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Assessment of In-Plane Rigidity Adequacy of Light Timber Frame Diaphragms for Implementation in Isolated Part 9 Buildings

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ABSTRACT

In Canada, single-family wood frame residential structures located in areas prone to earthquakes are typically built using non-engineering methods, following the guidelines set forth in Part 9 of the National Building Code or equivalent provincial standards. While these structures are expected to provide adequate life safety during strong ground motions, they are susceptible to significant economic losses. Seismic isolation is an innovative method that offers superior life safety and economic viability. The in-plane stiffness of the diaphragm situated above the isolation interface significantly affects the seismic response of isolated structures. The objective of this research is to evaluate the in-plane rigidity of plywood-sheathed floor diaphragms and assess their components for use in isolated Part 9 structures. To achieve this goal, an equivalent simplified finite element model of the diaphragm was created using SAP2000. The impact of the nailing schedule and sheathing panel arrangement on the diaphragm behaviour was taken into account by applying a shear stiffness modifier, which eliminated the need to represent each panel and nail connection individually. The validity of the equivalent model was confirmed, using the results of prior full-scale experimental studies. To investigate the adequacy of the diaphragm under seismic loads, a three-dimensional numerical model of a one-story, base-isolated wood frame structure was developed and subjected to dynamic time-history analysis. The results of this study indicate that the in-plane rigidity of the blocked edge wood diaphragm, the shear resistance of the plywood sheathing, and the axial resistance of the framing members are sufficient to withstand seismic forces. Furthermore, it was found that for typical wooden diaphragms to be used above a seismic isolation layer, strengthening of the rim joists is necessary to provide sufficient gravity load carrying capacity and transfer to the bearings.

Keywords: isolated Part 9 structures, wood diaphragm, in-plane rigidity, wood building modeling, dynamic time-history analysis.

INTRODUCTION

In Canada, the majority of single-family residential buildings are constructed using light timber frames (LTF) and designed in accordance with Part 9 of Division B of the National Building Code of Canada (NBCC) (henceforth referred to as Part 9 structures for brevity). Part 9 stipulates that no engineering involvement is required and is only applicable to relatively small and simple structures that are no more than three storeys high and have a total plan area of 600 m^2 or less [1]. Although Part 9 buildings are generally resilient in terms of life safety, they are often unable to prevent non-structural components and contents from experiencing significant damage during moderate to strong earthquakes. Non-structural components and contents make up a significant portion, approximately 60-80%, of the building's value [2]. Even superficial damage can result in considerable losses, and the cost of repairs can be a significant financial burden for homeowners. In recent years, the performance of wood frame residential structures has been criticized due to the significant economic losses resulting from several seismic events that occurred near urban centers [3-4]. These losses, in addition to the loss of property and the potential need to seek temporary accommodations while repairs are underway, can cause significant disruption to affected individuals' lives. Therefore, there is a need for methods to improve the seismic resilience of these structures.

Seismic base isolation is an effective method available in the NBCC for achieving excellent life safety performance while safeguarding the structure and its contents from damage. This emerging technology can minimize the seismic response of the superstructure by decoupling it from strong ground motions [5]. The decoupling process involves creating a horizontally flexible layer, commonly known as the isolation layer, between the foundation and superstructure. This layer permits the base of the structure to displace significantly laterally during an earthquake, which favourably alters the dynamic response and allows the superstructure to remain elastic. Figure 1 illustrates the idealized fixed base and isolated deformed shapes of structures during seismic excitation. As a result of this significant reduction in response, isolated structures and their contents can withstand even the most substantial earthquakes without damage. Therefore, seismic isolation is a promising approach for safeguarding economically vulnerable Part 9 structures [6-7].

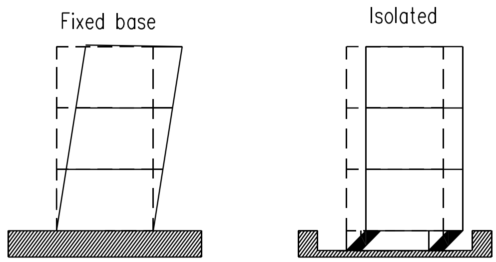


Figure 1. Idealized deformed shapes of fixed base and base isolated structures due to strong ground motions.

Irrespective of the construction material used, diaphragms play a crucial role in the structural behaviour of a building. Not only do they act as slabs under gravity loads, but they also connect all other structural elements and transfer horizontal loads to the vertical lateral load resisting system (LLRS). Since diaphragms are the first element to resist most gravity and horizontal forces, a loss of their action can potentially compromise the behaviour of the entire structure. In recent years, there has been increased research on the behaviour of diaphragms and their impact on the response of the entire building during dynamic loading scenarios [8]. In isolated structures, the diaphragm situated above the isolation layer holds significant importance due to two primary reasons. Firstly, it serves as a crucial system for transmitting both the vertical and lateral loads from the superstructure to the isolation system, thus ensuring structural stability. Secondly, the diaphragm must exhibit sufficient in-plane rigidity to ensure uniform motion across all bearings during seismic events. Therefore, maintaining the integrity of the isolated diaphragm and its components is critical in designing earthquake-resistant isolated structures. Symans et al. [9] stated that due to the typically insufficient in-plane stiffness of floor diaphragms, the process of installing base isolation systems in wood frame buildings can present difficulties.

To ensure structural integrity, it is necessary to transfer any forces generated within or applied to the edges of a floor diaphragm to the foundation. This requires all components of the diaphragm, including the chord beams, joists, panel elements, and connections, to withstand these anticipated forces and provide a clearly defined load path from the load points to the foundation. When subjected to seismic forces, it is essential for the LLRS to exhibit energy dissipation and ductility properties that allow the diaphragms to effectively function as a unified system within the elastic range. Additionally, the incorporation of an isolation system can further augment energy dissipation, further promoting the unified performance of the diaphragms within the elastic range [8].

This paper presents an assessment of the performance of LTF diaphragms for application in isolated Part 9 buildings through numerical simulation. The study proposes shear stiffness modification factors that incorporate the impact of sheathing panel edge nailing connections. Using these factors, a finite element model of a one-story base isolated LTF building is established with the aid of SAP2000 [10]. Nonlinear time-history dynamic analysis is then performed to evaluate the adequacy of the isolated diaphragm. Lastly, the paper assesses the performance of the diaphragm components by comparing the maximum applied stresses/forces with the nominal resistance specified in the building code.

FINITE ELEMENT MODELING

Plywood sheathing stiffness modifier determination

The objective of this study was to develop a practical method for constructing a simplified finite element model of a plywood-sheathed floor diaphragm typically used in single-family residential buildings. The assessment of the diaphragm's ability to withstand shear forces involved comparing the necessary shear demands with the combined capacity provided by the sheathing, framing, and fastening elements. The diaphragm's shear capacity is influenced by various factors, including the construction type (blocked or unblocked), framing species, framing member size, sheathing grade and thickness, panel arrangement, and nailing type, size, and spacing.

LTF diaphragms consist of various elements such as chord beams, joists, blocking, panel components, and connections. In order to perform an accurate simulation of a building's diaphragm, it is necessary to conduct detailed modeling of all these elements, which includes accounting for variables such as nail type and size, panel arrangement, and boundary nailing patterns. However, this process can become computationally intensive due to the high number of degrees of freedom involved [11]. Previous research has modelled individual fasteners using "zero length link elements" to represent each nail [12-14], but this method proves to be inefficient for complex 3D structures [15]. An alternative method has been proposed by some researchers, employing the equivalent truss method instead of the diaphragm and its constituents [11,16]. However, this method lacks the ability to verify the sufficiency of individual components, such as joists, blocking, and sheathing. Therefore, a simplified model that balances simulation accuracy with computational efficiency has been developed.

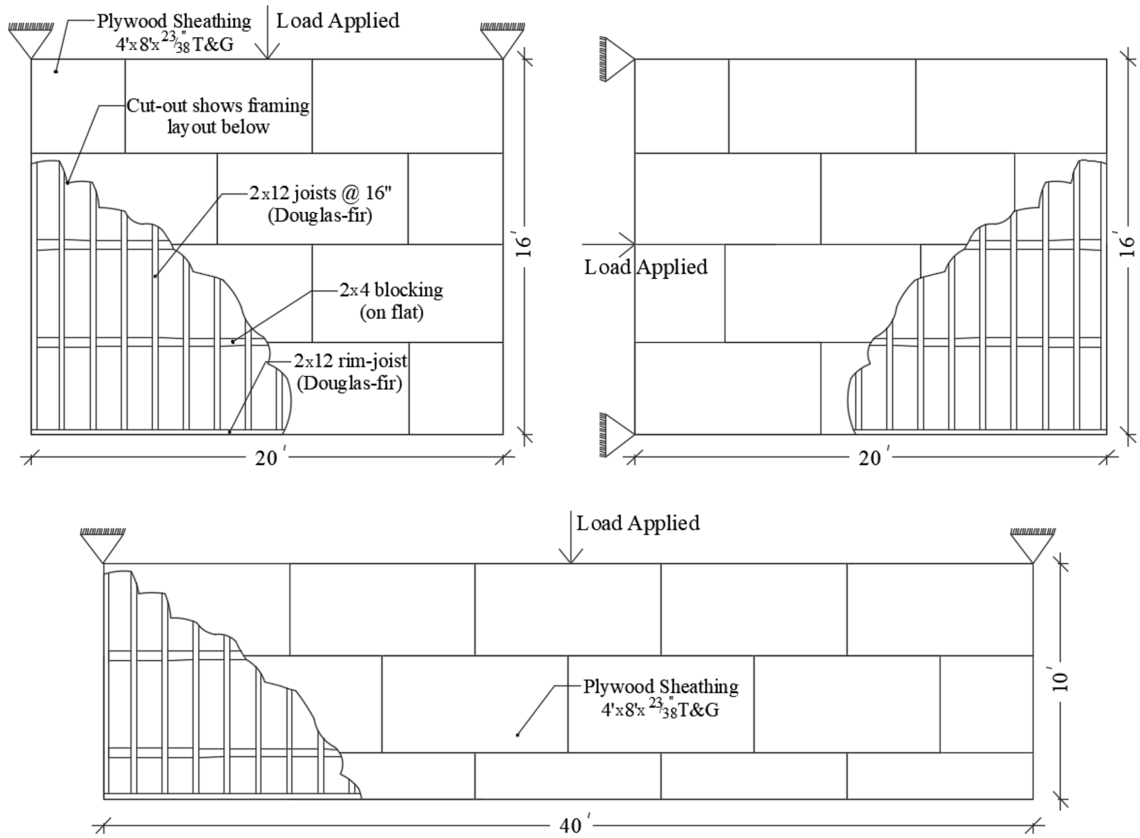


Figure 2. The three diaphragm configurations (without openings) selected for numerical modeling (adapted from [17]).

The blocked edge wood diaphragm in SAP2000 was modeled using chord beams, joists, and blocking, and instead of separate panels and nail connections, a homogenous plane element was used with shear stiffness modification coefficients. This approach assumed that internal forces were transferred continuously across the joints between actual sheathing panels measuring 2.43×1.21 m (4 x 8 ft). This assumption was valid due to the presence of joists and blocking along all panel edges and high nailing density [15]. Martin [18] provides more information on this topic. The connections between framing members and sheathing panels, which are widely used in light-frame wood construction, are known to have a significant impact on the structure's behaviour. In addition to dissipating energy during seismic loads, these connections have a significant impact on the diaphragms' strength and stiffness [19].

The shear stiffness modification coefficients for the considered LTF diaphragms were determined using full-scale experimental results from Bott [17]. The specimens were framed using Douglas-fir 38×286 mm (nominal 2×12 in) joists with 406 mm (16 in) spacing and sheathed with 2.43×1.21 m (4 x 8 ft) sheets of 18 mm (nominal 23/32 in) tongue-and-groove plywood arranged in a staggered panel configuration. The connections between framing members and sheathing panels were made using 10d common nails spaced in a 152/305 mm (6/12 in) pattern, meaning nails are spaced at 152 mm (6 in) around the perimeter and at 305 mm (12 in) on the interior supports of each sheathing panel. To provide support where each line of unsupported sheathing panel joints was located, 38×89 mm (nominal 2x4 in) blocking was laid flat and installed between the joists. The blocks were fastened on each side of the joists using two 16d common toe-nails, and the plywood panel edges that overlapped the blocks were nailed every 152 mm (6 in) with 10d duplex nails [17].

To determine the stiffness modification coefficients, the seven blocked edge diaphragm configurations were modelled in SAP2000. The first three configurations were diaphragms with dimensions of 4.88×6.10 m (16×20 ft), loaded parallel to the direction of the joists on the 6.10 m (20 ft) side. The first configuration had no sheathing opening, the second had a corner sheathing opening with dimensions of 1.22×2.44 m (4×8 ft), and the third had a center sheathing opening with dimensions of 2.44×3.66 m (8×12 ft). The fourth to sixth configurations were identical in dimensions and specifications to the first three specimens but were loaded perpendicular to the direction of the joists on the 4.88 m (16 ft) side. Finally, the seventh specimen had dimensions of 3.05×12.2 m (10×40 ft) and was loaded parallel to the direction of the joists on the 12.2 m (40 ft) side. This configuration was chosen to examine the effect of the aspect ratio of the diaphragm on the shear stiffness modification coefficient. Experimental details for these seven selected tests are provided in Bott [17]. Figure 2 shows the three main diaphragm configurations without any openings.

Finite element model of experimentally tested diaphragms

SAP2000 was utilized to simulate the seven selected configurations of diaphragms, along with their respective loading, boundary conditions, and sheathing openings (if any). The study is primarily concerned with the in-plane behaviour of LTF diaphragms; therefore, all out-of-plane degrees of freedom in the models were restricted. In the experiments, steel frames were placed on the two edges of the diaphragms parallel to the direction of loading. To replicate this boundary condition in the simulations, both sides of the diaphragm models were constrained in terms of translational degrees of freedom. Specifically, the diaphragm models were constrained such that displacements along the two edges parallel to the loading direction were fixed in both the transverse and longitudinal directions, while allowing rotations within the diaphragm plane [11].

A modulus of elasticity of $E = 11{,}000$ MPa has been considered for Douglas-fir type structural joists and framing elements [20]. A plywood shear modulus of $G = 600$ MPa has been used for plywood sheathing, according to Filiatrault [21]. In the numerical simulation, all diaphragm framing members, including joists and blocking were modeled with their actual sections. The plywood sheathing was modeled as a single integrated plane using SAP2000's thin shell element along with a shear stiffness modification factor. SAP2000 offers the capability to partition the modeling shell element into several analysis shell elements via meshing. This process allows for enhanced user control over the definition of the mesh and the ability to review the resultant diaphragm shell element model. Then, for a given peak load value, the diaphragm deflections obtained in experiments (from Bott [17]) were compared with those predicted by the SAP2000 model. The goal was to match the finite element model to each of the experimental deflections by iteratively changing the shear stiffness modifier, f_{12}, in SAP2000. Once a value of f_{12} was found to give almost the same deflection, the updating process was complete for that diaphragm configuration. The modification coefficients were extracted with two decimal places of precision.

The results from the seven software simulations are presented in Table 1, along with the estimated shear stiffness modification coefficients for diaphragms with edge nail spacings of 152 mm (6 in) for perimeter edges and 305 mm (12 in) for interior edges. These coefficients were determined based on the presence of an opening in the plywood sheathing and the loading direction. The modification factor values range from 0.07 to 0.15, providing insight into the varying levels of stiffness modification observed in the diaphragms. For simulations with the direction of loading along the joist, the modification factors range from 0.11 to 0.15, while for loading perpendicular to the joists, the modification factors range from 0.07 to 0.10. Furthermore, changing the aspect ratio of the diaphragm from 1.25 to 4 resulted in a decrease in the modification factors from 0.15 to 0.07 when loaded in the direction of the joists.

Table 1. Diaphragm configuration and the obtained values for shear stiffness modification coefficients.

No.	Diaphragm Configuration	Loading Dir. Relative to Joists Dir.	Experimental Results [17]			Simulated Model	Shear Stiff. Mod. Coeff.
			Peak Load kN	Cyclic Stiff. kN/mm	Deformation mm	Deformation mm	f_{12}
1	4.88×6.10 m (16×20 ft) No opening	parallel	55.5	11.6	4.79	4.93	0.15
2	4.88×6.10 m (16×20 ft) Corner sheathing opening	parallel	44.7	7.79	5.73	5.71	0.12
3	4.88×6.10 m (16×20 ft) Center sheathing opening	parallel	34.9	6.04	5.79	5.79	0.11
4	4.88×6.10 m (16×20 ft) No opening	perpendicular	49.7	11.4	4.37	4.35	0.10
5	4.88×6.10 m (16×20 ft) Corner sheathing opening	perpendicular	39.3	8.70	4.51	4.52	0.09
6	3.05×12.2 m (10×40 ft) Center sheathing opening	perpendicular	28.4	6.18	4.59	4.47	0.07
7	40×10 ft (6.0×4.8 m) No opening	parallel	32.5	1.65	19.7	19.9	0.07

* In all configurations, the joists are placed in the direction of the smaller diaphragm dimension.

The results show that the presence of an opening in the plywood sheathing reduces the shear stiffness of the diaphragm in both primary directions, resulting in lower values of the shear stiffness modification coefficients. Additionally, the modification coefficients that were extracted can be used with relatively good accuracy for diaphragms that have similar components and aspect ratios within the investigated range. The sensitivity of the in-plane deformation of diaphragms to changes in the shear stiffness modification coefficient was investigated in this study. Specifically, the effects of decreasing or increasing the modification coefficient by 10% and 20% were analyzed for diaphragm configuration 1. It was found that a 10% reduction in the shear stiffness modification coefficient resulted in a 10.8% increase in the maximum deformation of the diaphragm, while a 20% reduction led to a 24. 1% increase. Conversely, an increase of 10% in the coefficient caused a decrease of 8.72% in the maximum deformation, while an increase of 20% resulted in a decrease of 16.1%. The findings suggest that variations in the shear stiffness modification coefficient have a substantial effect on the in-plane stiffness and deformation of diaphragms. Consequently, the precise consideration of this parameter is deemed essential in diaphragm analysis. Figure 3 illustrates the deformed shape of the seven diaphragms simulated using SAP2000.

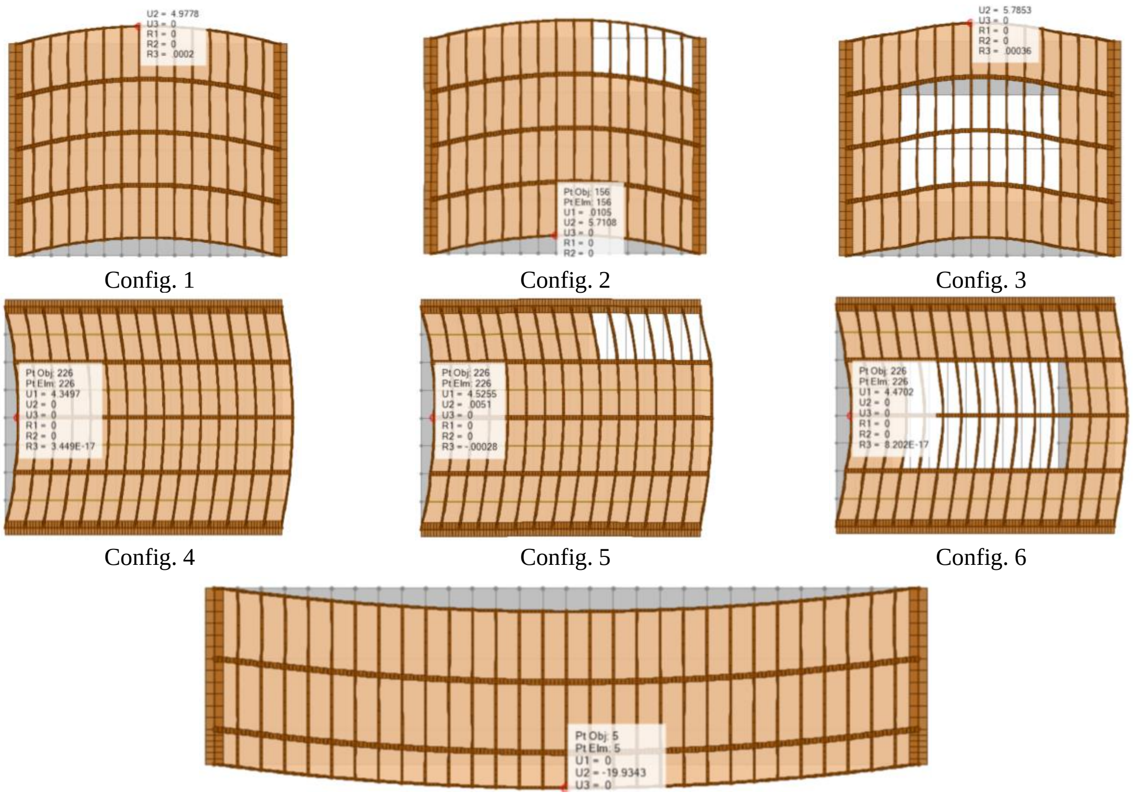


Figure 3. Deformed shape of the finite element model of the seven diaphragms simulated in SAP2000.

Isolated light timber frame building model

To evaluate the suitability of a wood diaphragm in meeting the criteria specified by the NBCC for use as a diaphragm above isolators in isolated Part 9 structures, a finite element model of a single-storey wood frame building has been developed in SAP2000. Nonlinear response history analyses have been conducted to investigate the structural performance of the wood diaphragm.

The building has a height of 3.5 m and a structural plan with dimensions of 5.5 m by 3.6 m, resulting in an aspect ratio of 1.53. The structural components of the walls and roof system were designed based on NBCC [1] and CSA-O86 [20]. The wall framing elements and roof trusses were constructed using 38 x 89 mm (nominal 2 x 4 in) Douglas-fir lumber, with studs and trusses spaced at intervals of 406 mm (16 in). The roof and wall sheathing were modeled using SAP2000's shell element, with the roof sheathing having a thickness of 11.9 mm (nominal 1/2 in) plywood and the wall sheathing having a thickness of 11.2 mm (nominal 7/16 in) oriented strand board (OSB). The diaphragm components, including the joist, blocking, and sheathing, were assumed to be similar to those simulated in the previous section. This consisted of Douglas-fir 38×286 mm (nominal 2×12 in) joists with 406 mm (16 in) spacing, 38×89 mm (nominal 2x4 in) blocking, and 2.43×1.21 m (4 x 8 ft) sheets of 18 mm (nominal 23/32 in) tongue-and-groove plywood sheathing.

The wall framing members and roof truss elements were assumed to have a modulus of elasticity of $E=11{,}000$ MPa. The elastic orthotropic material properties of the wall and roof sheathing in the SAP2000 model were assigned in accordance with Martin et al. [15], and are presented in Table 2. The effect of edge nail spacing is accounted for in the proposed shear modulus (G_{12}) for the wall sheathing. No additional stiffness modifiers were applied to the wall and roof sheathing.

Table 2. Elastic orthotropic material properties for wall and roof sheathing [15].

Item	**Modulus of Elasticity (MPa)**		**Shear Modulus (MPa)**		**Poisson's ratio**
	E_1	**$E_2 = E_3$**	**G_{12}**	**$G_{13} = G_{23}$**	**$\mu_{12} = \mu_{13} = \mu_{23}$**
Wall Sheathing	5102	1586	230*	827	0.08
Roof Sheathing	13100	1999	1034	1034	0.08

* Proposed by Martin et al. [15] for 7/16 in OSB wall sheathing with edge nail spacing of 152 mm (6 in).

The building is located in an area of high seismicity in Vancouver, BC and has a Site Class of C. The building falls under the normal weight class of Part 9, and therefore the dead loads for the floor, roof, and partition wall are considered to be 0.5 kPa, while the exterior wall's dead load is considered to be 0.32 kPa. The live loads for the floor and roof are 1.9 kPa and 1.0 kPa, respectively, while the snow load for Vancouver is 1.2 kPa. The building's isolators were designed as square, fiber-reinforced elastomeric isolators. The floor diaphragm was equipped with four elastomeric isolators, which were placed at each of the four corners. The sizing of these isolators was conducted in accordance with the methodology proposed by Stratton [22]. Specifically, the square isolators were designed with a 200 mm side length and were composed of 10 layers of rubber, each having a thickness of 11 mm. This configuration resulted in a total rubber thickness of 110 mm and an isolator shape factor of 4.55, defined as the ratio of the loaded area to unloaded area of a single layer of elastomer. The shear modulus of the rubber was assumed to be 0.3 MPa. To model these isolators, the rubber isolator link element in SAP2000 was used. The elastomeric bearing isolators have an approximate equivalent linear viscous damping ratio of 10%, whereas the damping ratio for the superstructure is considered to be 2.5%.

The three-dimensional finite element model of the building is presented in Figure 4a. The periods of the first and second modes of the isolated building, which correspond to the primary Y and X axis of the structure, have been determined to be 1.31 and 1.30 s, respectively. As seismic loads applied in the Y-direction have been found to cause greater forces and stresses in diaphragm components, this section aims to evaluate the diaphragm's performance under such loads. The simulation conducted in SAP2000 involved the modeling of all diaphragm framing members, including joists and blocking, with their respective actual sections. To model the plywood sheathing, a single integrated plane was created using SAP2000's thin shell element, and a shear stiffness modification coefficient was applied as elaborated in the previous section. The meshing technique was employed to partition the plywood sheathing into several analysis shell elements. The diaphragm plywood sheathing was considered as an isotropic material with a shear modulus of 600 MPa and a Poisson's ratio of 0.3. To enhance the dynamic analysis accuracy in the joist direction (Y-direction), a shear stiffness modification coefficient (f_{12}) of 0.15 was applied due to the isolated diaphragm's aspect ratio of 1.53 and the absence of any openings.

Selection and scaling of ground motions

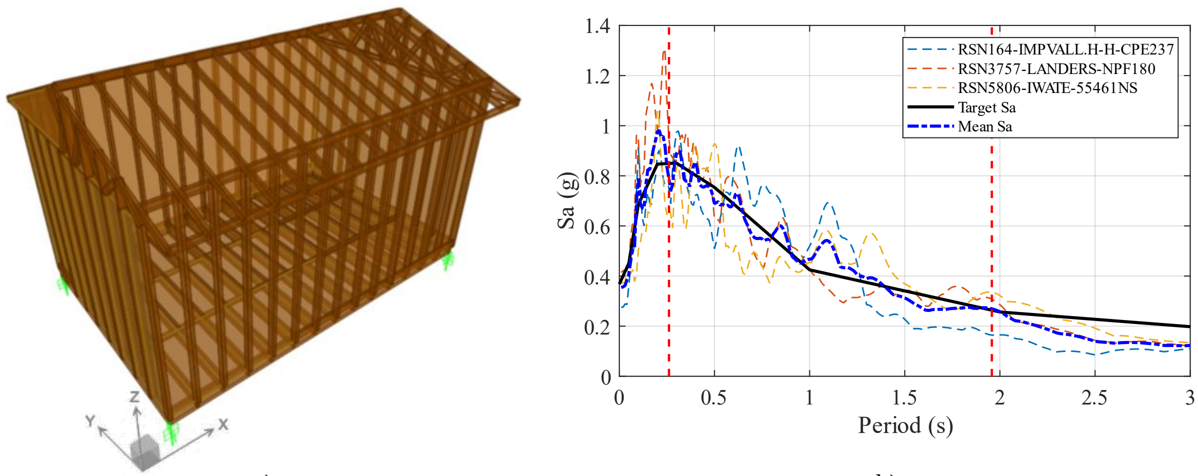
In order to simulate the seismic response of the prototype building, the 5% damped response spectrum for the site were procured from Natural Resources Canada [23] at a hazard level of 2% probability of exceedance in 50 years (2/50). Subsequently, ground motions were selected from the PEER-NGA database [24]. The ground motions were amplitude-scaled to ensure that the mean spectrum for the scaled ground motions did not fall below the target spectrum by more than 10% over the period range from $0.2T_1$ to $1.5T_1$, where T_1 represents the period of the isolated structure. To avoid over-scaling, the scale factors were limited to between 0.5 and 4, in accordance with the commentary of the NBCC. Table 3 provides a summary of the selected ground motions utilized in this study, while Figure 4b illustrates the scaled response spectra and the target spectrum for the hazard level under consideration.

Table 3. Summary of ground motions representing 2/50 hazard level.

Event	**Year**	**Station**	**Magnitude**	**Scaling Factor (2% in 50 years)**
Imperial Valley-06	1979	Cerro Prieto	6.53	1.76
Landers	1992	North Palm Springs Fire Sta #36	7.28	2.99
Iwate, Japan	2008	Yuzawa Town	6.90	1.57

Results and discussions

In order to evaluate the adequacy of the diaphragm components under lateral seismic loads, it is necessary to compare the compressive and tensile forces generated in the joists and the in-plane shear stresses induced in the plywood sheathing, respectively, with the axial capacity of the joists and the minimum of the panel buckling resistance and the shear resistance of the sheathing-to-framing connection. Additionally, a comparison is made between the maximum relative deformation that occurs at the center of the diaphragm and the values that correspond to the predefined damage states. This analysis is crucial to ensure the structural integrity of the diaphragm system and to prevent any potential failure or damage during seismic events.



a) b)

Figure 4. a) 3D SAP2000 model of the isolated building and b) scaled response spectra and the target spectrum for the hazard level of 2/50.

The overall deflection of the diaphragm can be decomposed into three distinct components: flexural deflection arising from the chord beams, shear deformation resulting from the sheathing panels, and slip of the fasteners. Table 4 presents the maximum displacement of the diaphragm and the maximum in-plane shear stress in the plywood sheathing resulting from seismic load in Y-direction. The Landers earthquake record caused a maximum absolute displacement of 69.3 mm in the Y direction at the middle of the diaphragm, while the average displacement of the elastomeric isolators in the same direction was 65.9 mm. This resulted in a maximum in-plane deflection of the diaphragm plane of 3.40 mm.

By defining diaphragm drift as the ratio of the maximum in-plane deformation of the diaphragm to half the dimension of the diaphragm perpendicular to the loading direction, the maximum drift caused by the Landers earthquake record was determined to be 0.12%. Christovasilis et al. [25] presented the damage states and associated maximum drift range for wood framing and OSB sheathing, based on visual damage observations. According to their findings, in order to prevent significant damage, drifts should be maintained below 0.5%. Drifts below this threshold are expected to result in only superficial damage, which can be easily repaired. Furthermore, drifts below 0.1% are unlikely to cause any damage. As the measured maximum drift of 0.12% approaches the acceptable threshold of 0.1%, it is not expected to cause any damage.

Figure 5 illustrates the maximum in-plane shear stress, denoted as S_{12}, in the diaphragm sheathing. The figure shows that the highest shear stress was observed in the upper left corner of the diaphragm, reaching a value of 0.188 MPa. Multiplying this stress by the thickness of the plywood sheathing yields the maximum in-plane shear demand of 3.39 kN/m as presented in Table 4. According to the Wood Design Manual [26], the shear resistance of the sheathing-to-framing connection for wood structural panel diaphragms with 10d nails spaced at 152 mm (6 in), a panel thickness of 18 mm, and Douglas-fir stud species is 7.01 kN/m. This finding indicates that the in-plane rigidity of the diaphragm with blocking and the shear resistance of the plywood sheathing are adequate to withstand the earthquake forces, as demonstrated by a demand-to-capacity ratio of 0.48. Additionally, it should be noted that the buckling resistance of the panel with the mentioned edge nailing is 50.8 kN/m, which is not dominant when compared to the shear resistance of the sheathing-to-framing connection.

Table 4. Maximum diaphragm deflection and shear stress generated in the plywood sheathing.

EQ-Y Direction	**Max Diaphragm Deflection in Y dir. [mm]**	**Max Shear Stress in Sheathing S_{12} [MPa]**	**Max Shear Demand in Sheathing [kN/m]**	**Shear Resistance [kN/m]**
Imperial Valley	2.97	0.180	3.24	
Landers	3.40	0.188	3.39	7.01
Iwate	1.88	0.123	2.21	

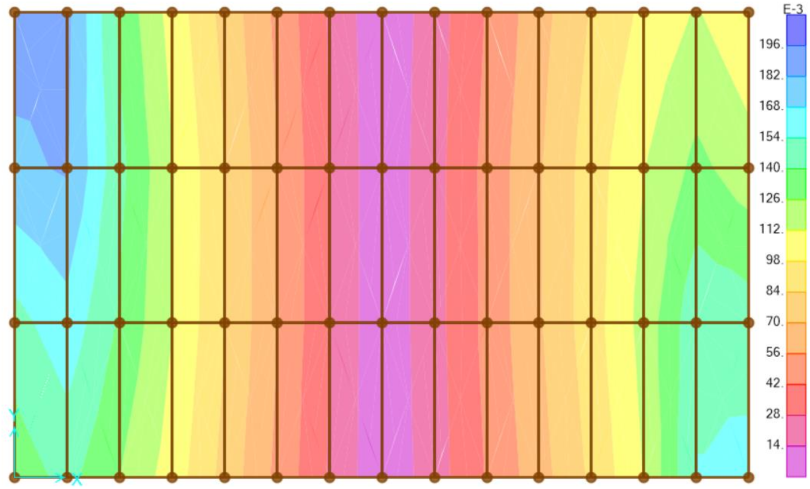


Figure 5. Max shear stress in plywood sheathing (S_{12}) [MPa] under the Landers record.

The section used for all floor joists, including the rim joists and end joists, is a visually graded No.1/No.2 Douglas-fir 38×286 mm (nominal 2×12 in). According to the Wood Design Manual [26], the resistance of the 38×286 mm Douglas-fir joist to tension, bending, and shear is 56.7 kN, 4.66 kNm, and 12.4 kN, respectively. Table 5 presents the maximum axial and shear forces, as well as the bending moment in the rim joists. Figure 6 illustrates the maximum axial force observed in the joists during the Landers earthquake record, while Figure 7 shows the maximum bending moment experienced by the joists under factored gravity loads in the seismic load case.

Due to the short unsupported length of the rim joists (406 mm/16 in) caused by the connection of the perpendicular joists, tension will predominate over compression in this diaphragm configuration. Wood framing members subjected to combined bending and axial tension loads must be designed to satisfy the appropriate strength interaction equation [26]

$$\frac{T_f}{T_r} + \frac{M_f}{M_r} \leq 1 \tag{1}$$

where T_f represents the factored axial tensile load, T_r represents the tensile resistance, M_f represents the factored bending moment, and M_r represents the bending moment resistance.

Table 5 indicates that the maximum tensile force in the joists is less than ten percent of their tensile resistance, or $T_f / T_r < 0.1$. Therefore, the floor joists adequately withstand the axial forces resulting from seismic lateral loads. The maximum bending moment occurs in the middle of the rim joists and is caused by the gravity load of the building's floor, walls, and roof, while the end joists only carry the load of the walls. The rim joists are subject to a combination of axial force resulting from lateral seismic loading and bending moment caused by factored gravity loads. The bending moment generated by lateral seismic loads is considered negligible and therefore disregarded. Analysis of the Landers earthquake record reveals that the rim joists experience a maximum tensile force of 5.2 kN and a maximum bending moment of 17.6 kNm. The bending moment resistance of the joists is 4.66 kNm, resulting in a high M_f / M_r value of 3.78 in Equation (1), which exceeds the limit. Additionally, the maximum ratio of demand induced by gravity load to the shear capacity of the rim joist is 0.99.

Table 5. Maximum tensile force, tensile resistance, and bending moment in joists.

EQ-Y Direction	Max Ten. /Comp. Force in Joists [kN]	Tensile Resistance [kN]	Max Bending Moment in Joists [kN.m]	Max Shear Forces in Joists (vert.) [MPa]
Imperial Valley	4.94			
Landers	5.20	56.7	17.62	12.28
Iwate	3.08			

In fixed base structures, the 38×286 mm (nominal 2×12 in) Douglas-fir rim joists adequately transfer gravity loads to the foundation. However, in an isolated diaphragm where the rim joists are placed on isolators, the reduced support surface area significantly weakens the rim joists' gravity load transfer. Therefore, it is necessary to replace these sections with larger or built-up sections to ensure adequate bending and shear strength (or place intermediate isolators).

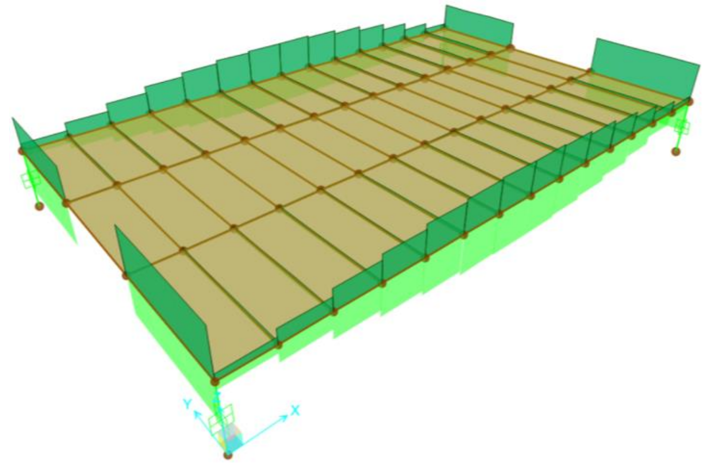


Figure 6. Maximum axial force [kN] in the joists under the Landers record in Y direction (roof and walls not shown for clarity).

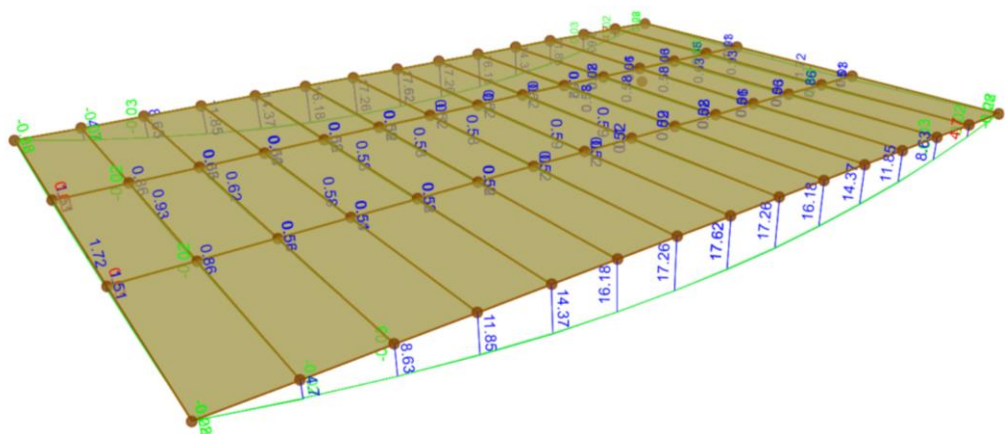


Figure 7. Maximum bending moment [kNm] in joists under factored gravity loads in the seismic load case (links, roof and exterior walls not shown for clarity).

CONCLUSIONS

This paper presents an analysis of the behaviour of plywood-sheathed floor diaphragms commonly found in residential single-family structures. The paper is structured into two main sections. The first section develops a practical approach for incorporating the effect of edge nailing and panel arrangement into the SAP2000 model by deriving shear stiffness modification coefficients. The equivalent model is validated by comparing the results with full-scale experimental studies conducted by Bott [17]. In the second section, the adequacy of the diaphragm under seismic loads is evaluated by developing a simplified finite element model of a one-story base isolated building in SAP2000, using the coefficients obtained in the previous section, and conducting nonlinear time-history analysis. Based on the experimental findings, the paper draws the following conclusions:

- For diaphragms with edge nail spacing of 152 mm (perimeter edges) and 305 mm (interior edges), the minimum and maximum shear stiffness modification factor (f_{12}) ranged from 0.07 to 0.15. The loading direction had an impact on these values, with the range being 0.11-0.15 for loading along the joist and 0.07-0.10 for loading perpendicular to the joists.
- The presence of an opening in the plywood sheathing diminishes the diaphragm's shear stiffness in both principal directions, causing a reduction in the shear stiffness modification coefficients.
- By varying the diaphragm aspect ratio from 1.25 to 4, the modification coefficients decrease from 0.15 to 0.07 in the loading direction of joists. The extracted modification coefficients can be used with relatively good accuracy for modeling LTF diaphragms with similar components and aspect ratios within the investigated range. The sensitivity of in-plane deformation of diaphragms to variations in the shear stiffness modification coefficient was examined, revealing that alterations in the modification coefficient significantly affect the in-plane stiffness and deformation of diaphragms. Therefore, accurate consideration of this parameter is crucial in diaphragm analysis.

- The results of dynamic nonlinear time-history analysis showed that the in-plane rigidity of the blocked edge diaphragm, shear resistance of plywood sheathing, and axial resistance of the framing members were sufficient to withstand seismic forces. Additionally, the diaphragm deflections were found to be within a range that is unlikely to cause damage. Nonetheless, it was discovered that rim joists in conventional wooden diaphragms utilized above a seismic isolation layer were identified as weak sections due to the introduced spans. This necessitates their strengthening to enable them to support an adequate amount of gravity loads and effectively transfer them to the isolators.

Further full-scale experiments are required to investigate the behaviour of LTF diaphragms with complex and irregular configurations, varying aspect ratios, and components. Additionally, it is imperative to develop more sophisticated numerical models of isolated Part 9 structures to effectively evaluate the adequacy of the diaphragm and its components against the applied seismic loads. These investigations are necessary to ensure adequate performance for typical LTF diaphragms in isolated single-family low-rise buildings.

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