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Modelling Force Flows in a Wood Light-Frame Buildings

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ABSTRACT: Questions associated with analysis and design of wood light-frame buildings are addressed with emphasis on three-dimensional system analysis. Specifically, concepts for very detailed finite element modelling are presented along with verification based on full system and substructure experiments. Such detailed modelling is not intended for application in normal design practice but is a necessary precursor to framing decisions about design level practice and development of associated provisions in timber design codes. Ongoing activities are applying what is discussed within the context of the Canadian code system and potentially others.

1 BACKGROUND

Many authors have presented analytical approaches for analysis of responses of wood light-frame structures by numerical methods (e.g. Foschi 1977; Itani and Cheung 1984; Falk and Itani 1989; Dolan 1989; Paevere 2002; Mi 2004; van der Lind 2004; van de Lindt and Rosowsky 2005; Kasal et al. 2005; Doudak et al. 2006; and Asiz et al. 2009). Methods have varied greatly in scope ranging from very simple to very complex multiple-degree-of-freedom representations of systems. In most instances complexity of models has increased over time to reflect growth in knowledge, parallel increases in computing power and metaphorical explosion in capabilities of software tools. The underpinning goal and concept was that ideal numerical models are those able to predict load-displacement responses and force flows at any level of resolution within physical systems. From a research perspective microscopic precision in analytical methods is undeniably desirable, but needs associated with normal design practices are normally different. What is discussed here relates to answering question about applicable design level analysis using sophisticated research models as vehicles for that. The context is contemporary Load and Resistance Factory Design (LRDF) design practices. Although not yet fully pervasive LRFD has become the normal worldwide for engineering design of wood structures (Breyer et al 2006; Larsen and Enjily 2009).

Wood light-frame buildings are always low-rise constructions and are designed based on equivalent static load force analysis practices, with forces calculated based on the assumption of a linear-elastic structural response (ASCE 2010; IRC 2010). Component (e.g. framing members, sheathing panels, fastenings) or substructure (e.g. wall segments, stressed skin panels) design strengths are based on apparent capacities that can reflect either or both responses in elastic and in-elastic ranges (ASCE 1996; CSA 2010). Thus, design practices only approximately represent how loads occur during, for example, hurricanes or earthquakes, and how components respond to internal force flows within building superstructures. Accepted design practices have evolved for all types of construction systems (i.e. not just wood light-frame buildings) over many decades, and in some instances centuries. Resulting design practices meld discrepancies on both sides of component level design equations and structural analysis practices in ways judged to achieve suitable solutions (i.e. solutions that equitably balance considerations of structural safety, economy of construction and avoidance of unnecessarily complex engineering practices. The “Engineering Guide for Wood Frame Construction” is an example of documents that en-

capsulate best contemporary design practices (CWC 2004).
Accepted engineering design approaches have served well for the design of relatively simple buildings of types for which there is much empirical experience in support of professional judgement of what constitute safe and serviceable design solutions. However, that warrants no guarantee that accepted practices will result in safe and serviceable designs in new situations. It follows therefore that design practices for new and evolving situations involving wood light-frame construction methods must be rationally evaluated based on more than professional judgements.

A number of large research projects have been completed around the world aimed at assessing how modern small and relatively large wood light-frame systems behave via full-scale tests and analytical modelling approaches (e.g. Paevere 2002; Lam et. al. 2002, 2004; Doudak 2005; Kasal et. al. 2005; Doudak et. al. 2006; Pei et. al. 2010). Such activities continue. This paper puts emphasis on approaches being developed within the context of the Canadian LRFD based code system.

Approaches the authors and colleagues are developing are centred on recognizing that at core choosing design level analytical methods for light-frame buildings is no different from analytical decisions made in connection with practical application of fracture- and damage-mechanics, or other, engineering theories. The responses of physical systems to applied forces whatever the scale of the objects reflects how their substructure parts behave in harmonically averaged ways. Element level design of parts of buildings is equivalent to analyzing the substructure of objects, neglecting that their individual behaviours do not sum to the homogenized behaviour. As is well known by experts who study materials at the microscopic scale and relate that to behaviour of objects at meso and macro scales, homogenized responses at high order scales are typically not predicted well by models that seek to penetrate too far into the micro scale (Herrmann and Roux 1990; Davids et. al. 2003; Smith et. al. 2003, 2007). In the vernacular of damage- and fracture-mechanics, the goal of system level modelling is to determine the scale of substructure representation that best characterizes the homogenization of smaller scale behaviours; rather than to seek to mimic real small scale behaviours. The best known implementation of such matching of the analysis method to material/substructural characterization is Linear Elastic Fracture Mechanics (LEFM). As is widely implemented in engineering design practice, LEFM enables simple but exact calculation of capacities of cracked bodies using representation of materials that only approximately mimics real behaviour of material around crack tips. Robustness of LEFM predictions depends on contextualized restriction of the scope of application. Transformed to analysis underpinning normal engineering design of wood light-frame (or any other) buildings, the need is to determine what resolution in the representation of the substructure embodied in wall segments, floors plates, roof trusses, connections and other elements best suits determination of force flows in completed structural systems. Answers to this question must be conditional and must depend on factors like the classification of the loading event(s) to be modelled, the structural arrangement(s) involved and required precision. It is necessary to contemporaneously be consistent with design information and practices embedded in or implied by applicable loading codes and material/timber design codes.

Experimental evidence and field observations of failed structures supports the expectation that, just like other complex objects, wood light-frame buildings behaviour before and during failure events is controlled by homogenized behaviours of substructures (Foliente 1998; Paevere 2002; Lam et. al. 2002; Doudak 2005; Kasal et. al. 2005; Langenbach 2008; Pei et. al. 2010). The behaviours of complete buildings are not controlled by how, for example, individual nails or studs embedded in substructures behave. Therefore, concepts just explained are fundamentally valid, as indeed they logically must be. However, given the unconstrained nature of types of structural systems engineers will wish to design in the future, design decisions cannot practically be made on the basis of experimental and field evidence alone. Detailed research level analytical models must be used to perform artificial experiments in computers, and thereby create a sufficient database to fully articulate the form of design level practices for Canada and elsewhere.

The remainder of this paper presents an example of one of the research level analytical modelling techniques and the associated verification exercises, plus some further brief commentary on steps lying ahead.

What is presented her applies to low-rise (circa 1 or 2 storey) building superstructure systems responses to wind and other pseudo static loads. Other modelling techniques being created by the authors and co-workers apply to taller buildings and other loading situations.

2 MODELLING CONCEPTS AND PROCEDURE

Following North American construction practices, nearly all wood light-frame buildings have structural walls made from small dimension nailed together lumber framing to which wood and sometimes other sheathing materials are mechanically fastened. Floors are most typically made as platforms separating walls in storeys, with the primary structural elements in floors being lumber or modern engineered-wood joists to which wood sheathing is nailed on the upper surface. Roofs normally are made from trussed rafters having small dimension lumber members mechanically interconnected with punched metal plate fasteners, with nailed on wood sheathing attached to upper surfaces of trusses. Undersides of floors and roofs often have mechanically fastened in place sheathing (e.g. plasterboard/gypsum-board). Connection of wall to floors is normally achieved via simple mechanical connections, as is attachment of superstructures to foundations. Wall studs, joists and trusses are parallel arranged and closely spaced so that such elements act compositely to resist effects of various applied forces. Walls and floors often contain "perforations" for doors and windows which can range in size from small to large proportions of wall surface areas. Structurally wood light-frame buildings are best characterised as arrangements of interconnecting and/or interlocking rib-stiffened plates.

Because of the construction methods and because of architectural geometry, the mechanical behaviour of substructures (e.g. wall segments) and complete systems can be very complex and the paths by which forces induced by external loading quite opaque. Figure 1 shows a partially constructed "typical" wood light-frame building.

Figure 1. *Wood light-frame building under construction*.

Numerical modelling of such buildings can be achieved based on using beam, plate and connection/joint elements within a finite element approximation strategy. What is presented below was implemented using the SAP2000 commercial finite element software (CSI 1997), because the authors have found it to be a reliably robust analysis tool. Other software products have similar capabilities and could be used instead.

Linear Hermitian "Frame" elements with six degrees-of-freedom per node were used to model all the framing elements (studs, top and bottom chords, lintels, floor joists, and roof trusses members). Therefore, all frame elements in the 3-D whole structure model included the effects of biaxial bending, torsion, axial deformation and biaxial shear deformation. Individual sheathing panels were modeled as linear "Thin Shell" elements which had four nodes, with six degrees of freedom at each node. All nailed sheathing-to-framing and framing-to-framing connections were modelled using nonlinear link elements (NL-Link in SAP2000) composed of springs with axial, transverse (shear) and rotational degrees of freedom. As shown schematically in Figure 2a, the NL-Link links two joints designated ***i*** and ***j*** and consists of six independent springs per joint. Note: To avoid confusion the diagram does not show all available spring. Assigning realistic load-deformation properties to the NL-Links is very important. Figure 2b shows a typical example for transverse shear loading of a 63.5 mm (2 ½″) common nail connecting the Oriented-Strand-Board (OSB) sheathing to a 38 x 89 mm (nominal 2-by-4) lumber framing member.

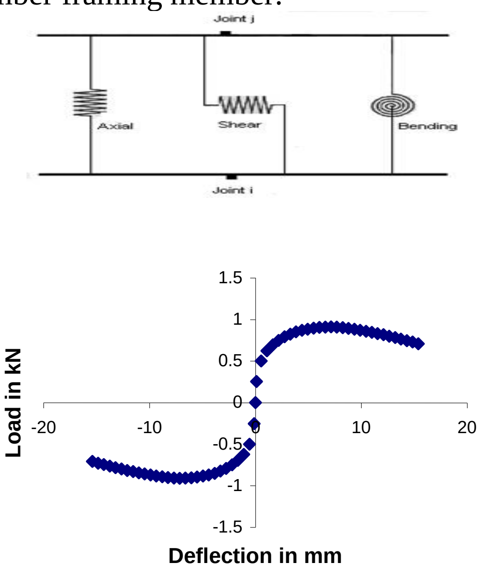

Figure 2. *NL-Link element*

a) *Arrangement with three of six available independent springs shown*

b) *Sheathing-to-frame nail connection response (transverse shear loading)*

As can be seen modelling of links included the softening regime that occurs when nails have begun to yield, which is necessary for accurate representation of failure in systems because often they are the primary source of system ductility. The curve shown employs a multi-linear load-deformation function fitted directly to connection test data. The approach in creating SAP2000 models for research was to represent the microstructure of light-frame systems down to every framing member, sheathing panel and nail.

3 VERIFICATION OF MODELLING CONCEPTS AND PROCEDURES

Two verification examples are summarised here aimed at assessing modelling technique capabilities to predict deformations and force flows in the linear response range for a building system, and capabilities to predict the overload responses of wall substructure loaded in-plane. The former is, as most analysts recognise, the most difficult response regime to model for complex structures situations, and the latter represents the critical behaviours of what post-disaster investigations indicate are highly vulnerable substructures in wood light-frame structure under extreme loading situations (Foliente 1998). Fuller details of both types of verification exercises are given by Doudak (2005).

3.1 *Full scale bungalow in Fredericton, NB*

Figure 3 shows some details of the specially built bungalow located in Fredericton, New Brunswick, Canada.

It is rectangular in plan with an 8.5 x 17 m footprint and a duo-pitch roof with 4/12 slopes. The construction details were typical of light-frame buildings in North America, i.e. lumber framing in walls, trussed-rafter roof and a joisted floor platform. All structural sheathing was OSB. However, unlike in normal buildings, the complete superstructure “floated” on a set of fifteen three-axis load cells sandwiched between the bottom of the floor platform and a perimeter reinforced concrete frost-wall foundation (the floor platform has no other support).

Figure 3. *Test bungalow in Fredericton, finished with siding*

This allowed instantaneous and accurate determinations of total and point horizontal and vertical force exchanges/flows. Other instrumentation installed to understand force flows and the deformation response were vertically oriented one-axis load cells at selected truss to wall attachment points, and displacement measuring devices characterizing vertical and horizontal displacements at various superstructure locations (e.g. bottom and tops of walls, roof ridge). Controlled vertical and horizontal static loads were sequentially applied to exterior surfaces of the bungalow (i.e. walls, roof) and internally to the floor, Figure 4, and associated force flows measured. All loads were insufficient to cause damage.

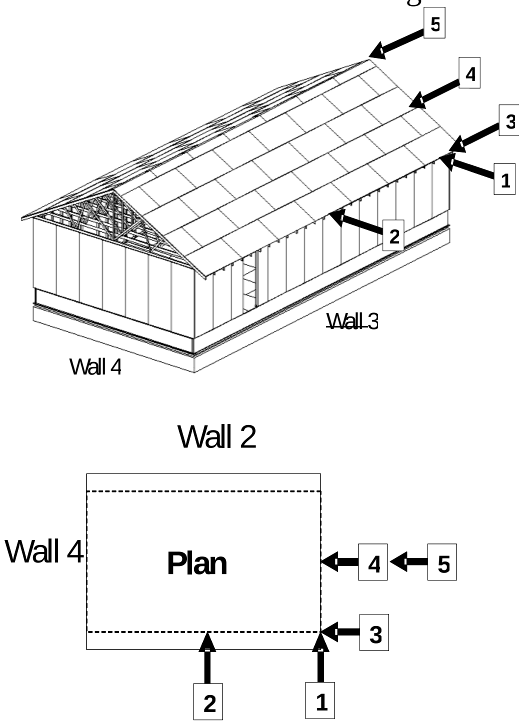

Figure 4. *Test bungalow in Fredericton, finished with siding*

Figure 5 shows the finite element model of the superstructure of the building.
Table 1 compares measured and predicted force flows at the superstructure to foundation interface, for each of the exterior surface load conditions illustrated in Figure 4. The shown comparisons are

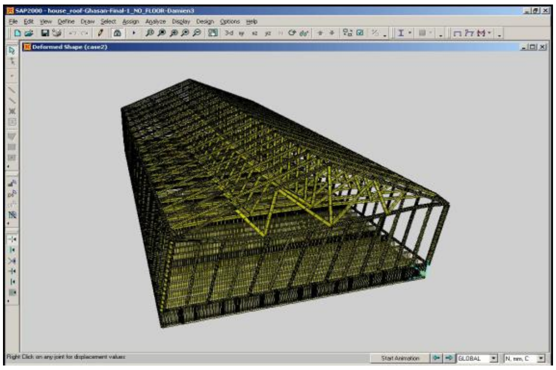

Figure 5. *SAP2000 model showing superstructure framing arrangement*

made on the basis of total horizontal force flows at interface line connections beneath each of Walls 1 to 4. Vertical force flows are segregated on the basis of tensile (uplift) and compressive total force flows in the same interfacial line connections, which is necessary for assessing model capabilities when only horizontal forces were applied to the building. Doudak (2005) gives details of force flows in individual load cells and observed and predicted deflections.

Table 1: Comparison between measured and predicted superstructure to foundation force flows in the test bungalow

		Wall 1		Wall 2		Wall 3		Wall 4	
Load case	D	T kN	M kN	T kN	M kN	T kN	M kN	T kN	M kN
1	Y	3.0	3.3	0.9	1.0	1.1	1.1	0.4	0.4
	Z^+	1.1	1.3	1.0	1.1	0.2	0.2	0.3	0.4
	Z^-	1.5	1.7	NA	NA	1.4	1.5	0.1	0.03
2	X	0.9	1.0	0.2	0.2	0.2	0.2	0.6	0.6
	Z^+	0.5	0.5	0.3	0.3	0.7	0.6	0.4	0.3
	Z^-	0.4	0.5	NA	0.3	0.6	0.5	0.3	0.5
3	Y	0.5	0.6	1.1	1.0	4.2	4.1	0.3	0.4
	Z^+	NA	NA	0.1	0.1	1.4	1.7	0.4	0.5
	Z^-	1.9	1.7	1.1	1.1	1.2	1.0	NA	NA
4	X	0.01	0.01	0.3	0.3	0.4	0.3	0.02	0.02
5	X	0.01	0.03	1.1	1.1	1.5	1.5	0.07	0.07

Notation:
Walls 2 and 3 are oriented parallel to the roof ridge, and Walls 1 and 4 are oriented perpendicular to the roof ridge.

T = Test
M = Model
X = sum of forces parallel to the roof ridge
Y = sum of forces perpendicular to the roof ridge
Z+ = sum of uplift forces in the vertical direction.
Z- = sum of compressive forces in the vertical direction.
NA = not applicable or insignificant.

The overall finding is that the adopted modelling techniques were able to accurately predict the response of the test bungalow. Precise predictions were also obtained by Paevere (2002) and Kasal et al (2005) who used similarly precise finite element modelling approaches to predict the cyclic displacement response of an L-shape plan wood light-frame bungalow built and tested under laboratory conditions. In the case of their investigation walls were directly connected to a reinforced concrete slab. It can be concluded, based on the two studies, that employed modeling techniques (as discussed here) remain robustly reliable when geometries and construction details of buildings are altered.

To be noted is that subsequent to experiments mentioned here, the bungalow in Fredericton has been structurally altered several times and reloaded with static forces to assess the effects of those modifications. The response of the building in undamaged condition has also been assessed under artificial dynamic impact forces and natural wind loads. Results of that additional work will be reported via future publications.

3.2 *Walls segments with in-plane forces*

Results of a comparison of finite element predictions with destructive tests on a series of seven light-frame wall segments tested destructively under in-plane loading are summarized here, to illustrate that research level analyses can predict substructure responses very accurately. Figure 6 shows a typical SAP2000 model of one of the tested wall segments and Figure 7 shows a comparison of the load test and model load versus deformation responses.
The relationships in Figure 6 are so-called racking deflection versus racking force relationships, which are commonly used as the basis of deciding how light-frame walls will respond if called on to act as "shear walls". Within the series of tests influences of construction variable investigated to determine their influences on wall segment stiffness and strength were: interaction of sheathing and framing, alternative base hold-down methods and segments with or without perforations. Table 2 summarises the experimental and model results.

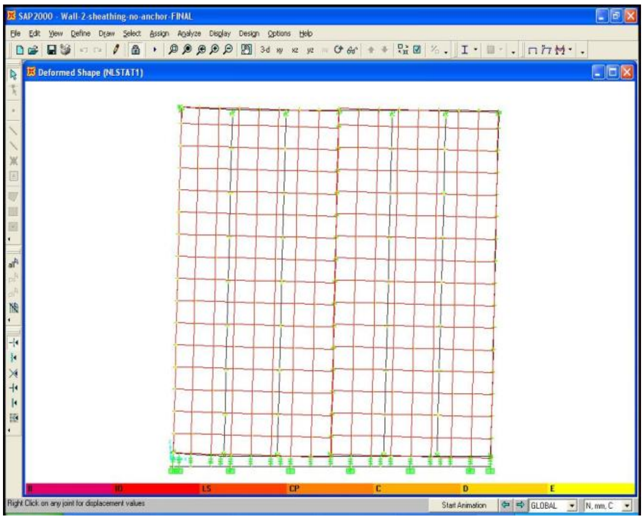

Figure 6. *Wood light-frame wall segments, SAP2000 model*

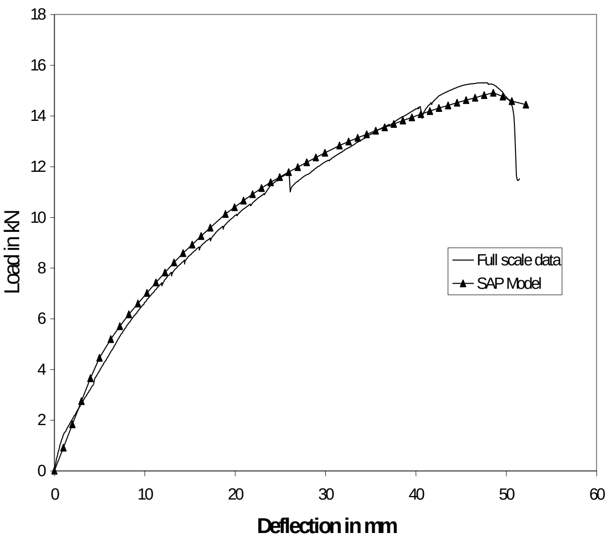

Figure 7. *Comparison of the load test and model load versus deformation responses*

Table 2: Comparison between measured and predicted responses of wood light-frame wall segments

Wall No.	Max load (kN)		Initial stiffness (N/mm)	
	Exp.	Model	Exp.	Model
1	0.3	0.3	8.7	8.4
2	20.8	20.1	558	695
3	9.3	9.3	313	326
4	11.2	11.4	380	392
5	10.9	10.6	416	544
6	12.1	12.2	562	550
7	15.3	14.9	603	737

Exp.: Full-scale test measurement
ER: Relative error in model prediction

As indicated model predictions were very accurate in all instances. This indicates that responses from initiation of loading to peak capacity and beyond (i.e. into the post peak-load softening regime) can be modelled with confidence. It should be emphasised however that models of earlier vintage did not achieve such precision. Ability to obtain the type of accuracy exhibited in Fig. 6, and other instances, is highly dependent on proper modelling of the component responses. In particular it is necessary to precisely replicate connection responses, as is illustrated in Fig. 2b for example. Fuller details of the wall segment study are given elsewhere (Doudak 2005; Doudak and Smith 2009).

4 DISCUSSION

As identified by Kasal et al (2004), there are many variations in current design level analysis of wood light-frame superstructures that individual designers employ, but those methods are mostly relatively crude and based on “engineering judgement” rather than scientifically proven understanding. Practices adopted include simple assumptions about how walls interact with floor and roof diaphragms to resist effects of lateral forces occurring during strong wind and seismic events; simple assumptions about which walls participate in resisting all types of design forces; and simple assumptions about the distribution of forces flows along line connections between layers in the building (e.g. between floor or roofs and walls, between superstructures and foundations). As already indicated fully verified modelling techniques discussed here and others become useful for investigating whether or not simplified approaches are reliable in various contexts. The likely scenario is that some applications of wood light-frame construction will be adequately addressed following established design practices but others will not. In the opinion of the authors, in situations where established design practices are inadequate, as might well be the case for large and tall modern buildings, definition of appropriate scale(s) at which to represent the microstructure will lead to design approaches that are reliably robust, consistent with or able to be made consistent with information in loading and material/timber design codes, and are not excessive impositions of complexity on design engineers.

In Canada the national timber design code committee has accepted in principle that all types of wood structural systems should be designed based on system level thinking, contextualized within the framework of LRFD practices. It is believed that wood design committees in other countries are moving in the same direction. What is discussed in the paper is one part of putting flesh on the bones of accepted principles.

5 CONCLUSION

No clear understanding exists about the capability/acceptability of established design practices for wood light-frame construction if applied in new contexts (e.g. design of relatively large and tall buildings). Fortunately researchers have created advanced analytical techniques that apply to new and old forms of such construction, but the resulting models are highly complex and not likely to be readily accepted for normal design practice. Plus, in some instances research models employ higher order concepts than do contemporary design codes. It is necessary to close the gap between what researchers can provide and what designers need. Discussion here frames the issues and presents a strategy for transitioning advanced capabilities into usable practices.

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