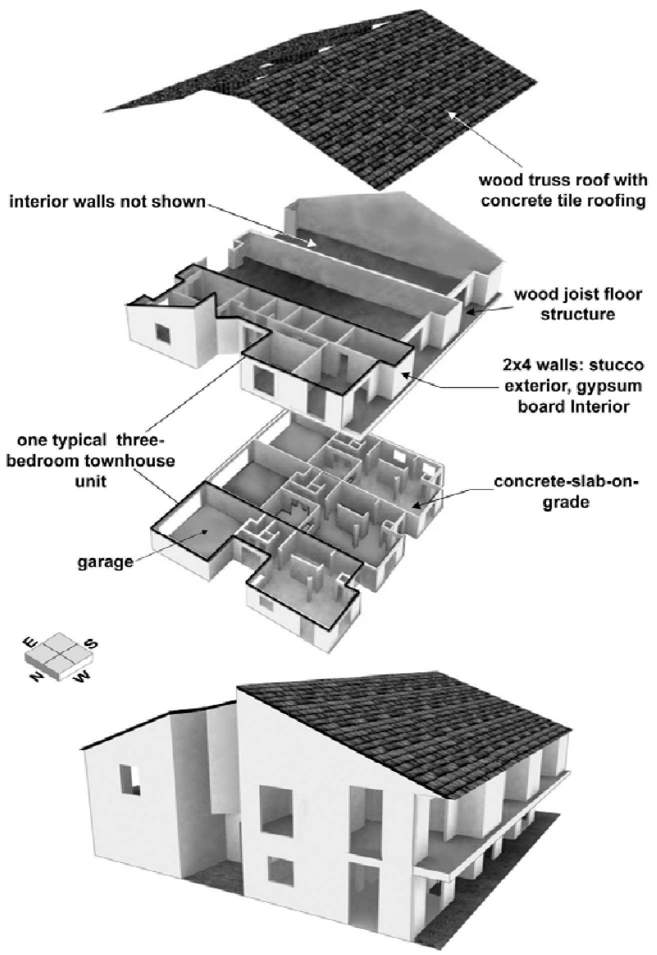

Fig 1 Townhouse index building [6]

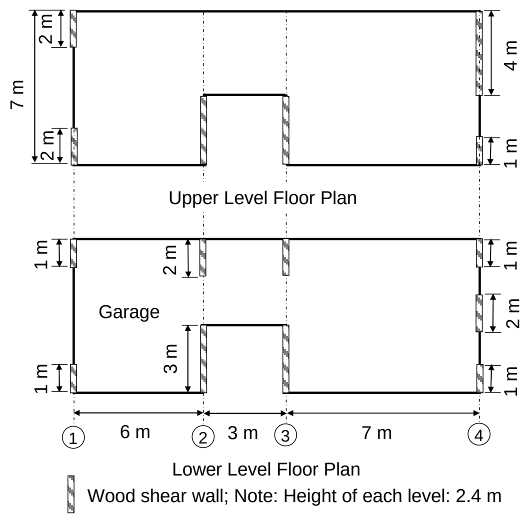

Fig 2 Floor plans for index building.

Seismically relevant characteristics that were intentionally featured in this townhouse building include the integral garage and for the end units, the imbalance in plan stiffness.

Only one townhouse unit of the building (weighing 280 kN) was considered in this example. Figure 2 shows plan views of both floors indicating the locations of the shear walls acting as the lateral load-resisting system in the North-South direction. Only this direction was considered for this example.

All framing materials for the shear walls are 38 mm x 89 mm dimensional lumber. The top plates and end studs consist of double members, while the sole plate and the interior studs are single members. Studs are spaced at 400 mm on center. Conventional corner hold-downs are used to prevent overturning of the walls and to ensure a racking mode of deformation. The sheathing panels are 9.5 mm thick oriented strand board (OSB), with an assigned elastic shear modulus of 1.5 GPa, installed vertically. The sheathing-to-framing connectors are pneumatically driven 50 mm long spiral nails.

It is assumed that the building is located in a seismic zone 4 and founded on a soil type D according to the 1997 edition of the Uniform Building Code [8]. The seismic design is to be performed for the life safety performance level with an associated limit interstory drift of 2% in any wall line. The nailing pattern for the upper level walls was kept constant at 150 mm spacing at panel edges and 300 mm spacing at all interior studs. Five different potential nail patterns are considered for the lower level walls of the building, as shown in Table 1. Nailing Pattern No. 1 represents the original design of the building according to the 1988 Uniform Building Code force-based procedure [7].

Table 1 Nailing Patterns for Lower Level Walls of Index Building

Nailing Pattern No.	Nail Spacing at Panel Edges (mm)*			
	Line 1	Line 2	Line 3	Line 4
1**	100	150	100	75
2	75	150	100	75
3	75	75	75	75
4	100	100	100	100
5	150	150	150	150

*Nail spacing for all interior studs = 300 mm for all nailing patterns
**Original design of the building according to 1988 UBC force-based procedure

4.2 Definition of Seismic Hazard

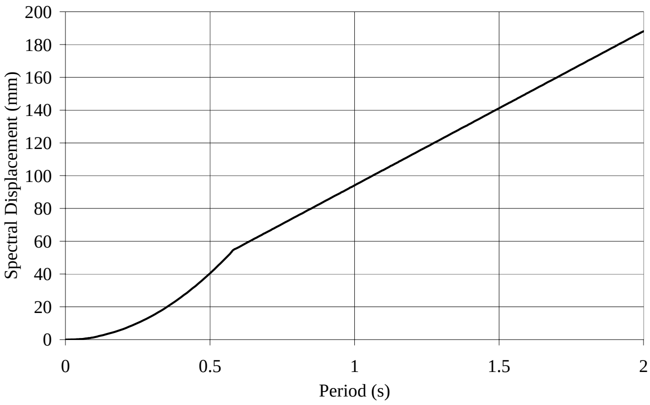

Fig 3 Design displacement spectrum, 18% damping

Figure 3 presents the design relative displacement response spectrum for the life safety performance level considered and for an equivalent viscous damping ratio of 18% of critical, using Equation [1]. This design relative displacement response spectrum was obtained by modifying the absolute acceleration response spectrum contained in the 1997 edition of the Uniform Building Code [8] for a seismic zone 4 and soil type D, $S_{A\ Code}$, into corresponding spectral displacement values $S_{D\ \zeta_{eq}}$ [1]:

$$S_{D\ \zeta_{eq}} = \left[\frac{T_{eq}^2}{4\pi^2}\right]\left[\sqrt{\frac{0.07}{0.02+\zeta_{eq}}}\right] S_{A\ Code} = 0.015 T_{eq}^2 S_{A\ Code} \qquad [4]$$

where T_{eq} is the equivalent elastic period of the building at the target displacement Δ_t.

4.3 Pushover Analysis

Monotonic pushover analyses were performed in order to determine the envelope of the Base Shear – Roof Central Displacement relationship for the index building in the North-South direction for the five different nailing patterns of the shear walls described above. For this purpose, the computer software SAWS: (Seismic Analysis of Wood Structures) was used [9], [10]. In SAWS, the building structure is composed of two primary components: rigid horizontal diaphragms and nonlinear lateral load resisting shear wall elements. The actual three-dimensional building is degenerated into a two-dimensional planar model using zero-height shear wall spring elements connected between the diaphragms and the foundation. The pinched, strength and stiffness degrading hysteretic behavior of each wood shear wall in the building can be characterized using an associated numerical model that predicts the walls load-displacement response under general quasi-static cyclic loading [11]. The hysteretic behavior of each shear wall is represented by an equivalent nonlinear shear spring element. With this approach the response of the building is defined in terms of only three-degrees-of-freedom per floor.

For the pushover analysis, lateral loads were applied in proportion to the weight distribution at each floor level. The loading was applied until the first wall in the building reached an interstory drift of 2% corresponding to the life safety performance level. Figure 4 presents the results of the monotonic pushover analyses for the index building along its North-South direction for the five different shear wall nailing patterns considered. On each curve, the central roof displacement corresponding to the first wall in the building (Line 1, bottom level) reaching the inter-story drift to 2% is indicated by a dot. As a result of the significant torsional response of the building, when the first shear wall in the building reaches an inter-story of 2%, the roof central displacement is less than the value corresponding to a building drift of 2% (96 mm).

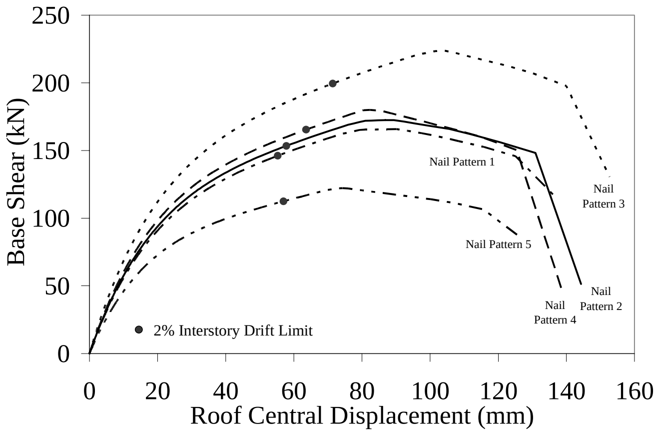

Fig 4 Pushover analysis results

4.4 Equivalent Natural Period

Using the life safety design response spectrum for 18% damping in Fig. 3 at the target roof central displacement associated with 2% interstory drift limit (indicated by a dot in Fig. 4) an equivalent natural period can be obtained for each nailing pattern considered, as reported in Table 2.

4.5 Required Equivalent Lateral Stiffness

Using Equation [2], a required equivalent lateral stiffness can be computed for each nailing pattern to satisfy the life safety performance level. This value is also given in Table 2.

Table 2 Displacement-Based Design Example

Lower Level Shear Wall Nailing Pattern No.	ζ_{eq}	Roof Central Displacement @ 2% Inter-story Drift Limit (mm)	T_{eq} (s)	k_{eq}^{r} (kN/mm)	k_{eq}^{a} (kN/mm)	k_{eq}^{a}/k_{eq}^{r}
1	0.18	55.3	0.59	3.24	2.64	0.81
2		57.8	0.62	2.93	2.65	0.82
3		71.4	0.76	1.95	2.79	1.43
4		63.6	0.68	2.44	2.60	**1.07**
5		57.0	0.61	3.03	1.97	0.65

4.6 Actual Equivalent Lateral Stiffness and Final Design

From the pushover curves shown in Fig. 4, the actual equivalent lateral stiffness of each nailing pattern can be obtained at the target displacement. These values are shown in Table 2 and are compared to the required equivalent lateral stiffness. Nailing Patterns No. 3 and No. 4 meet the life safety performance objective. Nail Pattern No. 4 meeting more closely the design objective and being more economical is adopted as the final design. Note that Nailing Pattern No. 1, corresponding to the original force-based design of the building according to the 1988 Uniform Building Code, does not meet the life safety performance objective according to the proposed direct displacement-based approach.

5. Design Appraisal

In order to evaluate the performance of the index building designed according to the direct displacement-based approach, non-linear time-history dynamic analyses were performed in the North-South direction of the index building for an ensemble of 20 ground motions consistent with the seismic hazard associated with the life safety performance level considered in the design process. Once again, the SAWS software was used for this purpose.

Table3. Set of 20 Ground Motion Records Used for Dynamic Analyses

Earthquake Event / Year	Station	Peak Ground Acceleration (g)	
		Actual	Scaled for Life-Safety
Superstition Hills 1987	Brawley	0.116	0.604
	El Centro Imperial County Center	0.258	0.584
	Plaster City	0.186	0.398
Northridge 1994	Beverly Hills 14145 Mulhol	0.416	0.470
	Canoga Park – Topanga Can	0.356	0.599
	Glendale – Las Palmas	0.357	0.472
	LA – Hollywood Storage	0.231	0.482
	LA – North Faring Road	0.273	0.609
	North Hollywood – Coldwater	0.271	0.485
	Sunland – Mt Gleason Ave	0.157	0.472
Loma Prieta 1989	Capitola	0.529	0.423
	Gilroy Array # 3	0.555	0.473
	Gilroy Array # 4	0.417	0.520
	Gilroy Array # 7	0.226	0.410
	Hollister Differential Array	0.279	0.415
	Saratoga – West Valley	0.332	0.600
Cape Mendocino 1992	Fortuna Boulevard	0.116	0.530
	Rio Dell Overpass	0.385	0.532
Landers 1992	Desert Hot Springs	0.154	0.542
	Yermo Fire Station	0.152	0.399

The characteristics of the 20 earthquake records used in this study are shown in Table 3. These records are representative of the 10/50-hazard level for Los Angeles conditions, and were used for the development of the CUREE-Caltech testing protocol [12]. Note that these ground motions are recorded far enough from the fault rupture to be free of typical near-fault pulse characteristics. For the life-safety limit state, each record was scaled such that its mean 5% damped spectral value between 0.1 and 0.6 s matches the 1997 Uniform Building Code design spectral value of 1.1 g for the same period range [8]. These scaled peak ground accelerations used in the analyses are also listed in Table 3.

Figure 5 presents the results of the non-linear dynamic time-history analyses in terms of the cumulative probability distribution of the maximum interstory drift across all shear wall lines in the building. The results are presented for Nailing Pattern No. 1 representing the original forced-based seismic design of the building and for Nailing Pattern No. 4 obtained herein with the proposed direct displacement-based seismic design procedure.

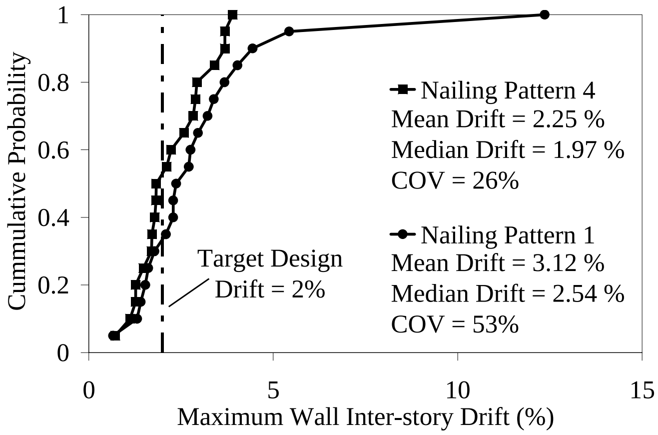

Fig 5 Cumulative distribution functions of maximum wall inter-story drifts from nonlinear dynamic analyses

For Nailing Pattern No. 4, the mean value of the maximum wall inter-story drift is 2.25%, which is very close to the target design inter-story drift of 2%. According to the current NEHRP Seismic Provisions [13] when using non-linear dynamic analyses, the design can be based on the mean value when more than seven analyses are performed. Therefore, it can be considered that the design of the building based on the proposed direct displacement-based procedure meets the design objective for life safety. Note that 50% of the records produce maximum inter-story drifts smaller than the 2% design target. Note also that the median value of the maximum wall inter-story drift is 1.97%, which is essentially identical to the target design inter-story drift of 2%. Finally there is a significant variability in system response associated with the ensemble of earthquake records selected (coefficient of variation of 26% in maximum inter-story drift).

For Nailing Pattern No 1, the mean value of the maximum wall inter-story drift is 3.12%, which is significantly larger than the target design drift of 2%. Therefore, it can be considered that the original 1988 Uniform Building Code design of the building based on the force-based procedure fails to meet the imposed design objective for life safety. Note that 70% of the records produce maximum inter-story drifts larger than the 2% design target and 10% of the records produce inter-story drifts in excess of 5%, which would likely correspond to the complete collapse of the building. Note also that the median value of the maximum wall inter-story drift is 2.54%, which is also larger than the target design drift of 2%. Finally there is even more variability in system response associated with the ensemble of earthquake records selected (coefficient of variation of 53% in maximum inter-story drift).

6. Conclusion

This paper discussed the application of direct displacement-based seismic design to light-frame wood buildings. This approach is appropriate considering that most of the damage to wood framed buildings observed following recent earthquakes has been related to excessive drift levels. This design procedure requires a numerical model capable of providing a pushover curve for the entire building. The SAWS numerical models developed under the CUREE-Caltech Woodframe Project incorporates this capability. As an application, the direct displacement-based seismic design of a two-story townhouse was presented in a step-by-step format. The validity of this proposed direct displacement-based design procedure was confirmed by evaluating the response of the townhouse structure through non-linear dynamic time-history analyses using earthquake records representative of the hazard levels that were associated with the life safety design performance level.

7. References

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