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PERFORMANCE BASED PUSHOVER ANALYSIS OF WOOD FRAMED BUILDINGS

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SUMMARY

This paper presents a nonlinear pushover analysis to evaluate the structural performance of existing light-frame wood structures subjected to earthquake-induced ground motions. The seismic evaluation of such buildings requires analysis techniques to determine lateral load resisting capacities and predict inelastic performance parameters. The initial phase of evaluation presented herein comprises of assessing the mass and stiffness characteristics of the building through available plans or on-site building investigation. The stiffness of the building lateral force resisting structural elements (walls) are obtained from experiments performed on similar elements at the University of California, Irvine as part of the City of Los Angeles and the University of California, Irvine (COLA-UCI) Light-Frame Test Program directed by the Structural Engineering Association of California (SEAOC) [1]. Utilizing this force-deflection information, a pushover curve (capacity) for a light-frame wood building can be developed with a finite element computer program such as SAP2000 [2]. FEMA-356 [3] procedures are then adopted to evaluate the performance (response) of the building to a given level of ground motion (demand). Expected levels of drift, consequent damage and any deficiencies in the lateral force resisting capacity at a given level of ground motion intensity can be identified through this process and corrective measures can be implemented.

INTRODUCTION

A large number of two and three storied wood-framed buildings were damaged by the January 17, 1994 Northridge Earthquake. Most of these buildings were built in the middle part of the twentieth century and many prior to the 1971 San Fernando earthquake, lessons learned from which initiated significant changes in the codes and design practices at the time. Most of these buildings are sheathed with materials such as stucco, gypsum plaster or drywall that have relatively low ductility. In addition, the inherent lateral force resistance capacity of these structures was further diminished due to the damage caused by the Northridge

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Earthquake induced ground motions. Lessons learned from the Northridge earthquake have led to the upward revision of the definition of design basis ground motion intensity. Consequently, during the process of evaluating the damage sustained by these buildings, it was determined that the lateral force resistance capacity of many of these buildings is inadequate to resist a current code level design basis earthquake, such as that defined by the 1997 Uniform Building Code (1997 UBC) [4]. However, the basic performance objective of the UBC series of codes has always been life safety protection in the event of a design basis earthquake. The purpose for undertaking this study was to determine the performance of these buildings during the earthquake, to estimate the loss in capacity experienced as a result of earthquake induced ground shaking, to determine whether the lateral force resisting capacity is adequate to resist a code level design basis earthquake and to determine appropriate methodology for restoring the loss in capacity and ensuring life safety protection in the event of a future design level earthquake.

LITERATURE REVIEW

Prior to the commencement of the work presented in this paper, a comprehensive review of available literature on experimentally determined lateral load-deformation characteristics of light-framed wood structures was undertaken. Some of the relevant research on the subject that was reviewed is summarized below.

Testing of wood framed walls was performed at the University of California, San Diego by Arnold, et. al. [5]. The testing consisted of cyclically loading to failure two 8 ft x 16 ft wall specimens sheathed with stucco and drywall finishes and with different opening configurations provided for doors and windows. The study simulated the presence of a second story on the specimen by the application of vertical axial load and enhancement of rigidity along the top plane of the test specimen. Damage induced in both specimens was repaired and the specimens were retested to determine their post-repair performance characteristics. Results from this set of experiments are presented as cyclic load deformation curves. The authors attributed the high level of strength to the greater length of effective anchorage relative to the net wall length, and the wrapping of stucco at the corner studs.

Dynamic testing of a two-story single-family wood-frame house was performed at the University of California, San Diego by Fischer et al. [6] as part of the CUREE-Caltech Wood-Frame Project in order to determine the dynamic characteristics and seismic performance of the structure. The structure had a footprint of 16 ft x 20 ft and was 18 feet 10 inches in height. The walls were sheathed with oriented strand board (OSB) and stucco on the exterior and gypsum wall board on the interior. The testing was performed at different stages of construction. Between each phase, the structure was repaired. Re-testing performed without repair of structure following damage from initial earthquake indicated a significant degradation in seismic performance during subsequent earthquakes.

Dynamic testing of a large-scale three-story wood-frame residential building with tuck-under parking was conducted at the University of California, Berkeley by Mosalam et. al. [7] as part of the CUREE-Caltech Wood-frame Project. The experiment consisted of shake table test of a 16 ft x 32 ft footprint building with plywood walls along the perimeter of the building. Three tests were conducted: In the first test, the building was tested without finishes; in the second, the building was tested with finishes (stucco and drywall); and in the third, the building was tested with finishes and retrofit (moment resisting frame around garage opening). The results show that the stucco and interior gypsum boards considerably reduced the maximum story drift in the open front but increased the story shear. The steel moment frame retrofit in the open front considerably reduced the maximum story drift in the open front of the finished building and reduced the story shear. The rotation of the second level was considerably reduced by the finishes and the retrofit.

Most of these research focuses on certain aspects of performance, such as full-scale behavior, method and adequacy of repairs or sub-assembly performance. Manufactured wood sheathing products were also used in a number of experiments, although such elements rarely existed in the majority of pre-1971 built wood frame structures. Therefore, it is difficult to distill characteristics of individual component (stucco, plaster, drywall, etc.) performance from these tests for direct application to the development of force-deformation curves for any building.

LATERAL FORCE RESISTING STRUCTURAL ELEMENTS FORCE DEFORMATION

In older light-framed wood structures, the lateral force resisting structural elements typically consists of exterior stucco walls and interior drywall or plaster partition walls. The properties of the building lateral force resisting structural elements are obtained from experiments performed on similar elements at the University of California, Irvine between 1996 and 2000. The experimental program was directed by the City of Los Angeles (COLA), the Structural Engineers Association of Southern California (SEAOSC) and the University of California, Irvine (UCI) and is known as the COLA-UCI Light-Frame Test Program. Cyclic tests were performed on 8 ft x 8 ft walls with various sheathing materials (stucco, plywood and drywall) and connector (Nails, Staples, etc.) configurations. The loading protocol was adopted from SEAOSC documents on cyclic load testing for shear resistance of framed walls. The tests were performed on at least three specimens of each configuration. Figure 1 shows a plot of the cyclic test data for 7/8 inch stucco with 3/8 inch furring nails spaced at 6 inches on center. Figure 2 shows a plot of the cyclic test data for 5/8" drywall on two sides with 1-7/8 inch drywall nails spaced at 4 inches. The test data provides the backbone curve data which connects the points on the load-deformation curve that correspond to the maximum measured force when the shear wall is subjected to a predetermined displacement level and the degraded backbone curve which connects the points of the load-deformation curve that correspond to the maximum load measured when the shear wall is repeatedly subjected to the same predetermined displacement level. For application of this experimental data, the force-displacement behavior of all components in a building can be calculated reasonably accurately by scaling the backbone data with the length and height of exterior stucco walls and the interior partition walls determined from available building plans or from on-site building measurements.

BUILDING LOAD DEFORMATION CAPACITY

A building's lateral load deformation capacity (pushover curve) can be calculated from a simplified two dimensional SAP2000 analytical model of the building. The building's lateral force resisting elements (exterior stucco walls and interior drywall or plaster walls) are represented by equivalent single-degree of freedom (SDOF) nonlinear shear elements (springs). The nonlinear shear elements are assigned the full backbone force-deformation behavior derived from the COLA cyclic analysis data. The floor and roof diaphragms are assumed to be stiff compared to the walls. The building seismic weight is calculated using the weight of the floors, walls and roof and the mass is lumped at the floor nodes. The lateral loads are applied to the model in proportion to the distribution of inertia forces in the plan of each floor diaphragm. The analytical model is subjected to progressively increasing force using a triangular, uniform or any other pattern of lateral load distribution and the displacement is monitored. The result of the analysis is a push over curve of the total earthquake inertial force, or base shear, versus the roof displacement such as the one shown in Figure 6.

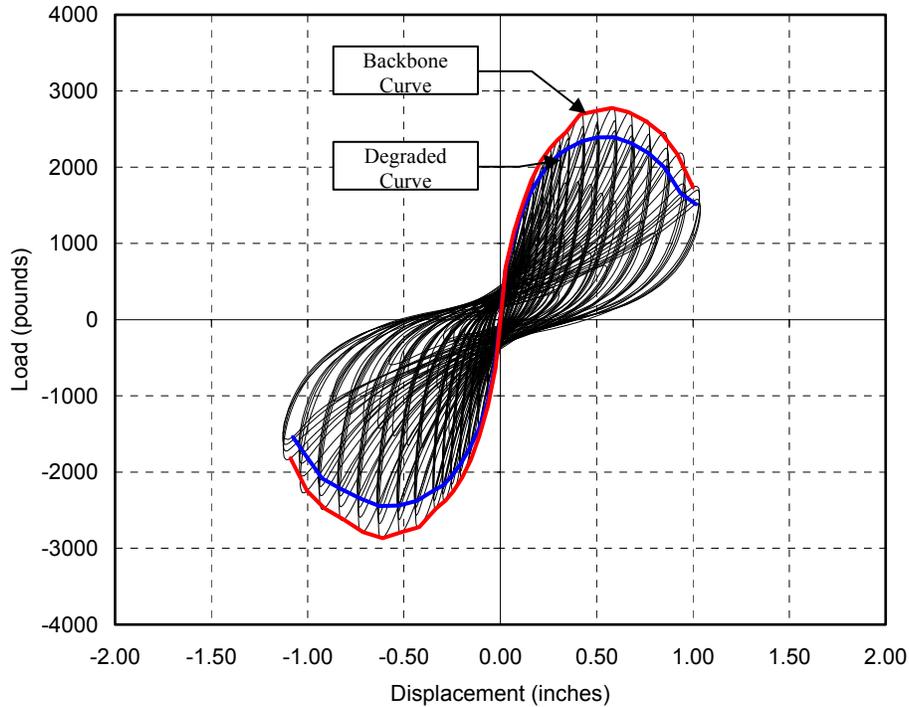


Figure 1 Cyclic Load-Deflection Curve of 8 ft - 7/8" Stucco Wall (One Side)

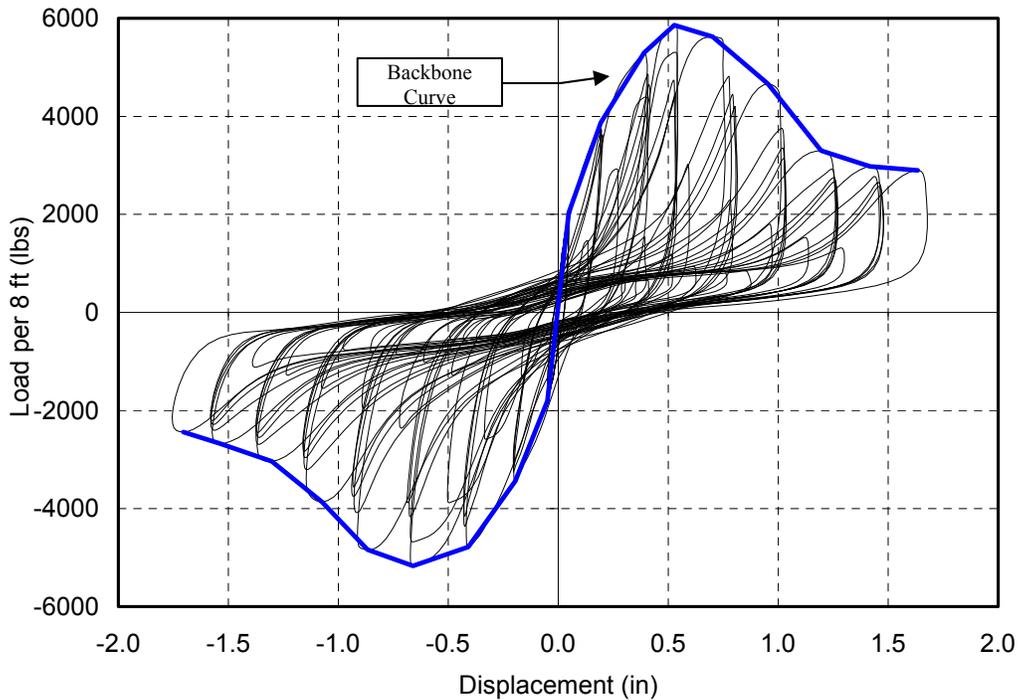


Figure 2 Cyclic Load-Deflection Curve of 8 ft - 5/8" Drywall (Two Sides)

EVALUATION OF BUILDING PERFORMANCE

The FEMA 356 nonlinear static analysis procedure is followed in order to calculate the building's target displacement. The target displacement is the maximum displacement at the roof level likely to be

experienced by the building for a given level of ground motion. The nonlinear force-displacement relationship obtained from the SAP2000 nonlinear push over analysis is idealized to calculate the effective lateral stiffness, K_e , and the effective yield strength, V_y , of the building as shown in Figure 3. The line segments on the idealized force-displacement curve are located using an iterative graphical procedure that approximately balances the area above and below the curve.

The effective fundamental period, T_e , is determined from the equation

$$T_e = T_i \sqrt{\frac{K_i}{K_e}} \quad (\text{FEMA 356 EQ. 3-14})$$

Where T_i is the elastic fundamental period, K_e is the effective lateral stiffness of the building and K_i is the elastic stiffness. The target displacement δ_t , is given by

$$\delta_t = C_0 C_1 C_2 C_3 S_a \frac{T_e^2}{4\pi^2} g \quad (\text{FEMA 356 EQ. 3-15})$$

where C_0 is a modification factor to relate spectral displacement of an equivalent SDOF system to the roof displacement of the building MDOF system given in Table 1. C_1 is a modification factor to relate expected maximum inelastic displacements to displacements calculated for linear elastic response, C_2 is a modification factor to represent the effect of pinched hysteretic shape, stiffness degradation and strength deterioration given in Table 2, C_3 is a modification factor for P- Δ effects, S_a is the response spectrum acceleration and g is the acceleration of gravity.

$$C_1 = 1 \text{ for } T_e \geq T_S$$

$$C_1 = [1.0 + (R-1) T_S / T_e] / R \text{ for } T_e \leq T_S$$

Where T_S is the characteristic period of the response spectrum associated with the transition from the constant acceleration segment of the spectrum, defined as the period associated with the transition from the constant acceleration segment of the spectrum to the constant velocity segment of the spectrum.

R is given by

$$R = \frac{S_a}{V_y / W} \cdot C_m \quad (\text{FEMA 356 EQ. 3-16})$$

where V_y is the yield strength as defined on Figure 3, W is the effective seismic weight and C_m is the effective mass factor, taken as 1 for light-frame wood structures and the modification factor for for P- Δ effects, C_3 is defined as:

$$C_3 = 1.0 + \frac{|\alpha|(R-1)^{1.5}}{T_e} \quad (\text{FEMA 356 EQ. 3-17})$$

Where α is the ratio of post-yield stiffness to effective elastic stiffness defined on Figure 3.

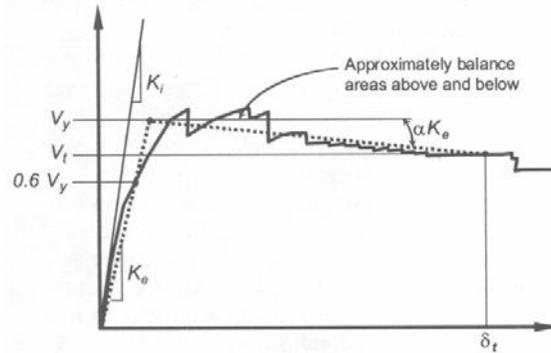


Figure 3 Idealized Nonlinear Force Displacement Relationship (From FEMA 356, Figure 3.1b)

Table 1 Values for Modification Factor C_0 (FEMA Table 3-2)

Number of Stories	Shear Buildings		Other Buildings
	Triangular Load Pattern	Uniform Load Pattern	Any Load Pattern
1	1.0	1.0	1.0
2	1.2	1.15	1.2
3	1.2	1.2	1.3
5	1.3	1.2	1.4
10+	1.3	1.2	1.5

Table 2 Values for Modification Factor C_2 (FEMA Table 3-3)

Structural Performance Level	$T \leq 0.1$ second		$T \geq T_s$ second	
	Framing Type 1	Framing Type 2	Framing Type 1	Framing Type 2
Immediate Occupancy	1.0	1.0	1.0	1.0
Life Safety	1.3	1.0	1.1	1.0
Collapse Prevention	1.5	1.0	1.2	1.0

Estimation of target displacement in each orthogonal direction for a specified intensity of ground motion and comparison with the pushover curve lends valuable insight into the expected performance of the building at the level of ground shaking. The target displacement provides an estimate of the total deformation of the structure at the roof level. Inter-story drifts can also be inferred from the estimates of target displacement.

LOSS IN LATERAL FORCE RESISTANCE

During an earthquake, the lateral force resisting elements of a building are exposed to a certain level of deformation. Depending on the number of cycles of shaking experienced by the structure at each level of deformation, there will be an associated loss in the strength of the lateral force resisting elements due to cracking of the exterior stucco walls and interior drywall or plaster walls, and loosening, pullout or shearing of the fasteners that attach them to the wood framing. The loss in lateral force resisting capacity for each lateral force resisting element type can be typically estimated from the difference between the backbone curve and the degraded backbone curve at the target displacement calculated from the FEMA 356 procedure.

STRENGTHENING OF BUILDING

The FEMA 356 procedure is used to calculate the building's target displacement for a given level of ground motion. The FEMA 356 procedure can also be used to back calculate the ground motion level that would result in a target displacement for any desired performance limit corresponding to a defined risk level. The ratio of the code level design basis earthquake ground motion spectral acceleration to the spectral acceleration that would result in an acceptable target displacement can be used to scale the yield strength V_y of the existing structure. Using an equal displacement assumption, the shortfall in yield strength between the scaled yield strength and the yield strength of the existing building can be compensated with addition of plywood shear panels.

ANALYSIS OF A WOOD-FRAMED BUILDING

A two-storied building of wood frame construction is used as an illustrative example of the method described above. The building consists of two units of almost identical floor plans. The floor plans of the building are shown in Figure 4. The exterior of the building is sheathed with stucco while the interior walls and ceilings are built of drywall. The force-displacement behavior of the walls is calculated by scaling the backbone data from the COLA-UCI Light-Frame Test Program with the length of exterior stucco walls and the interior partition walls. The building's load deflection capacity is calculated from a simplified two dimensional SAP2000 analytical model of the building where the building's lateral force resisting elements (exterior stucco walls and interior drywall) are represented by equivalent single-degree of freedom (SDOF) nonlinear shear elements (springs) as shown in Figure 5. Multiple springs in parallel are used at each level to represent the different types of wall sheathing and finishes that are present in the structure. A separate spring-mass model is developed for each orthogonal direction of the building. The floor and roof diaphragms are assumed to be stiff compared to the walls. The building's seismic weight is calculated using the weight of the floors, walls and roof and the mass is lumped at the floor nodes. The lateral loads are applied to the model in proportion to the distribution of inertia forces in the plan of each floor diaphragm. The analytical model is subjected to progressively increasing force using a triangular load distribution and the roof displacement is monitored. The result of the analysis is a push over curve of the total earthquake inertial force, or base shear, versus the roof displacement as shown in Figure 6.

An estimate of the response spectra from the Northridge earthquake at the building site in the East-West direction is shown in Figure 7. From this estimate of the response spectra, an Effective Peak Acceleration (EPA) of approximately 0.3g was computed and used to determine the spectral acceleration value experience by the building during the earthquake. Details on the procedure to estimate site-specific response spectra were obtained from King et al [7]. The UBC 1997 response spectra in the East-West direction for this building site is also shown in Figure 7.

The FEMA 356 procedure is then followed to calculate the building target displacement during the earthquake. Using the EPA value calculated from the estimated response spectra at the site, the target displacement is determined using an idealized bi-linear curve of the push over curve. The idealized bi-linear curve is located using an iterative graphical procedure that approximately balances the area above and below the curve as shown in Figure 6. A roof target displacement of 0.58 inch was computed for this level of ground shaking applied to the East-West direction. For a target displacement of 0.58 inch at the roof level, the inter-story drift was concentrated between the second floor level and the foundation level and was determined from the SAP2000 analytical model to be 0.51 inches. From Figures 1 and 2, it is apparent that for this level of inter-story drift, there is a substantial difference between the backbone and the degraded backbone curves which indicates a loss in the lateral force resisting capacity of the building.

The loss in the lateral force resisting capacity of the lateral force resisting elements is calculated as the difference between the backbone curve and the degraded backbone curve at the second floor level displacement (0.51 inch). On-site damage observations of the building indicated crack patterns and sizes that were consistent with this level of ground motion and resultant target displacement. The loss in load capacity of the lateral force resisting system of the building can be compensated by proper repairs and or addition of plywood shear wall panels in the building and the appropriate procedure has to be evaluated on case by case basis.

Similarly, the roof target displacement for the code basis design level ground motion was estimated at 2.62 inches which exceeds the displacement capacities of the building. The FEMA 356 procedure can be used to determine the amount of strengthening required to preserve life safety in the event of a code level earthquake. The ratio of the code level design basis earthquake ground motion spectral acceleration to the spectral acceleration that would result in an acceptable target displacement can be used to scale the yield strength V_y of the existing structure. Using an equal displacement assumption, the shortfall in yield strength between the scaled yield strength and the yield strength of the existing building can be compensated with addition of plywood shear panels.

The calculations performed show that the loss in capacity of the stucco and plaster walls in the East-West directions was approximately 15% and the building's shear force resistance capacity prior to the damage caused by the Northridge Earthquake was inadequate to provide life-safety protection in the event of a current design basis code level earthquake. Therefore, the following repair is recommended. First, the capacity of the building needs to be restored to pre-earthquake level. Patching of the stucco and plaster are an inadequate repair since the discontinuities and fastener degradation created by the earthquake damage are not completely restored by patching and their integrity and function are thereby impairing for future performance. Secondly, the lateral force resistance capacity of the stucco is not adequate to resist a current code level design basis earthquake even on the assumption of undamaged stucco and plaster. Hence, plywood shear wall panels need to be added at the first floor level of the building to provide adequate lateral force resistance capacity at current design basis ground motion levels primarily to safeguard against loss of life, not to limit damage or maintain function. Based on preliminary calculations, Douglas Fir-Larch plywood shear walls need to be added at the first floor level per the schedule shown in Table 3. The length of plywood shear walls to be added to the building is based on calculations to make up for the expected shortfall between the lateral force resistance capacity of the existing building and the anticipated seismic demand on the building from current design basis ground motion and also for the damage caused by 1994 Northridge Earthquake. The distribution of plywood shear panels on the first floor level is shown in Figure 8. The addition of the plywood walls is not a code upgrade and does not meet the specific prescriptive requirements of the current codes. It does however, in addition to the repair recommendations described earlier, restore the structural strength of the buildings and provide life-safety protection for the building occupants.

Table 3 First Floor Shear Wall Panel Schedule

Direction	Panel Grade	Plywood Shear Wall 		Length of Panel (feet)
		Thickness (in.)	Nailing	
Transverse (E-W)	Structural I	15/32	10d @ 4:12	24

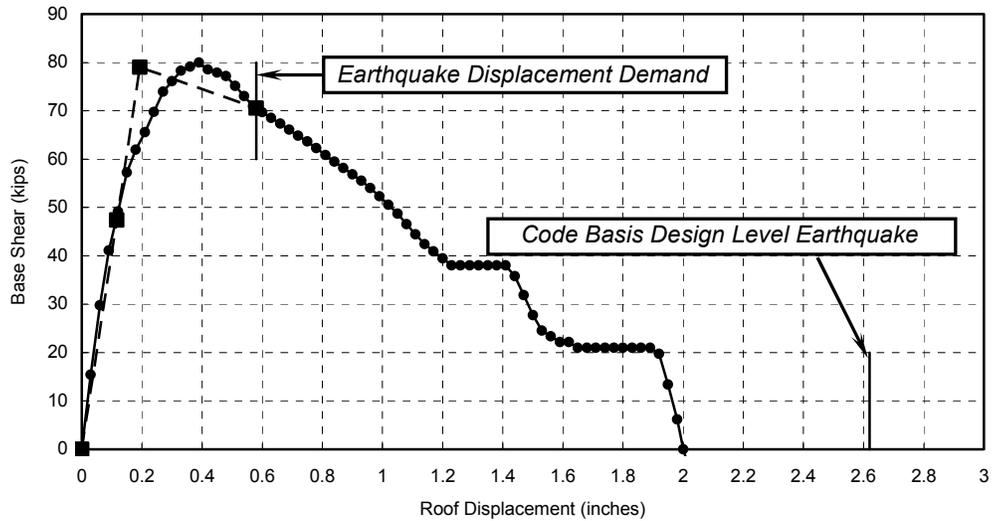


Figure 6 Base Shear versus Roof Displacement in EW Direction

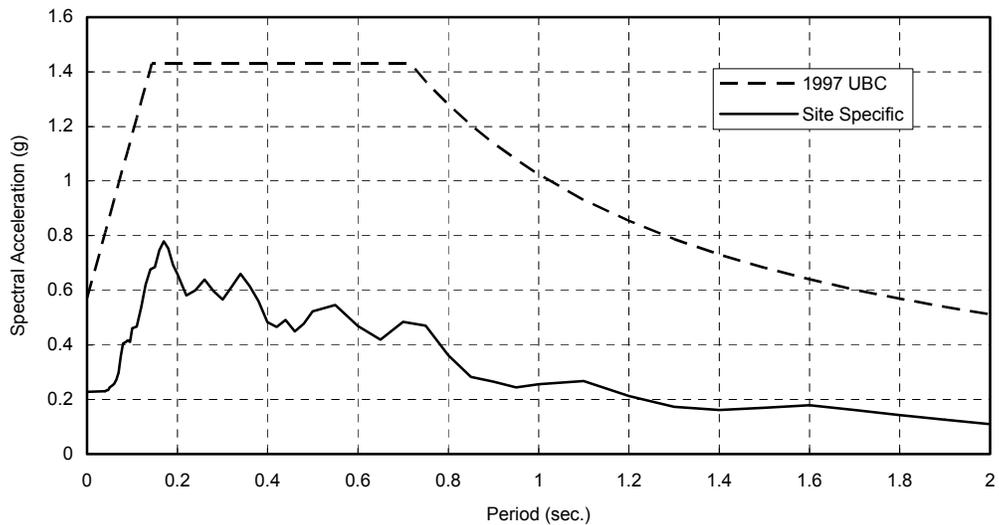


Figure 7 Estimate of Response Spectra in EW Direction

CONCLUSIONS

The stiffness of a wood framed building lateral force resisting structural elements obtained from experiments performed on similar elements as part of the COLA-UCI Light-Frame Test Program can be assembled to create a two dimensional analytical model for determining the nonlinear response of the building. FEMA-356 non linear static analysis procedure are then followed to calculate the building target displacement likely to be experienced by the building for a given level of ground motion. The FEMA-356 non linear static analysis procedure provide a tool for evaluating the damage in wood framed buildings and evaluate strengthening requirements. Buildings with significant torsional response may require additional analysis.

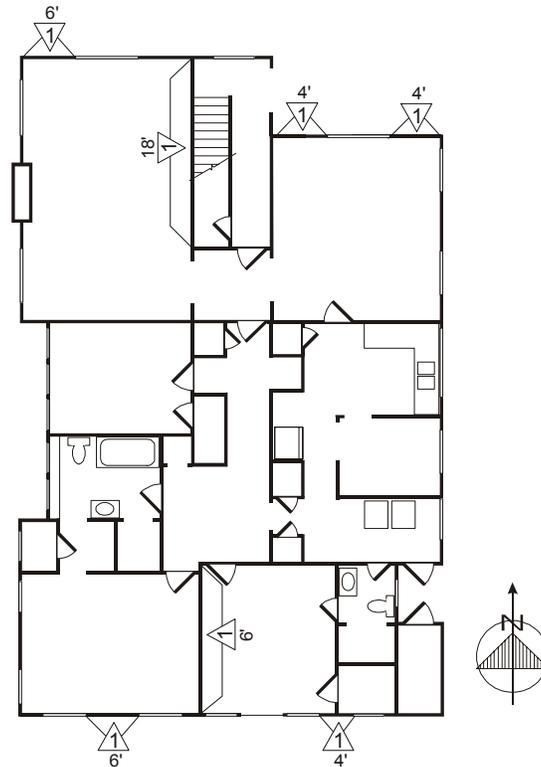


Figure 8 Distribution of Plywood Shear Walls at First Floor Level

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