Modelling the Structural Response of Wood Light-frame Structures

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Summary

Light-frame superstructures of wood buildings behave as complex systems, which makes intuitive understanding of the load-carrying mechanisms very difficult. It is hard to distinguish between structural and architectural components and how those interact with other complexity inherent to modern buildings. Consequently it is necessary to have verified models of complete buildings and subsystems like shear walls. This paper reports static non-destructive load tests on a whole bungalow-like building and destructive tests on isolated shear walls, and use of data to verify 2-D shear wall and 3-D whole building finite element light-frame (fe) models. Both shear wall and whole building models showed good agreement with test observations. The 3-D model was able to predict the appropriate wall and roof stiffness and therefore able to distribute the load realistically in the structure. Verified models will be to assess simpler design level representations.

1. Introduction

Experimental studies in structural engineering have mostly been conducted on the behaviour of elements and subsystems, ignoring the three-dimensional (3-D) effects that are present when the whole structure is considered. Design of wood light-frame structures is based on an assessment of the capacity of isolated rib components such as floor joists, wall studs, or roof trusses. Practical experience proves that traditional light-frame systems can be strong and robust, provided that interfaces between subsystems are properly constructed [1]. It is generally presumed that adequately reliable systems result if components assessed in isolation have adequate reliability. Presumptions rely on the accuracy of the analysis and assuming that system effects on structural reliability are negligible. However, for highly indeterminate systems (like light-frame buildings) validity of either of these presumptions is very doubtful because the stiffnesses and strengths of individual subcomponents are correlated. For wood buildings the typically high variability in mechanical properties of subcomponents compounds issues of system behaviour and simplified concepts are undoubtedly inaccurate in their case. Design inaccuracies can be expected to be greater than for similar systems made from materials like cold-formed steel. Poor understanding of system versus isolated subcomponent behaviour is undoubtedly a major reason why wood systems have better field performance than can be predicted by traditional simplified design methods. Although there is clear evidence that low-rise timber buildings are vulnerable to extreme storm and seismic events
[2], that results mainly from underestimation of forces that occur during extreme events and poor quality control [3]. Accurate design depends on ability to accurately predict both design loads and how buildings resist those loads.

This paper is concerned with providing verified modelling tools, based on the finite element (FE) method of analysis, for accurate prediction of the system response of highly indeterminate light-frame wood buildings and subsystems of such buildings. Future papers by the authors and co-workers will deal with issues of characterising design loads, interpretation of field monitoring data for light-frame buildings subjected to wind loads and statistical aspects of system performance.

2. Goals and scope of research

The goals of research reported were to model the structural responses of a typical whole light-frame building and typical isolated shear walls. Ability to model shear walls is essential to modelling of whole buildings because of the strong influence that walls have on complete systems [4]. Because the selected whole building is also being used in field monitoring studies of wind-structure interaction it was loaded non-destructively. Shear walls and other subcomponents were loaded destructively. It is presumed that if low load response of a whole building and behaviour of subsystems and components can be predicted to destruction, predictions of complete building failure will be reliable. This is valid provided that modelling techniques are consistent. Specific objectives were to:

- Measure internal forces and deformations in a building and shear walls and correlate them with applied loads.
- Develop numerical structural models based on FE analysis.

3. Testing method

3.1 Shear wall tests

Shear wall tests encompassed 2.4 x 2.4 m wall panels. Arrangements represented situations without openings, with a door opening, with a window opening, alternative base hold-down detailing, and alternative levels of compressive top loads. The panels had two sheets of vertically 11.1 mm thick oriented strand-board (OSB) nailed to 38 x 89 mm softwood framing. There were six wall tests in total representing construction situations typical of light-frame wood wall panels in North America. The case of a wall without sheathing was also investigated as a control situation.

The purpose was comprehensive and robust verification of FE models and therefore each wall was extensively instrumented to record the deformation response under a lateral (racking) force applied to one top corner. Fig. 1 shows a typical shear wall test set-up. Walls were restrained only against out-of-plane translation. Base hold-down anchoring (of various types) restrained overturning and base sliding, but otherwise wall rotation was unrestrained. Prior to construction every element of the walls’ framing and OSB sheathing was identified and stiffness properties measured for replication in FE models. Behaviours of connections were determined via matched specimen tests to determine lateral and withdrawal load-slip responses and capacities. During tests the piston applying the racking force moved horizontally in parallel with a steel top beam used to ensure uniform application of force along the top edge of the wall. Full details are given elsewhere [4].
3.2 Whole building

The whole building was a single storey bungalow-like structure located in Fredericton, New Brunswick, Canada (Fig. 2a) that has a light-frame superstructure above a floor platform supported on a concrete frost wall. Footprint dimensions are 8.5 x 17 m and the duo-pitched roof has slopes of 1:3. The walls are panelised (panels 3.6 m in length) with each panel having 38 x 89 mm S-P-F framing and studs spaced at 600 mm. Wall framing is covered by 9.5 mm thick OSB nailed in place. The roof consists of trussed rafters at 600 mm spacing and sheathed with 13 mm thick OSB nailed in place. The house meets the design specification of the Canadian Mortgage and Housing Corporation [5].

![Diagram of building](image)

**a) building as tested (no interior walls)**

**b) application of a horizontal (racking) force**

*Fig. 2 Whole test building*

Two series of load cells are installed at the roof-to-superstructure level and at the superstructure-to-foundation level as shown schematically in Fig. 3. These provide ability to make observations of internal force flows within the structural system, i.e. the system encompasses the superstructure, foundation, load cells themselves and applied loads. An important feature of the load cells is that the 3-axis load cells are the only mechanical links between the superstructure and the foundation wall interface and thus the building can be constantly weighed. This is important for model verification as discussed here, and ongoing monitoring of wind forces and their effects on the building (not discussed here). Load cells at the roof-to-wall interface are placed selectively and permit discrete internal force observations. Availability of internal force data in combination with discrete observations of structure deformations under defined loads provides a sound basis for model verification [6]. Controlled static loading tests have been conducted to assess the response of the structural system to point horizontal lateral (racking) forces; point and patch gravity loads placed on the upper roof surface and floor platform; and point loads hung from lower chords of roof trusses. At the time of the load tests considered here the building had no internal linings, no external finishes, no internal walls and the only wall openings were a pedestrian door in each long wall. This is a baseline condition and additional tests are in progress to assess the effects that various...
sequential construction modifications have on the flow of internal forces and structural deformation response of the building. All static loads were kept to a level that maintained an apparently elastic response and thus it is presumed the system is undamaged. As for shear walls, the elastic properties of individual framing and sheathing components were determined in subsidiary tests, as were responses of connections (mainly nailed joints) loaded to failure. The stiffness characteristics of each load cell were established on an individual basis. Fuller details of test methods are given elsewhere [4].

4. FE models

![Fig. 4 Examples of meshes for fe models](image)

Finite element models were built for each shear wall test configuration and the whole building using the SAP 2000 commercial software package. Although it may seem obvious, an important but often neglected feature of fe model verification is the need to accurately map geometry and material properties from test situations to the modelling environment. The approach taken by the authors was to ensure that the mapping was accurate for individual components of each shear wall or the whole building. Fig. 4 illustrates detailed models for a shear wall and the building. Linear “frame” elements were used to model all the ribs (studs, top and bottom chords and lintels) in walls. Frame elements in the 3-D whole-building model use three-dimensional beam-column elements that include the effects of biaxial bending, torsion, axial deformation and biaxial shear deformations. Sheathing panels were modelled as linear “shell” elements. Elastic orthotropic material properties were accurately mapped for both the frame and shell elements assuming an orthotropic response. Nonlinearity was included in the connections via the nonlinear links (option “NL-Links”). Strength degradation of the connections is included by using a multi-linear load-deformation function fitted to the experimental results. This reflects test observations by the authors and others that system failures are highly dependent on the connections [3, 4, 6]. Full details of the fe models are given elsewhere [4] and here the intent is to summarise what was learned concerning accuracy / verification of those models.

Overall ability of the fe models to predict the load versus deformation characteristics of shear walls was excellent and Tab. 1 summarises results. Fig. 5 shows representative comparisons of the test versus model results. The most salient features are ability to represent initial flexibility, development of non-linearity in the response, ultimate load capacity and post peak load softening in
the response. The ability to make accurate fe predictions is independent of the specifics of the wall situations. Detailed analysis of both test data and model output shows that, as expected, the connections are the primary control on non-linearity and the failure mechanism. Based on the study it is concluded that it is justified to assume that framing and sheathing elements remain elastic in response in the range of loads considered.

a) wall with: window opening (900 x 762 mm), anchor bolts at base, and 2.08 kN/m compressive vertical load

b) wall with: window opening (900 x 762 mm), anchor bolts and corner hold downs at base, and 2.08 kN/m compressive vertical load

Fig. 5 Load versus racking deformations in shear walls: Examples of test versus fe results

<table>
<thead>
<tr>
<th>Description</th>
<th>Initial stiffness (N/mm)</th>
<th>Ultimate racking load (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test</td>
<td>Model</td>
<td>Test</td>
</tr>
<tr>
<td>No opening, no additional anchoring. vertical load 4.16 kN/m</td>
<td>558</td>
<td>695</td>
</tr>
<tr>
<td>Door opening (1938 x 838 mm), no additional anchoring, vertical load 2.08 kN/m</td>
<td>313</td>
<td>326</td>
</tr>
<tr>
<td>Door size opening (1938 x 838 mm), additional anchoring inside bottom corners and at door, vertical load 2.08 kN/m</td>
<td>380</td>
<td>392</td>
</tr>
<tr>
<td>Window opening (900 x 762 mm), no additional anchoring, vertical load 2.08 kN/m</td>
<td>416</td>
<td>544</td>
</tr>
<tr>
<td>Window opening (900 x 762 mm), additional anchoring at inside bottom corners, vertical load 2.08 kN/m</td>
<td>562</td>
<td>550</td>
</tr>
<tr>
<td>Window opening (900 x 762 mm), additional anchoring at outside uplift corner, vertical load 2.08 kN/m</td>
<td>603</td>
<td>737</td>
</tr>
</tbody>
</table>
The load paths observed in the static load tests, as well as the load paths predicted by finite element analysis, were very different from those expected according to current simplified design assumptions based on the tributary areas.

The most critical issue in shear wall design is the proportioning of the force in the various shear walls that can resist the loading. To assess this, Tab. 2 summarizes the various static test results and the FE model prediction. In general, the error in the numerical prediction is small, with the exception of the cases where the load magnitude was very low.

**Tab. 2 Comparison of test results with 3-D FE model prediction**

<table>
<thead>
<tr>
<th>Test no.</th>
<th>Direction</th>
<th>Σ Wall 1</th>
<th>Σ Wall 2</th>
<th>Σ Wall 3</th>
<th>Σ Wall 4</th>
</tr>
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<tr>
<td></td>
<td></td>
<td>FS kN</td>
<td>Model kN</td>
<td>ER. %</td>
<td>FS kN</td>
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<tr>
<td>1</td>
<td>Y</td>
<td>3.0</td>
<td>3.3</td>
<td>10</td>
<td>0.9</td>
</tr>
<tr>
<td></td>
<td>Z+</td>
<td>1.1</td>
<td>1.3</td>
<td>24</td>
<td>1.0</td>
</tr>
<tr>
<td></td>
<td>Z-</td>
<td>1.5</td>
<td>1.7</td>
<td>8</td>
<td>NA</td>
</tr>
<tr>
<td>2</td>
<td>X</td>
<td>0.9</td>
<td>1.0</td>
<td>13</td>
<td>0.2</td>
</tr>
<tr>
<td></td>
<td>Z+</td>
<td>0.5</td>
<td>0.5</td>
<td>1</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>Z-</td>
<td>0.4</td>
<td>0.5</td>
<td>12</td>
<td>0.3</td>
</tr>
<tr>
<td>3</td>
<td>Y</td>
<td>NA</td>
<td>NA</td>
<td>12</td>
<td>1.1</td>
</tr>
<tr>
<td></td>
<td>Z+</td>
<td>NA</td>
<td>NA</td>
<td>11</td>
<td>0.1</td>
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<tr>
<td></td>
<td>Z-</td>
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<td>1.7</td>
<td></td>
<td></td>
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<tr>
<td>4</td>
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<tr>
<td>5</td>
<td>X</td>
<td>0.0</td>
<td>0.0</td>
<td>-</td>
<td>1.1</td>
</tr>
<tr>
<td>6</td>
<td>Z+</td>
<td>0.3</td>
<td>0.2</td>
<td>25</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>Z-</td>
<td>0.2</td>
<td>0.3</td>
<td>5</td>
<td>NA</td>
</tr>
</tbody>
</table>

NA: Not applicable or insignificant
FS: Full-scale
ER: Absolute values of error in %
Z+: Sum of positive values in the Z-direction
Z-: Sum of negative values in the Z-direction

The model was able to correctly predict the 3-D behaviour of the structure by predicting the forces in the wall under load as well as the transfer of forces to the remaining walls. The model correctly predicted the stiffness of the roof system and its ability to distribute the load in the structure, as well as the interaction between the roof diaphragm and the shear walls, and the interaction among the shear walls. Designers typically have very limited knowledge about this interaction, i.e., whether to assume a rigid or flexible roof diaphragm. Their assumption has a very strong impact on the design of shear walls, in particular. The finite element model confirmed that the roof system was stiff and the shear walls were relatively flexible. This means that in similar buildings shear walls should be designed according to their stiffnesses rather than their tributary area.
5. General discussion

Results from the shear wall tests and fe analysis indicate that the initial stiffness and ultimate load can be substantially enhanced by, for example, supplementation of bolting down of the bottom plate with tie-down anchors at wall ends and wall openings. It is clear that neither the strength nor the stiffness of a wall is linearly proportional to its ‘effective length’. Effective length is usually assumed to equal the sum of the lengths of ‘full height’ segments. It can be concluded therefore that this simple concept that is currently used by many designers cannot be reliable, i.e. the racking capacity of a wall or complete building is not proportional to the effective length(s) of the wall(s) parallel in plan to the direction of the lateral design forces.

The results from the static loads on the whole building indicated significant load sharing, even though at the time of work reported here many components (both nominally structural and non-structural) were omitted from the system. In some load cases, only about half of applied gravity loads on the roof were resisted by the wall directly supporting the loaded zone. Mostly this reflects that the roof is much stiffer than might be supposed and that it activates resistance of transverse walls irrespective of whether applied forces to the building envelope are lateral (e.g. lateral wind loads) or vertical (e.g. snow or wind uplift loads on the roof). Clearly system behaviour should not be ignored in design.

Fe models are suitable tools for assessing which are the most important variables. However, it is impractical to assume fe models will be applied routinely in design. What has to follow on beyond building and verifying complex models is to use those as benchmarks for assessing relative capabilities of simplified ‘design level’ concepts and representations of how light-frame buildings resist loads. If appropriate, new concepts, representations and design tools will have to be formulated. In the case of many low-rise constructions (1 or 2 storeys) it will generally be important to predict the ultimate capacity of building subject to wind forces, but for higher-rise constructions predicting initial stiffness of systems and their natural frequencies will be important for either wind or seismic design. Any simplified design method that ignores this will be inadequate.

6. Conclusions

System behaviour of light-frame wood buildings is too complex for intuitive understanding and the only reliable means of predicting how any given structure will resist applied loads is an appropriately formulated model. It is shown here that finite element models are accurate tools for this purpose and can be applied with confidence to predictions at the whole building or subsystem level. The primary basis of precise predictions is accurate representation of connections. Finite element models can be used to assess simpler design level models or acceptable routine modelling assumptions and strategies. The main difficulty in that respect will be balancing considerations of robustness, accuracy and simplicity of application because it is impossible to simultaneously achieve high levels in all these criteria.

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References


