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DAMAGE LEVEL ASSESSMENT OF RESPONSE LIMITS IN LIGHT-FRAME WOOD STUD WALLS SUBJECTED TO BLAST LOADING

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Abstract: Currently, no systematic approach exists for damage evaluation of light-frame wood structures subjected to blast loading. This paper presents a detailed assessment of the behaviour of thirty-three full-scale light-frame wood stud walls subjected to a total of forty-eight shots of simulated blast loading. Detailed documentation of the observed damage allowed for the development of an accurate evaluation strategy of the response limits. The observed response limits are compared to limits derived from single-degree-of-freedom modelling using scaled pressure-impulse diagrams and to current code performance levels. It was concluded that the assumption made in contemporary blast design codes overestimates the ductility ratios for light-frame wood stud walls, and that using a maximum ductility of 2 is more appropriate and safer for blast design. Based on the observed damage levels obtained from the experimental study, the authors propose new ductility ratios corresponding to four damage regions.

Key words: blast, light-frame, walls, CSA S850, response limits, scaled PI, pressure-impulse, timber, single degree-of-freedom, shock tube.

Introduction

The development of blast guidelines has mainly focused on materials such as reinforced concrete and steel due to their widespread use in blast resistant design. Blast design provisions have been enacted in design standards in the United States (Department of Defense 2008; ASCE/SEI 59 2011), Canada (CSA S850 2012), and the United Kingdom (Office of the Deputy Prime Minister 2004) as a result of events such as deliberate attacks (e.g. Oklahoma City 1995; World Trade Center 2001) and accidental explosions (e.g. Ronan Point 1968; BP Texas City 2005; Lac-Mégantic 2013) causing loss of lives and high property damage. Subsequently, provisions correlating potential building damage levels to expected levels of protection (LOP) have been defined for design and assessment purposes. Each LOP, corresponding to potential building performance, is in turn associated with a response limit defining the component's damage level. The response limits, denoted in the design standards as B1 through B4, correlate to specific values of support rotation or ductility ratio, which is the final deflection of the component divided by its elastic limit. By considering the damage level descriptions, as defined in blast design codes in North America (PDC-TR 06-08 2008; ASCE/SEI 59 2011; CSA S850 2012) and presented in Table 1, it can be noted that damage descriptions such as *“component has not failed, but it has significant permanent deflections causing it to be unrepairable”* and *“component has some permanent deflection. It is generally repairable”* are not applicable to light-frame wood structural elements. In such complex systems consisting of components that are known to have variability in their stiffness and strengths, the damage of one component (e.g. stud) may not accurately reflect the subsystem (wall) or overall building damage.

Light-frame wood stud walls consist of ribbed plates acting together in partial composite action through metallic fasteners (usually nails) joining the sheathing panels to the framing members. The slip in these joints, caused by the wall bending, provides some ductility in the wall system (McCutcheon 1986; Liu and Bulleit 1995a, 1995b; Lacroix 2013). One noticeable aspect of these systems is their ability to share the load through a distribution which is based on the studs' stiffness, as well as load redistribution when individual elements such as studs or nails experience different

levels of damage. Design under static load takes system effects into account indirectly by employing a factor which accounts for the load sharing between the individual studs. When this failure criteria was developed, it was recognized that basing the failure of light-frame wood floor or wall systems on the weakest element was too conservative in representing the failure of the subsystem as a whole (Vanderbilt, Goodman and Bodig 1974). The failure of a single stud, especially if debris is limited or contained, would not cause hazardous risk to the occupants and therefore should not be considered as the ultimate resistance of the wall subsystem. Several proposed approaches attempting to define the subsystem-level failure (i.e. entire floor/wall) have been developed, varying from failures based on first member rupture (Foschi 1982, 1984; Bulleit 1985; Folz and Foschi 1989) to multiple-member failure (Gromala 1983; Rosowsky and Ellingwood 1991; Liu and Bulleit 1995a, 1995b). It was shown that the ultimate lateral capacity of a subsystem is higher than that coinciding with rupture of the first member (Liu and Bulleit 1995b). Therefore, a more comprehensive approach at the system level considering multiple member failure would need to be considered.

Currently, the blast design standards (PDC-TR 06-08 2008; ASCE/SEI 59 2011; CSA S850 2012) use ductility ratios of 1, 2, 3, and 4 which correlate to response limits B1, B2, B3, and B4, respectively, for light-frame wood stud walls in flexure (see Table 1). The response limits in the blast standards correspond to superficial (less than B1), moderate (between B1 and B2), heavy (between B2 and B3), hazardous (between B3 and B4), and blowout (more than B4). These blast provisions for wood structures are based on limited data obtained primarily through live-explosive testing conducted by Marchand (2002). While no systematic approach for damage evaluation of light-frame wood structures exists, the test data available in the literature was reviewed by Oswald (2005) and an overall damage assessment was proposed based on the limited available information in form of damage photos and pressure-impulse combinations. In-situ properties of the structural members were not available and therefore published data was used to obtain the strength and stiffness of the specimens in order to conduct the assessment (Oswald 2005). Although the specific damage descriptors were not explicitly defined in Oswald's study, it is assumed that the maximum ductility ratio of 4 was chosen because this limit defined the most severe damage observed during the assessment. A "blow-out"

region is notoriously difficult to estimate, because if there is a lack of data points representing the onset of collapse or ultimate failure, the limit is likely to be overestimated.

A comprehensive research program was established at the University of Ottawa's blast laboratory with the aim to mitigate hazards associated with blast loads on light-frame wood structures. Presented in the current paper is a detailed assessment of the behaviour of thirty-three full-scale light-frame wood stud walls subjected to a total of forty-eight shots of simulated blast loading. Detailed documentation of the observed damage allowed for the development of an accurate evaluation strategy of the response limits. To assess the validity of the proposed approach, the observed response limits are compared to limits derived from single-degree-of-freedom (SDOF) modelling using scaled pressure-impulse (P-I) diagrams. The developed approach is also compared to current code performance levels for wood light-frame structures, and implications on the code are discussed.

Description of specimens and dynamic test setup

The data used to evaluate the damage levels stems from three separate experimental programs, denoted in Table 2 as test groups A, B, and C, all of which were conducted at the University of Ottawa's Blast facility. The tested walls consisted of the structural elements (i.e. studs, exterior sheathing panels, and sheathing-to-stud fasteners) only. Omitted from the construction of the specimens are non-structural components (e.g. exterior cladding, insulation, and interior sheathing) in order to isolate structural performance.

The walls were 2108 mm in width, with a clear span of 2032 mm. The wall segments tested were slightly shorter than typical light-frame walls, due to the limitation imposed by the 2032 mm x 2032 mm opening of the blast simulator. This deviation from typical stud wall sizes would have been of concern if the observed failure mode changed to be governed by, for example, shear. Since the governing failure in the tests was flexure, it is anticipated that using longer walls would yield similar behaviour.

The blast loads were simulated using a shock tube, a test apparatus capable of producing shock waves similar to those stemming from far-field detonation of high-explosives. The shock tube generates shock waves with reflected pressures and impulses of up to 100 kPa and 2200 kPa-ms, respectively. A sketch of the end-frame, located at the end of the expansion section of the shock tube, is shown in Figure 1. Depending on driver pressure and length, the positive phase of the blast wave can vary from 5 ms to 70 ms. For the purpose of this study, the range of reflected pressures consisted of 6.6 kPa to 70.9 kPa, which correlates to reflected impulses varying from 25.1 kPa-ms to 813.2 kPa-ms.

The walls were attached to the end frame of the shock tube (Figure 2a) with support details simulating idealized simply-supported boundary conditions (Figure 2b). Steel angles were attached to the shock tube frame using 6.35 mm thick 76 mm x 76 mm hollow steel sections (HSS) and allowed for free rotation at the outside face of the wall. Steel plates with rollers were fixed to each studs in order to avoid crushing of the wood at the point of contact. These rollers were in contact with a rectangular 6.35 mm thick 76 mm x 152 mm HSS which clamped the wall to the shock tube frame. While these support conditions are not representative of typical (toe-nailed) or designed boundary connections, the idealization of pin-ended boundary conditions allows for the development and verification of analytical expressions. The effects of connections on the performance of light-frame wood stud walls to blast is outside the scope of the current study. The reflected pressure for each test was measured using two dynamic piezoelectric pressure sensors, and strain gauges were installed on both the tension and compression face of the four middle studs of each wall. Coincidentally, the reflected impulse was determined by calculating the area under the pressure-time history up until the end of the positive phase. Linear variable differential transducers (LVDTs) were used to measure the mid-span stud deflections relative to a stiff steel bracket attached to the shock tube's end frame, as seen in Figure 2. A data acquisition system with a sampling rate of 100,000 samples per second and a high-speed camera, capable of recording videos at 500 frames per second, were used to collect the data output.

Complementing the dynamic tests presented in this paper are static full-scale bending tests conducted on both individual studs (Lacroix 2013; Viau 2016) and walls (Lacroix 2013). The static tests were

conducted in accordance with the ASTM D198 standard (2009). A total of 380 studs and ten walls with 38 mm x 140 mm studs were tested statically. Five walls were sheathed with 11 mm OSB and the other five with 18.5 mm plywood. While the latter is not typically used in construction, the use of 18.5 mm plywood was meant to simulate a retrofit option in an attempt to improve on the wall performance. Future studies will be dealing with the effect of varying sheathing thickness.

Results and Discussion

Evaluation of ductility ratios for light-frame wood stud walls

Static full-scale bending tests were conducted on both individual studs and walls prior to testing the walls dynamically. The predominant observed failure mode for both the studs and walls was flexural failure at the mid-span of the studs. Depending on the relative strength of the tensile fibres versus the compressive fibres, the failure mode was either initiated on the tension side, where a sudden failure took place, or in the compression zone, where crushing of the wood fibres permitted the stud to exhibit additional ductility.

The static wall test results yielded average ductility ratios of 2.0 and 1.7 for the OSB and plywood walls, respectively (Lacroix 2013). These observations are consistent with the findings from other published studies (Gromala 1983; Foschi 1985; Wheat et al. 1986; Liu and Bulleit 1995b; Bulleit et al. 2005). Figure 3 shows a representative load-displacement graph from the static testing conducted on the walls. It is clear from the graph that the ultimate average failure of this wall occurs near a ductility ratio 2. The results from the static tests as well as those obtained from the literature suggests that the assumption used in the design codes overestimates the ductility ratios for light-frame wood stud walls, and that using a maximum ductility of 2 is more appropriate and safer for blast design. This finding is crucial when ductility ratios and damage levels are being assessed for the walls tested dynamically. The code guidelines and the proposed ratios will be further evaluated analytically in a subsequent section of this paper.

Failure modes and wall behaviour under dynamic loading

Similar to static testing, the failure mode of walls subjected to simulated blast loading was predominantly flexural failure of the studs at mid-span. Compared to the failure observed in static tests (Figure 4a), the dynamic failure was dominated by a brash failure, with a noticeable “cut-through” the cross-section on studs with weak tension fibres (Figure 4b). Studs with stronger tension fibres experienced a more “fibrous” type of failure mode (Figure 4c), where the failed fibres tended to be longer and thicker than those found in weaker studs. This observation is consistent with that reported in a previous study on impact loading on clear wood specimens (Johnson 1986).

The deflections of the four middle studs can be seen to reach their peak at the same time due to the sheathing panel pushing on the studs and allowing for load distribution to occur between the studs, as seen in Figure 5. In the rebound phase, the sheathing-to-stud connection does not seem to have sufficient withdrawal capacity to maintain constant displacement and the studs no longer move in unison.

In the case of elastic shots, where the walls returned to their original position, the maximum displacement was easily identified as the actual peak displacement, while in destructive shots it was taken as the displacement corresponding to maximum strain recorded using the strain gauges. Strain- and displacement-time histories correlated well and the peaks in strains and displacements were observed to occur at approximately the same time.

Quantification of damage levels

The proposed evaluation criteria were developed based on the overall performance of the wall system and its estimated post-blast axial residual capacity. Typically, the lowest damage level corresponds to that of no observable damage on the structural elements. Due to the strength and stiffness variability between studs, a loading of relatively low magnitude may potentially cause cracking in the weakest studs. As mentioned in the introduction section, even though this is a permanent damage, it has little

to no impact on the post-blast axial capacity of the wall or the structure as a whole, and therefore, the proposed descriptor for “superficial damage” allows for limited splitting in an individual stud. Figure 6a shows an example of what the authors have characterized as “superficial” damage, where one stud experienced minor cracks on the tension face while the other studs did not experience any damage. This clearly does not meet the intent of the current damage level description in the blast design codes (ASCE/SEI 59 2011; CSA S850 2012), where a superficial damage level entails “no visible permanent damage”. Since the wall (and the “failed” stud) has no permanent deflection, the “moderate” and “heavy” damage description from Table 1 would also not apply. If the assessment was made based on the failure of a single stud (weakest member within the wall subsystem) then the description for “hazardous” damage, defined as “*Component has failed with no significant velocities*”, would apply to this wall. This was deemed, by the authors, as too conservative, and it reiterates the need for different and more appropriate descriptors specifically developed for light-frame wood stud walls. It is assumed here that the inherent variability in wood, which may lead to a stud with inferior capacity to experience some damage, should not mean that the level of damage of the entire wall is characterized for a higher damage level.

During the documentation of observed damage in the experimental phase, it was deemed very difficult to differentiate between moderate and heavy damage. For practical reasons, a merging of these two damage regions is proposed for the purpose of assessing wood stud walls. The proposed wording for the moderate-heavy range is based on observed wall failure being more appropriately defined by failure of two adjacent studs or any three studs. This is based on the assumption that overall wall failure is unlikely to occur when two adjacent studs or any three studs have completely failed in the wall system (Liu and Bulleit 1995b). Although the current study is based on the damage assessment of walls with six studs only, Liu and Bulleit (1995b) showed, through reliability analyses, that similar limit-state failure criteria could be applicable to walls with up to 32 members. Figure 6b shows a representative example of the moderate-heavy damage.

The proposed “hazardous” damage is defined by the failure of three or more adjacent studs where debris has no significant velocity. At this damage level it is assumed that the wall segment would be at the onset of structural collapse. An example is shown in Figure 6c. Finally, the blowout region is described by the structural component being overwhelmed by the load and where the debris have significant velocities. Figure 6d shows a representative observed damage level for what has been characterized as “blowout”.

The damage assessment of light-frame wood stud walls described above has been summarized in Table 3 in a format that resembles that currently found in the blast design codes (see Table 1). Table 4 shows the evaluation of all the test results based on the proposed damage level description. The table includes the test names, scaled pressures (P_{bar}), scaled impulses (I_{bar}), and assessments of the damage. The nomenclature of the test names allows for the identification of the specimen group, wall number, and shot sequence to which the specimen was exposed. For example, *B3-2* refers to group B, was the third wall of that group, and was exposed to a second combination of pressure and impulse. These assessments are based on the criteria proposed in Table 3 and in line with damage levels presented in Figure 6. For example, the initial damage of wall *B3-2* is defined by the damage assessment of wall *B3-1*. Detailed damage descriptions for all forty-eight tests can be found in Lacroix and Doudak (2012), Viau (2016), and Lacroix (2013).

It is noteworthy to mention that gypsum wallboard (GWB) has been intentionally omitted from this study. It is recognized that adding GWB to the tested walls would have an effect on the walls' behaviour as well as the damage level description. It is conceivable that flying debris from GWB panels may be generated when subjected to a blast load which correlates to moderate-to-heavy damage. On the other hand, adding GWB on the tension side of the studs may have a reinforcing effects and may help reduce the overall displacement and thereby damage of the wall studs.

Since this study constitutes the first step in the comprehensive evaluation of damage levels in light-frame wood stud walls, it was decided that the focus should be on the main structural elements only. Future studies will address the effect of non-structural elements, including cladding, insulation, and GWB panels, or other interior finishes, on the dynamic behaviour of the stud wall systems.

Implications of design and retrofitting were briefly investigated through the use of WWM. As per Table 3, the “blowout” damage level can be reached if the greater majority of structural components are overwhelmed by the blast load, or if failure of the sheathing creates debris with significant velocities. The latter damage criteria would tend to govern in the assessment of the damage level of walls with typical sheathing (i.e. 7/16 in. OSB), which has shown to fail prior to stud response and produce significant amounts of sheathing debris (Viau 2016). The addition of WWM has shown to adequately reinforce the sheathing, as observed in test C1-1 (see Table 4), and thereby reducing the damage level to hazardous.

In order to evaluate whether the proposed damage description and ductility ratios are consistent with the applied pressure and impulse imparted on the walls, analysis using SDOF is used as described in the next section.

Validation of observed damage

Scaled Pressure-Impulse Diagrams

A common method to describe the performance characteristics for a range of blast loading for a specific type of element and failure mode is through the use of pressure-impulse (P-I) diagrams. The different curves in a P-I diagram indicate different damage levels or levels of protection (i.e. iso-damage curves). These can be obtained using a SDOF analysis method, from experimental results, or from actual blast events. While useful, its main drawback is that if not normalized, P-I diagrams cannot be generalized and can only be used to assess the behaviour of structural elements of identical properties, geometries, loading and end conditions. To expand the applicability of P-I diagrams to a broader range of specimens, normalized pressure-impulse combinations can be used to directly compare results of similar specimens with different geometries and properties. The specimen stiffness, resistance, mass, and loaded-area are all taken into account and each data point is normalized by removing the direct effects of each of these properties. Curve fitting factors that take into account the effects of the negative phase of the blast wave are also available in the literature (Oswald 2005; PDC-TR 06-01 2008).

Scaled P-I diagrams contain non-dimensional scaled peak blast pressure (P_{bar}) and scaled positive blast impulse (I_{bar}) terms, which are obtained by dividing the pressure and impulse by the component properties using the conservation of energy principle (Oswald 2005; PDC-TR 06-01 2008; Krauthammer et al. 2008; Dragos and Wu 2013). Rather than describing a deflection limit, the scaled P-I diagram reflects a non-dimensional response criterion, which is often described as a ductility ratio or a support rotation. For wood, the common approach to relate the response limits to damage levels is through the use of the ductility ratio, μ , which consists of the ultimate deflection, x_{max} , divided by the elastic deflection of the system, x_e , as shown in Equation (1).

$$(1) \quad \mu = \frac{x_{\text{max}}}{x_e}$$

The difference between the static deformed shape and the first mode approximations is insignificant for blast loading, thus allowing the use of the static shape, which is more convenient (Department of Defense 2008). Most importantly, the time-scale between the real and equivalent systems is not altered; therefore, at any instance during the displacement-time history, the displacement of the equivalent system is equal to that of the real structure at the equivalent ordinate. Equations (2) and (3), reproduced from Oswald (2005), describe the non-dimensional response criterion of ductility ratio for a component with an elastic, perfectly plastic flexural response.

$$(2) \quad P_{\text{bar}} = \frac{P_R}{R_u}$$

$$(3) \quad I_{\text{bar}} = \frac{I_R}{R_u} \sqrt{\frac{K}{K_{\text{LM}}m}}$$

Where P_R is the reflected peak positive pressure, R_u is the ultimate flexural resistance of the component obtained from static testing and modified for strain rate effects, I_R is the reflected impulse associated with the positive phase, K is the stiffness of the system, K_{LM} is the load-mass factor, and m is the mass of the system. Based on the findings of a wall behaving in an approximate elastic-perfectly plastic flexural response, equations (2) and (3) can therefore be used to scale the data points of the forty-eight tests.

By scaling the pressure and impulse combinations, it is possible to include walls with different properties and geometries into the same P-I diagram, as shown in Figure 7. Figure 7 clearly shows the clustering of points identified with each damage level. Iso-damage curves could be developed to distinguish between these groups and each iso-damage curve would correspond to a well-defined damage level.

Implications on blast design codes

Figure 8 shows the response limits based on the current provisions of the Canadian blast design standard (CSA S850 2012). It is clearly shown that data points characterized as “superficial” and “moderate-heavy” fit reasonably well within the regions proposed by the code, however, damage regions for “hazardous” and “blowout” are non-conservative. The need to reduce the ductility ratio corresponding to response limit B3 and B4 is apparent when comparing the fit between the data points and the respective response limits.

Based on the observed damage levels obtained from the experimental study, the authors propose the use of only four damage regions, separated by three response limits, denoted B1, B2, and B3, corresponding to ductility ratios of 1, 1.5, and 2, respectively (see Table 3). Figure 9 shows the same data as in Figure 8 but with the proposed damage regions. It can be observed that test walls characterized as “blowout” fit well within the proposed blowout region, located to the right of the iso-curve corresponding to response limit B3 (i.e. $\mu = 2$). The damage region corresponding to “hazardous” can be seen to be relatively small, because it is often difficult to attain flexural failure of several load-bearing elements without any hazardous debris. This observation may imply that for all practical purposes and given the uncertainty associated with defining damage in the hazardous region, only three regions, namely superficial, moderate/heavy/hazardous, and blowout, may be adequate to describe the behaviour of light-frame wood stud walls. More data points are needed in the hazardous region before such recommendations can be made. The “superficial” region has not been changed, since it is based on the elastic limits.

In general, it can be concluded that the proposed damage assessment method and the proposed ductility ratios seem appropriate when compared with observations of actual damage and measured pressure and impulse combinations from the experimental program.

Conclusions

New damage assessment methodologies and ductility ratios, reflective of the behaviour of wood light-frame wood stud walls subjected to blast loads, are proposed. The proposed response limits were based on a series of experimental investigations and include observed damage using system level failure criteria. Scaled P-I diagrams were effectively used to demonstrate the proposed damage level descriptions and ductility ratios using walls from different tests. Simply-supported end conditions were utilized throughout the testing in order to investigate the behaviour of the wall in isolation.

Damage descriptions and associated response limits, suitable for damage observed in light-frame wood stud walls were proposed for “superficial”, “moderate-heavy”, “hazardous”, and “blowout” regions. The study also showed that current blast design code response limits overestimate the ductility found in light-frame wood stud wall systems, especially for the hazardous and blow-out regions. New ductility limits of 1, 1.5 and 2 are proposed and found to better describe the damage regions.

The effect of end fixities and other non-structural components as well as axial loads are currently under investigation by the authors and are outside the scope of the current paper.

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Table 1: Component damage versus response limits

Component damage level	Component damage level descriptions	Response limits
Blowout	Component is overwhelmed by blast loading causing debris with significant velocities	Response greater than B4
Hazardous failure	Component has failed with no significant velocities	Response between B3 and B4
Heavy damage	Component has not failed, but it has significant permanent deflections causing it to be unreparable	Response between B2 and B3
Moderate damage	Component has some permanent deflection. It is generally repairable, if necessary, although replacement may be more economical and aesthetic	Response between B1 and B2
Superficial damage	Component has no visible permanent damage	Response less than B1

*Reproduced from CSA S850 2012

Table 2: Description of test specimens

Specimens	Studs (mm x mm)	Sheathing Configuration				
		Type(s)	Grade(s)	Thickness(es) (mm)	Fasteners	Nailing pattern
A1-A5	38 x 89, SPF ¹ No. 2	OSB ³	24/16	11	3.25 mm x 64 mm nails	150 mm (edge), 300 mm (field)
B1-B5	38 x 140, SPF MSR ² -1,450fb	OSB	24/16	11	3.50 mm x 64 mm nails	150 mm (field & edge)
B6-B10	38 x 140, SPF MSR-1,450fb	Plywood	SHG ⁴	18.5	4.24 mm x 89 mm nails	150 mm (field & edge)
B11-B19	38 x 140, SPF No. 2	OSB	24/16	11	2.87 mm x 50 mm nails	150 mm (field & edge)
B20-B23	38 x 140, SPF No. 2	Plywood	SHG	18.5	2.87 mm x 50 mm nails	150 mm (field & edge)
B24-B26	38 x 140, SPF No. 2	OSB, Plywood	24/16, SHG	11, 18.5	2.87 mm x 50 mm nails/ 4.2 mm x 76 mm screws	150 mm (field & edge)
C1-C2	38 x 184, SPF No. 2	OSB w/ WWM ⁴	24/16	11	2.87 mm x 50 mm nails	150 mm (field & edge)

¹Spruce-Pine-Fir²Machine stress rated³Oriented strandboard⁴Sheathing grade⁵Welded wire mesh – 152 mm x 152 mm x 11.1 mm²

Table 3: Proposed damage level criteria for light-frame wood stud walls

Component damage level	Proposed component damage level descriptions	Response limits
Blowout	-Sheathing overwhelmed, and sheathing debris detached from the structure with significant velocities. Studs may or may not be damaged -Majority of studs failed. Complete loss of axial capacity. Debris with significant velocities created.	Response greater than B3
Hazardous	-Three or more adjacent stud failure with cracks extending to 50% or more of the stud depth. Debris is limited to small sized pieces of sheathing and/or stud fragments that have detached from the structural system with no significant velocities.	Response between B2 and B3
Moderate-heavy	-No more than two adjacent or three non-adjacent studs with cracks extending to 50% or more of the stud depth.	Response between B1 and B2
Superficial	-No more than one stud with cracks extending to 50% of the stud depth. No debris.	Response less than B1

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Table 4: Normalized dynamic test results

Test name	P_{bar}	I_{bar}	Damage level	Test name	P_{bar}	I_{bar}	Damage level
A1-1	0.70	1.03	Superficial	B9-1	0.24	0.92	Superficial
A2-1	1.13	1.46	Moderate-Heavy	B9-2	0.92	3.67	Moderate-Heavy
A3-1	0.63	0.90	Superficial	B10-1	0.29	1.12	Superficial
A3-2	0.68	1.00	Superficial	B10-2	0.71	2.11	Superficial
A4-1	0.61	0.76	Superficial	B10-3	0.89	3.25	Moderate-Heavy
A4-2	1.20	1.53	Moderate-Heavy	B11-1	0.40	1.46	Superficial
A5-1	0.90	1.51	Superficial	B11-2	1.57	4.63	Blowout
B1-1	N/A*	N/A*	Superficial	B12-1	1.35	4.02	Blowout
B1-2	0.74	3.00	Moderate-Heavy	B13-1	1.46	4.24	Blowout
B2-1	0.88	3.12	Moderate-Heavy	B14-1	1.63	4.62	Blowout
B3-1	0.25	0.20	Superficial	B15-1	1.28	4.39	Blowout
B3-2	1.32	1.38	Moderate-Heavy	B16-1	1.08	3.87	Blowout
B4-1	0.15	0.57	Superficial	B17-1	1.14	3.88	Blowout
B4-2	0.65	2.10	Superficial	B18-1	1.17	4.13	Blowout
B4-3	0.95	3.13	Moderate-Heavy	B19-1	1.13	5.56	Blowout
B5-1	0.25	1.45	Superficial	B20-1	1.34	4.56	Blowout
B5-2	0.66	3.76	Superficial	B21-1	1.32	4.65	Blowout
B6-1	0.25	1.04	Superficial	B22-1	1.34	4.61	Blowout
B6-2	0.72	2.92	Superficial	B23-1	1.39	4.94	Blowout
B6-3	0.75	3.32	Moderate-Heavy	B24-1	1.33	4.16	Blowout
B7-1	0.25	1.76	Superficial	B25-1	1.75	5.39	Blowout
B7-2	0.79	6.33	Moderate-Heavy	B26-1	1.90	5.65	Blowout
B8-1	0.27	0.23	Superficial	C1-1	1.02	4.20	Hazardous
B8-2	1.54	1.60	Moderate-Heavy	C2-1	1.26	4.97	Blowout

* Data acquisition malfunctioned during testing

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(a)

(b)

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(a)

(b)

(a)

(b)

(c)

(a)

(b)

(c)

(d)

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Table A.1: Dynamic test results

Test name	P _r (kPa)	I _r (kPa-ms)	K (kN/m)	R _u (kN)	m (kg)	Test name	P _r (kPa)	I _r (kPa-ms)	K (kN/m)	R _u (kN)	m (kg)
A1-1	8.0	62.7	226	9.5	9.7	B9-1	11.0	122.2	1175	37.8	12.8
A2-1	13.0	89.5	226	9.5	9.7	B9-2	42.1	450.4	1175	37.8	12.8
A3-1	7.2	55.3	226	9.5	9.7	B10-1	12.5	160.3	925	36.0	12.8
A3-2	7.8	61.3	226	9.5	9.7	B10-2	31.0	302.0	925	36.0	12.8
A4-1	7.0	46.4	226	9.5	9.7	B10-3	38.7	427.9	925	36.0	12.8
A4-2	13.8	93.8	226	9.5	9.7	B11-1	13.7	140.0	1084	28.5	10.7
A5-1	13.7	100.9	329	12.5	9.7	B11-2	54.2	408.1	1084	28.5	10.7
B1-1	N/A*	N/A*	1125	42.5	11.6	B12-1	46.5	354.5	1084	28.5	10.7
B1-2	38.3	403.0	1125	42.5	11.6	B13-1	50.5	373.5	1084	28.5	10.7
B2-1	38.2	384.9	940	35.7	11.6	B14-1	56.4	407.0	1084	28.5	10.7
B3-1	9.8	25.1	875	33.0	11.6	B15-1	44.3	387.3	1084	28.5	10.7
B3-2	52.6	163.0	875	33.0	11.6	B16-1	37.3	341.5	1084	28.5	10.7
B4-1	6.6	76.1	980	36.5	11.6	B17-1	39.5	342.2	1084	28.5	10.7
B4-2	28.9	282.3	980	36.5	11.6	B18-1	40.3	364.4	1084	28.5	10.7
B4-3	41.9	386.5	980	36.5	11.6	B19-1	39.1	377.4	1830	28.5	10.7
B5-1	11.6	207.0	985	39.0	11.6	B20-1	48.1	429.0	1145	29.7	11.9
B5-2	31.3	495.1	985	39.0	11.6	B21-1	47.3	437.0	1145	29.7	11.9
B6-1	11.5	128.1	1350	37.5	12.8	B22-1	48.3	434.0	1145	29.7	11.9
B6-2	32.6	331.5	1350	37.5	12.8	B23-1	49.8	465.0	1145	29.7	11.9
B6-3	34.1	377.6	1350	37.5	12.8	B24-1	49.7	481.0	1200	30.8	17.5
B7-1	12.6	245.7	1325	42.0	12.8	B25-1	65.1	623.0	1200	30.8	17.5
B7-2	40.1	813.2	1325	42.0	12.8	B26-1	70.9	654.0	1200	30.8	17.5
B8-1	9.5	27.3	925	29.5	12.8	C1-1	51.2	483.0	1881	41.5	15.0
B8-2	55.0	172.6	925	29.5	12.8	C2-1	63.4	572.0	1881	41.5	15.0

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* Data acquisition malfunctioned during testing