INVESTIGATING THE INFLUENCE OF DIAPHRAGM FLEXIBILITY ON THE SEISMIC RESPONSE OF COLD-FORMED STEEL STRUCTURES

V. Nikolaidou 1, C.A. Rogers2 and D.G. Lignos3

ABSTRACT

In recent years, there has been an increase in the construction of cold-formed steel (CFS) framed multi-story residential and commercial buildings. In the process of designing these structures, needed improvements to the current seismic design provisions of CFS framed buildings (AISI S400 & S100, CSA S136) have been identified. At present, there is no design procedure available for CFS framed diaphragms in Canada; as well, there has been little research on CFS framed diaphragm design and its isolated seismic performance. This paper contains the findings of an investigation of the effect of the in-plane diaphragm flexibility on the seismic response of a cold-formed steel framed structure subjected to earthquake excitations. The CFS-NEES Building was chosen as the case study structure and modeled in the OpenSees platform, including the detailed simulation of its floor and roof subassemblies. Three diaphragm stiffness conditions were considered, flexible, semi-rigid and rigid. The diaphragm flexibility influences the fundamental period, the mode shapes and total maximum floor displacement; results indicate that the resulting base shear forces and drifts for the three cases are dependent on the input ground motion.

1PhD Candidate, McGill University, Department of Civil Engineering and Applied Mechanics, Montreal, Canada, (violetta.nikolaidou@mail.mcgill.ca)
2Associate Professor, McGill University, Department of Civil Engineering and Applied Mechanics, Montreal, 817 Sherbrooke Street West, Montreal QC, Canada, H3A 0C3 (colin.rogers@mcgill.ca)
3Associate Professor, Swiss Federal Institute in Lausanne (EPFL), School of Architecture, Civil & Environmental Engineering, Lausanne, Switzerland (dimitrios.lignos@epfl.ch)

Investigating the Influence of Diaphragm Flexibility on the Seismic Response of Cold-Formed Steel Structures

V. Nikolaidou¹, C.A. Rogers² and D.G. Lignos³

ABSTRACT

In recent years, there has been an increase in the construction of cold-formed steel (CFS) framed multi-story residential and commercial buildings. In the process of designing these structures, needed improvements to the current seismic design provisions of CFS framed buildings (AISI S400 & S100, CSA S136) have been identified. At present, there is no design procedure available for CFS framed diaphragms in Canada; as well, there has been little research on CFS framed diaphragm design and its isolated seismic performance. This paper contains the findings of an investigation of the effect of the in-plane diaphragm flexibility on the seismic response of a cold-formed steel framed structure subjected to earthquake excitations. The CFS-NEES Building was chosen as the case study structure and modeled in the OpenSees platform, including the detailed simulation of its floor and roof subassemblies. Three diaphragm stiffness conditions were considered, flexible, semi-rigid and rigid. The diaphragm flexibility influences the fundamental period, the mode shapes and total maximum floor displacement; results indicate that the resulting base shear forces and drifts for the three cases are dependent on the input ground motion.

Introduction

Improving the seismic design of cold-formed steel (CFS) structures has been of great interest in recent years, leading multiple groups in Canada, the USA and elsewhere to carry out research [1,2,3,4,5,6,7,8]. The goal of these various research programs has been to facilitate the work of professional engineers by providing a comprehensive design process for individual components of a CFS structure, e.g. shear walls, gravity walls, chord studs, diaphragms, etc. Numerous numerical and experimental studies on shear walls have offered valuable information [9,10,11,12], while limited experimental and numerical work was recently completed on the lateral response of CFS framed/wood sheathed diaphragms including non-structural components [4,5,8]. The need for an ongoing investigation on the response and influence of the floor and roof

¹PhD Candidate, McGill University, Department of Civil Engineering and Applied Mechanics, Montreal, Canada, (violetta.nikolaidou@mail.mcgill.ca)
²Associate Professor, McGill University, Department of Civil Engineering and Applied Mechanics, Montreal, 817 Sherbrooke Street West, Montreal QC, Canada, H3A 0C3 (colin.rogers@mcgill.ca)
³Associate Professor, Swiss Federal Institute in Lausanne (EPFL), School of Architecture, Civil & Environmental Engineering, Lausanne, Switzerland (dimitrios.lignos@epfl.ch)

diaphragms, as well as the contribution of the non-structural elements is clearly reflected in the current design provisions in North America [13,14,15], where a comprehensive design process for the CFS framed diaphragm component is not available in Canada, while limited information is provided for the USA. Further, the non-structural components’ contribution to the lateral load carrying system is not explicitly included in any of the existing seismic design methods.

At the core of the research work discussed herein is a major research program realized at Johns Hopkins University entitled “Enabling Performance-Based Seismic Design of Multi-Story Cold-Formed Steel Structures”. It involved a two-story CFS framed / wood sheathed shear wall building (CFS-NEES Building) tested under earthquake loading using the Network for Earthquake Engineering Simulation (NEES) equipment site (shake table) at the State University of New York (SUNY) at Buffalo in the USA [1]. Studies for the shear walls and gravity walls of the building were carried out at Johns Hopkins and Virginia Tech University [9,16,17]. An effort to characterize and simulate the behavior of CFS beam and column members was also made by Padilla-Llano et al. [3], who produced analytical expressions describing the nonlinear response of CFS members based on the local, distortional and global slenderness. Chatterjee [4] created a 3D ABAQUS model of the floor subassembly of the CFS-NEES Building, while Florig et al. [5] conducted a full-scale test of the floor subassembly loaded in the z direction (short direction loading). Moreover, an experimental program involving CFS framed diaphragm configurations with oriented strand board sheathing (OSB) was conducted at McGill University [8] using as a basis for the diaphragm specimens the floor and roof configurations of the CFS-NEES Building. The numerical work presented herein is a continuation of the experimental work conducted at McGill University [8] for the CFS framed diaphragm subassemblies. The CFS – NEES Building is modeled in the OpenSees platform [18], based on the work of Leng [2], with the purpose of investigating the influence of the diaphragm stiffness on the overall seismic response. A simplified approach of simulating this structure is investigated, where only half of the gravity framing is included, in an effort to generalize and facilitate the numerical simulation of CFS structures. Non-structural interior gypsum and gravity walls were also included in the model. Three stiffness conditions were considered for the floor and roof of the structure; flexible, semi-rigid and rigid, based on the diaphragm flexibility definitions provided in ASCE 7 Standard [19].

### Numerical Simulation

The CFS-NEES Building was a 7.01 × 15.16m two-story structure with 2.74m story height and ledger framing construction for the floor-to-wall connections. Basic sections used in the structure were: 600s162-54/33 floor/roof gravity studs, double 600s162-54 shear wall chord members, 1200s200-54 roof joists, 1200t200-68 roof rim joists, 1200s250-97 floor joists, 1200t200-97 floor rim joists and 600t150-54 horizontal members [15]. A 3D model is illustrated in Fig. 1.

### Shear walls

Shear walls are the primary lateral force resisting components of a CFS structure; they typically consist of CFS framing, OSB sheathing and hold-down connections. The shear walls in the CFS-NEES Building were 2.74m high with widths that ranged from 1.24m to 3.82m. Shamim et al. [12] used a pair of pin-connected truss elements with Pinching4 uniaxial material [20], representing one OSB panel, in order to simulate the response of a shear wall under lateral loading. Leng [2] built on the work of Shamim et al. by explicitly modeling the hold-down/chord
stud connection and introducing the subpanel approach in order for a better transfer of force to be enabled between the shear wall and the rest of the gravity framing of the structure. In the model used herein, the shear wall was simulated using four subpanels, as shown in Fig. 2, to facilitate the interaction of the shear walls with the two horizontal gravity studs along the length of the wall line. Using one whole panel or four subpanels can yield a similar lateral response for the shear wall; thus, validating the subpanel approach.

![Subpanel Approach Diagram](image)

The shear wall Pinching4 material [20] was based on the shear wall tests by Liu et al. [9], specimens 4 & 14 (Fig. 1f), and is described in detail by Leng. The hold-down/chord stud connection simulation involved the use of two parallel zero-length springs [2]. One spring represented the tensile strength of the hold-down using Pinching4 material and the tensile...
properties as provided by the fabricator ($F_y = 43.5 \text{kN}, F_u = 66.75 \text{kN}, \delta_y = 5.94 \text{mm}, \delta_u = 10.51 \text{mm}$ for a Simpson S/HDU6 hold-down [21]). A uniaxial elastic perfectly plastic gap material with infinite compressive stiffness was used for the second spring, which represented a pin connection of the chord stud to the bottom track. The two materials are shown in Fig. 1g. The chord studs of the shear walls were simulated using nonlinear beam-column elements (Fig. 1a). A section aggregator was used to combine the uniaxial material properties defining the nonlinear response of the CFS member under axial load ($P_c$, $P_t$), bending moment ($M_y/M_z$) and torque ($T$) (Fig. 1). The only information available at present for the nonlinear response of CFS beam-column members is provided by Padilla-Llano et al. [3]. Chord studs were assumed to be braced against global buckling modes. The analytical expressions (Tables 7.3, 7.7, 7.8 and 7.9 in [3]) were used for the calculation of backbone points and pinching parameters of the Pinching4 model based on the local buckling slenderness of a CFS member under uniform compressive loading (axial strength). An elastic perfectly plastic material was used to define the flexural strength, and an elastic material was included to define the torsional stiffness of the chord studs. The direct strength method [13,22] was relied on to calculate the local and distortional buckling slenderness for the double 600s162-54 sections and to obtain the moment capacity in the strong and weak axis. The resulting member capacities are shown in Table 1. It should be noted that fixed connections were assumed throughout as a more practical and computationally efficient approach for the 3D model of a CFS structure.

Table 1. Gravity framing and shear wall chord studs; member capacities [3].

<table>
<thead>
<tr>
<th>Member Capacities</th>
<th>Double 600s162-54</th>
<th>Double 600s162-33</th>
<th>1200t200-97</th>
<th>1200s250-97</th>
<th>1200t200-68</th>
<th>1200s200-54</th>
<th>600t150-54</th>
</tr>
</thead>
<tbody>
<tr>
<td>$P^+$ (kN)</td>
<td>293.55</td>
<td>113.33</td>
<td>363.86</td>
<td>461.76</td>
<td>294.48</td>
<td>248.48</td>
<td>132.96</td>
</tr>
<tr>
<td>$P^-$ (kN)</td>
<td>-133.34</td>
<td>-42.10</td>
<td>-135.15</td>
<td>-190.66</td>
<td>-82.24</td>
<td>-65.46</td>
<td>-51.72</td>
</tr>
<tr>
<td>$M_y$ (kNm)</td>
<td>8.2</td>
<td>3.1</td>
<td>15.0</td>
<td>28.1</td>
<td>11.7</td>
<td>10.4</td>
<td>3.4</td>
</tr>
<tr>
<td>$M_z$ (kNm)</td>
<td>±1.82</td>
<td>±0.7</td>
<td>1.2/-1.3</td>
<td>2.6/-3.2</td>
<td>0.8/-1.2</td>
<td>1.0/-1.9</td>
<td>±0.4</td>
</tr>
</tbody>
</table>

Gravity framing

The same modeling approach as used for the shear wall chord studs was followed for the gravity framing members (nonlinear beam-column elements). The material properties can also be seen in Table 1. Testing of the CFS-NEES Building revealed that the gravity framing contributes considerably to the lateral stiffness of the structure [1]; thus, it should be included in the numerical simulation of the building apart from the main lateral force resisting elements (shear walls and diaphragms). In order to simplify the simulation of the gravity framing only half of the gravity studs were included and their sections were doubled, accordingly. Two horizontal gravity members were also included to ensure interaction between the shear walls and gravity studs. The connection of the gravity studs to the bottom track of the structure was simulated using a zero-length spring. The connection cannot resist uplift forces [2]; thus, a uniaxial elastic multilinear material was used with negligible tensile stiffness and infinite compressive stiffness (Fig. 1a).
Diaphragm

The floor and roof subassemblies of the CFS-NEES Building were simulated using a mesh of two-node-link pairs with Pinching4 material properties, as shown in Fig. 1. Initially, a 3.51m × 6.1m 2D model (Fig. 3) with one pair of two-node-links was created, for which the material parameters were calibrated using the data obtained from the experimental diaphragm program, as described in Nikolaidou et al. [8]. The experimental shear strength and stiffness values obtained for the roof and floor diaphragm configurations are 5.6 kN/m, 1.5 kN/mm and 7.9 kN/m, 3.2 kN/mm, respectively. A 1.53m × 1.75m mesh element, product of the initial 2D model, was used in order to construct the 7.01m × 15.16m diaphragms of the structure (Fig. 1b).

![Diaphragm Specimen 2D Model Cantilever Test Approach](image1)

Figure 3. Diaphragm simulation process.

A 2D model of half of the CFS-NEES Building floor and roof subassemblies was created (Fig. 3) to obtain their shear strength and stiffness for the two directions of loading (x and z). The resulting shear strength vs deformation response of the 2D diaphragm model was compared with the work of Chatterjee [4] and Florig et al. [5], as described in the “Introduction” section. From an initial comparison with their results it was shown that the shear stiffness obtained from the 2D model (Fig. 3) was lower than that obtained by the full-scale test and the ABAQUS model. This was attributed to the fact that both Chatterjee and Florig et al. included the ledger framing connection, which was not part of the test setup during the diaphragm experimental program at McGill University [8]. In addition, Chatterjee proposed two limits for the shear stiffness of the diaphragm; an upper limit where perfect friction is assumed between the wood panels and a lower limit when no friction is included [4]. In order to include the influence of the walls confining the diaphragm and increasing its resistance, the initial stiffness of the diaphragm mesh element was multiplied with a confinement factor (CF) of 1.5, which resulted in a match of the lower limit that Chatterjee proposed, as illustrated in Fig. 4a and b. This is a conservative CF value. Additional research is necessary to better define the effect of the wall’s presence on the diaphragm nonlinear response.

![2D OpenSees floor model compared to ABAQUS model and full scale test](image2)

Figure 4. 2D OpenSees floor model compared to ABAQUS model and full scale test; a) long direction loading (x direction), and b) short direction loading (z direction).
Non-structural gypsum sheathing and gravity walls

Non-structural interior gypsum and exterior OSB panels were installed throughout the wall line of the structure. To include non-structural gypsum for shear walls in the model, tests 3 and 13 by Liu et al. [9] (OSB + Gypsum + Hold-down) were relied upon for the material properties of the shear walls. The gravity walls (OSB + Gypsum, no Hold-down) followed the same simulation approach as the shear walls. There are no experimental data available at present for CFS framed/wood sheathed gravity walls of the type used in the CFS-NEES Building. The Pinching4 material for the truss elements of the gravity walls was calculated using the work of Leng [2]. Leng provided the lateral response of two 2.44m x 2.74m gravity walls, one with OSB and one with gypsum sheathing, using a fastener-based model as described by Bian et al. [17] and Buonopane et al. [16]. Work by Chen et al. [23], on wood framed walls, showed that summing the lateral response of a gypsum-sheathed and a separate OSB-sheathed shear wall yields the lateral response of a shear wall constructed with both OSB and gypsum. Experimental data from tests 12, 16 and 13 by Liu et al. [9] were used to verify this concept (Fig. 5a). A 2D shear wall model with two pairs of coincident truss elements was created and evaluated under reversed cyclic loading. One pair incorporated the response for Test 12 (shear wall with OSB) and the other pair for Test 16 (shear wall with gypsum). The resulting lateral response from this model (two panels) was compared to a one pair of truss elements shear wall model (one panel) based on Test 13 (shear wall with OSB and gypsum). In Fig. 5a it is shown that the lateral response of Test 12+16 slightly overestimates the lateral response of Test 13. In general, the two V-δ (shear strength – displacement) curves in Fig. 5a are in good agreement; thus, verifying the finding of Chen et al. A lateral response for both OSB and gypsum was calculated and incorporated in one pair of truss elements for each gravity wall; the subpanel approach was used for the various gravity wall sizes. Fig. 5b shows how well the calculated V-δ curve for one pair of truss elements matched the model with two pairs of truss elements representing each of the lateral response curves for OSB and gypsum as provided by Leng [2].

Figure 5. Non-structural components; a) shear walls and b) gravity walls

Results

The CFS-NEES Building was initially modeled only with structural components (Phase 1). Subsequently, non-structural components were added (Phase 2c) and three diaphragm conditions were considered in order to investigate the influence of the diaphragm flexibility on the seismic response of the structure.

Phase 1 – Model verification

The Canoga Park (CNP) ground motion (100%, design level earthquake in the USA) was applied
for the Phase 1 testing of the structure. The actual shake table signal was inserted as uniform excitation loading in the x, z (horizontal directions) and y direction (vertical direction). Table 2 includes a comparison of the fundamental period in the two directions, $T_x$ and $T_z$ and of wall line inter-story drift ratios in the x (u) and z (v) direction for floor (1) and roof (2) level. Table 3 includes the base shear, the maximum floor diaphragm deflection in the z direction (MDD$_z$) and the maximum total displacement (shear walls and diaphragm) experienced by the floor of the structure. Figures 6a and b show a comparison of the wall line inter-story drift ratios in the z direction at the floor level. From the information provided in Tables 2 and 3 it is shown that the simplified model, although more flexible, can adequately predict the lateral response of the structure. It should be noted that in Table 3 the base shear in the two directions is not the total force experienced by the structure; only the shear wall force is presented for the purpose of comparison. Gravity studs resist 40% of the lateral load.

### Table 2. Inter-story drift ratios and fundamental periods, Phase 1; Comparison

<table>
<thead>
<tr>
<th>Phase 1$x,y,z$</th>
<th>$\Delta u_1/h$ (%)</th>
<th>$\Delta u_2/h$ (%)</th>
<th>$\Delta v_1/h$ (%)</th>
<th>$\Delta v_2/h$ (%)</th>
<th>$T_x$</th>
<th>$T_z$</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Model</strong></td>
<td>1.54</td>
<td>1.22</td>
<td>0.75</td>
<td>0.51</td>
<td>0.34</td>
<td>0.39</td>
</tr>
<tr>
<td><strong>Test</strong></td>
<td>1.18</td>
<td>0.81</td>
<td>0.85</td>
<td>0.56</td>
<td>0.31</td>
<td>0.36</td>
</tr>
</tbody>
</table>

*Note: The drift was calculated based on the average displacement of two corner nodes.

### Table 3. Base shear, MDD$_z$ and maximum floor displacement Phase 1; Comparison

<table>
<thead>
<tr>
<th>Phase 1$x,y,z$</th>
<th>$V_x$ (kN) $^*$1</th>
<th>$V_x^{\text{tot}}$ (kN)</th>
<th>$V_z$ (kN) $^*$1</th>
<th>$V_z^{\text{tot}}$ (kN)</th>
<th>Max Displacement (mm)</th>
<th>MDD$_z$ (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Model</strong></td>
<td>96.3</td>
<td>154.1</td>
<td>65.9</td>
<td>109.6</td>
<td>28.2</td>
<td>7.5</td>
</tr>
<tr>
<td><strong>Test</strong></td>
<td>84.4$^*$2</td>
<td>-</td>
<td>79.1$^*$2</td>
<td>-</td>
<td>28</td>
<td>3.5</td>
</tr>
</tbody>
</table>

*Note: 1. Base shear values based only on the shear wall forces.  

![Figure 6](image1.png)  
**Figure 6.** Inter-story drift ratios, $z$ direction; a) floor level and, b) roof level.

**Phase 2c – Diaphragm flexibility**

The next step was to add the non-structural interior gypsum and gravity walls, as was done for
Phase 2c of testing (no interior walls at this phase). It should be noted that the x and z component of the 100% CNP ground motions were considered in the response history analysis, not 44% CNP as in the experiment. The purpose is not to compare with the experimental data in this case but to explore the influence of the diaphragm flexibility with the addition of non-structural components. Gypsum panels increased the shear wall stiffness by an average of 20-30%, while the gravity walls contributed in the lateral resistance of the structure resulting in smaller shear wall forces by 26% in the x direction and 21% in the z direction. The base shear was increased by 82% and 17% in the x and z direction, respectively. In the x direction, 75% of the forces were taken by the gravity walls while 25% by the shear walls. In the z direction, 62% of forces were taken by the gravity walls while 38% by the shear walls; it is shown that the gravity walls attain a primary role as lateral force resisting elements. The fundamental period of vibration values in the two directions were decreased as expected to $T_x = 0.21\text{sec}$ and $T_z = 0.31\text{sec}$ (equivalent decrease was recorded during the experiment for Phase 2c). Accordingly, wall line inter-story drift ratios were reduced on average by 65% in the x and by 50% in the z direction. The diaphragm stiffness for floor and roof subassemblies in the z direction was 3.5 kN/mm and 1.7 kN/mm, respectively. A factor of 0.1 was, subsequently, applied, which resulted in a flexible floor diaphragm in the z direction as per the ASCE 7 definition ($\text{MDD}_z \geq 2^*\text{Average drift of walls}$) [19] and a factor of 100 for the case of rigid diaphragm ($\text{MDD}_z \approx 0\text{mm}$). It should be noted that in order to investigate the influence of diaphragm flexibility, an elastic material with stiffness, $k$, was initially considered for the two-node link elements. In Tables 4 and 5, a comparison of the three diaphragm conditions is demonstrated in the form of fundamental periods, wall line inter-story drift ratios, MDD$_z$ values, maximum overall floor displacement values and base shear forces.

Table 4. Phase 2c, Fundamental period; flexible vs. semi-rigid vs. rigid diaphragm.

<table>
<thead>
<tr>
<th>Phase 2c, $x$, $z$</th>
<th>$T_x$ (sec)</th>
<th>$T_z$ (sec)</th>
<th>$\Delta_u/h$ (%)</th>
<th>Floor</th>
<th>$\Delta_u/h$ (%)</th>
<th>Roof</th>
<th>$\Delta_u/h$ (%)</th>
<th>Floor</th>
<th>$\Delta_u/h$ (%)</th>
<th>Roof</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexible</td>
<td>0.29</td>
<td>0.57</td>
<td>0.72</td>
<td>0.39</td>
<td>0.40</td>
<td>0.24</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Semi-rigid</td>
<td>0.21</td>
<td>0.31</td>
<td>0.69</td>
<td>0.35</td>
<td>0.41</td>
<td>0.26</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rigid</td>
<td>0.20</td>
<td>0.23</td>
<td>0.67</td>
<td>0.33</td>
<td>0.49</td>
<td>0.44</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 5. Phase 2c, MDD, max floor displacement values and base shear; flexible vs. semi-rigid vs. rigid diaphragm.

<table>
<thead>
<tr>
<th>Phase 2c, $x$, $z$</th>
<th>MDD$_z$ (mm)</th>
<th>Max Displacement (mm)</th>
<th>$V_x$ (kN)</th>
<th>$V_z$ (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Flexible, $x$</td>
<td>38.7</td>
<td>48</td>
<td>249</td>
<td>136.2</td>
</tr>
<tr>
<td>Semi-rigid</td>
<td>8.2</td>
<td>19.5</td>
<td>262.8</td>
<td>127.8</td>
</tr>
<tr>
<td>Rigid</td>
<td>-</td>
<td>14.6</td>
<td>260.5</td>
<td>129.3</td>
</tr>
</tbody>
</table>

In Tables 4 and 5, for the flexible diaphragm case, there is an 84% increase of the fundamental period of vibration $T_z$, which translates to over a 100% increase of the MDD$_z$ and the total displacement of the floor subassembly. In addition, the second mode shape of the structure changed to the second natural mode of the z direction (Fig. 7). For the rigid diaphragm case, the fundamental period, $T_z$ decreased by 26%; the roof level with a previously flexible diaphragm system was influenced more by the added rigidity, where the diaphragm deformations were
minimized and the walls’ story drifts increased by 69%. Base shear forces changed up to 6% in both directions. Overall, it was observed that there is no specific trend followed in the seismic response of the structure; i.e. for the flexible case, the increase of the fundamental period did not lead to a decrease of the forces and an increase of the drifts for both directions. This suggests that it is the input ground motion that plays the determining role in the resulting seismic response.

Figure 7. Second mode shape; semi-rigid vs. flexible diaphragm

**Conclusions**

In the work presented herein, the CFS-NEES Building was modelled in the OpenSees platform following a simplified modeling approach with explicit simulation of the floor and roof diaphragm subassemblies. The 3D building model was verified for Phase 1 of CFS-NEES Building testing and non-structural interior gypsum sheathing and exterior OSB was added along the wall lines of the structure. Three diaphragm conditions were considered, flexible, semi-rigid and rigid, and results were compared in the form of fundamental periods, base shear forces, wall line inter-story drift ratios, maximum diaphragm deflection and total maximum displacement values. It was shown that the simplified 3D model is able to predict adequately the lateral response of the structure. The diaphragm flexibility influenced primarily the fundamental period, mode shapes and diaphragm deflection; for the cases discussed, results indicate that the seismic response becomes sensitive to the input ground motion characteristics. This work is a first step in understanding the diaphragm influence. Further study is in progress considering ground motions suitable for a design level earthquake in Canada and other building structural characteristics.

**Acknowledgments**

The authors would like to thank Professor Benjamin W. Schafer and Dr. Cristopher D. Moen, as well as Dr. Kara Peterman, Dr. Aritra Chatterjee, Dr. Jiazhen Leng and Dr. David A. Padilla-Liano for the information and data they provided related to the CFS-NEES Building. The authors are also grateful to the Natural Sciences and Engineering Research Council of Canada (NSERC), as well as, the American Iron and Steel Institute (AISI) and the Canadian Sheet Steel Building Institute (CSSBI) for financially supporting this research project.

**References**


11. Pan, C. L. and Shan, M. Y. Monotonic shear tests of cold-formed steel wall frames with sheathing. Thin-Walled Structures 2011; 49(2), 363-370.


19. ASCE 7. Minimum design loads for buildings and other structures. American Society of Civil Engineers 2016, Reston, VA.


