Framework for Seismic Fragilities on Light-Frame Wood Structures and Retrofitting Strategies

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Abstract

Light-frame wood-dwellings in Canada are vulnerable to seismic hazards since most of the wood-frame residences were built before the introduction of a proper national building code. Although wood structures perform well under normal gravity and lateral loading, poorly designed seismic resistance can lead to damages to existing wood-frame buildings during hazards as well as causing substantial financial loss and widespread destruction. This paper reviews the mechanism and results from a full-scale shake table test performed under the NEESWood Project (2007), which provides valuable data on seismic responses of wood-frame structures. Next, the study is extended to the calibration of the Damage Index model, which provides empirical constants as a part of the fragility analysis. The paper first investigates how the results from the shake table test can be used in calibrating the Damage Index, followed by an implementation of the calibrated model in fragility analysis to study the probabilistic relationships between seismic motions and structural failures. A case study of a two-story baseline wood structure (Fisher et al.2001) is illustrated. Finally, by comparing changes in fragility curves before and after retrofits, the paper studies the relative effectiveness of different retrofitting strategies, such as the installation of sheathing nails, gypsum wallboards, and anchor bolts.

Key words: *Seismic Analysis, Wood-frame Structure, Shake Table Test, Seismic Design*

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1.0 Introduction

Earthquakes are one of the root causes of devastating damages to infrastructure and residential buildings, hindering social and economic activities and generating significant financial loss across the affected region. The catastrophic nature of earthquake hazards can be attributed to the simultaneous occurrence of building collapses and severe damages to city infrastructure. The disastrous impact of earthquakes could be significant, especially in urban cities with a high concentration of population. Since more than 4000 earthquakes occur in Canada every year, though the majority of these are below magnitude 3, it is necessary to reduce urban seismic risks and enhance seismic resistances of existing buildings. Generally, earthquakes occur most frequently along the western coast of Canada (Figure 1) around the Cascadia Subduction Zone. Eastern Canada has a relatively lower rate of earthquake activity as it falls into a stable continental region within the North American Plate.

Wood-frame structures are the most prevailing construction type for residential buildings in Canada. Approximately 40% of the existing wood-frame housing inventories were constructed before 1970, followed by 25% in the 1970s and 21% in the 1980s, and the rest were built after 1990 (Ventura, Finn &Onur, 2005). However, about half of the existing wood dwellings are non-engineered, which lacks sufficient resistance to seismic ground motions, as the implementation in the National Building Code of Canada (NBCC) was not implemented until 1974. Based on the historical records, the Natural Resource Canada (NRC) has estimated that a 5-6 magnitude earthquake occurs approximately every 25 years in Canada and every 100 years for earthquakes with a magnitude greater than 6. Therefore, analyzing the seismic performance for wood-frame structures is of vital importance to provide a better understanding of the structural response, in other words, forestall destructive damages during earthquakes. Moreover, for existing wood-frame buildings, seismic analysis can help in deciding the necessity of retrofits and in selecting appropriate modifications based on the relative effectiveness for each strategy.

Performance-based seismic design (PBSD) has been one of the most effective approaches for the design of structures, the extreme environmental hazards, and enables engineers to systematically analyze the performance of the selected building based on the probability of failure under specified loading. PBSD provides insights for the selection of structural systems, construction sites, and building configurations as well as analytical approaches used in the structural design. There are various ways to test whether a given structure has adequate strength and energy dissipation capacity subjected to a given earthquake. This paper mainly focuses on the fragility analysis method to evaluate the performance of a wood-frame structure under seismic ground motions. The word 'fragility' in the seismic analysis is defined as the probabilistic relationship between structural damages and earthquake ground motions at a specific spectral acceleration. Therefore, the characteristic of the selected earthquake and the reference for different damage levels of the structure has to be first defined as well as the source of uncertainties. Park and Ang (1985) proposed the use of a linear combination of the peak displacement and hysteretic energy dissipated during the earthquake, called Damage Index Method, to evaluate the level of damages for a reinforced-concrete structure. A damage index is a factor that represents the degree of damage of the structure, and typically ranges from 0 to 1, with the value of 1 representing complete collapse. Their method was then extended to light-frame wood structures by Van de Lindt (2005). Specifically, Van de Lindt modified the damage index method based on the experimental data. One example for the experimental data can be the shake table test performed by NESSWood Project in 2007, and it has been one of the most reliable references for the modification of the Damage Index as it is one of few wood-frame megaprojects in the past several decades. This multiyear project that studies the seismic response of wood-frame structures continued the work initiated by the CUREE-Caltech Project (CUREE, 2002) by performing and analyzing a series of shake table tests based on the CUREE prototype buildings (Van de Lindt et al. 2006). The results from the NESSWood Project provided a valuable basis for both experiment and analysis that all researchers could utilize. Thus, the damage-index based fragility analysis was developed and then applied to a single shear wall from the first floor of the test building in the NESSWood Project. Additionally, Van de Lindt further explained how to extend the damage index from a single shear wall to full structures. Based on the calibrated damage index, improved fragility analysis tends to predict the seismic response of wood-frame structures more accurately than traditional drift-based method. Lastly, different retrofitting strategies were evaluated through the fragility curve as it directly reflects the change in vulnerabilities, before and after the retrofit, for the selected wood-frame structure. Therefore, this analysis provides an answer for structural engineers in determining whether retrofitting is required and what is the most optimal mitigation method by displaying the relative effectiveness for each retrofit. The fragility analysis method effectively accelerates the speed of the decision-making process and simplifies complexities in the wood-frame seismic analysis.

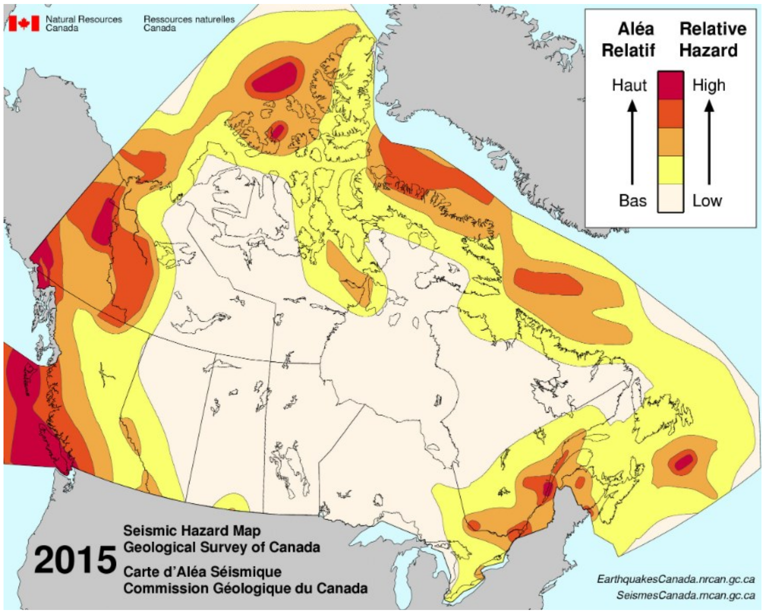


Figure 1. Simplified seismic hazard map for Canada (2015)

2.0 Shake Table Test

2.1 The necessity for conducting a shake table test

Almost half of the residential buildings in Canada are wood-frame structures. A significant improvement in structural engineering in the last two decades has been the development of the PBSD for wood-frame structures, which effectively strengthens the seismic resistance for these structures by quantitively estimating the probability of failure under various circumstances to achieve the damage control. However, before the NEESWood Project in 2007, a major difficulty to the development of the PBSD for light-frame wood buildings is the lack of a comprehensive understanding of factors that influence the seismic performance of structures. Seismic design provisions for wood-frame structures at that time were largely based on the traditional force-based design procedure, which was mainly concerned with providing adequate lateral strength to the design structure under a single seismic hazard level associated with life safety level. While considering only the lateral strength of a structure within a certain level does not guarantee safety nor successfully ensures the damage-control under various earthquake levels. Most costly damages to wood-framed structures from historical earthquake records have been related to many external factors rather than only life-safety damage controls, such as interior or exterior wall cracking caused by excessive roof drifts, or connection failures (anchor bolts, hinges, etc.) by high seismic accelerations. More importantly, previous studies performed in the 2000s mainly focused on concrete and steel structures. Lack of experimental knowledge and researches related to wood-frame structures at that time had hindered structural engineers from further improving the accuracy in predicting seismic responses of wood structures. Therefore, the desire for extensive experiments on wood-frame structures became intense for upcoming PBSD studies, so led to the shake table test for light-frame wood structures preformed by NEESWood Project, and this project intends to provide a better understanding of the many other external factors related to the seismic performance of wood structures.

2.2 Testing protocols

The objective of the NEESWood project is to develop an improved seismic design approach for wood-frame buildings. Multiple benchmark tests were conducted for buildings with different configurations (Table 1), which intended to evaluate the response of each configuration subjected to two types of ground motions. More specifically, Canoga Park Earthquake (1994, Northridge) with amplitude scaling factor of 1.2 and unscaled Rinaldi earthquakes (1994, Northridge) were used as the ordinary ground motions and near-field ground motions, respectively. The ordinary ground motion is corresponding to an earthquake with a probability of exceedance of 10% in 50 years, and the near-field ground motion has a probability of exceedance of 2% in 50 years. Furthermore, the dynamic characteristics of the buildings, including mode shape, and natural period were identified through low amplitude white noise tests as the damages accumulated during each test phase. To ensure that all experimental results were based on the same structural condition, the building was repaired after each shake table before proceeding to the subsequent test phase. The protocol for repairing test buildings was considerable, which included replacing its damaged gypsum wallboards, oriented strand boards, and wood studs.

From Table 1, Test phases 1 is considered as the reference in the comparison to the phase 3, 4, and 5 in order to investigate the effect on the structural performance from installing interior and exterior wall finishes. Test phase 2 along is dedicated to evaluating the feasibility of implementing dampers to enhance the seismic resistance as well as to assess passive energy dissipation during the shake table test. This paper mainly focuses on phases 1, 3, 4 and 5 as these four phases can be used as the comparison to investigate the impact on the structural response by several typical retrofitting approaches. Phase 1 consisted of only wood members without any wall finishes. Phase 3 was mainly based on Phase 1, but additional 12-mm-thick gypsum wallboards were used in the interior surface along perimeter walls. The difference between Phase 3 and 4 was that interior wall finishes in Phase 4 were applied to all interior partition walls and ceilings instead of only on one side. Lastly, Phase 5 was settled for the comparison to Phase 4 to investigate the effectiveness of additional stuccos on exterior wall partitions to building's seismic performances. All dynamic responses of the wood-frame structure were collected by extensive high-resolution photographs during the NESSWood shake table test.

Table 1: Test building phases for the shake table test (excerpted from NESSWood Project 2007)

Test phase	Test building configuration
1	Wood structural elements only
2	Test Phase 1 structure with passive fluid dampers incorporated into selected wood sheathed walls
3	Test Phase 1 structure with 12-mm-thick gypsum wallboard installed with #6 32-mm-long screws at 400 mm on center on structural wood sheathed walls
4	Test Phase 3 structure with 12-mm-thick gypsum wallboard installed with #6 32-mm long screws on all walls (400 mm on center) and ceilings (300 on center)
5	Test Phase 4 structure with 22-mm-thick stucco installed with 16 gauge steel wire mesh and 38-mm-long leg staples at 150 mm on center on all exterior Walls

2.3 Results and influences from the shake table test

The structural responses from the shake table tests are characterized by the relationship between the base shear forces and relative horizontal displacement at the inter-story level. Based on the experimental data (Filiatrault, Christovasilis et al. 2009), the installation of gypsum wallboard finishes (Phase 3) approximately reduced the transverse first-story displacements by 43% compared to the wood-only structure (Phase 1). And the wood-frame structure in Phase 3 was much stiffer than in Phase 1, confirming the significance of the gypsum wallboard in stiffening structural walls. Additionally, the transverse roof displacements were further reduced by 29% compared to Phase 3 when gypsum wallboards were installed on all partition walls and ceilings

(Phase 4). The introduction of stucco on the exterior walls (Phase 5) made the wood-frame structure stiffener, and it diminished the roof displacement from 24 mm in Phase 4 to 18mm.
As the largest full-scale three-dimensional shake table test ever performed in the U.S, the results of this series of shake table tests with various building configurations play a significant role in subsequent related studies, such as the calibration on the fragility analysis and the development of non-linear seismic models for wood-frame buildings. This valuable experiment also contributed to the development of direct displacement-based design (Sullivan et al, 2009) philosophy that focuses on controlling building damage by limiting inter-story deformations. Then, the PBSD was extended beyond only predicting structural failure to incorporate economic analysis to identify the most effective retrofitting strategy as well as to extend the lifespan of wood-frame buildings. The NESSWwood project not only provided answers to what externals factors might influence the seismic behavior of wood-frame buildings but also pointed out the direction for further researches on wood structures. The project culminated with the shake table test of a six-story building through an earthquake of magnitude 7.5 and confirmed that the large residential building could be successfully designed to withstand expected earthquake activities, which reached a significant milestone on the road towards constructing higher wood-frame buildings.

3.0 Seismic Fragility analysis

3.1 Fundamental theory of the Fragility Analysis

The word 'Fragility' is represented by the probabilistic relationship between the level of structural damages and the intensity of earthquakes or the conditional probability that a reaches a given level of damages at a given spectral acceleration S_a. The probability for the fragility curve is defined based on the normal distribution as:

$$P[d_s \geqslant C|S_a] = \phi\left[\frac{1}{\beta_{ds}}\ln\left(\frac{S_a}{S_{a,ds}}\right)\right] \quad (1)$$

where d_s is the threshold damage state calculated from the Damage Index Method (Park& Ang et al, 1985), C is the capacity of the structure. $S_{a,ds}$ is the median spectral acceleration at which the structure reaches its critical damage states d_s. β_{ds} is the standard deviation for the normal distribution function at the threshold damage state d_s. Φ is the symbol of the cumulative normal distribution function.

The standard derivation β_{ds} controls the shape of the fragility curve, which significantly affects the results of the prediction. In the ideal case where the demand and capacity can be defined with certainty, the area of the normal distribution is 'squeezed' into a vertical line, and the fragility curve essentially becomes a step function. Therefore, accurately defining the uncertainty β_{ds} is vital before applying the fragility analysis. The total uncertainty of structural damage state β_{ds} can be expressed by the following formula:

$$\beta_{ds} = \sqrt{\beta_c^2 + \beta_d^2 + \beta_m^2} \quad (2)$$

where β_c stands for the standard deviation parameter in the capacity, based on experimental data and the level of structural damage obtained from the Damage Index Method. The standard deviation β_d is related to the seismic demand variables such as roof drift, plate bearing force, and tensile forces along with the bolts. Although the β_d is derived from the same synthetic ground motions, the value of β_d varies with the configuration of the wood-frame structure. This is mainly attributed to the fact that buildings with different configurations behave differently subjected to ground motions. Lastly, the application of a numerical model may be affected by uncertainties β_m due to modeling errors. Substituting three uncertainties in Eq.2 back into Eq.1, the final expression for the fragility formula becomes:

$$P[d_s \geqslant C|S_a] = \phi\left[\frac{\ln(S_d)-\ln(S_{d,ds})}{\sqrt{\beta_c^2+\beta_d^2+\beta_m^2}}\right] \quad (3)$$

This formula is developed from the traditional fragility method that is entirely based on statistical distributions of inter-story drifts at a given spectra acceleration and can only be applied to the ideal situation. The damage-index based fragility equation (Eq.3) involves several considerations in realistic scenarios and tends to generate more reliable predictions than the traditional method. However, the damage level for wood structures is difficult to define as wood materials do not perform a perfectly elastic relationship between stress and strain nor behave plastically after the yielding point. Therefore, extensive calibrations are needed to obtain a reliable value of d_s. The following section further discusses the procedure of calibrating the damage index and how does this method improve the accuracy of seismic analysis by allowing structural engineers to adjust parameters under various environmental conditions.

3.2 Park-Ang (1985) Damage index model

Extensive studies for reinforced concrete structures were carried by Park and Ang (1985) to construct a linear combination of the peak displacement and hysteretic energy dissipated during the earthquake to estimate the damage level of a structure. As mentioned in section 3.1, the traditional fragility method mainly focuses on the distribution of maximum inter-story drifts, whereas it ignores some other factors such as energy dissipation, changing aspect ratio (width-to-height) in the ductile state, and cyclic loading. Park-Ang damage model complements the accuracy of the fragility method as it examines the effect of hysteretic energy dissipation with cyclic loading during an earthquake. After obtaining the damage index, the index can be directly used as the threshold damage state d_s and substituted back to Eq.3. However, the development of Park-Ang damage model was initially aimed at evaluating reinforced-concrete structures. Van de Lindt (2005) extends the Park-Ang damage model that enables a damage-based fragility analysis for wood shear walls, and constructs the relationship between shear walls and the whole structure. The damage model for a single shear wall is defined as:

$$DI = \frac{\Delta_m}{\Delta_u} + \frac{\psi}{F_{ey} * \Delta_u} \bar{\int} dE \tag{4}$$

where DI is the damage index of the wall, Δ_u is the ultimate deformation of the wall subjected to the predetermined monotonic load. Δ_m is the maximum deformation during cyclic loading. The damage index calculated from these two factors is generally used in the traditional fragility analysis, which only considers the wall drifts. The second portion of the Eq.4 involves the effect of the amount of hysteretic energy $\bar{\int} dE$ absorbed during the earthquake. The calibration parameter ψ is a regressive function that obtained from extensive comparations of damage survey results. F_{ey} stands for the yield strength of the wall.

The Damage Index model above can only be applied for a single shear wall. However, most fragility analyses require the damage index for the whole building to see the general performance during the earthquake. Van de Lindt investigated the relationship between a single shear wall and the entire structure, and proposed the equation:

$$DI_{structure} = \sum_{i=1}^{n} (\lambda_i)_{wall} (DI)_{wall} \tag{5}$$

where $DI_{structure}$ and DI_{wall} is the damage index for the whole structure and i^{th} wall, respectively. 'n' is the total number of walls in the selected structure, and λ_i is the relative weighting factor of the

i^{th} wall comparing to the entire wood-frame structure, which can be calculated through the following

$$(\lambda_i) = \frac{(E_i)_w}{\sum_{i=1}^{n}(E_i)_w} \tag{6}$$

where the relative weighting factor λ_i is the ratio of the hysteretic energy dissipated by the i^{th} shear wall to the total hysteretic energy dissipated by the entire building.

Therefore, the order for applying the damage index method is first to find the DI for a single shear wall (Eq.4), then calculate the weighting factor through Eq.6 based on the hysteretic energy. Lastly, substitute results from Eq.6 in Eq.5 to obtain the damage index of the entire structure.

3.3 Calibrations based on the shake table test results

The full-scale shake table test performed under the NEESWood Project in 2007 collected valuable data that contributed to the improvement of the damage-based fragility method. In this case, the dynamic data for the calibration is excerpted from Test Phase 4 in the NESSWood shake table test as its construction standard is similar to the majority of wood-frame residential buildings in Canada. Recall that this shake table test was recorded with high-resolution photographs that monitored shear deformations, building damages, and overall structural deformations. Although the hysteresis in vibrations was not documented in the test, it can be developed through a predetermined numerical model for each shear wall. The hysteretic response was calculated based on recorded top displacements of shear walls and theoretical restoring force obtained from Pei& Van de Lindt's (2008) model. The structural deformation of a single shear wall and the entire structure is mainly governed by two parameters: the aspect ratio and the perimeter nail spacing. Therefore, the calibration parameter ψ is described as a regressive function of these two parameters. Multiple linear regression based on damage survey results was used to find the best-fit relationship between numerical responses and experimental responses of the structure. The function is defined as:

$$\psi = \beta_0 + \beta_1 x_{NS}^2 + \beta_2 * x_{WH} * x_{NS}^2 \tag{7}$$

where ψ is the calibration parameter that used in Eq.4, and β_i is the regression coefficients, x_{NS} and x_{WH} are parameters for the perimeter nail spacing and the aspect ratio of the shear wall, respectively. It should be noted that, once the calibration is done, the equation does not need to be calibrated again for different wood-frame buildings as the model is regressive and only a function of two parameters. The subsequent analysis only requires the adjustment of x_{NS} and x_{WH}.

For example, Table 2 displays the computational results for regression coefficients based on the seismic response in Phase 4 from the NESSWood shake table test. Then, taking different values of x_{NS} and x_{WH} of each wall into the Eq.7 outputs different values of calibration parameter ψ (Table 3) in the Damage Index equation (Eq.4). Substituting the Eq.7 back to the Eq.4, the Damage Index equation becomes:

$$DI = \frac{\Delta_m}{\Delta_u} + \frac{\psi\ (= \beta_0 + \beta_1 * x_{NS}^2 + \beta_2 * x_{WH} * x_{NS}^2)}{F_{ey} * \Delta_u} \int dE \qquad (8)$$

Moreover, based on the Eq.5, a calibrated damage index for a single shear wall can be extended to the entire wood-frame structure. The DI for the whole structure is therefore applied to the normal distribution function (Eq.1) as d_s. Then, the probability of failure can be determined.

Table 2: Regression Coefficients based on the shake table results of a two-story wood structure. (Park& Lindt. 2009)

Regression coefficient	β_0	β_1	β_2
Value	1.120700	0.013881	−0.026316

Table 3: Calibration parameters subjected to different combinations of x_{NS} and x_{WH} (Park& Lindt. 2009)

Wall number	Story	Nail spacing	W/H ratio	*W* mm (in.)	*H* mm (in.)	ψ
1A	1	2	0.4091	1,371.6 (54)	3,352.8 (132)	1.1332
10A	3	4	0.8889	2,438.4 (96)	2,743.2 (108)	0.9685
D3	6	6	1.500	5,029.2 (198)	3,352.8 (132)	0.1994

3.4 Fragility analysis on a single shear wall

A single shear wall (No.1A) from the first floor of the wood-frame building in the NESSWood project (2007) is considered as an example in this section. Hundreds of nonlinear time analyses have been performed by the SAPWood model (Pang et al. 2007) to estimate the DI at a specific spectra acceleration. Although the SAPWood model does not output the DI directly, it gives the value of ultimate& maximum deformation as well as a nonlinear hysteretic data that can be substituted back to Eq. 8, then Eq.1, for each simulation. Therefore, each cycle of nonlinear time analysis plots a point on the fragility graph. Continuously calculating the average value of probabilities with the same DI at every Sa gives a line of the fragility curve of that DI. For simplicity, 0.3, 0.5, and 0.7 were selected as the reference damage index in the fragility analysis, as shown in Figure 2. To interpret the fragility curve, for instance, the probability of failure is 80% for 0.7 DI at 6g S_a. Furthermore, the fragility curve for the entire building can be obtained from repetitively calculating the fragility of each shear wall and applying them into Eq.5.

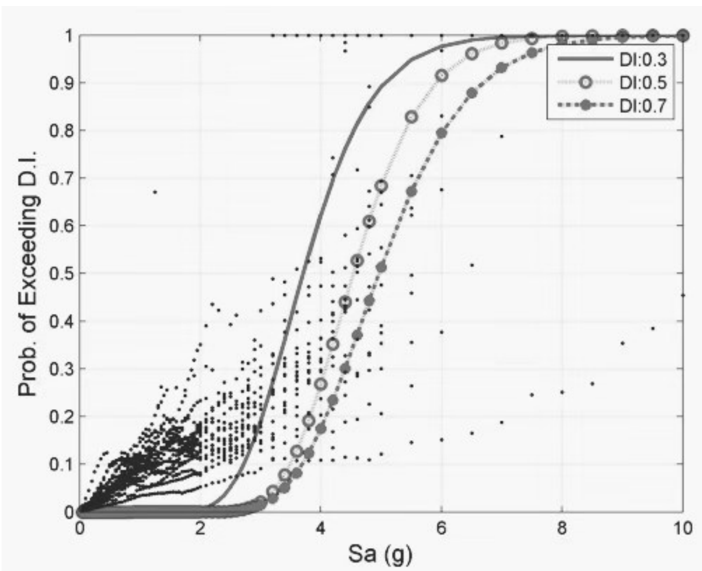


Figure 2. Fragility analysis for a single shear wall 1A in NESSWood Project (Park& Lindt. 2009)

3.5 Fragilities of other structural components in a wood-frame building

A wood-frame structure with brick masonry veneer (Figure.3) is a typical type of residential building in Canada as a wood structure with brick veneer walls have the advantage of, for example, better thermal insulation, moisture prevention, and erosion resistance than a wood-frame structure alone. The design concept of the brick masonry veneer is that assume wood backups (Figure.3) bear all lateral loads and gravity loads. Specifically, all lateral loads exerted on brick veneer are transferred to the wood-frame structure through metal ties. So that brick veneer wall damage frequently occurs at the connection point as the veneer moves away from the backup and causing the failure of metal ties during the earthquake. This failure is also named as 'out-of-plate' failure.

Therefore, the seismic response of veneer brick walls significantly relies on the performance of the tie connections. Seismic fragilities of veneers are evaluated as a function of three representative types of tie connections: (1) tie bend eccentricity (or spacing), (2) tie thickness, and (3) tie connection geometry. Reneckis& LaFave (2009) investigated the out-of-plane seismic performance of anchored brick veneer with wood-frame backup wall systems and concluded that the thinner tie was more vulnerable to the seismic hazards and a denser tie spacing was able to shift the fragility curve to the right, which reduced the probability of connection failures.

Moreover, as the most common component in wood-frame residential buildings, brick chimneys have a similar response as the brick veneers during earthquakes. But damages caused by earthquakes on brick chimneys are much more severe than on brick veneers as most chimneys are only connected at the bottom. Seismic ground motions may cause a large P-delta effect on bottom metal connectors, which quickly breaks connections between the roof and the chimney. Also, cement between bricks are too stiff to resist the seismic vibration, and the detachment of bricks is always regarded as the most probable failure that occur during an earthquake. Various retrofitting methods can be used to reduce the probability of failure, such as installing additional metal flashing at the bottom of the chimney (Figure.4) or placing metal bars to support the top of the chimney. Applying theses retrofits on the chimney can effectively reduce the probability of failures during earthquakes.

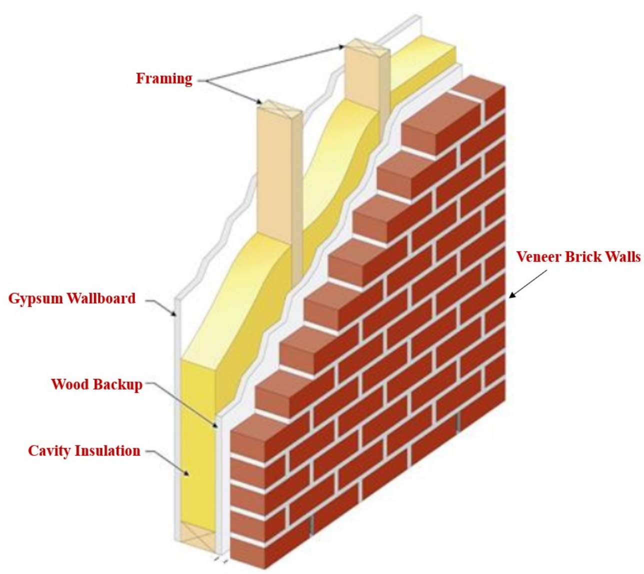


Figure 3: Veneer brick walls in the wood-frame structure (Innovations In Buildings, 2017)

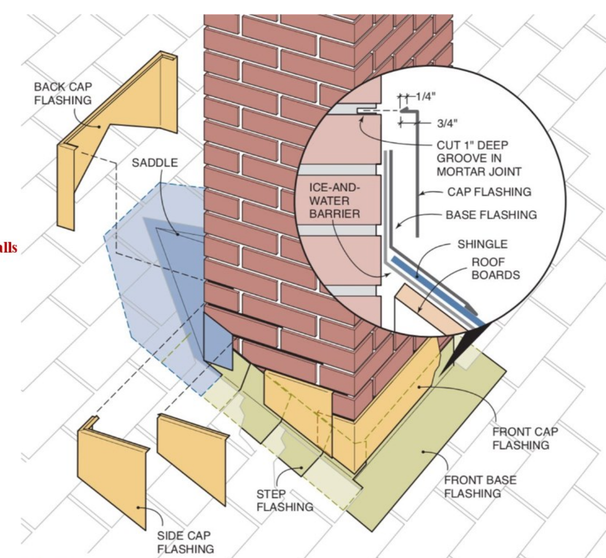


Figure 4: Typical brick chimneys with the flashing retrofit (Family Handyman, 2013)

4.0 Retrofitting for Wood-Frame Structures

Instead of estimating the conditional probability of structural damages, another essential role of the fragility analysis is evaluating the relative effectiveness of different retrofitting approaches based on the change in probabilities of structural failures. By comparing the critical damage state and capacity for one structural component fragility, with and without retrofit, it can be quickly concluded whether the retrofitting strategy is necessary for the current scenario. Beyond analyzing the performance of a single component due to the impact of retrofitting, assessment of the whole structural system should also be taken into consideration during the analyzing process. In this section, several modes of failure for the light-frame wood structure during the earthquake are discussed, as well as how fragility analysis can quantify the relative effectiveness of various retrofitting options.

4.1 Modes of failure for the Wood Structure buildings

The general failures (shown in Figure 5) for wood-frame structures due to seismic motions are (1) the inter-story horizontal drift, (2) base wall separation due to hold-down uplift force, and (3) sill plate splitting. The peak inter-story horizontal drift ratio can be represented by the ratio of maximum deflection of a story to its height. The building collapses when the drift ratio exceeds its capacity. Base wall separation is defined as the limit at which the anchor bolt at one end of the wall is pulled out from the foundation due to the tensile forces. The sill plate splitting, in some extreme cases, is when the building slides off its foundation due to exceedingly high shear forces.

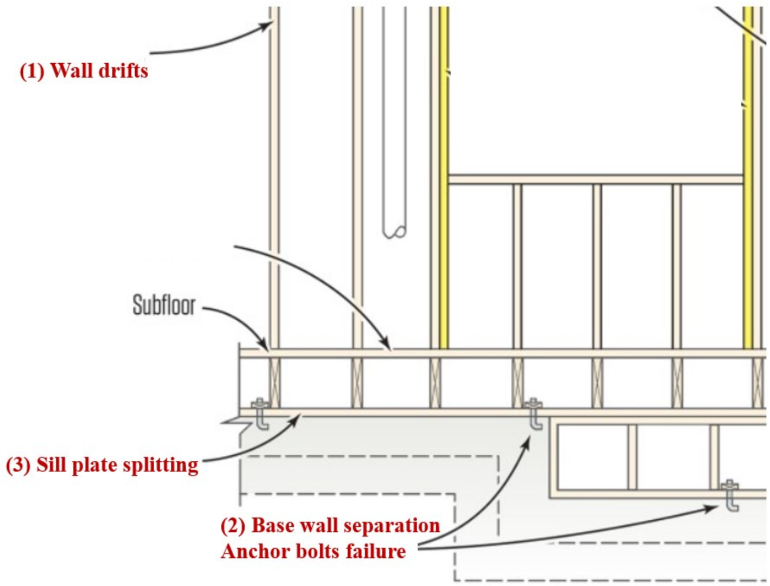


Figure 5: Three types of failures for the wood-frame structure (Light-Frame Wood construction Manual, 2015)

4.2 Retrofitting strategies

Approximately 40% of the existing wood-frame housing inventories were constructed before 1970, these wood structures were built without anchorage on their foundations, which could be severely damaged by the effect of the sill plate splitting and wall lift as their foundations lack the resistance to withstand earthquakes. The absence of anchorage may also contribute to the 'soft-story' (Figure 6), with one level of a building is significantly weaker in lateral load resistance than stories above it. The soft-story effect at the first level is one of the root causes for wood-frame building collapses as both seismic ground vibrations and the p-delta effect contribute to the building damage. Therefore, retrofit measures have to be introduced to deal with these potential threats. Three retrofitting strategies are generally used in wood-frame structures. (1) Adding more anchor bolts, (2) installing additional gypsum wallboards, and (3) inserting more sheathing nails (Figure.6). However, for existing wood-frame structures, there are certain limits for the installation of additional gypsum wallboard as it first requires partial removal of existing walls, which may potentially reduce the strength of the structure and cause accidents. Moreover, this approach is time-consuming and sometimes cannot achieve the expected performance after the retrofit, whereas the other two retrofit approaches are relatively economical and convenient to implement in most existing wood-frame buildings. Installing gypsum walls is most effective under the process of constructing new wood-frame structures, as mentioned in Section 2.3 (Phase 1 and Phase 3). Thus, the effect of additions of anchor bolts and sheathing nails are mainly investigated with the fragility analysis. The structural fragility curve quantitively describes how dose different retrofits impact the probability of structural behavior. Relative effectiveness of retrofits can be determined by the amount of fragility curve shifts after retrofitting.

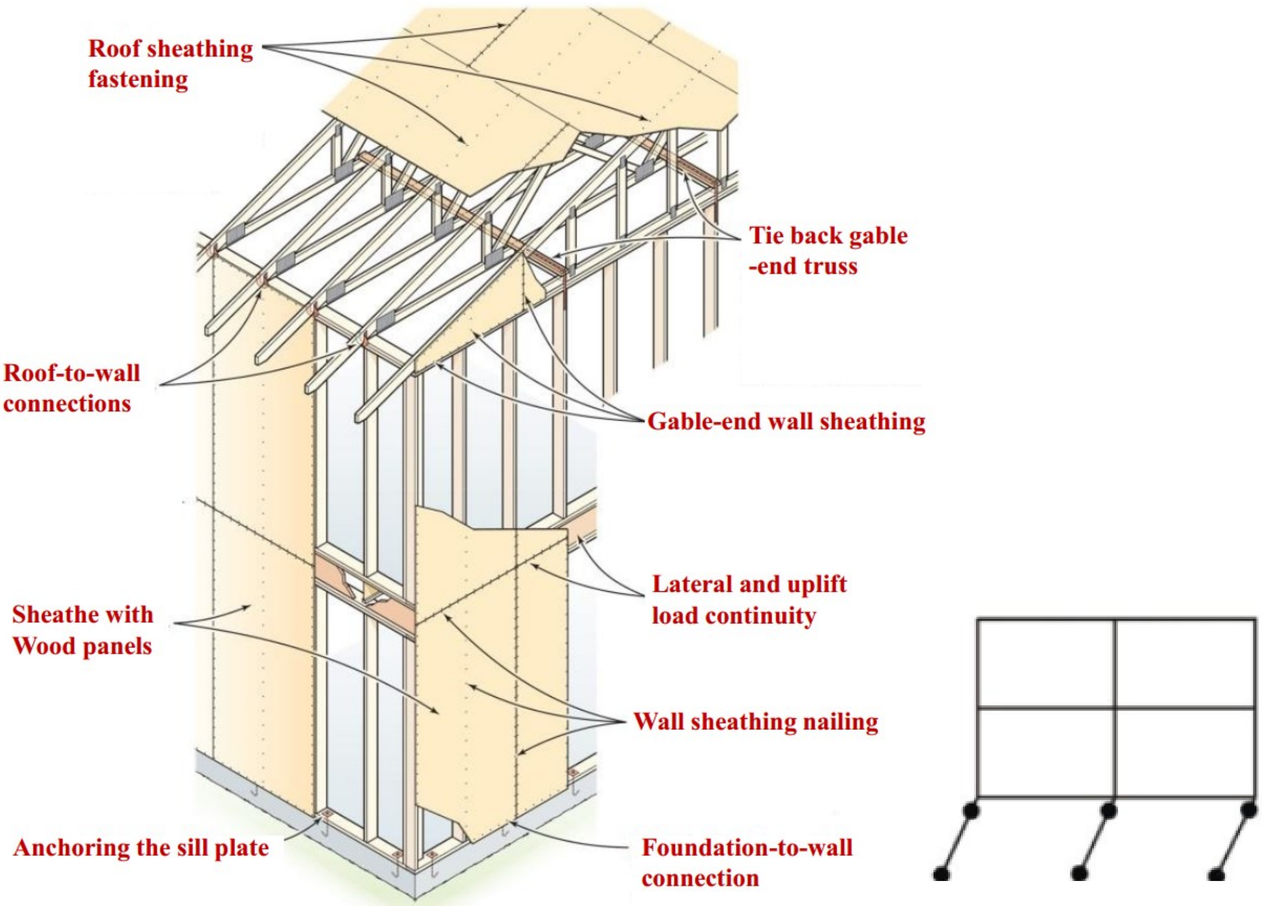


Figure 6. General retrofits on the wood-frame structures (left) and soft story effect (right) (Light-Frame Wood construction Manual, 2015)

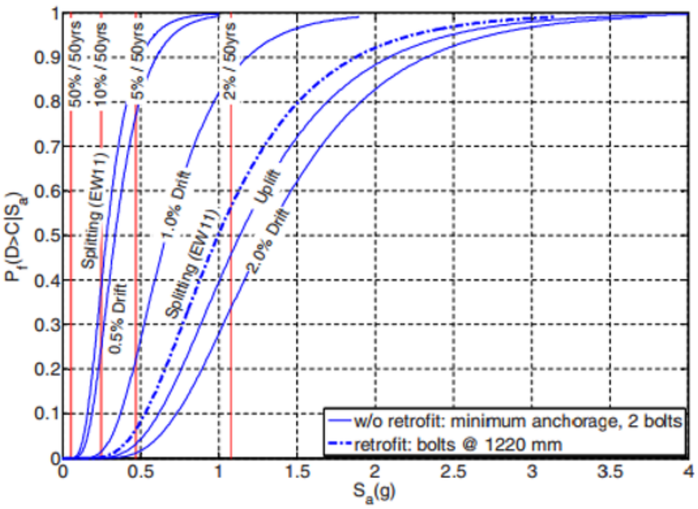
4.3 Improvements of the wood structure building after the retrofitting

Figure 7 and 8 describe the fragility analysis for a wood-frame structure, before and after two types of retrofit, performed by Pang, Rosowsky& Wang (2009). The prototype of the wood-frame structure for the fragility analysis is a two-story single-family building in Memphis, Tenn, and construction details are shown in Table 4. The fragilities for the test wood-frame building with and without anchor bolts retrofit are shown in Figure 7. The fragility analysis includes the study of vulnerabilities for three damage types: wall drifts, wall uplift, and sill plate splitting. Before the retrofit (rigid line in Figure 7), the risk of sill plate splitting at a 50-year (5%) earthquake is approximately 90%. The risk of wall splitting ultimately reduces to 8% after installing more anchor bolts. Because the total base shear in the fragility analysis for each shear wall was assumed to be equally distributed on every anchor bolt, the increase in the number of anchor bolts decreases the average in-plane shear force acted on each bolt, which effectively reduces the possibility of structural failure. However, additional anchor bolts do not necessarily improve the building's resistance to the wall drift as they only strengthen the shear and tensile resistance at the bottom of the wall, and it does not play a role in enhancing the structural stiffness for each shear wall. On the contrary, the use of denser sheathing nail spacing (Figure.8), for example, reduces the risk of failure caused by wall drifts at a 50-year (5%) earthquake from 87% to 30%. But it is companied by a greater probability of wall uplift as inserting more sheathing nails results in a higher overturning moment at the connection and greater uplift forces. Overall, the fragility analysis effectively facilitates the process of seismic analysis, in which allows structure engineers quickly determine pros and cons of different retrofitting approaches. This method still play an important role in nowadays seismic analyses.

Table 4: Construction Details for the two-story wood-frame structure in Memphis, Tenn. (CUREE-Caltech Woodframe Project, Fisher et al. 2001)

Structure ID		WF2-2
Number of stories		2
Foundation type		Slab
Plan dimension (m)		12.2×8.5
Nail spacing on OSB[a] Ext./int. (mm)		150/305
Screw spacing on GWB [b] Ext./int. (mm)		305/305
Stud spacing (mm)		610
Seismic	Cripple wall	n/a[c]
Weight	Floor1	167.7
(kN)	Floor2	128.1
Period (s)		0.286

[a]Oriented strandboard thickness=9.5 mm (3/8 in.), 8d common nails [2.78 mm (0.113 in.) diameter].
[b]Gypsum wallboard thickness=12.7 mm (1/2 in.), #6 bugle head dry wall screw.
[c]n/a=not available.



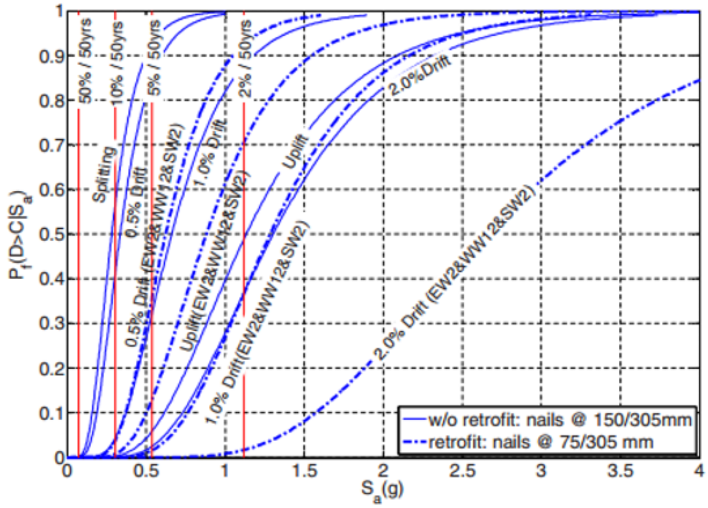


Figure 7: Fragility curves for two-story wood structure with anchor bolts (Pang, Rosowsky & Wang, 2009)

Figure 8: Fragility curves for two-story wood structure with denser sheathing nailing spaces (Pang, Rosowsky & Wang, 2009)

5.0 Conclusion

Existing wood-frame buildings in Canada, as almost half of them were constructed before the 1970s, are particularly vulnerable to earthquakes as they were built without a proper building code. The necessity for developing a seismic analyzing system is urgent for structural engineers to restrain the damages below a certain level. The paper discusses the damage-based fragility method that uses the probabilistic relationship to estimate structural failures as well as the calibration procedure for the damage index based on the full-scale shake table test performed by the NESSWood Project. The calibrated fragility method provides a better prediction for seismic responses of wood structures as it takes into account the effect of hysteretic energy dissipation with cyclic loading during an earthquake. Additionally, instead of applying fragility analysis only on shear walls, the study shows that the fragility analysis is also capable of evaluating other structural components such as brick veneers, chimneys, metal connectors. The study also pointed out that the fragility analysis can be extended to assess the relative effectiveness of different retrofitting approaches, which can set a reference for structural engineers to determine the most effective and economical approach. The fragility not only quickly indicates when the structural damage might occur, but also provides a simpler way to estimate the probability of structural failure. Despite new efforts to analyze structural responses and control building damages. More research is needed to develop an advanced building design, which can prolong the sheltering time for people to escape during a catastrophic earthquake. Structural engineers should be keen on constantly pushing the boundaries of the existing building design and buying time for people to survive during the catastrophe.

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