A BEAM-SPRING ANALOG MODEL FOR SEISMIC ANALYSIS OF SEMI-RIGID WOOD DIAPHRAGMS

WeiChiang Pang¹, David Rosowsky²

ABSTRACT: This paper presents a beam-spring analog model for use in seismic analysis of wood diaphragms in North American style light-frame wood construction. A roof or floor diaphragm in a wood building generally spans across multiple lateral force resisting elements (shear walls). The diaphragm serves as a horizontal beam that distributes forces to the shear walls. In seismic design, the wood diaphragm is commonly assumed to be either completely flexible or completely rigid. Strictly speaking, the behavior of a wood diaphragm is neither flexible nor fully rigid. Full-scale shake table tests of a two-story woodframe structure have confirmed that roof and floor diaphragms are semi-rigid. This paper examines the effect of diaphragm flexibility on shear wall deflections by considering the in-plane stiffness of the diaphragm to be semi-rigid. A beam-spring analog model is used to represent the diaphragm-shear wall system where the shear walls are modeled as springs and the diaphragm is modeled as an analog beam which acts as a load distribution mechanism. The resulting load sharing among the shear walls is examined and possible application of the beam-spring model to seismic design of wood-frame structures is discussed.

KEYWORDS: wood diaphragm, shear wall, semi-rigid, beam-spring model, load sharing

1 INTRODUCTION

In North America, light-frame wood construction is the dominant building method used in both low-rise single-family homes and mid-rise multi-story residential structures. Light-frame wood construction offers advantages over other building methods (e.g., reinforced concrete and steel moment frames) in the form of lower construction costs and faster overall construction time. However, light-frame wood buildings are extremely difficult to accurately model or analyze due to the complex interactions between the interconnected subassemblies and framing members.

A light-frame wood building typically consists of dimension lumber as framing members (wall studs, floor joists and roof trusses) to which various sheathing materials such as plywood or oriented strand board (OSB) is attached. The framing members and sheathing when oriented horizontally form the floor and roof diaphragms, and when oriented vertically form the partition walls and shear walls. A typical diaphragm in a wood building has two primary functions. The first is to carry gravity loads and transfer them to the load bearing walls. Additionally, a diaphragm acts as a horizontal beam that collects lateral forces due to earthquakes or high wind events and transfers them to the shear walls.

In a wood building, diaphragms and shear walls serve as the primary lateral-force-resisting systems. A diaphragm in a wood building generally spans across multiple shear walls. The distribution of lateral forces or load sharing among the shear walls depends on the flexibility of the diaphragm. For design purposes, it is common to assume the wood diaphragm is either completely flexible or completely rigid. In low-rise light-frame wood construction, wood diaphragms are almost always designed as flexible diaphragms, and modeled as simple beams spanning across two adjacent shear walls. Strictly speaking, neither the flexible nor the rigid assumption is accurate for modeling the behavior of a wood diaphragm. A series of full-scale shake table tests, conducted as part of the NEESWood project, has confirmed that roof and floor diaphragms are indeed semi-rigid [1]. In this paper, the effect of diaphragm flexibility on shear wall deflections is examined by analyzing the NEESWood test building using two beam-spring analog models: 1) a finite element-based, and 2) a simplified beam-spring models. In both cases, the shear walls are modeled as non-linear springs and the semi-rigid behavior of a diaphragm is modeled using an analog beam which acts as a load distribution mechanism.

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2 FLEXIBILITY OF DIAPHRAGM

Diaphragms are classified as either flexible or rigid in the US design codes [2][3] for the purpose of distributing the design seismic forces to the shear walls. A diaphragm that is designated as flexible is assumed to distribute the seismic forces based on tributary width or area while a rigid diaphragm is assumed to distribute seismic forces to the shear walls in proportion to the walls’ stiffness.

The provisions for determining diaphragm flexibility can be found in section 1613.6.1 of the current edition of the International Building Code [3] and in section 12.3.1.1 of the ASCE/SEI 7-05, Minimum Design Loads for Buildings and Other Structures [2]. According to IBC 2009, wood diaphragms are permitted to be idealized as flexible if 1) the diaphragms are constructed of wood structural panels with no more than 38 mm thick of non-structural topping; 2) each line of vertical elements (shear walls) complies with the allowable story drift of Table 12.12-1 in IBC 2009 [3]; 3) the shear walls are sheathed with wood structural rated shear panels; 4) the cantilevered portions of wood diaphragms are designed in accordance with section 4.2.5.2. of AF&PA Special Design Provisions for Wind and Seismic (SDPWS) [4].

The aforementioned criteria are applicable to most regular one- and two-story light-frame wood buildings, thus flexible diagrams can be assumed for most single-family dwellings. However, a two-story single-family house tested as part of the CUREE-Caltech Wood-frame Project [5] clearly exhibited rigid diaphragm behavior. In contrast, shake table results obtained from the NEESWood test program seem to suggest that the roof and floor diaphragms are semi-rigid [1]. Since the size of the CUREE test structure is significantly smaller than the average size of single-family homes in the US, it was not used in the numerical study presented in this paper. Rather, the larger two-story NEESWood structure is selected as the benchmark for evaluating the effect of diaphragm flexibility on shear wall deflections.

3 TWO-STORY TEST STRUCTURE

A two-story light-frame wood building (Figure 1) was built and a series of shake table tests were performed, as part of the NEESWood project, to investigate the seismic performance of the test structure in full-scale. This NEESWood Benchmark structure is representative of a typical townhouse building constructed in the 1980’s and located in Southern California. The test structure had approximately 170 m² of living space and an attached two-car garage. The story height of the structure was 2.74 m. All shear walls were constructed using nominal 51 mm × 102 mm dimension Hem Fir studs except for those shear walls located on the west side of the 1st floor (garage walls), where 51 mm × 152 mm studs were used. In order to study the influence of wall finish materials on the seismic response of the test building, multiple seismic tests were conducted at various stages of construction (Table 1). Only the Phases 1 and 3 test structures are considered in this study.

4 FINITE ELEMENT BEAM-SPRING MODEL FOR TEST STRUCTURE

A numerical model for the test structure was constructed using a specialized non-linear dynamic time-history analysis program developed for light-frame wood structures, called M-SAWS (MATLAB – Seismic Analysis of Woodframe Structures). In the M-SAWS model, shear walls are modeled as non-linear single degree-of-freedom (SDOF) springs using the Modified Stewart Hysteretic model [7] and diaphragms are modeled using linear two-node beam elements (Figure 2). The Modified Stewart Hysteretic model is essentially a non-linear single degree-of-freedom (S DOF) spring
which includes hysteretic pinching, strength and stiffness degradation. The modeling parameters for the Modified Stewart Hysteretic model are shown graphically in Figure 3 and the backbone equation for the hysteretic model is given below:

\[
F_b(\Delta) = \begin{cases} 
1 - e^{\frac{\Delta}{\Delta_u}} (rK_0\Delta + F_u) & \text{for } \Delta \leq \Delta_u \\
F_u + rK_0(\Delta - \Delta_u) & \text{for } \Delta > \Delta_u
\end{cases}
\]  

Further details on the Modified Stewart Hysteretic model are available in [7].

In order to consider the in-plane deformation of diaphragms, master nodes were placed on the floor and roof diaphragms of the test structure along the transverse wall lines 2, 4, 5 and 6 and these wall lines were connected together through beam elements (Figure 2). Each master node has one rotational and two in-plane translational degrees-of-freedom. The beam elements permit relative movements between the master nodes in the transverse direction (N-S) therefore can be used to model the semi-rigid behavior of diaphragms.

Supplemental weights were installed in the test structure to account for the weight of finish materials and to maintain similar total weight throughout the test phases. The effective seismic weights at several key locations of the test structure were determined experimentally through white noise tests [1]. To model the weight distribution, lumped seismic weights were assigned to slave nodes at these key locations and rigid axial elements were used to connect these slave nodes to the master nodes.

### 4.1 NON-LINEAR SHEAR WALL SPRING

The shear walls in the test building were sheathed with 11 mm thick OSB connected to the framing members using 8d common nails (63.5 mm long × 3.3 mm in diameter). The locations of shear walls are shown in Figure 1 along with the perimeter sheathing-to-frame nail spacing of each shear wall. All shear walls had the same interior nail spacing (305 mm). Also shown in Figure 1 are the locations of hold-down devices. Note that hold-down devices were installed in selected first-story shear walls only. As a result, significant wall uplift (separation between end studs and bottom plate) was recorded at some shear wall locations during the seismic tests. This indicates that the shear walls in the test building were not fully anchored to the foundation.

In order to model the rocking response of the partially anchored shear walls, a specialized shear wall analysis program, called M-CASHEW2 (MATLAB – Cyclic Analysis of wood SHEar Walls version 2), was developed [6]. The M-CASHEW2 program can be used to predict the non-linear force-displacement response at the top of the wall by explicitly modeling the relative movements of the wall components (nails, sheathing panels and framing members). The M-CASHEW2 was developed from its predecessor, a FORTRAN version of the CASHEW program [7]. The M-CASHEW2 program can be used to create detailed shear wall models that explicitly account for 1) the bending and axial elongation of framing members, 2) the shear deformation of the sheathing panels, 3) the non-linear force-slip response of sheathing-to-frame connections (i.e. 8d common nails in this study), 4) the anchorage effect of hold-down devices, 5) the withdrawal of end nail connections, 6) the effect of gravity loading, and 7) the separation/contact effect between the framing members.

![Figure 2: Finite Element Model for Test Structure.](image)

The M-CASHEW2 program was used to predict the cyclic response of each shear wall in the test building. Figure 3 shows an example deformed shape obtained from M-CASHEW2 for wall E11 located in the first story and parallel to wall line 4. In order to obtain the non-linear force-displacement response of wall E11, a reversed cyclic simulation was performed using a displacement controlled cyclic loading protocol applied at the top of the wall. Note that a uniform gravity load was applied on the double-top plate of the shear wall. Assume that the total gravity load (~340 kN) is carried by the load bearing walls in the first story, the average uniform gravity load on the top of the first story shear
walls is determined to be 6.38 kN/m (computed as the total seismic weight divided by the total length of the first story bearing walls, 53.3m). Using the same approach, the uniform gravity load for shear walls on the second floors is 1.97 kN/m (105 kN / 53.3 m). The top of wall force-displacement response was fitted to the 10-parameter Modified Stewart Hysteretic spring model. This modeling process was repeated for each shear wall in the test building and the complete list of equivalent SDOF parameters are provided in [8].

4.2 SEMI-RIGID DIAPHRAGM MODEL

The in-plane behavior of the floor and roof ceiling diaphragms is very similar to that of shear walls. Consider the floor diaphragm as an example, it is essentially a “shear wall” oriented horizontally. Therefore, M-CASHEW2 also was used to estimate the in-plane force-displacement response of the floor diaphragm. The floor diaphragm was constructed of nominal 51 mm × 305 mm Douglas Fir joists spaced at 406 mm on-center and sheathed with 19 mm thick OSB connected to the joists using 10d common nails (76.2 mm long × 3.8 mm in diameter). The 10d nails were spaced at 152 mm along the edges of the panel and 254 mm in the field of the panel. The roof system consisted of trusses spaced at 610 mm on-center. In this study, only the bottom chords of the roof trusses are considered as part of the roof ceiling diaphragm.

In the M-SAWS model, the floor diaphragm was divided into three segments and each segment was analyzed separately (Figure 2). The diaphragm model employed in this study is a one-way flexible diaphragm model. The axial elongation or compression along the longitudinal direction of the diaphragm is ignored. The relative movement between the wall lines parallel to the transverse direction is modeled using a two-node beam element and the stiffness matrix of the beam element is:

\[
K_{beam} = \begin{bmatrix}
\frac{AE}{E} & -\frac{AE}{E} & 0 & 0 \\
-\frac{AE}{E} & \frac{AE}{E} & 0 & 0 \\
0 & 0 & \frac{12EI}{L} & 0 \\
0 & 0 & 0 & \frac{12EI}{L}
\end{bmatrix}
\begin{bmatrix}
u_1 \\ v_1 \\ u_2 \\ v_2
\end{bmatrix}
\]

(2)

where \(u\) and \(v\) are the displacements parallel to the longitudinal and transverse directions, respectively. The length of each element or diaphragm segment, \(L\), is shown in Figure 1. Since the axial elongation of the element was not considered, a very large value was assigned to the element cross-sectional area, \(A\), to restrain the elongation of diaphragm along the longitudinal direction. \(E\) and \(I\) are the modulus of elasticity of Douglas Fir lumber (12410 MPa) and the equivalent moment of inertia. In order to estimate the effective \(I\) value, a displacement controlled monotonic pushover analysis was conducted for each diaphragm segment to obtain the force-displacement backbone curve. Figure 5 shows the backbone curve for the middle segment of the floor diaphragm obtained using M-CASHEW2. The effective \(I\) can be estimated using the following equation:

\[
I = \frac{K_d L^2}{E}
\]

(3)

where, \(K_d\) is the stiffness (slope) of the backbone curve. From the middle segment of the floor diaphragm, the effective \(I\) was 22,476 cm^4 \((0.0742\ kN/mm \times (3.35\ m)^3 / 12410\ MPa)\). The effective \(I\) values are given in Table 2.

![Semi-rigid Diaphragm Model](image)

Table 2: 2-node (diaphragm) beam element properties.

<table>
<thead>
<tr>
<th>Element ID</th>
<th>Location</th>
<th>(E) (MPa)</th>
<th>(I) (cm^4)</th>
<th>(L) (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B1</td>
<td>Floor West</td>
<td>12410</td>
<td>265,556</td>
<td>5.87</td>
</tr>
<tr>
<td>B2</td>
<td>Floor Middle</td>
<td>12410</td>
<td>22,476</td>
<td>3.35</td>
</tr>
<tr>
<td>B3</td>
<td>Floor East</td>
<td>12410</td>
<td>261,810</td>
<td>6.86</td>
</tr>
<tr>
<td>B4</td>
<td>Roof West</td>
<td>12410</td>
<td>224,765</td>
<td>5.87</td>
</tr>
<tr>
<td>B5</td>
<td>Roof Middle</td>
<td>12410</td>
<td>16,649</td>
<td>3.35</td>
</tr>
<tr>
<td>B6</td>
<td>Roof East</td>
<td>12410</td>
<td>191,466</td>
<td>6.86</td>
</tr>
</tbody>
</table>

5 TIME-HISTORY ANALYSES

The 1994 Northridge, Canoga Park earthquake motions scaled to peak ground accelerations (PGA) equal to 0.05g (Level 1), 0.22g (Level 2) and 0.36g (Level 3) were used as the input table motions for Phases 1 and 3.
tests [1]. Eight seismic tests were conducted in Phase 1 and four tests were performed in Phase 3 (Figure 6). The designations “3D” and “2D” mean tri-axial and bi-axial ground motions, respectively. Similarly, “1X” and “1Y” mean the applied ground motions were uni-axial ground motions parallel to the longitudinal and transverse directions, respectively. The vertical ground accelerations are not shown in Figure 6. In dynamic time-history analyses, only the horizontal ground accelerations were considered. The Rayleigh damping ratios assigned for the test structure were 0.01 (1% of critical damping) and were associated to the 1st and 3rd vibration modes. Similar damping ratios have been used in other numerical studies [Error! Reference source not found. , , , ].

Figure 6: Ground motions for Phases 1 and 3 Tests.

6 COMPARISON BETWEEN SEMI-RIGID AND RIGID DIAPHRAGM MODELS

The M-SAWS program was used to obtain all numerical predictions presented in this paper. In addition to the previously discussed beam-spring model (for semi-rigid diaphragm), a rigid diaphragm model also was created for each test phase. Table 3 shows the comparison between the initial natural periods estimated through white noise tests [1] and the M-SAWS model predicted periods. In general, the numerical models over-predicted the initial periods. This may be attributed to the fact that the M-SAWS model considers only the in-plane stiffness of shear walls while the additional out-of-plane wall stiffness is ignored.

In general, both semi-rigid and rigid diaphragm models were able to predict the first story drifts well. As observed in the test, both models predicted maximum first story drifts at the garage wall (wall line 6). In the second story, however, the beam-spring model was able to reproduce the test deformed shape while the rigid diaphragm model was not (Figure 7). The rigid diaphragm model predicted peak second story drift in the exterior wall (Line 6) while the actual recorded peak second story drift occurred in one of the interior wall lines (Line 4). In the rigid diaphragm model, the roof ceiling diaphragm deformed and rotated in a rigid body motion. Hence, the maximum drift in rigid diaphragm could only occur in one of the exterior walls. This means a rigid diaphragm model will under-predict the drift demand in interior wall lines (Figure 8).

Table 3: Initial Periods of Test Structure (second).

<table>
<thead>
<tr>
<th>Mode</th>
<th>Test</th>
<th>Beam-Spring</th>
<th>Rigid Diaphragm</th>
<th>Test</th>
<th>Beam-Spring</th>
<th>Rigid Diaphragm</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.33</td>
<td>0.38</td>
<td>0.36</td>
<td>0.30</td>
<td>0.32</td>
<td>0.30</td>
</tr>
<tr>
<td>2</td>
<td>0.23</td>
<td>0.32</td>
<td>0.31</td>
<td>0.22</td>
<td>0.27</td>
<td>0.26</td>
</tr>
<tr>
<td>3</td>
<td>0.18</td>
<td>0.26</td>
<td>0.26</td>
<td>0.17</td>
<td>0.23</td>
<td>0.23</td>
</tr>
</tbody>
</table>

Figure 7: Maximum Inter-story Drifts (Phase 1).

Figure 8: Example Deformed Shapes (Phase 1).
The test and models predicted maximum inter-story drifts for Phase 3 structure (structural panels + GWB) are presented in Figure 9. As seen in the Phase 1 simulation results, the Phase 3 rigid diaphragm model also displayed linear deformed shapes in both the first and second stories. Compared to the Phase 1 results, the discrepancies between the test and the rigid diaphragm model were even more apparent in Phase 3. The rigid diaphragm model greatly underestimated the drifts in the second story (by as much as 50%). This is attributed to the fact that the rigid diaphragm model overestimated the load sharing between the shear walls. Under-predictions of drift response in the second story were also reported in a separate study in which a rigid diaphragm assumption was utilized to model a two-story wood building [9]. In contrast, the beam-spring model predicted drifts for all four wall lines are in good agreement with the test results.

In the simplified beam-spring model, each spring carries a point load which is equal to the lumped mass, m, times the spring acceleration, a. A similar beam-spring model subjected to uniform load has been used by others to study the load-sharing effect of floor joists [ ] and wall systems [ ]. Following the modeling technique employed in the uniformly loaded beam-spring model, the system of equations for this point-load beam-spring model is derived using the consistent deformation approach [ ]:

\[
[Y_i + \lambda_{ij}] = \begin{bmatrix} 0 & 0 & 0 & 0 & 0 & 0 & 0 & 0 & \cdots & 0 & 0 & 0 & 0 \\
0 & k_1 & 0 & 0 & 0 & 0 & 0 & 0 & \cdots & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & k_{i-1} & 0 & 0 & 0 & 0 & \cdots & 0 & 0 & 0 & 0 \\
0 & 0 & 0 & 0 & 0 & \cdots & 0 & 0 & \cdots & 0 & 0 & 0 & 0 \\
\end{bmatrix} \{\Delta\} = \begin{bmatrix} 0 \\
k_1 \\
k_{i-1} \\
0 \\
\end{bmatrix} \{m_i a_i\}
\]

where subscripts i and j are the matrix row and column numbers, respectively. I is the identity matrix, k is the shear wall (spring) stiffness and \(\Delta\) is the shear wall deflection. The \(\gamma\) and \(\lambda\) matrices are given in the following equations:

\[
\gamma_{ij} = \begin{cases} 
\frac{x_i (L-x_j) (x_j^2 - 2Lx_j + x_i^2)}{6EIL} & x_i < x_j \\
\frac{x_j (L-x_j) x_i^2 - 2Lx_j + x_i^2}{3EIL} & x_i = x_j \\
\frac{x_j (L-x_i) x_j^2 - 2Lx_j + x_i^2}{6EIL} & x_i > x_j \\
\end{cases}
\]

(5)

\[
\lambda_{ij} = \begin{cases} 
\frac{1}{EIL} \left( \frac{(L-x_j)(L-x_i)}{k_i} + \frac{x_j x_i}{k_n} \right) & \end{cases}
\]

(6)

where \(EIL\) is the in-plane bending stiffness of the diaphragm (beam), \(x\) is the shear wall location measured from the left end of the diaphragm, and \(L\) is the length of the diaphragm.

8 ILLUSTRATIVE EXAMPLE

As an illustrative example, the load sharing behavior among the shear walls in the second story of the Phase 1 test structure is analyzed using the simplified beam-spring model. In the beam-spring model, non-linear
shear walls are approximated using equivalent elastic springs. The elastic stiffness is determined by connecting a straight line between the origin and a point on the non-linear backbone curve at 75% of the peak backbone force, $F_a$ (Figure 3). Table 6 summarizes the properties of the roof beam-spring model. The spring stiffness, $k$, was determined using the M-CASHEW2 program and the shear wall backbone equation (1). It should be noted that wall E21 is not directly in line with wall E23 (Figure 10a), but for simplicity, the stiffness for spring no. 4 (wall line 3) is computed as the sum of the elastic stiffnesses of wall E21 and E23.

In seismic design, one will determine the spring acceleration, $a$, based on the seismic hazard for the location of the structure. The spring acceleration can be approximated using the design spectral acceleration, $S_a$, specified in building codes ([2][3]). In this example, the roof ceiling acceleration is taken as 1.5g. This magnitude of acceleration is in line with the code specified spectral acceleration at the Maximum Considered Earthquake (MCE) level for wood structures located in Southern California [14]. While an assumed spectral acceleration is used in this example, further study is needed to determine an appropriate procedure for estimating the design acceleration for use with the beam-spring model.

Three diaphragm flexibility conditions are investigated: 1) semi-rigid, 2) rigid, and 3) flexible. For the semi-rigid condition, $EI$ is equal to the bending stiffness of the beam element B4 shown in Table 2 (2.780×1010 kNm²). Note that $EI$ cannot equal to zero. Thus, for the flexible diaphragm condition, $EI$ is set to a very small value (1×10^{-6}EI). Similarly, for the rigid diaphragm condition, $EI$ of 1000 times the stiffness of the semi-rigid diaphragm is used. For all three conditions, the length of the beam-spring model is equal to 16.21 m.

Substituting the properties listed in Table 4 into equations (4) to (6) and solving for the deflections, $Δ$, yields the results shown in Figure 11. The deformed shape of the semi-rigid condition predicted using the simplified beam-spring model is very similar to that predicted by the more detailed FE model and the actual test results. As expected, the rigid diaphragm model has a linear deformed shape. Compared to the results of the semi-rigid and flexible models, the rigid diaphragm analysis yields smaller displacements along the interior wall lines 5 and 4 and larger displacements along the exterior walls. This phenomenon was also observed in the previous FE analysis (Figure 7). The ability of the simplified beam-spring model to reproduce the deformed shape of the actual test data makes it a suitable model for use in a performance-based seismic design framework [14] where accurate prediction of shear wall displacements is needed.

<table>
<thead>
<tr>
<th>Wall Line</th>
<th>6</th>
<th>5</th>
<th>4</th>
<th>2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spring No.</td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
</tr>
<tr>
<td>$m$ (kN/g)</td>
<td>17.65</td>
<td>33.15</td>
<td>33.95</td>
<td>19.65</td>
</tr>
<tr>
<td>$x$ (m)</td>
<td>0.00</td>
<td>5.94</td>
<td>9.30</td>
<td>16.21</td>
</tr>
<tr>
<td>$k$ (kN/mm)</td>
<td>1.401</td>
<td>0.908</td>
<td>1.040</td>
<td>1.186</td>
</tr>
</tbody>
</table>

9 CONCLUSIONS

A finite element-based and a simplified beam-spring models are developed to model the interaction between diaphragms and shear walls in light-frame wood structures. In a beam-spring model, shear walls are modeled as springs and the diaphragm is modeled as an analog beam which acts as a load distribution mechanism. The first beam-spring model presented is a finite element (FE) model which can be used in non-linear dynamic time-history analyses of the complete light-frame wood structure. The second simplified beam-spring model is developed specifically for use in seismic design.

The FE beam-spring model is coded into a specialized time-history analysis program developed for light-frame wood structures (M-SAWS). Using the M-SAWS program, two numerical models were constructed to predict the actual shear wall displacements of a two-story wood-frame structure, namely the NEESWood Benchmark structure, tested in a series of full-scale shake table experiments. The shear walls in the test structure were modeled using non-linear single degree-of-freedom (SDOF) springs. A separate shear wall analysis program, M-CASHEW2, was developed to predict the non-linear shear wall responses (i.e., SDOF springs). The effect of diaphragm flexibility on shear wall deflections is examined by considering the in-plane stiffness of diaphragm to be semi-rigid and completely rigid. Good agreement was observed, in terms of the magnitude of the displacements and deformed shapes, between the numerical predictions of the semi-rigid FE beam-spring model and the actual experimental results. On the other hand, the FE analyses showed that the rigid diaphragm model failed to reproduce the actual deformed shapes observed in the shake table experiments, especially of the roof diaphragm.

Finally, the formulation of a simplified beam-spring model is described and possible application of this model to seismic design was presented. Three diaphragm flexibility conditions (semi-rigid, rigid and completely flexible) were modeled using the proposed simplified beam-spring model. The modeling results confirmed that the simplified beam-spring model is capable of estimating the displacements predicted using the more detailed FE model with reasonable accuracy. While this simplified beam-spring model has been shown to be an...
attractive tool for use in performance-based seismic design, further study is still needed to determine an appropriate method for estimating the acceleration demand.

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