Fatigue performance of wood frame roof-to-wall connections with elastomeric adhesives under uplift cyclic loading

Bilal Alhawamdeh, Xiaoyun Shao*
Dept. of Civil and Construction Engineering, Western Michigan University, MI 49008, USA

ARTICLE INFO

Keywords:
Wind loads
Fatigue analysis
Roof-to-wall connection
Elastomeric adhesives

ABSTRACT

Roof-to-wall-connection (RTWC) is critical in the loading path of wood-frame residential buildings, whose fatigue performance under varying wind loading is investigated in this paper. To get an insight on the wind-induced fatigue behavior at low to moderate hourly mean wind speeds and to demonstrate the effects of adhesives on the fatigue performance of RTWC, two types of fatigue experiments, namely the constant and the varying amplitude loading tests, were conducted on three RTWC configurations with and without elastomeric construction adhesives. Based on the constant amplitude loading test results, fatigue life prediction models were developed, and the reduction in the static load capacity due to cyclic loadings were estimated. Adhesives are shown to increase the endurance limit of the RTWCs, which is desirable to enhance the life-cycle performance of wood buildings. The varying amplitude loading test results indicate that buildings in non-hurricane regions are vulnerable to fatigue damage at a low-level mean wind speed. It may induce loadings above the endurance limit of the RTWCs. On the other hand, the linear Miner’s cumulative fatigue damage model can be reasonably used to predict fatigue damage of the RTWCs when subject to multi-amplitude wind loadings. Toenailed connections generally fail in a less ductile manner at a certain number of load cycles with no warnings compared to the connections with adhesives that fail in a more ductile manner. The testing results presented herein provide essential data on the hysteresis behavior and failure modes of RTWCs to facilitate future implementation of adhesives in wood constructions.

1. Introduction

Roof-to-wall connections (RTWCs) and roof sheathing in residential wood-frame buildings significantly influence the roof performance under wind loads. The critical role of these connections was also revealed from many post-hurricane/storm damage surveys (e.g., [1–3]). Roof failures not only endanger occupants of the houses, they also lead to water intrusion, resulting in subsequent damage to household items inside, such as furniture and appliances.

An experiment conducted by the Insurance for Business & Home Safety (IBHS) on a full-scale house under the impact of open wind turbines shows that the roof failure initiated at the rafter-to-top plate connections due to inadequacy in resisting and transferring loads [4]. Toenails are the most common fasteners used in RTWC in North America, and significant roof structure failures were due to the failure of these conventional connections, among which many were observed at wind speeds below the design level [5–7]. The underperformance of the roof connection can mainly attribute to the improper selection and application of construction materials (i.e., fasteners, wood framing, and sheathing) or strength degradation due to aging and long-term service within the intended life span [8].

The capacity of toenail connections to uplift loads has been the subject of many studies. For example, [9–17] examined various connection strengthening approaches, such as commercial metal straps and construction adhesives. Research on the effect of adhesive materials on wood construction has gained attention. Generally, better performance of structural members (i.e., roof connection, sheathing) under natural hazardous loading conditions was observed when adhesive materials were adopted in the construction [18]. Monotonic loading tests of RTWC specimens demonstrated that increased uplift resistances were achieved with the application of elastomeric adhesives, which may provide an affordable, efficient, and nonintrusive solution for roof connections in highwind areas.

Monotonic loading tests with a constant displacement rate ranging from 0.25 to 6.35 mm/min were generally conducted to determine the uplift capacities of RTWC specimens. However, whether these static...
tests can realistically reproduce the connections’ failure mechanisms is questionable, considering that RTWCs are usually subjected to fluctuating wind loads. Therefore, an attempt was made to investigate the effects of realistic wind loads on the nail withdrawal performance of RTWC [19]. It was found out that toenailed RTWCs tend to fail by incremental damage accumulation caused by severe short duration peaks of the applied wind load, where the failure occurred later at a load lower than the applied peak load. It was concluded in [19] that fatigue would accumulate in the connections over the service life under generally low to moderate wind speeds because of their frequent occurrence. This fatigue behavior requires further study to ensure the integrity of wood-frame structures. The effect of storm duration on RTWC failure in a wood-frame structure was also numerically investigated using a combined load sharing/nail-slip model [20], employing the concept of “design cyclone” and a validated roof structural model. The analysis results revealed a substantial increase in the probability of roof failure with increased storm duration.

One way to evaluate the connection’s capacity under long-duration wind load might be through low cyclic fatigue experiments, which were adopted in several studies to investigate the fatigue damage of metal roof claddings. A fatigue testing program of mechanical fixation elements of roofed low-rise structures was developed based on the design wind pressure [21]. Wind cycles of certain wind speeds were estimated considering the cumulative probability distribution of the 50-year return period. Fatigue performance of light gauge roofing was evaluated based on the cycles to fatigue failure versus loading levels determined using the wind loading spectrum of a design wind event [22]. Another procedure for estimating the wind-induced fatigue damage of roof claddings was developed in [23,24]. The rainfall count method was employed to determine the fatigue loading from a measured cyclone wind load history based on a wind tunnel testing of a model house. The S-N curve, where S represents the stress amplitude, and N is the number of cycles until failure, was used to estimate the fatigue damage in conjunction with Miner’s rule.

It shall be pointed out that the damage accumulation mechanism in low cycle fatigue for metal roof claddings is different from those of nailed connections in wood-framed buildings [20]. Understanding whether the fluctuating wind loading of longer duration and relatively lower amplitude induces fatigue failure is critical for wind resistance performance evaluation of RTWCs, especially in non-hurricane regions where toenails still dominate the wood frame constructions. These regions are exposed to winds with low to moderate speeds all year-long, where the damage is not expected due to overloading [25]. Therefore, the objectives of this paper are twofold: (i) to estimate the wind-induced fatigue damage of toenailed RTWCs; (ii) to evaluate the wind-induced fatigue mitigation performance of the proposed strengthening method using elastomeric adhesives.

2. Methodology

Estimating wind-induced fatigue damage in RTWC requires knowledge of fatigue analysis concepts. This section first presents the predefined wind-force time history adopted in this study. Then the rainfall count method for developing the fatigue loading protocol (i.e., the varying amplitude loading protocol), the development of S-N curves, and Miner’s rule for predicting fatigue life are introduced. A flowchart illustrating the fatigue analysis method adopted in this study is provided in Fig. 6 at the end of this section.

2.1. Realistic wind-force time history

The real wind loading’s effects on buildings vary not only based on the mean wind speed but also the geometry of the building, its location, exposure, and topography. Therefore, there is almost an endless number of realistic wind-load time histories for numerical simulations and experimental investigations. This study adopts a realistic, fluctuating roof wind pressure measured from a wind tunnel experiment of a 1/50 scale model house with 52 RTWCs [26], shown in Fig. 1. Specifically, the RTWC with the largest peak magnitude of the force coefficient (CF) in the time history was chosen, and a representative portion of that time history was selected [27]. The full-scale force was obtained using:

\[ F(t) = 0.5\rho V^2 CF(t)A \]  

(1)

where \( F(t) \) is the full-scale force–time history, \( \rho \) is the density of air, \( V \) is the hourly mean wind speed at roof height, and \( A \) is the geometric tributary area of the corresponding RTWC.

The wind loads were gradually increased by a wind speed of 5 m/s at a time for the same \( CF \); therefore, the same number of peaks were generated at each wind speed level (see Fig. 1). Note that complying with the scaling law shown in Eq. (2) [27], the time duration of the wind load decreases as the scaling wind speed increases.

\[ (VT/L)_{model-scale} = (VT/L)_{full-scale} \]  

(2)

where \( T \) is the total duration of the time history, \( V \) is the scaling wind speed, and \( L \) is the length scale. Table 1 lists the wind-force history duration and the maximum force at each scaling wind speed shown in Fig. 1. This force–time history was then analyzed to develop the wind-induced fatigue loading that could cause premature failure, as discussed next.

2.2. Rainflow count method

As a first step in estimating wind-induced fatigue damage, a cycle counting analysis is necessary to convert the irregular variations of wind loading cycles into a set of simple reversals. Then Miner’s rule, which is introduced later, can be employed to assess fatigue damage accumulation. While there are many cycle counting methods described in the ASTM E1049-85 [28], the rainfall count method is considered as a standard procedure to predict the wind-induced cyclic load on roofs [29]. Therefore, the rainfall count method was adopted herein to determine the number of occurrences of a specific mean load and load range. In executing the rainfall count cycling, a cycle is identified if it meets the criterion illustrated in Fig. 2. For a peak-valley-peak or a valley-peak-valley combination consisting of three loading points \( P_y, P_y \), if the second load range, \( \Delta P_y \), is greater than or equal to the first load range, \( \Delta P_y \), the cycle of the first load range (i.e., \( \Delta P_y \) and the three points \( x-y-x' \)) is counted. The mean value of this cycle (\( P_{ave} \)) is the average of \( P_y \) and \( P_y \). These two points forming the cycle are discarded, and the cycle counting is repeated starting from \( P_y \) and its following two consecutive points (i.e., \( P_y \) becomes \( P_y \) point in the next cycle counting). The counting continues until all the load points are consumed. More details on this method, including an example, can be found in ASTM E1049-85 [28].

In this study, the wind force–time history shown in Fig. 1 was analyzed utilizing the rainfall count cycle method, during which each loading cycle is sorted according to its mean and range values. Fig. 3 shows the distribution of the counted cycles with the corresponding mean and load ranges. The load ranges are distributed between 1.3 × 10^-6 kN to 6.37 kN, with 98% of the load ranges less than 0.048 kN, while the mean loads are distributed more evenly compared to the load.
range with a maximum and minimum value of 6.18 kN and 2.6 × 10^5 kN, respectively.

2.3. S-N curve and Miner’s rule

Continuous cyclic loading causes engineering materials and components to fail at a load level lower than its inherent strength. This lower level of load can be predicted using an S-N curve that relates the number of load cycles at the failure to a corresponding load level. The S-N curve is usually established following a stress-based fatigue approach, in which specimens are subjected to constant amplitude cyclic loading with different load ranges [30]. In this study, the constant amplitude uplift wind loading cycles (F) were used to develop the curve as a substitute for the stress (S), as illustrated in Fig. 4. The load varies between a maximum \( F_{\text{max}} \) and a minimum \( F_{\text{min}} \) with a mean value of \( F_{\text{m}} = (F_{\text{max}} + F_{\text{min}})/2 \). The load range \( \Delta F \) is defined as \( |F_{\text{max}} - F_{\text{min}}| \), while the amplitude \( F_a \) equals to half of the load range \( (\Delta F)/2 \). Therefore, for the cyclic uplift loading adopted in this study where \( F_{\text{min}} \) equals zero, \( F_a = F_{\text{max}}/2 \).

Based on the predefined load amplitudes \( F_a \) and its cyclic loading protocols, constant amplitude loading tests are conducted to obtain the number of cycles to failure \( N_f \), which is also known as fatigue life. The load amplitudes and the corresponding failure cycles are then plotted on a logarithmic scale to establish the S-N curve according to ASTM E468-18 [31]. A fatigue load-life model is therefore developed using regression analysis, which is mathematically represented as:

\[
F_a = AN_b^B
\]

where \( A \) and \( B \) are the fitting constants estimated from the regression analysis.

A typical S-N curve follows a slope that intersects with the load axis at the static load capacity \( F_S \), as shown in Fig. 5. As the loading amplitude decreases, fatigue life increases and eventually reaches a fatigue limit or an endurance limit. The fatigue limit is defined as the load level below which fatigue failure will not occur regardless of the number of load cycles. While the endurance limit \( F_E \) is the load level at a higher number of cycles, which is considered the upper limit of the number of load cycles that a component might be subjected to during its intended life span [22]. The determination of the fatigue limit is beyond the current study’s scope due to the extensive testing requirements on both the testing time and the number of specimens. Therefore, the endurance limit \( F_E \) is adopted as an index of fatigue performance of the RTWC specimens. \( F_E \) was estimated as the loading amplitude corresponding to \( N_f \) of 10^6 cycles (see Fig. 5) based on the S-N curves, representing the approximate upper limit of the low cyclic fatigue life of roofing components, such as RTWC and cladding [22].

A constant amplitude loading protocol, from a zero-minimum load to various maximum loads representing a percentage of \( F_S \), was adopted to develop the S-N curve (see Fig. 4). To account for the varying mean loads, the Smith, Watson, and Topper (SWT) approach was used to determine an equivalent load amplitude, \( F_{\text{eq}} \), in a fully reversed form with a constant mean load of zero. The SWT approach, as expressed below, has the benefit of not counting on any material constant [30].

\[
F_{\text{eq}} = \sqrt{F_{\text{max}}F_a}
\]

where \( F_{\text{max}} \) is the maximum load, \( F_a \) is the load amplitude in the constant loading protocol.

To simulate the wind-induced fatigue loadings on roof connections, varying amplitude loading tests were performed based on the identified

![Fig. 2. Criterion for counting a cycle following the rainflow count rule [30].](image)

![Fig. 3. Total load cycle distribution from the rainflow count method.](image)

![Fig. 4. Constant amplitude loading cycles and the associated nomenclature.](image)

![Fig. 5. Typical S-N curve (semi-log scale).](image)
cycles from the rainflow counting. Miner’s linear cumulative damage model was utilized in previous studies to predict the fatigue damage and fatigue life of steel roof connections subject to varying amplitude cyclic loading [32–34]. In this study, the cumulative fatigue damage index \( DI \) is defined based on the Miner’s model to quantify the specimen’s fatigue damage under multiple load amplitudes:

\[
DI = \sum_{j=1}^{m} D_j = \sum_{j=1}^{m} \frac{N_j}{N_j^f}
\]

where \( D_j \) is the proportional fatigue damage of the \( j \)th loading amplitude \((1 \leq j \leq m)\), and \( m \) is the total number of loading amplitudes. \( N_j \) is the number of applied cycles at the \( j \)th loading amplitude, and \( N_{j}^f \) is the number of cycles to failure under the constant loading of the \( j \)th amplitude. According to Miner’s rule in Eq. (5), fatigue failure is expected when \( DI \) reaches unity, that is when 100% of the life is exhausted [35].

2.4. Procedure for evaluating the fatigue performance of RTWC

The flowchart shown in Fig. 6 illustrates the fatigue performance evaluation of the roof connections employing both the constant and the varying amplitude loading tests. On the left, steps to develop the S-N curve are demonstrated, including the determination of the endurance limit based on the constant amplitude loading tests. The mathematical relationship between the applied load and fatigue life, known as the fatigue load-life model, is then established based on regression analysis. On the right, the rainflow count method is used to determine the wind-induced cyclic load (i.e., the varying amplitude loading protocol) from the wind-force time history. Fatigue damage of the test specimen of each configuration under the varying amplitude loadings is quantified using the \( DI \) defined in Eq. (5), where \( N_j^f \) is estimated using the fatigue load-life model for the \( F_{s} \) values resulted from the rainflow cyclic counting analysis, while the number of applied cycles \( (N_j) \) is directly obtained from the varying amplitude loading tests. The hysteresis curves and displacement behavior are also analyzed to provide reasonable explanations for the connections’ fatigue performance and failure modes observed from both tests.

3. Experimental program

3.1. Test specimen

The rafter-to-double top plates connection was selected as the RTWC specimen, and their fatigue performance was studied. Specimens representing three configurations were fabricated. Nailed specimens were built first. Either the polyurethane or the polyether adhesive, the two elastomeric adhesives adopted in this study, was then applied to the RTWC specimens making two more configurations. Monotonic uplift loading tests were conducted on the three configurations to determine their respective static load capacity \((F_{s})\) [18], which were then utilized to develop the fatigue testing program. Table 2 summarizes the three specimen configurations, their indices, and the \( F_{s} \) values.

Rather than merely testing the fastener and adhesive in an ideal test specimen configuration, attention was paid to achieve realistic test specimens that account for as-built conditions in a wood-frame structure, as shown in Fig. 7(a). Therefore, a licensed carpenter was contracted to prepare the 2 by 6 rafters with a “birdsmouth” notch used to fit on the top plates. Douglas Fir wood was selected as the material of the test specimens, and the assembly of the connection used the ring shank nails with a higher withdrawal design capacity (533 N/nail) compare to that of the typical smooth nails (355 N/nail) specified in the International Building Code (IBC) [36]. The double top plates 2 by 4 were first nailed together using four ring shank nails and then fastened to the rafter using three-ring shank nails driven by a pneumatic nail gun (see Fig. 7(b)). A formwork was used to control the angle and placement of the nails so that the same nail penetration depth can be obtained (see Fig. 7(c)). For the toenail connections with adhesives, toenailing was performed after applying the adhesives, during which a caulk gun was used to spread the adhesives covering the entire cross-section of the notch with an approximate thickness of 3 mm.

Table 2

<table>
<thead>
<tr>
<th>Configuration status</th>
<th>Configuration index</th>
<th>( F_{s} ) (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nails only</td>
<td>NA</td>
<td>1.9</td>
</tr>
<tr>
<td>Nails + polyether</td>
<td>N-PE 7.0</td>
<td>7.0</td>
</tr>
<tr>
<td>Nails + Polyurethane</td>
<td>N-PU 8.8</td>
<td>8.8</td>
</tr>
</tbody>
</table>

Fig. 6. Fatigue performance evaluation using constant and varying amplitude loading tests.
The ten-day standard curing process was followed before testing the specimens with adhesives. Photos of the representative specimens are shown in Fig. 8. Table 3 lists the properties of the materials used in the construction of the test specimens. The shear strength of the two adhesives used in this study was determined in [18] according to the standard lap shear tests specified in ASTM D2339 [37].

3.2. Test setup

A rigid steel fixture was designed and fabricated to hold the test specimen in the loading frame and to transfer the uplift loads (see Fig. 9). The MTS 318.10 testing machine with a capacity of 100 kN (22.4 kips) was employed to apply the loading. While welding points rigidly fixed the lower part of the steel fixture, the upper part could swivel freely as a universal joint to minimize the undesirable effect of loading misalignment. The uplift loading effects were introduced to the test specimens by imposing the load from the bottom of the test specimen through the lower steel fixture when keeping the upper steel fixture in place, as shown in Fig. 9.

3.3. Load protocols

Two loading protocols were developed, namely the constant and the varying amplitude loading protocols, as introduced in Section 2.4. The constant amplitude loading (see Fig. 4) ranges from a minimum of zero force to a maximum of a percentage of $F_S$, as listed in the second column of Table 4 in the next section.

The varying amplitude loading protocol was developed based on the rainflow cycle counting method, which consists of a sequence of loading cycles extracted from the wind force–time history with a maximum force less than $F_S$ of each configuration. Correspondingly, the wind force–time histories (see Fig. 1) up to a wind speed of 20 m/s and 40 m/s was utilized for the NA and the NPU/NPE configurations, respectively, based on Table 1 and Table 2. The loading cycles below a cut-off amplitude in the force–time history were excluded due to the minimal damage they could cause to the test specimen. And this cut-off amplitude was determined based on the specimen’s respective endurance limit $F_E$. Accordingly, the varying amplitude loading protocol for NPU configuration contains more load cycles than the NPE configuration due to its lower $F_E$ value (i.e., lower cut-off amplitude leading to a greater number of cycles included). The resulting varying amplitude loading protocols are shown in Fig. 10, which were repeatedly applied to the test specimens until failure.

In this study, all the tests were force-controlled, aiming to replicate the mechanical responses of roof connections under wind loads until they failed in fatigue. Both loading protocols adopted a cyclic uplift loading with a sinusoidal waveform at a frequency of 1.0 Hz. Synchronized data of applied forces, displacement responses, and elapsed times were recorded at a rate of 100 Hz. All the tests were discontinued under two conditions: 1) test specimens’ failure and the numbers of cycles to failure were recorded; 2) reaching the Run-out number of cycles (i.e., 250,000, which is explained later.

<table>
<thead>
<tr>
<th>Wood</th>
<th>Type and grade</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rafter and</td>
<td>Douglas Fir No. 2</td>
<td>38.1 mm $\times$ 184.2 mm (1.5 in.$\times$ 7.25 in.)</td>
</tr>
<tr>
<td>Top plates</td>
<td></td>
<td>38.1 mm $\times$ 88.9 mm (1.5 in.$\times$ 3.5 in.)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Fasteners</th>
<th>Description</th>
<th>Dimensions</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ring shank nails</td>
<td>Paper tap offset round head</td>
<td>2.9 mm $\times$ 60.3 mm (0.113 in.$\times$ 2.3/8 in.)</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Adhesive</th>
<th>Shear strength</th>
<th>Service temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Polyurethane</td>
<td>4.94 MPa (717 psi)</td>
<td>$-17.8^\circ C$ (0 $^\circ F$) to $-71.1^\circ C$ (160 $^\circ F$)</td>
</tr>
<tr>
<td>Polyether</td>
<td>2.88 MPa (418 psi)</td>
<td>$-23.3^\circ C$ (-40 $^\circ F$) to $-93.3^\circ C$ (200 $^\circ F$)</td>
</tr>
</tbody>
</table>

Fig. 7. Test specimen and construction: (a) schematic of the test specimen, (b) gun nailing method, and (c) construction formwork.

Fig. 8. Test specimen configurations: (a) NA; (b) NPE; (c) NPU.
4. Results

4.1. S-N curves based on constant amplitude loading results

The applied constant amplitude loading and the corresponding cycles to failure \( N_f \) are listed in Table 4. The constant amplitudes were converted into equivalent reversed load amplitudes using the SWT relationship, as discussed in Section 2.3, so the S-N curves can be developed based on the testing results.

As listed in Table 4, only one load amplitude was tested for each of the three configurations that did not fail the specimen at 250,000 cycles. However, such load amplitude cannot exclude other higher or lower loading values being the fatigue limit as failure might occur if more loading cycles were tested. Therefore, it was decided that the endurance limit instead of the fatigue limit to be determined as an index of the fatigue performance of connections, as explained earlier in Section 2.3.

The number of Run-out at 250,000 cycles was based on the limitations of a practical testing program with a limited amount of access to the shared loading equipment and the duration of each test (i.e., approximately 70 h to complete 250,000 cycles). However, the loading point corresponding to the Run-out cycles was not utilized in developing the fatigue load-life models since the specimens did not reach failure at the Run-out cycles. The S-N curves of the three configurations were then plotted on a semi-log scale, as shown in Fig. 11(a). The squared correlation coefficients \( R^2 \) of the regression lines range between 0.91 and 0.98 for the tested configurations, indicating that the regression equations match very well with the test results.

The comparison in Fig. 11(a) shows that at lower load amplitudes (i.e., less than 3 kN), the NPE specimens have a longer fatigue life than that of the NPU specimens. While, under higher load amplitudes (i.e., more than 3 kN), the polyurethane adhesive (i.e., NPU specimen) is more effective in expanding the fatigue life than the polyether adhesive. The S-N curves of NA and NPU configurations show a comparable slope in Fig. 11(a). These similar slopes reflect the same trend of the fatigue life of the two configurations, however, at different load levels, with the load levels of the adhesive connections being approximately three times higher than that of the NA configuration at the same \( N_f \). The endurance limit \( F_E \) was computed based on the fatigue load-life model equations, as illustrated in Fig. 11(a) and listed in Table 4.

![Fig. 9. Test setup [18].](image)

![Fig. 10. Varying amplitude loading protocol applied to the configurations NA, NPU, and NPE.](image)

![Table 4: Constant amplitude loading tests results.](table)

<table>
<thead>
<tr>
<th>Loadings</th>
<th>Results</th>
</tr>
</thead>
<tbody>
<tr>
<td>NA</td>
<td></td>
</tr>
<tr>
<td>0-40</td>
<td>0-0.76</td>
</tr>
<tr>
<td>0-50</td>
<td>0-0.95</td>
</tr>
<tr>
<td>0-71</td>
<td>0-1.35</td>
</tr>
<tr>
<td>0-79</td>
<td>0-1.50</td>
</tr>
<tr>
<td>0-90</td>
<td>0-1.70</td>
</tr>
<tr>
<td>NPU</td>
<td></td>
</tr>
<tr>
<td>0-40</td>
<td>0-1.96</td>
</tr>
<tr>
<td>0-51</td>
<td>0-3.60</td>
</tr>
<tr>
<td>0-61</td>
<td>0-4.30</td>
</tr>
<tr>
<td>0-71</td>
<td>0-5.00</td>
</tr>
<tr>
<td>0-80</td>
<td>0-5.6</td>
</tr>
<tr>
<td>0-90</td>
<td>0-6.3</td>
</tr>
<tr>
<td>NPE</td>
<td></td>
</tr>
<tr>
<td>0-40</td>
<td>0-3.52</td>
</tr>
<tr>
<td>0-45</td>
<td>0-4.00</td>
</tr>
<tr>
<td>0-50</td>
<td>0-4.40</td>
</tr>
<tr>
<td>0-70</td>
<td>0-6.20</td>
</tr>
<tr>
<td>0-80</td>
<td>0-7.00</td>
</tr>
<tr>
<td>0-91</td>
<td>0-8.00</td>
</tr>
</tbody>
</table>
adhesives show a 250~330% increase in the $F_{\text{E}}$ compared to the NA configuration. The $F_{\text{max}}$ of the corresponding $F_{\text{E}}$ for the three configurations was found to be about 35% of their respective $F_{\text{E}}$, which is comparable (i.e., approximately 5% increase) to the results of flexural fatigue cyclic tests of wood products incorporated with wax and resin [38].

Although adding more nails to RTWC generally increases its static capacity, it may not necessarily increase $F_{\text{E}}$ and $N_f$ values proportionally. The fatigue behavior is more complicated and dependent on many factors, such as stress concentration [30]. Besides, construction codes, such as the IBC [36], generally are strict on using extra nails over the specified fastening schedule, which may result in reduced net sectional area of the substrate, increased stress intensity, and then lower the split capacity. Therefore, adding adhesives to RTWCs provides an effective means to increase their endurance limits, which has the potential to further impact on the fatigue performance of wood residential buildings considering the critical role of RTWC in resisting cyclic wind uplift loading during its intended life cycle.

In addition to the fatigue load-life models shown in Fig. 11(a), the results from the constant amplitude loading tests were further utilized to determine the reduction in static capacity caused by the cyclic loadings. Fig. 11(b) depicts the relationship between the load reduction factor ($R$) and fatigue life can be made between the NA and the NPE/NPU specimens.

### 4.2. Varying amplitude loading tests

#### 4.2.1. Fatigue damage

Table 5 summarizes the results obtained from the varying amplitude loading tests, including the calculation of the cumulative fatigue damage index ($DI$) of the three specimens. The varying amplitude loadings are listed in the second column, after being converted into $F_{\text{ar}}$ using the SWT. The corresponding number of cycles applied ($N$) of each specimen at failure are listed in the third column. Noting that half-cycles were counted for some loading amplitudes, as shown in Fig. 10, and their corresponding $N$ values are half of the others. Although the loading amplitudes were different for each specimen, they were normalized based on the $F_{\text{ar}}$ of the specimens, as discussed in Section 3.3. The NPE specimen showed a relatively longer fatigue life of 955 repetitions (i.e., cycles of the total varying amplitude loading protocol) compared to the NPU specimen with a life of 876 repetitions. No direct comparison of fatigue life can be made between the NA and the NPE/NPU specimens because the maximum hourly mean wind speed tested for these two specimens was 40 m/s and was only 20 m/s for the NA specimen.

The number of cycles to failure ($N_f$) listed in the fifth column of Table 5 were either directly obtained from the constant amplitude loading testing results listed in Table 4 (e.g., $N_f = 8,900$ cycles when $F_{\text{ar}} = 0.95$ kN for the NA specimen), or computed using the fatigue load-life
models (see Fig. 11(b) and also listed in Table 4). The DI of each specimen under the varying amplitude loading was then computed using Miner’s rule, i.e., $DI = \sum (N/N_f)$, as listed in the last column. As mentioned previously, Miner’s rule is a linear fatigue damage model. Ideally, $DI$ equals 1 when 100% of the specimen’s life is exhausted due to the fatigue damage. $DI$ being greater or less than 1 represents an over-estimating or underestimating of the fatigue damage. As can be seen from Table 5, $DI$s of the NA and NPU specimens are 0.88 and 0.95, respectively, indicating that Miner’s rule slightly underestimates their fatigue damage. In contrast, there is an overestimation of the fatigue damage of the NPE specimen with a $DI$ value of 1.45.

The $DI$s being relatively close to 1 of all three specimens prove that the linear damage model (i.e., Miner’s rule) can be reasonably used to predict the fatigue damage of the roof connections under a wind event (e.g., storm) with varying amplitude loadings if the underestimation and overestimation seen from the test results are accounted for. It is also worth noting that the magnitude of loading amplitude is the dominant factor causing the most considerable fatigue damage when compared to the number of cycles. Take the NPE specimen as an example, at $F_{av} = 4.16$ kN and $N = 478$ cycles, $DI$ was computed as 0.3747. Whereas, at $F_{av} = 2.93$ kN and $N = 955$ cycles, $DI$ was only 0.0266.

Table 5 also shows that the force–time history corresponding to a wind speed of 20 m/s has peak loadings above the endurance limit (0.48 kN) of the NA configuration. Comparatively, for the NPE and NPU configurations, whose endurance limits are 2.10 kN and 1.72 kN, respectively, this wind speed is between 30 and 40 m/s. These hourly mean wind speeds (i.e., 20 m/s and 30–40 m/s) may be considered as the minimum wind speeds that generate peak loadings above the endurance limit of the respective configurations. Therefore, a mean wind speed of 40 m/s for buildings in non-hurricane regions [39] may induce loads above the endurance limit, causing fatigue damage in RTWCs, which needs to be considered in future development of design requirements. The fatigue performance of building components such as roof connections depends on the intensity and duration of a wind event, or in other words, the number and amplitude of load cycles [40]. Under wind loads, most cycles are of low amplitude, with only a few cycles with amplitudes above the endurance limit of roof connections. For example, in the 900-sec wind force–time history of 20 m/s wind speed shown in Fig. 1, there are about 10,600 cycles with only 13 cycles having loading amplitudes above $F_E$ of the NA configuration. Therefore, the intensity and duration of wind events at the design-level need to be assessed for typical structure events using the regional climatic data (i.e., metrological information).

4.2.2. Hysteresis curves and displacement behavior

The hysteresis load–displacement curves of the three configurations, under the varying amplitude loading defined in Fig. 10, are shown in Fig. 12. The hysteretic response of all three configurations exhibit sliding and stiffness degradation, as indicated in the figure. When the connection parts (i.e., rafter and top plate) slipped under the load, a horizontal sliding in the hysteretic curve is observed. During the horizontal sliding, noticeable displacement increase with no load drop of the current cycle, results in considerable energy dissipation during sliding (i.e., the area under the hysteresis curve).

The stiffness degradation is observed when the slopes of hysteretic curves decrease progressively in each loading cycle. The secant stiffness corresponding to the peak load cycle was computed and related to the percentage of the number of cycles to failure (%$N_f$), as shown in Fig. 13. As can be seen from the figure, the NPE and the NA specimens have a relatively linear stiffness degradation throughout most of their loading cycles, while the NPU specimen maintains a linear degradation up to 70% of its fatigue file and the degradation slightly accelerates after that. The linear regression analysis was then performed to find the best-fitted

![Fig. 12. Hysteresis force–displacement curves: (a) NA, (b) NPU, (c) NPE.](image-url)

![Fig. 13. Stiffness degradation.](image-url)
lines and their equations, as shown in Fig. 13. The slopes of these lines can, therefore, be used to estimate the stiffness degradation rate of the three configurations. It was found out that the stiffness of the NPE specimen degrades at a 130% higher rate than the NPU specimen. The NA specimen shows the minimum stiffness degradation (-0.0006 kN/mm/cycle) among the three configurations and maintains this rate until the last few failure cycles when a sharp drop in the stiffness occurred.

Stiffness degradation can be used as an indicator of the dissipated energy by the area captured under the hysteresis curves [41]. Higher stiffness degradation rate generally leads to a higher amount of energy dissipation. With similar load capacity levels, the NPE specimen dissipated more energy than that of the NPU specimen due to its higher rate of stiffness degradation. The NA specimen showed the minimum amount of energy dissipation due to its much lower static load capacity.

Displacement responses versus fatigue life percentages are plotted for the three specimens, as shown in Fig. 14. The increase in the displacement response (i.e., damage) with the number of cycles is evident for all the specimens representing the continuous accumulation of fatigue damages to failure. The displacement responses shown in the figure can be approximately defined using two phases, where phase I represents most of the fatigue life of the connections, and phase II involves a rapid increase in the displacement response up to failure. In phase I, the three specimens showed stable displacement responses of 1.0–1.5 mm. In phase II, the NA specimen had a shorter fatigue life percentage, which was only 5%. While the adhesives induce more plastic response of the RTWCs and exhibited 20–30% of their fatigue life in Phase II. Also, it can be observed that when the varying amplitude loading reduced to zero, residual displacements existed in the NPU specimen, which may be explained by the unrecoverable displacement that occurred in the wooden parts of the connection.

4.3. Failure modes

At the end of each constant and varying amplitude loading test, the failure behavior of the test specimen was observed to help interpret fatigue performance and hysteresis behavior. Pulling nails from the top plate was the predominant failure mode in the NA configuration (see Fig. 15(a)). Surface friction is the primary mechanism holding the embedded ring shank nails in the connection, which is, to some extent, a function of the nail diameter and its penetration. Therefore, the larger the friction surface, the higher the static load capacity and fatigue capacity (i.e., longer fatigue life). The nails are forced in-between wood fibers, so wood fibers are subjected to considerable tension and providing resistance to nail pulling out. The failure of the NA configuration under both loading tests occurred suddenly without noticeable warning. This behavior can be seen from Fig. 12(a) that when the applied loads overcome the friction/resistance of the nails due to the cumulative damage, the failure immediately followed with fewer numbers of cycles compared to the NPU/NPE configurations.

The elastomeric adhesives promoted adhesions in the substrate of the NPU and NPE configurations. Therefore, depending on the relative strengths of the wood and the adhesives, the connection might undergo cohesive failure or wood damage. When the adhesion to the substrate provided by the adhesive was stronger than the shear strength of the adhesive itself but less than the wood strength, cohesive failure would occur in the bulk layer of the adhesive (see Fig. 15(b)). This type of failure was observed in the polyether configuration (i.e., NPE). Differently, when both the adhesion to the wood substrate and the adhesive strength within itself was stronger than the wood strength, the failure would occur to the wood fibers in a less ductile manner compared to cohesive failure occurred to NPE configuration. Such failure would split the rafter (see Fig. 15(c)), which is dominant in the specimens using the polyurethane adhesive (i.e., NPU) because of its higher bonding and shear strength compared to the polyether (see Table 3).

5. Conclusion

An experimental study was conducted to investigate the wind uplift fatigue performance of roof-to-wall connections (RTWCs) under both constant and realistic varying amplitude loadings. Three configurations of RTWCs, including the toenailed connections (NA configuration) and the connections with two types of elastomeric construction adhesives (NPU and NPE configurations), were tested so the effects of the adhesives on the fatigue behavior of the RTWCs can be evaluated.

The S-N curves of each configuration are plotted based on the results from the constant amplitude loading tests, from which a fatigue load-life model was developed using the regression analysis, and the endurance limits were determined. Test results show that adding adhesives to the toenailed connections significantly increases the endurance limit by 250–330%. The hourly mean wind speeds of 20–40 m/s were found to be able to generate peak loadings above the endurance limit of the tested RTWCs and cause potential fatigue damage, that needs to be accounted for in design. Equations were derived to predict the reduction in the static capacity of an RTWC at a given number of cycles. Although the developed models and equations provide a means to estimate the fatigue life and load capacity reduction of the RTWC, they cannot be broadly adopted yet due to the limited number of specimens tested in this study.

Varying amplitude loading protocols were developed based on a realistic wind force–time history using the rainflow count method, and they were applied to one specimen of each configuration. The NPE specimen showed a longer fatigue life with 955 repetitions compared to the NPU specimen with 876 repetitions. The cumulative fatigue damage quantification was performed using Miner’s rule, and it was found out that Miner’s rule can be reasonably used to predict fatigue damage of the RTWCs when subject to multi-amplitude wind loadings. The stiffness degradation was computed based on the hysteretic curves and related to the percentage of fatigue life. The stiffness of the NPE specimen degraded at a 130% higher rate than the NPU specimen, while the NA specimen showed the minimum stiffness degradation. Failure modes of specimens under constant and varying loading tests are discussed to provide interpretations of the observed fatigue behavior.

In summary, toenailed connections may be exposed to fatigue damage at a low-level wind speed due to their lower endurance limit and tendency to experience a sudden failure at a certain number of load
cycles. Adding elastomeric adhesive to the toenailed connections provides a wind-induced fatigue mitigation alternative by increasing the endurance limit and significantly expand the fatigue life at the same load level or higher than that applied to toenailed connections. The testing results presented herein provide essential data on the hysteresis behavior and failure modes of RTWCs that are expected to promote the implementation of adhesives in the wood constructions. Further tests covering more load ranges are needed to refine the developed models to be reliably used.

**CRediT authorship contribution statement**

**Bilal Alhawamdeh**: Conceptualization, Methodology, Investigation, Formal analysis, Data curation, Writing - original draft, Writing - review & editing. **Xiaoyun Shao**: Supervision, Validation, Project administration, Funding acquisition, Writing - review & editing.

**Declaration of Competing Interest**

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

**Acknowledgements**

The authors are grateful to Prof. Daniel Kujawski of Western Michigan University and Dr. Mohammad Al Paralleh of Mutah University for the helpful discussion on the topic of this study. The authors would like to acknowledge the funding received through the Bronco Construction Research Center under the project grants program.

**Appendix A. Supplementary material**

Supplementary data to this article can be found online at https://doi.org/10.1016/j.engstruct.2020.111602.

**References**


