



VERIFICATION OF CODE PROCEDURES FOR NON-LINEAR ANALYSIS OF WOOD BUILDINGS

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SUMMARY

Although not widely used in practice, there is a growing need for non-linear analysis of wood framed buildings. Damage incurred by wood frame buildings during the 1994 M6.7 Northridge Earthquake and the 2001 M6.8 Nisqually Earthquake was valued in the millions of dollars. Non-linear analysis can provide better understanding of building response to earthquakes as well as suggesting ways to mitigate costly damage. The 1997 Uniform Building Code (1997 UBC) and the Federal Emergency Management Agency Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356) provide procedures for determining the displacement of wood framed buildings in the non-linear range under lateral loads.

This paper compares the procedures outlined in the 1997 UBC and FEMA 356 with experimental results obtained from full scale tests of a wood frame buildings. Full-scale shake table tests were conducted at the University of California at San Diego and the University of California at Berkley as part of the CUREE-Caltech Wood Frame Project undertaken for the Consortium of Universities for Research in Earthquake Engineering (CUREE). A comparison of the analysis results with the experimental findings on the Building tested in San Diego indicates that at moderate loads the procedures in the 1997 UBC and FEMA 356 provide a reasonably accurate representation of the displacement experienced in an earthquake. However, at loads approaching code design earthquake levels, the FEMA 356 procedures highly over-estimate the building deflections.

INTRODUCTION

The magnitude 6.7 Northridge Earthquake of January 17, 1994 was the most costly natural disaster in United States History. Estimates of losses, including rebuilding costs, directly attributable to the Northridge Earthquake range from \$20 to \$40 billion. Wood frame buildings were among the most severely damaged structures. Twenty-four of the twenty-five deaths that occurred as a result of building damage due to the Northridge Earthquake ground shaking occurred in wood frame structures.

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After the Northridge Earthquake, investigations were undertaken by various agencies and individuals, including the United States Geological Survey (USGS), the Federal Emergency Management Agency (FEMA) and the California Department of Transportation (Caltrans), to determine the factors contributing to building damage and failure, and to suggest ways to prevent future losses of this magnitude.

Largely as a result of these investigations, it was recognized that design techniques for wood frame buildings need to be improved. Although prescriptive methods of design for light-frame wood structures currently prevail in the Western United States it is becoming increasingly clear that, for other than the simplest buildings, it is important to accurately calculate the deformation of wood frame walls at various levels of displacement. Since wood shear walls behave inelastically at low displacements, accurately determining their deformation requires non-linear analysis. In addition, with the continuing emergence of performance based design, the need for non-linear analysis of wood framed structures continues to increase.

Currently, the predominant model code adopted by municipalities and state agencies in California is the 1997 Uniform Building Code (1997 UBC) [1]. The 1997 UBC includes methods for calculating the displacement of wood structural shear walls. In addition, FEMA has published a Prestandard for the Seismic Rehabilitation of Buildings (FEMA 356) [2] that includes methods not only for calculating displacement at yield but for constructing force displacement curves and analyzing the target displacement for specified lateral loads.

These methods will be used to provide an analytical comparison to experimental values obtained by researchers involved in the CUREE-Caltech Wood Frame Project. The CUREE-Caltech Wood Frame Project was undertaken to provide a better understanding of the behavior of wood-frame buildings under earthquake loads. As part of the project full scale shake-table tests were performed at the Universities of California at San Diego [3] and Berkley [4]. This paper presents the significant results obtained by the testing in San Diego, and compares them to the analytical methods provided by the 1997 UBC and FEMA 356.

PROCEDURES FOR THE CALCULATION OF SHEAR WALL DEFLECTIONS

1997 Uniform Building Code (1997 UBC)

The calculation of shear wall deflection as prescribed by the 1997 UBC is found in Standard 23-2. It is essentially a summation of the individual deformations of the various elements of the shear wall including bending of the vertical boundary members, Δ_b , shear deformation of the plywood panels, Δ_v , deformation of the nailing, Δ_n , and deformation of the hold-downs, Δ_a . Figure 1 further illustrates those components.

$$\Delta = \Delta_b + \Delta_v + \Delta_n + \Delta_a \quad (1)$$

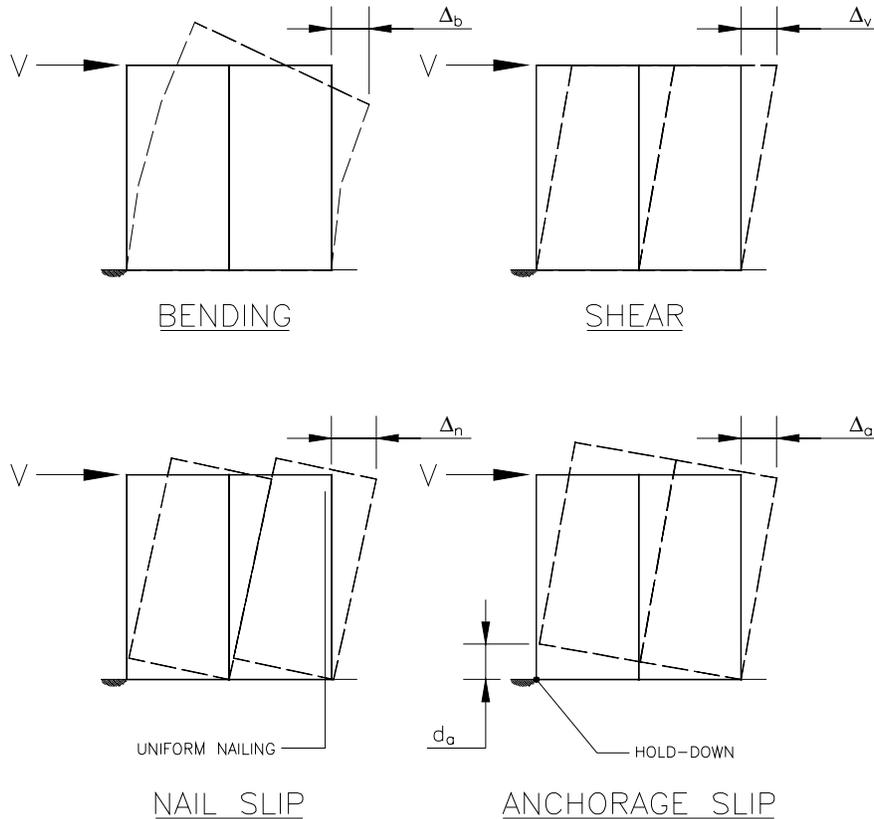


Figure 1 Deflection Components of a Shear Wall (Breyer [5])

In a shear wall, the vertical boundary elements, or posts, carry the moment and act like the flanges of a very deep I-beam. The bending deflection of the shear wall is related to the elongation and shortening of these members. Thus,

$$A_b = \frac{8vh^3}{EA b} \quad (2)$$

where:

v = maximum shear due to design loads at the top of the wall (plf)

h = wall height (ft)

E = Elastic modulus of the boundary element (psi)

A = Area of the boundary element cross-section (in²)

b = wall width (ft)

Similarly, the sheathing in a shear wall carries the shear forces and acts like the web of a very deep I-beam. For Equation (1) the sheathing is assumed to be a pure shear element as shown in Figure 1.

$$A_v = \frac{vh}{Gt} \quad (3)$$

where:

G = modulus of rigidity of the sheathing (psi)

Values for G are published in the 1997 UBC in Table 23-2-J

t = effective thickness of sheathing for shear (in)

Values for t are published in the 1997 UBC in Tables 23-2-H and 23-2-I

Deformations due to the nailed connections, Δ_n , between the sheathing and the framing allow the shear wall panels to move relative to each other and to the framing. This introduces a further component of deformation into the panel and reduces the shear carrying capacity of the sheathing.

$$\Delta_n = 0.75he_n \quad (4)$$

where:

e_n = nail deformation (in)

Values for nail deformation, e_n , are published in the 1997 UBC. These values are based on the results of experiments performed by The American Plywood Association – The Engineered Wood Association (APA) and published in Table B-4 of, “Plywood Diaphragms, Report 138.” [6] APA Table B-4 is reproduced here in part as Table 1.

Table 1 Fastener Slip Equations (APA [6])

Fastener	Minimum Penetration (in)	For Maximum Loads up to (lb)	Approximate Slip, e_n (in) ^{(a)(b)}	
			Green/Dry	Dry/Dry
6d Common	1 1/4	180	$(V_n/434)^{2.314}$	$(V_n/456)^{3.144}$
8d Common	1 7/16	220	$(V_n/857)^{1.869}$	$(V_n/616)^{3.018}$
10d Common	1 5/8	260	$(V_n/977)^{1.894}$	$(V_n/769)^{3.276}$

(a) Fabricated green/tested dry (seasoned); fabricated dry/tested dry, V_n = fastener load

(b) Values based on Structural I plywood fastened to Group II lumber, specific gravity 0.50 or greater. Increase slip by 20% when plywood is not Structural I.

Deformation of the hold-down, Δ_a , allows for rotation of the sheathing panels as shown in Figure 1. Hold-down deformation can occur in both the anchor bracket and the post.

$$\Delta_a = \frac{h}{b} d_a \quad (5)$$

where:

d_a = deflection due to anchorage details (in)

The deflection due to anchorage details, d_a , is dependant on the type and construction of the hold-down assemblies. Hold-down manufacturers typically publish hold-down deflections at the highest allowable design load in their catalogs or product specifications [7]. Interpolation can then be used to calculate d_a at design loads for the 1997 UBC equation.

When the design basis ground motion is applied to the structure using the equation for deflection from Standard 23-2, the resulting value is the design level response displacement, Δ_s . To determine the maximum inelastic response displacement, Δ_M , the design level response displacement must be amplified to account for over-strength and ductility.

$$\Delta_M = 0.7R\Delta_s \quad (6)$$

where:

R = Response modification coefficient

Values for R are published in the 1997 UBC in Table 16-N

This value must be checked against the story drift limitation in §1630.10.1 of the 1997 UBC which states that the calculated story drift using Δ_M shall not exceed 0.025 times the story height for structures having a fundamental period of less than 0.7 seconds.

Federal Emergency Management Agency, Prestandard and Commentary for the Seismic Rehabilitation of Buildings (FEMA 356)

The calculation of shear wall deflection at yield as determined by FEMA 356 Section 8.5.9 is similar to the 1997 UBC formula:

$$\Delta = \frac{8v_y h^3}{EAb} + \frac{v_y h}{Gt} + 0.75he_n + \frac{h}{b}d_a \quad (7)$$

where:

v_y = shear at yield in the direction under consideration (plf)

This value is then used in constructing the generalized force-deformation relation shown in Figure 2.

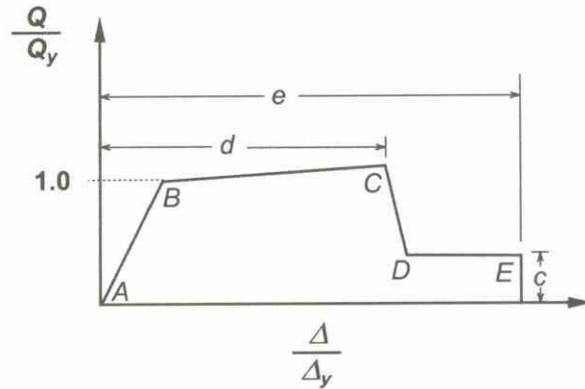


Figure 2 Generalized Force-Deformation Relations (FEMA 356 [2])

Point A on the curve represents the unloaded component. Point B is the effective yield point where the yield deflection, Δ_y , is determined by FEMA 356 Eqn (8-2) and the yield strength is determined using the Load and Resistance Factor Design Manual for Engineered Wood Construction (ASCE-16) [8] with a resistance factor, ϕ , of unity. Points C and D represent the first loss of strength in the component. Since the yield strength is determined using calculation, the strength at point C is taken as 1.5 times the yield strength. At point D the residual strength is determined by taking a percentage, c , of the yield strength of the component as determined in FEMA 356 Table 8-4. The maximum deflection at the point of first loss of strength, d , is determined from FEMA 356 Table 8-4. Generally when going from point C to point D on the curve, a small slope is incorporated to ease computations using nonlinear software. Point E represents the maximum deflection the component can sustain at the residual strength. The deflection at point E is determined using FEMA 356 Table 8-4 which is reproduced here in part as Table 2.

Table 2 Modeling Parameters and Numerical Acceptance Criteria for Nonlinear Procedures – Wood Components (FEMA 356)

		Modeling Parameters		
		Δ/Δ_y		Residual Strength Ratio
Diaphragms ^(a)	Height/Width Ratio (h/b)	d	e	c
Wood Structural Panel, Blocked, Chorded ^(b)	$L/b \leq 3$	4.0	5.0	0.3
	$L/b = 4$	3.0	4.0	0.3
Wood structural Panel, Unblocked, Chorded ^(b)	$L/b \leq 3$	3.0	4.0	0.3
	$L/b = 4$	2.5	3.5	0.3

(a) For diaphragm components with aspect ratios between maximum listed values and 4.0, deformation ratios shall be decreased by linear interpolation between the listed values and 1.0. Diaphragm components with aspect ratios exceeding 4.0 shall not be considered effective in resisting lateral loads.

(b) Linear interpolation shall be permitted for intermediate values of aspect ratio.

Once the generalized force displacement curve has been constructed the target displacement is determined using the Procedures in FEMA 356 §3.3.3.3.

CUREE-CALTECH WOOD FRAME PROJECT

The California Institute of Technology (Caltech) has undertaken a project funded by the Federal Emergency Management Agency (FEMA) whose stated goal is to significantly reduce earthquake losses to wood-frame construction [9]. The Consortium of Universities for Research in Earthquake Engineering (CUREE) subcontracted to perform and coordinate the non-Caltech work. The project, called the CUREE-Caltech Wood Frame Project, is divided into five separate elements which are being concurrently undertaken by engineers and students at participating universities. Element 1 consists of three full scale shake-table tests that were undertaken to determine the performance of wood-frame structures under seismic loads at a system level. Task 1.1.1 was performed at the University of California, San Diego, and Task 1.1.2 was performed at the University of California, Berkley. Task 1.1.3 included full scale testing of simplified box structures and was undertaken at the University of British Columbia. This paper will focus on Task 1.1.1, the results of this task were published by CUREE as, “Shake Table Tests of a Two-Story Wood-frame House” (CUREE W-06) [3].

Shake Table Tests of a Two-Story Wood-frame House, UCSD

The wood-frame building for Task 1.1.1 at the University of California, San Diego was designed and constructed to incorporate recent trends in residential wood construction in California. The building is a simplified example with no vertical or horizontal irregularities. This type of design was chosen so that the results would be easily interpretable and allow for more general extrapolation. Figure 3 shows plan views of the first and second story and Figure 4 shows elevations of the exterior walls.

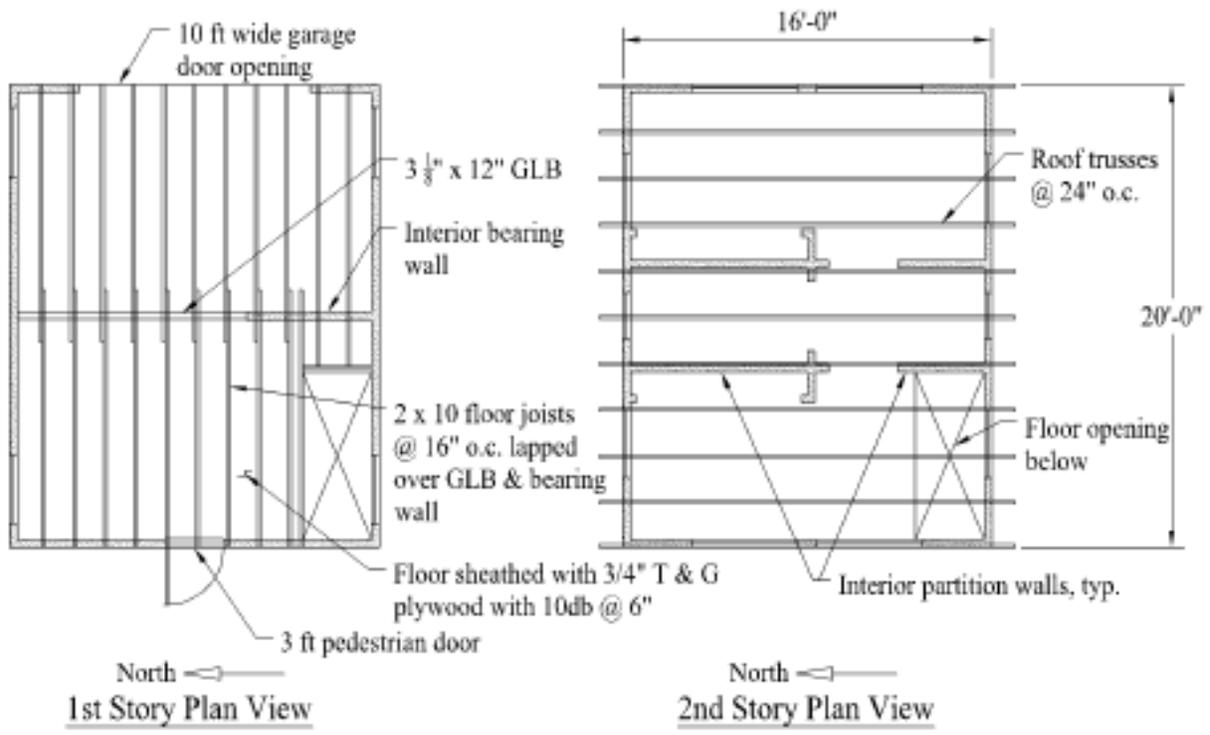


Figure 3 Plan View of Test Structure Showing Major Structural Components (CUREE W-06)

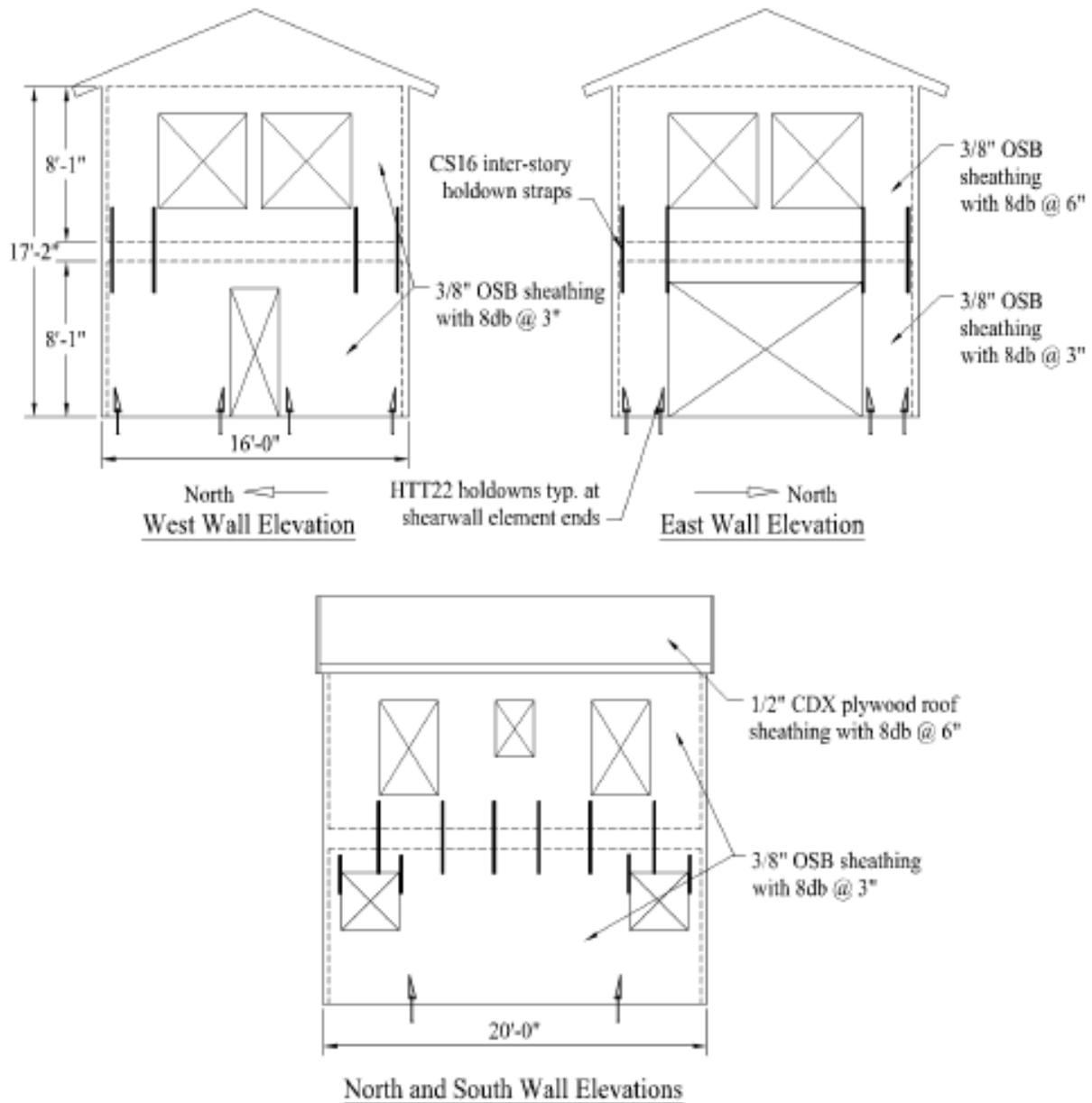


Figure 4 Elevations of Test Structure Showing Major Structural Components (CUREE W-06)

The building was tested in several phases under a variety of structural and loading configurations. Structural conditions for the testing included: unsheathed with various sub-floor configurations (Phases 1 through 4), sheathed with no openings (Phase 5), sheathed with small openings in the East and West elevations (Phase 6), sheathed with various hold-down and anchorage configurations (Phases 7 and 8), sheathed with large first floor opening (Phase 9), sheathed with plywood, gypsum wallboard, and stucco (Phase 10).

Seismic ground motions were selected as part of Task 1.3.2 which was organized to provide common testing protocols for all aspects of the project. The 1994 Northridge Earthquake ground motions were selected in order to match the National Earthquake Hazards Reduction Program (NEHRP) design spectra for ordinary ground motion, 10% probability of exceedance in 50 years (475 year return period) and near-

field ground motion, 2% probability of exceedance in 50 years (2475 year return period). The Northridge Earthquake ground motion at Canoga Park was selected for the ordinary ground motion using an amplitude scaling factor of 1.2. The Northridge Earthquake ground motion at Rinaldi Receiving Station was selected for the near-field ground motion using an amplitude scaling factor of unity. In order to test the structure under various ground motions with more frequent return periods, the Canoga Park ground motion was applied with various levels of amplitude scaling factors as shown in Table 3. Seismic loads were applied to the structure parallel to the short (North-South) side.

Table 3 Ground Motions for Seismic Tests (CUREE W-06)

Seismic Test Level	Ground Motion	Hazard Level	Amplitude Scaling Factor	PGA (g)
1	Canoga Park	99.99%/50 years	0.12	0.05
2	Canoga Park	50%/50 years	0.53	0.22
3	Canoga Park	20%/50 years	0.56	0.36
4	Canoga Park	10%/50 years	1.20	0.50
5	Rinaldi	2%/50 years	1.00	0.89

For the purposes of this paper we will consider the Phase 9 structure without gypsum wall board or stucco sheathing under the near-field ground motion. This allows us to test the analytical models for wood shear panels at large displacement without considering the effects of the drywall and stucco sheathing. The relative roof displacement time histories for the Phase 9 Structure under near-field ground motions are shown in Figure 5.

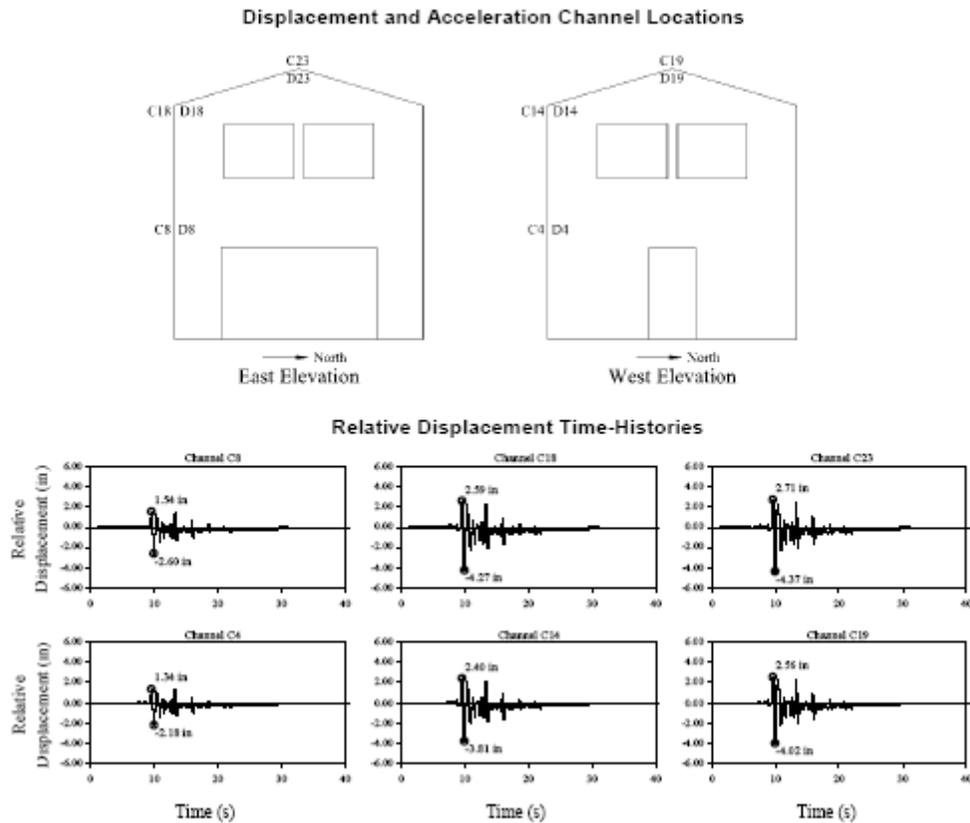


Figure 5 Relative Displacement Time-Histories (CUREE W-06)

Building Displacement Based on 1997 UBC and FEMA 356 Methodology

To calculate the base shear, V , the effective peak acceleration was obtained from the spectral acceleration for seismic test level 4. Effective peak acceleration was calculated by finding the average acceleration between 0.1 and 0.5 second periods and dividing by 2.5. The base shear was then calculated using the procedures in Section 1630.2 of the 1997 UBC. Using this method the base shear was calculated as 0.19 times the total seismic dead load. Vertical distribution of the base shear was done following Section 1630.5 in the 1997 UBC.

The design seismic force at each level was horizontally distributed to the shear walls according to their relative stiffnesses assuming a rigid diaphragm. The maximum inelastic response displacements for the second floor and roof levels were calculated using Formula 1 and an iterative procedure. The results are given in Table 5. Torsional effects were observed in the numerical results due to the large opening on the first floor at the East elevation. The CUREE results in Table 5 were obtained from Appendix J, Fischer [3].

The generalized force deformation relations from FEMA 356 were used to construct a “pancake” model of the building in SAP2000 [10]. The “pancake” model used nonlinear springs to represent the shear walls. A pushover analysis was performed in SAP and the resulting force-displacement curve for the building was used to calculate the target displacement at the center of mass using Section 3.3.3.3 in FEMA 356. The pushover curve generated by SAP is shown in Figure 6. Once the target displacement at the center of

mass was calculated the displacements at the second floor and roof were generated for the East and West elevations. Those displacements are given in Table 5.

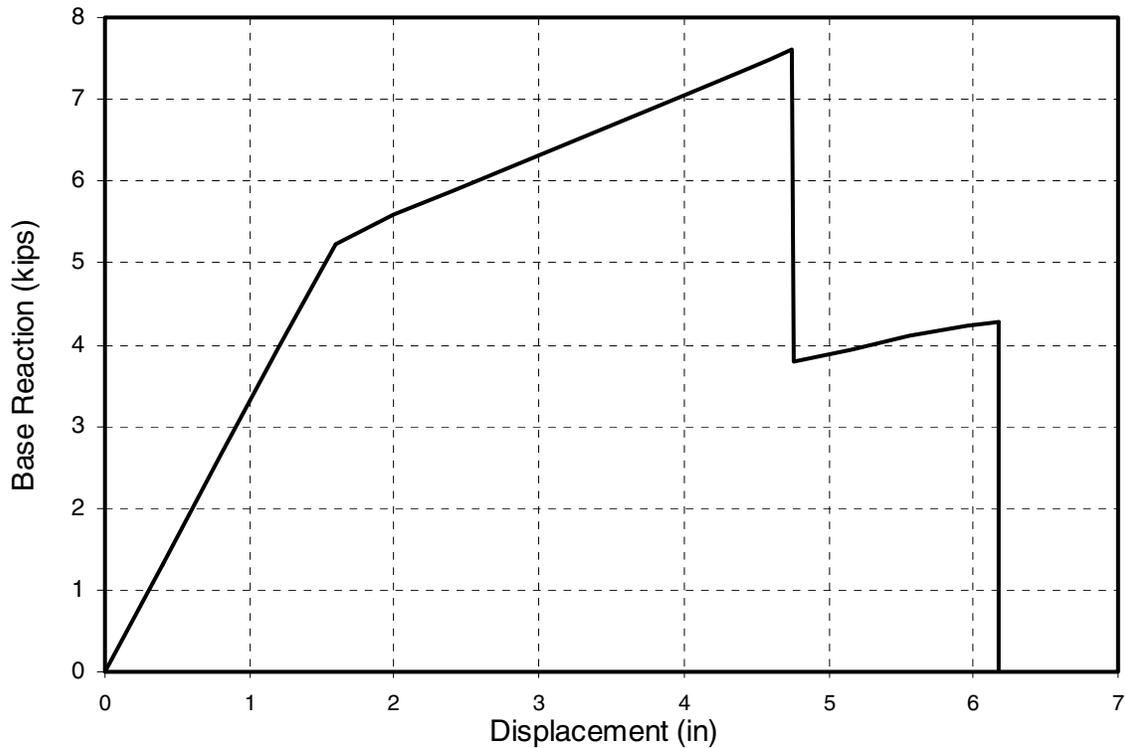


Figure 6 Push-over Curve at Roof Center of Mass

Table 5 Comparison of 1997 UBC Calculations to CUREE Test Data

Level	Building Elevation	1997 UBC Design Seismic Force (kips)	Displacement from CUREE (in)	Calculated Displacement (in)	
				1997 UBC	FEMA 356
Roof	West	2.86	2.41	1.73	4.29
	East		2.62	2.64	4.5
2 nd Floor	West	1.84	1.37	0.25	2.7
	East		1.56	1.14	2.9

CONCLUSIONS

For the building considered, the 1997 UBC equations and methodology for calculating the building displacements generate an accurate picture of the building response observed during the CUREE tests. However, the 1997 UBC methodology over-estimated the torsional effects of the non-symmetric first floor stiffnesses. Approximating a rigid diaphragm using numerical methods is difficult and complex, in order to mitigate the opportunities for errors in the calculations three a three dimensional model of the structure should be constructed using non-linear analysis software.

Although the FEMA 356 equations and methodology provide a more accurate description of the torsion and relative deformations of the structure, FEMA 356 exaggerates the displacement significantly. Estimation of the building period, in the absence of experimental data, becomes critical and can cause large discrepancies in calculation of the target displacement. The FEMA target displacement at the roof center of mass in the previous section was calculated using the building period from modal analysis in SAP2000. When the Period is calculated using Section 3.3.1.2 of FEMA 356 the target displacement changes from 5.6-inches to 4.7-inches.

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