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Modelling the Behaviour of Light-Frame Wood Stud Walls Subjected to Blast Loading

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Abstract: This paper presents analysis using a non-linear material predictive model considering partial composite action for wood light-frame stud walls subjected to blast loading. The proposed model was verified against data from an experimental program, where walls were tested statically and dynamically. The validity of the current provisions of the Canadian blast standard is evaluated and discussed. The modeling results show a tendency for increased capacity and stiffness under high strain rates. The results also demonstrate that neglecting partial composite action yields predictive capacity values that are too conservative.

Keywords: High strain rate, blast loading, flexural response, wood light-frame, static capacity, dynamic capacity, shock tube testing, partial composite action, dynamic increase factor, SDOF modelling.

1 Introduction

Modelling the response of a real structure requires special attention to the details such as the representation of the boundary conditions and the type of elements used to represent the structural components under consideration. Although real structures have theoretically an infinite number of modes of deformation, capturing such behaviour generally requires significant computational effort. Consequently, it is sometime possible to provide a reasonable approximation of structural response based on a single dominant deformation mode. A commonly used approach is equivalent single-degree-of-freedom (SDOF) systems in which member properties, such as the mass and stiffness, and external loading are converted into an equivalent SDOF using deformed shape factors (Biggs 1964). SDOF models can take strain rate effects and ductility into account, but the influence of modes other than the one assumed is neglected. All three aspects can be incorporated in finite element simulations, but at the cost of generality and comprehensibility. Refined SDOF techniques, even those which considers complex response modes, are available in the literature (e.g. Nebuda and Oswald 2004, Oswald 2005). Most often the best parameter inputs to the models are determined from static experiments

Studies on the behaviour of light-frame structures under high strain rates has been mostly limited to research focusing on small clear specimens, free of defects (e.g. Wood 1951, Mindess and Madsen 1986, Jansson 1992). Limited full scale tests (e.g. Lloyd et al. 2011, Kimbell and Fies 1953, Randall 1955, Marchand 2002) have shown that under high pressures, an increase in the capacity under dynamic loading is observed. Moreover, it has been shown that the performance of the wood frame structures was greatly dominated by the performance of the rafters, joists, and studs in flexure rather than the connections between them, thus allowing the focus on a single dominant response mode.

This paper discusses the non-linear material predictive model developed for light-frame wood stud walls under high strain rates using a SDOF approach. The constitutive model includes relevant numerical implementation methods of material behaviour such as strain rate dependency. Emphasis is placed on evaluating and assessing the effect of the composite action between the sheathing and the studs. Different approaches are investigated varying from considering the capacity of a single stud modified to include partial composite action between the sheathing and the studs to models using static capacity of full scale stud walls as input. The model is validated with experimental data, and implications of the outcome on the design code are discussed.

2 Single-Degree-of-Freedom (SDOF) Analysis

In a SDOF analysis, the member properties such as mass, stiffness and the external loading are converted into an equivalent SDOF system by using deformed shape factors that relate to the motion of a single point of interest, generally where maximum deflection occurs (Biggs 1964). In order to idealize the system into an equivalent SDOF, the displaced shape function must be determined to obtain the appropriate mass factor (k_M) and load factor (k_L). The transformation factors are obtained by equating the work done, kinetic energy and strain energy, of the real structural system based on the assumed static deflected shape to the corresponding relationships of the equivalent system. The dynamic equation for the equivalent system is then obtained by applying the mass and load factors to their respective terms. The dynamic response of the SDOF system was obtained by solving the equation of motion of the equivalent system, shown in Equation 1, where each term was divided by the load factor, thus yielding the load-mass factor (k_{LM}).

[1] $\mathrm{k_{LM}m\ddot{y}(t) + Ky(t) = F(t)}$

Where m is the mass, K is the stiffness of the system, F(t) is the reflected pressure as a function of time, multiplied by the loaded area, $\ddot{y}(t)$ and y(t) are the acceleration and displacement of the equivalent ordinate taken at mid-span, respectively. Damping is omitted from the analysis as it has negligible contribution to the equation of motion under very short duration of loads and during the first cycle up to maximum displacement (TM 5-1300 1990, Biggs 1964). Typical curves of idealized pressure-time history and impulse used in the SDOF analysis are shown in Figure 1.

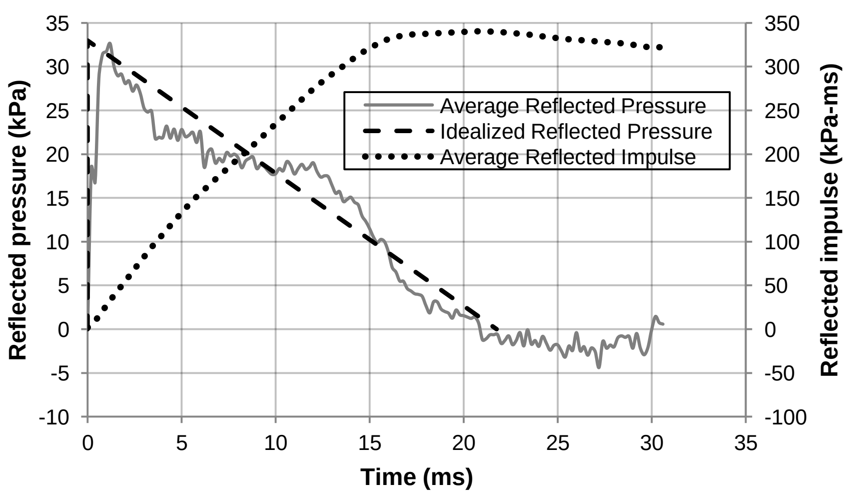


Figure 1: Typical reflected pressure and impulse time history

The displacement-time history of the equivalent system and real system is not altered. Lumping the applied load as a point load and the stiffness as an inertia force acting on the equivalent mass enables the displacement of the equivalent system at any instance in time to be equal to the one of the real system.

For simply supported end conditions and a uniformly distributed load, the resistance, R_e, and stiffness, K_e, are determined as shown in Equations 2 and 3, respectively. For the uniformly distributed load with simply supported end conditions the ultimate resistance, R_u, is equal to the resistance at yield, R_e.

[2] $R_e = R_u = \frac{8S_D}{L^2}$

[3] $K_e = \frac{384EI}{5L^3}$

Where S_D is the dynamic bending strength, L is the clear span, E is the modulus of elasticity (MOE), and I is the moment of inertia of the system. The idealized resistance curve for simply supported end conditions and uniformly distributed load assumes a linear elastic behaviour until a maximum yield resistance, which is maintained until failure. The stiffness of the structural component is determined with respect to a unit displacement in the direction of the chosen degree of freedom due to the actual applied load. The forcing function is determined for every combination of pressure and impulse as the reflected pressure-time history multiplied by the loaded area. The load-mass factor for simply supported end conditions is equal to 0.78.

2.1 SDOF Material Predictive Model

Several studies (e.g. Polensek et al. 1972, Foshi 1985, McCutcheon 1986, and Wheat et al. 1986) have shown that neglecting the contribution of the sheathing in a wall system could yield deflection values that are too conservative. In order to account for the partial composite action in the wall system, a method accounting for the interlayer slip between the sheathing and the stud was used in this study (McCutcheon 1986). The approach incorporates the load-slip relationship as a function of the fastener diameter and spacing, sheathing type and thickness, and the elastic bearing constants (Wilkinson 1972 and 1974). The interlayer stiffness is subsequently used to modify the bending stiffness of the wall system. The material model proposed here is developed based on the behaviour of a T-section, consisting of a stud and a segment of the sheathing width equivalent to its stud spacing (406 mm).

In order to verify the modelling approach, a comparison was made with experimental results based on tests conducted at the University of Ottawa’s Structural Laboratory. Twenty walls consisting of 38 mm x 140 mm machine-stress-rated (MSR) lumber, spaced 406 mm on center and with a single top and bottom plates, were tested to failure under static and dynamic loading. From the static tests, properties such as the modulus of elasticity (MOE), modulus of rupture (MOR) were derived and used as input in the models. Tests were conducted on individual studs as well as full scale walls, all under similar loading and boundary conditions.

The dynamic testing was done using the Shock Tube facility at the Blast Research Laboratory at the University of Ottawa, where blast loading can be simulated to induce high strain rates in the specimens. The specimens were subjected to various combinations of pressure and impulse in the impulsive, dynamic and quasi-static regions. More details on the experimental program are reported in a companion paper in this proceeding.

Figure 2 shows a representative resistance-deflection relationship based on the static tests of the wall system including studs and sheathing. The curves for each of the four middle studs are shown, together with the average force-deflection curve. The static resistance of the T-section using partial composite action was found using Equation 4 with the average MOR found from stud static testing along with the section modulus resulting from partial composite action.

[4] $R_e = R_u = \frac{6*MOR*S}{L}$

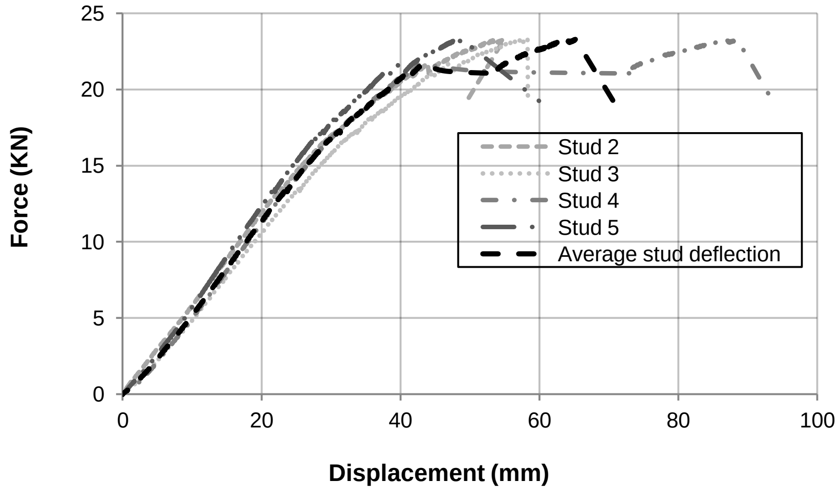


Figure 2: Experimental static resistance curve

The resulting dynamic resistance curve for an OSB T-section is shown in Figure 3. The dynamic peak load was not measured experimentally and therefore a direct calculation of the dynamic MOR and MOE is not possible. An indirect iterative method, using the SDOF procedure was employed to determine the dynamic strength and stiffness increase of the specimens at high strain-rates. The mass of the T-section used as an input in Equation 1 for OSB and plywood were 11.86 and 13.38 kg, respectively, which comprises of the mass of the sheathing and stud within the clear span.

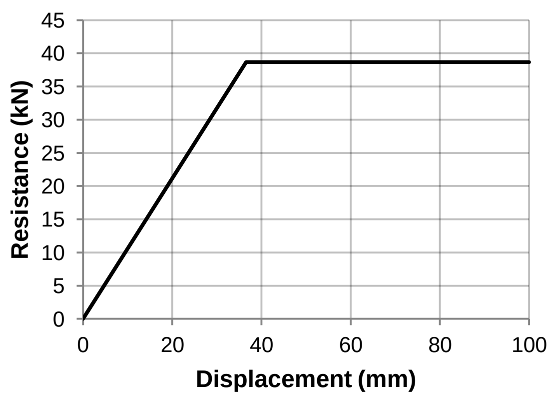


Figure 3: Dynamic resistance curve-partial composite method

2.2 Deriving the Inputs Parameters Based on the Canadian Blast Standard Provisions

In order to evaluate the level of safety and conservatism of the code provisions for wood structures, a material model, similar in principle to the one described earlier, but with all the assumptions and simplifications in the code provisions, was used. Of course, in reality, the designer may not have access to static data on individual studs or wall systems, thus the model includes only published strength and stiffness data. The recently developed Canadian blast standard "Design and assessment of buildings subjected to blast loads" (CSA S850 2012) provides guidance on how to obtain the dynamic design strength. Although not specified in the design standard, it will be assumed here that the weight of the system is that pertaining to the stud and the sheathing. Contrary to the previous approach, only the mass of the sheathing is considered to contribute whereas the stiffness of the sheathing is considered negligible. The dynamic bending resistance, S_D, is determined based on the CSA O86 provisions modified for high strain rate effects, as shown in Equation 5.

[5] $S_D = \mathrm{SIF \; x \; DIF \; x} \; S_S$

Where SIF is the strength increase factor, DIF is the dynamic increase factor which reflects the increase in capacity due to the high strain rate the member is experiencing during the blast loading. According to the CSA S850, the DIF for MSR lumber is 1.4 and the SIF is 1.5. The static member strength is determined according to Equation 6 (CSA O86 2009) where the load duration factor, K_D, and the material resistance factor, ϕ, are taken as unity as specified in appendix to the CSA S850.

[6] $S_S = \emptyset[f_b(K_D K_H K_{Sb} K_T)] S K_{Zb} K_L$

The service condition for bending at extreme fiber, K_{Sb}, treatment factor, K_T, size factor, K_{Zb}, and lateral stability factor, K_L, are determined according to the CSA O86 provisions and are all equal to unity for this experimental program. The bending strength, f_b, of the stud element is based on published strength data (CSA O86 2009) and the section modulus, S, is that of the stud only. The assumption made for calculating the section modulus may vary depending on the designer; here, the approach is the most conservative one, and is believed by the authors to also be the most common. Figure 4 shows the resulting dynamic resistance curve for a T-section along with a system effect factor, K_H, of 1.2 using the code approach.

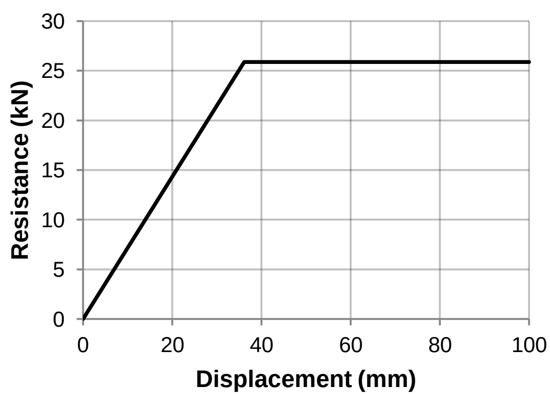


Figure 4: Resistance of T-section using the code approach

3 Analytical Results

SDOF analyses were performed and the results were compared with all the experimental data. A representative displacement-time history curves for two different response regions, one at very low load level within the elastic region and the other near failure are shown in Figure 5 and 6, respectively. Figures 5 and 6 show the displacement-time history records for the middle 4 studs of the wall. It can be seen that in general, the first maximum peak displacement of the individual wall studs was reached at approximately the same time. However, and as illustrated in Figure 6, the succeeding maximum peaks were often out of phase. This observation may have some implication whether the assumption of “no load sharing” (i.e. K_H=1.0) is appropriate.

The iterative modelling procedure showed that wood experiences an increase in both ultimate capacity and stiffness under high strain rates. Since no increase in stiffness is allowed in the CSA S850 standard, it is observed that the maximum predicted displacement is reached later than those measured experimentally or predicted using the partial composite action material model (Figures 5 and 6). It is also clear that using the CSA S850 approach for MSR lumber is conservative. The difference in the predicted and measured displacement at lower load levels (Figure 5) is not significant because the member capacity is not yet reached. The difference is much more pronounced at higher load levels (Figure 6), where the predicted deflection using the design code approach is significantly higher than that observed from the experimental tests. The observed conservatism is attributed to several factors, including values for SIF and DIF. The SIF is based on a conservative assessment derived from in-situ relative to design strength values. Therefore, it is expected that a sample of tests specimens may yield higher capacity than that proposed by the code. The ratio between the average measured stud capacity for the samples used in the current testing program relative to the design values for the same stud grade is 2.2 compared to 1.5 provided by the CSA S850 standard. The authors are not suggesting modifying this value as it is well know that wood has large variability and another test sample might have shown a ratio closer to the 1.5 provided by the code.

It is clearly demonstrated in Figures 5 and 6 that using a material model based on the capacity of the individual studs modified for added stiffness and strength from the sheathing panels show satisfactory results when compared with the experimental test data. The predicted displacement and time to first peak deflection match those from the experimental data very well.

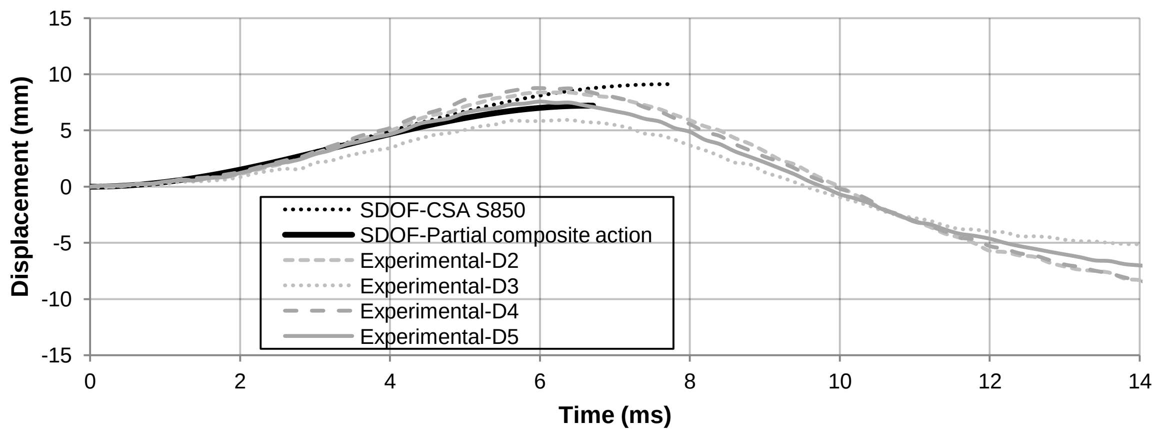


Figure 5: SDOF models comparison for a typical OSB wall

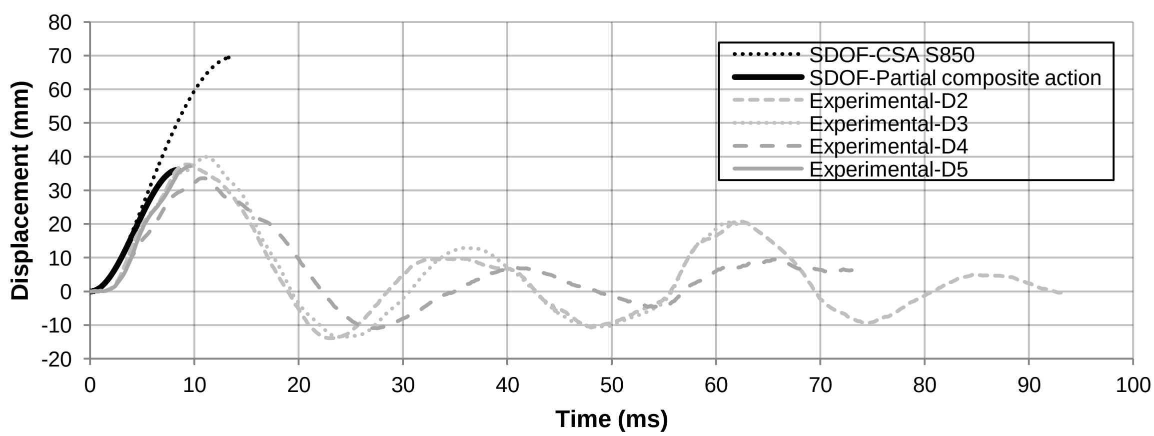
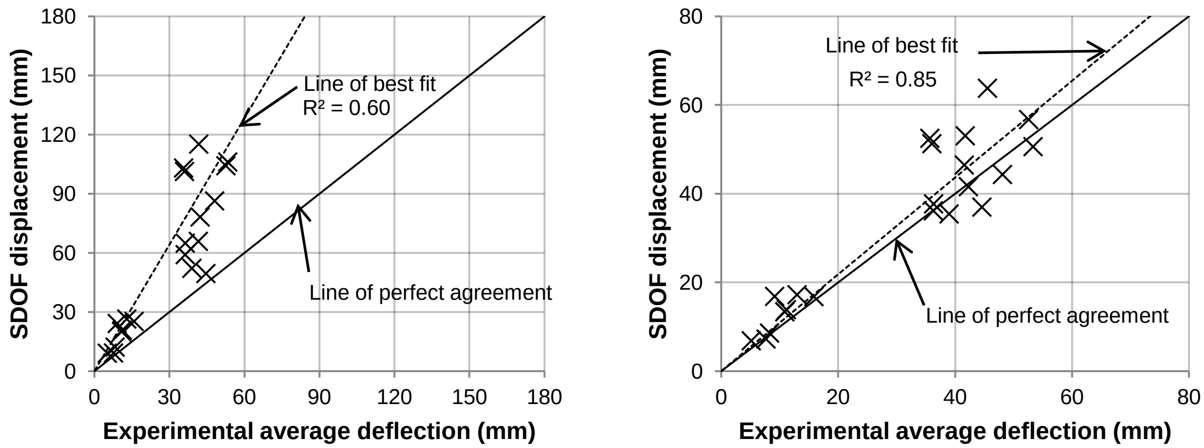


Figure 6: SDOF models comparison for a typical plywood wall

Figure 7 shows the two models' ability to predict the behaviour for all the experimental test data. Figure 7 (a) shows the comparison between the model results using the CSA S850 approach and Figure 7 (b) shows the model with stud capacity modified with partial composite action. Again, it is clear that neglecting the composite action between the sheathing and the stud might yields results that are too conservative especially near or at the failure level.



(a) CSA S850 code approach

(b) Partial composite action

Figure 7: SDOF and experimental displacements comparison

As mentioned earlier, experimental data from static tests was collected for both stud elements in isolation as well as walls consisting of studs from the same sample lot. Since studs (e.g. 2"x6") are not likely to be constructed in isolation, the modeling approach described thus far has been focussed on a material model that can represent the behaviour of the entire wall but uses individual stud capacity and stiffness as input. This approach, if successful and repeatable, would eliminate the need for testing full scale stud walls and rather rely on stud tests alone. In order to evaluate the current approach, static test data of full scale walls were used directly as input in the SDOF model. The values obtained from the static test include all system parameters except the DIF. Clearly, no additional modifications need to be made to account for the composite action between the stud and sheathing, nor is there a need to incorporate any "system" behaviour because identical systems were tested under static and dynamic loading.

Figure 8 shows comparison between the calculated dynamic displacements based on stud capacity, modified for partial composite action and that based on full scale wall tests. The close correlation between the two approaches is a testament to the validity of the proposed approach (i.e. that based on the partial composite action). There seems to be a minor tendency towards under predicting the displacement using the proposed model. The error may be attributed to some of the assumptions made in calculating the nail slip and the elastic bearing constants. Additional material tests are underway aiming to improve the material model.

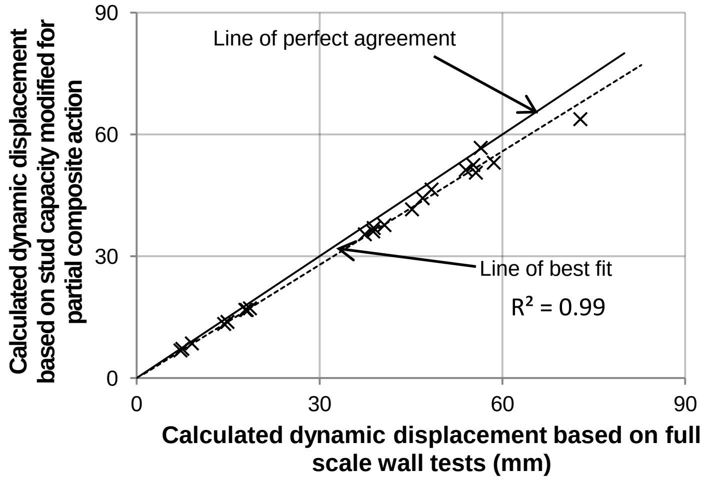


Figure 8: SDOF method comparison between partial composite approach and resistance curves from static testing

4 Conclusion

Behaviour of light-frame wood stud walls can effectively be captured using a SDOF approach that uses a non-linear material predictive model under high strain rates, such as blast loading. The proposed model considering partial composite action between the sheathing and the stud showed that there is an increase in the resistance as well as stiffness under high strain rates. Both the predicted displacement and time to first peak deflection were captured well by the model. It was also shown that the CSA S850 assumptions are too conservative for MSR lumber based on the sample lot tested. Comparison between the calculated dynamic displacements based on stud capacity, modified for partial composite action and that based on full scale wall tests showed close correlation between the two approaches. More work is underway to further improve the proposed material model.

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