Field Determination and Modeling of Load Paths in Wood Light-Frame Structures

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Summary

Low-rise buildings, constructed using wood, are vulnerable to extreme wind storms and earthquakes. While several experimental measurements of the environmental loads (mostly wind) on the building envelope have been made at full scale, none of these studies directly linked these external loads with the internal forces and displacements of the structure, as achieved in this research.

The thesis presents the experimental and analytical work on two light-frame wooden structures, where one already existed (Forintek shed in Québec City) and the other (UNB house) was built specifically for the research project on the University of New Brunswick campus in Fredericton. The research goal was to devise and demonstrate methods of identifying load paths in light-frame wood buildings subject to environmental loads. The objectives were also to improve the knowledge on the magnitude of the forces generated by environmental loads on typical low-rise buildings; to measure forces and deformations in test buildings and correlate them with the applied loads; and finally to develop accurate numerical whole-building structural models.

These goals were achieved by carrying out experiments at the element level (studs, sheathings), subsystem level (shear walls) and on the whole-building level (finished and "realistic" light-frame timber buildings). The responses of these buildings to controlled static tests as well as natural environmental loads were observed and compared with a wind tunnel study and with detailed finite element models with good agreement.

Shear walls were tested in isolation and as a part of the whole structure. The tests indicated that neither the strength nor the stiffness decreased by the same magnitude as the wall effective length is reduced. Therefore, the simple concept of effective length, being used presently, is invalid.

For the Forintek shed, the structural monitoring was based on measurements of deformations within a representative segment of the wall and roof surfaces and a matching grid of wall and roof wind pressure taps supplemented with a wind tunnel study at Concordia University. In general, it was shown that the building surroundings had a great effect on the pressure distribution of the surface on the structure and that these effects are cannot always be determined intuitively. Both mean and peak pressure coefficient were measured and they compared well with corresponding values obtained in the wind tunnel tests. In general, the peak pressure coefficients from the full-scale tests were higher than those obtained from the wind-tunnel tests.

The results from controlled static loads on the UNB house indicated that the load was distributed to all walls, and significant load sharing was observed. Mostly, this reflected not only the rigidity of the roof, but also the rigidity of transverse walls. The stiffness of the roof was sufficient to distribute load to walls farthest away from the load application point. Also, the expected vertical paths for load were not observed. It was also found that the internal forces are concentrated near the corners of the building. Under vertical loading on the roof, the load at the roof-to-wall interface was concentrated in a small region of the building plan around the application point. This was not the case at the superstructure-to-foundation interface. The test results also showed that the load was transferred to the transverse walls, even though there were only nominal connection between the wall and the roof trusses.

The results from the analytical modeling showed good agreement with the full-scale test results for shear walls as well as for the whole building. The 3-D model was able to simulate the sharing of racking forces between shear walls, based on experiments reported in the literature. It was also able to reproduce static test results and predict the force measurements obtained from load cells underneath the house structure. In general, the errors in the numerical prediction were small. The model was able to predict the interaction between the roof system and the walls and the interactions amongst walls.

The research relied on the collaboration of several researchers in industry and academia, and was funded by a CRD grant of the Natural Sciences and Engineering Research Council of Canada.

Sommaire

Les bâtiments à ossature en bois sont particulièrement vulnérables aux tempêtes de vent fort (bourrasques, tornades, ouragans) et aux tremblements de terre. Plusieurs études se sont déjà penchées sur la mesure des charges environnementales sur l'enveloppe de bâtiments en pleine grandeur, mais aucune de ces études n'avait encore établi le lien direct entre les charges appliquées et la réponse structurale (forces internes et déplacements).

La thèse porte sur des études analytiques et expérimentales sur deux constructions légères à ossature en bois : l'une était une structure existante (entrepôt Forintek) située à Québec, et l'autre (structure UNB), représentant un bungalow résidentiel de construction nord-américaine typique, a été conçue et construite spécialement pour cette recherche sur le campus de l'Université du Nouveau-Brunswick à Frédéricton. Le but de la recherche était de proposer et de démontrer des méthodes pour identifier le cheminement ou la distribution des charges environnementales dans les bâtiments à ossature en bois. Les objectifs plus spécifiques étaient : d'améliorer les connaissances sur la grandeur des forces générées par les charges environnementales (le vent en particulier) sur des structures types ; de mesurer les forces internes, réactions et déplacements dans les bâtiments d'essai et de corréler ces mesures avec les charges appliquées ; et enfin de développer des modèles numériques précis et fiables pour l'analyse de structures tri-dimensionnels réalistes.

Ces objectifs ont pu être atteints en combinant des études expérimentales et numériques au niveau des composants individuels (poteaux, panneaux), des sous-systèmes (murs de cisaillement) et enfin au niveau global du bâtiment complet combinant l'ossature et les composants et finis architecturaux. L'auteur a étudié (observé, mesuré et simulé) en détail la réponse des deux bâtiments à des essais statiques contrôlés et à des charges environnementales. Certains de ces résultats à l'échelle réelle ont été comparés avec ceux obtenus sur modèle réduit en tunnel de vent (couche limite). Toutes les comparaisons ont été concluantes, y compris celles entre les mesures expérimentales et les résultats des analyses détaillées par éléments finis.

Les murs de cisaillement ont fait l'objet d'une étude spéciale en laboratoire, comme sous-système isolé, et ensuite dans l'ensemble de la structure complète. En autres, les essais en laboratoire ont montré que ni la résistance ni la rigidité des murs n'étaient réduites au même degré en fonction de la longueur effective du mur. Ce concept simple de longueur effective, abondamment utilisé dans le domaine, s'est avéré invalide.

Le monitoring de l'entrepôt Forintek était concentré sur la mesure des déformations d'une section intérieure complète du bâtiment (deux sections de murs continues à une section de toit), avec mesures des pressions de vent sur les surfaces de mur et du toit. Les mesures de charges de vent ont été complétées par une étude sur modèle réduit en tunnel de vent réalisée à l'Université Concordia. En général, les résultats indiquent que l'environnement du site et surtout la présence d'obstacles influencent beaucoup la distribution de pression externe sur les surfaces du bâtiment et que ces effets ne sont pas toujours faciles à prédire ou expliquer. Les valeurs moyennes et les valeurs maximales des coefficients de pression ont été comparées aux mesures en tunnel de vent : en général les coefficients de pression de pointe sont plus élevés sur la structure réelle que sur le modèle réduit.

Les résultats d'essais de la structure UNB sous charges statiques contrôlées ont montré comment les forces internes sont réparties à tous les murs et ont révélé l'importance du phénomène de répartition des charges entre l'ossature et tous les éléments de contreventement. En particulier, la rigidité du toit et des murs transversaux s'est avérée déterminante : La rigidité du toit était suffisante pour redistribuer les charges jusqu'aux murs les plus éloignés du point d'application de la charge. Aussi, le cheminement des charges verticales n'a pas suivi la trajectoire prévue par la méthode des aires tributaires des composants de l'ossature, les forces internes étant plutôt concentrées dans les coins plus rigides du bâtiment. Sous charge verticale appliquée au toit, la zone chargée de l'interface toit-mur était réduite au droit du point d'application alors que la charge était très dispersée à l'interface mur-fondation. Les essais ont aussi montré le transfert de charge aux murs transversaux même si la connexion entre le mur et les fermes du toit n'était pas rigide.

Les résultats des simulations sur modèles d'éléments finis détaillés ont été systématiquement en accord avec ceux des essais en pleine grandeur sur les murs

de cisaillement isolés et sur les deux structures d'essai. Le modèle tri-dimensionnel détaillé peut simuler correctement le transfert des charges latérales entre les différents murs de cisaillement et ces résultats ont également été validés avec des résultats d'essais publiés par d'autres chercheurs. Le modèle détaillé de la structure UNB a pu reproduire les mesures des cellules de charge à l'interface mur-fondation avec beaucoup de précision. Les interactions entre les murs et entre le toit et les murs sont clairement reproduites dans les simulations numériques.

Cette recherche a impliqué la collaboration de plusieurs intervenants de l'industrie et chercheurs universitaires et a été financée en partie par une subvention de recherche du Conseil de Recherche en Sciences Naturelles et en Génie du Canada.

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It is a great pleasure to acknowledge the valuable contributions of the many people who have helped to make this project possible. Firstly, thank you to my supervisor Professor Ghyslaine McClure and my co-supervisor Professor Ian Smith for their help, encouragement and continuous support. Also, thank you to my Danish contact person, Professor Preben Hoffmeyer from the Technical University of Denmark (DTU) for his help and support.

I would also like to thank the collaborating colleagues, students and technical support staff, especially Professor Ted Stathopoulos, Concordia University, for tremendous support and help with the wind related issues; Dr. Mohammad Mohammad, Forintek Canada Corp, for his help with the data collection in relation to the Forintek test structure; Professors Mike Bartlett, University of Western Ontario; Professor Ying Hei Chui, University of New Brunswick; Dr. Bo Kasal, North Carolina State University; Drs. Greg Foliente and Phillip Paevere, CSIRO. Special thanks to technical staff of the Forintek Canada Corp., in particular Mr. Anes Omeranovic, for facilitating the shear wall tests and for their tremendous help with the monitoring of the Forintek test structure. Also, special thanks to the staff at the Wood Science and Technology Centre, in particular Mr. Dean McCarthy, for their help with the building of the UNB test house as well as the installation of the instrumentations and the static load tests. Also thank you to Mr. Kasper Koeppen, Mr. Damien Gilles, Mrs. Alicia Ghallagher,

Finally, I would also like to thank my wife, Sandy, for her inspiration, patience and support throughout my Ph.D. work. This thesis is dedicated to my beautiful Samantha.

Original Contributions

To the best of the candidate's knowledge, this research is the first thorough study on the effects of wind loads on the structural response of light-frame buildings. The research is also unique in its methodology as it combined real-scale experiments on prototype structures under controlled loads, response monitoring under natural loads, reduced-scale models tested in the boundary layer wind tunnel, and detailed finite element modeling.

Monitoring of wind loads (wind speed and surface pressures) at the real scale has posed several technical difficulties. The experimental set-up and procedure first developed for the Forintek building have been successfully utilized in the UNB test house. This new and instrumented house will serve as a valuable laboratory in future years to better understand the effects of wind loads and the influence of the various architectural components on the load paths. Apart from the uniqueness of its wind instrumentation, the UNB test house is also the only real-scale construction equipped with load cells at its two main interfaces: roof-to-wall and superstructure-to-foundation. Although the load cell design for the lower interface had been developed in Australia for the CSIRO test house project, the load cell for the upper interface (roof-to-wall) has been designed and built from scratch. Several prototypes have been studied in the laboratory before the final three-dimensional load cell design was satisfactory. The technical challenge was to measure forces in the three orthogonal directions without imposing local stiffness incompatibilities.

In addition to these original contributions related to experimental aspects of the research program, improvements have been made in the application of finite element analysis to wood structures. Wood light-frame structures (the same applies to steel light-frame structures) are very complex systems where the framework components, panelling and connections need be modeled: it is neither accurate nor realistic to separate the framework from the other architectural components in any structural analysis. This study has shown that detailed and careful modeling of all important components, including the nailed connections, is feasible. The agreement obtained between the numerical simulations and the physical measurements suggests that finite element modeling is an invaluable tool to study wood structures and improve rational design methods.

In short, the sound and holistic methodology used in this research will likely have an important influence on the way structural engineering wood research will be conducted in the future. This will necessarily lead to important improvements in the design of wood structures, as the whole structural performance can now be predicted with accuracy.

The candidate has focused on the original contributions with likely high impact on future work in related fields. Several specific conclusions, many of them are new or in contradiction with previous knowledge on the various aspects of the research are stated in the corresponding chapters and summarized in Chapter 7.

The manuscripts presented in the following are the primary author's original work, except where due acknowledgment is made in the text. Comments have been provided by my supervisor, Professor Ghyslaine McClure, and co-supervisor, Professor Ian Smith. Comments have also been provided by Professor Ted Stathopoulos whenever relevant to wind monitoring and wind tunnel issues (Chapters 4 and 5). All of the work described is the result of the first author's direct involvement.

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CHAPTER 1

Introduction

1.1 Problem definition

Low-rise buildings constructed from wood are vulnerable to extreme storm and seismic events, with associated insurance claims averaging more than US$ 10 billion annually in North America alone. Low-rise timber buildings accounted for 70 percent of insured damage from Hurricane Andrew in 1992 (Foliente 1998). It has been estimated that between 1983 and 1995 the worldwide economic losses due to wind and earthquake events were US$ 230 billions (Smolka 1995), with the majority attributed to hurricanes and tornadoes. Failures are a clear indication that, at times, design and construction practices are inadequate. While several experimental measurements of the environmental loads on the building envelope have been made at full scale, none of these studies directly linked these external loads with the internal forces in the structure. This is despite considerable evidence that designs for low-rise timber buildings are often far from optimal. Also, in spite of the huge losses in human and economic terms, the amount of research oriented toward mitigation of damage to low-rise timber construction has been minimal (Foliente 1998). Establishing the load path within this type of structure is the first crucial step in understanding its behaviour.

Surprisingly few timber buildings have been tested at real scale, even under laboratory conditions, with none (prior to now) having been studied systematically in situ under the action of "real" environmental loads.

This research project relies on the collaboration of several researchers in industry and academia, and is a part of an NSERC/CRD project. The research involved researchers from McGill University in collaboration with the University of New Brunswick, Concordia University, the University of Western Ontario, and the University of Manitoba. The project was also supported by industrial partners, Forintek Canada Corp. and the Canadian Wood Council, as well as the Institute for Catastrophic Loss Reduction.

The candidate believes that this collaborative project is the first attempt to systematically monitor the structural behaviour of wood structures under "real" rather than simulated environmental loads. The intent of the project is to improve understanding of loads on, and 'load paths' within typical low-rise wood buildings.

1.2 Light frame structures and design codes

Light-frame wooden structures behave as assemblages of folded and interlocked plates stiffened by ribs. In most North American constructions, the primary plate elements are wood-based sheathing materials such as plywood or Oriented Strand Board (OSB) in 1.22 x 2.44 m sheets. Ribs consist of dimension lumber, wood I-joists or open-web trusses. Typically, plate elements are structurally attached to ribs by steel nails or glue.

Sheathed light-frame structures are redundant and this should make system analysis an essential part of understanding the ways in which external loads are resisted. In particular the behaviour of sheathed wood structures is based on two important system effects: composite action and load sharing. In the composite action mechanism, the element that connects the members together, for instance sheathing, acts in conjunction with the main structural members to produce a larger effective cross section. This composite action is usually only partly effective because complete shear transfer between the connected sheathing element and the main member is difficult to achieve in wood construction.

Load distribution refers to how the system components share the load prior to failure of any member. Load sharing among the various components may take place with or without composite action.

Load redistribution is the ability of the system to redistribute loads to unfailed or stiffer members, as members that are failed or partially failed lose their ability to carry the increased load. Load redistribution effects are not studied here but they are important to the reliability of the whole structure.

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. The tributary area method is used for load distribution proportional to rib spacing. System effects such as load sharing and composite action are allowed for by multiplying the bare rib capacity by a series of factors that reflect: rib spacing, the nature of the plate, the type of connection between the plate and ribs, and whether ribs are single piece or mechanically laminated (Foschi et al. 1989). For example, the bending capacity of dimension lumber ribs can be increased by up to 40% if such members are spaced less than 610 mm apart, if there are at least three members resisting a common load, and if a plate is providing sufficient connection between the ribs (CSA O86 2001).

Practical experience proves that traditional light-frame systems can be strong and robust, provided that interfaces between subsystems are properly constructed. Most small light-frame construction falls in the category of non-engineered or prescriptive construction buildings. In Canada such buildings must meet requirements of Part 9 of the National Building Code (NRC/IRC 1995). Implicitly, 'Part 9 type' construction is traditional in nature and such buildings embody high degrees of structural redundancy. However, the engineering community is questioning this, especially in the case of buildings constructed with non-lumber wood-based products and when building shapes are irregular or there are large openings in walls (Foliente 1998). An aim of this project is the verification and improvement of structural performance of non-engineered wood buildings.

Larger light-frame buildings have to be engineered with loads estimated according to provisions of documents such as Part 4 of the National Building Code of Canada (NRC/IRC 1995). It is assumed that environmental loads (wind, snow) can be represented as surface pressures on sheathings, with loads on supporting members being proportional to the projected tributary areas. This simplified approach is reasonable in situations where the subsystems are statically determinate, but this is rarely the case in light-frame structures. It is generally presumed that adequately reliable systems result if individual components have adequate reliability, but this relies partly on the accuracy of the analysis, i.e. correct prediction of the force in every component under various loading scenarios. For example, gravity loads on buildings are usually known to a high level of precision, because the geometry, construction materials and contents are all well characterized. It is feasible to predict environmental loads such as snow, wind, rain and flooding using stochastic models. This can be done by examining historical events and extrapolate the data to predict future events. Environmental loads due to wind and earthquakes can also be predicted within a statistical framework, but less precisely than snow loads on ground and roofs. Likely peak wind speeds can be predicted based on statistical estimates combined with wind tunnel determination of pressure coefficients on building surfaces. Pressure coefficients reflect the condition for a specific building with a specific shape and surrounding terrain conditions. Overall, it is fair to say that design estimates for natural loads on buildings are less precise than might commonly be assumed.

Due to their generality, current system-effect adjustment factors in design codes are often highly conservative. For example, there is no reason to assume that system effects become nil when the rib spacing exceeds 610 mm. No account is usually taken in design of secondary plate-layers such as plasterboard applied to interior surfaces of buildings, other than their dead load, even though it is well known that such layers can contribute significantly to the stiffness, and in some cases, to the strength of the various systems (Sherwood and Moody1989). On the other hand, studies have confirmed that commonly accepted assumptions regarding load sharing are in some cases un-conservative (Boughton 1988). Clearly, a system approach must be adopted towards structural analysis and design of ribbed-plate wood sub-systems and whole light-frame buildings for accuracy of design, reliability of performance and economic viability of the solutions. In particular, system analysis needs to recognise composite action and load sharing in an explicit manner.

1.3 Performance-based building codes

Performance-based design has been a major focus amongst researchers and designers in recent years. The codes will provide criteria for what a building system is expected to achieve in terms of building physics and structural behaviour, rather than the current prescriptive approach. Both the demands for the building design as well as the building materials are becoming increasingly sophisticated. This means that the current approach of prescriptive design and design based on elements and subsystems is being challenged. It is important, however, to understand that the assumption behind the performance-based philosophy is that performance can be predicted with accuracy and consistency (Paevere 2002). Both environmental loadings on structures as well as their structural response are random or semi-random in nature, and hence they are best described in stochastic terms. With the current tools, neither the loads on buildings, nor how those loads are distributed are well understood. Therefore, the safety and serviceability of light-frame buildings remains unclear, and it is virtually impossible to suggest how their structural performance can be improved. This raises the issue of understanding the load paths in structures in order to develop recommendations for improvement in their performance.

When fully developed and implemented, new codes are intended to lead to greater efficiencies in consumption of materials, and construction and operating costs based on life-cycle analysis and environmental considerations. Whole-structure testing and modeling must be utilised to understand the behaviour of light-frame structures, if accurate performance prediction is to be achieved. The development of experimentally validated analytical models of light-frame structures is essential in working towards this goal.

1.4 Research objectives

The goal of this research is to devise and demonstrate methods of identifying load paths in light-frame wood buildings subjected to environmental loads. The objectives are:

- Improving the knowledge of the magnitude of the forces generated by environmental (wind) loads on typical low-rise buildings under Canadian conditions
- Correlating field observations of wind pressures with those expected in the wind tunnel tests
- Measuring forces and deformations in test buildings and correlating them with applied loads
- Developing accurate numerical whole-building structural models
- Quantitatively assessing system effects and load sharing in the structure
- Studying the accuracy/adequacy of present analysis methods

1.5 Research strategy

1.5.1 Why full-scale testing?

Most experimental and analytical studies have been conducted on the behaviour of elements such as joists, studs and sheathing elements, or subsystems such as shear walls and roof diaphragms. As mentioned earlier, this approach ignores system effects and does not account for the effects of boundary conditions, despite the evidence that such effects influence the behaviour of the structure (Boughton 1988). It is important that analytical models be able to predict the behaviour of the structure (serviceability and ultimate limit states) on an elemental level, a subsystem level as well as on a whole system level.

Full-scale testing is paramount, as it is very difficult if not impossible to consider reduced-scale models in timber engineering, because it is difficult to reproduce the physical and mechanical characteristics of a building. For example, the joints cannot easily be scaled for both strength and stiffness.

Even though full-size structures within the laboratory are “realistic” in size, they often represent stripped-down construction. Also, the loading on these structures usually requires some modifications to prevent local damage. These modifications can be quite considerable and may therefore affect the response of the test structure, and lead to unrealistic failure mechanisms. Special attention must be paid to altering the test structures by adding various instruments such as load cells. The stiffness of these instruments should not affect the stiffness of the building and thereby create artificial load paths. This is especially important when the roof-to-wall interface is considered. Therefore, full-size tests done in situ are the only way to fully account for the effects of construction details, physical environment, moisture movement, loading and ageing processes.

1.5.2 Research method

The methodology in this research is to carry out experiments on finished and realistic light-frame timber buildings, and to observe the responses of those buildings to natural environmental loads. This implies that loads applied to buildings and their structural response must be monitored simultaneously. There is no intent or expectation that the test buildings will fail during the project, and they are expected to remain serviceable following the completion of experiments. The emphasis of the research program is on linking the in situ monitoring activities with wind tunnel studies as well as the development of numerical whole-building finite element models. This is represented schematically in Fig. 1.1.

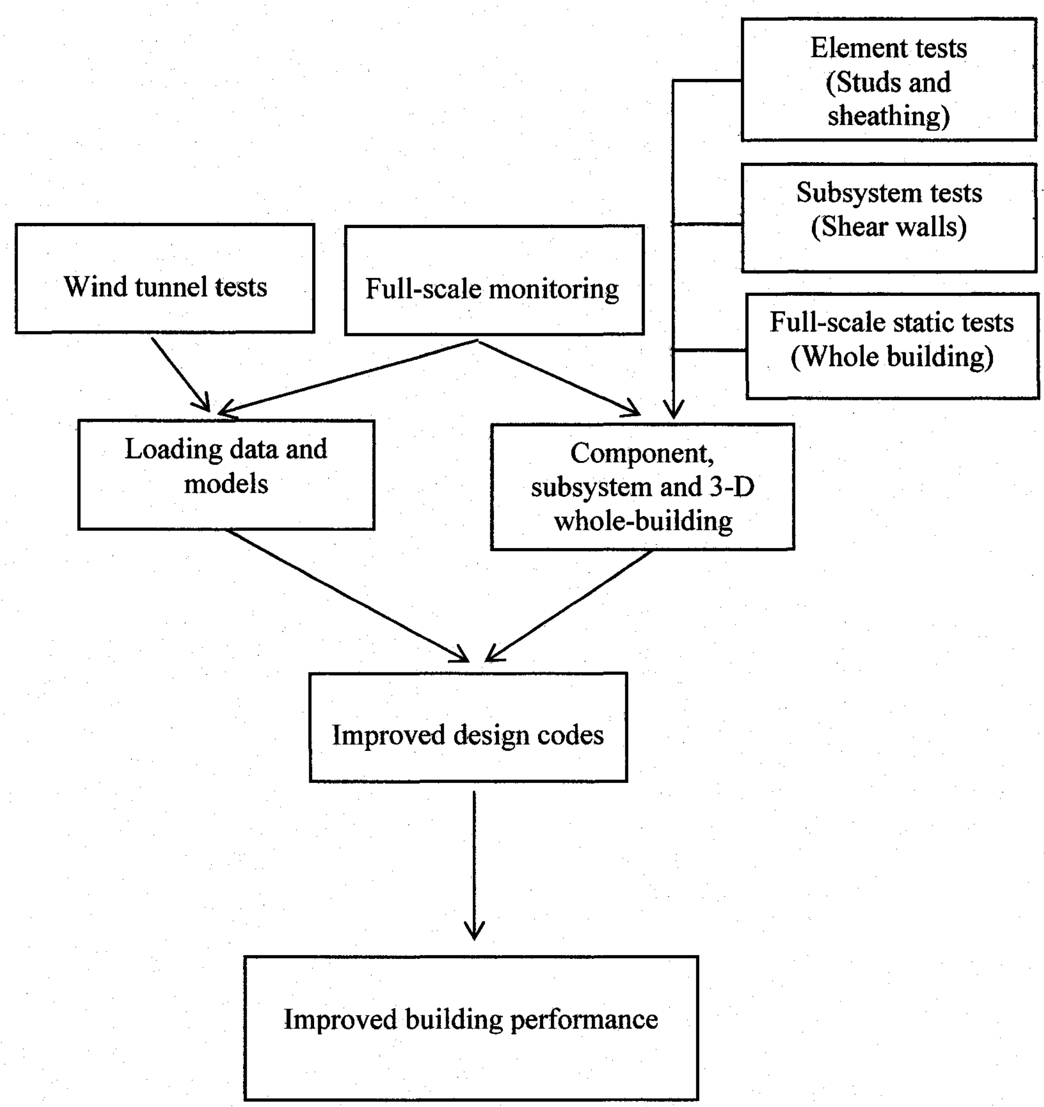

Figure 1.1: The methodology of the collaborative research study

The current collaborative research study is approaching the problem from different aspects and at different levels. The study is attempting to analytically predict the behaviour of a single element, a sub-system consisting of single elements, and finally a whole-building model that combines the two previous levels. Ideally, the analytical models should be able to link the individual elements with the whole-building response and *vice versa*.

The first test building is an industrial shed that has been studied previously without pressure measurements (Doudak 2000, Doudak et al. 2005). The second test structure was specially built for the project and is a typical North American residential construction (single storey bungalow-type building). It is located on the campus of the University of New Brunswick. The design of this house meets the Canadian Mortgage and Housing Corporation specifications (CMHC 1997), and is based on the so-called platform construction method.

Measurements taken for the UNB house included: wind speed and direction, internal and external wind pressures on wall and roof surfaces, internal forces at foundation-to-superstructure and wall-to-roof interfaces, and displacements (deflections and distortions) of stud walls and roof trusses. Measurements for Forintek shed were similar, except that there were no observations of internal forces, because this would have involved too much reconstruction. The UNB house is equipped with two series of 1- and 3-axis load cells (force measurements: vertically and horizontally parallel to external wall directions).

Simulated wind load tests on a 1:200-scaled model were carried out in the boundary layer wind tunnel at Concordia University for the Forintek shed and another model with similar scale is planned for the UNB test house. Models were tested for airflow in each of the principal wind directions, with upstream exposure representative of the upstream terrain roughness in the field. Due to the limited full-scale data collected, because only few predominant wind directions are usually present, the wind tunnel data can then complete the information missing from the full-scale study. Both full-scale data and wind tunnel data can then be used as loading data or input for the finite element model. The model can in turn be used as a prediction tool for different loading scenarios.

The author's contributions to the research study include: Element tests including studs, sheathing as well as some connection tests; Subsystem tests, such as shear walls; full-scale whole building static tests; full-scale monitoring of the surface pressures of the first test structure (Forintek shed); comparison with wind tunnel tests performed at Concordia University; analysis and design of the new house structure at UNB; planning, installation and calibration of the instruments, and analysis of data collected; development of preliminary analytical models, initial prediction of expected load path and determination of location for load cells and displacement transducers, and finally building two-dimensional and three-dimensional models of substructures as well as the whole structure.

The *in situ* tests will only provide data and verify whole building response at low load levels. As mentioned previously, the test buildings are not expected to be damaged to any significant extent. This could raise a concern about the usefulness of the results in predicting the behaviour at failure level. The key argument is that failure processes in structural systems always initiate on the local or subsystem level. Local failures and their propagation, to cause catastrophic whole-system failure, normally occur well beyond the design load levels. It is, therefore, important to know the load paths and what the forces and deformations are along boundaries of the subsystems or the components. This information makes it possible to predict the failure behaviour of subsystems or components. Subsystem failure mechanisms can be studied in isolation through relatively simple and inexpensive testing and analysis, for example in the laboratory.

1.6 Structure of thesis

Chapter 1 describes the background and the motivation for the research program.

Chapter 2 contains a detailed literature review. All literature relevant to the full-scale study of wood buildings and wind-related studies is presented.

Chapter 3 is the manuscript of a journal paper to be submitted to the ASCE Journal of Structural Engineering, titled: "Static loading and modeling of wood shear walls in isolation and in systems". The paper contains the results of tests on shear wall along with two-dimensional and three-dimensional finite element model response predictions. The logic behind the shear wall study is to attempt to analytically predict the behaviour of a single element, a sub-system, and a whole-building. The models in this study are precursors to more detailed 3-D model described in Chapters 4 and 6.

Chapter 4 is the manuscript of journal paper to be submitted to the ASCE Journal of Structural Engineering, titled: "Monitoring wind effects in an industrial wood light-frame building". This Chapter investigates the feasibility of structural monitoring on an existing building and compares the measurements to the results of a reduced scale wind tunnel model, as well as the results obtained from the analytical models.

Chapter 5 contains a description of the monitoring program at the UNB house including installation of wind instruments and preliminary data.

Chapter 6 is the manuscript of journal paper to be submitted to the ASCE Journal of Structural Engineering, titled: "Structural response and whole building modeling of a single-storey wood light-frame house to applied static loads". The paper describes controlled static tests to measure load sharing and composite action effects in the structure, as well as 3-D finite element modeling and calibration of UNB house model.

Chapter 7: Concluding chapter linking work together, drawing final conclusions and stating recommendations for future work.

Appendix A: A survey of full-scale tests on wood structures.

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CHAPTER 2

Literature Review

2.1 Full-scale tests and related research on wood structures

During the last fifty years, several investigations of the behaviour of whole buildings have been attempted. These are briefly described in the following sections. A survey of full-scale tests on wood structures is also summarized in a tabular form in Appendix A.

2.1.1 Tests in the 1950's and 1960's

One of the first and most important studies on full-scale wood structures was conducted by Dorey and Schriever (1957) from the National Research Council of Canada. They evaluated the response of a single-story house under simulated wind and snow loads. The walls were constructed with diagonal wind braces and covered on the inside with gypsum wallboard, while the external sheathing was purposely omitted. The goal of the structural test on this house was to obtain information on the strength and stiffness of a single-story house without exterior sheathing, and to obtain experience in full-scale testing and evaluation of strength of house frames. The loading was increased to the point of relatively minor damage. It was shown that the corner braces, combined with the various finishing materials, provided sufficient racking strength and that exterior wall sheathing was not required. The joist-rafter roof system withstood a load before failure that was 43 percent above the design load. The load that the walls could withstand was also considerably above the design wind load. Lateral distortion of the walls was relatively small but there was cracking in the gypsum wall board. This test demonstrated that light-frame structures built without exterior sheathing could demonstrate good field performance. Despite the tremendous contribution of this paper, it was only evaluating one particular house and there was no generalised conclusion because of lack of information about other full-scale structural tests. However, this study was a precursor to a series of similar laboratory full-scale studies that confirmed that such structures can be over-designed.

Another experimental house was evaluated by Hurst (1965) in Washington D.C. What made this study unique was that the test structure was specifically built for the research and was evaluated during various stages of construction, much like the test house for the present research (Chapters 5 and 6). The study provided useful information about the interactions between the various elements in the structure and the effect of openings and sheathing materials. Unfortunately, it did not quantify the additional contribution of the interior walls and the gypsum wallboard sheathing. Also, specific information on the load versus deformation behaviour of the shear walls was not obtained. The minor racking distortion of the end walls and the ballooning of the loaded walls were similar to those found by Dorey and Schriever (1957).

2.1.2 Tests in the 1970's

In this period, some studies confirmed the work of Hurst (1965) and Dorey and Schriever (1957), but some new types of tests were conducted such as studies on multi-storey structures, along with the dynamic response and the visual quantification of cracks during testing.

Yokel and his collaborators (1973) evaluated the performance of a two-story wood-frame structure under simulated wind loads. Two series of tests were conducted. The first was to determine the stiffness of the house when subjected to a simulated wind loading. The second was to determine the dynamic response of the house to a single impulse load. Visual observations of the interior and the exterior surfaces of the building were made before and after each test. In this test, the second story drift (lateral translation) of the building measured was considerably less than that derived using design criteria for medium-rise buildings as applied at the time. Another noteworthy fact was that only a small portion of the distortion of the exterior walls was transmitted to the interior gypsum board finishing material.

Yancey and Somes (1973) evaluated both the stiffness and strength of a house unit typical of factory-built modules. Six tests were conducted to quantify some of the structural characteristics of the wood-frame module. The structural tests were performed subsequent to a series of tests relating to transportation by rail. The data collected from these tests were supplemented with visual observations. All crack formations on interior surfaces were recorded and described. Most of the test methods were *ad hoc* since no existing standards were applicable. It is noteworthy that the unit evaluated by Yancey and Somes was considerably more flexible than the conventional house tested by Yokel et al. (1973).

Another important study on single-story light frame structures under various stages of construction was by Tuomi and McCutcheon (1974). The study evaluated the lateral resistance of the light frame structure to determine the effects of window and door openings, exterior gypsum board and other structural components. The structure was loaded until failure, which in this case occurred by splitting of the sill plate. The study was systematic and produced very important findings that were ground-breaking for the understanding of light-frame systems. This research was the first to show that light-frame structures could be over-designed in some respects but also under-designed in others. This was confirmed and studied in greater detail by Boughton (1988).

2.1.3 Tests in the 1980's

Extensive testing was conducted during the 1980's at the James Cook Cyclone Structural Testing Station in Townsville, Australia to evaluate light-frame wood structures under wind loading. Although houses were constructed outdoors, loads were artificial with suction pressures simulated using Wiffle-tree loading arrangements. The objective was to determine whether similar houses are likely to have adequate performance under certain design wind speeds. Findings confirmed the acceptable performance of the structures and that stiffening effects of secondary (nominally non-structural) elements can be considerable. No attempt was made to explain realistic load paths.

Boughton and Reardon (1982) conducted a test on an existing 40-year-old house. The building was constructed utilizing a timber frame and bolted timber trusses. The aim of the study was to determine the strength of the whole house in its assembled state, and to relate the performance of structural elements assembled in the house to the performance of the matched individual elements in the laboratory tests. The results showed that the measured failure loads of the various elements in the structure were higher than those predicted in most cases. It appeared that weatherboards assisted in transferring loads from the tested stud to the adjacent studs, and a significant reduction in the ultimate load was obtained by removing the weatherboards. Very little load was carried by bending of wall claddings and wall timbers. The contribution of each element to the load-carrying mechanism of the whole house was determined. This paper was one of the very few reports that discussed the distribution of forces throughout the house.

Reardon and Boughton (1985) also worked on testing of the Togan Hurricane house, simulating the pressure of the cyclone winds. The elastic response was measured, and an analysis was made to determine the load sharing between the walls, together with the effect of the roof diaphragm. The study showed that the internal wall, which was not designed as a bracing wall, had the capacity to act as one. Another important (and maybe unexpected at the time) result was that although the end walls had identical stiffnesses, they attracted different percentages of the applied load. This finding is of special interest to the present research program because it emphasizes the importance and need to quantify and understand the load paths in the structures.

Boughton (1988) conducted another full-scale structural test of a single-storey house. The test used static non-destructive lateral loads, and the applied load and the resulting deflections were monitored. The house was subjected to cyclic loading simulating wind loads. Also, to compare the fracture properties of the tested house and the observations made in damage surveys, parts of the house were subjected to overload. Loads were monitored at key application points with load cells. Simultaneously, deflections were measured using specially constructed displacements transducers. The results of this study showed that the stiffness of the loading frame and the support system play a significant role in the determination of the failure mechanism. The test also confirmed that architectural (non-structural) elements could play a role in distributing the loads to the structural components and the foundation.

In 1988, several important studies on full-scale wooden structures were conducted in Japan. Some of the most important work was done by Sugiyama et al. (1988 a). They performed a full-scale test of a two-story wood-frame house subjected to lateral load. The study investigated the influence of shear walls on the racking resistance of the house. A series of six tests was conducted by applying lateral load to the top of the first story during progressive stages of construction as the house advanced from the structure with solely let-in bracing to that covered with the exterior wall siding. Even though this was not the first study to test a structure under various stages of construction, the research yielded many useful results, where some were less expected than others. The contribution of the walls perpendicular to loading to the overall shear resistance was found negligible. The application of wall sheathing and /or wall siding to the wall spaces above and below the window and door openings provided some increase in the racking resistance. This work provided useful insight into the mechanisms of the structural deformations.

Sugiyama and his collaborators (1988 b) also compared the lateral stiffness of a frame obtained from full-scale testing and that estimated by racking tests on Japanese wooden frame constructions. The racking resistance measured in the whole-house test was about one and a half times the one estimated by using the unit resistance of the shear walls obtained from subsystem racking tests, regardless of the type of shear wall. This result was explained by the rotation of a shear wall resulting from the uplift of a column, which was smaller in the actual house than in the racking test.

Ohashi and Sakamoto (1988) tested a two-story structure with two partition walls in each story. The research was particularly focused on the influence of the floor rigidity on three-dimensional behaviour. Deflection was measured as a cycled load was applied horizontally to the top corner of one wall at a time. The load-deflection curves showed that the structure responded nonlinearly with degrading stiffness. The test indicated the strong influence of the connections on the overall behaviour of the structure.

Yasumura and his collaborators (1988) tested a three-storey wooden frame building, subjected to horizontal loads. Exterior walls were sheathed plywood outside and gypsum board inside, while interior walls were sheathed with gypsum board on both sides. Horizontal static load was applied at the top of the structure with three hydraulic jacks and the horizontal displacements of each story were measured. Forced vibration tests were also carried out before and after the static loading test by employing a rotating-mass generator. The shear deformations of the diaphragm were quite small and it was assumed that the diaphragm was rigid.

Also in the same year, Stewart and his collaborators (1988) tested two manufactured houses in Colorado, USA. The primary purpose of the research program was to evaluate the degree of participation of transverse walls in providing the racking resistance to lateral loads. The study identified the contribution of the transverse shear walls to the structural capacity of the buildings subjected to concentrated as well as uniformly distributed transverse loading. Architectural items were not installed to make them easier to instrument the houses. Due to torsion of the building shell, the interior walls did not exhibit as much racking deformation as the end walls. Due to the flexibility and energy dissipation characteristics, the buildings were able to sustain load three times that expected from a design wind, with relatively minor damage.

2.1.4 Tests in the 1990's

During the 1990's, several full-scale studies were conducted, most of them confirming findings established by previous studies. The novelty for this period was the interest in timber pole houses and the study of multi-storey structures. These studies also became more multi-purpose, integrating the study of moisture movement, accidental collapse, and fire safety. One of the most complete efforts was undertaken by the Building Research Establishment (BRE) in the United Kingdom.

Moore and his collaborators (1993) conducted a series of tests on five full-scale multi-storey steel frames. Each frame was three stories high and was tested under gravity loading. The primary requirements of the instrumentation were to measure the applied loads, the distribution of the internal forces around the frame, the deflected shape of the structure, and the moment-rotation response of the connections. In all tests significant interactions were observed with restraining effects being transmitted and moments transferred from beams to columns and *vice versa* via the connections. Of particular interest was the observation of moment redistribution away from collapsing columns towards restraining beams, which is regarded as the key feature in improved column behaviour as compared with predictions of simple construction.

Reardon and Henderson (1996) from the James Cook Cyclone Testing Station performed a test on a two-story house. They wanted to determine whether the structural redundancy and load sharing in a normal house construction designed for a 28 m/s wind speed was adequate to perform satisfactorily for 33 m/s winds. Wind tunnel tests were conducted to determine the total uplift forces, drag forces and overturning moment. The single-story section was able to resist twice the design pressure for 28 m/s and 33 m/s winds, without structural failure. For the two-story section, the frame increased in lateral stiffness as the lining elements were added and finally became a very stiff box, which demonstrated only small displacements at design wind loads. There was no evidence of failure during the overload strength tests in combined racking and uplift, which were conducted at 2.4 times the pressure for the 33 m/s wind.

More full-scale studies were conducted in Japan in the 1990's. Hirashima and Suzuki (1996) performed a full-scale horizontal loading test on a two-story wooden house. Two types of experiments were conducted: horizontal loading test on the whole construction and racking tests of the bearing walls. The horizontal load was applied to the top of both the first and second stories. Results from this study showed that the ultimate loads obtained from the tests were more that 2.8 times the design load in the ridge direction, and more that 2.5 times the design load in the transverse direction. The result showed high stiffness for both stories. The reactions in the first story were about 1.7 times the design level values.

Wood and Bullen (1996) studied the response of timber pole houses to lateral loading simulating wind forces at Queensland University of Technology in Australia. The test house was evaluated at various stages of construction to allow collection of information on how different structural elements resist lateral wind forces. The choice of pole type, timber size and grading, cladding and flooring was made as typical as possible. The initial testing consisted in characterizing the individual timber elements. The poles deflected similarly through the series of tests. The addition of the floor did not change the eave deflections, but a marked reduction (by a factor of two) was obtained after the bracing walls were constructed. The pole house showed increased lateral stiffness as architectural elements were added during construction.

Hirashima and Suzuki (1998) reported work on a full-scale horizontal loading test on a two-story wood construction. The tests conducted in this study were, racking of the elements using the plywood shear wall and bracing frame, and a horizontal loading test where the load was applied to both the top of the first and second stories. The results showed that the final load applied to the construction was equivalent to about three times the design load, without any apparent failure in the construction. The study also showed that the second story had higher stiffness than the first one. In addition, the load sharing of the structural elements tended to increase as the deformations increased.

More recent work conducted at the BRE in United Kingdom is the TF 2000 collaborative project between the British Government, TRADA Technology Ltd. and the UK timber industry. The test building, described by Steer (1999), was a typical six-story multi-occupancy residential block with four two-bedroom apartments per floor. The project studied in depth accidental collapses, differential movements between the timber frame and exterior masonry cladding and fire safety, rather than focusing on strength, stability and stiffness of the structure alone. It also dealt with the increase of quality assurance and building maintenance. Ninety three instruments were installed at various locations in the building to measure the displacements. Load cells were positioned selectively beneath the studs after completion of the first floor, to give an indication of the "actual" loads in the studs. The results showed that the greatest load was observed following installation of plasterboard linings. This indicates that forces are attracted to the stiff regions of building, as in the present work (Chapter 6). In general the distribution of load did not follow that assumed in the design theory. The TF 2000 project proved that the contribution of plasterboard to racking resistance is underestimated in design, as is that of the brick cladding.

2.1.5 Recent research studies

An extensive testing program was conducted at the University of California, San Diego, on a two-story timber frame house as a part of the Consortium of Universities for Research in Earthquake Engineering, CUREE-Caltech Wood frame project. This study was in response to the damage to light-framed timber constructions in the 1994 Northbridge Earthquake. The objective was to determine the seismic performance under different levels of shaking for different structural configurations. The scope of the project was very broad, including field investigations, full-scale house testing, component testing, and analytical modeling.

The whole-building testing involved a shaking table test of a two-story house (Fischer et al. 2001). The results confirmed that a fully engineered timber-frame house has better seismic performance than a conventionally constructed house. The test examined the effect of the contribution of different components and found that the non-structural wall finishes considerably stiffened the structure and reduced the displacement response level. Even though the CUREE project is one of the most extensive work conducted to date to improve the performance of light-frame structures, a detailed description of the load paths throughout the structure was not obtained.

In 2002, the Commonwealth Scientific and Industrial Research Organization (CSIRO) in Australia carried out a test on a single-storey house in partnership with North Carolina State University. An L-shape plan bungalow was adopted to promote torsion through the height of the system under various load arrangements. The building was subjected to a series of non-destructive tests prior to final destructive testing. Reaction forces beneath all walls as well as the displaced shape were measured in detail under static and static-cyclic lateral loading. A number of analytical models were developed and validated against the experimental results. The main purpose and value of the study is that it collected comprehensive data for verification of whole-building models. The study confirmed the potential for significant load sharing and redistribution of applied lateral loads as well as the importance of the roof and ceiling diaphragm under elastic and inelastic response conditions. The CSIRO experiment is the most accurately documented work available providing the detailed pattern of reaction forces under lateral loading (Paevere, 2002). In fact, some of those results have been used to validate finite element models in the present research as discussed in Chapter 3. However, the applied load was simple static or slow cyclic concentrated loads. One of the main differences between the CSIRO experiment and the present research is that forces at the roof-to-wall interfaces are also measured in the latter, in addition to forces at the superstructure-to-foundation interface. Also, the present study dealt with environmental loads (wind), in addition to the statically applied loads.

The University of Western Ontario is planning to develop a new test facility that will accommodate two-story houses. The facility will consist of a reinforced steel floor and a steel space frame that will envelop specimens to facilitate load application. Hydraulic jacks, pumps and controllers will be used to apply loads to the full-scale specimen. The test house itself will be designed to capture the external and internal pressures generated, as well as the overall house deflection and local deformations of its components. The facility is still under development and is expected to be ready for testing in 2008 (UWO 2002).

As a part of the current NSERC (Natural Sciences and Engineering Research Council) collaborative research project, including the present research program, monitoring work is ongoing at the University of Manitoba on a post-frame building with widely spaced columns, straw bale infill panels, open interior and a duo-pitch roof with laterally braced trusses (Smith et al. 2004). Instrumentation of the facility is related to both the structural behaviour and the building envelope performance. Ten posts are being monitored for axial force with load cells at each end. These load cells monitor both snow and uplift due to wind forces. Forces in the adjacent truss webs will also be measured. To obtain some insight into the behaviour of the metal cladding on the roof and the ceiling, strain gauges will be attached to these components at various locations. Construction of the building started in October 2003, and the remainder of the instrumentation was installed during Spring 2004. No results have been published so far.

2.2 Full-scale wind studies and wind-tunnel studies

Because most low-rise buildings are relatively low cost structures, little attention has been given to accurately measure the wind pressure distributions on these buildings. Increasingly more buildings have engineered elements such as trusses and I-joists and modern buildings have more irregular shapes and functions. This has led to the realization that pressure coefficients given in building codes might not be sufficient to satisfy design needs. Discussion of some of the most important research projects involving tests on full-scale structures follows.

Eaton and Mayne (1975) described an extensive full-scale experiment on houses constructed in Aylesbury, England. The main contribution to wind engineering that came from this project was the construction of an experimental building with a variable pitch roof, with possible variations between 5 to 45 degrees. Several wind tunnel studies have been conducted worldwide (Holmes 1982) in connection with this full-scale experiment. The location of a reference pit and its design are frequent problems associated with the field measurements, and the Aylesbury house was no exception. Variations in the pressure coefficients were attributed to the measurement of the reference pressure; an issue that is addressed in detail in the work of Levitan (1992). A summary of the international comparative study of the 1:100 scale model of the Aylesbury House is given by Sill and Cook (1989).

The results of full-scale and wind tunnel pressure measurements on a single-story residential house at the Malmstrom Air Force Base in Montana, USA are reported by Marshall (1975). Pressure transducers were referenced to the static port of a van-mounted pitot-static tube, located about one building height above the roof. There were some difficulties with the measurements and the comparison to the wind tunnel results was not conclusive due to the errors in the reference pressure measurements. The mean data were in reasonable agreement between the model and field observations, although some correction was required for the static pressure source.

The Silsoe experimental building, described by Robertson and Glass (1988), was constructed by the Building Research Establishment (BRE) in England in the late 1980's. Extensive measurements have been made on the envelope of the structure, providing data which was compared with wind tunnel test results. Strains were measured in some structural members but the attempt to describe load paths was very limited. The wind tunnel models were later retested at the University of Western Ontario (Surry 1989). The mean data taken on the full-scale Silsoe

Structure Building were not accurately replicated in the wind tunnel. The field observations appeared to be underestimated by 30% in the BRE study and overestimated by as much as 50% in the UWO wind tunnel study by Uematsu and Isyumov (1999).

A current initiative by the Florida Department of Community Affairs (DCA) launched the Florida Coastal Monitoring Project (FCMP) in collaboration with Clemson University (Reinhold et. al., 2000). Sensors to monitor wind speed and pressure were installed on roof surfaces, walls, attic spaces and interior rooms of 30 South Florida homes. The objectives were to collect uplift pressure data during a hurricane or high-wind event to study the response of the houses to hurricane force winds and to track any destruction that occurred in the houses. This information will also be used to compare the performance of houses that were retrofitted with houses that were not, to better assess the benefits and techniques of retrofitting houses.

More details on low-rise wind-engineering research have been presented by Stathopoulos (1984), Holmes (1993 and 2001), and Uematsu and Isyumov (1999).

2.3 Conclusions

Various aspects of building performance have been investigated experimentally during the past fifty years. Of these studies, most of the work has been undertaken on one– and two-story structures on rigid foundations. These reports provided insight into the mechanism of structural deformation and suggested important load sharing within the structure. The behaviour of the system under various stages of construction was studied, resulting in findings confirming that both the ultimate load and stiffness were improved by the addition of architectural components.

Some studies demonstrated that houses are two to three times as strong as contemporary design practices suggest. This implies that there are many design inefficiencies and that economies are possible, when simultaneously improving the vulnerability to various structural and non-structural damages.

Despite the availability of fairly sophisticated (usually finite-element based) computer models, it seems that complexities of wood-based and other low-rise constructions cannot be predicted with sufficient certainty and testing will continue to play a major role in the development of better buildings.

The majority of the studies surveyed here recognize the importance of load sharing and composite action between the components in the structural system. Very few studies actually measured the forces within the members and none have quantified the load paths in the structure, especially at the interface between the roof and the wall.

As later chapters show, the present work is making an important contribution towards elimination of gaps in knowledge.

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CHAPTER 3

Shear Wall Testing and Modeling

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. Analytical models must be able to predict the behaviour of the structure on an elemental level, a subsystem level as well as on a whole system level. The modeling approach adopted proceeds with increasing levels of details and complexity attempting to predict the behaviour of a single element, a sub-system consisting of single elements, and finally a whole-building model that combines the two previous levels. The approach is to build a detailed 3-D finite element model that can successfully describe the structural behaviour.

By combining all the individual components with their properties measured in the laboratory, a two-dimensional (2-D) finite element model is used for nonlinear static analysis and validated with the shear wall test data. The 2-D model is extended in three dimensions and checked once again. This 3-D model is later used to predict the response of a whole structure.

Static loading and modeling of wood shear walls in isolation and in systems

By G. Doudak[1], G. McClure[2] and I. Smith[3]

Abstract: Shear walls provide the strength to resist horizontal loads on buildings, typically from wind and earthquake. Recent advances have been made in earthquake engineering, but the wind response is very complex and still needs to be researched. The diversity of construction products introduced in the last decades has led to construction details that deviate from the conventional practices. This shift has made it more challenging to describe the behaviour of light frame structures, and has resulted in the traditional strength and stiffness studies on walls without openings being insufficient. The analysis is simply not sophisticated enough to be realistic. An example of a more realistic approach is one that would consider the stud ends as being semi-rigid and not pinned as currently assumed. Also, it is important to recognize the interaction between the studs and the sheathings in a system approach.

Seven full-scale shear wall tests were carried out to calibrate 2-D finite element models. The model predictions and the full-scale results agree quite well. The

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study concluded that the simplistic concepts of shear wall behaviour derived from isolated wall observations are invalid. The model predictions show features such as the rotation of the sheathing panels, the bending of the top-beam and the uplift of the studs, as observed during the tests. Finally, a detailed 3-D finite element model of a complete building was idealized, and the computed predictions were compared with full-scale data results published in the literature. This example validates that the modeling approach for shear walls is suitable for finite element simulation of load paths within three-dimensional, realistic wood light frame construction, rather than simply being able to replicate behaviour of walls in isolation.

Key Words: Shear walls, timber construction, modeling, load paths.

Introduction

Conventional wood light frame walls are very easy to fabricate and erect, and their relative low cost in North America has bolstered this type of construction, especially for housing. Most shear walls in residential buildings are constructed according to the traditional prescriptive rules, such as those given in Part 9 of the National Building Code of Canada (IRC/NRC 1995). Typically, shear walls consist of members of dimension lumber spaced no further than 610 mm apart combined with nailed sheathing such as plywood, wafer board and oriented strand board (OSB).

Shear walls provide strength to resist horizontal loads on buildings, typically from wind and earthquakes. Design of such components for gravity is very well understood especially since the estimate of the load is easily predicted. On the other hand, recent advances have been made in earthquake engineering, but wind response is very complex and still needs to be researched. The diversity of construction products and complexity in wall openings introduced in recent times has led to construction details that deviate from the conventional practice. This shift has made it more challenging to describe the behaviour of light-frame structures and has resulted in the traditional strength and stiffness studies on walls without openings being insufficient. The analysis is simply not sophisticated enough to be realistic. An example of a more realistic approach is one that would consider the stud ends as being semi-rigid and not pinned as currently assumed. Also, it is important to recognize the interaction between the studs and the sheathing in a system approach. This can only be achieved by understanding the behaviour of the structural system as a whole and the behaviour of the interface connections in particular, and how the forces are transmitted through the structural system.

Project Goal

This study combines the experimental tests and numerical modeling of unblocked wood shear-wall specimens with various configurations and loadings. A detailed finite element modeling approach is presented for shear walls and a complete single-story house. The goal is to model both the response of the shear walls in isolation and to predict the response and flow of the forces in complete buildings that contain shear walls.

Methodology

Seven series of full-scale shear wall tests were performed at the Forintek Canada Corp. facility in Quebec City, Canada. The purpose of the tests was to understand and replicate the behaviour of wooden shear walls. In particular, the effects of sheathing, openings and anchoring details to the foundation were studied.

The standard shear wall test protocol described by ASTM E564 /ASTM E72 (ASTM 1998 a and b) requires that the wall be restrained from moving vertically as well as out-of-plane. In reality, a wall in a structure is subject to deformation modes such as rigid body rotation as well as racking. For this reason, a different testing procedure was adopted.

The shear walls were constructed especially for the project, in accordance with the Canadian Mortgage and Housing Corporation specifications (CMHC 1991): they are typical for North American residential applications.

Every element was identified and its material properties were established. For wall studs, the modulus of elasticity (MOE) was determined. Bending and shear tests were conducted on coupons for each individual sheathing panel after the test. Some connection tests, such as nail bearing capacity and nail withdrawal capacity, were also performed. The modeling approach adopted proceeds with increasing levels of details and complexity. The first step is to predict the behaviour of a single shear wall element. Then a sub-system consisting of an assembly of single elements is studied and finally a whole building model is constructed. By combining all the individual components with their properties measured in the laboratory, a 2-D finite element model is used for nonlinear static analysis and validated with the shear wall test data. The 2-D model is extended in three dimensions and checked once again. This 3-D model is later used to predict the response of a whole structure.

Material property tests

All components used in the full-scale shear wall tests were characterized to assess their essential static mechanical properties, which also served as input to the finite element models. The characterization tests were carried out at the Forintek Canada Corp. laboratory in Ste-Foy, Quebec, and at the Wood Science and Technology Center (WSTC) in Fredericton, New Brunswick.

The modulus of elasticity of each stud (a total of 112 specimens) was determined using a nondestructive dynamic response test (Ross et al. 1991). The studs were tested in their two principal transverse directions. The average MOE for the flat-wise and the edge-wise directions was 9625 MPa and 9590 MPa, respectively. For both cases, the coefficient of variation was about 20%. All wood was preconditioned to the laboratory environment and the moisture content (11-13% MC) was maintained throughout the test.

For the OSB panels (construction grade), the shear modulus in the edgewise in-plane direction and the static modulus of elasticity were determined. The OSB panels were tested according to the ASTM D 1037-99 (ASTM 1999). The specimens were cut from undisturbed portions of the OSB panels after completion of the full-scale tests.

At first, edgewise shear tests were performed on four coupons 254x89 mm (10"x31/2") from a single OSB panel to verify that the variation within a single panel was low. In total, 17 specimens were tested. The modulus of elasticity for each OSB panel was established from the bending test described in ASTM D1037-99. Since OSB is not an isotropic material, the MOE of each panel was determined for bending about the major and minor in-plane axes. The test specimens were 75x315 mm coupons; the test set-up is shown in Fig. 1.

The OSB test specimens had an average density of 600 kg/m^3 with a standard deviation of 25 kg/m^3. Table 1 summarizes the measured material properties of the OSB panels. The results indicate that the variation in shear modulus is small (COV less than 10%), both within a single panel and also between the panels. This is due to the consistent distribution of the wood wafers in the panels. The tests also confirm that the modulus of elasticity is highly dependent on the panel orientation.

Connection tests

The connection tests were performed at Forintek Canada Corp. and at the Wood Science and Technology Center of the University of New Brunswick. Both frame-to-frame connections and sheathing-to-frame connections were tested in the appropriate directions, as well as withdrawal tests. A typical example of the connection test results is shown in Fig. 2.

Full-scale shear wall tests

Shear walls have been studied extensively during the past decades (e.g. Gupta 1981, Ceccotti 1990, Dolan 1989, Foliente 1994 and 1997). The main purpose of these studies was to establish the ultimate shearing capacity, or a relationship between the applied load and the resulting deflection. In this study, the purpose is to generate load-deflection data and failure mode observations that verify the finite element models. There was no attempt to statistically characterise the shear wall capacities. As the properties of components were uniquely defined for each specimen, it was deemed sufficient to build only one replicate of each wall configuration.

Test set-up and procedure

Seven wall configurations were tested, as summarized in Table 2. All of the walls were 2.4 x 2.4 m, with 38 x 89 mm (nominally 2 by 4) grade no. 2, Spruce-Pine-

Fir studs, spaced at 400 mm (16 in.) on centres. The studs were end-nailed using 75 mm (3 in.) common spiral nails. Walls nos. 2 through 8 had two 1200 x 2400 mm (4 x 8 ft.) 11.1 mm (7/16 in.) OSB panels nailed to the frame with a 150/300 mm (6/12 in.) nail pattern (150 mm on centres at the edges and 300 mm on centres in the interior of the panels). The sheathing to framing nails were 2-3/8 in. long power driven nails.

Figure 3 shows a schematic of a typical shear wall test set-up with anchoring details, loading points and the location of the instrumentation. All tests were carried out on a MTS test machine capable of applying loads in the horizontal and vertical directions simultaneously. The forces were applied to a steel square-tube beam bolted to the wall using two 12 mm bolts. The wall was restrained only against out–of-plane translation, thereby allowing rotation. The piston that supplied the racking force moved horizontally in parallel with the beam. A series of rollers inserted between the loading beam and a rigid steel beam on top of the wall allowed free horizontal motion of the wall. The horizontal force was applied by an actuator that allows vertical movement to retain its horizontal alignment.

All walls were anchored at their base using 12.7 mm (1/2 in.) steel bolts driven into the support beam under the wall. There were steel washers (1-3/8 in. diameter) under the heads of the bolts. Additional steel brackets, built specifically for the tests, were used in some configurations as tie-downs to anchor the stud and bottom plate to the strong floor.

Four LVDT's were used to measure the deformation of the shear wall at the locations shown in Fig. 3. Two LVDT's were added to measure the deformation of the window in test nos. 5 to 7. The lateral displacements of each stud were monitored using digital dial gauges. These were read manually for every load increment of 2 kN. Digitized data were collected at a sampling rate of 1 Hz.

Results

Full scale tests

As indicated in Table 2, Wall No. 1 consisted of frame members only with no sheathing panels attached. As expected, the behaviour of this wall is very different from that of the other walls in the test series. The initial stiffness of 8.7 N/mm as well as the ultimate lateral load of approximately 0.3 kN are very low (Table 3). The wall showed very little stiffness and no sign of in-plane rigid body rotation. There was only racking deformation, and no uplift in any of the studs was observed. The test was stopped when the wall reached the maximum allowable horizontal displacement of 100 mm in the set-up.

Wall No. 2 was sheathed with two vertically placed OSB panels. The test set-up is shown in Fig. 4, and the lateral load vs. displacement curve is shown in Fig. 5. Wall No. 2 showed nonlinear softening horizontal behaviour from the very beginning of the test. The wall had two modes of deformation: racking and rigid body rotation. In the initial part of the loading, the wall was deformed by almost pure racking, but after approximately 20% of the ultimate capacity was reached, a more complex combination of racking and rigid body rotation was observed.

There were no signs of failure in the studs as well as no evidence of frame-to-frame connection separation or failure during this test. This suggests that the uplift forces developed during the test were not large enough to create separation. The predominant failure was observed in the sheathing-to-lumber connections. Large plastic deformations were observed in the nails connecting the sheathing to the lumber as well as shearing of the sheathing panels near the edges. Figure 6 shows the relative rotation of the panels in the centre of the shear wall specimen.

Wall No. 2 has obviously a more complex horizontal load transfer mechanism than Wall No. 1. The load is first distributed in the stiff top plate. Horizontal movement of the wall causes the OSB panels to rotate in-plane around their centroidal axes. The nailing to studs restrains the OSB panels from out-of-plane bending and buckling. The bottom plate is bent and finally the axial forces are transferred through the anchors to the strong floor.

Wall No. 3 had a door opening (1938 x 838 mm) but no anchoring other than hold-down bolts. As expected the wall had a much lower ultimate capacity (9.3 kN) than the reference configuration No. 2 (20.8 kN). Two lintels of dimensions 38 x 140 mm (nominal 2 by 6) were installed above the door opening, thus restraining any bending deformations.

Wall No. 4 had a door opening as well as four tie-down anchors placed inside the bottom corners of the wall and on the outside of the door opening as shown in Fig. 7, for a total of eight anchoring points including the tie-downs. This wall reached an ultimate capacity of 11.2 kN. The initial part of the load-deformation curve was almost linear with a stiffness of 380 N/mm.

The many anchoring points forced the dominant mode of deformation to be racking; at 50 % of the ultimate load, the relative vertical movement of studs was still less than 1 mm.

Wall No. 5 had a window opening (900 x 762 mm) but no additional anchoring. The ultimate load was 10.9 kN, and the dominant deformation mode was in-plane rigid body rotation. The stud in the lower left corner had an uplift of 38 mm at the end of the test and the end nails at that location almost completely pulled out, as shown in Fig. 8. There was also failure of the nailing between the panel and the bottom plate in this corner. The stiff behaviour of the headers above the window confirmed the observations made in test Wall No. 3.

Wall No. 6 is similar to No. 5 but with tie-down anchors in the two bottom corners. The wall sustained an ultimate load of 12.1 kN and the initial stiffness was estimated at 560 N/mm. Rotation of the OSB panels caused vertical (uplift and downwards) movement of the studs. However, the tie-down-anchor in the lower left corner limited uplift at stud No. 1, therefore causing the largest uplift, 12 mm, to occur at the interior stud No. 2.

Wall No. 7 had the same layout as Wall Nos. 5 and 6, except that the lower left corner was fixed to prevent uplift, as shown in the detail of Fig. 9. Accordingly, the ultimate capacity was increased to 15.3 kN, and the deformation mode was almost exclusively racking deformation.

Comparison of the full scale wall test results

Table 3 lists the initial lateral stiffness and the ultimate load obtained for all the specimens tested. Predictions of the numerical models are also shown for comparison.

Effect of openings

The influence of openings can be observed by comparing Wall Nos. 2, 3, 5 and 4, 6, which had the same anchoring arrangement but different opening layouts. As expected, the wall that was stiffest and strongest is Wall No. 2 with no openings. The least stiff and strong is Wall No. 3 with a door opening and no additional anchorage.

The stiffness of Wall No. 3 was found to be only about 56% of the stiffness of the wall with no openings (Wall No. 2). The ultimate capacity was also affected by the presence of the opening. Even though the window opening consisted of 28% of the total wall area, the ultimate capacity was about 45% of that of wall No. 2.

Similarly for the wall with a window opening, the stiffness was about 75% and the ultimate load, 52% of the values obtained for the reference Wall No. 2. The door opening in Wall No. 3 was approximately 2.3 times the surface of the window opening in Wall No. 5. The larger opening affected mainly the stiffness, but also the ultimate capacity to a lesser extent. The racking stiffness of Wall No. 3 was only 75 % and the ultimate load was 85 % of that of Wall No. 5.

Wall Nos. 4 and 6 also had the same anchoring, but different opening sizes. Again, a reduction in both initial stiffness and ultimate load is expected when an opening is created and when its size is increased. The initial stiffness of wall No. 4 is 77 % of Wall No. 6.

A first level design approach is to ignore parts of the wall length with openings. Table 4 shows the reduction in strength and stiffness due to a door or window opening. It is clear that neither the strength nor the stiffness are being reduced by the same amount as the wall effective length is reduced. This suggests that the simplistic concept of shear wall behaviour according to its effective length is invalid.

Effect of anchoring

To study the effect of anchoring, walls with the same layout and opening size but with different anchoring systems are considered. These groups are Wall Nos. 3 and 4 with a door opening, and Nos. 5, 6 and 7 with a window opening.

The additional tie-down on Wall No. 4 caused the ultimate capacity to increase by 20 % and the initial stiffness to increase by 21 %. Wall No. 3 experienced rotation at a low load level whereas wall No. 4 had racking as its main mode of deformation for almost the entire duration of the test. However, the header block over the door opening behaved in the same way in both tests.

Wall Nos. 5, 6 and 7 all had an opening the size of a window. The additional anchoring used on Wall Nos. 6 and 7 led to an increase in the ultimate capacity of 11% and 40 %, respectively. Also due to the additional tie-downs, the initial stiffness of Wall Nos. 6 and 7 was higher than that of Wall No. 5; the increase was 35% and 45%, respectively. There was only 7% increase in the stiffness by adding the hold-down bracket on the outside of the wall. This bracket provides a direct link between the wall stud and the foundation and it minimizes the rotational deformation of the connection.

FE-modeling

Background

Numerical modeling is an ideal complement to laboratory testing, and essential to proper interpretation and extrapolation of the experimental findings. Analytical models include detailed finite element models of substructures and inter-component connections, and global models of the entire building systems. Multi-degree-of-freedom (MDOF) models of subsystems like shear walls require material and connection parameters that are obtained from experiments conducted on matched specimens. These models are used to establish the load-deformation and stiffness characteristics that will be used in global (whole structure) models. In this study, the software selected for the finite element modeling is SAP2000 Non-Linear Version 8 (CSI 1997).

In the past decades, many attempts have been made to analytically describe wood shear walls and other structural sub-systems (e.g. Foschi 1977, Itani and Cheung 1984, Falk and Itani 1989, Dolan 1989, Paevere 2002).

It is important to establish whether or not it is possible to predict the response of an assembly of walls in the structure based on a 2-D wall model. The most critical issue in shear wall design is the proportioning of the force to the various shear walls that can resist the loading. To assess this, it was decided to use data from a full-scale L-shaped house test structure (Paevere 2002). That work is unique in that the reactions that represent the shear flow beneath the walls were directly measured. Detailed analysis of the structure was performed but because it is a complex topic in its own right, attention here is restricted to the shear flow.

Modeling procedure

The finite element models were built by representing all physical elements in the structure. Figure 10 shows a typical example of the detailed modeling of a shear wall.

Linear "frame" elements are used to model all the ribs (studs, top and bottom chords and lintels) in walls. All frame elements in the 3-D whole structure model use three-dimensional beam-column elements, which include the effects of biaxial bending, torsion, axial deformation and biaxial shear deformations. The sheathing panels were modeled as linear "shell" elements. A series of elastic orthotropic material properties are assigned to both the frame and shell elements. The modulus of elasticity, shear modulus and Poisson's ratio can be defined in all three directions of the material. Nonlinearity was included in the connections, represented by nonlinear links (option "NL-Links" in SAP 2000). Strength degradation of the connections is included by using a multi-linear load-deformation function fitted to the experimental results such as those presented in Fig. 2 (Mi 2004).

The NL-Link is used to connect the frame elements to each other and the shells to the frames. As shown schematically in Fig. 11, the NL-Link is a link between two joints ***i*** and ***j*** and consists of six independent linear or nonlinear springs per joint; the springs that are not shown in Fig. 11 represent torsion, bending and shear in the perpendicular plane.

Shear wall tests reported here are linked to a study on a full-scale experimental house currently monitored at the University of New Brunswick (Doudak et al. 2004). These tests form part of a comprehensive research project dealing with load paths in wooden light-frame buildings.

Shear wall modeling results

Figure 12 shows a graphic example of the comparison between the load deflection curve for the test walls and the corresponding prediction of the finite element model. The load is the horizontal racking force and the horizontal displacement is taken at the top right corner as measured by LVDT 1 on Fig. 3 and corrected for any movement in LVDT 3.

Generally, there is good agreement between the full scale test data and the model prediction. The model prediction seems to be inaccurate only in the case of Wall No. 1 with no sheathing applied. As mentioned before, the wall cannot transfer the shear forces when there is no sheathing applied to it; this causes a large slip in the connections, which the model cannot predict. The overall trend in the behaviour is obtained analytically, however, the quantitative response is not accurately predicted.

Table 3 summarizes the results from both the full-scale tests and the model predictions in terms of ultimate capacity (maximum horizontal load) and initial stiffness (slope of the load-deflection curve at the origin).

For the ultimate capacity, the model predictions and the full-scale test results agree very well in all cases. However, the prediction of the initial stiffness is not as accurate in some instances. This is attributed to "shake down" effects in experiments at low load levels, which make it very difficult to accurately define the initial trajectory of the response. The results of the full-scale tests, especially at low load level, are also affected by workmanship, i.e. variability in the tightness of the bolts, nailing spacing and alignment, etc. Overall, agreement in results is considered reasonable.

An interesting feature of the finite element model is that it can predict the location of the failure mechanism and thereby allowing design solutions to avoid premature damage. An expected result from the shear wall tests was that the uplift in the lower left corner was so large that it resulted in the base connection failure; the model correctly predicted this result.

Although finite element models have the capability to compute and predict the force flow within any structural system, these results could not be verified since force sensors such as load cells and strain gauges were not utilized in the tests. To demonstrate this capability to predict the load paths in the structural system, a complete 3-D model of a single-story house tested at the Commonwealth

Scientific and Industrial Research Organization (CSRIO) in Melbourne, Australia (Paevere 2002), is presented next. The CSIRO test building was equipped with a series of three-directional load cells at the floor-to-base interface. In addition to predicting the force flow in the structure, the 3-D model verifies the approach used in the shear wall models and confirms its validity for applications to realistic wood light-frame constructions.

3-D model: The CSIRO test house

The CSRIO single-story wooden house had an L-shaped plan, with a footprint of approximately 95 m^2. The framing of the house consisted of 38 x 89 mm studs spaced at 400 mm on centres. The roof consisted of light frame trusses spaced at 600 mm centres. The house had gypsum boards on the inside of the walls and on the ceiling, and plywood sheathing boards on the outside. The house had various types of openings such as windows, pedestrian doors and a large garage door. More detailed information about the structure is presented by Paevere (2002). The house was loaded at several locations by a lateral static point load and the deformations were limited so the response was kept within the elastic range.

The 3-D finite element model of the CSIRO house was idealized following the same approach and assumptions used for the detailed 2-D shear wall models. Most of the necessary property data was estimated from the literature (Paevere 2002), but some of the properties had to be approximated from generic material properties. Figure 13 shows the 3-D model with only studs (Figs. 13 a and b), and with studs, sheathing and NL-links (Fig. 13c), and finally a detail of a corner connection (Fig. 13 d). This detail illustrates the complexity of the model where each connection, nail, bolt etc., is considered. Such meticulous detailing is very useful when the force distribution is considered, and it allows studying various changes in the configurations in the connections and the studs.

The lateral loads applied to the structure are presented schematically in Fig. 14 (plan view). Table 5 compares the reactions measured underneath the four main load transferring shear walls (W1 to W4 in Fig. 14) and those obtained from the 3-D SAP2000 model.

As indicated in Table 5, the predictions of the finite element model are within 20% in error. Errors are primarily attributed to uncertainty about the true material properties of the components and connections in the test structure. A higher level of accuracy, such as the one obtained for the shear wall tests could have been obtained if the material properties of the individual components had been known accurately. This example demonstrates that the detailed finite element model is capable of predicting the distribution of the applied load through the house structure with reasonable accuracy.

Conclusion

Seven full scale shear wall tests were carried out to calibrate 2-D finite element models. It was shown that the drop in initial stiffness observed is reduced in the presence of a tie-down at the base. The tests indicate that the increase in capacity and stiffness depends on the size of the opening. It is clear that neither the strength nor the stiffness are being reduced in the same proportion as the wall effective length is reduced. This leads to the conclusion that the simplistic concept of effective length to describe shear wall behaviour is invalid.

Generally, the model predictions and the full-scale test results agree quite well. The model prediction error is within 20% for the initial horizontal stiffness. The model predicted the ultimate strength with higher accuracy. This is expected since the initial stiffness of systems is notoriously difficult to measure experimentally or to predict, with nonlinearity present even at early stages of loading.

In addition to predicting the ultimate capacity and the initial stiffness of the shear walls, it is also important to predict the deflection response. The model predictions show features such as the rotation of the sheathing panels, the bending of the top beam and the uplift of studs, as observed during the tests.

A detailed 3-D finite element house model was built to simulate the sharing of racking forces between shear walls, based on experiments reported in the literature. Considerations were restricted to the initial stiffness (linear elastic) response. The model predictions are within 20% of the full-scale test results, which is consistent with the findings for isolated shear walls. Based on all of the comparisons reported here it is shown that detailed finite element models of isolated wood shear walls are capable of acceptable predictions of stiffness responses and failure loads.

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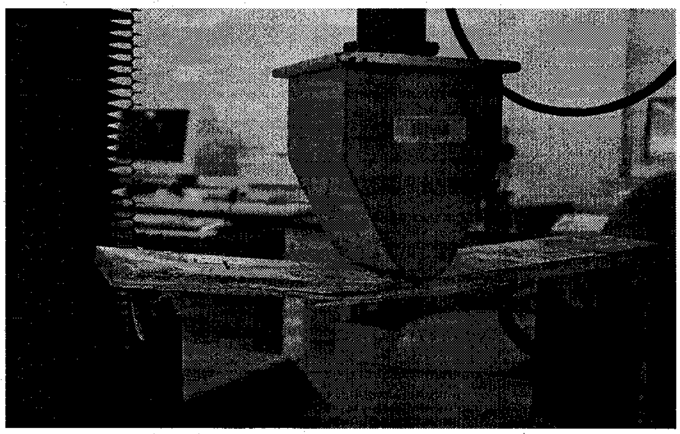
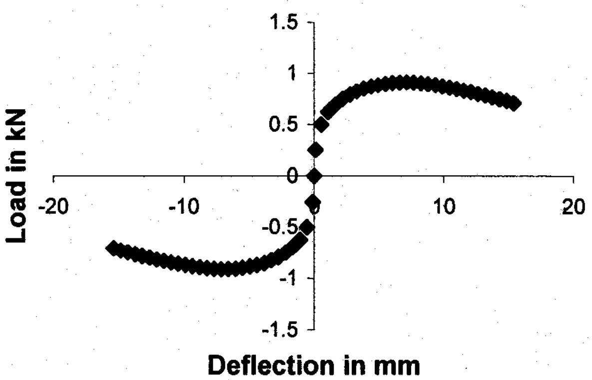


Figure 1. Bending test of OSB coupon



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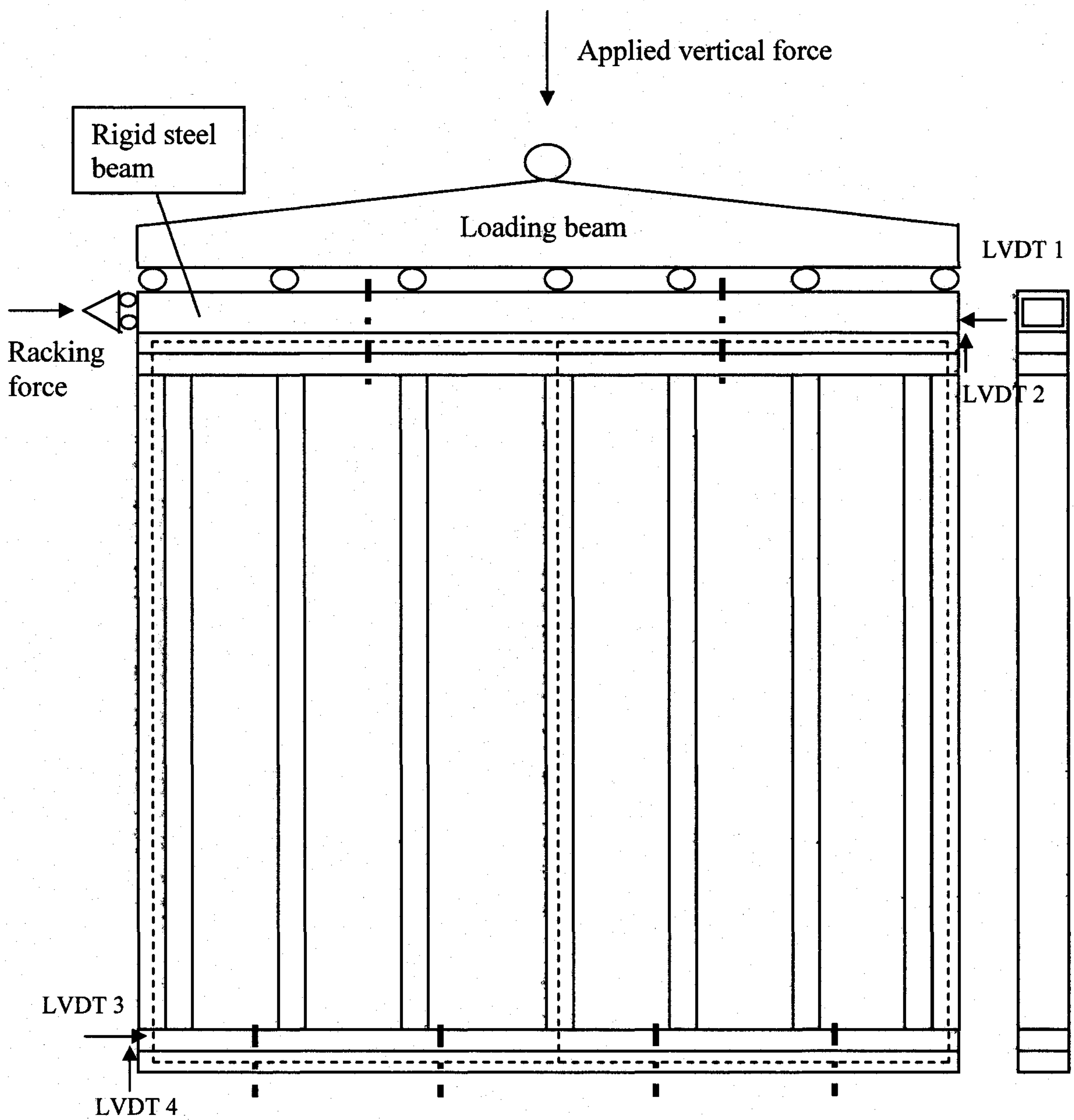

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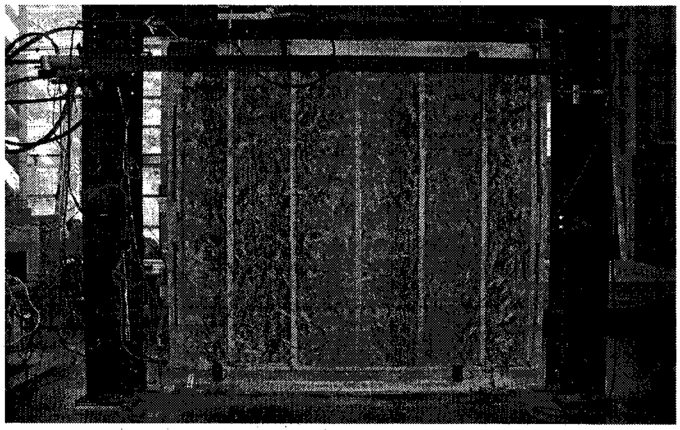

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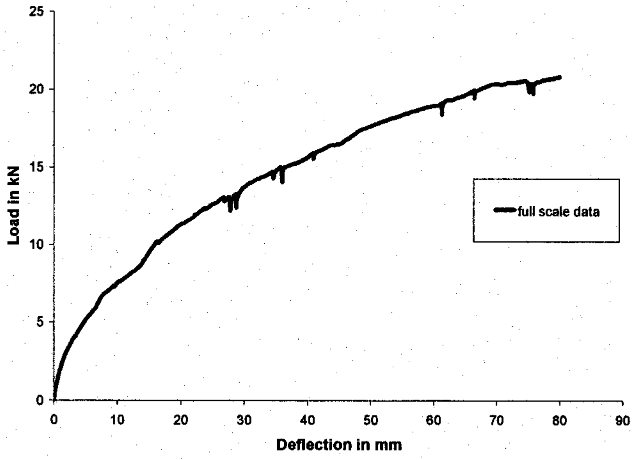

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Figure 6. Relative rotation of OSB panels in Wall No. 2 (scaling in cm)

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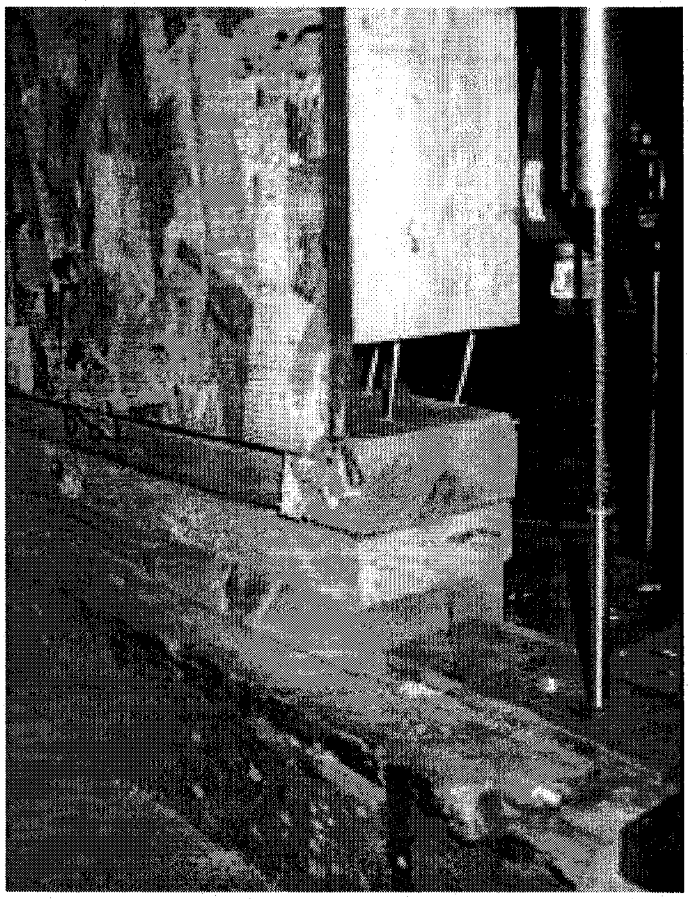

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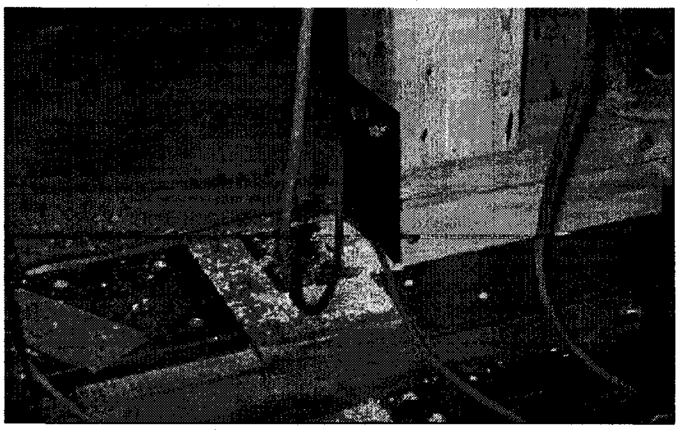

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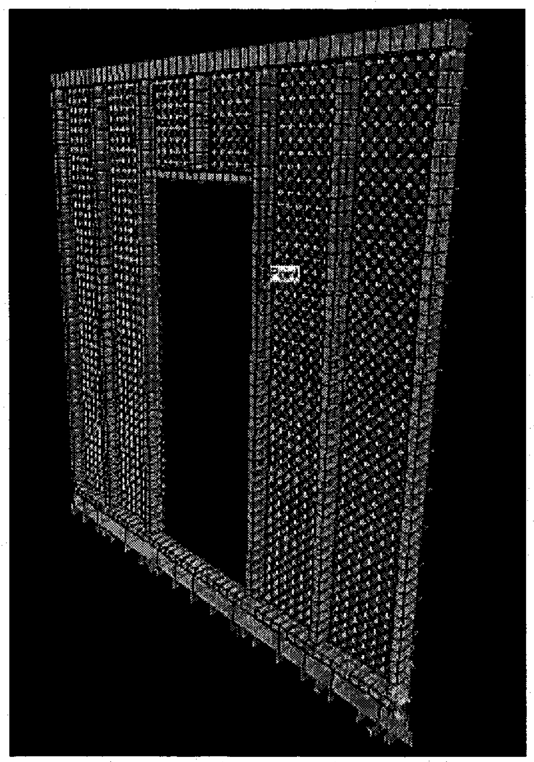


Figure 10. Example of mesh showing detailed modeling of all physical elements of shear Wall No. 4

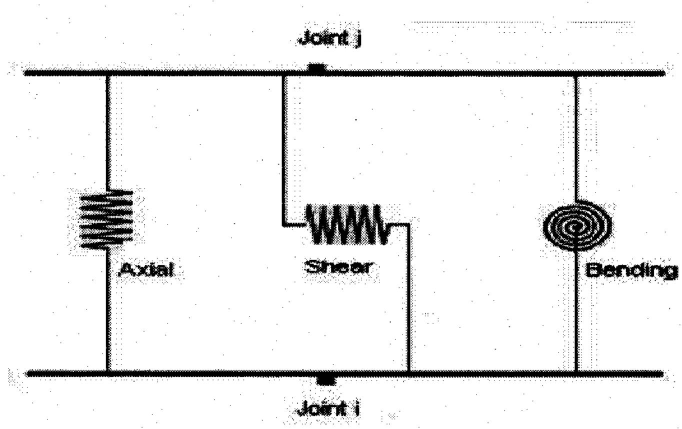
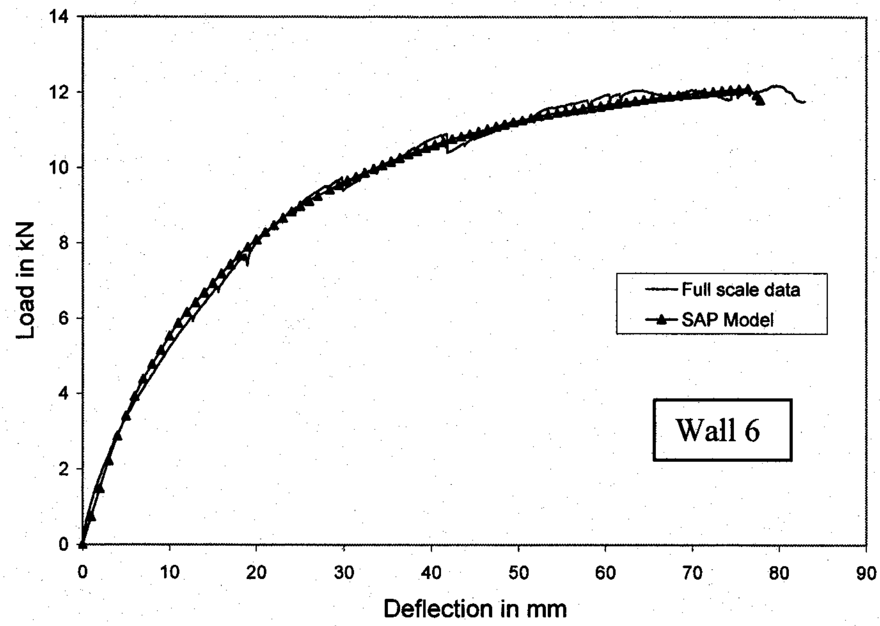


Figure 11. Three of six independent springs in an NL-Link element (CSI 1997)



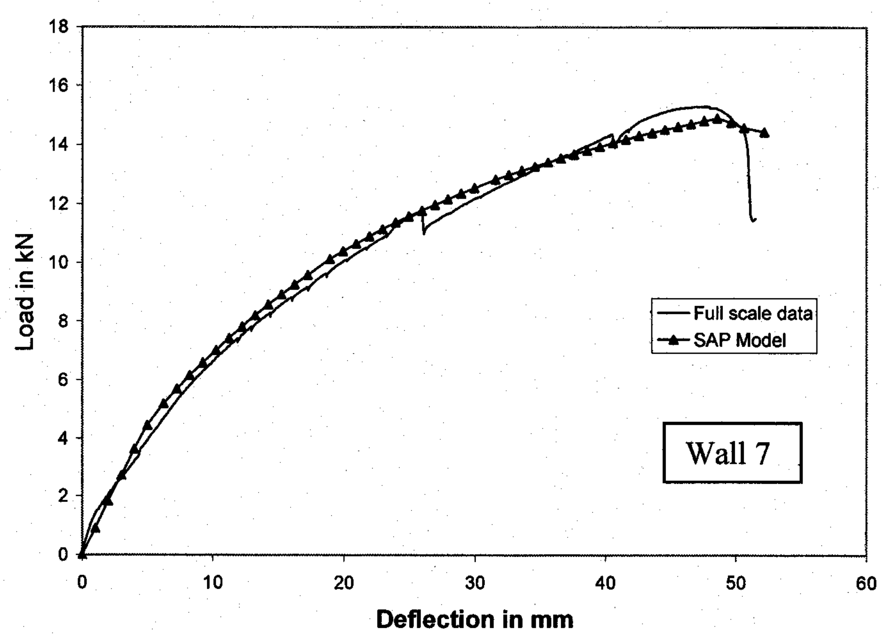


Figure 12 a-b. Experimental load-deflection versus model prediction

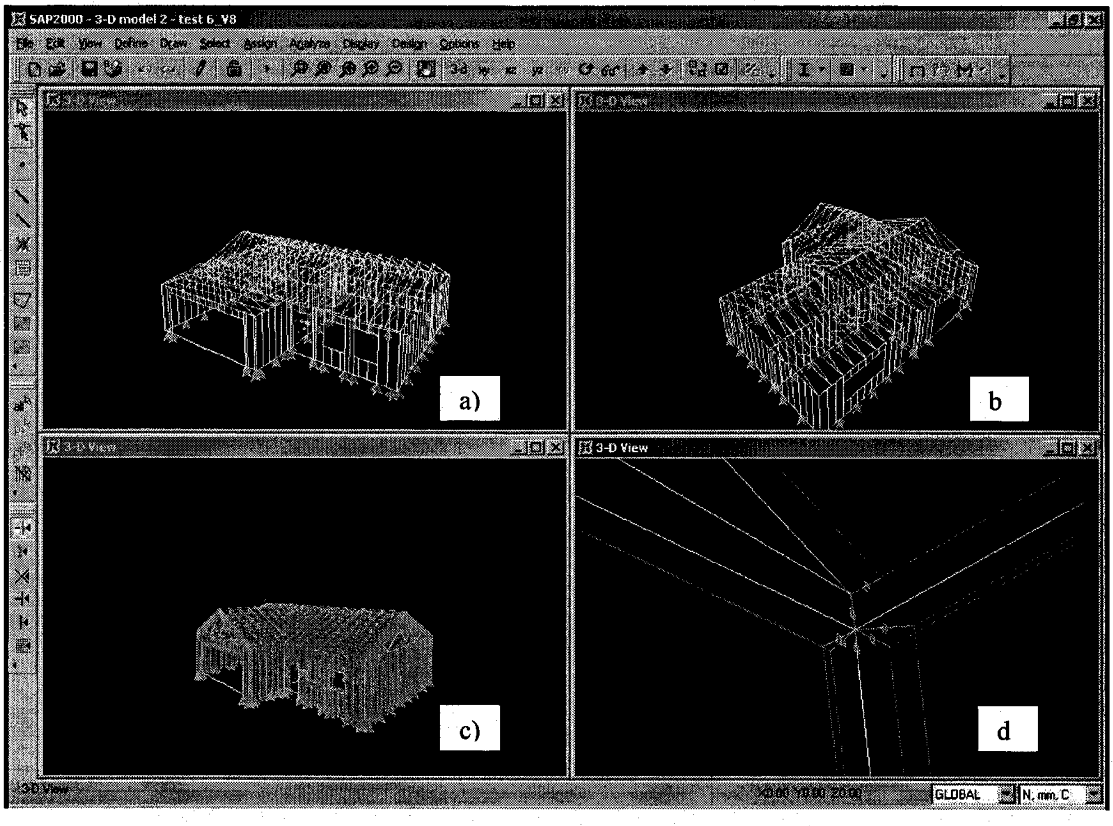


Figure 13 a-d. Detailed finite element model of CSIRO Test House

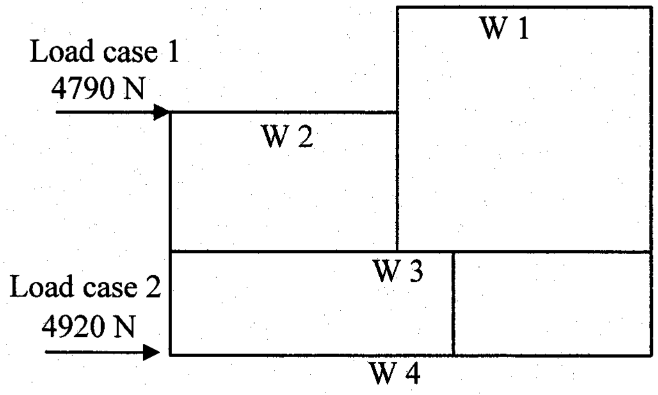


Figure 14. Plan view of loading configurations of CSIRO Test House

Table 1. Material properties of OSB panels

Property **(1)**	**Average Modulus** **[MPa]** **(2)**	**COV** **%** **(3)**
Edgewise shear : G	790	10
Parallel bending : MOE	5100	13
Perpendicular bending: MOE	1600	13

Table 2. Overview of full-scale wall test setups

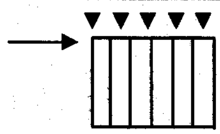
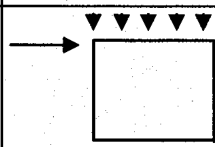
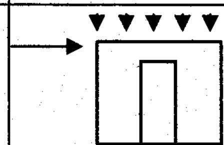
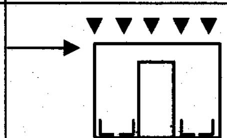
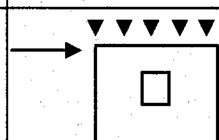
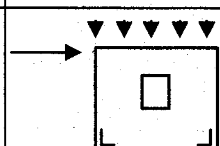
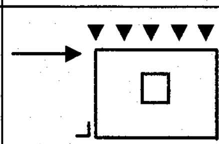
Test No.	Setup	Description
1		No OSB panel (framing only). No anchoring. Hold-down bolts only 5 kN vertical force (2.08 kN/m)
2		2 OSB-panels. No additional anchoring. 10 kN vertical force (4.16 kN/m)
3		2 OSB-panels. Door size opening (1938 x 838 mm). No additional anchoring. 5 kN vertical force
4		2 OSB-panels. Door size opening (1938x838 mm). Anchoring in the inside bottom corners and door. 5 kN vertical force.
5		2 OSB-panels. Window size opening (900 x 762 mm). No additional anchoring. 5 kN vertical force.
6		2 OSB-panels. Window size opening (900 x 762 mm). Additional anchoring in the inside bottom corners. 5 kN vertical force.
7		2 OSB-panels. Window size opening (900 x 762 mm). Additional anchoring outside the lower left-corner. 5 kN vertical force.

Table 3. Comparison between full-scale test results and finite element model predictions

Wall No. (1)	Maximum load (kN)			Initial stiffness (N/mm)		
	Exper. (2)	Model (3)	Predic. error % (4)	Exper. (5)	Model (6)	Predic. error % (7)
1	0.3	0.3	0	8.7	8.4	-3.4
2	20.8	20.1	-3.4	558	695	24.6
3	9.3	9.3	0	313	326	4.2
4	11.2	11.4	1.8	380	392	3.2
5	10.9	10.6	-2.8	416	544	30.8
6	12.1	12.2	0.8	562	550	-2.1
7	15.3	14.9	-2.6	603	737	22.2

Table 4. Effect of openings

Wall No. (1)	Maximum load (kN)		Initial stiffness (N/mm)	
	Exper. (2)	**Reduction % (3)**	**Exper. (4)**	**Reduction % (5)**
2	20.8	-	558	-
3	9.3	55	313	44
5	10.9	48	416	25

Table 5. Comparison between full-scale test results and model predictions for CSRIO test house

Wall	**Load case 1**			**Load case 2**		
	Total shear force CSIRO N (1)	**Total shear force FE model N (2)**	**Model Error % (3)**	**Total shear force CSIRO N (4)**	**Total shear force FE model N (5)**	**Model Error % (6)**
W1	310	333	7.4	67	73	9.0
W2	1933	2303	19.1	438	525	19.9
W3	1098	957	-12.8	1195	1345	12.6
W4	917	955	4.1	2995	2768	-7.6

CHAPTER 4

Monitoring Wind Effects in an Industrial Building

Full-scale testing is essential to predict the behaviour of structures. It is important to establish a relationship between the behaviour of the full-scale and the reduced scale of the structure, wherever possible. A successful application is in the wind tunnel, where a complete and accurate description of the pressure distribution on building surfaces can be obtained. When dealing with detailed structural response, it is difficult to produce a scaled model of the structure that follows all the essential rules of similitude to reproduce the effects of the real physical and mechanical properties of materials, details of structural geometry and the details response of structural form and idealized loads, which simulate the "real" loads on the structure.

The chief advantage of having both the full-scale monitoring program and the wind tunnel tests is that the full-scale data can be used to calibrate the wind tunnel model. Both full-scale data and wind tunnel data can then be used as loading data or input for the finite element model. The model can in turn be used as a prediction tool for different assumed loading scenarios.

The research presented in Chapter 3 established the feasibility of predicting the behaviour of 2-D wall subsystems. This chapter investigates the feasibility of structural monitoring on an existing building and compares the measurements to a reduced-scale wind tunnel model. In addition, the research establishes the feasibility to predict the whole structure behaviour based on 3-D finite element analysis. The goal is to combine the knowledge on the 2-D shear wall study with the 3-D modeling to establish a monitoring study on a new building as will be seen in later chapters (Chapters 5 and 6).

Monitoring wind effects in an industrial wood light-frame building

By G. Doudak[1], G. McClure[2], I. Smith[3], T. Stathopoulos[4]

Abstract: An emerging trend is the acceptance that the performance of whole buildings can only be reliably assessed via full-scale tests. This paper describes the structural monitoring experiments on a single storey industrial shed building located in Quebec City, Canada. The structural response monitoring is based on measurements of deformations within a representative segment of the wall and roof surfaces that constitute a continuous strip encompassing three adjacent roof joists and the three supporting wall studs at either end of those joists. Wind pressure was measured by a grid of 20 wall and roof wind pressure taps, with a matching set of LVDT's measuring the structural response. The pressure measurements on the envelope of the building in full scale were supplemented with a wind-tunnel study.

Both mean and peak pressure coefficients, C_p, measured on the envelope of the building compare well with the values obtained in the wind tunnel. It was found

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that the ratio between peak and mean values varied between 2.5 and 4 with a mean value of about 3. This ratio is higher for higher C_p values.

The results from both 2-D and 3-D finite element models for the structural response of the building agreed with the full-scale data. It was shown that more joists than assumed in traditional design are participating in the load sharing. Also, the results indicated a large degree of load sharing in the wall. Analytical and finite element numerical models were built to represent the structural response of the building to wind load. The key findings were that analytical models of ribbed plates are capable of very accurate predictions of the response of the structure.

Key Words: Wind loads, wind tunnel, mean pressure, peak pressure, load paths, monitoring, timber construction.

Problem definition

In designing wood frame structures, it is assumed that the environmental loads (wind, snow) can be represented as surface pressures on sheathings, with loads on supporting members being proportional to their projected tributary areas. This simplified approach is reasonable in situations where structural subsystems are essentially statically determinate, but this is rarely the case in light-frame structures. Other related issues are the current arbitrary nature of decisions about how the roof and floor subsystems distribute the load laterally to the shear walls, and there is an uncertainty about the contributions of the architectural elements to the building response to the applied loading. Uncertainty, and presumably inaccurate expectations, about how loads flow through buildings are common.

Wood structures are built to take advantage of the interaction between the members in the system which must be considered in any structural analyses. Present design criteria (CSA O86 2001) include system effects in a cursory manner, or not at all. The one area where system effects are explicitly included is for parallel member wood systems where the "repetitive member use factor" is used to adjust the allowable stress. The structural response of parallel member wood systems makes such system analysis necessary.

Project goals

Research was initiated to monitor the structural response of a single storey light-frame wooden industrial shed building located in Sainte-Foy, Quebec, Canada, to environmental loads. The long term goal is to determine how the forces induced by the environmental loads flow through the selected light-frame wooden building and similar other structures, requiring determination of load paths and the correlation of observations to the prediction from the models that embody various levels of sophistication. The immediate objectives were:

- To investigate the feasibility of full-scale wind measurement in terms of pressure on the walls and roof

- To compare data from full-scale monitoring with wind tunnel data generated on a model of the shed and the simulated surroundings
- To relate deflections measured in the full scale structure to those calculated from the finite element model.

Background

Wind

Wind measurements have a great importance in several fields, including engineering. For example, the dynamic response of tall buildings to high wind speeds has long been a serious concern. However, in North America light-frame wood structures comprise the majority of the building stock most vulnerable to high wind hazards. In recent years, hurricanes and other natural hazards have caused extensive damage to these types of construction. Hurricanes Hugo in 1989, Andrew in 1992, and Opal in 1995 have served to increase the awareness of the vulnerability of light-frame structures to wind damage (Crandell and McKee 2000).

The goal of this research is to contribute to better understanding of the problems of wind-structure interaction. Some of the major parameters that affect this interaction are wind characteristics, aerodynamics of the structure, wind loading on main components and claddings, as well as structural response to the wind loads.

Reference pressure

The errors usually associated with the measurement of reference pressure are quite complex. Previous work (Levitan 1992) discussed the issues of reference pressure in details. Several systems can be utilized to establish the reference pressure. One could imagine using a constant reference pressure from a Pitot tube, but this can be applicable only to one-dimensional flow, such as that in the wind tunnel. Since field applications deal with three-dimensional wind flow, that technique cannot be used. The building internal pressure can also be a valid choice, especially for special types of buildings where the change in atmospheric pressure will be reflected in both the external and internal pressures. In fact, it is almost impossible to separate the internal pressure component from the external component. The internal pressure can also be affected locally by air conditioning equipment, but even with a constant internal pressure, it is not always possible to represent the variation in the atmospheric pressure. This observation suggests that the atmospheric pressure is the ideal choice for the reference pressure, further considering that the static pressure is everywhere on the building even when there is no wind (or rather negligibly low wind speed). The approaching wind will induce pressures on the various surfaces of the building; that is, at any point on the surface of the building, the net pressure is equal to the sum of the atmospheric static pressure and the pressure due to wind flow around the building.

The presence of the structure affects the static pressure in the air around the building, and a measuring location must be chosen where the static pressure is not significantly influenced by the flow of air around the structure (Levitan 1992, ASHRAE 2001).

Wind effects on buildings

Even simple building shapes can generate flow patterns that are too complex to generalize for design. Wind effects on any building are also highly affected by its surroundings, such as any adjacent buildings, trees and the like.

The mean speed of wind, V_{mean}, approaching a building increases with height above the ground. The airflow typically separates at sharp edges to generate circulating flow zones that cover the downwind surface of the building (roof, sides, and leeward walls) and extend for some distance into the wake (ASHRAE 2001). It is important to note that for strong wind episodes, streamline patterns are independent of the wind speed and depend only on the building shape and the upwind conditions.

Surface pressures can fluctuate due to the turbulence or gustiness of the approaching wind and the unsteady characteristic of the separated flows. The peak pressure also varies significantly from the average pressure reaching values two, three or more times the mean values. The approaching wind exerts pressures on the building surface that could be positive or negative (inward or outward relative to the interior). This pressure is affected by several variables, such as wind speed and direction, building geometry and surroundings, barometric pressure and the air density. Because of the large variability in several of these factors, a dimensionless factor is used to represent the pressure, C_p. This local surface pressure coefficient is also influenced by the presence of nearby buildings, vegetation, and terrain features (ASHRAE 2001).

According to the Bernouli's law, the time-averaged (static) surface pressure, P_v, is given by:

$$P_v = ½ (\rho_a U_H^2) \qquad (1)$$

where U_H is the approaching wind speed at roof height and ρ_a is the outdoor air density. The difference between the surface pressure on the building, $P_{surface}$ and the atmospheric pressure, P_{atm} is labelled as P_s:

$$P_s = P_{surface} - P_{atm} = C_p P_v \qquad (2)$$

where C_p is the local surface pressure coefficient for the given surface (wall or roof), which depends on the shape of the building and the wind direction, but for strong winds, it is independent of wind speed. Thus (2) can be expressed as:

$$P_s = ½ (C_p \rho_a U_H^2) \qquad (3)$$

It should be noted that U_H can be measured on site or estimated from a nearby meteorological station (U_{met}). In that case, the hourly-average wind speed U_H at the wall height H is:

$$U_H = U_{met}\,(\delta_{met}/H_{met})^{a,met}(H/\delta)^{a} \qquad (4)$$

where δ is the wind boundary layer thickness and a is an exponent characteristic of the upstream building terrain. The total pressure on a wall comprises of the pressure difference between the surface pressure, C_p, obtained from the exterior, and the interior pressure coefficients, C_{pi}.

$$C_{p,total} = C_p - C_{pi} \qquad (5)$$

Full-scale testing

The test building

The test building is located in Sainte-Foy, Quebec City, Canada. The building and its immediate surroundings are shown schematically in Fig. 1.

This shed is an industrial building constructed in 1998 as an extension to a similar structure built in 1994. It is on level ground and has a rectangular plan with outside dimensions, 8.0 m by 15.0 m. The interior is open. The roof surface is flat and 5.1 m above grade, with a 0.5 m high parapet (Fig. 2). The longitudinal building axis is oriented NE-SW. The surrounding area consists mainly of low-rise industrial buildings.

The walls consist of 38 mm x 140 mm S-P-F (Spruce Pine Fir) lumber studs class 1650f-1.5E spaced at 406 mm on centres and sheathed externally with 13 mm thick 1.22 x 2.44 m plywood panels. Sheathing panels are oriented with the long axis horizontal and are attached using 76 mm nails spaced at 150 mm at the edge supports and at 300 mm at the intermediate supports. There is an external 75 mm thick rigid insulation as well as horizontal and vertical, 19 mm x 64 mm (1"x 3") strapping, spaced at 400 mm centres in the vertical direction and 610 mm centres in the horizontal direction. There is a 19 mm x 140 mm (1"x6") drop siding of stained wood (9/16" or 14 mm thickness). Sill and header plates are 38 mm x 140 mm lumber. Horizontal 38 mm x 140 mm lumber blocking is installed between studs at 1219 mm centres (4 ft), starting from the bottom and coinciding with horizontal joints in the plywood sheathing panels. The roof has 457 mm deep wood I-joists sheathed with plywood. Roof plywood is of the exterior type and I-joists are spaced at 406 mm. The flanges of the I-joists have dimensions of 38 mm x 89 mm and the web thickness is 11 mm. All I-joists are seated on light gauge steel hangers and aligned directly with studs in supporting walls.

The building foundation is a continuous concrete strip footing. The footing dimensions are 600 mm wide and 300 mm high. The height of the foundation wall is 1900 mm; it is 200 mm in thickness and raised 200 mm above the concrete floor inside the shed. A 38 mm x 140 mm sill plate is anchored to the top of the foundation wall.

Measurements

The structural response monitoring is based on measurement of deformations within a representative segment of the wall and roof surfaces. To measure wind pressure on the building, a grid of 20 wall or roof wind pressure taps and LVDT's were installed, as shown in Fig. 3a. Apart from the monitoring experiment, load-deformation response under controlled static loads was measured. As shown in Fig. 3b, the LVDT's for measuring roof deflection were referenced from beams suspended from the I-joists at locations near to the joist support points. Both I-joist and sheathing displacements were monitored. For wall deflection measurements, LVDT's were referenced from towers attached to the concrete building foundation (see Fig. 3c).

A wind-measurement station was attached to the Southwest face of the building to record wind speed and direction 10 m above grade level (central picture in Fig. 2). Apart from helping establish the building's response to loads, field pressure tap data is valuable for validation and calibration of the wind tunnel test results. It is noted that the limitations of the data acquisition system made it possible to collect data at a maximum sampling rate of 1Hz.

In addition to the direct wind pressure measurement, wind data from two accredited meteorological stations near the test location were used. One station is located at a small airport west of the building at approximately 5 km and the other is located on Laval University campus at a distance of approximately 2.5 km.

Pressure taps

Description

As indicated in Fig. 3a, a total of 20 pressure taps were installed on the test building surfaces, 14 on the walls and 6 on the roof. The taps are essentially tubes of 4.8 mm inside diameter and approximately 178 mm long. The taps on the walls were mounted flush with the outside surface and sealed with silicon. It was not possible to drill holes in the roof of the shed and therefore the pressure sensors were specially designed and built to fit on plates that were glued to the roof as shown in Fig. 4a.

Sensitive differential pressure transducers were used to measure the surface pressures. The transducers were calibrated in the Wind Tunnel Laboratory at Concordia University, and each transducer was also checked again after its installation on the building.

Ambient atmospheric pressure was used as the reference pressure for the measurements. It was obtained from a box below ground (Fig. 4b) with a small hole of 12.7 mm, located about 25 m from the test building: It was assumed that the building and the reference pressure box are located far enough apart so that the presence of the building had no effect on the static pressure measurement. A 4.8 mm diameter tube transmitted the ambient pressure to the building, and the tubing then connected the reference pressure to all the transducers.

Data collection and treatment

The climate in Quebec City is such that the building experienced a range of wind directions and speeds. Climatic data for this area are collected by Environment Canada's weather office at the Jean-Lesage International Airport (YQB). On a yearly basis, the maximum wind speeds dominate from the South-West and then the North-East directions (see detailed wind rose in Fig. 5). Due to the limitations of obtaining a wide range of values for wind incidences, full-scale data could only be obtained for directions of 90, 100, 110, 260, and 280 degrees due North. These were the only directions in which wind speeds were large (>10 km/h) and the wind was steady in its direction.

The data were sorted by isolating incidences when the local wind speed was exceeding the threshold of 10 km/h. It was also checked that the recorded wind direction was consistent with the entries recorded just before and after it with a tolerance of 5 degrees. Pressure coefficients were calculated based on Equation 3, where the velocity U_H was determined at the roof height and the values of ρ_A were adjusted with ambient temperature. The mean pressure was calculated based on five-minute averages.

Wind tunnel study

The boundary layer wind tunnel

The wind tunnel study was conducted at the boundary layer wind tunnel in the Building Aerodynamics Laboratory at Concordia University, in Montreal. The tunnel has dimensions of 1.8 m width x 1.8 m depth x 12 m length. It is an open-circuit, blow-down type, with an adjustable ceiling and a turntable base, allowing measurements to be taken for all wind directions. A roughness board fitted with egg-carton boxes is placed directly following the inlet. This set-up is designed to develop the boundary layer profile along the length of the tunnel.

Terrain roughness simulation

Roughness boards, as seen in Fig. 6, were placed along the tunnel to simulate the appropriate upstream roughness having similar characteristics with those in the field. Styrofoam cubes of two different sizes were used to simulate the terrains for the so-called suburban (S) and urban (CB) panels.

Measurements

Differential transducers were used to measure the pressure on the surfaces of the model in the wind tunnel. The transducers were then connected to a multiplexer, transferring the differential pressure to the data acquisition software. Velocity measurements in the tunnel were conducted using a hot film anemometer connected to a data acquisition system

Wind tunnel model

The model used in the wind tunnel was scaled at 1:200 compared to the actual building dimensions. Pressure taps were installed on the model at the same locations, corresponding to the tap locations in the real building. The building surroundings were also modeled using Styrofoam cut in the exact shape and size of the surrounding buildings as in the Forintek complex. Further away from the building, roughness panels were also used to model the wind exposure.

From the South-West and West directions, the exposure consisted of a forested area of deciduous trees with an average height of 12.2 m (40 ft). The forest density of this area was approximately 12.5 trees per 1000 square feet (92.9 m^2) or 140 stems per ha, as measured on site. The main Forintek building, shown in Fig. 1, is located on the eastern side of the shed. The first two rows of trees were modeled accurately for similar size and shape, and the rest of the forest was simulated using undulated foam sheets. The main Forintek building model was cut to the exact shape and scaled size, as shown in Fig. 7.

This particular upstream roughness configuration established a velocity profile with a power-law exponent of 0.34 in the wind tunnel (See Equation 4 and Table 1 in ASHRAE (2001)). This was the value determined by the analysis of wind speed records obtained from the Jean-Lesage Airport located 5 km west of the site and the anemometer readings at the test building site.

Testing and calculation

Wind tunnel data was obtained for wind directions at intervals of 10, 15 or 20 degrees, by rotating the turntable in the test section. Data were recorded at a rate of 250 Hz. During testing, a manometer reading was taken to determine the wind velocity in the tunnel at the reference height of 0.6 m (in the wind tunnel), and a velocity meter was used to verify this data. From this compiled data, mean and peak pressure coefficients (C_p) were calculated based on:

$$C_p = P\,k/M_v \tag{6}$$

P = Differential pressure measured by the transducer (Volts).

k = $(V_{ref} / V_h)^2$

V_{ref} = Velocity at reference height of 600mm (Volts).

V_h = Velocity at roof height (Volts).

M_v = Manometer reading (Volts)

The mean pressure measured by the transducer was calculated based on the average differential pressures obtained from the analysis of the complete raw data set. The peak pressure measured was evaluated by considering the most critical peak values from every second of data and averaging them accordingly for the given time period. Wind tunnel testing velocities were recorded between 12.7 and 13.3 m/s at the reference height.

Finite element modeling

Introduction

Numerical modeling is a useful complement to field monitoring and wind tunnel testing. The analytical models include detailed finite element models of the substructures and the inter-component connections, and complete models of the whole structure. The load history obtained from full-scale in situ experiments is used to construct a loading function.

The commercial software selected for the finite element modeling was *SAP2000* Nonlinear Version 8 (CSI 1997).

Modeling procedure

The finite element models were built by representing inasmuch as possible all the physical elements of the structure. Linear frame elements were used to model all the ribs (studs, top and bottom plates as well as lintel beams). The panels were modelled as linear thick plate shell elements. All nonlinearity was included in the connections between these elements represented by nonlinear links. At low load levels, the response of the structure is expected to be linear. However, the model functionality included nonlinearity to enable the prediction of the structure's behaviour at higher load levels. All frame elements in the 3-D model use three-dimensional beam-column formulation, which includes the effects of biaxial bending, torsion, axial deformation and biaxial shear deformations. The shell elements use a general plate formulation that allows in-plane deformations, as well as out of plane deformations. The model can also detect instabilities such as buckling. Orthotropic material properties are assigned to both the frame and shell elements to represent the anisotropy of the wood and the sheathing materials. The modulus of elasticity and the shear modulus as well as the Poisson's ratio can be defined for all three directions of the material.

The nonlinear link is used to connect the frame elements to each other and the shell to the frame elements. This link is defined between two nodes i and j and consists of six independent nonlinear springs. Static analysis is used to predict the behaviour of the structure under quasi-static wind pressures and applied static loading.

Results

Wind measurements

Wind data collection started during the spring of 2004. Apart from yielding useful load and response data for the particular type and shape of this building, the shed structure served as a test bed for assessing several instrumentation devices to be used in future projects.

Since the shed structure is not an isolated building, its response to wind is influenced by two main features in the two predominant wind directions. For wind blowing from the North-East and Easterly directions, the structure falls in the wake of the main Forintek building with a height of 9 m. From the North-West and Westerly directions, the structure is in the wake of a forested area. The direction of wind incidence on the building and its deviation due to surrounding topography are the main contributing factors to the pressure distribution observed on the structure.

Critical angles of wind incidence were those where a 90-degree angle was created between a particular surface of the building and the wind direction. For this structure, these angles were about 110 degrees for the southern wall, 200 degrees for the western wall, and 290 degrees for the northern wall.

Comparison with airport data

The wind velocities at both the reference and roof heights were taken from the velocity profiles evaluated for westerly and easterly winds. These represented the two upstream wind possibilities: forested area for the West side and the Forintek main building for the East side. Both profiles provided a power-law exponent of 0.34 (Equation 4). Figure 8 shows an example of comparison between time series for the wind direction from the monitoring data (Forintek) and the airport data.

Fig. 8 a) and b) show that the results for the wind direction agree well with the airport data, considering that small differences in the data sets are always expected due to the local conditions. The tendency of the full-scale data to be consistently slightly higher than those from the airport can be attributed to the presence of trees close to the shed. For this case, a deviation of around 20% is observed. The actual wind direction used in the calculations is that taken from the anemometer located on top of the building.

Figures 9 a) and b) show wind speed records measured at the airport and, in parallel, with the anemometer at the top of the Forintek building. The measurements are taken for the wind direction shown in Fig. 8. In addition, the airport data has been corrected to reflect the exposure influence (terrain roughness) on the wind blowing from the airport to the building location (retardation effect). The graphs confirm that the exponent of 0.34 is a reasonable choice.

Mean and peak pressure coefficients

The mean pressure coefficients on all surfaces were calculated using Equation 3. Figure 10 shows a sketch of the position and numbering of the wall and the roof pressure taps used in the wind-tunnel and the full-scale measurements.

The pressure coefficients on the western wall increase gradually as the wind incidence becomes closer to 200 degrees, where an angle of approximately 87 degrees is formed with this wall, as shown in Fig. 11. Mean pressure coefficients at this angle were observed to be 0.27, 0.80, 0.18 and 0.79, for taps 1, 2, 3 and 4 respectively. Beyond this azimuth, the C_p values continue to decrease steadily.

Figure 12 shows that pressure coefficients for the northern wall increase as the wind incidence changes from 45 to 90 degrees. The mean pressure coefficients were -1.44, -1.84, -1.19, and -1.90 for tap locations 13, 14, 15 and 16, respectively. As the wind incidence shifts to a more westerly direction, pressure coefficients increase steadily and experience local maximum values at 300 degrees (flow almost normal to face of wall) where mean pressure coefficients were found equal to -0.30, -0.16, -0.36, and -0.39 for tap locations 13, 14, 15 and 16, respectively. Afterwards, C_p values decrease, as the angle approaches 360 degrees and it becomes less perpendicular.

The southern wall taps experience increasing pressure coefficients from 45 to 110 degrees as the wall is in the wake of the Forintek main building and the angle of incidence to the wall becomes normal. Figure 13 shows the large reduction of pressure coefficient values for this wall as the wind approaches the azimuth of 200 degrees, where the influence of the Forintek main building becomes clear. At this angle, the mean pressure coefficients of -1.44, -1.19, -1.42, and -1.22 were obtained for taps 5, 6, 7 and 8, respectively. As the wind comes through the forested area, i.e. from the West, pressure coefficients increase gradually with locally high mean values at 285 degrees, for which the wind is almost normal to the northern wall; the mean C_p values were then measured as -0.27, -0.59, -0.40, and -0.25 for taps 5, 6, 7 and 8, respectively. As the wind incidence becomes more northerly, the pressure coefficients decrease but remain relatively stable.

In addition to the surroundings, the roof taps were influenced by the presence of a 0.5 m high perimeter parapet. Tapings 9, 10, and 11 are located approximately 500 mm from the wall edges. It should be noted that for azimuths 45 to 110 degrees the roof tapings experience either positive pressure, or very low negative pressure due to the flow path over the Forintek main building. Figure 14 shows the variation of roof pressure coefficients with the azimuth. For the wind direction of 70 degrees, the mean pressure coefficients are 0.31, 0.05, 0.16, and 0.04, for tap locations 9, 10, 11, and 12, respectively. The flow seems to be re-attached at these points. High suction appears on the roof for the 200-degree direction, as wind flows over the parapet. This is consistent with the previous findings by Stathopoulos et al. (2002). As the wind turns westerly afterwards, the C_p values steadily increase as flow passes through the forested area where it is less restricted since the trees create a porous barrier.

Peak pressure coefficients were also determined from the in situ measurements and compared with those of the wind tunnel tests. Figure 15 shows mean and peak pressure coefficients for all azimuths measured in the wind tunnel on the western wall as well as peak pressure coefficients from the full-scale data. Generally, the ratio between peak and mean values was calculated to 2.5 to 4 with an average of about 3. This ratio is higher for higher C_p values. Furthermore, peak pressure coefficients appear to have the same trend in their variation with direction as with the mean pressure coefficients.

The peak pressures measured on the envelope of the building compare well with corresponding values obtained in the wind tunnel tests. In general, the peak pressure coefficients from the full-scale tests were higher than those obtained from the wind-tunnel test.

Comparison between full-scale data and wind tunnel data

In general, full-scale and wind tunnel mean C_p values were found to agree well and to follow a similar trend for the various directions studied.

This comparison was not so good for the southern wall tapings, where although field C_p values followed the same trend as the wind tunnel data, their magnitudes were typically lower than expected for the 90, 100, and 110 directions. This could be attributed to the proximity of this wall to the Forintek main building and a possible inability to properly simulate the recirculation in this restricted area for these wind directions. For 260 and 280 degrees, the values are slightly lower as well for tap locations 5 and 7 but within a reasonable deviation. Both these tapings are located on the edge of the southern wall (corner with the western wall) and the flow might not have separated similarly along this edge.

The western wall tap locations 1, 3 and 4 also experience lower values than expected for the 90, 100, 110 degrees. Flow separation could therefore be slightly different in the wind tunnel test and in the full-scale tests, for the reason mentioned above. Tap location 2, on the centre of the wall, provided more consistent data and the comparison with the field is encouraging.

Finally, the roof taps also showed some differences in the C_p values measured in the field and the wind tunnel for wind directions 90, 100 and 110 degrees. These deviations occur again when the shed is in the wake of the Forintek building. However, for wind directions 260 and 280 degrees, the field and wind tunnel values correlate very well.

3-D finite element model

General

A 3-D finite element model of the test shed was created (Fig. 16). A detailed description of the model can be found in Chapter 3. Static controlled loading (Doudak et al. 2005), was used to calibrate the model. Model loading includes four cases: Uniform and concentrated loads on the roof, as well as concentrated loads at the top and mid-height of the wall.

Comparison between applied static load tests and 3-D model

A uniform load of 0.7 kPa was applied on the roof by filling the roof with water. The load caused a mid-span deflection of 2.8 mm for an interior joist. The measured maximum deflection on the roof is compared to the deflection calculated for a single joist (3.9 mm) and the value calculated by the model (2.9 mm). It is clear from this result that a large reduction in deflection was observed when considering the system effects in the model. The measured value is 28% smaller than the calculated value for a single joist. It can also be concluded that the model prediction of the roof deflection under uniform load is reasonably accurate.

A 3 kN concentrated load was applied at mid-span of an interior roof joist. The bending deflection was 1.45 mm for the joist directly under the load, and 0.80 mm for the adjacent joist. The model predicted the deflection of the joist under the load to be 1.43 mm and the deflection of the adjacent joist to be 0.81 mm. These results show the model's ability to predict the deflection in the roof joists accurately. The loaded joist carried about 26% of the applied load, while those adjacent received most of the remainder load through two-way action. The model suggests that four joists on each side of the loaded joist are sharing the load; the fourth joist away from the loading point is carrying only approximately 3% of the load. One can conclude that more joists than presumed in conventional design are participating in the load sharing and that ribbed plate models clearly are capable of accurate predictions.

A lateral load was applied to the wall at mid-height location relatively remote from door or window openings. The results, presented in Table 1, indicate that the stud immediately adjacent to the loaded stud deflected as much as 74% of the deflection of the loaded stud. The model was able to predict the deflection of the wall with good accuracy, with greater accuracy for predicting the deflection of the loaded stud (only 1% error for the loaded stud and 12% error for the stud 800 mm away from the loaded stud). The model also suggested a large degree of load sharing in the wall: Approximately 10 studs adjacent to the loaded stud (on each side) are active in the load resisting mechanism.

A similar load test was conducted on the top of the wall. Although not presented here, the results indicate that in this case also, the model was able to predict the deflection of the shed with good accuracy. Relatively small displacements (below 1 mm) occurred at the top of walls irrespective of the height of the point of application of the loading, since the roof has considerable in-plane rigidity and can be regarded as a rigid diaphragm. This is an important finding in the context of design level analysis for distribution of wind forces between shear walls in buildings similar to the one investigated.

Summary and conclusions

This paper describes structural monitoring experiments on a single storey industrial shed building located in Quebec City, Canada. The structural response monitoring is based on measurements of deformations within a representative segment of the wall and roof surfaces and a matching grid of wall and roof wind pressure taps. The pressure distribution on the envelope of the building in full scale was supplemented with a wind tunnel study.

Critical angles of wind incidence were those where a 90-degree angle was created between a particular surface of the building and the wind direction. For this structure, these angles were about 110 degrees for the southern wall, 200 degrees for the western wall, and 290 degrees for the northern wall. The wind velocities at reference height and at roof height were taken from velocity profiles evaluated for westerly and easterly winds. Both profiles provided a power-law exponent of 0.34.

In general, it was shown that the building surroundings have a great effect on the pressure distribution at the surface of the structure and that these effects are not always intuitive and can be hard to predict using code considerations alone. More specifically, it was important to note on the western wall the higher values of the two taps near the centre of the wall as well as the relatively smaller values near the edges. Also, the roof taps were influenced by the presence of a perimeter parapet. It should be noted that for azimuths 45 to 110 degrees the roof tapings experience either a positive pressure or a very small negative pressure due to the flow path over the adjacent main building. High suction appears on the roof for the 200-degree direction, as wind flows over the parapet.

Peak pressure coefficients were also determined from the full-scale tests and compared with those measured in the wind tunnel. Generally, the ratio between peak and the mean values was found to be 2.5 to 4 with a mean of about 3. This ratio is higher for higher C_p values. Furthermore, peak pressure coefficients appear to have the same trend in their variation with direction as with the mean pressure coefficients. The peak pressures measured on the envelope of the building compared well with the corresponding values obtained in the wind tunnel test. In general, the peak pressure coefficients from the full-scale tests were higher than those obtained from the wind-tunnel test.

The mean C_p values obtained from the full-scale and wind tunnel tests agreed well and follow a similar trend for the directions studied. This comparison was not so good for the southern wall tapings, where although field C_p values followed the same trends as far the wind tunnel data, their magnitudes were typically less than expected for the 90, 100, and 110 directions. This could be attributed to the proximity of this wall to a larger adjacent building and a possible inability to properly simulate the recirculation in this restricted area for these wind directions. Also, the roof tapings showed some differences in the C_p values measured in the field and the wind tunnel for wind directions 90, 100 and 110 degrees. These deviations occur again when the shed is in the wake of the larger building. However, for wind directions 260 and 280 degrees, the field and wind tunnel values correlated well.

The 3-D model predictions agree in general with the results of static load tests on the shed structure. It was shown that more joists than initially presumed are participating in the load sharing and that ribbed plate models clearly are capable of very accurate predictions.

Acknowledgements

The authors gratefully acknowledge support and contributions to this project from The Canadian Wood Council, Forintek Canada Corp. Eastern Division in Québec City and the Natural Sciences and Engineering Research Council of Canada. Dr. Mohammad Mohammad and the technical staff of the Forintek Canada Corp. are especially thanked.

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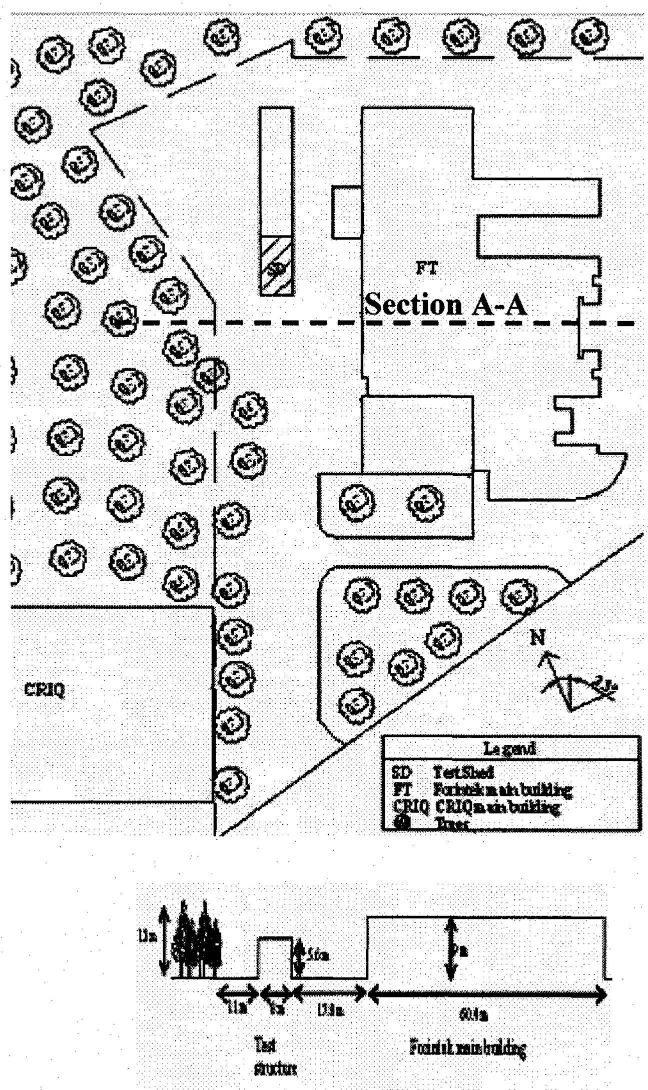
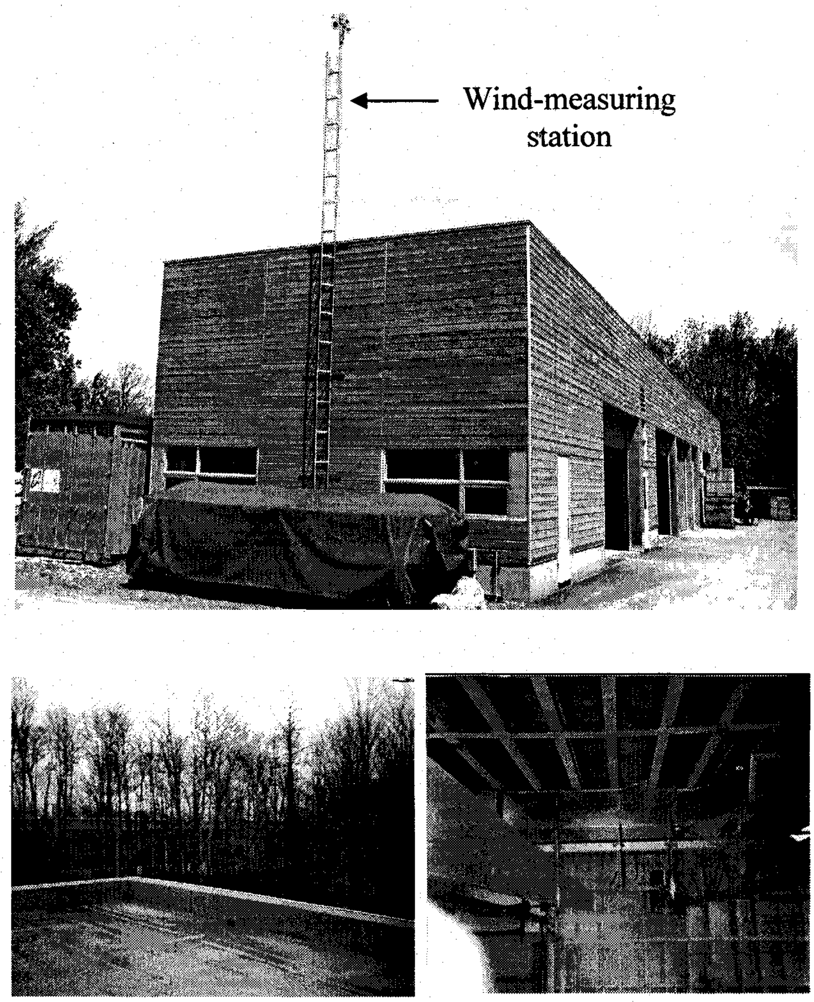


a)

b)

Figure 1: The Forintek complex
a) Plan view
b) Section A-A



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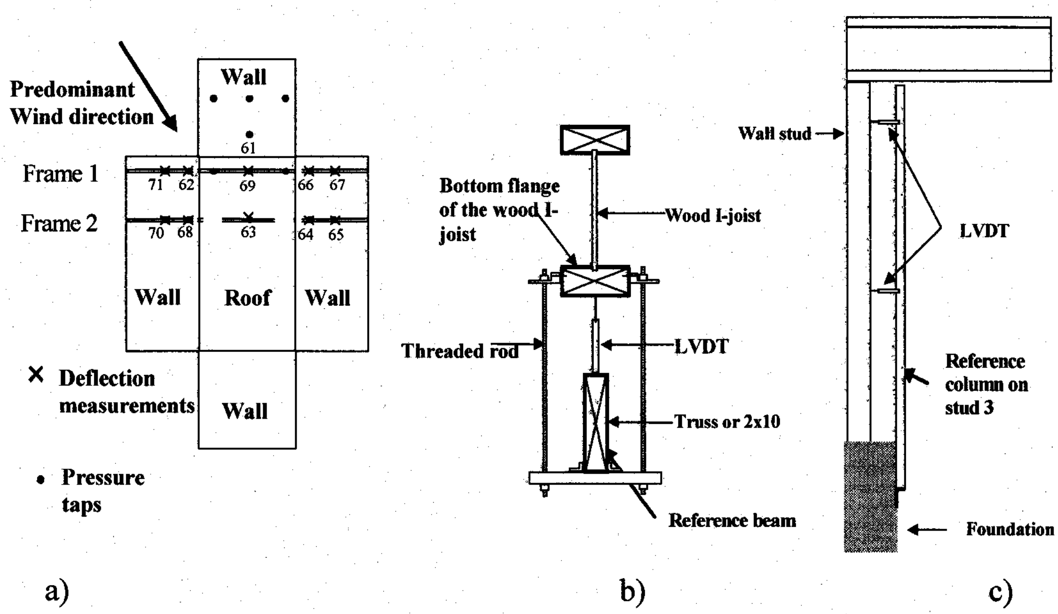

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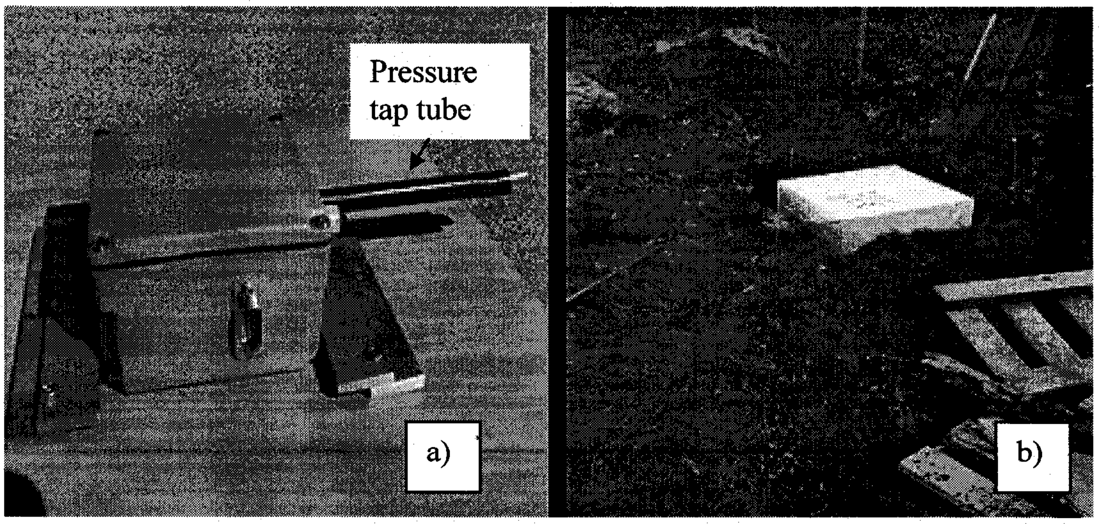

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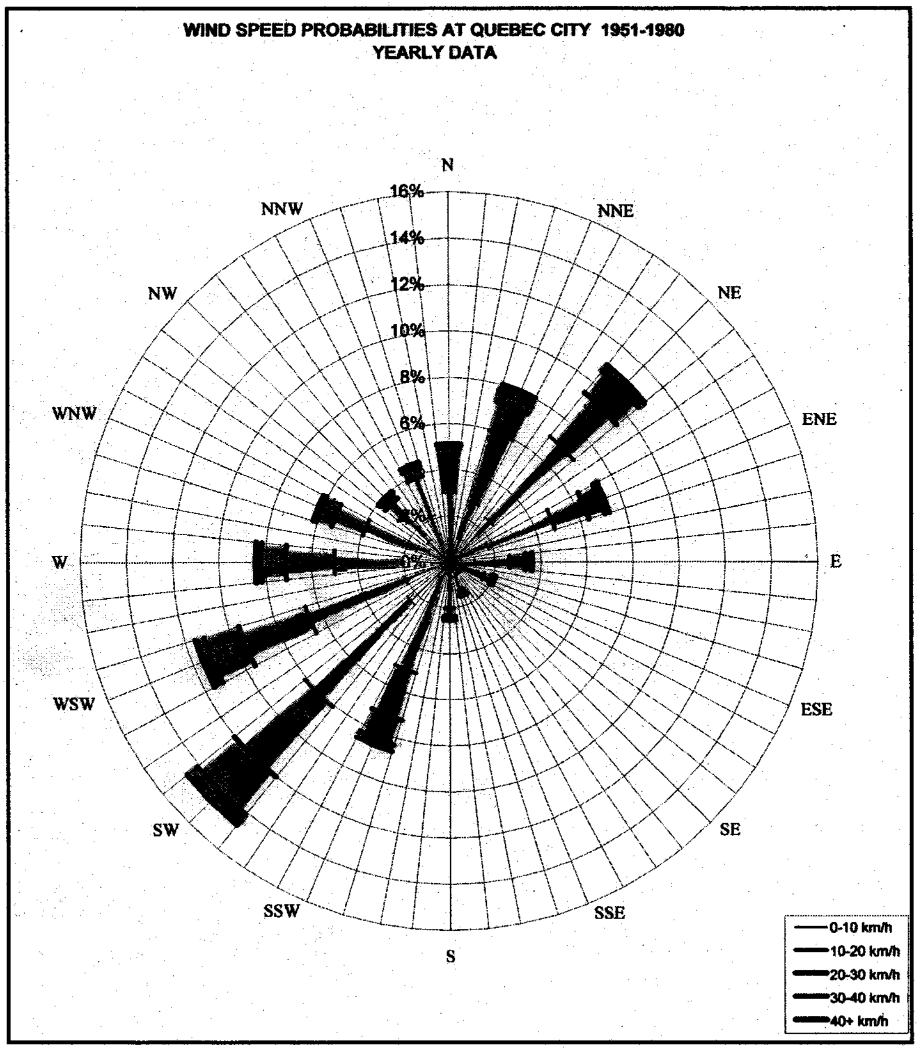
a) Pressure sensor on the roof,
b) Reference pressure box



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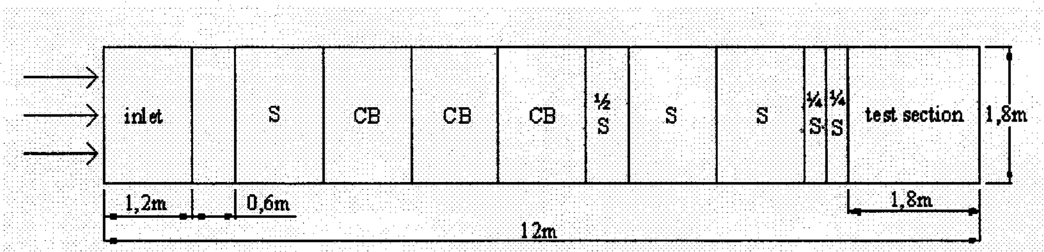

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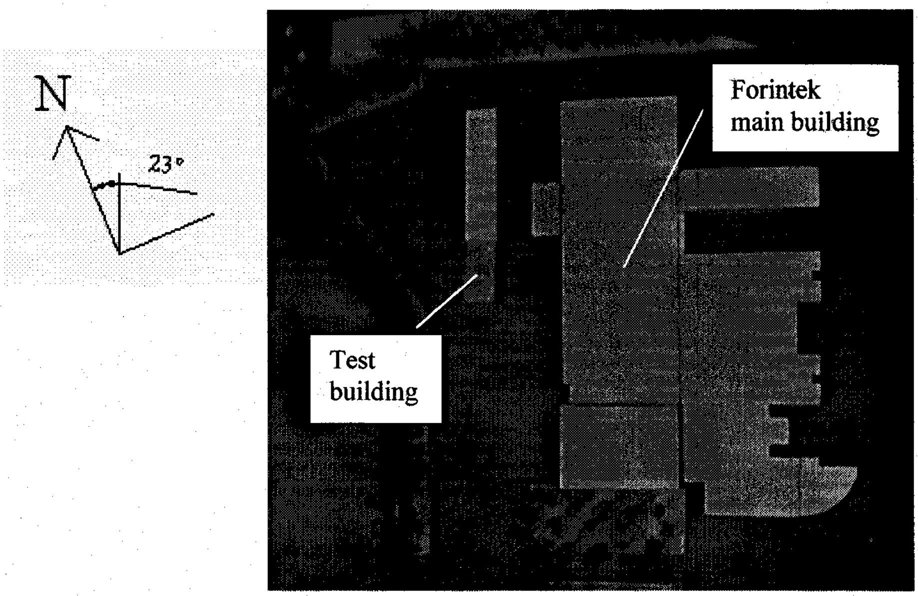

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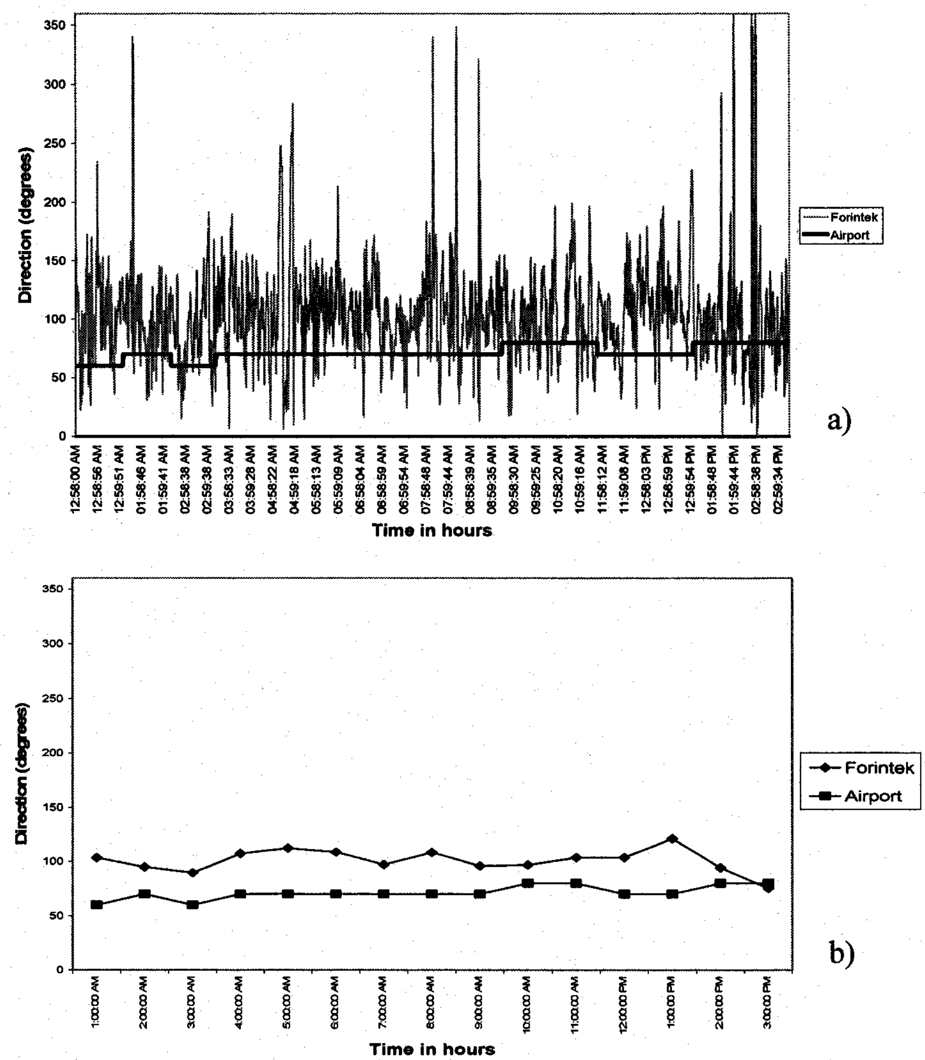

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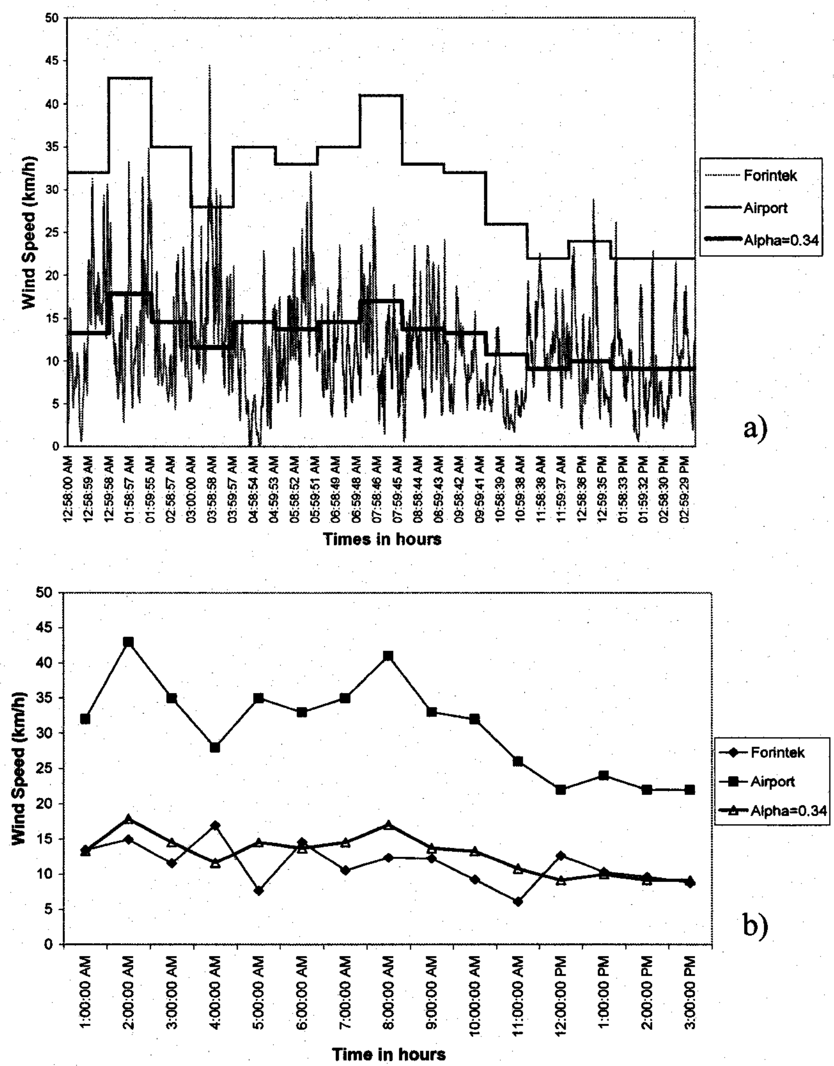
a) typical record
b) hourly average



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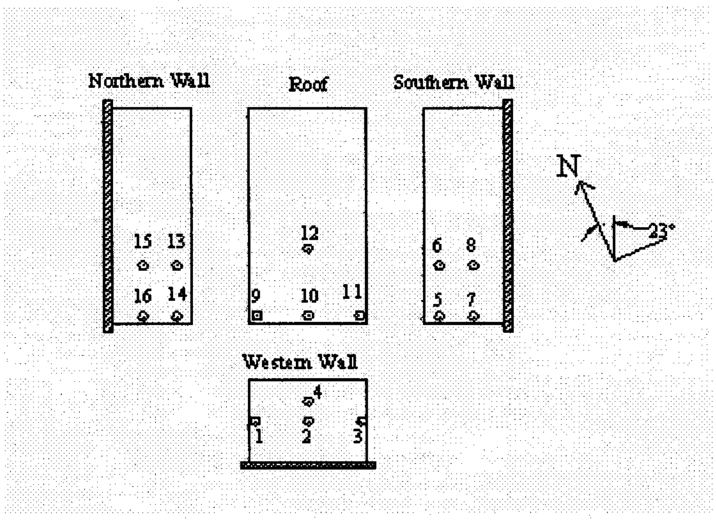
a) typical record,
b) hourly average



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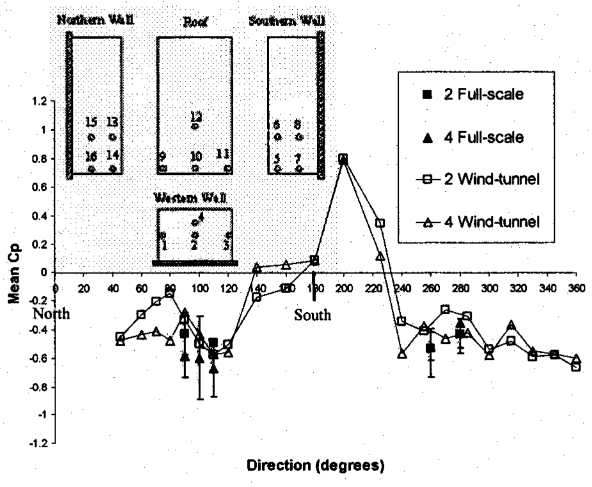

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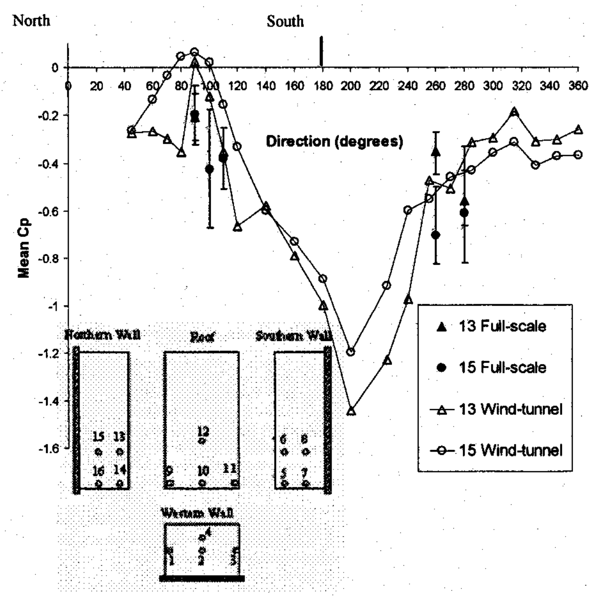

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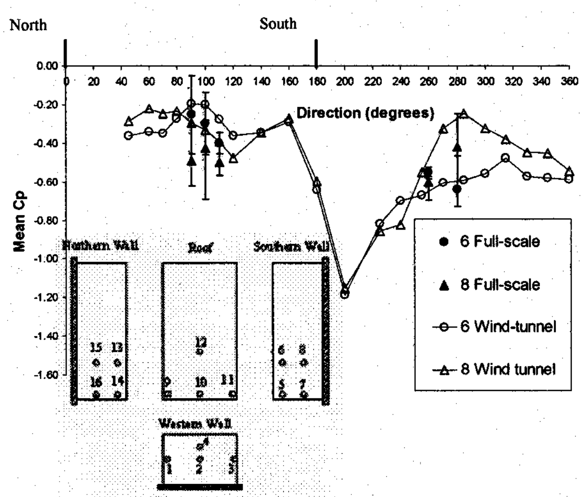

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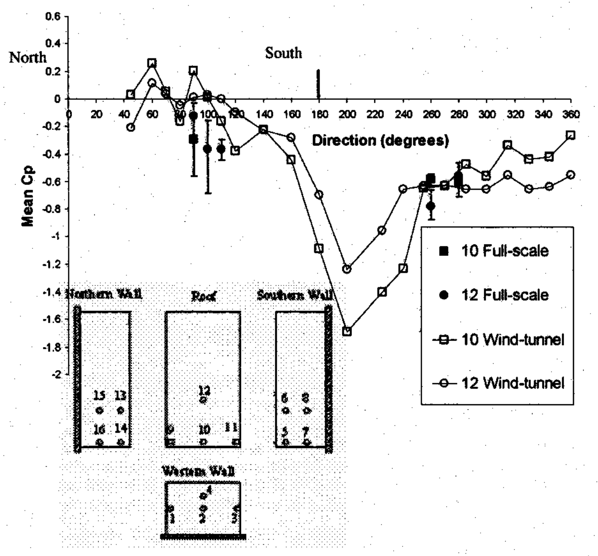

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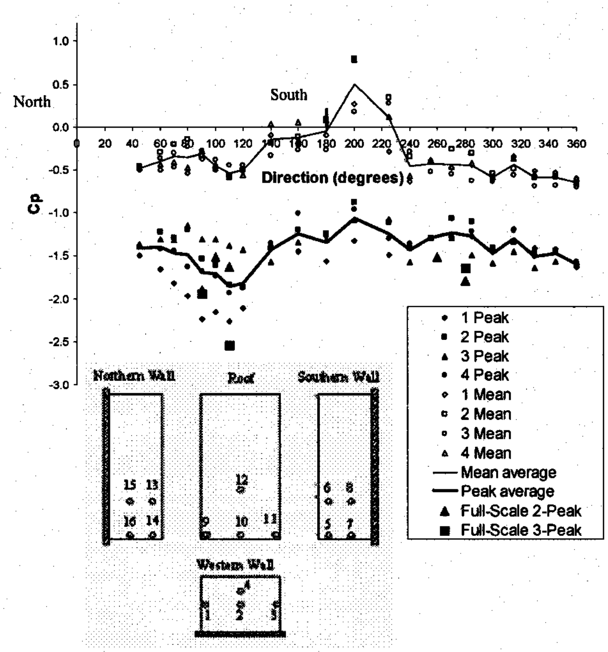

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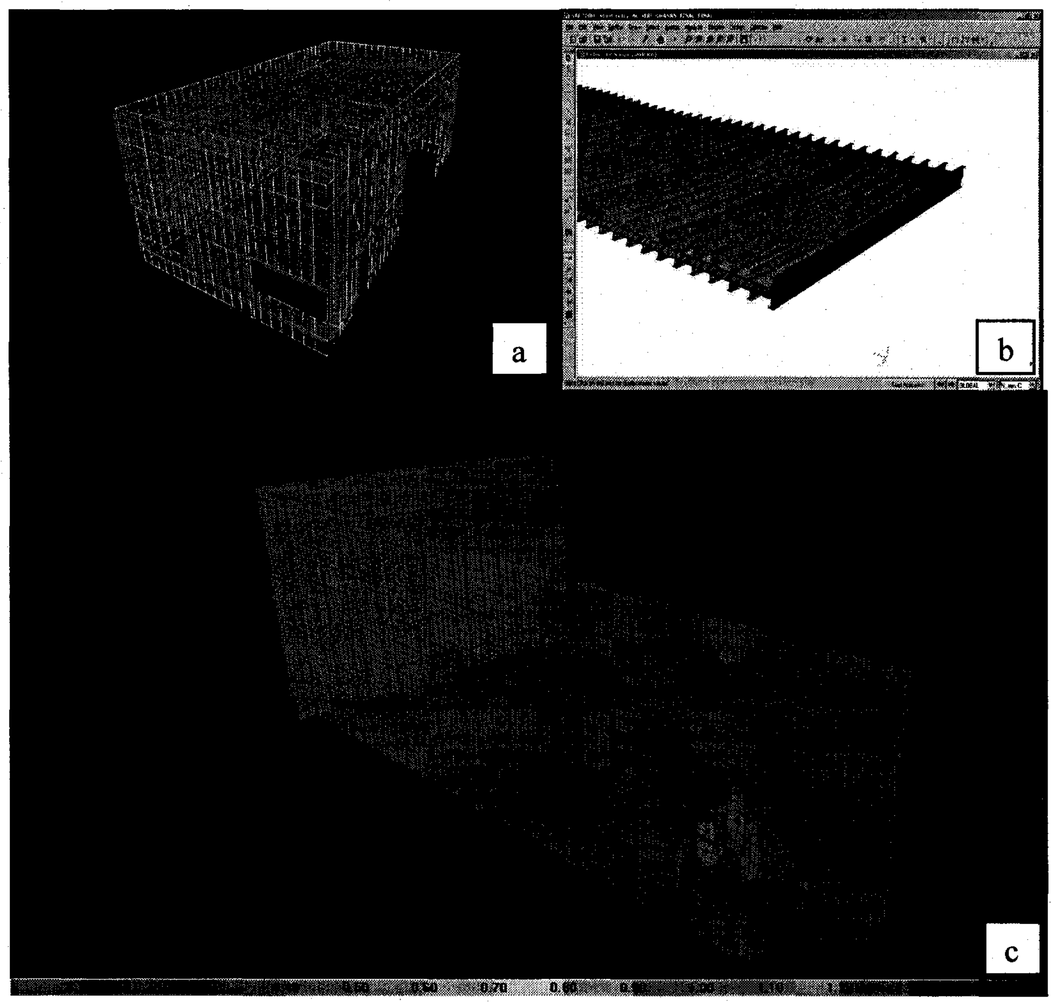

Figure 16: 3-D mesh of FE model
a) entire model
b) roof joists
c) wall deformation under point load at mid-height

Table 1: Deflections (in mm) under concentrated load for loading at mid-height of wall

		Loaded stud (2)	**Immediately adjacent stud (3)**	**Two studs away from loaded stud (4)**
Deflection, mm (1)	*Full-scale test*	3.4	2.5	1.7
	Model prediction	3.45	2.4	1.9
	%-error	1	4	12

CHAPTER 5

Structural Response of a Wood Light-Frame House to Wind

While in past studies, several experimental measurements of the environmental loads on the building envelope have been made at full scale, none of these studies attempted to link the external loads with the internal forces in the structure.

Full-size structures within the laboratory are "realistic" in size, but they often represent stripped-down constructions. Also, the application of artificial loading on these structures usually requires some modifications to prevent local damage. These modifications can be quite considerable in some instances and can therefore affect the response of the test structure, and lead to unrealistic failure mechanisms. Special attention has to be given to altering the test structures by adding various instruments such as load cells. The stiffness of the instruments should not affect the stiffness of the building and thereby create artificial load paths. This is especially important when the roof-to-wall interface is considered. Full-size tests done *in situ* are the only ones that can fully account for effects of construction details, physical environment, moisture movement, loading and ageing processes.

The study of the Forintek test house in Chapter 4 was limited due to the fact that the building was already built at the time of testing. The UNB test structure on the other hand was especially built for this research and has instruments embedded in the framework to establish the load paths within the structure. As it was the case for the Forintek building, the external load is measured and compared with the internal forces. Comparison with wind tunnel tests on reduced-scale model of the UNB house will be performed at a later time: This is not within the scope of the current research project.

Monitoring the structural response of a wood light-frame house to wind: Instrumentation and preliminary results

By G. Doudak[1], G. McClure[2], I. Smith[3], T. Stathopoulos[4]

Abstract: Environmental loads due to wind can be predicted within a statistical framework, but these estimates are usually only rough approximations. The goal of this research is to characterise the wind loads and identify the load paths in light-frame wood buildings subject to wind loads. The test building is a single-storey house located in Fredericton, New Brunswick, Canada. The building is set on a flat and relatively open site within a research park setting. Pressure taps were installed on the test building to establish the pressure distribution on the building surfaces. 1-D and 3-D load cells were successfully designed to measure the forces in three orthogonal directions without disturbing the stiffness of the roof-to-wall connection. The load cells have performed adequately in all tests.

Key Words: Load distribution, monitoring, timber construction, wind loads.

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Introduction

Most low-rise buildings in North America are wood light-frame systems, which typically present much complexity and structural redundancy. Therefore, their load-carrying behaviour (load sharing and load paths) cannot be based on intuition and simple notion of tributary areas.

The prediction of the structural safety of any building depends on factors such as the ability to predict the external loads, adequate knowledge to predict the force flow through the building and into the components and, finally, the reliability of the measurements to ensure acceptable performance. Environmental loads due to wind can be predicted within a statistical framework, but these estimates are usually only rough approximations. Pressure coefficients available in the literature reflect generic or very specific building shapes and specific surrounding terrain conditions. It follows that loads and their distribution on light-frame buildings are not well understood. The level of safety and serviceability of light-frame buildings under unusual (extreme) loads is unclear, along with the techniques which can improve their structural performance at ultimate load.

Modern performance-based building codes, such as those that will be embodied in the next generation of Canadian "Objective Based" building codes lay down what a building system is to achieve in terms of either building physics or structural behaviour. When fully developed and implemented, new codes are intended to lead to improved efficiencies in the consumption of materials, and construction and operating costs based on life-cycle analyses. In order to achieve this goal, there must be development of new construction technologies, including structural design methods.

Research objectives

The goal of this research is to demonstrate methods of identifying load paths in light-frame wood buildings subject to wind loads and improving the knowledge of wind loads on typical low-rise buildings (under Canadian conditions).

Test building

The test building is a single-storey house structure located in Fredericton, New Brunswick. The building is set on a flat and relatively open site within a research park setting (Fig. 1). Open exposure is important because it facilitates the description of the approaching wind. The house has regular plan geometry (two to one ratio of the length to width on plan) and a duo-pitch roof (4/12 slope), which represent a standard case for defining wind pressures in the National Building Code of Canada (NRC 1996) and similar international standards.

The house meets the design specification of the Canadian Mortgage and Housing Corporation (CMHC 1997) for platform construction, with some modifications to accommodate for placement of monitoring instruments. The building is representative of typical North American "bungalow" style single-family residence with a floor platform over a perimeter concrete foundation (8.5 m x 17 m footprint). The foundation is a continuous reinforced concrete strip footing supporting a 255 mm (10 in) thick and 1226 mm (4 ft) high frost wall.

The walls consist of 3678 mm (12ft) panels nailed together with 89 mm (3.5") nails. The wall frames are assembled from studs, 38x89 mm (2"x4") S-P-F (Spruce Pine Fir) lumber framing at 600 mm spacing. There are double 38x89 mm top plates tying the wall panels together and one bottom plate. Studs are end-nailed to the top and bottom plates with 3.5" nails. The walls have oriented strand board (OSB) with wood clapboards on the exterior. The structural response is assessed with and without the clapboards. Also, internal plasterboards and window openings will be added later. Wall sheathing is 9.5 mm, 1.22 x 2.44 m exterior OSB panels nailed at a 150/300 mm spacing (150 mm on perimeter, 300 mm inside). The roof consists of trussed rafters laid out without blocking on 600 mm spacing and it is sheathed with 13 mm OSB, fastened by nails at 150/300 spacing. The roof trusses are shaped as Fink trusses (W-trusses) and comprise of lumber elements of 38x89 mm (2"x4").

The test house is equipped with two series of load cells to measure the vertical and horizontal forces parallel to the external wall directions. There are a total of 27 load cells arranged around the perimeter of the building at the foundation level, with an inverted steel C-shape sitting above. It is important to note that there is no continuity in the structural system across the wall-to-foundation interface other than what is provided by the load cells. Thus the building can be weighed continuously to determine the resultant and/or directional total forces. The weighing data is to be used in conjunction with field point observations of wind pressure and matched reduced-scale wind tunnel test data to determine pressure distributions over the external surfaces.

A series of load cells are installed between the roof and the wall to monitor the vertical roof load transfer. Where a load cell is not used, a wood spacer replaces the load cell. The intent is to measure the vertical forces so that the load path of the total gravity load and some imposed loads on the roof will be observed. Continuity at the gable walls prevents the whole roof to be weighed as done for the superstructure-to-foundation interface.

Measurements made on the test structure include: wind speed and direction, external and internal wind pressures on wall and roof surfaces, internal forces (at superstructure-to-foundation and wall-to-roof interfaces), and displacements (deflections and distortions) of stud walls and roof trusses.

Wind instrumentation

Anemometer measurements

A wind-measuring station is installed on a tower located approximately 30 m west of the test building to measure wind speed and direction at two levels. One anemometer is placed at roof height, approximately 5.5 m above ground, and another is placed at 10 m, which is the reference height for wind collection in meteorological stations accredited by Environment Canada. A picture of the wind station tower can be seen in Fig. 2.

Pressure taps

At this stage, 28 pressure sensors have been installed on the building surface, 10 on the walls and 18 on the roof, in the arrangement shown in Fig. 3. At a later time, additional pressure taps will be added as needed: It is already planned to add pressure taps on the eave, which explains the missing numbers in Fig. 3. The distribution of the pressure taps on the house surface was based on considerations of the predominant wind direction. The pressure taps are plastic tubes of 4.8 mm inside diameter. Sensitive differential pressure transducers are used to measure surface pressures.

Moisture and especially rain can influence the accuracy of pressure measurements, and several set-ups were tested to minimize these effects. All pressure taps are equipped with a 90-degree angle elbow (Fig. 4 a) to prevent rain from entering the tube. This is especially critical on the roof. A supplementary way of overcoming this humidity problem is by attaching a Y-shaped air gap to the pressure tap (Fig. 4 b). This provides a reservoir for the water to be collected and emptied as needed.

Ambient atmospheric pressure is used as the reference measurement pressure for the transducers. It is obtained from a box below ground with an air intake tube of 25.4 mm diameter. A 25.4 mm diameter pipe transmits the ambient pressure to the building and from there the tubing connects the reference pressure to all the transducers. It is assumed that the building and the box are located far enough apart (approximately 30 m) so that the building has no effect on the static pressure at the reference pressure box (see also Chapter 4).

Internal force instruments

Design of roof-to-wall load cells

1-D roof-to-wall load cells

One of the greatest challenges in the instrumentation when considering the load paths within the structure is to avoid altering the structure significantly and thereby creating artificial load paths. The instrumentation challenge in this research was to create a system to measure forces at the roof level with minimum change in the stiffness of the construction detailing.

Several possible load cells designs were investigated, following which an "instrumented ring" design was chosen (Fig. 5).

3-D load cells

The initial concept for measuring 3-D (X,Y,Z) force transfers between the trusses and the walls was based on the use of unidirectional ring load cells, measuring forces in the orthogonal directions (Fig. 6). Each ring load cell was tested individually and seemed to work with good accuracy (3% of full-scale). Since the stiffness is different in each principal direction of the roof-to-wall connection, the ring load cells in the assembly was designed accordingly. This introduced, in theory, six different load rings with six different stiffnesses, designed specifically for a particular direction in space. The concept was validated in tests on the load cell assembly loaded in isolation from the actual building. However, this load cell assembly was not implemented because of the large number of load rings needed, and concerns about the possibility that components might get loosened in service, with a consequential loss of calibration.

The focus in the load cell design shifted to creating a device that could measure forces in the three orthogonal directions and have stiffness comparable to the "real" roof-to-wall connections (Fig. 7). A load cell was designed using finite element models that were successfully refined, with an error of less than 5% deviation from full-scale. The measuring segment of the load cell was made in a square shape with two couples of SG- 3/120-LY41 strain gages for the x-direction and another two for the y-direction. Four couples of the same strain gages are used for measuring the z-direction normal stresses (compression and tension). All gages were configured as full bridges. The load cells were calibrated and the calibration curves indicated linear behaviour throughout the range of its capacity, and with negligible cross-effects between the responses in the three measuring directions. Before utilizing the load cells in the test structure, they were calibrated in the laboratory. They were also installed under shear walls for a series of full-scale static tests performed at the University of New Brunswick (Mi 2004). The load cells performed adequately in all tests. So far, only one 3-D load cell and six uniaxial load cells have been installed at the roof level. More load cells will be installed later (Fig. 8).

Superstructure-to-foundation load cells

The set of load cells located at the superstructure-to-foundation interface, i.e. under the floor platform, register the total load transferred in the three orthogonal directions. The location of the load cells is shown in Fig. 9. This enables the combined effects of the dead loads and the imposed loads on the superstructure to be determined at any instant. Each unit, as shown in Fig. 10, is manufactured from three identical shear-beam load cells, which are connected together and oriented such that each individual beam allows measurement of the load in one orthogonal direction. Equilibrium checks were performed at various stages during their installation in the test building.

There are a total of 27 load cells arranged around the perimeter of the building, with a continuous steel channel sitting above them. This arrangement of load cells was designed based on the detailed finite element (FE) analysis of the test building that indicated a combination of the load cells that would smooth forces in the load cells and ensure adequate precision in the force measurements.

Installation of load cells

Superstructure-to-foundation load cells

The base plates of each load cell were manufactured specifically for the test building. They have slotted adjustment holes to allow for levelling of the load cells. Since concrete foundation cannot be cast with a large geometric accuracy, holes for anchor bolts were drilled after the concrete was cured and the bolts were set in place using epoxy resin. There are two locking nuts per bolt (Fig. 11 a), which were tightened once the plate was properly aligned and levelled. This was especially important for accurate measurements because load cell readings are highly sensitive to the boundary conditions and to any misalignment between the framework axes and the axes of the load cell. Another crucial installation step was the alignment of the load cells both in the horizontal and vertical directions. This is shown in Fig. 11 b) and c). Each of superstructure-to-foundation load cells was calibrated individually under laboratory conditions. Once the load cells were installed in situ, a special steel box was constructed around each load cell, and they were checked to ensure that the calibration was still valid.

Roof-to-wall load cells

The second series of load cells is located at the roof-to-wall interface, i.e. between the top wall plate and the roof framework to register the loads transferred by the roof to the wall. The roof load cells were installed by attaching (nails) a steel plate welded to the bottom of the load cell. The top part of the load cell was attached to the roof truss by means of a metal strap as shown in Fig. 12 a. Two types of load cells were used (Fig. 12).

Results and discussion

Wind data

Comparison between the two anemometers

Two anemometers are installed on a tower located near the test building. One of the anemometers is placed at the roof height and the other is placed at 10 m above ground (same as the reference height at the airport).

The wind speed depends on the height above ground. This can be described according to the following simplified Power law:

$$U_1 = U_2 (Z_1/Z_2)^{\alpha} \qquad (1)$$

where U_1 is the wind speed at height Z_1 and U_2 is the wind speed at height Z_2. α is the dimensionless index reflecting the terrain roughness conditions at the site.

Fig. 13 shows two-minute average of wind speed at 10 m height and at roof height, respectively, and the wind speed at roof elevation obtained from the 10 m high record corrected with the power law.

The velocities at reference height from the airport and at the two anemometers on the tower near the house were taken from velocity profiles obtained for the oncoming wind incidences. These profiles provided a power-law exponent of 0.22. Fig. 14 shows an example of comparison between the time series for the wind direction from the test building and the airport. The results for the wind direction match well with the airport data. Small differences in the two data sets are observed due to the local conditions. The actual wind direction used in the calculations is that taken from the anemometer located on the building top.

Where direct correlation between wind direction at the structure and at the airport is possible, it must be recognized that the wind speed is affected by the wind boundary layer thickness and the local building terrain in the wind path. Fig. 15 compares the average wind speed between the anemometer at the test structure and the airport. The graph emphasizes that the value of the exponent of 0.22 is a reasonable assumption.

Preliminary wind data

So far, only a few data sets were collected during the fall of 2004. Due to the limitations of the results, it is premature to draw any conclusions and therefore more monitoring is needed.

Conclusion

A 3-D load cell was successfully designed to measure the forces in three orthogonal directions with an error of less than 5% of full-scale and without disturbing the stiffness of the roof-to-wall connection.

The velocities at the reference height from the airport and at the two anemometers on the tower near the house were taken from velocity profiles obtained for the oncoming wind incidences. These profiles provided a power-law exponent of 0.22. The results for the wind direction match well with the airport data. Small differences in the two data sets are expected due to the local conditions.

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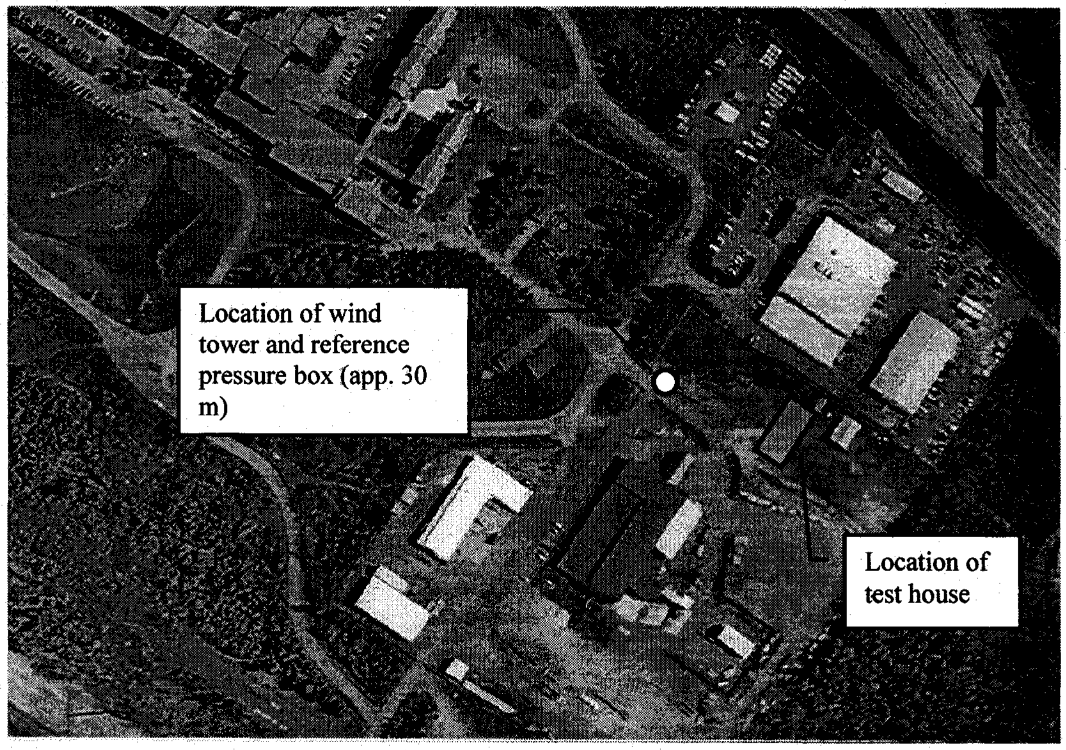

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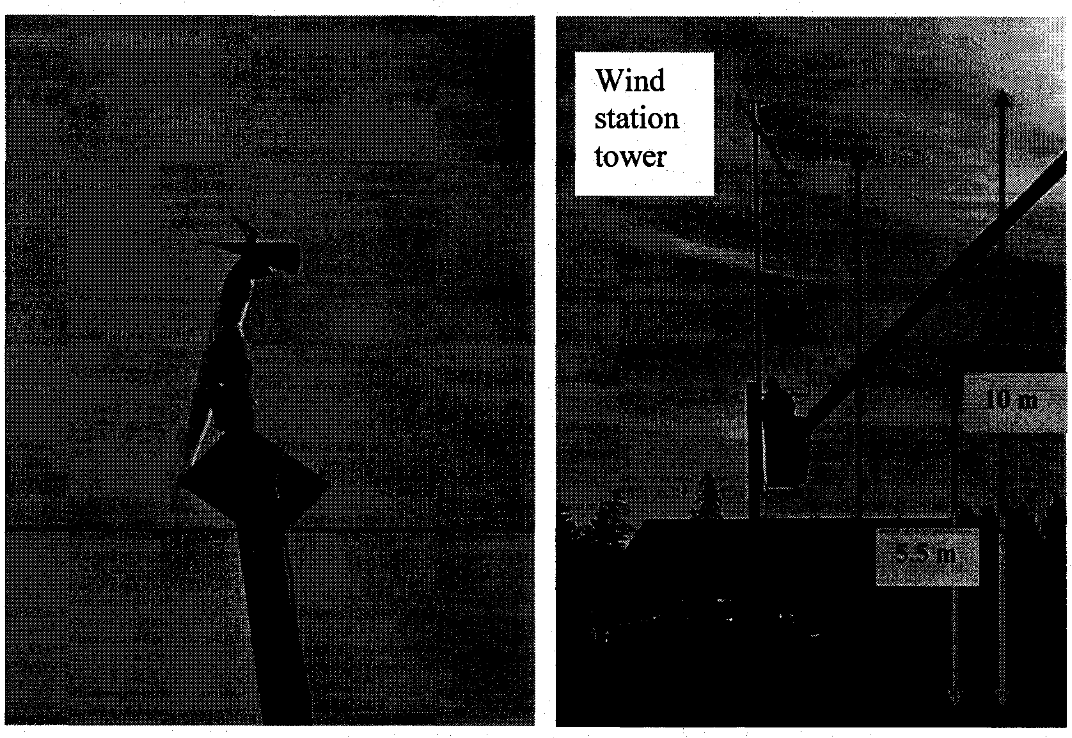

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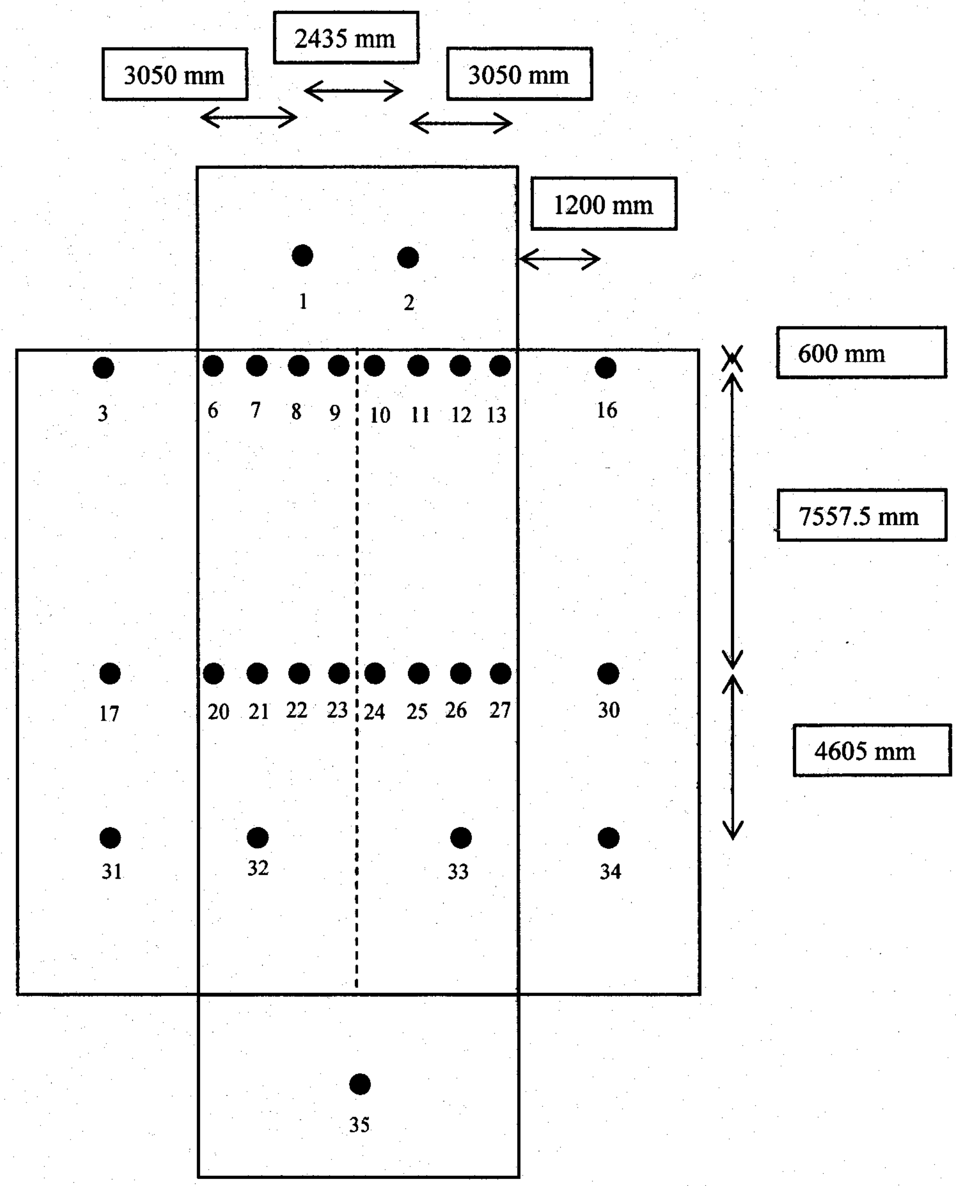

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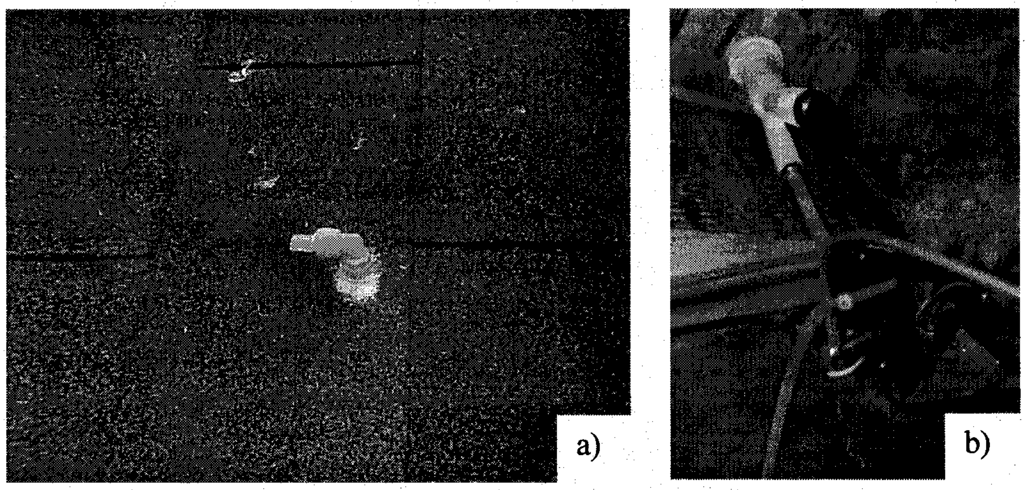

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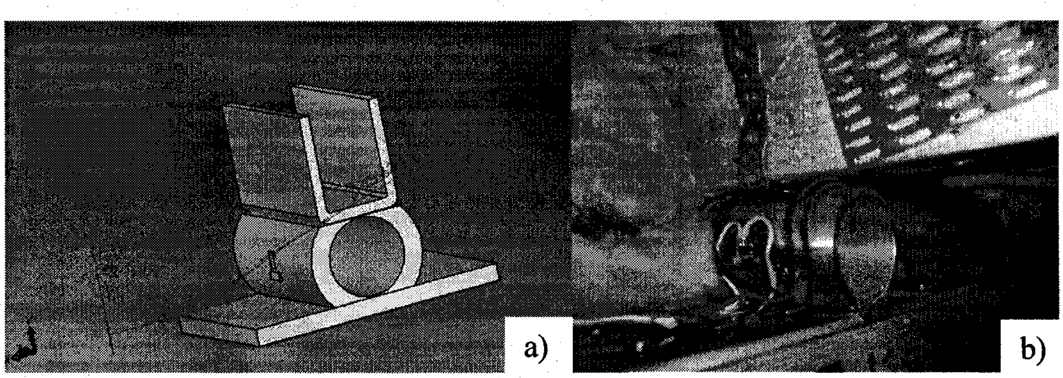

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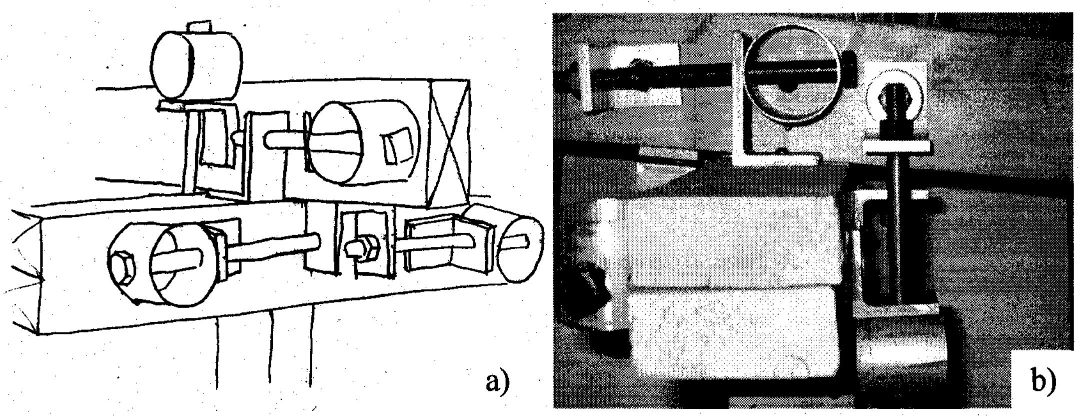
a) concept
b) actual



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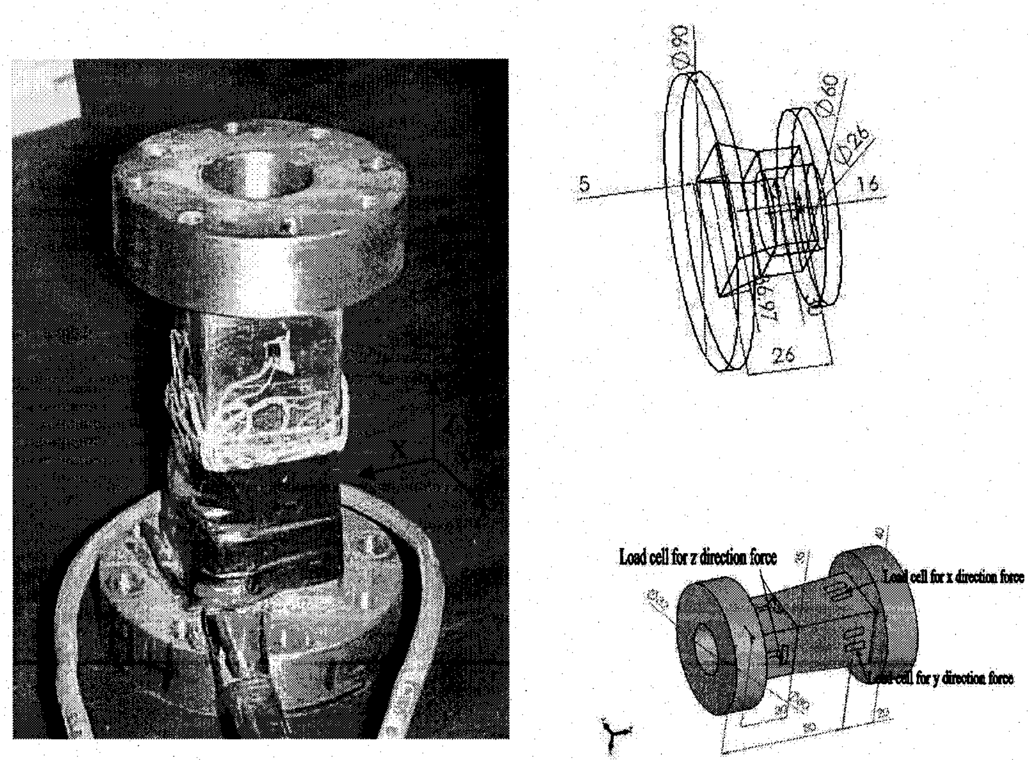
a) concept
b) actual



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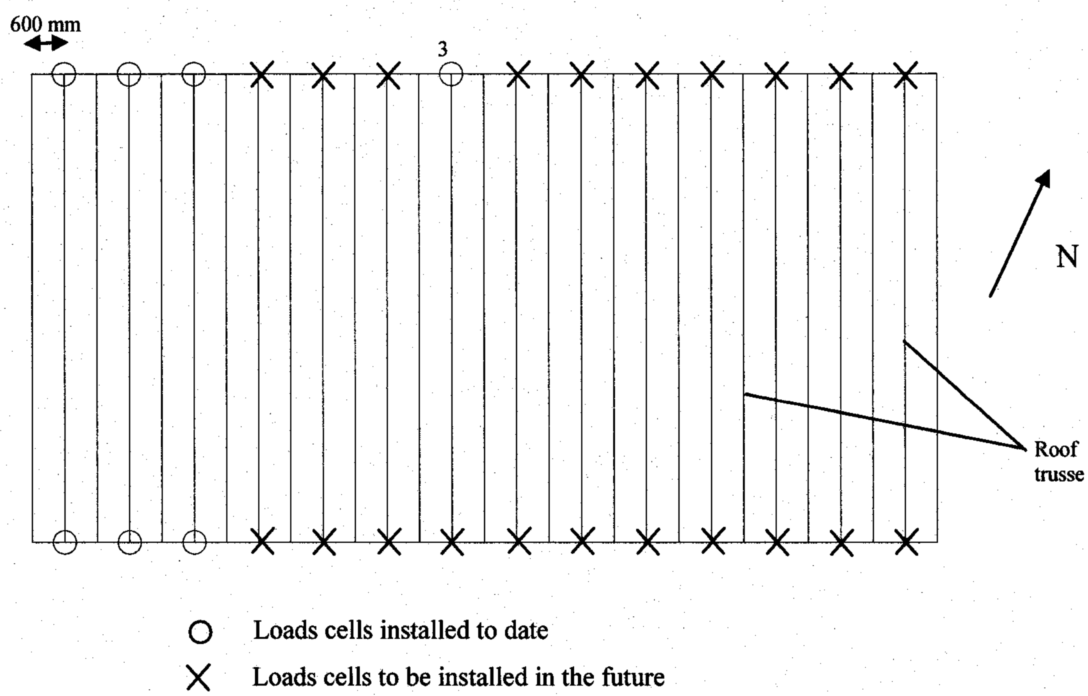

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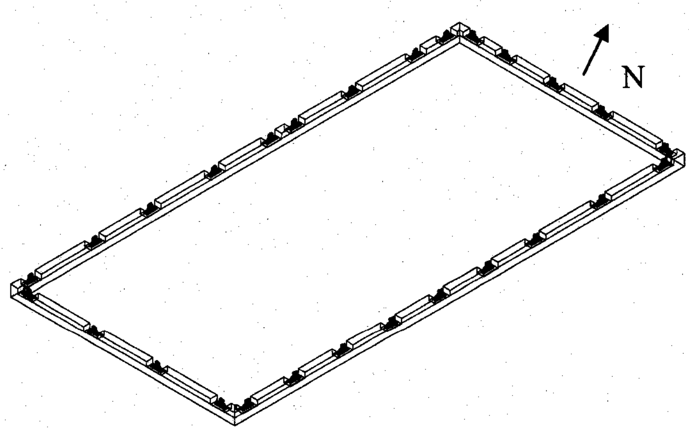

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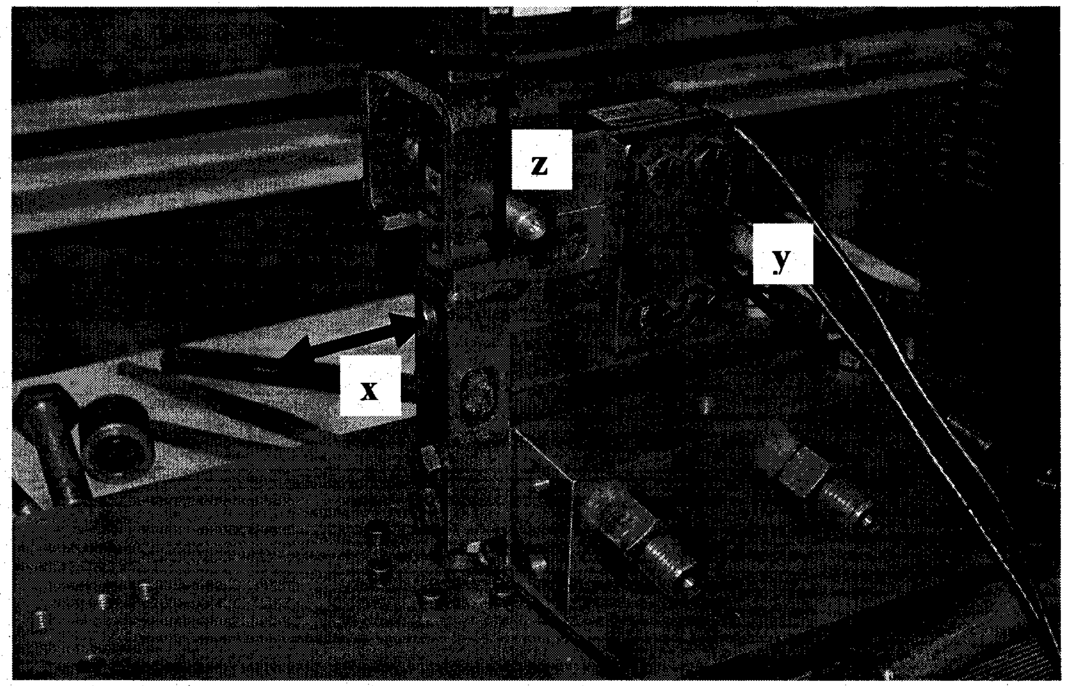

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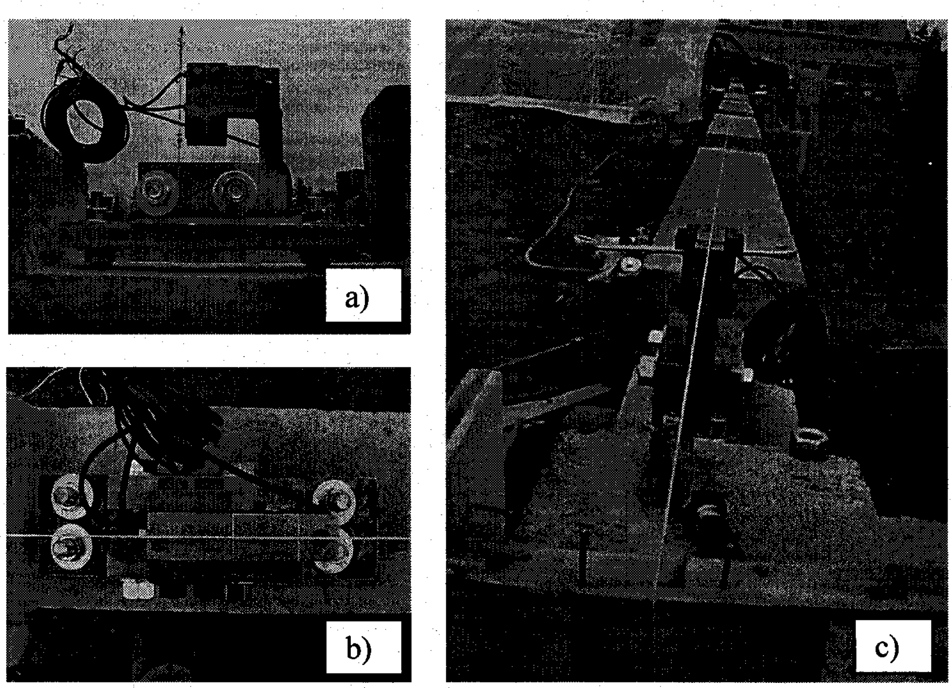

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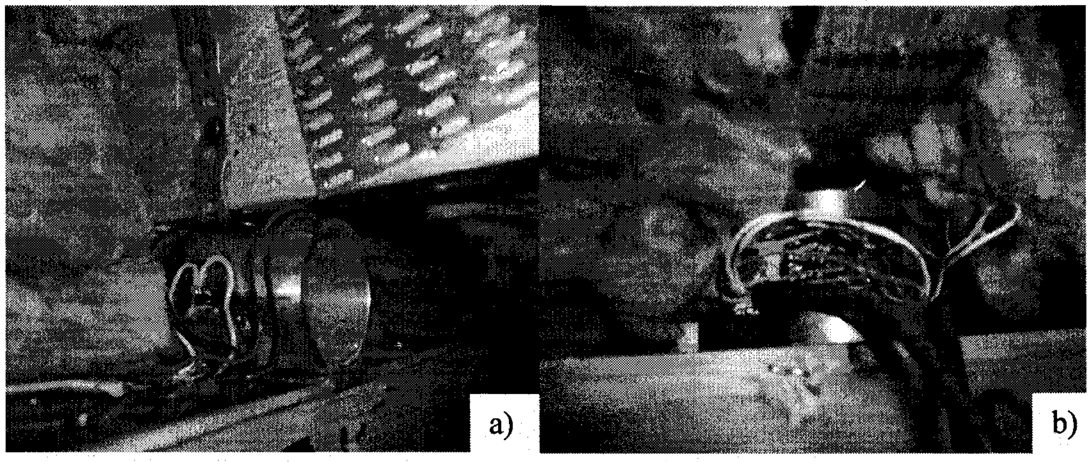

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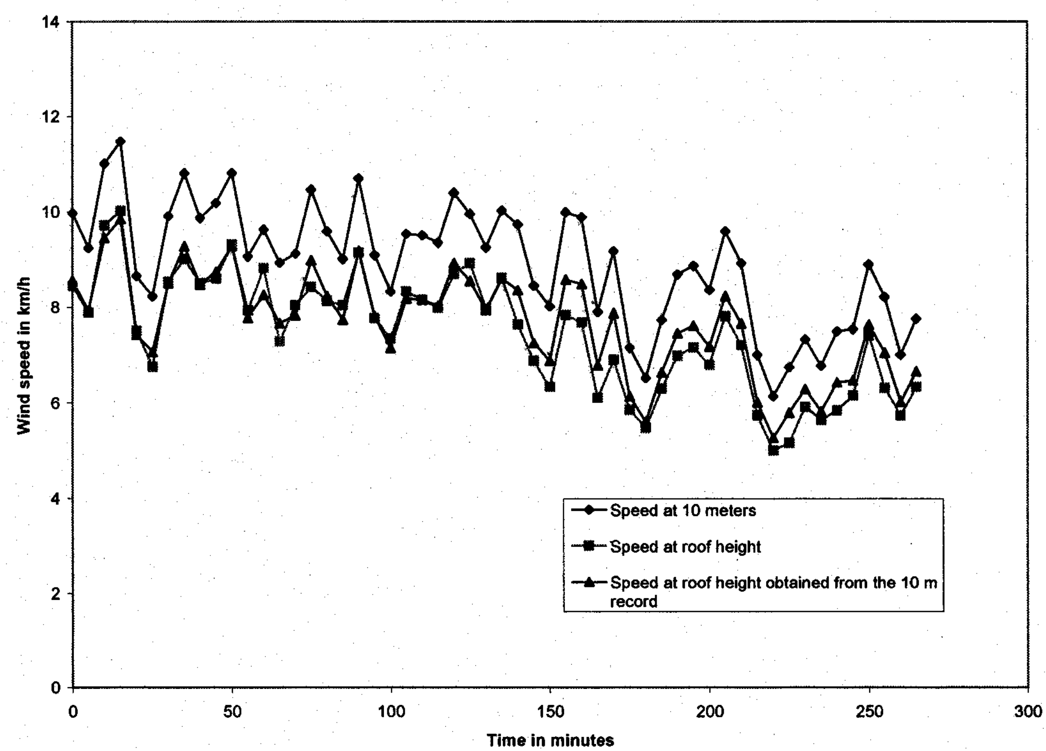

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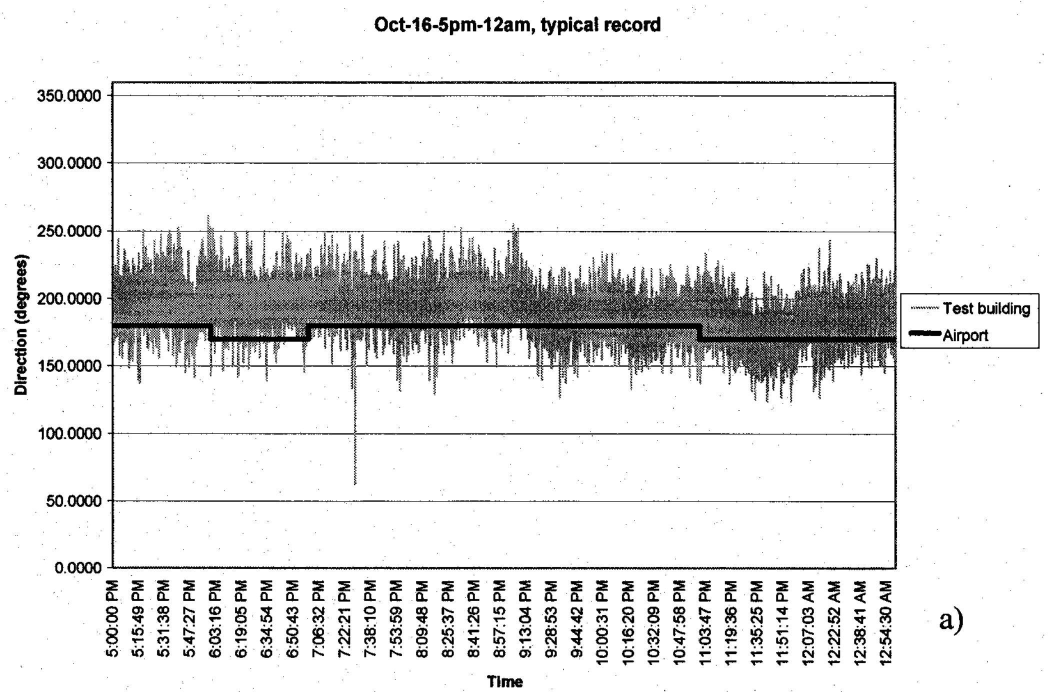
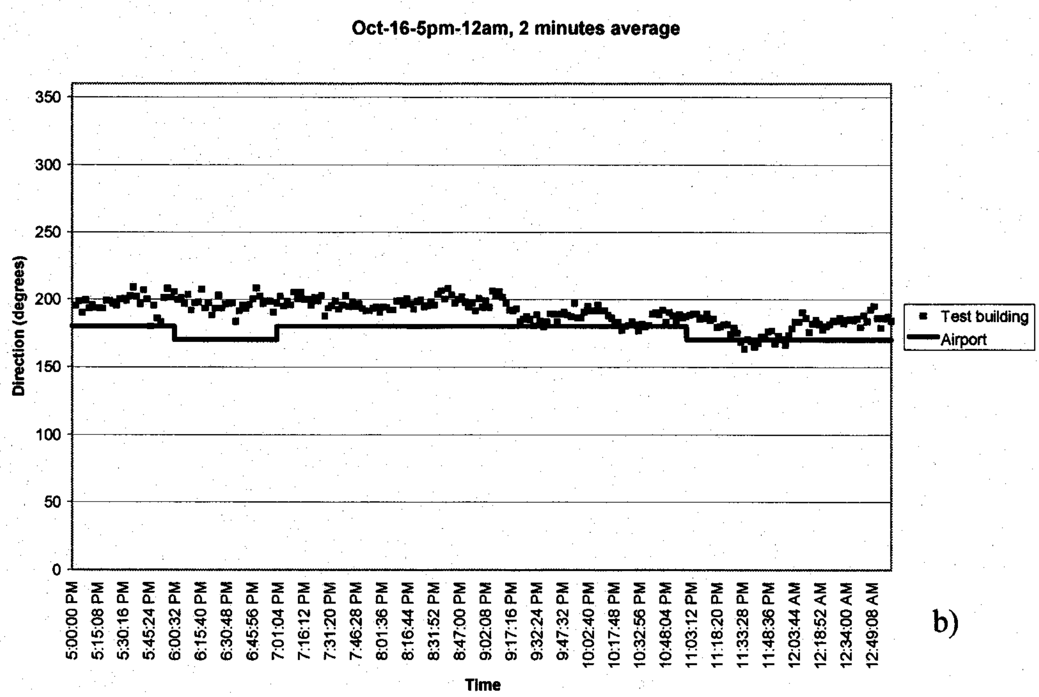


a)



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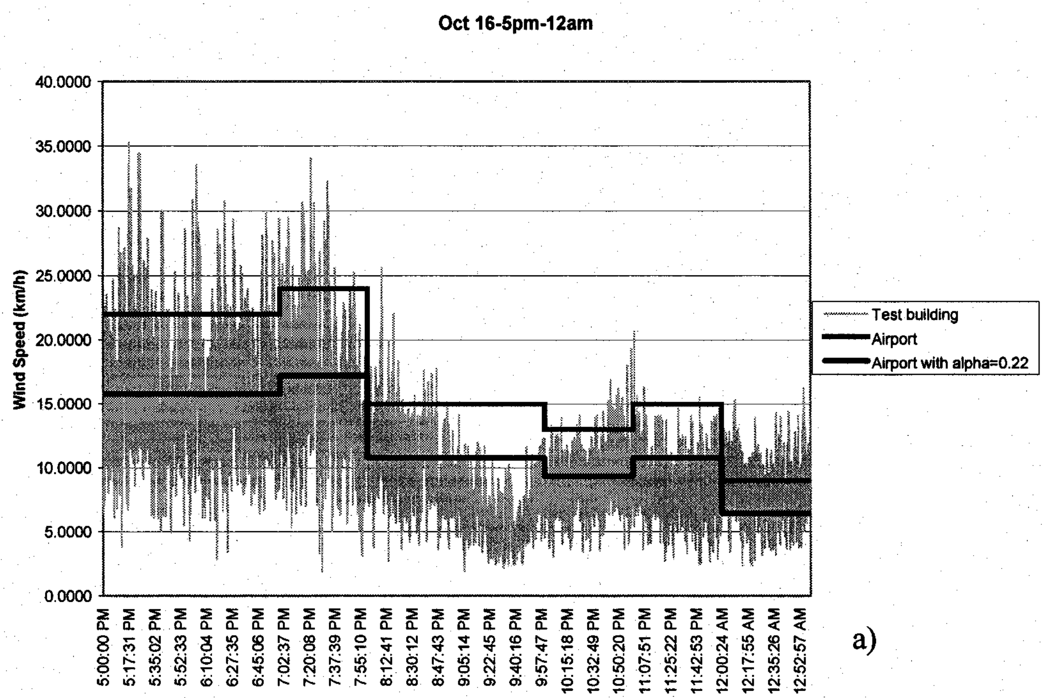

a)

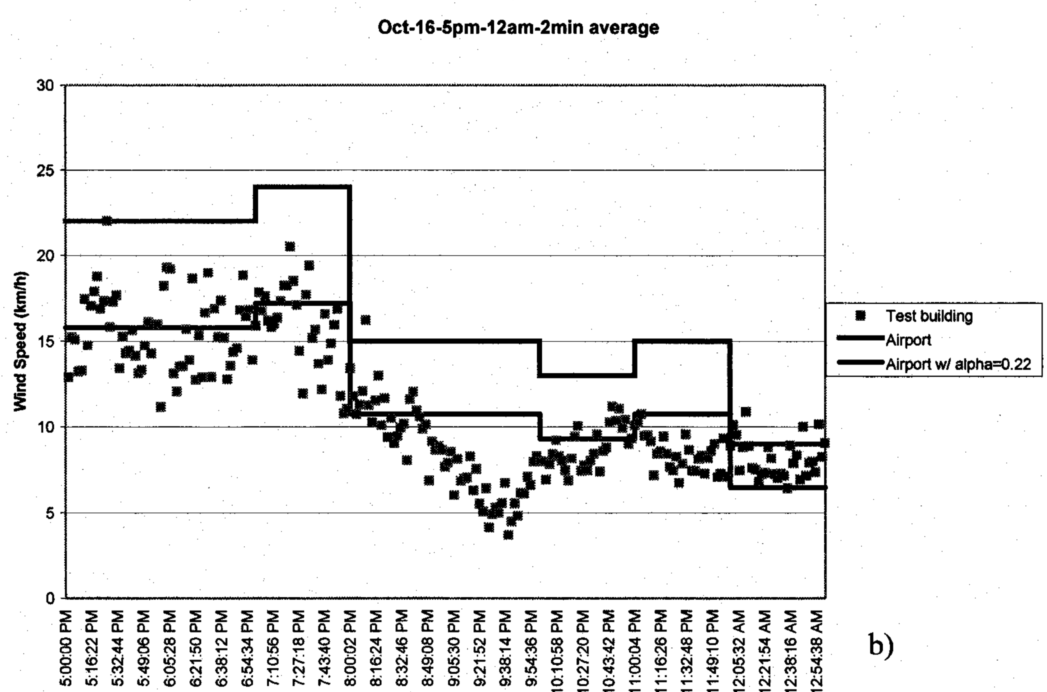

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Figure 15: Comparison of speed data with the airport,
a) typical record
b) two-minute averages of data in a)

CHAPTER 6

Controlled Load Tests

The monitoring research study in Chapter 4 established the link between the full-scale pressure distribution on the test structure and the reduced-scale model studied in the wind tunnel. Since the building already existed (Forintek shed), it was not possible to characterize the properties of the various components, which limited the modeling aspects of the study.

Many experimental and analytical studies have been conducted on the behaviour of elements such as joists, studs and sheathing elements, or subsystems such as shear walls and roof diaphragms. This approach ignores system effects and does not take into account the effects of realistic boundary conditions as found in structures, despite the evidence that such effects have large influence on the structural behaviour.

The methodology in this chapter is to carry out controlled loading experiments on finished and realistic light-frame timber buildings.

The *in situ* tests will only provide data about and verify whole-building models for the building response at low load levels. It is, therefore, important to know the load paths and the magnitude of the forces and deformations along the boundaries of the subsystems or components. Subsystem failure mechanisms can be studied in isolation through relatively simple and inexpensive testing and analysis, for example in the laboratory.

Structural response and whole building modeling of a single-storey wood light-frame house to applied static load

By G. Doudak[1], G. McClure[2], I. Smith[3]

Abstract: Despite clear evidence that low-rise timber buildings are vulnerable to extreme storm and seismic events, few timber buildings have been tested at full-scale, even under laboratory conditions. The goal of the research project is to correlate internal forces and deformations to those predicted by numerical whole-building models. The results from the controlled static loads on the walls indicated significant load sharing caused by the roof system and through the transverse walls. The expected load path was not observed, and there was a clear distinction in the roof stiffness between the longitudinal and transverse direction of the house. In the vertical loading of the roof, the load was concentrated in a small region around the load application to the roof surface, but was more dispersed at the foundation level. A 3-D finite element model correctly predicted the behaviour of the structure by predicting the forces in the wall under load and the transfer of forces to the remaining walls, along with the interaction between the roof and the shear walls and among the shear walls.

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Key Words: Full-scale testing load distribution, monitoring, timber construction.

Introduction

Light-frame superstructures behave as an assembly of ribbed plates that are folded and interlocked. It is very difficult to distinguish between structural and architectural components due to the complexity of modern building shapes combined with their structural redundancy (Reardon and Henderson 1996). This also makes understanding of the load-carrying mechanisms very difficult, besides rendering the prediction of forces in the components based on simple notions of tributary areas is unreliable. Despite clear evidence that low-rise timber buildings are vulnerable to extreme storm and seismic events (Smolka 1996), few timber buildings have been tested at full scale, even under laboratory conditions. To establish the characteristics of the house structure and to develop a reference to calibrate a finite element model, static load tests were performed on the structure.

Research objectives

The goal of the research is to devise and demonstrate methods of identifying load paths in light-frame wood buildings subject to controlled static loading. Specific objectives are to:

- Measure internal forces and deformations in the test building and correlate them with applied loads.

- Develop numerical whole-building models for structural analysis.

Test building

The test building is a single-storey house-like structure located in Fredericton, New Brunswick, Canada. The house meets the design specification of the Canadian Mortgage and Housing Corporation (CMHC 1997) for platform construction, with some modifications to accommodate for placement of monitoring instrumentation. The building is a typical North American "bungalow" style single-family residence with a floor platform over a perimeter concrete foundation (8.5 m x 17 m footprint) (Fig. 1). The house has regular plan geometry and duo-pitch roof (4/12 slopes). The foundation is a continuous concrete strip footing with 250 mm width. Rectangular cut-outs (180 mm x 545 mm) in the frost wall (Fig. 2) accommodate tri-axial load cells. The wall frames are assembled from studs, 38x89 mm (2"x4") S-P-F (Spruce Pine Fir) lumber framing at 600 mm spacing. There is a double 38x89 mm top plate tying the wall panels together and one bottom plate. Wall sheathing is 9.5 mm thick, 1.22 x 2.44 m exterior OSB panels nailed at a 150/300 mm spacing (150 mm on perimeter, 300 mm inside). The roof is constructed of trussed rafters laid out without blocking at 600 mm spacing and it is sheathed with 13 mm thick OSB, fastened by nails at 150/300 spacing.

The load cells, at the foundation level, are the only continuity between the superstructure and foundation, which permits the super structure with or without applied loads to be weighed in horizontal and vertical senses. The load cells provide point and total weights to be estimated. A second series of single-axis loads permits point estimates of vertical forces flowing from the roof system to tops of walls. In addition to giving precise distributions of reaction forces, the lower series of load cells enable the measurements of the total applied forces, whereas the upper series is intended to provide local observations of the incoming roof forces.

Presently, the only openings the test house are two pedestrian doors. The intent is to add window openings later. The wall cladding, plasterboards as well as internal partitions will also be added later. More details about the house construction and the installation of the load cells can be found in Chapter 5.

Instrumentation

Measurements made on the test structure include: The applied load, internal forces at foundation-to-superstructure and wall-to-roof interfaces, and displacements (deflections and distortions) of stud walls and roof trusses. The test house is equipped with two series of load cells to measure vertical and horizontal forces parallel to external wall directions.

A sketch showing the attachment of load cells between the roof and the wall and between the walls and the foundation is shown in Fig. 3. The schematic detail in Fig. 3 indicates the connection of the foundation load cells to the bottom plate via a steel channel section, by mean of 12.5 mm (½ in.) bolt, which creates a pin connection releasing the moments on the load cells in the plane of the panels. A detailed description of the development and the installation of the load cells can be found in Chapter 5.

Finite element modeling

The numerical simulation models include detailed finite element models of substructures and inter-component connections, and the global models of the entire system. The software chosen was *SAP2000* Nonlinear Version 8 (CSI 1997). All physical elements of the structure are represented in the model. The main framing and sheathing elements were modeled as linear elements, whereas all nonlinearity was included in the connections represented by nonlinear links. Detailed description of the model can be found in Chapters 3 and 4. A *SAP2000* rendering of the 3-D house model is shown in Fig. 4.

Testing program

Several controlled static loading tests were conducted to calibrate the 3-D finite element model built for the UNB house structure. The goal of this static testing program is to determine the distribution of the reaction forces for a range of load configurations while remaining in the elastic range of response. Several loading set-ups were conducted to ensure a complete description of the behaviour of the house. They are described in Table 1 and represented schematically in Fig. 5.

Point load on the wall

All point loads on the walls were applied by pushing against a calibrated 15 kN load ring (Fig. 6 a) pushing on a stiff steel plate. All loads were applied gradually in a quasi-static regime, and each loading lasted no longer than 15 minutes. The maximum applied load was set to be 10 kN, based on calculation of the total drag with estimated pressure coefficient (C_p) values in the two main building directions. The deflection at the point loaded was also monitored to ensure that the response remained elastic.

Uniform load on the roof

A patch of pressure load (1350 mm x 1040 mm), consisting of bundles of roof shingles, was applied on the roof (Fig. 6b). The applied loads weighted before its application were measure to be 4.49 kN (3.2 kN/m^2). This load was applied at several locations on the roof, on top of a shear wall and on top of the two door openings (Fig. 1).

Deflection measurements were taken at several locations using LVDT's (Fig. 6c), which measured the horizontal displacement of the house at the top-plate level in the case of point loading of the walls, and the vertical deflection of the roof trusses in the case of roof loading.

Results and discussion

Applied load tests

It should be noted in the following that when dealing with the horizontal load application, the relevant forces, referred to in this section, are measured using the wall-to-foundation load cells. Also, the results are reported for the main direction of response.

Test 1: Horizontal point load in the Southeast direction (parallel to Wall 1)

The point load in Test 1 was applied to the second truss located approximately 600 mm away from the edge, as shown in Table 1. The load was applied monotonically in 1 kN increments, up to a maximum value of 8 kN. The load distribution is represented in Fig. 7. The size of the circles in the figures indicates the magnitude of load. The dark colour represents compression or positive pressure, and the light colour implies suction or negative pressure. The location of the circles corresponds to the position of the load cells in the real structure.

The results from this test showed that only approximately 52% of the applied load was taken by shear Wall 1 in the Y-direction, with the remainder of the load being distributed to the other three walls; Walls 2 and 3 took about 19 and 22 % of the load, respectively, and Wall no. 4 took 8 % of the load in the Y-direction. It is important to note that even though most of the load is taken by Wall 1, the rest of the load is distributed almost evenly to Walls 2 and 3. The stiffness of the roof system was also sufficient to distribute the load to Wall no. 4, far away from the load application point. In the Z-direction (vertical), the load was concentrated near the stiffer corners, which emphasizes the importance of the rigidity of the roof system under the sway deformation of the structure.

Overall, the internal forces are rather concentrated near the corners of the plan: the load cell near the corner between Walls 3 and 4, carries 15 % (2.6% of total load) of the load cell near the load application.

The relationship between the loads in roof-to-wall load cells and the applied load was linear. Approximately 33% of the roof load was transferred to the load cell nearest the load application point via the wall top plate.

Test 2: Horizontal point load in the Southeast direction at mid-span of Wall 3

This loading scenario consists of applying a maximum load of 3 kN to Wall 3 at mid-span (Table 1). The static load was applied gradually to the top-plate of the wall.

The results indicate that the load applied (in the Y direction) is distributed to all walls even though it is assumed in existing design practices that such load would only be resisted by the two shear walls (Walls 1 and 4) parallel to the direction of load application. It is also noteworthy that the load has the tendency to be transferred to the four corners (Fig. 8). The Y-load at the corner load cell (Load cell 22 under Wall 1) is four times larger than the load measured at the load cell under the load. Also, it is noted that the house structure is much more flexible under this loading scenario compared to the loading in Test 1.

Test 3: Horizontal point load in the Northeast direction (parallel to Wall 3)

A static load of approximately 8 kN was applied in the longitudinal direction of the house in the Northeast direction at the level of the wall top plate.

Significant load sharing was observed in this loading scenario, where as much as 1/3 of the load was distributed to the non-loaded walls, mostly by the roof system but also through the transverse walls. It is important to note that the reactions in the X-direction (Fig. 9) for the transverse walls are very small compared to the reaction under the in-plane walls. This indicates that most of the load transfer/redistribution occurred via the rigid roof system. The vertical reactions (Z-components) are also distributed to the non-loaded walls.

The roof diaphragm is more rigid in the NE-SW direction than in the perpendicular direction (NW-SE). In this loading scenario, more loads were transferred to the parallel non-loaded wall than in the Test 1 case.

Test 4: Horizontal point load in the Northeast direction at mid-span of Wall 1

A small load of 1.14 kN was applied to Wall 1 at mid-span, in the Northeast direction (Table 1). The reason for this low load level is the large flexibility of the structure at this location. The limiting criterion was therefore set by the deformation of the structure.

The load was distributed almost evenly to the two parallel shear walls with almost no significant reactions under the two transverse walls. This is evidence of the rigidity of the roof system in the direction of the building axis.

Test 5: Horizontal point load in the Northeast direction at mid-span at top of roof (Wall 1)

The load was applied to Wall 1 at mid-span and at the roof ridge, in the Northeast direction, similar to the configurations in Test 4.

In the X-direction, it is obvious that the assumption of "stiff roof on flexible shear walls" is valid (Fig. 10). The load clearly is distributed according to the stiffness of the studs.

In all load cases of applied horizontal load on the walls, even though the load was applied as much as possible along the axis of the house structure, a small rotation occurred. The geometry in the tested structure was very simple with no irregular shapes and no internal partitioning. The only difference between Walls 2 and 3 was the location of the door opening. For more complex building shapes or complex opening layouts, torsion will always play a large role, even if the load is parallel to the plan axis. Table 2 summarizes the distribution of the load among the walls of the test structure. It is obvious that the applied load was distributed to the non-loaded walls by means of roof system and the transverse walls.

Test 6-a and b: Vertical point load on the roof

The point load on the roof was applied on top of roof load cell 4, in alignment with Wall 3. The roof load cell immediately under the load carried about 57% of the applied load.

At the roof level, the load is concentrated in a small region around the load application point, with as much as 92% of the load being carried by the three roof load cells in the vicinity of the loading point. This is not the case when the foundation load cells are considered, where the load is more dispersed away from the line of application of the external load (Fig. 11). With the traditional tributary area assumption, a load of 4.2 kN would be transferred to Load cells 28 and 31. However, only 62% of that load is actually measured at these two load cells. Most of the remainder of the load is carried by the adjacent wall (Wall 1), attracted by the stiffness of the building corner. Also, the traditional approach suggests that the roof would transfer the applied vertical load via the trusses to the two long walls (Walls 2 and 3). However, the test results show that 19% of the load went to Load cell 22 in Wall 1, even though there is no direct connection between Wall 1 and the roof trusses. Only the OSB sheathing at the gable end provides continuity between the roof and Wall 1. This could be explained by the load being transferred directly from Wall 3 to Wall 1. The entire wall (Wall 1) carried a considerable amount of the total vertical load, approximately 28%.

By conventional design practice, load cells 28 and 31 should have the same reading, but in fact Load cell 31 has only 60% of the load measured by load cell 28. This is due to the stiffness of the roof and the location of Load cell 28 near the stiff corner. This is also explained schematically in Fig. 12.

The same load was moved toward the interior of the building (on plan), to a position approximately 1 m from the outside wall (Case 6 b). The force measured in the load cell under the load application point decreased by 18%. This is considerably higher than predicted by simple tributary area theory, where the load is expected to decrease by about 8% only. Another difference in the load distribution between the two cases was in transfer of the load to the adjacent wall (Wall 1). In the case where the load was applied immediately on top of the wall, 19% of the load was transferred to the first load cell under Wall 1 (Load Cell 22). In the case where the load was applied further away from the wall (1 m away), 14% of the applied load was transferred to Load Cell 22. This is due to the fact that more loads were transferred to the parallel wall (Wall 2) in the latter case. The roof diaphragm plays a more important role in this case. The load in the load cells under Wall 2 increased by approximately 40%.

Test 6-c: Vertical point load on the roof, on top of a door opening

When the load is applied on top of the door opening, it is not distributed as far away from the load application point as in the case where load is applied away from a wall opening. Only 1.6% of the load was transferred to the parallel wall (Wall 2), while 3.6% was obtained in Test 6-a. It appears that the force would rather flow in the framing around the door and stay in the vicinity of the load application point.

It is seen from Fig. 13 that even though the load cells 19, 47 and 16 under Wall 2 are expected to receive almost the same load magnitude, load cells 16 and 19 experienced slightly higher loads. The wall studs immediately on top of these particular load cells are double studs at the connection between the two wall panels, which provide further evidence that stiffer studs attract more load.

Comparison to FE model

The main goal of the project is to predict load paths in the structure. As was clearly seen from the physical tests, the load did not always follow a path that was obvious.

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, Table 3 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. 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.

Summary and conclusions

Controlled static loads were applied to a single-storey house and the force flows in the system were measured, and a finite element model was built to predict the load path in the structure.

The results from controlled static loads on the walls indicated that the load was distributed to all walls. In some load cases, only about half of the applied load was taken by a wall directly beneath an applied load, with the remainder of the load being distributed to the other walls. Significant load sharing was observed. Mostly, this reflected not only the rigidity of the roof, but also the rigidity of transverse walls. The stiffness of the roof was sufficient to distribute load to walls farthest away from the load application point. In addition, the expected vertical paths for load were not observed. It was also found that the internal forces are concentrated near the corners of the building, due to their high stiffness.

There was a clear distinction in the roof stiffness between the longitudinal and transverse direction of the house, with the roof being more rigid in the longitudinal direction than in the transverse direction.

Under vertical loading on the roof, at the roof-to-wall interface, the load was concentrated in a small region of the building plan around the load application point. This was not the case at the superstructure-to-foundation interface. At this level, the load was dispersed away from the line of load application. Even at the foundation level, the load did not follow the common tributary area assumption. The test results also showed that the load was transferred to the transverse walls, even though there was only a nominal connection between the wall and the roof trusses. The load must then have been transferred directly from the longitudinal wall to the transverse wall. It was found that the load was attracted to locations where there was extra framing, for example at corners.

A 3-D finite element model was built to predict the point force measurements obtained from the load cells. In general, the errors in the numerical prediction were small. 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 was able to predict the very crucial interaction between the roof system and the walls and the interactions amongst walls.

Acknowledgements

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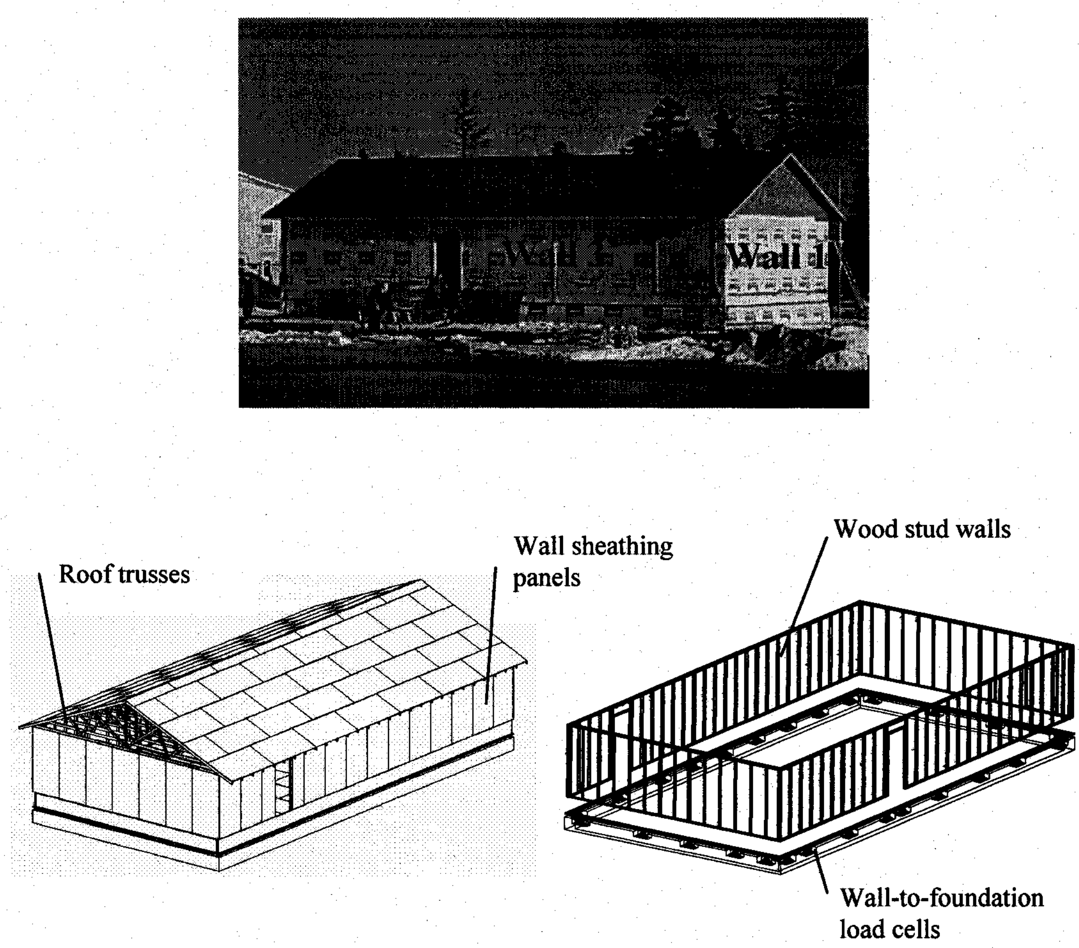


Figure 1: Schematic configuration of UNB test house

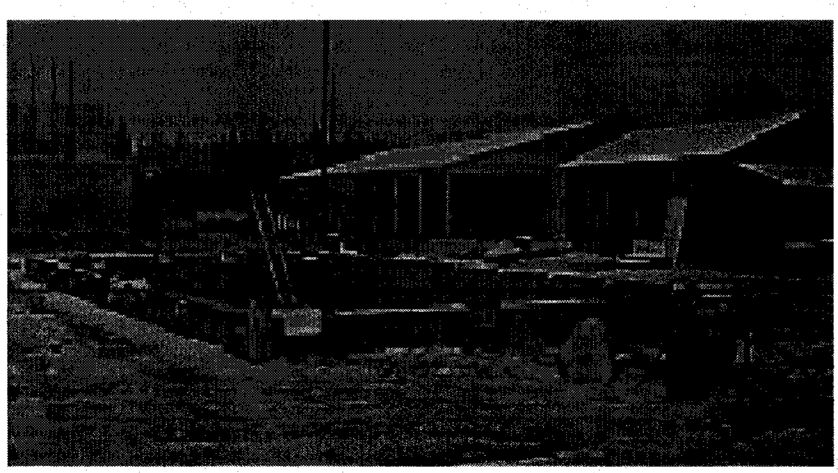


Figure 2: UNB test building foundation

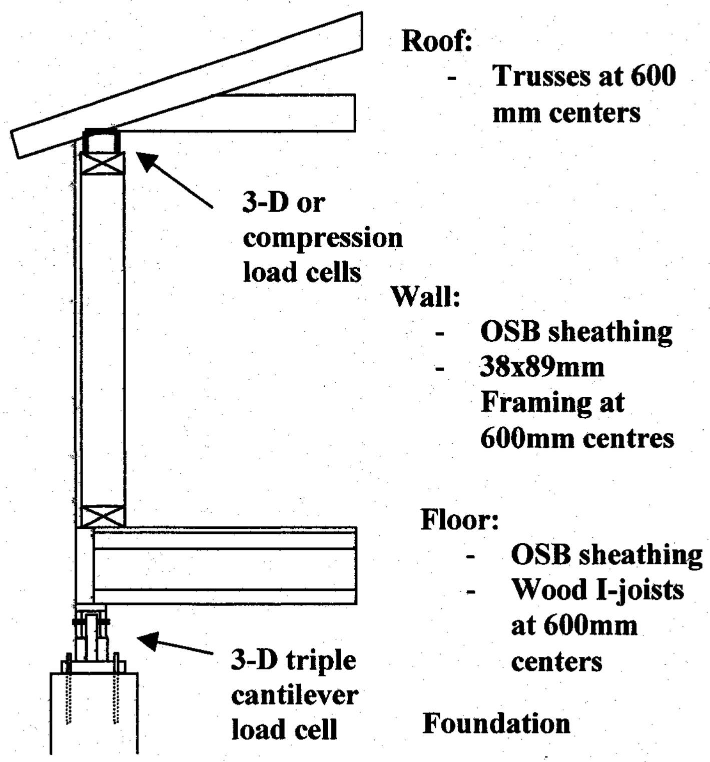


Figure 3: Placement of load cells at building interfaces

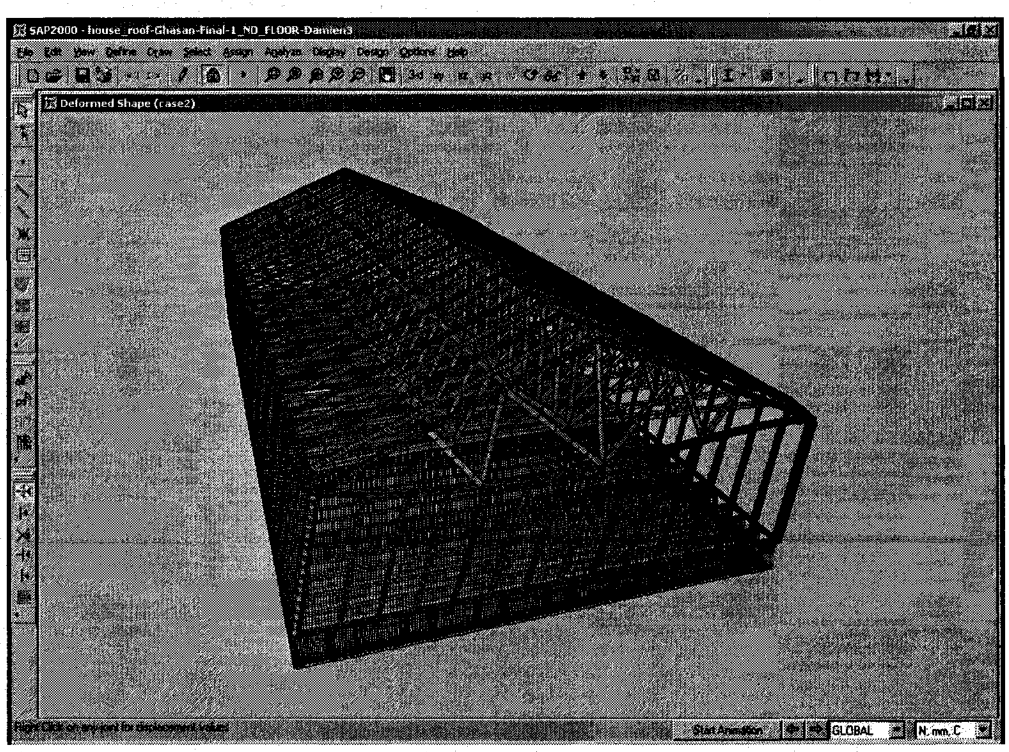


Figure 4: 3-D model

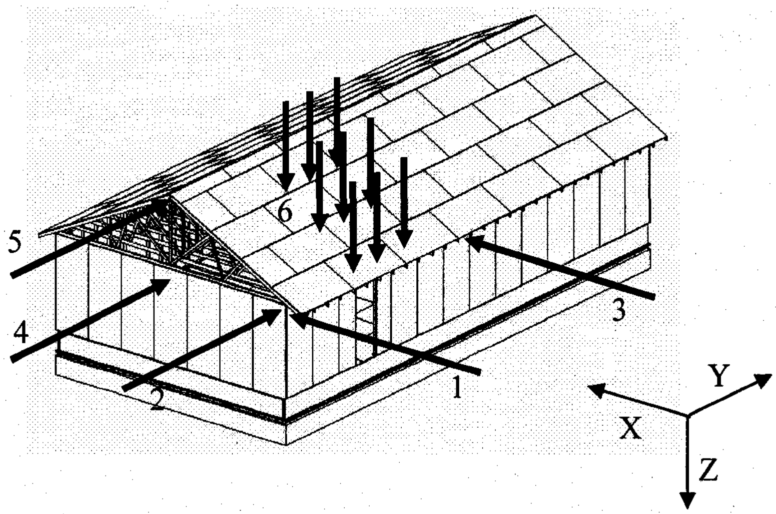


Figure 5: Applied load tests (see Table 1)

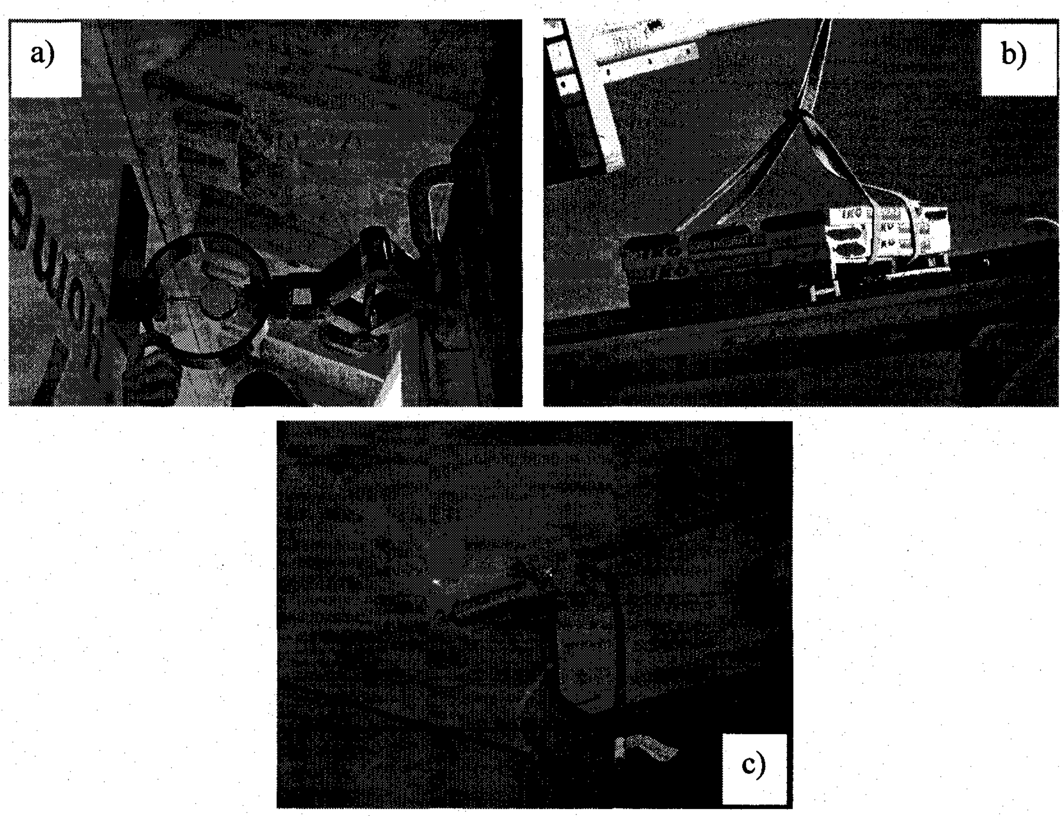


Figure 6: Full-scale testing and response
a) Load application for point load on the wall (load ring)
b) Patch pressure load application on the roof
c) LVDT for deformation measurements

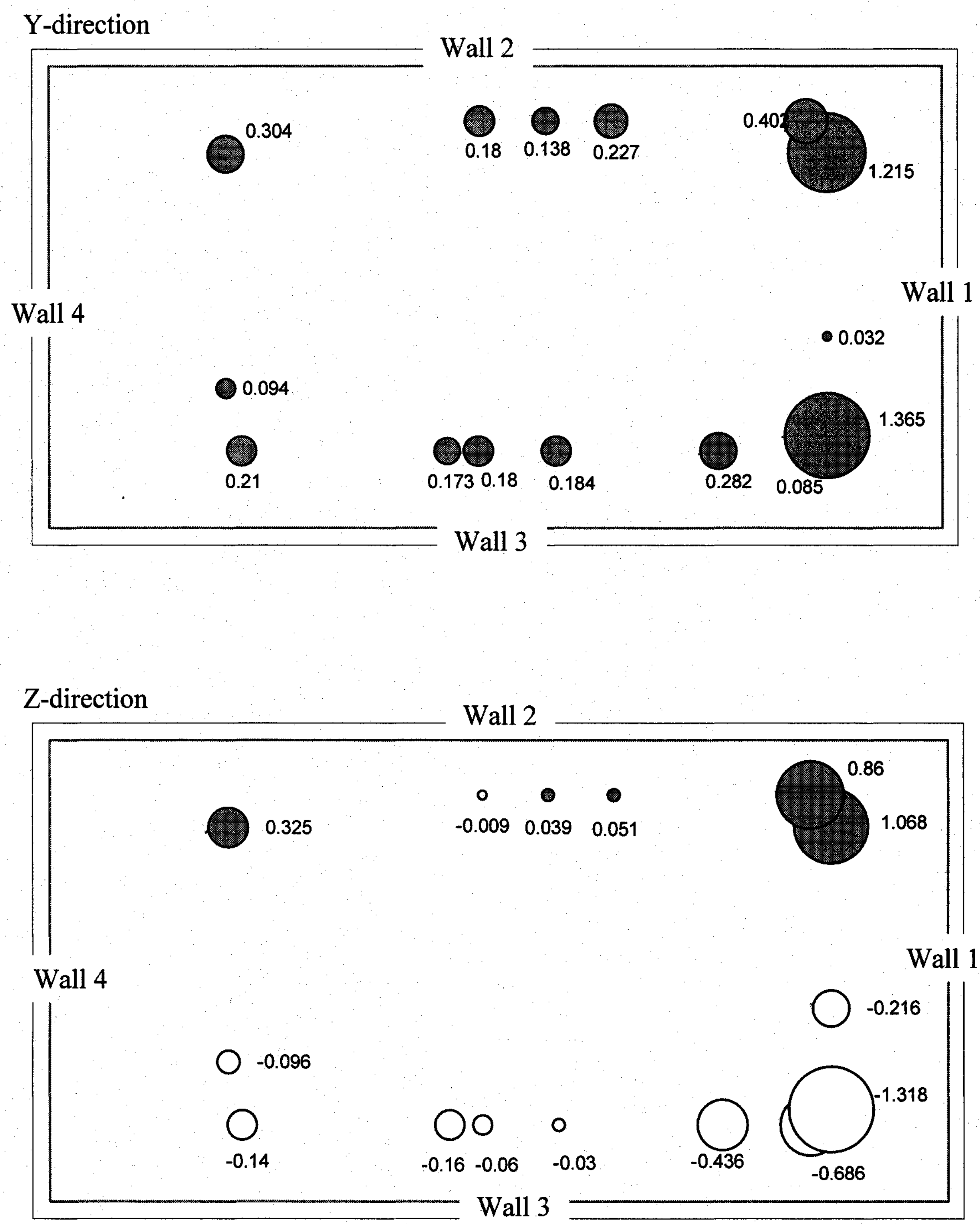


Figure 7: Results from test 1 in the Y, and Z directions (foundation load cells)

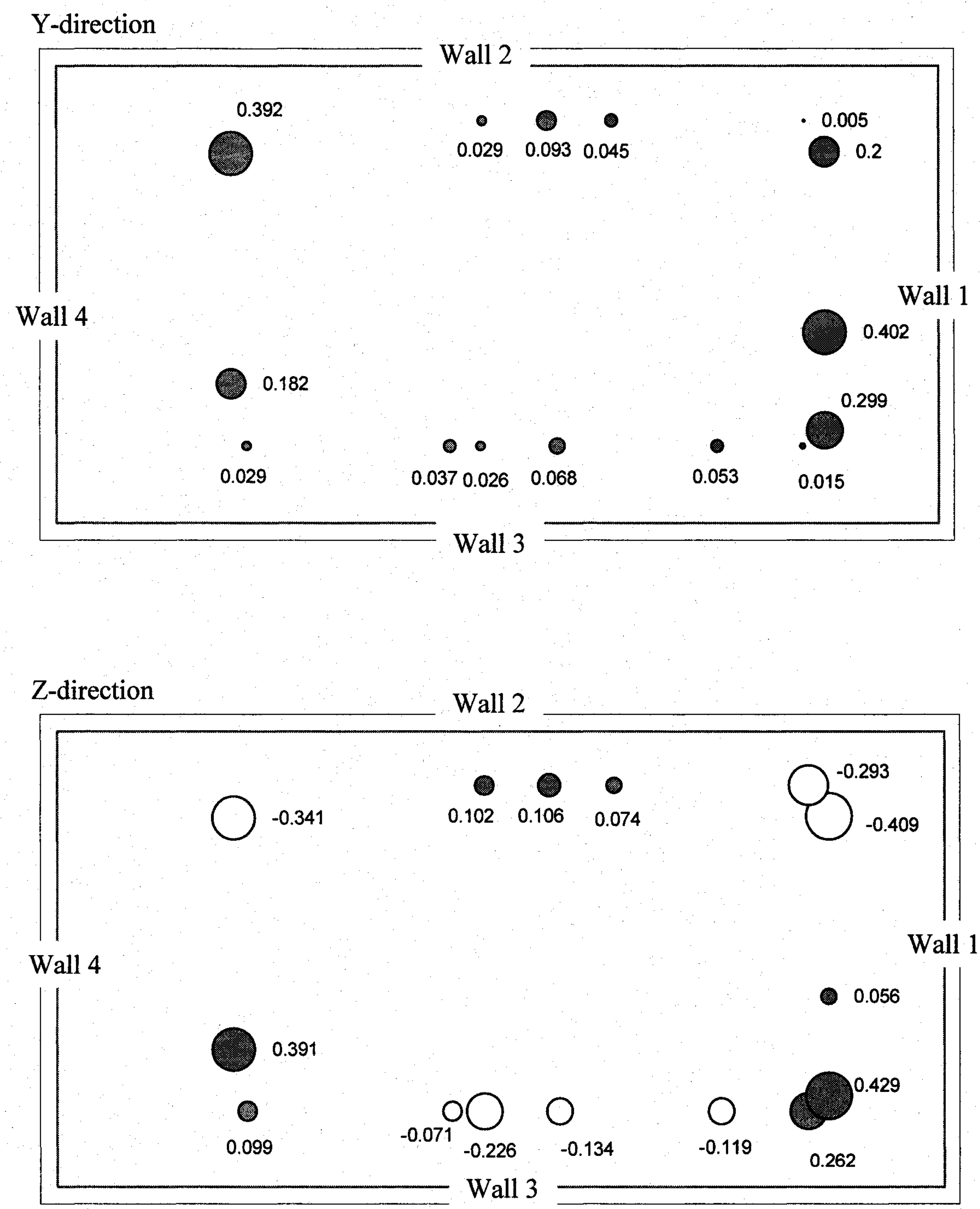


Figure 8: Results from test 2 in the Y and Z directions

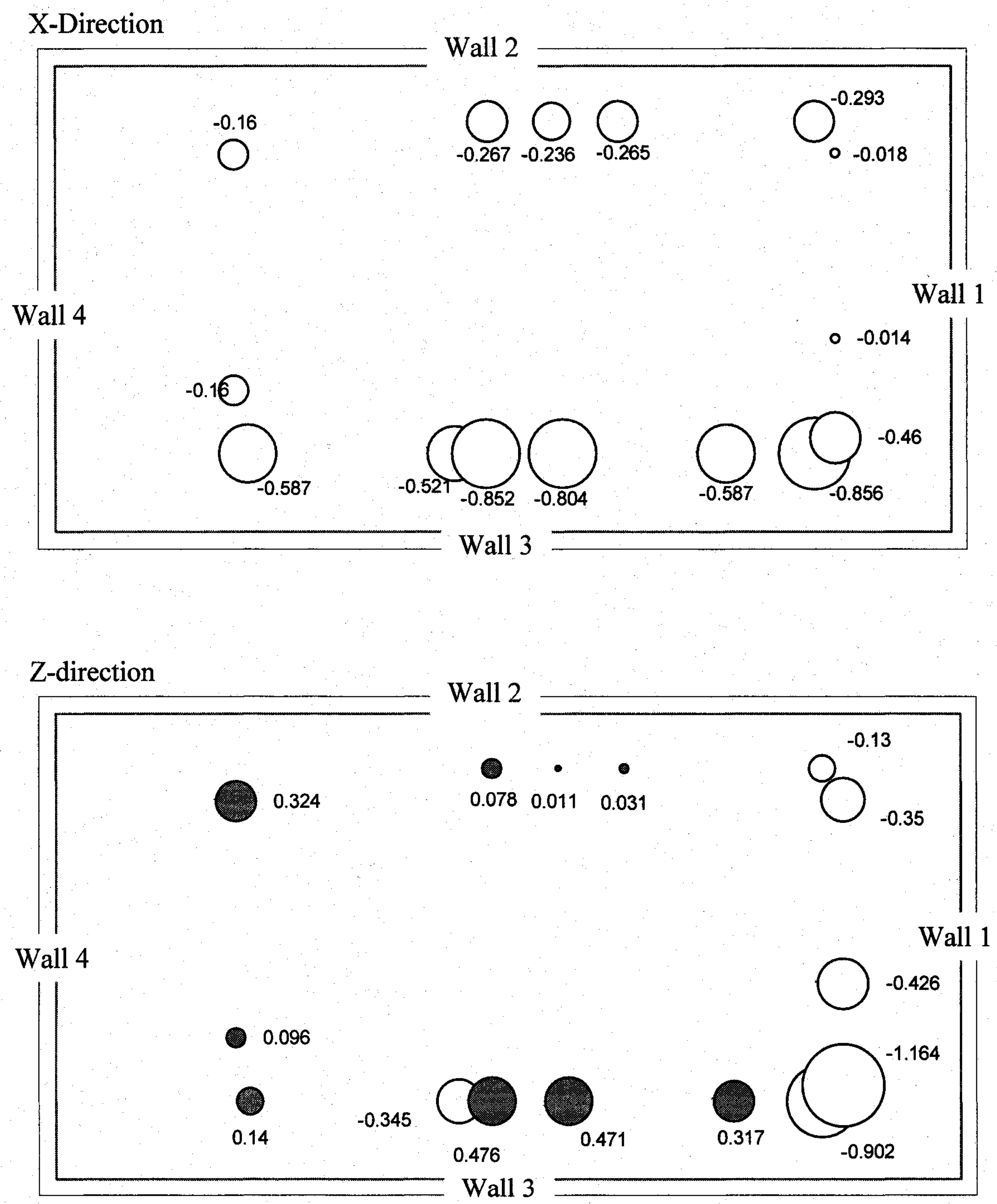


Figure 9: Results from test 3 in the X and Z directions

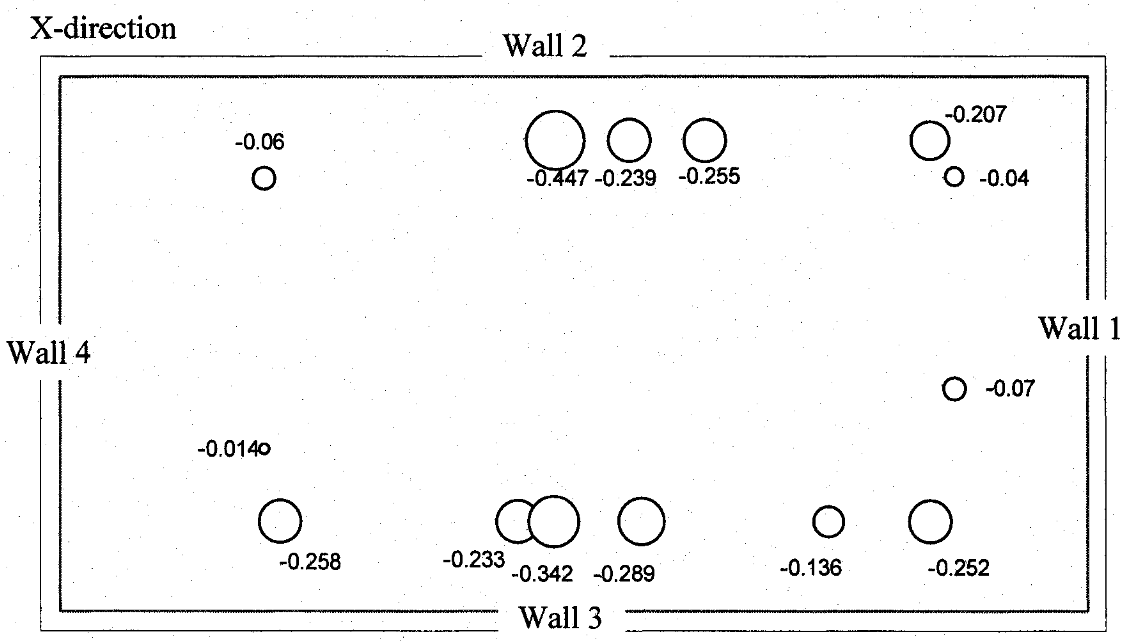


Figure 10: Results from test 5 in the X direction

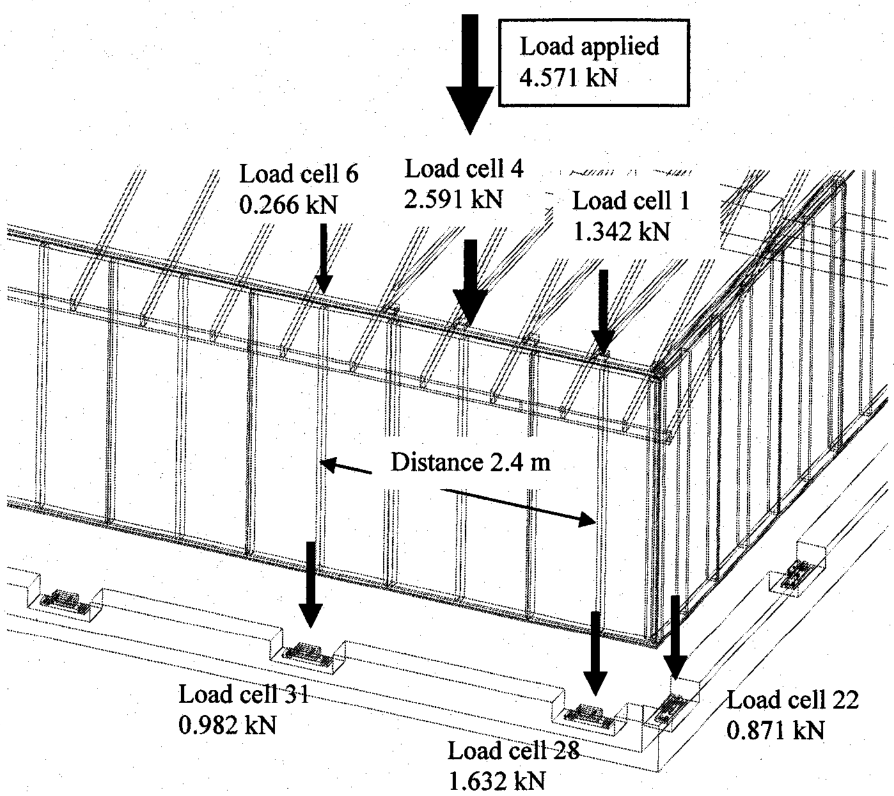


Figure 11: Flow of forces to roof and foundation load cells due to a point load on the roof

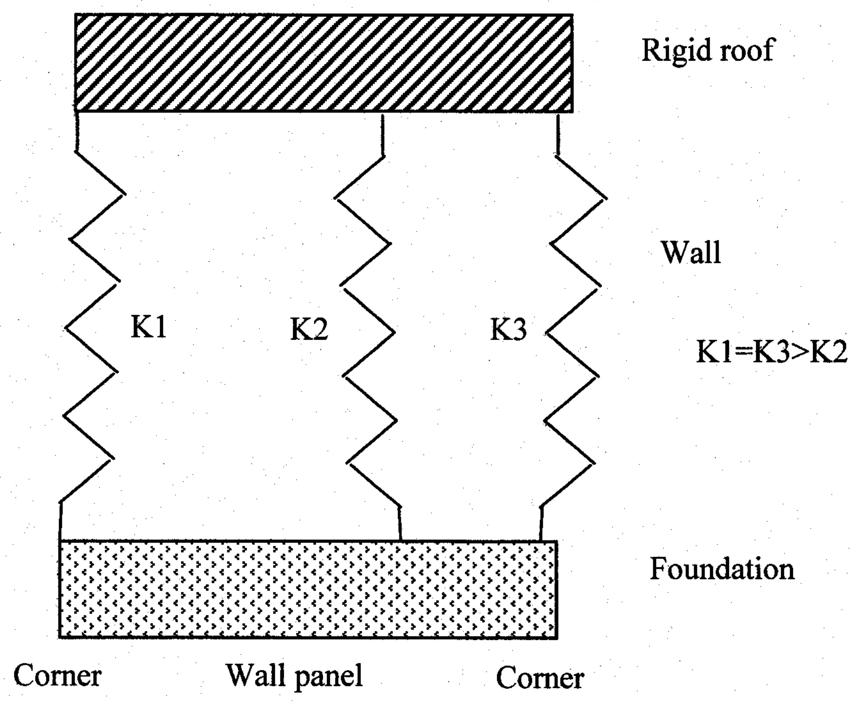


Figure 12: Analogy for the load transfer to the stiff corners

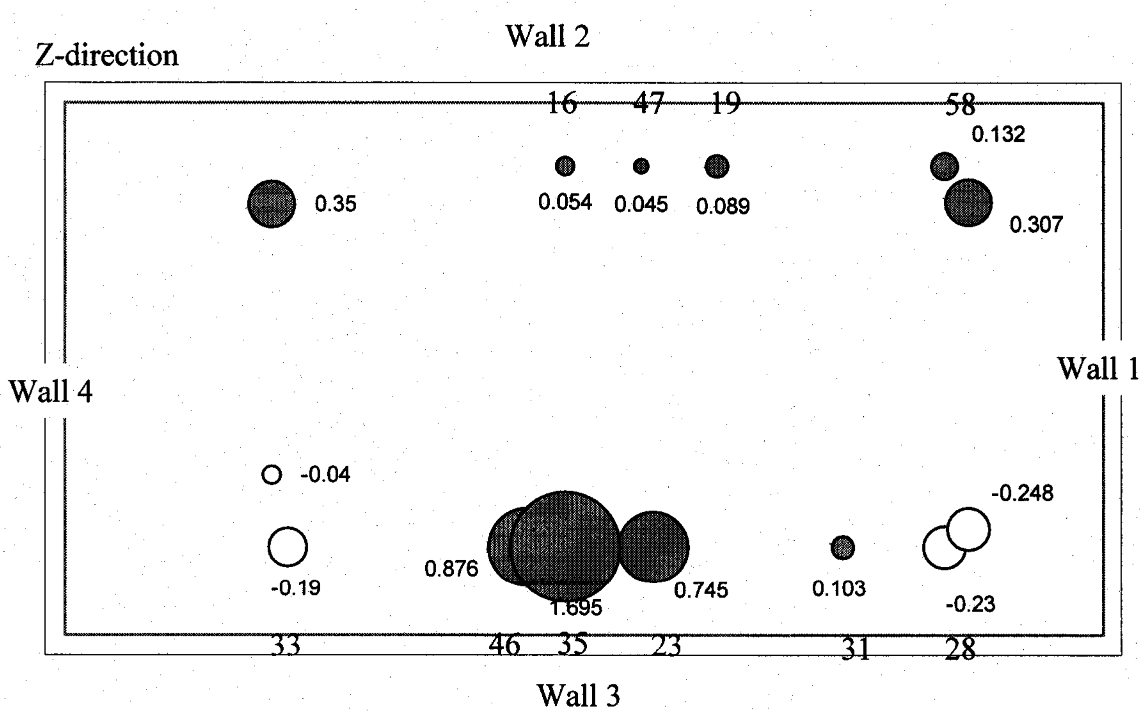


Figure 13: Foundation load cells, Z-component for point load on the roof on top of door opening

Table1: Summary of controlled static tests

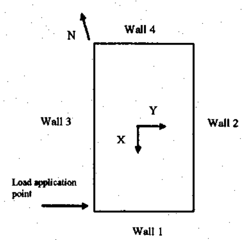
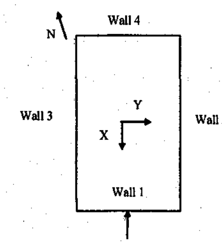
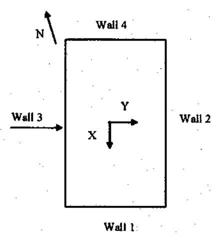
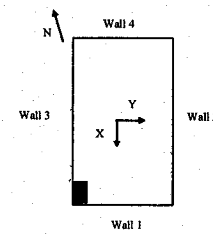
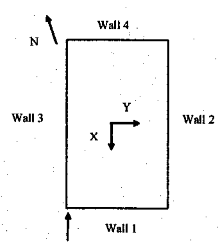
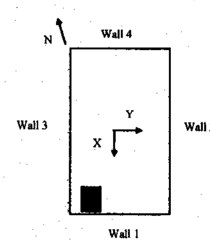
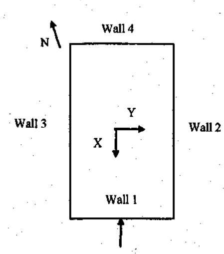
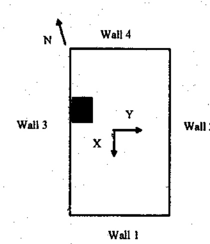
Test no. (1)	Test lay-out (2)	Test no. (3)	Test lay-out (4)
1		5	
2		6-a	
3		6-b	
4		6-c	

Table 2: Load distribution among the walls in the principal tests

	Summation of lateral forces (kN)				
Test no. (1)	**Direction (2)**	**Wall 1 (3)**	**Wall 2 (4)**	**Wall 3 (5)**	**Wall 4 (6)**
1	**Y** **Z+** **Z-**	2.61 1.07 1.53	0.95 0.95 NA	1.11 NA 1.51	0.40 0.33 0.10
2	**X** **Z+** **Z-**	0.90 0.49 0.41	0.17 0.22 0.29	0.23 0.36 0.55	0.57 0.39 0.34
3	**Y** **Z+** **Z-**	0.49 NA 1.94	1.06 0.12 0.13	4.21 1.40 1.25	0.32 0.42 NA
4	**X**	0.01	0.27	0.37	0.02
5	**X**	0.11	1.15	1.51	0.07
6	**Z+** **Z-**	0.31 0.25	0.32 NA	3.42 0.42	0.35 0.04

Table 3: Comparison of test results with FE model prediction

Test no.	Direction	Σ Wall 1			Σ Wall 2			Σ Wall 3			Σ Wall 4		
		FS kN	**Model kN**	**ER. %**	**FS kN**	**Model kN**	**ER. %**	**FS kN**	**Model kN**	**ER. %**	**FS kN**	**Model kN**	**ER. %**
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
1	Y	2.60	2.92	**12.3**	0.95	1.10	**16.2**	1.11	1.23	**10.4**	0.40	0.48	**20.6**
	Z+	1.07	1.21	**13.3**	0.96	0.97	**1.1**	0.00	0.14	**NA**	0.33	0.43	**32.3**
	Z-	1.53	1.56	**1.7**	NA	0.04	**NA**	1.50	1.25	**16.7**	0.10	0.04	**NA**
2	X	0.90	0.62	**31.2**	0.17	0.16	**7.0**	0.23	0.14	**38.6**	0.57	0.58	**1.0**
	Z+	0.49	0.42	**13.4**	0.22	0.16	**27.3**	0.40	0.50	**25.0**	0.39	NA	**NA**
	Z-	0.41	0.40	**2.2**	0.29	0.23	**21.5**	0.55	0.70	**27.3**	0.34	0.40	**17.3**
3	Y	0.49	0.50	**1.6**	1.06	0.78	**26.5**	4.21	3.89	**7.5**	0.32	0.30	**6.3**
	Z+	NA	NA	**NA**	0.12	NA	**NA**	1.40	1.47	**4.7**	0.42	0.44	**4.8**
	Z-	1.94	1.05	**45.9**	0.13	0.17	**30.8**	1.25	0.95	**23.8**	NA	0.08	**NA**
4	X	0.01	0.03	**NA**	0.27	0.25	**8.4**	0.37	0.31	**15.8**	0.00	0.01	**NA**
5	X	0.10	0.09	**10.0**	1.15	1.11	**3.3**	1.51	1.51	**0.0**	0.07	0.06	**14.3**
6	Z+	0.31	0.23	**25.1**	0.32	0.27	**15.6**	3.42	3.31	**3.2**	0.35	0.26	**25.7**
	Z-	0.25	0.23	**7.3**	NA	NA	**NA**	0.42	0.30	**28.6**	0.04	0.05	**25.0**

NA: Not applicable or insignificant
FS: Full-scale
ER: Error, absolute values
Z+: Sum of positive values in the Z-direction
Z-: Sum of negative values in the Z-direction

CHAPTER 7

Summary and Conclusions

7.1 Summary of literature review

Various aspects of wood building performance have been investigated experimentally during the past fifty years. Of these studies, most of the work has been done on one– and two-story structures on rigid foundations. These reports gave new insight into the mechanism of structural deformation and suggested important load sharing within the structure. All of these studies dealt with applied load on the structure within controlled laboratory conditions. Many of the studies dealt with the behaviour of the system under various stages of construction, and these studies showed that both the ultimate load and stiffness were much increased by the addition of any architectural components.

Many studies went well above the design load in their destructive tests, thus showing the conservatism of the conventional design of wood light-frame constructions. Some studies showed that tested buildings were over-estimated in some aspects and under-estimated in others, which emphasizes the importance of knowing the load paths in a building to predict its behaviour. The majority of the studies reviewed in the survey recognize the load sharing and composite action between the components in the structural system. However, few of these studies actually measured forces within the members and none of them quantified the load paths in the structure, especially at the interface between the roof and the wall. These measurements were accomplished in this research program.

7.2 Structural response

The load paths observed in the physical tests and during monitoring, 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.

Seven full-scale shear wall tests were carried out with the main objective of calibrating the 2-D finite element models. The parameters that were varied included: the presence and size of openings and anchoring details. Physical observations indicated that the drop in the initial stiffness is reduced in the presence of a tie-down at the base. It is clear that neither the strength nor the stiffness are being reduced in the same proportion as the wall effective length. This leads to the conclusion that the simple concept of the effective length to describe shear wall behaviour is invalid.

The results from the controlled static loads on the UNB house indicated significant load sharing. In some load cases, only about half of the applied load was taken by a wall directly underneath an applied load, with the remainder of the load being distributed to the other walls. Mostly, this reflected not only the stiffness of the roof diaphragm action, but also the stiffness of transverse walls.

The stiffness of the roof was sufficient to distribute the load to the walls farthest away from the load application point. Also, the expected vertical load paths were not observed. It was found that the internal forces are concentrated near the corners of the building, which display high stiffness.

There was a clear distinction in the roof stiffness between the longitudinal and transverse directions of the house, with the roof being stiffer in the direction parallel to the roof trusses than in the transverse direction. This confirms that a design assumption of "stiff roof on flexible shear walls" is reasonable for structures similar to the test building.

Under vertical loading on the roof, the load at the roof-to-wall interface was concentrated in a small region of the building plan in the vicinity of the load application point. This is not the case at the superstructure-to-foundation interface, where the load is dispersed away from the line of load application. Even at the foundation level, where large load dispersion effects were observed, the load paths did not follow the common tributary area assumption. Unexpectedly, the test results showed that load was transferred to the transverse walls, even though there is only a nominal connection between the wall and the roof trusses. The load must then have been transferred directly from the longitudinal wall to the transverse wall, with the stiffer studs attracting more load.

7.3 Finite element modeling

Observations and measurements made on full-scale subcomponents and structures such as the shear walls, the Forintek shed, and the UNB house, are necessarily limited and often difficult to generalize. This is why the finite element modeling is an essential companion approach to the field monitoring. The first step was to validate the finite element models created.

Generally, the shear wall model predictions and the full-scale results show good agreement especially for the ultimate load. The prediction of the initial horizontal stiffness was less accurate. The latter is expected since the initial stiffness of systems is notoriously difficult to measure experimentally or to predict, with the nonlinearities always present at early stages of loading.

In addition to predicting the ultimate capacity and the initial stiffness of the shear walls, it is also important to predict their deflection response for checking design serviceability criteria. The model predictions are able to replicate features such as the rotation of the sheathing panels, the bending of the top beam and the uplift of studs, as observed during the tests.

The 3-D model predictions of the Forintek shed under controlled static load agree in general with the experimental results. It was shown that more joists, than initially presumed, are participating in the load sharing and that ribbed plate models clearly are capable of very accurate predictions of displacements, provided that connections at interfaces are properly modeled.

The 3-D finite element model of the UNB house was built to predict the point force measurements obtained from load cells. The model was able to predict the appropriate wall and roof stiffnesses and therefore able to distribute the load realistically in the structure. The 3-D model was able to predict the very important interaction between the roof system and the walls and the interactions amongst the walls.

The 3-D finite element model of the CSIRO house simulated the sharing of racking forces between the shear walls, based on the experimental results reported in the literature. Considerations were restricted to the initial stiffness (linear elastic response).

7.3 Wind effects

7.3.1 The Forintek shed

The structural response monitoring was based on measurements of deformations within a representative segment of the wall and roof surfaces and a matching grid of the wall and roof wind pressure taps. The pressure distribution measured on the envelope of the building in full scale test was supplemented with data from a wind tunnel study.

The wind velocities at the reference height and at roof height were taken from velocity profiles evaluated for the westerly and easterly winds. Both profiles provided a power-law exponent of 0.34.

In general, it was shown that the building's surroundings have a great effect on the pressure distribution at the surface of the structure and that these effects cannot always be determined intuitively and can be difficult to predict using code considerations alone.

Peak pressure coefficients were also determined from the full-scale tests and compared with those measured in the wind tunnel. Generally, the ratio between peak and mean values was found to be 2.5 to 4 with a mean of about 3. This ratio is higher for higher C_p values. Furthermore, the peak pressure coefficients appear to have the same trend in their variation with the direction as with the mean pressure coefficients. The peak pressures measured on the envelope of the building compare well with corresponding values obtained in the wind tunnel. In general, the peak pressure coefficients from the full-scale tests were higher than those obtained from the wind-tunnel test.

The roof tapings showed some differences in the C_p values measured in the field and the wind tunnel for wind directions 90, 100 and 110 degrees. These deviations occur when the shed is in the wake of the larger main Forintek building. However, for wind directions of 260 and 280 degrees, the field and wind tunnel values correlated very well.

7.3.2 The UNB test house

The velocities at reference height from the airport and from the two anemometers on the tower near the house were taken from velocity profiles obtained for the oncoming wind incidences.

The results for the wind direction match well with the airport data. Small differences in the two data sets are normally expected due to local conditions. Where direct correlation between the wind direction at the structure and at the airport is possible, it must be recognized that the wind speed is affected by the wind boundary layer thickness and the local building terrain in the wind path. The comparison of wind speeds between the anemometer near the test house and the airport indicates that the terrain roughness exponent of 0.22 seems reasonable for the site.

7.4 Future work

The findings in the current research have indicated the need for more research work to be conducted. A few suggestions follow.

The present experimental work is based on simple shape structures without large openings and interior divisions, whereas the presence of architectural elements is more realistic of typical housing structures. Adding more openings as well as interior finishing and other architectural components to the UNB house would permit quantification of the effects of such non-structural components on the load paths.

This study has yielded important information about the typical rectangular wood light-frame bungalows. Once the structural response to controlled static tests and wind and snow loads is established, structural changes can be made to the structure based on finite element model predictions to improve the structural behaviour. As planned in the early stages of this project (but not retained due to limitations of funding), a twin house could be built equipped with these modifications and a comparison between the two structures could be studied. This could yield important findings for retrofitting of buildings and it would also validate some modeling aspects.

The rate with which the data was collected was limited to 1 Hz. It is recommended that higher frequency data acquisition is utilized to link peak wind values to structural dynamic response.

The load level at which the tests and the analysis have been conducted is very low, realistic of normal serviceability conditions rather than limit states design conditions. It is important to start by establishing the load path at the service load level but it would also be interesting in the next phase to study the effects of higher load levels. Eventually, the failure of components would alter the overall load paths as forces would be redistributed to unfailed members.

The 3-D finite element models used in this research yielded good results when compared with the controlled static load tests and the average of wind loads. The model has the option to input a time-history of the wind and it produces a time history of the response. The detailed finite element model can then provide a picture of the load path of the signal from the very outer layer in the structure and all the way down to the foundation. The damping effect of the various structural elements on the external load can also be established.

APPENDIX A

A Survey of Full-Scale Tests on Wood Structures

Investigators	Location	Type of Structure	Main findings
Dorey and Schriever (1957)	Canada	Single-story house	The corner braces, combined with the various finishing materials, provided sufficient racking strength. The joist-rafter roof system withstood a load at the onset of failure that was 43 percent above the design load.
Hurst (1965)	USA	Single-story house	The minor racking distortion of the end walls and the ballooning of the loaded walls were similar to those found by Dorey and Schriever (1957).
Yokel et al. (1973)	USA	Two-story building	The floor-ceiling diaphragm between the first and the second stories tended to translate as a rigid body, while the upper ceiling diaphragm experienced significant in-plane distortion.
Yancey and Somes (1973)	USA	Factory-built modules	The unit was considerably more flexible than the conventional house tested by Yokel et al. (1973).
Tuomi and McCutcheon (1974)	USA	Single-story light-frame structure	The structure was over-designed in some respects but under-designed in others. Addition of gypsum wallboard and siding nearly doubled the lateral strength.
McCutcheon et al. (1979)	USA	Single-story house	The inter-component connections were identified as weak links. Interaction between the joist and

			sheathing material improved the floor performance over that of the joist and the sheathing acting separately.
Hirashima (1981)	Japan	Single-story house	The allowable shear load and the maximum load for the frames with bracing were greater than those for the frame without reinforcement.
Boughton and Reardon (1982)	Australia	Existing 40-year-old house	The measured failure loads of various elements in the structure were higher than those predicted in most cases. Weatherboards assisted in transferring loads from the tested stud to the adjacent studs. Substantial loads were carried through bracing action. Very little load was carried in actual bending of the wall.
Reardon and Boughton (1985)	Australia	Togan Hurricane house, single story	The internal wall, which was not designed as a bracing wall, had the capacity to act as one. Although the end walls had identical stiffnesses, they attracted different percentages of the applied load.
Sugiyama et al. (1988)	Japan	Two-story wood-frame house	The application of wall sheathing and /or wall siding to the wall spaces above and below the window and door openings provided some increase in the racking resistance.
Sugiyama et al. (1988)	Japan	Shear wall units and two-story house	The racking resistance measured in the whole house test was about one and a half time the one estimated by using the unit resistance of shear walls obtained from the racking tests, regardless of the type of shear wall.
Wolfe et al. (1988)	USA	Light frame roof assemblies	Load sharing and composite action mechanisms were shown to vary with framing member stiffness variability, and the predominant

			failure mechanism varied as the truss configuration was varied.
Boughton (1988)	Australia	Single-story house	The load did not follow paths through the major structural components. Non-structural elements played a role in distributing the loads.
Stewart et al. (1988)	USA	Two manufactured houses	The interior walls did not exhibit as much racking deformation as the end walls. The building systems were able to sustain the loads to three times the load that expected from a design wind, with minor damage.
Ohashi and Sakamoto (1988)	Japan	Two-story structure	The structure behaved as a nonlinear system with degrading stiffness. The test indicated the strong influence of connections on the overall behaviour of the structure.
Yasumura et al. (1988)	Japan	Three-story wooden building	The shear deformation of the diaphragm proved to be small enough to assume that the diaphragm was rigid.
Reardon (1989)	Australia	Two full-scale timber brick veneer houses	Design wind pressure was resisted with only a minimum of load transfer to the timber frame. No racking load transfer was observed between the veneer and the stud wall. The internal walls acted as bracing walls even though they were not designed to do so.
Phillips (1990)	USA	Light-frame building	Stiffness did not appear to be the determining factor with respect to wall shear load. The interior walls' contribution to system performance was not proportional to their relative racking stiffness.
Hansen and Mortensen (1991)	Denmark	Prefabricated timber frames	The strength and stiffness of a frame are strongly dependent on the strength and stiffness of the knee

			joint.
Moore et al. (1993)	UK	Five three-story steel frames	Significant interactions were observed with the restraining effects being transmitted and moments transferred from beams to columns and *vice versa* via the connections.
Xiaoming (1993)	Japan	Floor system and a single joist	The combination of sheathing and joists improved the effective stiffness of the joists by more than 100 percent. The system tended to reduce the variability of the load carried by the loaded joist by transmitting the loads to adjacent joists.
Ceccotti (1994)	Italy	Glued-laminated timber portal frame	Seismic tests confirmed the suitability of the theoretical model proposed by the authors for predicting the earthquake behaviour of multi-degree-of-freedom timber frames with moment rotation semi-rigid joints at the corners.
Reardon and Henderson (1996)	Australia	Split-level two-story house	The single-story section was able to resist two times the design pressure for 28 m/s winds without structural failure. For the two-story section, the frame increased in lateral stiffness as the lining elements were added and finally became a very stiff box, with only small displacements occurring at design wind loads.
Hirashima and Suzuki (1996)	Japan	Two-story wooden dwelling house	The ultimate loads obtained from the test were more than 2.8 times the design load in the ridge direction and more than 2.5 times the design loads in the span direction. The reactions of the construction were around 1.7 times the design values.
Suzuki et al. (1996)	Japan	3 different construction types of two-story houses	Impact-hammer excitations and electric excitation are available to establish the frequency response function for full-scale wooden

			houses. Experimental modal analysis is available to assess their dynamic properties.
Wood and Bullen (1996)	Australia	Timber pole house	Reduction by a factor of two in the deflection was obtained after the bracing walls were constructed. The pole house increased its lateral stiffness as more elements were added.
Gad (1997)	Australia	One-room brick-veneer house	The boundary conditions imposed on the wall by the surrounding structure have a significant effect on the lateral load resisting capacity.
Ceccotti and Karacabeyli (1997)	Canada	Four-story wood frame platform structure	The result of this study confirmed the current Canadian seismic force modification factor and the European factor. The presence of gypsum wallboard sheathing has a stiffening influence on the response of the structure.
Ohashi et al. (1998)	Japan	Single-story Japanese-style house	The load-deflection curves showed that the structure behaved as a non-linear system with degrading stiffness. The test indicated the strong influence of the connections on the overall behaviour of the structure.
Hirashima and Suzuki (1998)	Japan	Two-story wood construction	The final load applied to the construction was equivalent to three times the design load. The load sharing of the structural elements increased as the deformation increased.
TF 2000	UK	Typical six-story multi-occupancy residential building	The load paths did not follow those assumed in the design theory. Significant contribution of the plasterboard lining and brick cladding were noted. The sheathing was showed to contribute to the

			resistance to vertical loads. The diaphragm action of floors adds to the stability of the building, which is a factor that is usually ignored in design.
Fischer et al. (2001)	USA	Two-story timber-frame house	A fully engineered timber-frame house has a better seismic performance than a conventionally constructed house.
Paevere et al. (2002)	Australia	Single-story structure	The study showed the potential for significant sharing and redistribution of applied lateral loads as well as the importance of the roof and ceiling diaphragms under elastic and inelastic response conditions. The study provided the most detailed picture of the reaction forces under the lateral loading.
Doudak et al. (2005)	Canada	Light-frame industrial building	Measured response under snow loads confirmed theoretical expectations that composite action and load sharing are important mechanisms for light-frame buildings. This project proved the feasibility of real-time monitoring and was the precursor for a larger monitoring project currently in progress.
UWO (2008)	Canada	Testing facility	The University of Western Ontario is planning to develop a new test facility that will accommodate a full-size two-story house. The test house itself will be designed to capture the external and internal pressures generated as well as the overall house deflections and local deformations of its components.
University of Manitoba (on going)	Canada	A 'post-frame' building	A 'post-frame' building, open interior and a duo-pitch roof with laterally braced trusses. Instrumentation of the facility is

			related to both structural behaviour and building envelope performance. Ten posts are being monitored for axial force with load cells at each end. Strain gauges will be attached to components at various locations. Construction of the building started in October 2003, and the remainder of the instrumentation was installed during Spring 2004.