Seismic Evaluation of Wood-Frame Shear Wall on Podium

Francisco X. Flores[1], Philip Line [2], Bradford K. Douglas[3], M. Omar Amini[4]

ABSTRACT: The Two-Stage analysis procedure is a design methodology used where a flexible building is placed atop a rigid platform. ASCE 7-22 allows both portions of the structure to be designed independently using this methodology if the lateral stiffness ratio between the lower portion (podium) and the upper portion (tower) is at least ten, and if the period of the whole structure is smaller than 1.1 times the period of the tower. Implicitly, this procedure relies on the fact that the acceleration at the top of the podium is approximately equal to the ground acceleration, which is true if the podium is rigid enough to cause the relative acceleration to be zero. Multi-story wood-frame towers over concrete podiums are a common configuration of tower-podium structures. This investigation evaluates the seismic performance of a 5-story wood-frame structure with wood structural panel shear wall vertical elements atop a 2-story concrete podium using the FEMA P-695 methodology. Three key outcomes were obtained from the study. First, the total acceleration at the top of the podium decreased when the podium yielded. Second, the total acceleration at the top of the podium exhibited amplified peaks compared to the ground acceleration, and its frequency content was altered if the lateral stiffness ratio of the podium to the tower was not large enough. Finally, when the lateral stiffness ratio is sufficiently large, the collapse performance of the tower collapse performance remains unaffected by the podium.

KEYWORDS: *Wood light-frame shear wall, two-stage analysis, seismic analysis, collapse performance, FEMA P*-695

1 – INTRODUCTION

The two-stage analysis is a typical design method used in the United States when a flexible structure (tower) is placed atop a rigid structure (podium). ASCE/SEI 7-22 [1] Section 12.2.3.2 two-stage analysis provisions allow the design of the tower and podium independently if the lateral stiffness ratio between the podium and the tower is at least 10 and if the period of the combined structure is smaller than 1.1 times the period of the tower. It is conceivable that the concept behind the standard design procedure is that the total acceleration at the top of the podium, which is the input acceleration for the tower, is the same (opposite sign) as the ground motion acceleration. This statement is true only when the podium is unrealistically rigid to have near zero relative displacements, relative velocities, and relative accelerations. However, a large enough podium to tower lateral stiffness ratio is intended to provide adequate performance of the tower atop a podium. Section 12.2.3.2 has deviated from the original concept by establishing a stiffness ratio between the podium and the tower equal to 10 as the main requirement to use the 2-stage design method. As a result, the ground motion measured at the top of the podium presents amplifications, and its frequency content is modified.

This investigation aims to evaluate the effect of varying the lateral stiffness ratio and the influence of podium yielding on the seismic performance of a structure designed using the two-stage analysis procedure. The evaluation is done by performing nonlinear time history analyses based on the FEMA P695 methodology [2] to compare a 5-story wood-frame tower fixed to the ground with the same tower atop a 2-story concrete podium.

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2 – BUILDING OVERVIEW AND SEISMIC DESIGN

The building that was analyzed was taken from the Structural Engineers Association of Northern California (SEAONC) study titled, "Evaluation of Two-Stage Seismic Analysis and Design Provisions for Multi-Story Buildings" [3], which investigated an example building designed by Tipping Structural Engineers for the San Francisco, CA Richmond District. The structure was designed using the two-stage analysis procedure per ASCE/SEI 7-22 [1]. The 5-story wood-frame shear wall (WFS) tower was designed using *Special Design Provisions for Wind and Seismic* (ANSI/AWC SDPWS-2021) [4] requirements.

The strength capacities of the first and second stories of the podium, according to the SEAONC report, were designed with an overstrength (capacity/demand) equal to 2.57 and 2.75, respectively. On the other hand, the length of the light frame wood shear walls, which would be used to estimate the assigned capacity, is not described in the report, but it can be inferred that the maximum strength capacity per story is 2.6 times the allowable stress design. Tests on Light-Frame Wood Shear Walls show that the maximum capacity per shear wall compared to the ASD capacity is around 3.0; Therefore, the capacity per story was increased by 15% (3/2.6).

3 – NUMERICAL MODELS

The tower and podium numerical models were created using OpenSees [5]. The approach used to represent the cyclic behavior response at each level used a shear spring at each story, as illustrated in Fig. 1.

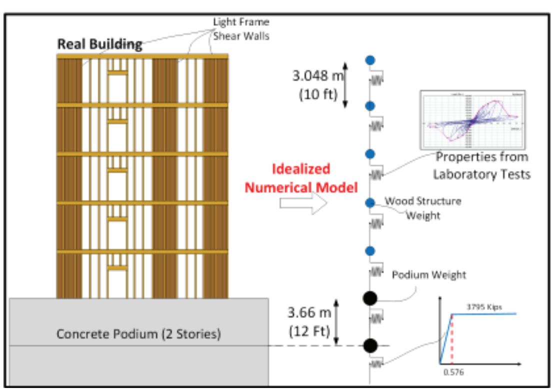

Fig.1 Idealization of Structure with Numerical Model

In the case of the wood-frame structure, each shear spring at each level was created using the SAWS [6] model. This model represents the nonlinear hysteretic behavior that accounts for pinching, stiffness, and strength degradation and is governed by 10 parameters. As shown in Fig. 2, these parameters were calibrated using shear wall test data. The modeled cyclic behavior, shown as AWC, was fitted between the cyclic and monotonic response of the tested wood structure shear walls.

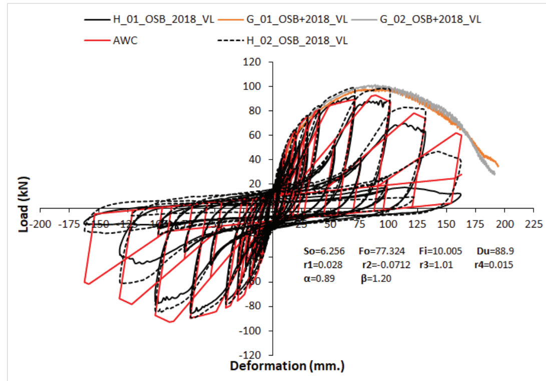

Fig. 2. SAWS material calibration

The test data shown in Fig. 2 was performed on a 2.44 m x 2.44 m (8x8 ft) shear wall which used a continuous rod tie-down system. The drift at the maximum strength capacity was 3.6%, equivalent to 111 mm (4.375 in.) for a 3.04 m (10 ft) wall. The modeling properties were then scaled up to match the required strength per story. The parameters used to model the 5-Story Light Frame Wood Shear Wall are shown in Table 1.

Table 1. Light Frame Wood Shear Wall SAWS Modeling Parameters

Tag	F0 - kN (Kips)	FI - kN (Kips)	Du - mm (in)	S0 - kN/mm (Kip/in)	R1	R2	R3	R4	alph	beta
1	4378.93 (984.42)	597.32 (134.28)	111.13 (4.38)	373.61 (2133.35)	0.028	-0.071	1.01	0.015	0.89	1.2
2	4378.93 (984.42)	597.32 (134.28)	111.13 (4.38)	373.61 (2133.35)	0.028	-0.071	1.01	0.015	0.89	1.2
3	4378.93 (984.42)	597.32 (134.28)	111.13 (4.38)	373.61 (2133.35)	0.028	-0.071	1.01	0.015	0.89	1.2
4	4040.70 (908.39)	551.18 (123.91)	111.13 (4.38)	344.75 (1968.57)	0.028	-0.071	1.01	0.015	0.89	1.2
5	2808.42 (631.36)	383.09 (86.12)	111.13 (4.38)	239.61 (1368.22)	0.028	-0.071	1.01	0.015	0.89	1.2

The concrete podium structure was modeled using two different approaches. First, the podium was considered to behave elastically, and then, it was modeled using an elastoplastic constitutive model where yielding could be captured. This approach was carried out to assess the effect of podium yielding on the input accelerations of the tower and the tower seismic performance.

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Since diaphragms were idealized as flexible, torsion was not considered, and 2D numerical models were considered adequate to represent the buildings.

In addition to the material nonlinearities incorporated in the numerical models, geometric nonlinearities (P-delta effects) were included explicitly. P-delta effects were modeled using a leaning column with no flexural capacity and large axial stiffness carrying the gravity load. Finally, mass and initial stiffness proportional Rayleigh damping was used with a critical damping of 3% assigned to the first and second modes.

4 – LATERAL STIFFNESS

The lateral stiffness of the podium and the tower were computed using the methodology described in ASCE7-22, where the design lateral forces were applied to the structure to obtain the maximum displacement at the roof. The stiffness was then calculated by dividing the total design lateral load (base shear) by the maximum displacement at the roof.

The lateral stiffness ratio computed using this procedure for the analyzed building was equal to 6.644. Lateral stiffness of the tower was computed using the OpenSees numerical model, and not as it would be done in common practice using SDPWS or using a commercial software.

In order to evaluate the influence of the lateral stiffness ratio, the podium stiffness was modified, but the strength capacity was considered to remain constant per story. This approach was implemented to assess the impact of inelastic behavior in the podium on the performance of the tower. The podium stiffness (*Ko*) was scaled up by different factors to evaluate the influence of lateral stiffness ratios. Table 2 summarizes the periods, stiffnesses, and lateral stiffness ratios used in this study.

Table 2. podium-tower Lateral Stiffness and Period Ratios.

	Stiffness - kN/mm		Period of Vibration (s)			
Lateral Stiffness Ratio	**Tower (Kip/in)**	**Podium (Ko) (Kip/in)**	**Tower (Tt)**	**Podium (Tp)**	**Podium + Tower (Tp+Tt)**	**Period Ratio (Tp+Tt)/Tt**
K_ratio=5	95.504 (545.342)	477.52 (2726.71)	0.655	0.371	0.780	1.192
Original Design (6.64)	95.504 (545.342)	634.51 (3623.188)	0.655	0.322	0.746	1.139
K_ratio=10	95.504 (545.342)	955.04 (5453.42)	0.655	0.262	0.712	1.088
K_ratio=15	95.504 (545.342)	1432.56 (8180.13)	0.655	0.214	0.692	1.056
K_ratio=20	95.504 (545.342)	1910.08 (10906.84)	0.655	0.185	0.682	1.041
K_ratio=25	95.504 (545.342)	2387.60 (13633.55)	0.655	0.166	0.676	1.033
K_ratio=30	95.504 (545.342)	2865.12 (16360.26)	0.655	0.151	0.673	1.027
K_ratio=50	95.504 (545.342)	4775.20 (27267.1)	0.655	0.117	0.665	1.016
K_ratio=80	95.504 (545.342)	7640.32 (43627.36)	0.655	0.093	0.661	1.010
K_ratio=500 (Rigid Podium)	95.504 (545.342)	47752.01 (272671)	0.655	0.032	0.656	1.001

This table shows that, for this building, the ASCE7-22 period ratio requirement is met when the lateral stiffness ratio exceeds 10. Additionally, period of the podium plays an important role in the total acceleration atop the podium, which is discussed later in the paper.

5 – METHODOLOGY

The numerical models were subjected to static pushover and incremental dynamic analyses to evaluate their collapse performance according to the FEMA P-695 methodology. Static pushover analyses were performed to quantify several parameters, such as overstrength and ductility. Incremental dynamic analyses were performed using the 44 Far Field ground motions where spectral accelerations were increased until collapse was determined, defined as when the structure could no longer carry either vertical or lateral load (dynamic instability) such as from side-sway collapse or exceedance of a specified inter-story drift of 8%. From these results, the probability of collapse for the maximum considered earthquake (MCE) was computed and compared among models.

6 – NONLINEAR STATIC PUSHOVER RESULTS

Nonlinear static pushover analysis was performed to compute the maximum load capacity and ultimate displacement, which were then used to compute the overstrength (Ω) and period-based ductility (μ) of the tower. The vertical distribution of the lateral force was proportional to the fundamental mode shape of the tower and was applied to the tower only.

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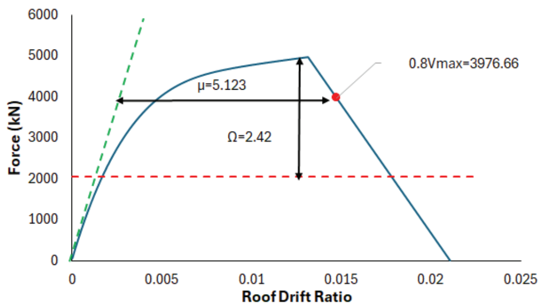

Fig.3 tower Pushover Curve

Fig. 3 shows the pushover curve computed for the tower, where the 5-story Light Frame Wood Structure has an overstrength of 2.42 and a period-based ductility of 5.123. The overstrength is defined as the ratio between the maximum load capacity and the design load. The period-based ductility was used to compute the Spectral Shape Factor (SSF) following the FEMA P-695 methodology, which in turn was required to evaluate the collapse performance of the structure.

The capacity of the tower (pushover curve) should not be affected by the podium because it is an intrinsic characteristic of the structure. However, Fig. 4 shows the pushover curves when the tower is placed atop podiums with different stiffnesses, where it can be seen that the maximum load capacity of the tower increases when the podium is flexible (lower stiffness ratio) and is equal to the original pushover curve of the tower when the podium is rigid.

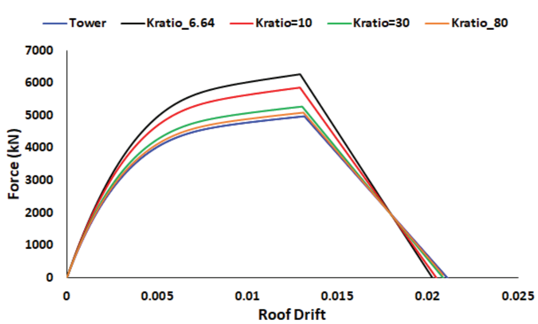

Fig.4 Pushover Curves Varying Lateral Stiffness Ratios

It is important to point out that the apparent gain in strength and ductility (Fig. 4) that a flexible podium provides influences the collapse performance evaluation when using the FEMA P-695 methodology. Table 3 shows the parameters computed using the results from the nonlinear static analyses. It can be seen how the period-based ductility increases for every case, including the case where the podium is rigid (K_ratio=500), even though for this specific case, the pushover curve is virtually the same as the one for the case where the tower is fixed to the ground. The increase in ductility is attributed to a reduction in the effective yielding displacement when the podium is considered in the analysis. However, this increase is not real, rather a problem that arises from applying the formulas outlined in the FEMA P-695 methodology when combined structures (podium + tower) are analyzed. Consequently, the period-based ductility and SSF for the tower fixed to the ground was used for all cases.

Table 3. Overstrength and Period Based Ductility Parameters

	Overstrength			Period Based Ductility				
Model	**Vmax - kN (Kips)**	**Vdesign - kN (Kips)**	**Ω**	**δu - mm (in)**	**T1**	**CuTa**	**δy,eff - mm (in)**	**μT**
Tower Tt=0.65	4971.29 (1117.59)	2053.03 (461.54)	**2.421**	224.79 (8.85)	0.655	0.530	43.88 (1.727)	**5.123**
K_ratio=6.64 (Original Design)	6264.80 (1408.38)	2053.03 (461.54)	**3.051**	219.46 (8.64)	0.746	0.530	36.01 (1.418)	**6.094**
K_ratio=10	5854.75 (1316.12)	2053.03 (461.54)	**2.852**	219.96 (8.66)	0.712	0.530	29.88 (1.176)	**7.362**
K_ratio=20	5418.66 (1218.16)	2053.03 (461.54)	**2.639**	221.74 (8.73)	0.682	0.530	24.15 (0.951)	**9.181**
K_ratio=30	5270.03 (1184.75)	2053.03 (461.54)	**2.567**	222.50 (8.76)	0.673	0.530	22.37 (0.881)	**9.948**
K_ratio=80	5803.02 (1142.71)	2053.03 (461.54)	**2.476**	223.77 (8.81)	0.661	0.530	20.24 (0.797)	**11.054**
K_ratio=500 (Rigid Podium)	4984.21 (1120.49)	2053.03 (461.54)	**2.428**	224.54 (8.84)	0.656	0.530	19.17 (0.755)	**11.712**

7 – TOTAL ACCELERATION VS GROUND ACCELERATION

The structural dynamics concept proposed for the Two-Stage analysis methodology assumes that the total acceleration is the same as the ground acceleration (opposite sign) when the structure is infinitely rigid [7].

The total acceleration at the top of the podium was measured for various stiffness ratios. The performed analyses consider both elastic and inelastic behavior of the podium. The ground motions were scaled following the FEMA P-695 methodology to represent the Maximum Considered Earthquake.

Fig.5 a) and b) show the total acceleration measured at the top of the podium at MCE and 1.5MCE, respectively, for the case where the lateral stiffness ratio is 6.64 (original design). In order to have a better visual of the results, only ten seconds of the total acceleration time is shown.

Different conclusions were drawn from these results. One of the most evident conclusion was that total acceleration decreases when the podium yields, with a more significant reduction as ground motion intensity increases (i.e., more yielding). Another key finding emerged when comparing total acceleration atop the podium with ground acceleration. Two observations stood out: (1) total acceleration at the top of the podium exhibited amplified peaks compared to the ground

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acceleration, and (2) the frequency at the top of the podium changes, revealing a dominant frequency that matched the natural frequency of the podium. Figures 5a and 5b highlight instances where total acceleration peaks occur, with a time interval of 0.32s between them, corresponding to the period of the podium (Table 2).

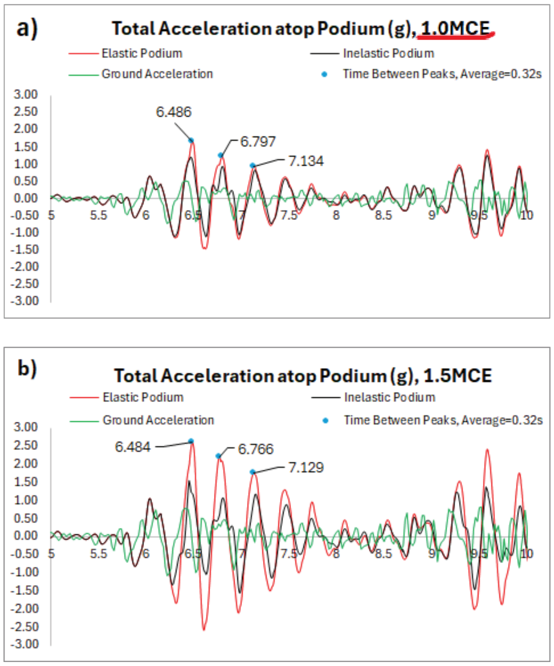

Fig.5 Total vs Ground Acceleration Kratio=6.64 (Original Design): a) MCE, b)1.5MCE

Fig. 6a and 7a display the total acceleration results for lateral stiffness ratios of 30 and 80 between the podium and tower, respectively. Although a lateral stiffness ratio of 30 is three times higher than the ASCE 7 requirement, the total acceleration exhibited some peak amplifications compared to the ground motion, with the frequency of the podium also being dominant in the signal.

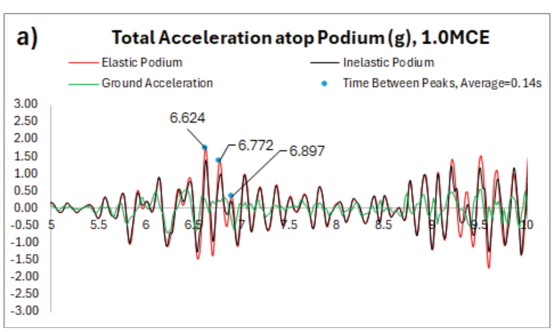

Fig.6 Total vs Ground Acceleration Kratio=30

It is important to note that even with a large lateral stiffness ratio of 80 (Fig. 7a), the total acceleration still exhibited small peak amplifications. In this case, the total acceleration was nearly equal to the ground acceleration.

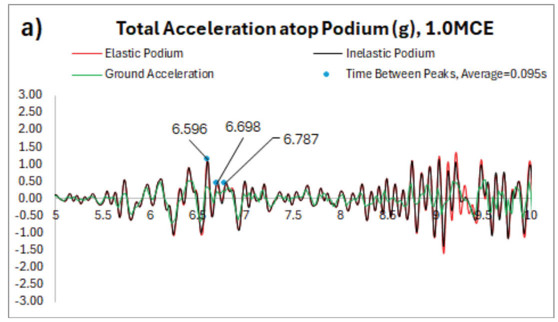

Fig.7 Total vs Ground Acceleration Kratio=80

8 – COLLAPSE PERFORMANCE EVALUATION

The performance of the building numerical models was evaluated using 44 Far-Field ground motions, and Incremental Dynamic Analysis was conducted to assess collapse. Based on previous studies, collapse was assumed to occur when the story drift reached 8% or when dynamic instability was observed. Once the scale factor that triggered collapse for each ground motion was determined, the collapse margin ratio (CMR) was calculated. The CMR is defined as the ratio of the spectral acceleration that causes a median collapse (22 out of 44 ground motions) to the spectral acceleration at the MCE level.

While varying the lateral stiffness ratio, the probability of collapse was used as a metric to compare the performance of the tower fixed to the ground with that of the same tower placed atop a podium. In addition, the effect of yielding in the podium was studied by analyzing the tower placed atop podiums that exhibited either elastic and inelastic behavior. The probability of collapse was computed at the MCE level using the lognormal of Adjusted Collapse Margin Ratio (ACMR) as the mean and the uncertainties ($\beta = 0.5$) as the standard deviation. ACMR, as defined in the FEMA P-695 methodology, is obtained by multiplying the Collapse Margin Ratio (CMR) with the Spectral Shape Factor (SSF). This accounts for differences between the spectral shape of rare ground motions in California and the design spectrum or a uniform hazard spectrum.

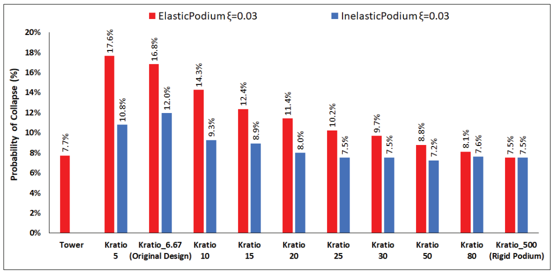

Fig. 8 Collapse Evaluation for Varying Kratio

Fig. 8 presents the collapse evaluation results for two scenarios: the tower fixed to the ground and the tower placed atop a podium that behaves elastically and inelastically. As seen in the figure, the collapse performance of the tower worsens when placed atop the podium for the original design, which had a stiffness ratio of 6.64. Notably, the ASCE7 lateral stiffness requirement of 10 did not entirely prevent the podium from affecting the performance of the tower. Specifically, for a lateral stiffness ratio of 10, the probability of collapse increased from 7.7% for the tower fixed to the ground to 14.3% and 9.3% for the tower atop elastic and inelastic podiums, respectively.

Another important finding was that, as seen earlier in Fig. 5 that total acceleration decreased when the podium yielded, there was improvement in the probability of collapse when the podium behaved inelastically.

These results also demonstrated that the total acceleration matched the ground motion acceleration when the podium was rigid (Kratio=500). Consequently, the probability of collapse remained the same whether the tower was fixed to the ground or placed atop the podium, regardless of whether the podium behaved elastically or inelastically.

9 – CONCLUSIONS

After performing analyses using the FEMA P-695 methodology and comparing the collapse performance of the tower fixed to the ground with the tower atop the podium, the following conclusions can be made:

- The lateral stiffness ratio between the podium and the tower can affect the performance of the tower, and the lateral stiffness ratio of ten recommended by ASCE 7 did not guarantee a similar performance when compared the tower fixed to the ground with the tower atop the podium.
- The inelastic behavior of the podium reduces the input accelerations for the tower, improving the performance compared to the results when the podium behaves elastically.
- The total acceleration measured at the top of the podium is not the same as the ground acceleration unless the lateral stiffness ratio of the podium to the tower is very high.
- The ground acceleration is amplified at the top of the podium, and its frequency content is modified. The natural frequency of the podium is embedded and predominant in the total acceleration atop the podium. Thus, resonance with higher modes could occur and more research is recommended to study this effect.

10 – REFERENCES

1. American Society of Civil, E., Minimum design loads and associated criteria for buildings and other structures: provisions. 2022, American Society of Civil Engineers: Reston, Virginia.
2. FEMA, Quantification of building seismic performance factors : component equivalency methodology. 2011, U.S. Department of Homeland Security, FEMA, NEHRP [Washington, D.C.]: [Washington, D.C.].
3. Mcbain, M., Evaluation of Two-Stage Seismic Analysis and Design Provisions for Multi-Story Buildings. 2023, SEAONC.
4. AWC, Special Design Provisions for Wind and Seismic. 2021, AWC Leesburg, VA.
5. McKenna, F., OpenSees: a framework for earthquake engineering simulation. Computing in Science & Engineering, 2011. 13(4): p. 58-66.
6. Folz, B. and A. Filiatrault, SAWS-Version 1.0, A computer program for the seismic analysis of woodframe structures. Structural Systems Research Project: SSRP-2001, 2001. 9.
7. Chopra, A.K., Dynamics of structures : theory and applications to earthquake engineering. Sixth edition ed. 2025, Hoboken, NJ: Pearson.

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