DEVELOPMENT OF A DESIGN METHODOLOGY FOR STEEL FRAME / WOOD PANEL SHEAR WALLS

by

Aaron E. Branston

McGill

Department of Civil Engineering and Applied Mechanics

McGill University, Montréal, Québec, Canada

June, 2004

A thesis submitted to the Faculty of Graduate and Postdoctoral Studies in partial fulfillment of the requirements of the degree of Master of Engineering

Your file Votre référence
ISBN: 0-494-06548-6
Our file Notre référence
ISBN: 0-494-06548-6

ABSTRACT

It is anticipated that the construction of homes and multiple storey buildings which incorporate light gauge steel frame / wood panel shear walls as primary lateral load resisting elements will increase across Canada in coming years. This includes sites that have a relatively high seismic risk, such as found along the West Coast of British Columbia and in the Ottawa and St. Lawrence River Valleys. With this rise in construction activity comes an accompanying increase in the probability that a light gauge steel frame structure will be subjected to the demands of a severe earthquake. Currently, guidelines for engineers with which the design of laterally loaded light gauge steel frame / wood panel shear walls can be carried out are not available in Canada. For this reason an extensive shear wall research program has been undertaken at McGill University.

This thesis provides details on the 109 specimen main testing program as well as a summary of past wood frame and steel frame shear wall research. An extensive review of existing data interpretation methodologies is presented. The equivalent energy elastic-plastic (EEEP) technique is chosen as most suitable for the wall systems under study to deduce key design parameters including the yield wall resistance, elastic stiffness, and system ductility. It is recommended that the EEEP methodology be implemented for all future steel frame / wood panel shear wall data interpretation. The calibration of a resistance factor for use with the limit states design philosophy consistent with the upcoming draft version of the 2005 National Building Code of Canada (NBCC) is also presented.

It was found that a resistance factor (Φ) of 0.7 provided sufficient reliability and a reasonable factor of safety under the NBCC wind loading case. Final nominal strength and unit elastic stiffness values for use in design are presented in tabular format according to given perimeter fastener schedules. Finally, recommendations for future research and testing are outlined.

RÉSUMÉ

Il est attendu que la construction d'habitations et de bâtiments à plusieurs étages incorporant des murs de refends à montants en acier forgé à froid / panneau de revêtement en bois comme élément principal fournissant la résistance latérale augmentera au Canada dans les prochaines années. Ceci inclut des sites qui ont un risque sismique relativement élevé, particulièrement sur la côte Ouest du Canada, dans la région de la rivière des Outaouais et le long de la vallée du St-Laurent. Avec l'accroissement de la popularité de ce type de construction vient une augmentation de la probabilité qu'un bâtiment construit en acier forgé à froid soit sujet aux accélérations d'un séisme majeur. Actuellement, des directives spécifiques de calculs pour la capacité prévue des murs de refends à montants en acier forgé à froid / panneau de revêtement en bois sujets aux charges latérales ne sont pas disponibles au Canada. Par conséquent, un important programme de recherche était entreprit à l'Université McGill.

Ce mémoire fournit des détails sur les 109 essais qui ont fait partie du programme de recherche principal et présente aussi un sommaire des essais effectués sur les murs de refends à montants en acier et en bois de construction jusqu'à ce jour. La méthode de l'énergie équivalent élastique-plastique (EEEP) est utilisée pour déterminer les paramètres clés de conception, incluant la limite élastique des murs de refends, leur rigidité et la ductilité du système. Il est suggéré que la technique EEEP soit employée pour tout autres programmes d'interprétation des données des murs de refends à montants en acier forgé à froid / panneau de revêtement en bois. L'évaluation d'un coefficient de résistance qui pourrait être utilisé avec la philosophie de conception aux états limites promulguée dans la prochaine édition (2005) du code national du bâtiment du Canada (CNBC) est aussi présentée.

Il a été démontré qu'un coefficient de résistance (Φ) de 0.7 assure un facteur de sécurité acceptable lors des charges de vent. La capacité nominale et la rigidité des murs sont données sous forme de tableau en fonction de l'espacement des attaches. Finalement, des recommandations pour les études et essais futurs sont présentées.

ACKNOWLEDGEMENT

The completion of my graduate studies at McGill University could not have been possible without the guidance and support of Professor Colin A. Rogers. A special thank you goes out for always being available, for offering advice, and for teaching me what there is to know about research. Your countless efforts are much appreciated.

Thanks are also extended to numerous researchers who have provided invaluable information and insight along the way: R.L. Serrette, Prof. G.C. Pardoen, E. Karacabeyli, R.O. Foschi, E. Jones, H. Prion, T. Skaggs, Z. Martin, N. Nagy, P. Jaehrlich, W.D. Cook, and D.P. Janssens. I would also like to acknowledge the support provided by numerous organizations, including the Natural Sciences and Engineering Research Council of Canada, the Canada Foundation for Innovation, the Canadian Sheet Steel Building Institute, Simpson Strong-Tie Co. Inc., the Structural Board Association, Grant Forest Products Inc., MTS, and the Canam Manac Group. Assistance with the procurement of steel framing was provided by Mr. John Rice of Bailey Metal Products Ltd.

I would like to express my sincere gratitude to our shear wall testing team during the summer of 2003: Katherine Hikita, Damien Soulier, Regis Lambert, and Jason Hui. The assistance of laboratory technicians M. Przykorski, J. Bartczak, R. Sheppard and D. Kiperchuk and the administrative staff of the Department of Civil Engineering is also greatly appreciated.

To my partners on this research project, Felix-Antoine Boudreault and Chang Yi Chen, thank you for all your help, dedication and encouragement. To all my friends, thanks for making my studies at McGill all the more enjoyable.

To my father, mother, sisters, and Lindsay, I thank you for never leaving my side. You have paved the way for my future and I could never thank you all enough.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

CHAPTER 1 INTRODUCTION

1.1 GENERAL OVERVIEW

Since the late 1940s, the majority of residential or low-rise construction in North America has consisted of wood frame, platform-type construction in which stud walls are assembled on a working platform and lifted manually into place *(Kesik and Lio, 1997)*. The subsequent floor area can be built using these walls as supports, and, in turn, provides another working platform at the second storey of the structure. Platform frame construction has proven to be both economical and efficient since the structural system acts as a unit in load sharing between the elements. Light gauge steel framing is becoming ever-increasingly more popular as an alternative framing method in residential and industrial / commercial type construction. When properly constructed, these light gauge steel framed walls sheathed with wood panelling may be relied on to act as shear walls. As shown in Figure 1.1, light gauge steel stud structures can be built using the same platform framing technique.

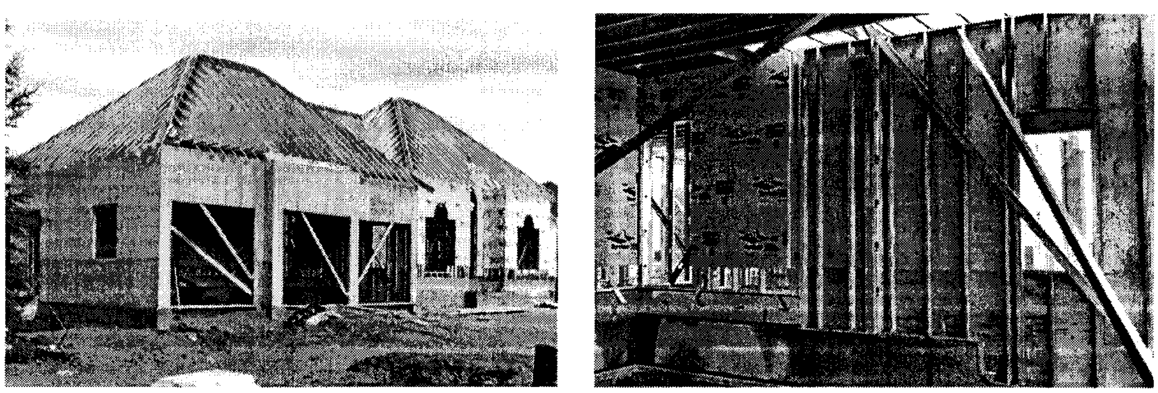

Figure 1.1: Light gauge steel stud wall using platform framing technique (left: exterior view; right: interior view of side wall)

Why has there been an increase in the use of light gauge steel stud framing in recent years? In many ways, light gauge steel is advantageous as a framing material. Light gauge steel products are very light thereby allowing members to have high strength-to-weight ratios. Manufacturers are able to fabricate members to accurate dimensions, of uniform quality, and are also able to easily produce special orders for unusual configurations and sizes. Light gauge steel is also not affected by humidity variation; that is dimensional changes, warping, splitting, or checking, and it is resistant to termites and rotting. Furthermore, on average, steel manufactured to meet today's demand contains a minimum of 70 % recycled material *(AISI, 2004)*. An average 2 000 sq. ft. (186 m^2) home framed with solid lumber can require approximately 40 – 50 trees: about one acre of forest. The same home constructed of light gauge steel framing would only require six recycled automobiles *(Steel Recycling Institute, 2003)*. This practice, to a great extent, takes pressure off of our renewable resources.

The construction industry has also witnessed an escalation in lumber prices over the past decade due to an increasingly competitive market and more selective cutting of trees to preserve the environment and endangered species. It is estimated that approximately 90 % of North America's old growth forests have been harvested for solid lumber. This price escalation, which has caused an extensive search for alternative framing materials, is demonstrated in data provided by the National Association of Home Builders (NAHB) *(Waite, 2000)* and Madison's Canadian Lumber Reporter *(Madison's Report, 2004)* in Table 1.1.

Table 1.1: Average lumber cost for a typical home (USD)

	Average lumber cost for a 2 000 sq. ft. (186 m^2) home
1990	$4 882
2004	$9 764

The increase in total lumber costs for a home is directly related to the increase in the price of lumber framing, which is estimated to consume approximately 20 % of the total cost of a home. The NAHB provides average lumber costs for a given home based on the cost of lumber per 1 000 board feet, while Madison's Canadian Lumber Reporter provides the current key prices of lumber. The doubling of the average lumber cost for a home from 1990 to 2004 is based upon an increase of the price of lumber from $200 (USD) to approximately $400 (USD) per 1 000 board feet.

While light gauge steel framing does have its advantages and the material costs are now competitive with lumber costs, this alternative material also has its drawbacks. Because light gauge steel studs and track are usually fabricated from a thin (typically 0.90 – 2.0 mm) sheet steel coil by continuous roll forming, these members can easily be damaged during transportation and handling. Also, because the steel base material is usually quite thin, there is very little reserve strength if corrosion does occur over the thickness. To prevent the onset of corrosion, light gauge steel must be coated with a sacrificial element such as zinc or a combination of zinc and aluminum. It is also of great importance that designers and constructors take into consideration building science issues so as to minimize the possibility of moisture build-up in the wall, floor, and roof cavities. Proper insulation must also be installed, as steel does not perform well as an insulator of heat. Even though light gauge steel is not itself combustible, it must still be protected from fire to prevent it from losing stiffness and load carrying capacity at very high temperatures. This is usually accomplished by sheathing a light gauge steel frame wall with gypsum wallboard. A number of complexities also exist in light gauge steel design which do not exist in the design of the more stocky hot-rolled steel and wood members. Because thin plate elements in compression tend to buckle locally while remaining elastic, designers must employ an effective width approach, which considers an average stress acting on only a reduced area of the cross-section. These thin members are also able to redistribute stresses to stiffer portions of the cross-section, *i.e.* corners of the section, internal stiffeners, *etc.*, and therefore post-buckling capacity must also be taken into account in design *(AISI, 2002a)*.

Other barriers which exist are not dependant upon the material itself, but are related to labour, engineering, and consumer awareness. Because this type of structural system is still in development, not all designers and contractors know how to work with light gauge steel. This makes it difficult to find competitive skilled framers, inspectors, and designers at reasonable prices. Finally, because the consumer tends to first consider cost and tradition, the conventional home builder will often choose lumber framing over a new, alternative means. In summary, the use of steel framing presents both advantages and disadvantages, nonetheless it can be considered as a viable alternative for the framing market, with an even greater potential for use once more detailed design documents have been developed and greater awareness has been achieved.

The use of plywood and oriented strand board (OSB) sheathing on shear walls is economical in that, unlike solid lumber, an engineered wood product can be produced from second growth trees which are much smaller in diameter. These younger trees are easier to replace and Canada has seen a 400 % increase in the number of trees re-planted between 1975 and 1990. In the United States, two billion trees are planted every year, the forest products industry being responsible for approximately one billion of these re-plantings. This represents a re-planting rate of approximately three million trees per day *(APA PS2, 1992)*. In addition to preserving the old growth forests, the emergence of engineered wood products has also led to less waste during manufacture and a final product that is nearly free of strength reducing defects and has a more uniform quality and strength than solid lumber. Certain manufacturing techniques can also be used such as aligning strands or veneers and cross-banding, giving plywood and OSB similar strength characteristics with an increased capacity along the major axis of the panel.

1.2 Shear Walls as Lateral Load Resisting Elements

Although light-framed load bearing walls do often support gravity loads, they can also be designed as engineered shear walls, sheathed with wood panelling, which withstand and transmit lateral forces from the upper storeys of a structure to the foundation. Lateral loads are typically caused by wind or seismic events. Lateral earth pressure, an out-of-straight wall, and secondary effects such as P-Δ forces can also lead to shear forces developing in a wall system. Even though wind and seismic events apply loads to a structure in very different manners, for most low-rise buildings with relatively short fundamental periods of vibration the National Building Code of Canada (NBCC) *(NRCC, 1995, 2004)* allows designers to use an equivalent static force approach.

In the case of a seismic event, since ground shaking causes the building mass to accelerate, inertial forces are produced in the floors and roof of the structure and an equivalent base static shear force can be estimated given the overall mass and configuration of the structure. The total lateral seismic base shear is then distributed to each individual floor / roof level according to the seismic mass of that individual level, as well as its elevation from the base of the structure. Wind loads, derived from a reference velocity pressure available in the NBCC *(NRCC, 1995, 2004)*, are applied directly to the faces of the structure as uniform loads and are transferred to the floors or roof diaphragms via wind bearing elements.

Horizontal roof or floor diaphragms are designed to have adequate capacity in their own plane to distribute lateral loads to the shear wall elements. Shear walls are also commonly referred to as "vertical diaphragms" in the literature *(Diekmann, 1997)*. As shown in Figure 1.2, the shear wall acts a deep cantilever beam where shear forces are transmitted from the framing through the connectors to the wood panel sheathing acting as the web. To accomplish the transfer of forces from the framing members to the panel sheathing, the connectors must be spaced sufficiently close to provide the necessary stiffness and strength. The wood panel sheathing resists the shear force and the two boundaries of the wall, or chords, resist the overturning moment. These members must be designed to withstand the large tensile and compressive forces that develop in the chords of the wall and any additional gravity loads which may be present. For this reason, back-to-back light gauge steel chord studs are typically incorporated for added strength. Shear anchors transmit the shear force in the wall to the foundation while hold-down anchors resist and transfer the uplift force from the tension chord to the storey below or to the foundation.

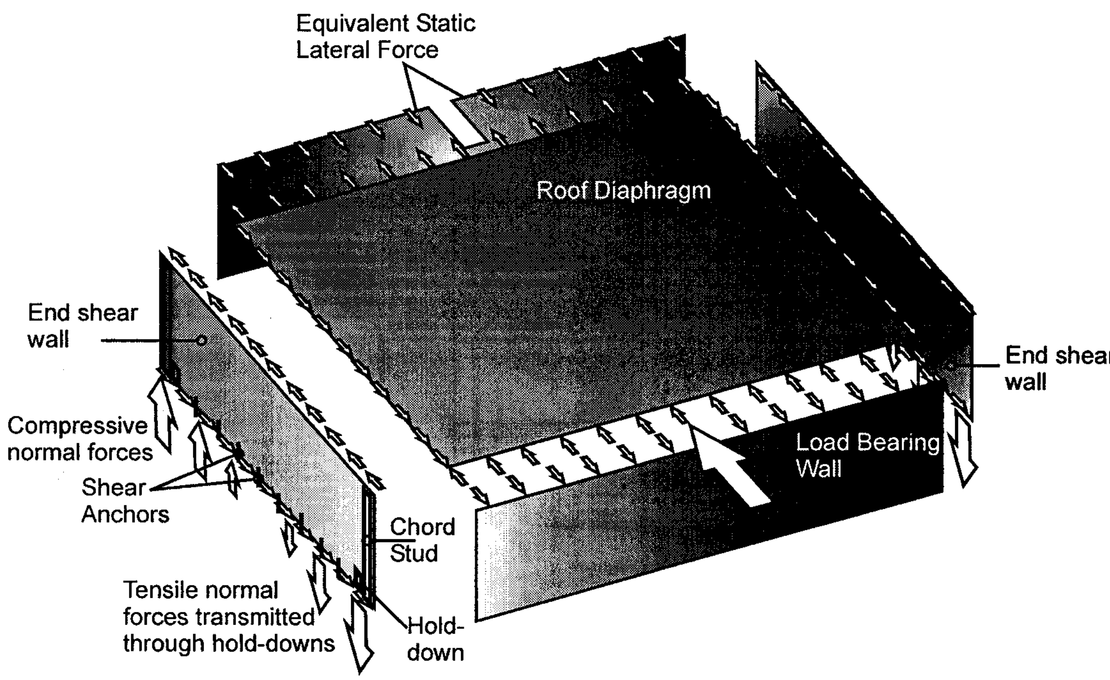

Figure 1.2: Transfer of lateral loads through roof diaphragm to end shear walls *(CWC, 2001, 2002)*

1.3 Statement of Problem and Need

For light gauge steel frame shear walls sheathed with wood panelling there are currently no prescribed engineering design guidelines outlined in Canadian literature or standards. Because of the highly non-linear load-deflection behaviour of light frame shear walls sheathed with wood panelling subject to racking loads, and the many factors which could potentially affect their performance, testing programs must be carried out in order to determine design parameters. Factors which could potentially affect both steel frame and wood frame shear wall performance may include, amongst others, the type, orientation and blocking of sheathing, the fastener schedule and types of fasteners, the spacing and dimensions of the framing studs, the aspect ratio of the wall, the presence of hold-downs and the loading condition or loading protocol applied to the wall. In addition, because of complexities such as compression failure in the chord studs and the thinness of steel framing members, as demonstrated above, steel frame shear walls share some, but not all of their overall performance characteristics with wood frame walls.

Due to the considerable seismic risk in certain regions of Canada, it is important, in addition to investigating the monotonic behaviour of light-framed shear walls, to investigate their behaviour under reversed cyclic loads. Earthquakes are usually caused by a release of elastic energy built up at the boundary of continental plates. This sudden release of energy causes waves to propagate through the earth's crust, which results in ground shaking. Along the West Coast of Canada, a subduction zone exists in which the Pacific plate meets the North American continental plate. Even though there is greater seismic hazard along the West Coast of Canada, intra-plate seismic hazard also exists in the Ottawa and the St. Lawrence River Valleys, where the weak crust is subject to compressive stresses from differential movement of surrounding plates *(Rainer and Karacabeyli, 2000a)*. Areas of the Maritimes as well as in the Yukon and the high Arctic also have been identified as having a substantial seismic hazard.

Since the major earthquakes that have occurred in Canada during the 20th century (Cornwall, ON – 1944, Courtney, BC – 1946, Miramichi, NB – 1982, Nahanni, NT – 1985, Saguenay, QC – 1988) *(Rainer and Karacabeyli, 2000a, 2000b)* haven't produced significant structural damage and loss of life in wood-framed houses, it can be concluded that, according to the standards set out by the NBCC *(NRCC 1995, 2004)*, past performance has been satisfactory. In the United States, on the other hand, considerable damage to wood frame shear walls was identified due to the Northridge (CA – January 17, 1994) earthquake. Property damage to wood frame construction was estimated at US $40 billion, and in the aftermath of the earthquake, 48 000 housing units were reduced to uninhabitable status resulting in 24 fatalities *(Krawinkler et al., 2000)*. Noting that some performance characteristics are common to both wood frame and steel frame shear walls, these shortcomings could be expected to affect light gauge steel frame walls constructed in Canada subject to an earthquake of similar magnitude, the likes of which could occur along the West Coast of BC. With a lateral force resisting system such as the light gauge steel frame shear wall with wood panelling, studies must be conducted for this specific type of construction to determine design parameters which could be incorporated into a Canadian Standard for use by designers.

1.4 OBJECTIVES

The objectives of this thesis include: i) To review existing shear wall test programs as well as to examine current prescriptive design approaches, including the design approach for wood-framed shear walls according to CSA O86 *(2001; CWC, 2001)* and the design approaches employed in the United States. ii) To explore numerous approaches for the interpretation of test results, and to choose a suitable method with which a yield capacity and several other key design parameters may be derived from test data. iii) To carry out a suite of shear wall tests and to determine the appropriate design information from the test results. iv) To propose a resistance factor for light gauge steel frame / wood panel shear walls, which can be applied to a nominal strength in order to obtain a factored resistance consistent with the limit states design philosophy in Canada. And v) to provide recommendations for future study in light gauge steel frame / wood panel shear walls to expand on the coverage of this body of research.

1.4.1 Scope of Study

A large shear wall testing program was conducted during the summer of 2003 in order to assemble a bank of data for various wall configurations constructed with Canadian steel products and sheathing products (Douglas Fir Plywood *(CSA O121,* *1978)*, Canadian Softwood Plywood *(CSA O151, 1978)*, Performance Rated OSB *(CSA O325, 1992)*). In total, 109 tests were completed, 43 of which the author was solely responsible for. All test data, results and observation sheets were assembled in a stand-alone document *(Branston et al., 2004)*. Fourteen existing data interpretation techniques were analyzed and a chosen method was used to interpret all data resulting from the testing program. The resulting test data was then incorporated into this study to accomplish the main objectives.

1.4.2 Limitations of Study

The findings presented in this thesis are limited to single-storey light gauge steel frame / wood panel shear walls subject to lateral load only. This research does not attempt to address the issue of inter-storey connections for shear transfer in multi-storey shear walls and buildings nor does it address the issue of combined gravity and lateral loadings. These aspects were considered beyond the scope of this thesis. The design guidelines presented, including nominal shear strengths and stiffnesses, apply only to walls constructed in an identical manner to those described herein until further research is conducted.

1.5 THESIS OUTLINE

This thesis, which consists of four main parts, is primarily concerned with the derivation of a design procedure for light gauge steel frame / wood panel shear walls subject to wind and seismic loads in Canada. In Chapter 2, a review of existing shear wall test programs, both in and outside of North America, is provided. A discussion of the various product and performance standards both in Canada and the United States for structural sheathing materials is also presented in Chapter 2. Chapter 3 focuses on both the preliminary and main experimental programs conducted on light gauge steel frame / wood panel shear walls using a self-equilibrating test frame designed by Zhao *(2002)*. The contents of Chapter 4 include a review of: i) Various existing data interpretation methodologies. ii) The prescriptive design approach for wood frame shear walls according to CSA O86 *(2001; CWC, 2001)*. And iii) both the load and resistance factor design (LRFD) and allowable stress design (ASD) approaches currently employed in the United States for wood frame and steel frame shear walls. In addition, a focus is placed on the interpretation of the results obtained from the experimental program and the derivation of nominal strengths and stiffnesses for light gauge steel frame shear walls. Chapter 5 incorporates these derived strength values in a calibration of a resistance factor for shear walls consistent with the limit states design philosophy of the National Building Code of Canada *(NRCC 1995, 2004)* to obtain a satisfactory level of reliability and margin of safety when subject to wind and seismic loads. Finally, Chapter 6 provides conclusions and recommendations for further research in this domain.

CHAPTER 2 LITERATURE REVIEW

Light frame timber shear wall research and testing have been carried out since 1929, with Report R896 *(Trayer, 1929)* published by the Forest Products Laboratory. An extensive experimental program was then undertaken by the National Bureau of Standards (US) in the 1930s *(Tissell, 1989)*. The Douglas Fir Plywood Association (DFPA) began to sponsor tests at the National Bureau of Standards, and, after being re-named the American Plywood Association (APA – The Engineered Wood Association) in 1964, still produces many of the reports pertaining to the testing and behaviour of timber frame shear walls. The first separate table of design values for wood frame shear walls was included in the 1967 Uniform Building Code (UBC) *(PCBOC, 1967)* and has been reviewed, updated and expanded upon due to ongoing research. A conversion from allowable stress design (ASD) to limit states design (LSD) allowed the values listed in the U.S. model building codes to be included in the 1989 version of the Engineering Design in Wood Standard (CAN/CSA-O86.1-M89) in Canada *(CSA O86-M89, 1989)*. A review of much of the early research conducted on wood frame shear walls is detailed in bibliographies prepared by Carney *(1975)* and Peterson *(1983)*. Van de Lindt *(2004)* provides a literature review on the evolution of wood shear wall testing from 1982 to present including, among others (Table 2.1), testing programs conducted by Atherton *(1983)*, Patton-Mallory and Wolfe *(1985)*, Dolan and Madsen *(1992)*, Karacabeyli and Ceccotti *(1996, 1998)*, Durham *et al. (2001)*, *etc.* In addition, CUREE *(Filiatrault, 2001)* provides a detailed literature review of past wood frame shear wall research including studies in the specific areas of testing protocols, wide / narrow shear walls, perforated shear walls, shear wall analysis, shear wall design, testing on full-scale houses, nailed connections and framing (Table 2.1).

Table 2.1: Summary of existing wood frame shear wall test programs *(van de Lindt, 2004; Filiatrault, 2001)*

Study	Walls tested	Sheathing type(s) and connector	Loading protocol
Atherton *(1983)*	Ten 16' x 48'	7/16" and 5/8" particleboard; 8d and 10d nails	Cyclic
Nelson *et al.* *(1985)*	Four 8'; Three 10.5'	Glue	Monotonic
Patton-Mallory and Wolfe *(1985)*	Three 8' x 8'; Two 8' x 16'; Six 8' x 24'; 200 22" x 2,4,6,8'	1/2" gypsum with 1.25" drywall nails; 1/2" gypsum and 1/2" CDX plywood	Monotonic
Soltis and Patton-Mallory *(1986)*	200 small-scale specimens	One and two-sided wall sheathing	Cyclic
Falk and Itani *(1987)*	Four 8' x 24'	1/2" plywood with 6d com. Nails; 1/2" gypsum with 1/2" drywall nails	Sine waves at varying frequencies, free vibration
Cheung *et al.* *(1988)*	Seven 8' x 8'	1/2" Douglas fir; 8d smooth galvanized boxnails	Monotonic
Stewart *et al.* *(1988)*	Eleven 2.4 m x 2.4 m	7.5 mm and 12.0 mm sheathing	Quasi static, sinusoidal shaketable, El Centro 1940 shaketable
Zacher and Gray *(1989)*	13 2.44 m x 2.44 m 15 406 x 457 mm	9.5 mm plywood	Reversed cyclic
Polensek and Schimel *(1991)*	Five 2.44 m x 2.44 m	9.5 mm plywood; 12.7 mm gypsum	Static cyclic
Deam *et al.* *(1991)*	Five 9.0 m x 3.6 m	Plywood	Cyclic
Dolan and Madsen *(1992)*	Eleven 2.4 m x 2.4 m	9.0 mm waferboard; 9 mm plywood	Monotonic, slow cyclic
Tissell *(1993)*	8' x 8'	unblocked shear walls, stapled shear walls, double-sided walls, panels over gypsum wallboard	Monotonic
Schmid *et al.* *(1994)*	Three 8' x 4'	1/2" plywood; 10d vinyl coated nails	Reversed cyclic pseudostatic
Leiva-Arevena *(1996)*	2.4 m x 2.4 m	12.0 mm Radiata-pine; helically threaded 50 mm nails	Reversed cyclic: BRANZ P21 test procedure (NZ)
Karacabeyli and Ceccotti *(1996)*	Six 2.4 m x 4.9 m	9.5 mm OSB; 9.5 mm plywood; 65 mm nails	Ramp and cyclic
Kamiya *et al.* *(1996)*	25 2.42 x 1.82 m	7.5 mm, 9.0 mm, 12.0 mm plywood	Shaketable: El Centro 1940, Taft 1952
Enjily and Griffiths *(1996)*	14 walls of varying dimensions	Plywood, OSB, chipboard, tempered hardboard	Monotonic
Johnson and Dolan *(1996)*	Ten 2.4 x 12 m	13.0 mm plywood on one side; 13.0 mm gypsum on other side	Monotonic and reversed cyclic: SPD
Lam *et al.* *(1997)*	Eleven 2.4 m x 7.3 m	1.2 m x 2.4 m and 1.2 x 7.3 m panels	Monotonic and reversed cyclic

Table 2.1 cont'd

Study	Walls tested	Sheathing type(s) and connector	Loading protocol
Rose and Keith *(1997)*	Seven 8' x 8'; Four 2.44 x 3.66 m	Plywood and gypsum	Monotonic
Yamaguchi and Minowa *(1998)*	2.94 m x 3.64 m	9.0 mm plywood; N50 50 mm nails	Shaketable: Kobe 1995
He *et al. (1998)*	2.4 m x 7.2 m	OSB	Reversed cyclic, three different protocols
Shenton *et al. (1998)*	Eight 2.4 m x 2.4 m	15/32" plywood; 1/2" OSB; 8d nails	Reversed cyclic SPD
Dinehart and Shenton *(1998a)*	12 2.44 m x 2.44 m	11.9 mm plywood; 12.7 mm OSB; 8d nails	Static cyclic, dynamic reversed cyclic
Kawai *(1998)*	19 2.79 x 3.64 m	Plywood, braces, gypsum, plywood with sliding board	Pseudodynamic, cyclic
Shepherd and Allred *(1998)*	2.44 x 0.70 m	Plywood	Reversed cyclic
Dolan and Heine *(1998)*	22 2.4 x 12 m	12.0 mm plywood; 12.0 mm OSB	Monotonic, reversed cyclic: SPD
Ficcadenti *et al. (1998)*	24 2.4 x 2.4 m	9.5 mm plywood	Reversed cyclic: various
Dinehart and Shenton *(1998b)*	Four 2.4 x 2.4 m	11.9 mm plywood; 8d nails	Reversed cyclic: SPD
Karacabeyli and Ceccotti *(1998)*	2.44 x 4.88 m	-	Monotonic, reversed cyclic, pseudodynamic: various EQ's
Bracci and Jones *(1998)*	8' x 8'	15/32"	Reversed cyclic
Rose *(1998)*	Eight 8' x 8'	Various thicknesses of plywood and OSB; 8d com.; 10d com. nails	Reversed cyclic: SPD
Salenikovich and Dolan *(1999a)*	55 walls of various dimensions	OSB	Monotonic and reversed cyclic
He *et al. (1999)*	Eight 2.4 x 7.32 m	9.5 mm OSB; 50 mm 6d nails	Monotonic, reversed cyclic
Dinehart *et al. (1999)*	8' x 8'	15/32" plywood; 8d nails	Reversed cyclic: SPD
Shipp *et al. (2000)*	14 8' x 8'	15/32" and 3/8" plywood; 8d and 10d com. Nails	Reversed cyclic
Karacabeyli *et al. (2001)*	13 8' x 4' 20 8' x 8'	7/16" OSB; 82 mm nails	Shaketable: scaled Kobe 1995, Landers 1992
Higgins *(2001)*	2.4 x 2.4 m	11.9 mm plywood; 8d nails	Reversed cyclic
Durham *et al. (2001)*	12 2.4 x 2.4 m	9.5 mm OSB; 50 mm spiral nails	Monotonic, cyclic, shaketable: Landers 1992

Light gauge steel frame / wood panel shear wall research commenced in the late 1970s with the efforts of Tarpy at Vanderbilt University. The first major experimental research program was sponsored by the United States Steel Corporation (USS) and is described in the following publications: Tarpy and McCreless *(1976)*, Tarpy and McBrearty *(1978)*, McCreless *(1977)*, McCreless and Tarpy *(1978)*. This initial test program as well as subsequent research under the guidance of Tarpy and sponsored by the American Iron and Steel Institute (AISI) are summarized and assembled in a report by Klippstein and Tarpy *(1992)*. Prescriptive light gauge steel frame / wood panel shear wall design values appeared in the United States in the 1997 version of the UBC *(ICBO, 1997)*, and have since been expanded upon in the 1998 Shear Wall Design Guide *(AISI, 1998)*, the 2000 International Building Code *(ICC, 2000)*, and the draft 2002 version of the Standard for Cold-Formed Steel Framing: Design Provisions Lateral Resistance *(AISI, 2002c)*. In Canada, a design method including prescriptive design values for light gauge steel frame / wood panel shear walls does not currently exist in standard or guide form.

Zhao *(2002)* has written a literature review of existing test programs on light gauge steel frame shear walls that have been completed both in and outside of North America. For test programs undertaken in North America, Zhao provided details on research conducted by:

- McCreless and Tarpy *(1978)*
- Tarpy and Hauenstein *(1978)*
- Tarpy *(1980)*
- Tarpy and Girard *(1982)*
- Tissell *(1993)*
- Serrette *et al. (1996a, 1996b)* and Serrette *(1997)*
- Serrette and Ogunfunmi *(1996)*
- National Association of Home Builders (NAHB) *(1997)*
- Serrette *et al. (1997a)*

- Serrette *et al. (1997b)*
- Salenikovich and Dolan *(1999b)* and Salenikovich *et al. (2000a)*
- City of Los Angeles (CoLA) – University of California at Irvine (UCI) *(2001)*

A summary of these test programs is presented in Table 2.2.

Table 2.2: Summary of existing light gauge steel frame shear wall test programs *(Zhao, 2002)*

Study	Walls tested	Sheathing type(s) and connector	Loading protocol
McCreless and Tarpy *(1978)*	16 various dimensions	1/2" gypsum wallboard; No. 6 x 1" drywall screws	Monotonic
Tarpy and Hauenstein *(1978)*	18 8' x 8' and 12' x 8' walls	1/2" gypsum wallboard; No. 6 x 1" drywall screws	Monotonic
Tarpy *(1980)*	12 8' x 8' and 12' x 8' walls	1/2" gypsum wallboard, 7/8" cement plaster; No. 6 x 1" or No. 8 x 1/2" pan head (sheathing)	Monotonic and reversed cyclic
Tarpy and Girard *(1982)*	14 8' x 8' and 12' x 8' walls	1/2" gypsum wallboard, plywood; No. 6 x 1" drywall screws	Monotonic
Tissell *(1993)*	Eight 4' x 8'	11.1 mm OSB and 9.5 mm and 15.9 mm plywood; No. 10-24 (14, 16 gauge), No. 8-18 (18 gauge), 3.7 mm dia. Steel pin	Monotonic
Serrette *et al. (1996a, 1996b)* and Serrette *(1997)*	48 8' x 8' and 4' x 8' walls	15/32" plywood, 7/16" OSB, 1/2" gypsum wallboard; No. 8 x 1" (ply and OSB), No.6 x 1-1/4" (gypsum)	Monotonic and reversed cyclic: SPD
Serrette and Ogunfunmi *(1996)*	13 8' x 8' walls	50.8 mm x 0.88 mm flat strap bracing, 1/2" gypsum wallboard, 1/2" gypsum sheathing board; No. 8 x 1/2" (steel straps), No. 6 x 1" drywall screws (gypsum)	Monotonic
National Association of Home Builders (NAHB) *(1997)*	4 40' x 8' walls	7/16" OSB, 1/2" gypsum wallboard; No. 8 (OSB), No. 6 (gypsum)	Monotonic
Serrette *et al. (1997a)*	44 2' x 8' and 4' x 8' walls	15/32" plywood, 7/16" OSB, 0.84 mm flat strap bracing, 0.69 mm and 0.46 sheet steel; No. 8 screws of various length	Monotonic and reversed cyclic: SPD
Serrette *et al. (1997b)*	Full scale: 8' x 8' Small scale: 2' x 2'	11.9 mm plywood, 11.1 mm OSB, 12.7 mm FiberBond, 12.7 mm GWB; No. 6 x 1", 3.7 mm dia. Steel pin, No. 8 x 1-1/2", No. 6 x 1-1/4"	Monotonic
Salenikovich and Dolan *(1999b)* and Salenikovich *et al. (2000a)*	16 40' x 8'	7/16" OSB, 1/2" gypsum wallboard; No. 8 screws	Monotonic and reversed cyclic: SPD
City of Los Angeles (CoLA) – University of California at Irvine (UCI) *(2001)*	12 8' x 8'	15/32" plywood, 7/16" OSB; No. 8 screws	Reversed cyclic: SPD
Serrette *et al. (2002)*	20 4' x 8' and 8' x 8' walls	11.0 mm OSB, 0.69 mm sheet steel, 12.5 mm gypsum wallboard	Monotonic and reversed cyclic
Fülöp and Dubina *(2002, 2003)*	15 8' x 12' walls	Corrugated sheet steel, 10.0 mm OSB, flat strap bracing, gypsum wallboard	Monotonic and reversed cyclic
Branston *et al. (2003)*	6 4' x 8' walls 6 8' x 8' walls	11 mm OSB and 12 mm plywood	Monotonic and reversed cyclic

Zhao also presented details on light gauge steel frame shear wall test programs conducted outside of North America, particularly in Australia at the University of Melbourne. These research programs included:

- Gad *et al. (1997, 1998, 1999a, 1999b, 1999c, 2000)*

This literature review will not attempt to review the above-mentioned research programs for which details can be found in Zhao *(2002)*. Included in the following literature review will be an analysis of existing North American shear wall test programs on light gauge steel frame shear walls and test programs carried out outside of North America that were not previously reviewed. In addition, a comparison between the current performance and manufacturing standards for structural-use wood panels in both Canada and the United States will be presented. A literature review of existing data interpretation methodologies to obtain design parameters as well as a description of past test results that have been incorporated into current design codes for both wood frame and steel frame shear walls is presented in Chapter 4.

2.1 EXISTING NORTH AMERICAN LIGHT GAUGE STEEL FRAME SHEAR WALL TEST PROGRAMS

Serrette *et al.* *(2002)*

The purpose of this experimental research program was to investigate the behaviour of light gauge steel frame shear wall configurations not permitted in the U.S. model building codes at the time of publication (2002). The test program was carried out by the Light Gauge Steel Research Group (LGSRG) of Santa Clara University and was assembled in a report submitted to the NAHB Research Center. Since the most recent design values for light gauge steel frame shear walls are contained in the 2000 IBC *(ICC, 2000)* and the shear wall design guide draft *(AISI, 2002a)*, Serrette addresses the limitations (both design and testing limits) that designers must face when it comes to choosing a suitable shear wall configuration. Some of these limitations arise from results of the research (design limits) while others arise due to the lack of coverage of the test data. These limitations include:

- The maximum and minimum uncoated base metal thickness for the framing members is limited to 0.048" (1.22 mm) and 0.033" (0.84 mm), respectively.
- Stud dimensions are limited to a minimum of 1-5/8" (41.3 mm) flange, 3-1/2" (88.9 mm) web, with a return lip of 3/8" (9.5 mm).
- Track dimensions are limited to a minimum of 1-1/4" (31.8 mm) flange, with a 3-1/2" (88.9 mm) web.
- Back-to-back chord studs must be used in order to prevent local and flexural buckling of boundary members.
- No. 8 screws are required as minimum to connect the sheathing (plywood and OSB) to the underlying framing members.
- No. 6 screws are required as minimum to connect gypsum wallboard sheathing to the underlying framing members.
- Aspect ratios are limited (up to 4:1 for certain wall configurations and applications provided a reduction in shear capacity is applied *(AISI, 2002a)*).
- Walls sheathed on both sides with identical materials are not considered to have an increased capacity.
- Gypsum wallboard sheathing must be installed perpendicular to the framing with blocking.

In order to address these limitations and to provide test data for wall configurations not currently (2002) designated in the U.S. model building codes, the Light Gauge Steel Research Group investigated four areas of performance including:

- Reversed cyclic tests on specimens sheathed on one side with 7/16" (11 mm) OSB and framed with 0.054" (1.37 mm) and 0.068" (1.73 mm) light gauge steel.
- Reversed cyclic tests on specimens sheathed on two sides with 7/16" (11 mm) OSB and framed with 0.054" (1.37 mm) and 0.068" (1.73 mm) light gauge steel.
- Reversed cyclic tests on specimens sheathed with 0.027" (0.69 mm) sheet steel oriented perpendicular to the framing with simple lap shear connections between sheets.
- Monotonic tests on specimens sheathed on one side with 1/2" (12.5 mm) gypsum wallboard with blocking configurations not recognized in the 2000 IBC *(ICC, 2000)*.

In total, 20 tests were performed: 10 monotonic and 10 reversed cyclic. All reversed cyclic tests consisted of 4' x 8' (1220 x 2440 mm) shear wall specimens, while the monotonic tests on gypsum wallboard sheathed shear walls were 8' x 8' (2440 x 2440 mm) specimens. For each wall configuration considered, two replicate wall specimens were tested. In general, the first two performance groups consisted of tests on OSB sheathed walls (either on one side or on both sides of the wall) with a fastener spacing of 2" (50 mm) around the perimeter of the sheathing panels and 12" (305 mm) in the field of the panel. Eight reversed cyclic tests were conducted in order to investigate differences in wall behaviour due to the use of No. 8 and No. 10 sheathing-to-framing screws. Two reversed cyclic tests were performed on wall specimens sheathed with sheet steel fastened to the framing members with No. 8 screws. The fastener schedule for these walls was also 2"/12" (50/305 mm). Ten tests were then carried out on wall specimens sheathed with gypsum wallboard fastened to the framing members with No. 6 drywall screws and various fastener schedules (4"/4" (102/102 mm), 7"/7" (178/178 mm), 8"/12" (203/305 mm), 4"/12" (102/305 mm)). In addition, these 5 wall configurations incorporated several different blocking scenarios. Additional details on the test program are contained in Table A.1 (Appendix A).

The reversed cyclic test procedure consisted of three consecutive displacement cycles at increasing amplitude displacement levels, *i.e.* 0.2" (5 mm), 0.4" (10 mm), 0.6" (15 mm), 0.8" (20 mm), *etc.*, with no decreasing or degradation cycles. The wall specimens were subject to a maximum displacement amplitude of 4.0" (100 mm) at a cyclic frequency of 0.2 Hz. The monotonic tests consisted of displacing the top of the wall at a rate of 0.02"/sec. (0.5 mm/sec.) to a maximum displacement of 4" (100 mm).

The authors concluded that both No. 8 and No. 10 fasteners in OSB shear walls framed with 0.054" (1.37 mm) and 0.068" (1.73 mm) light gauge steel failed in a ductile manner. This is a favourable mode of failure since it allows the shear wall to dissipate energy while deforming in the inelastic range. The walls sheathed on both sides with OSB did not reach their full capacity due to compression buckling in the chord studs when the shear walls were framed with 0.054" (1.37 mm) thick studs. With the 0.068" (1.73 mm) thick framing, the compression chords did not buckle, however, the connection between the hold-down screws and the chord studs was not sufficient to carry the increased tension loads. There was still a noticeable increase in shear capacity for the double sided walls (70 - 75 % more than the single sided walls). The authors predicted that had the chord studs not buckled and the hold-down connections withstood the increased loads, the capacity of the double sided walls would be almost twice that of the single sided walls.

In the walls sheathed with sheet steel, because of high tension field action, the screw connection at the mid-height lap joint failed. This failure did not allow the steel sheet to reach its maximum capacity. The gypsum wallboard walls performed in a similar manner to walls that were tested previously by the Light Gauge Steel Research Group in that an "un-zipping" of the sheathing to framing connections was observed along the "un-papered" edges and a pull-through of the connectors resulted when the edges were "papered". The authors also comment that the inelastic drift (drift at peak load) for all tests was less than the code prescribed limit of 2.5 % (2.4" (61 mm) for an 8' (2440 mm) high wall).

2.2 Existing Light Gauge Steel Frame Shear Wall Test Programs Outside of North America

Fülöp and Dubina *(2002, 2003)*

At the Politehnica University of Timisoara in Romania, Fülöp and Dubina initiated a test program on six light gauge steel frame shear wall configurations for a total of 15 tests. The wall specimens were all 12' (3600 mm) in length with a storey height of 8' (2440 mm). The C-shaped steel studs and U-shaped tracks incorporated into the walls, according to their Romanian designation, were C150/1.5 and U154/1.5 profiles, respectively, supplied by a local manufacturer. Back-to-back chord studs and hold-downs were used and intermediate framing studs were spaced at 600 mm on centre. The wall configurations tested are outlined in Table 2.3.

The corrugated steel sheets used as sheathing were oriented in the horizontal position, perpendicular to the framing members. One corrugation was overlapped and seam fasteners spaced at 200 mm were used to provide the connection between the multiple sheets incorporated into each wall specimen. The corrugated sheet steel sheathing was secured to the light gauge steel framing using self-tapping screws with a diameter of 4.8 mm spaced at every corrugation along the sheet ends and spaced at every second corrugation along the intermediate connections to the interior studs. The gypsum wallboard used to sheath the interior side of wall configuration II was 12.5 mm thick and connected to the vertical studs at 250 mm on centre.

Table 2.3: Description of wall specimens tested by Fülöp and Dubina *(2002, 2003)*

Series	Opening	Strap Bracing	Sheathing Material	Interior Shthg	Load Protocol	Loading Rate	No. of Tests
I	-	-	Corrugated sheet steel	-	Mono	10 mm/min	1
	-	-	Corrugated sheet steel	-	Cyclic	6 min/cyc.	1
						3 min/cyc.	1
II	-	-	Corrugated sheet steel	Gypsum wallboard	Mono	10 mm/min	1
	-	-	Corrugated sheet steel	Gypsum wallboard	Cyclic	6 min/cyc.	1
						3 min/cyc.	1
III	-	Yes	-	-	Mono	10 mm/min	1
	-	Yes	-	-	Cyclic	3 min/cyc.	1
IV	1200 mm door	-	Corrugated sheet steel	-	Mono	10 mm/min	1
	1200 mm door	-	Corrugated sheet steel	-	Cyclic	6 min/cyc.	1
						3 min/cyc.	1
OSB I (V)	-	-	10 mm OSB	-	Mono	10 mm/min	1
	-	-	10 mm OSB	-	Cyclic	3 min/cyc.	1
OSB II (VI)	1200 mm door	-	10 mm OSB	-	Mono	10 mm/min	1
	1200 mm door	-	10 mm OSB	-	Cyclic	3 min/cyc.	1
						Total No. of Tests	15

For wall configuration III, strap bracing was used in order to provide the necessary strength and stiffness to resist lateral load in the shear wall. The strap braces were made up of the same material as the light gauge steel studs and track and had dimensions of 110 mm wide x 1.5 mm thick. Two straps were applied to each side of the wall; screw connections at opposite corners of the wall were over-designed in order to ensure the yielding of the straps themselves.

Three 1200 x 2440 mm OSB panels were used to sheath one side of the specimens which made up wall configurations OSB I and OSB II. The OSB panels were 10.0 mm thick and were fastened to the steel framing members using bugle head self-drilling screws (dia. = 4.2 mm) at 10.5 cm intervals.

All wall configurations were subject to both monotonic and reversed cyclic loading protocols as per Table 2.3. The monotonic tests consisted of displacing the top of the wall at a rate of 1 cm/min. The walls were restrained against out-of-plane movement and the loading was restricted to shear only; it was beyond the scope of the research to account for both lateral and vertical loading on the shear wall. Based on the monotonic test results, an elastic displacement limit was determined as the intersection between the line denoting the elastic stiffness (secant stiffness through the origin and the point on the load-deflection curve corresponding to 40 % of the ultimate wall resistance) and the line tangent to the curve with a slope of 10 % of the elastic stiffness (Chapter 4, Section 4.1.2). The displacement amplitudes for the reversed cyclic tests were then made up of multiples of the elastic limit displacement, *i.e.* 0.25, 0.50, 0.75, 1.0, *etc.* For wall types I, II, and IV, the reversed cyclic tests were conducted at a rate of 6 minutes per cycle and 3 minutes per cycle, whereas all other wall specimens were conducted at a rate of 3 min/cycle only.

The authors concluded that all wall specimens provided a significant and recognizable shear resistance that could be relied on in practice to resist wind and seismic loads. Failure usually commenced at the connections on the bottom track, close to the wall corner where the highest shear forces existed. The authors note the importance of properly detailing the bottom corner of the shear wall including the hold-down connectors and the anchor bolts. The high forces in the chord studs induced by the shear wall loading must be transferred directly from the chords themselves, through the hold-down apparatus and the anchor bolt. A load path through the sheathing, the connectors, and finally through the bottom track is not favourable since the bending capacity of the weak axis of the bottom track is not sufficient to allow for the transfer. In the case of the corrugated sheet steel sheathed walls, similar to the walls tested by the Light Gauge Steel Research Group at Santa Clara University *(Serrette et al., 2002)*, it was observed that the overlapping connection between multiple sheets represented a critical connection and severely affected the overall performance of the shear wall. The failure of this seam connection does not allow for the steel sheets to reach their "full" capacity. When comparing the performance of the different wall configurations tested it was found that the walls sheathed with gypsum wallboard on the interior exhibited approximately 17 % increased strength, despite being controlled by the seam fasteners. Considering the walls with openings, both series IV and OSB II demonstrated a significant decrease in initial stiffness (approx. 60 %) and a lesser, but nevertheless noticeable, decrease in ultimate resistance (20 – 30 %).

2.3 SHEATHING MATERIALS

2.3.1 Overview of Sheathing Materials used in Experimental Program

In terms of sheathing components, three types of structural-use panels were included in the main testing program described in Chapter 3. Because structural-use panels are strong and stiff in their own plane, when applied over light gauge steel framing members, they provide a significant shear resistance to the overall shear wall assembly. With the assembly acting as a vertical cantilever beam, the sheathing behaves as a deep web which takes the shear force while the tension and compression forces are carried by the perimeter members (chords) of the wall. Forces are transferred between the sheathing and framing members via screw connectors. Since the overall capacity of the shear wall is usually governed by the failure of these connections, the wood sheathing has a substantial effect on the lateral load carrying performance. Hence, a review of the fabrication requirements and materials of the wood sheathing was carried out. The three types of panels considered were:

- 12.5 mm CSA O151 Exterior Sheathing Canadian Softwood Plywood (CSP) *(CSA O151, 1978)*
- 12.5 mm CSA O121 Exterior Sheathing Douglas Fir Plywood (DFP) *(CSA O121, 1978)*

- 11 mm CSA O325 Grade O-2 Oriented Strand Board (OSB) *(CSA O325, 1992)* rated 1R24/2F16/W24

The standards for the above mentioned plywood products (CSP, DFP) are product standards, whereas that for the oriented strand board (OSB) is a performance standard. Plywood manufactured to meet the product standards is able to withstand certain exposure conditions including extreme moisture and temperature environments. This ensures that the material is suitable for exterior exposure and is graded as an EXTERIOR-type sheathing panel. The product standards specify requirements for the grades and species of veneer which form the panel lay-up, and also set out requirements to be met to ensure that the proper bond durability exists between these layers. A performance standard differs from a product standard in that instead of specifying by what means the product must be manufactured, certain requirements must be met for the panel to be graded according to a specified end-use. The performance standard in Canada *(CSA O325, 1992)*, however, does not classify the panel as being suitable for exterior type conditions. CSA O325 *(1992)* lists panel grades according to certain end-use designations, including:

- *Floor Sheathing (F)*: 1F16 or 2F16, where 1F designates that the panel is to be used as a floor sheathing and the sheathing alone can meet the structural requirements for a span of a 16" (406 mm) and 2F designates that the performance requirements can be met if the panel is used with an underlay, which is a second layer of panelling placed on top of the performance rated OSB. Other span ratings also exist, *i.e.* 20" (508 mm), 24" (610 mm), 32" (813 mm), *etc.*
- *Roof Sheathing (R)*: 1R24 or 2R24, where 1R designates that the panel is to be used as a roof sheathing and the sheathing alone can meet the structural requirements for a span of 24" (610 mm) and 2R designates that the performance requirements can be met if the panel is used with additional support elements such as edge support provided by H-clips.

- *Wall Sheathing (W)*: W16, where this end-use grade designates that the panel is to be used for a maximum span of 16" (406 mm) in a wall.

As an example, a panel with an end-use designation of 1R24/2F16/W24 can be used for structural purposes on roofs with rafters or trusses spaced at 24" (610 mm) on centre or less, on a floor with a panel-type underlay and joists spaced at 16" (406 mm), or less, or on a stud wall with studs spaced at a maximum of 24" (610 mm). It must be noted that the final grade marking includes only the name or logo of the manufacturer and the mill identification number, the name or logo of the mill's certification organization, the date of manufacture, the designation "CSA O325" and the end-use designation, *e.g.* 1R24/2F16/W24, and the nominal thickness of the panel. The panel is not classified as being suitable for exterior conditions and so must be used in an interior application where dry conditions exist. It must also be noted that panels of different thicknesses may have the same rating depending on the material used to make up the panel. While any wood-based material may be used to make construction sheathing, in Canada OSB is most commonly certified to CSA O325.

Waferboard was originally developed in the 1940s but has been superseded by the stronger and stiffer OSB panel developed in Germany. OSB is now widely used in North America as a reliable alternative to plywood. OSB is made up of strands that are approximately 80 mm long, 25 mm wide and 1 mm thick and are aligned along the long dimension of the panel in the face layers. This feature gives OSB its increased resistance along its major axis. The strands originate from aspen or poplar species of trees and, after being dried and sorted, are glued together with a phenolic adhesive under heat and pressure to form the rigid panel. Strands in the middle layers of the panel can either be oriented perpendicular to those in the face grain or placed randomly resulting in slightly different structural properties for the different grades of OSB *(SBA, 2001)*.

Two other standards for OSB also exist in Canada: CSA O452: Design Rated OSB *(1994)* and CSA O437: OSB and Waferboard *(1993)*. CSA O437 is a product standard similar to CSA O121 and CSA O151 in that it specifies the make-up of the structural panels. OSB panels are fabricated according to CSA O437 and are then graded under CSA O452. According to the design rating standard, three types of OSB panels are recognized: Type I (standard), Type II (plus) and Type III (proprietary). A Type II (plus) panel will have increased strength properties of 10 % over a Type I panel. Within strength designations Type I and Type II, rating grades of A, B, and C also exist, where grade A designates a higher bending capacity, for example. OSB panels can be manufactured as grade O-2, where strands are aligned in the face layers and strands making up the inner core are aligned perpendicular to the face layers, as grade O-1, where strands are aligned in the face layers and are placed randomly in the core layers, or as R-1 (waferboard), where strands are placed randomly throughout.

Plywood is made up of thin veneers or plies oriented with their grain direction either parallel or perpendicular to the long dimension of the panel. In the outer plies, the grain direction of the veneers is usually aligned with the long dimension of the panel in order to provide the greatest bending resistance along this axis. The inner plies of the panel are usually aligned perpendicular to the outer plies or with their grain direction parallel to the short dimension of the panel giving plywood its two-way strength. Veneers, usually 2 – 4 mm in thickness, are peeled from steamed logs, dried to approximately 5 % moisture content at temperatures ranging from 160 to 200°C, coated with an adhesive, and then bonded together with pressure and heat (approximately 1.4 MPa and 150°C) to form a rigid panel which is cut to dimension and graded *(CANPLY, 1999)*.

2.3.2 Comparison of Standards for Structural-Use Panels in Canada and the United States

Performance Rated OSB

Performance rated OSB is graded according to CSA O325 *(1992)* in Canada and according to the American Plywood Association (APA) voluntary product standard PS2-92 *(APA PS2, 1992)* in the United States. According to both of these standards, certain tests on the panels themselves are carried out to ensure that the final product meets the intended end-use requirements. The panel tests can be classified into three main groups:

- Structural adequacy
- Dimensional stability
- Glue bond durability

The Structural Board Association (SBA) represents a number of member OSB manufacturing companies in Canada, the United States, as well as overseas. Because this association represents companies internationally, OSB panels are usually fabricated and performance tested to meet both CSA O325 and PS2-92. PS2 was developed in the United States in conjunction with members of the Canadian wood panel industry under the U.S. / Canada Free Trade Agreement, and, for this reason, the performance requirements set out by both standards are very similar *(SBA, 2001)*. This similarity is demonstrated in Table 2.4 which was adapted from the SBA OSB: Performance by Design Manual *(SBA, 2001)*.

OSB panels are qualified by third-party certification agencies for performance requirements. As shown in Table 2.4, OSB panels graded to CSA O325 and PS2-92 are very similar but differ mainly only in the wall strength requirements.

Table 2.4: Performance requirements for CSA O325 and PS2-92 *(SBA, 2001)*

Property	CSA O325	PS2-92
Thickness tolerance	1.5 mm range	1.6 mm range
Length and width, from stated dimensions	+0, -4 mm	+0, -3.2 mm
Squareness, maximum deviation from square along diagonal	4 mm	3.6 mm
Straightness, maximum deviation from straight	1.5 mm/edge	1.6 mm/edge
Ultimate concentrated load		
▪ Roof -static	1.78 kN	1.78 kN
-following impact	1.33 kN	1.33 kN
▪ Subfloor with underlay		
-static	1.78 kN	1.78 kN
-following impact	1.78 kN	1.78 kN
▪ Subfloor maximum 24" span		
-static	2.45 kN	2.45 kN
-following impact	1.78 kN	1.78 kN
▪ Subfloor 32" to 48" span		
-static	3.12 kN	3.12 kN
-following impact	1.78 kN	1.78 kN
Maximum deflection under 0.89 kN load	varies with application[1]	varies with application[1]
Ultimate uniformly distributed load		
▪ Roof	7.2 kPa	7.2 kPa
▪ Floor		
-max 32" span	15.8 kPa	15.8 kPa
-max 48" span	10.8 kPa	10.8 kPa
▪ Wall	no requirements	3.6 kPa
Maximum deflection under uniform load		
▪ Roof (1.68 kPa load)	span/240	span/240
▪ Floor (4.79 kPa load)	span/360	span/360
Linear expansion, maximum		
▪ One sided wetting and relative humidity exposure (50 – 90%)	0.30% major axis 0.35% minor axis	0.30% major axis 0.35% minor axis
▪ Oven dry to vacuum pressure soak	0.50%	0.50%
Thickness swell, maximum		
▪ One sided wetting after 14 day exposure (single floor only)	25%	25%
▪ After 24-hr soak		
-12.7 mm and thinner	25%	N/A
-thicker than 12.7 mm	20%	N/A
Bond durability		
▪ 6 hour cycle	50% strength retention	50% strength retention
▪ 2 hour boil	50% strength retention	N/A

[1]Varies with application, however, requirements for specific applications are identical in both standards

1" = 25.4 mm

PS2-92 has higher strength requirements for panels incorporated into structures as wall sheathing. In addition, CSA O325 does not require racking tests for panels designated as "W" end-use only. For this reason, in Canada, for sheathing to be incorporated into shear walls and diaphragms, panels with end-use markings of a combination of "W" and "R and / or F" should be used to circumvent this problem.

In addition to the requirements listed above in Table 2.4, both CSA O325 and PS2-92 require a minimum performance for fasteners tested under a lateral resistance load and a withdrawal load. The minimum requirements are dependant on the end-use application, however, the standards are identical in specifying the minimum loads to be attained in both lateral and withdrawal resistance. This property is quite important with respect to shear walls, since it is the sheathing to framing connection which typically dictates the overall behaviour and performance of the assembly. Light gauge steel frame / wood panel shear walls usually fail in a way which is governed by the sheathing fastener tearing out of the side of the panel or the screw head pulling through the sheathing.

PS2-92 gives end-use ratings or span ratings that are somewhat different than CSA O325 as described in Section 2.3.1. In the United States, span ratings consist of two numbers, *i.e.* 24/16. The first number in this designation represents the maximum allowable span, in inches, for roof sheathing applications (it can also be assumed that the same span can be achieved for a wall application), while the second number in the designation represents the maximum allowable span, in inches, when the panel is used in flooring applications in conjunction with a panel-type underlay (equivalent to a 2F16 designation according to CSA O325). When a single number is followed by the letters "OC" (on centre), this panel is to be used in a flooring application (single floor – equivalent to a "1F" designation in CSA O325) and would replace the need for an underlay acting together in combination with a subfloor. APA Rated sheathing *(APA PS2, 1992)* may also be performance tested for use specifically as wall sheathing. The span ratings for this type of application are designated as "Wall-16" or "Wall-24", where 16 and 24 represent the allowable span (in inches). PS2-92 also specifies two other grades of performance rated structural-use panels, namely, sheathing grade and Structural I sheathing. The performance characteristics listed in Table 2.4 are for sheathing grade panels. Structural I grade panels require supplementary performance tests and must meet additional requirements for cross-panel strength (ultimate uniformly distributed load) and stiffness (maximum deflection under uniform load) and for racking shear. Performance rated panels graded under PS2-92 may be classified as Exposure I. Exposure I designation assures that the panel is not to be used in permanent exterior applications, however, is suitable for exposure to the elements during construction delays, water leakage, *etc.*

According to these requirements, as well as the comparison of requirements listed in Table 2.4, it can be assumed that OSB panels graded to CSA O325 *(1992)* and APA PS2-92 *(1992)* (rated sheathing grade) are equivalent and can be used interchangeably for light gauge steel frame / wood panel shear wall applications, provided the end-use rating and span rating are equivalent, *e.g.* PS2 24/0 is equivalent to CSA O325 2R24/W24 and PS2 32 OC is equivalent to CSA O325 1F32. A typical grade stamp found on a performance rated OSB panel graded according to both CSA O325 and PS2-92 is shown in Figure 2.1.

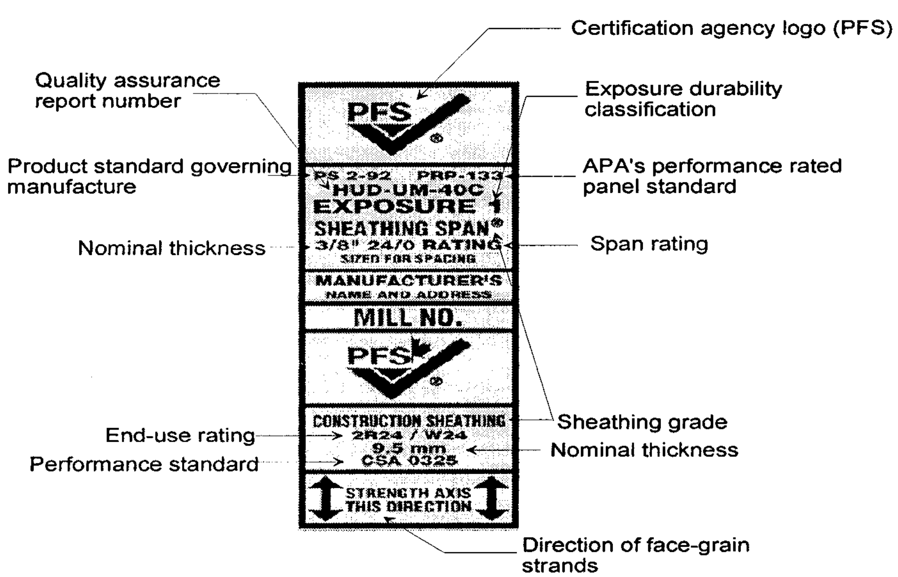

Figure 2.1: Typical grade stamp (dual-stamp) for performance rated OSB

Plywood

Douglas Fir Plywood *(CSA O121, 1978)* and Canadian Softwood Plywood *(CSA O151, 1978)* differ by the species which make up their veneers or plies. Poplar plywood *(CSA O153, 1980)* also exists in Canada, however it is typically not used for structural purposes. In general, DFP is made up of outer plies of Douglas Fir, while the inner plies can be any of the species listed in the standard (Table 2.5). CSP can be made up of most of the species listed as inner plies of DFP, even as outer plies (Table 2.5). Veneers are graded as A, B or C depending on the quality of the sheet. Grade A represents the highest quality, however, strength values listed in CSA O86 *(2001)* are for panels made up of C-grade veneers in the face, back, and inner core plies. Veneers are visually graded according to the presence of knots, splits, surface roughness, streaks, discolorations and grain imperfections. Wood inlay or synthetic patches are often used to fill gaps or holes. Unsanded grades of plywood include sheathing grade (SHG) (consisting of all C-grade veneers), select grade (SELECT) (consisting of B-grade veneers on the face, C-grade veneers in the back and inner plies), and select tight-face (SEL TF) (same grade veneers as select grade, however, of superior visual quality). Unsanded grades are used for structural purposes whereas sanded grades consist of A-grade veneers and are typically used in furniture applications.

CSA O121 *(1978)* and CSA O151 *(1978)* both require that for panels of thickness between 6.0 mm and 11.0 mm, a minimum of three plies must be used while a panel of 14.0 mm thickness must contain 4 plies and a panel between 17.0 mm and 19.0 mm thickness must contain 5 plies. The greatest nominal thickness allowed for inner plies and outer plies is 5.0 mm and 3.2 mm, respectively. The minimum permissible nominal thickness of all plies is 2.4 mm. Both Canadian standards also set out identical requirements for manufacturing tolerances. In terms of thickness, for example, -0.5 mm to +1.0 mm is considered an acceptable deviation from the nominal thickness for regular unsanded grades of plywood. Limits are also set out for width and length, panel squareness and edge straightness. In both cases, bond durability tests are conducted on conditioned specimens. Conditioning cycles are performed in order to represent extreme in situ moisture and temperature conditions. The main goal of the bond durability tests is to have a failure in the wood rather than at the interface of the adhered plies.

Table 2.5: Permissible species for Canadian plywood *(CSA O121, 1978; CSA O151, 1978)*

Canadian Softwood Plywood (CSP) CSA O151		**Douglas Fir Plywood (DFP) CSA O121**	
Face Plies	**Inner Plies**	**Face Plies**	**Inner Plies**
	Douglas Fir	Douglas Fir	Douglas Fir
Western Hemlock	Western Hemlock		Western Hemlock
True Fir	True Fir		True Fir
Sitka Spruce	Sitka Spruce		Sitka Spruce
Western White Spruce	Western White Spruce		Western White Spruce
Western Larch	Western Larch		Western Larch
	Western White Pine		Western White Pine
	Ponderosa Pine		Ponderosa Pine
Lodgepole Pine	Lodgepole Pine		Lodgepole Pine
	Balsam Fir		Balsam Fir
Eastern Spruce	Eastern Spruce		Eastern Spruce
Jack Pine	Jack Pine		Jack Pine
Tamarack	Tamarack		
Yellow Cedar	Yellow Cedar		
Red Pine	Red Pine		
Eastern Hemlock	Eastern Hemlock		
Eastern White Pine	Eastern White Pine		
	Balsam Poplar		
	Trembling Aspen		
	Western Red Cedar		

Plywood panels manufactured according to APA PS1 *(1995)* differ considerably from those manufactured according to CSA O121 *(1978)* and CSA O151 *(1978)* in terms of the type of species that form the panel lay-up. APA PS1 allows for the use of a number of species which are not covered by the Canadian standards including; Apitong, Beech, Kapur, Keruing, Maple, Tanoak, Cypress, Lauan, Meranti, Mersawa, Alder, *etc. (APA PS1, 1995)*. Wood behaviour varies greatly with species mainly due to differences in density. When considering the behaviour of light gauge steel frame / wood panel shear walls, the wood-to-frame connections have a direct impact on performance. Because, when loaded, the screw mainly tilts and rocks when fastened through the steel framing member, it is mostly the wood sheathing that dictates this connection behaviour. Since the wood species used for the PS1 and CSA plywood standards are different, it is suggested at this time that one not interchangeably use US and Canadian plywood in a shear wall assembly assuming that a similar design capacity will exist. Since shear wall capacity can be assumed to be dependant upon the lateral and pull-through resistance of a screw fastening the wood panel to the framing member, the following recommendations are made for future research in order to draw a link between panels manufactured under their respective National Standards:

- Make use of connection test data to establish a link between the lateral screw resistance in Canadian plywood *(Okasha, 2004)* and APA PS1 plywood *(APA E830C, 1995)* in hope to extend this conclusion to overall shear wall behaviour.
- Undertake a test program on light gauge steel frame / APA PS1 *(1995)* plywood sheathed shear walls in order to provide bottom line design values applicable for all types of plywood manufactured in Canada and the United States.
- Complete a detailed investigation of the bearing capacity (for connections) of the various species of wood veneers in order to identify the similarities between the two standards, and, ultimately, to conclude on whether or not the plywood types can be used interchangeably.

2.4 SUMMARY

The findings of this literature review on existing test programs, in addition to the information provided by Zhao *(2002)*, Van de Lindt *(2004)* and CUREE *(Filiatrault, 2001)*, helped shape the preliminary and main testing programs of this body of research, later described in Chapter 3. Many aspects of the test set-up, procedure and test protocols were replicated from past bodies of research. A literature review on existing data interpretation techniques and prescriptive design methods for light frame shear walls both in the United States and Canada (Chapter 4) provided the basis necessary to select the most suitable data analysis methodology and to propose a design format to be used following the limit states design procedure of the upcoming version of the National Building Code of Canada *(NRCC, 2004)*. A comparison of sheathing standards in Canada and the United States provided the necessary evidence to conclude that design values derived later in this body of research may be applied for OSB-sheathed shear walls when the panel is graded either according to CSA O325 *(1992)* or APA PS2 *(1992)*, provided equivalent span ratings exist. On the other hand, differences in plywood panel make-up with respect to species type between Canadian plywood standards (CSA O151 *(1978)* and CSA O121 *(1978)*) and the plywood standards in the United States (APA PS1 *(1995)*) dictate that further research must be conducted to determine the correlation between shear wall test results in Canada and the United States.

CHAPTER 3 EXPERIMENTAL PROGRAM

In the summer of 2002, the Department of Civil Engineering and Applied Mechanics at McGill University installed in its structures laboratory a frame designed specifically for the testing of shear walls (Figures 3.1 and 3.2) *(Zhao, 2002)*. The test frame would function in much the same way as a laboratory strong wall providing a rigid reaction against which a load can be applied to the top of a shear wall that is anchored at its base. An advantage of the test frame is that it is able to transfer the applied shear force internally through the frame so that only vertical forces are applied to the supporting floor. The test frame is 11 m in length, just over 5 m in height, and has a clear height of 4 m to allow for the testing of shear walls up to 12' (3.66 m) in height and length. During design, strict deflection limits of *L*/800 were imposed in order to minimize the distortion of the frame during testing, and each member was designed to remain elastic at a limiting stress of $0.4F_y$ *(Zhao, 2002)*. The test frame was also designed to accommodate for an actuator having twice the capacity (500 kN / 110 kip) than that actually installed. This conservative design allows for retrofitting and upgrading of the test frame and its equipment if need be.

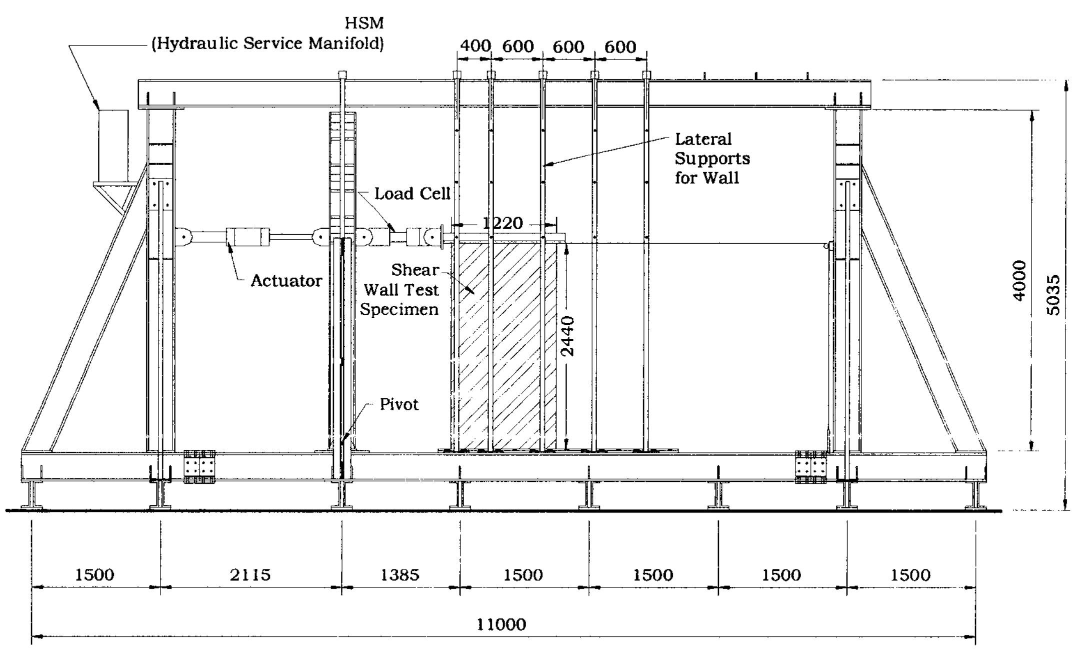

Figure 3.1: Test frame with 4' x 8' (1220 x 2440 mm) wall specimen

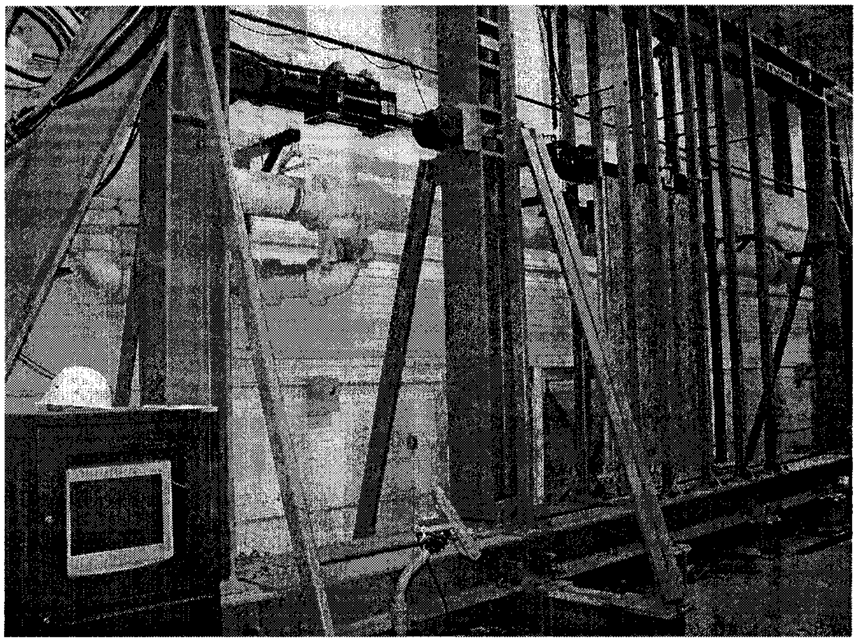

Figure 3.2: Shear wall test frame in structures laboratory at McGill University

The apparatus is comprised of several components including a 250 mm (10") stroke (± 125 mm (5")) dynamic actuator and a 250 kN (55 kip) load cell. As shown in Figures 3.1 and 3.2, the secondary column, which was designed to pivot at its base, serves two purposes: i) The column supports the weight of the actuator, thereby allowing the main component of the force transmitted to the wall to be horizontal. ii) It also allows for the actual actuator displacement to be amplified at the top of the wall by lowering the actuator's attachment position on the column while maintaining the original attachment position of the loading beam. The lateral braces prevent the out-of-plane movement of the wall while Teflon guides coated with grease ensure that developed friction forces are negligible.

This chapter provides an overview of both the preliminary and main test programs that were carried out at McGill University, as well as the detailed results of the 43-specimens that were the responsibility of the author. As stated in Chapter 1, the testing program was conducted in order to build a bank of data for various configurations of shear walls that are commonly used in current construction practice. The final result of the testing program will be to incorporate the nominal strengths from the test results into a limit states design format, for use with the upcoming National Building Code of Canada *(NRCC, 2004)* to estimate a design capacity for light gauge steel frame / wood panel shear walls subject to wind and earthquake loads. The development of the limit states design approach for these shear walls is the topic of Chapters 4 and 5.

3.1 OVERVIEW OF PRELIMINARY TESTING PROGRAM

In the fall of 2002, a preliminary testing program consisting of twelve shear wall specimens was carried out *(Branston et al., 2003)*. Even though design methods for light gauge steel frame / wood panel shear walls are not well developed or documented in Canada, research on these wall systems has been extensive in other countries. In the United States, the results of the research have been used to develop a Shear Wall Design Guide *(AISI, 1998, 2002a)* and nominal load design tables in the IBC and UBC model building codes *(ICC, 2000; ICBO, 1997)*. Design values provided in the aforementioned documents are direct results from the research conducted by Serrette and the Light Gauge Steel Research Group (LGSRG) of Santa Clara University *(Serrette et al., 1996b, 1997a, 2002)*. In the United States, for allowable stress design (ASD), the allowable load can be derived from the nominal load by dividing by a safety factor (Ω). For load and resistance factor design (LRFD) the design load can be obtained from the nominal load by multiplying by a resistance factor (Φ). The design values are valid for shear walls constructed with U.S.-based products which have not yet been proven to be applicable in Canada. The University of California-Irvine (UCI) *(CoLA-UCI, 2001; Freund, 2001; Larsen, 2000; Shah, 2001; Smith, 2001)* has also conducted an extensive research program on shear walls, with a subsection of their research devoted towards a suite of light gauge steel framed shear walls with wood panelling.

The purpose of the preliminary testing program was to replicate a limited number of shear wall specimens and to draw a link to previous tests done by Serrette

(Serrette et al., 1996b) and UCI *(CoLA-UCI, 2001)*. A comparison of the results would allow for an evaluation of the suitability of the test frame and the test set-up. If needed, modifications to the test frame and/or test set-up could be implemented prior to the initiation of the main testing program.

A complete discussion of the series of twelve preliminary tests can be found in Branston *et al. (2003)*. In general, the series of match tests consisted of 4' x 8' (1220 x 2440 mm) and 8' x 8' (2440 x 2440 mm) light gauge steel frame / wood panel shear walls. The materials used to construct the walls were purchased from the United States in order to replicate the wall specimens that were tested by Serrette *et al. (1996b)* and CoLA-UCI *(2001)*. ASTM A653 *(2002)* steel studs and tracks were used to construct the wall frame. All steel was nominal grade 230 MPa (33 ksi) and of 0.84 mm (0.033") nominal thickness (20 gauge). The wall sheathing, either 11 mm (7/16") oriented strand board (OSB) or 12.5 mm (15/32") four-ply plywood, was then laid on the light gauge steel framing and connected with sheathing screws at various edge and field spacings. All walls consisted of American Plywood Association (APA) rated sheathing *(APA PRP-108, 2001; APA PS1, 1995; APA PS2, 1992)* on one side, which was placed vertically on the wall (parallel to framing). The walls, once installed in the test frame, were subjected to monotonic and reversed cyclic testing protocols as defined in the original research reports *(Serrette et al., 1996b; CoLA-UCI, 2001)*.

Three nominally identical tests were performed for each of the four wall configurations under study. The monotonic tests followed the protocol as defined by Serrette *et al. (1996b)*. This stroke controlled test protocol displaces the top of the wall at a rate of 0.3" (7.62 mm) per minute to failure. The Sequential Phase Displacement (SPD) protocol, identical to that used by the respective researchers, was utilized in reversed cyclic testing. This protocol was originally proposed by the Joint Technical Coordinating Committee on Masonry Research (TCCMAR) *(Porter, 1987)* and was adopted by the Ad Hoc Committee on Testing Standards for Structural Systems and Components – Structural Engineers Association of

Southern California (SEAOSC) *(1997)*. The four configurations of walls tested are outlined in Table 3.1.

Table 3.1: Preliminary tests matching
Serrette *et al. (1996b)* and CoLA-UCI *(2001)*

McGill Preliminary Tests	Previous Tests	Sheathing	Nominal Wall Dimension	Loading Type
OSB 4-8 US M-A,B,C	Serrette *et al.* OSB – 1D3,4	OSB[1,3]	4' x 8'	Monotonic
OSB 4-8 US C-A,B,C	Serrette *et al.* AISI OSB 3,4	OSB[1,3]	4' x 8'	Reversed cyclic
PLY 8-8 US M-A,B,C	Serrette *et al.* PLY – 1A6,7	Plywood[2]	8' x 8'	Monotonic
PLY 8-8 US C-A,B,C	COLA-UCI Group 14 (14A,14B,14C)	Plywood[2]	8' x 8'	Reversed cyclic

[1]OSB 11 mm (7/16") APA Rated 24/16 sheathing Exposure 1
[2]Plywood 12.5 mm (15/32") APA Rated 32/16 4-ply sheathing Exposure 1
[3]OSB also CAN CSA O325 rated 2R24/2F16/W24 exterior adhesive
1 foot (ft) = 305 mm

Complete details of wall construction, test set-up, testing procedure, results and comparisons for the preliminary testing program can be found in Branston *et al. (2003)*. In general, certain variations in the performance of the replica tests from the previous tests existed. Based on these variations and other comparisons of performance, slight modifications to the test set-up and procedure were made. Some of the disparity, however, could be attributed to differences in material properties and hold-down installation.

3.2 Test Matrix for Main Testing Program

For the summer of 2003, a total of 100 light gauge steel frame / wood panel shear wall tests were planned. In most cases, six specimens (3 monotonic and 3 reversed cyclic) were tested per wall configuration to provide a minimum level of validity /

reliability for the test data, however, in some cases, where the initial series exhibited large variation (> 10 %), it was deemed necessary to perform supplementary tests. As stated in Chapter 1, the author was responsible for carrying out only a portion of the overall testing program and the scope of this research includes the study and analysis of 43 out of the total 109 tests. The test specimens were grouped so as to isolate sheathing materials, screw spacing, wall size, *etc.*

All 43 specimens were 4' x 8' (1220 x 2440 mm) in size and were constructed of the following components:

- Either 12.5 mm CSA O151 Exterior Canadian Softwood Plywood (CSP) *(CSA O151, 1978)*, 12.5 mm CSA O121 Exterior Douglas Fir Plywood (DFP) *(CSA O121, 1978)*, or 11 mm CSA O325 Grade O-2 Oriented Strand Board (OSB) *(CSA O325, 1992)* rated 1R24/2F16/W24 for wall sheathing on one side oriented vertically (strength axis or face grain parallel to framing).

- Light gauge steel studs manufactured in Canada to ASTM A653 *(2002)* with nominal grade and thickness of 230 MPa (33 ksi) and 1.12 mm (0.044", 18 gauge), respectively. The nominal dimensions of the studs were 3-5/8" (92.1 mm) web, 1-5/8" (41.3 mm) flange and 1/2" (12.7 mm) lip.

- Light gauge steel top and bottom tracks manufactured in Canada to ASTM A653 *(2002)* with nominal grade and thickness of 230 MPa (33 ksi) and 1.12 mm (0.044", 18 gauge), respectively. The nominal dimensions of the tracks were 3-5/8" (92.1 mm) web and 1-3/16" (30.2 mm) flange.

- Back-to-back chord studs connected by two No. 10-16 x 3/4" (19.1 mm) long Hex washer head self-drilling screws at 12" (305 mm) on centre. The built-up member was incorporated in order to prevent both flexural and local buckling failure of a single chord stud on its own. The remaining interior studs were spaced at 24" (610 mm) on centre.

- Industry standard Simpson Strong-Tie S/HD10 *(Simpson, 2001)* hold-down connectors attached to the chord studs with 33 No. 10-16 x 3/4" (19.1 mm) long Hex washer head self-drilling screws. An ASTM A307 *(2003)* 7/8" (22.2 mm) anchor rod was used to fasten the hold-down to the test frame.

- No. 8 x 1/2" (12.7 mm) long wafer head self-drilling framing screws. These screws were used to connect the track and studs.

- No. 8 x 1-1/2" (38.1 mm) long Grabber SuperDrive *(SuperDrive, 2003)* bugle head self-piercing sheathing screws. In all walls, the sheathing-to-framing screws were inserted at a distance of 1/2" (12.7 mm) away from the edge of each sheathing panel. The screw spacing (fastener schedule) varied between 3" (76.2 mm) and 6" (152.4 mm) for the different wall configurations as shown in Table 3.2, and all interior (field) fasteners were spaced at 12" (305 mm) on centre.

Seven different wall configurations were tested by the author. Six tests (3 monotonic and 3 reversed cyclic) were performed for all but one wall configuration (Group 14). An extra reversed cyclic test was carried out for series 14 because one of the walls (14D) was fabricated with a damaged panel. All components listed above remained constant throughout the testing of the different wall configurations. The only two factors that were not consistent from group to group were the wood sheathing type and the fastener schedule, as demonstrated in Table 3.2.

A complete set of details was recorded on individual test data sheets which can be found in an assembled document of test data and results *(Branston et al., 2004)*.

Table 3.2: Light gauge steel frame / wood panel shear wall test program matrix

Specimen ID	Loading Protocol[1,2]	Length of Wall (ft)	Height of Wall (ft)	Panel Type	Thickness of Panel (mm)	Fastener Schedule[3] (in.)
7 – A,B,C	Monotonic[1]	4	8	CSP	12.5	6/12
8 – A,B,C	CUREE[2]	4	8	CSP	12.5	6/12
9 – A,B,C	Monotonic	4	8	CSP	12.5	3/12
10 – A,B,C	CUREE	4	8	CSP	12.5	3/12
11 – A,B,C	Monotonic	4	8	DFP	12.5	6/12
12 – A,B,C	CUREE	4	8	DFP	12.5	6/12
13 – A,B,C	Monotonic	4	8	DFP	12.5	3/12
14 – A,B,C,D	CUREE	4	8	DFP	12.5	3/12
21 – A,B,C	Monotonic	4	8	OSB	11.0	6/12
22 – A,B,C	CUREE	4	8	OSB	11.0	6/12
23 – A,B,C	Monotonic	4	8	OSB	11.0	4/12
24 – A,B,C	CUREE	4	8	OSB	11.0	4/12
25 – A,B,C	Monotonic	4	8	OSB	11.0	3/12
26 – A,B,C	CUREE	4	8	OSB	11.0	3/12

[1]The monotonic testing protocol is explained in detail in Section 3.4.1

[2]The CUREE reversed cyclic protocol for ordinary ground motions is described in detail in Section 3.4.2

[3]Fastener schedule (*e.g.* 3"/12") refers to the spacing between sheathing to framing screws around the edge of the panel and along intermediate studs (field spacing), respectively.

1 foot (ft) = 305 mm

1 inch (in.) = 25.4 mm

3.3 SHEAR WALL FABRICATION, MATERIALS, AND COMPONENTS

Upon receiving all wood panels for the main testing program, the sheets were stacked after being separated with stickers in order to allow for air circulation (Figure 3.3). The panels were stored at room temperature in the laboratory for a period of time to enable the wood to reach an equilibrium moisture content (EMC) with the surroundings which, in turn, allowed for any dimensional changes to take place prior to assembly with the wall framing. Structural-use panels are usually fabricated to approximately 4 - 5 % moisture content. At an ambient temperature of 20°C and a relative humidity level of 85 % (typical of most buildings), the wood panel would reach an EMC level of approximately 15 % *(CWC, 2001, 2002)*. If the movement of the panel is inhibited by the fasteners to the metal framing, and if a substantial change in moisture content occurs, cracking or splitting could take place thereby reducing the capacity of the sheathing to framing connections. Since the overall capacity of the shear wall is highly dependent on these connections, a large change in moisture content could have a considerable influence on the performance of the system and would have to be addressed during design.

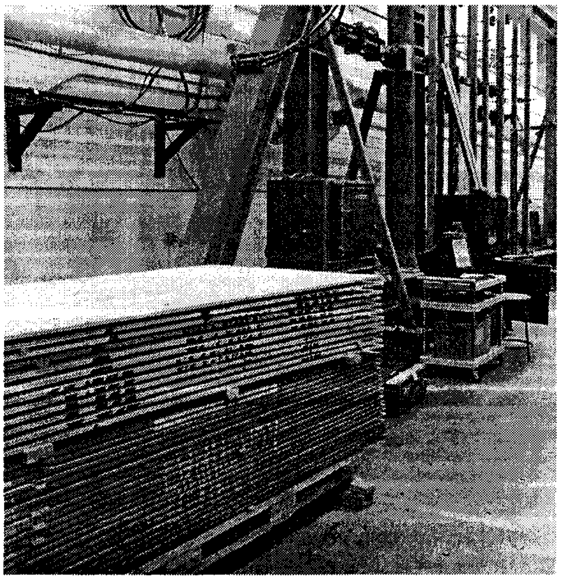

Figure 3.3: Storage of wood sheathing panels to allow for air circulation

Figure 3.4: Drilling of 4' (1220 mm) long light gauge steel tracks

All tracks (top and bottom) had holes drilled in them prior to fabrication of the shear walls. The bottom tracks were drilled to accommodate two shear anchors and two hold-downs. The holes for the shear anchors and hold-downs were drilled to 1/16" (1.6 mm) larger than the 3/4" (19.1 mm) ASTM A325 *(2002)* bolt and 7/8" (22.2 mm) ASTM A307 *(2003)* anchor bolt, respectively. The top tracks were drilled to accommodate six bolts which would transfer the load from the loading beam to the top track of the shear wall. ASTM A325 *(2002)* bolts, 3/4" (19.1 mm) in size, were used requiring all holes in the top track to be drilled to 13/16" (20.6 mm). The exact location for the holes in the top and bottom tracks of a 4' (1220 mm) long wall is demonstrated in Figure 3.16.

As previously mentioned, the back-to-back chord studs were connected using two No. 10-16 Hex washer head self-drilling screws at 12" (305 mm) on centre (Figure 3.5). The hold-downs (Figure 3.6) were also installed on the chord studs with 33 No. 10-16 Hex washer head screws. According to the manufacturer's literature *(Simpson, 2001)*, the average ultimate load for the S/HD10 hold-down is 29 000 lbs. (129 kN) and the allowable load for the hold-down connected to 18 gauge (1.12 mm) studs is 9 665 lbs. (43 kN).

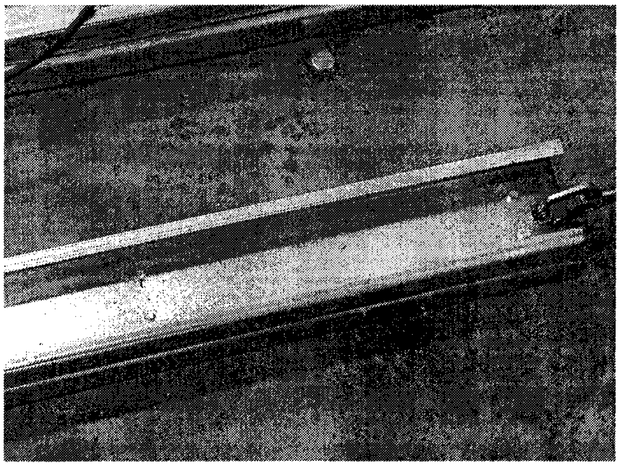

Figure 3.5: Back-to-back chord studs connected by two screws at 12" (305 mm) on centre

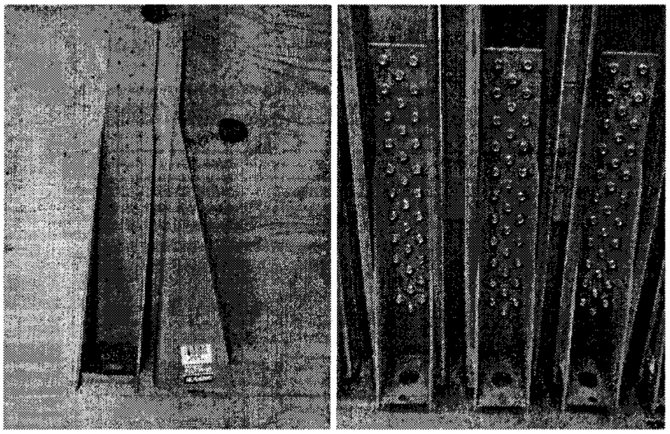

Figure 3.6: Industry standard Simpson Strong-Tie S/HD10 hold-downs *(Simpson, 2001)* used on back-to-back chord studs in all tests

In must be noted that all 43 tests carried out in this test program contained hold-down connectors on the chord studs. In contrast to the shear anchors in the bottom track that are installed to transfer the shear force to the foundation or subsequent storey, the hold-downs are in place to transfer the uplift force from the tension chord to the foundation or the storey below. If hold-down anchors had not been installed the tensile force from the chord would have to be transferred through the sheathing to the nearest shear anchor because the flexural capacity of the bottom track is not adequate to carry the resulting bending forces. The shear anchors would therefore not only take a shear component of the force, but also a tensile component. In addition, the overall capacity of the shear wall would be reduced because of the contribution of the sheathing and the connections between the sheathing and the framing in transferring the axial chord forces. The study of light gauge steel frame / wood panel shear walls without hold-down connectors is beyond the scope of this research and the test results and recommended design values apply only to shear walls containing hold-downs.

It is also important that hold-down connectors at the bottom corners of shear wall segments be installed correctly according to the manufacturer's literature. Improper installation of the connectors can lead to unwanted and unintended load transfer mechanisms such as above. It is important that the specified number of fasteners be used to connect the hold-down to the stud in order to prevent slip at that interface. It is also best to use a restrained anchor bolt nut so that cyclic loading does not cause the nut to become loose. A lack of nut tightening from the onset, as well as an accidental gap between the seat of the hold-down and the bottom track can also lead to excessive deformations in the whole shear wall system due to a lack of stiffness and unnecessary stud rotation. An overtightening of the hold-down nut is also not advised since it leads to increased wall stiffness, a concentration of demand on a small number of sheathing fasteners, and hence, a less ductile behaviour of the overall system *(Branston et al., 2003)*.

The two back-to-back chord studs along with the top and bottom pre-drilled tracks and the intermediate stud were assembled and connected on the ground in a horizontal position. All framing components were connected on one side (Figure 3.9), checked for squareness, turned over and connected on the other side and checked for squareness once again before installing the sheathing panel on the front side. One framing screw was installed at each track to stud interface (on each side). Once the light gauge steel framing was connected and square, the sheathing panel was laid on the front side of the framing and then fastened at a given fastener spacing (Table 3.2) and a 1/2" (12.7 mm) edge distance (Figure 3.10). The screw schedule for a 3"/12" (76.2 mm/305 mm) spacing is shown in Figure 3.7. The depth cone on the screw gun controlled the depth of penetration of the sheathing fasteners so that all screw heads were flush with the panel surface.

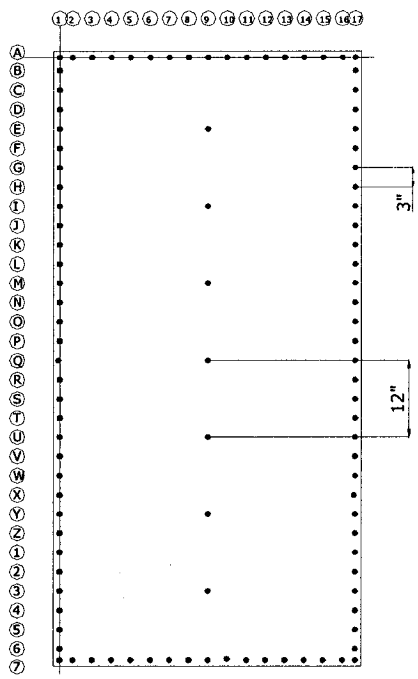

Figure 3.7: Screw schedule for 3”/12” (76.2 mm / 305 mm) spacing

In some cases a small pilot hole was pre-drilled through the wood sheathing and the steel studs and track to allow for ease of sheathing screw installation. This was done for all connections which engaged the wood sheathing and two thicknesses of light gauge steel (the track and the stud). This technique also kept the second layer of steel (stud flange) from bending away from the track flange during screw installation. In addition, for fastener schedules of 3” (76.2 mm) (Figure 3.7) and 4” (101.6 mm) around the perimeter, a No. 9 x 1” (25.4 mm) long bugle head screw was used at the connection location adjacent to each bottom corner instead of the usual No. 8 x 1-1/2” (38.1 mm) screw. For the fastener schedule shown in Figure 3.7, this corresponds to screw locations 7-2 and 7-16. This adjustment had to be made in order to ensure that the screw tip would not come into contact with the side of the hold-down once the screw penetrated the steel track. All fastener details are recorded on Test Data Sheets in Branston *et al. (2004)*. The four different types of screws used are shown in Figure 3.8.

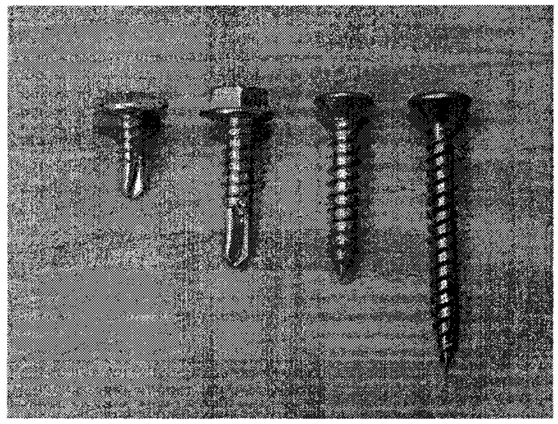

Figure 3.8: From left to right; No. 8 x 1/2" wafer head framing screw, No. 10-16 x 3/4" Hex washer head self-drilling screw, No. 9 x 1" bugle head self-piercing sheathing screw, No. 8 x 1-1/2" bugle head self-piercing sheathing screw (1" = 25.4 mm

Upon completion of the fabrication process, the moisture content of the wood panel was measured and recorded with an electronic moisture meter (Delmhorst Instrument Co. RDM-2 *(Delmhorst, 2003)*). Five readings at various locations on the panel were recorded for an average value for the whole panel. This procedure was done in order to ensure that the panel moisture content was not in excess of approximately 10 %. In addition to recording the moisture content, all relevant information from the grade stamps on the panel, as well as imperfections in the assembled wall were recorded on the Test Data Sheets and Test Observation Sheets. This information is included in a stand-alone test data and results document *(Branston et al., 2004)*.

Figure 3.9: Back-to-back chord stud and bottom track connection with framing screw and framing screw gun

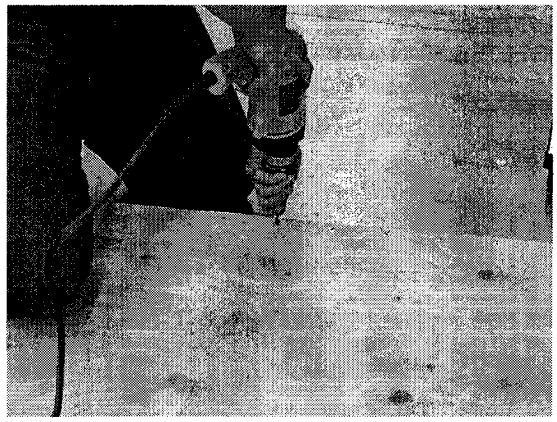

Figure 3.10: Sheathing-to-framing fastening utilising a screw gun with a depth-cone

As described in Section 3.2, three different types of wood sheathing (CSP, DFP and OSB) were used for the shear walls. These three types of sheathing, and their respective thicknesses, were deemed to be the most commonly used in today's construction industry. Example grade stamps for the three types of sheathing are shown in Figure 3.11.

After completing each test, the moisture content of the sheathing panel was also determined by APA Test Method P-6 *(APA PRP-108, 2001)*. Two specimens were cut using a 3" (76.2 mm) diameter hole saw at a distance of more than 2" (50.8 mm) away from any edge of the panel. The specimen weight was obtained immediately following the test (W_w) and then placed in a drying oven at approximately 200°F (93.3°C) for 24 hours. The oven-dry weight (W_d) was then obtained and the moisture content (MC) was determined according to Equation (3-1).

$$MC = \left(\frac{W_w - W_d}{W_d} \right) \times 100 \qquad (3\text{-}1)$$

where,

MC = moisture content of specimen (%)

W_w = initial weight (g)

W_d = oven-dry weight (g)

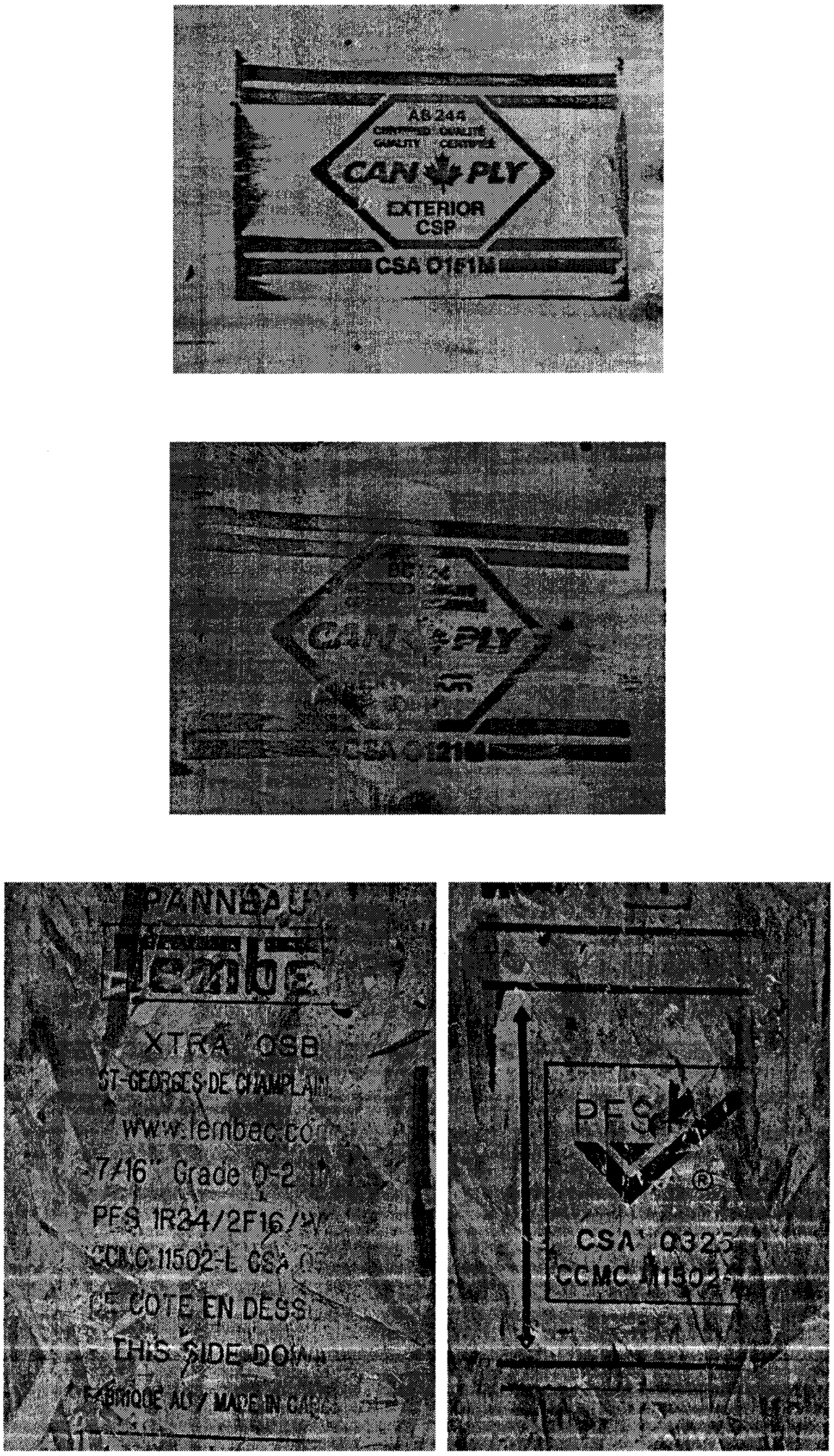

Figure 3.11: Grade stamps of sheathing panels for CSP, DFP and OSB (from top to bottom)

3.4 Test Set-Up and Protocols

Once the shear wall specimen had been fabricated, it was lifted vertically onto the test frame apparatus and slid into place under the loading beam. The height of the loading beam could be adjusted with chain blocks to allow for the insertion of the wall specimen from the side. Figure 3.12 shows a 4' x 8' (1220 mm x 2440 mm) shear wall specimen being loaded into the test frame.

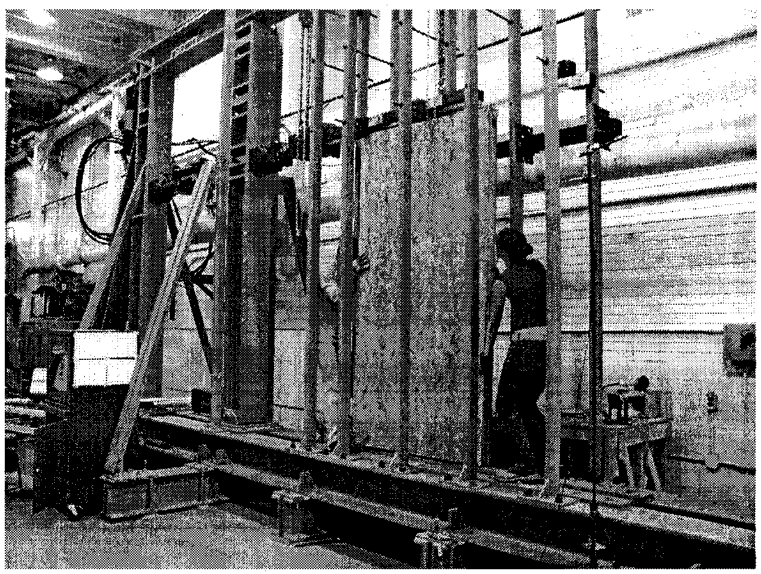

Figure 3.12: Wall specimen being loaded into the test frame from the right side

The wall specimen was aligned both vertically and in-line with the loading beam and load cell. With the shear wall specimen in place, all shear anchors, top bolts, and hold-down anchors were placed. The shear anchors and top bolts consisted of 6" (152.4 mm) long 3/4" (19.1 mm) ASTM A325 *(2002)* bolts. The shear anchors and top bolts were chosen to be 6" (152.4 mm) in length so that they would extend fully through the test frame and the loading beam apparatus (Figures 3.13 and 3.14). Load cells were also installed on the shear anchors and the hold-down rods. In total, two shear anchors were installed at the bottom of the shear wall to fasten the bottom of the wall to the base of the test fixture and six bolts were installed to connect the top of the wall to the loading beam (for a 4' (1220 mm) long wall). It was assumed that this connection would be adequate in order to transfer the shear load along the top of the wall. For all bolted connections (both at the top and bottom of the wall) a square (2.5" x 2.5" (63.5 x 63.5 mm)) steel plate washer 3/16" (4.8 mm) thick was used between the track and the bolt head or nut (Figures 3.13 and 3.15). The hold-down anchors consisted of two (one at each bottom corner of the shear wall) 250 mm long 7/8" (22.2 mm) ASTM A307 *(2003)* steel rods. A 1/2" (12.7 mm) steel plate was also incorporated into the hold-down connection under the base plate of the test fixture. This extra plate was used to stiffen the base plate and to limit any possible deformations which may occur in the test frame due to the high axial loads developed in the chords of the test specimen. A cross-section detail of the bottom shear connection as well as the top bolted connection is shown in Figure 3.13 and the bolts and anchor rods used are shown in Figure 3.14.

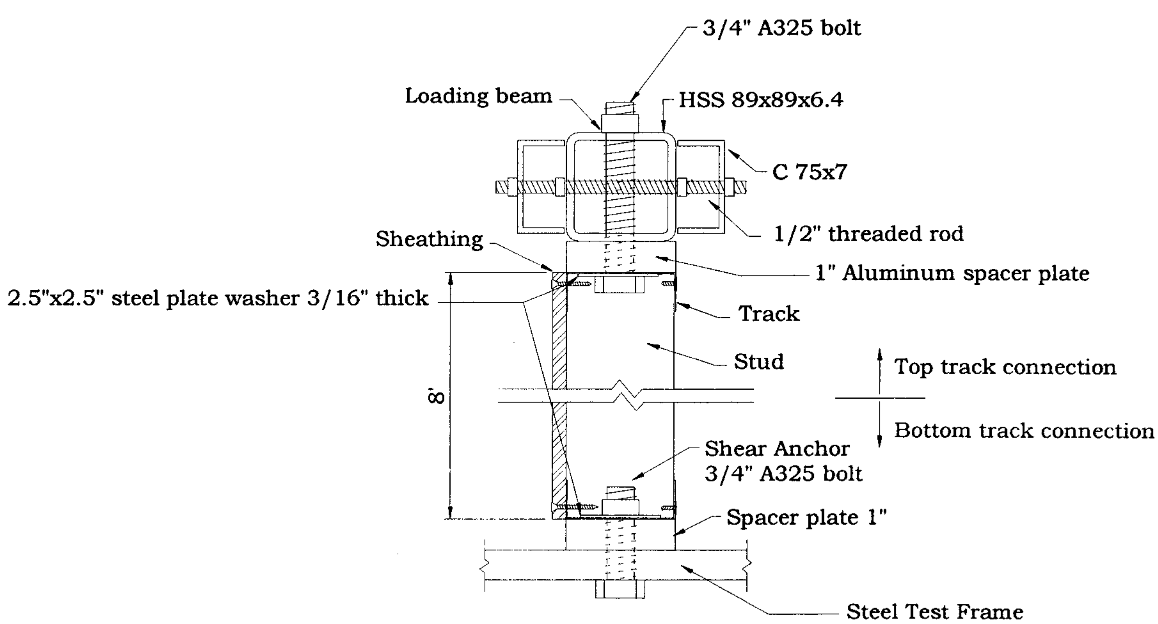

Figure 3.13: Detail of loading beam and its components as well as the top and bottom track connections

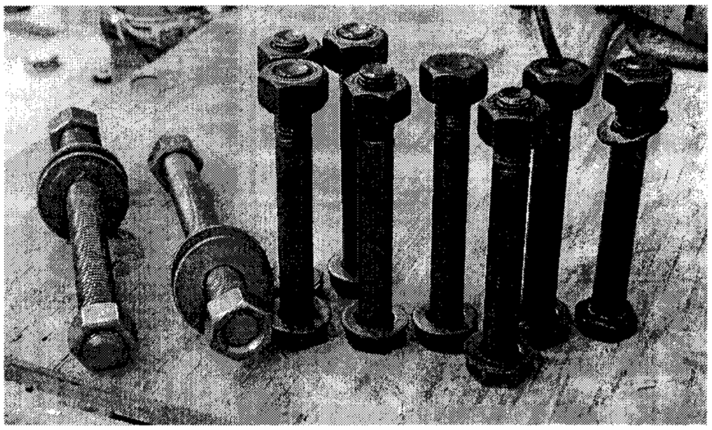

Figure 3.14: Shear anchors, top bolts, and hold-down anchors used to secure the shear wall test specimen to the testing apparatus

Using the readings from the load cells, the shear anchors were tensioned to approximately 80 kN axial load for all tests. All bolts were torqued using an electric impact wrench (Figure 3.15) with a capacity of 300 ft-lbs. (0.4 kN-m). The hold-down anchors were secured to finger tight and then turned an extra half-turn with a wrench, as suggested in the manufacturer's literature *(Simpson, 2001)*.

Figure 3.15: Tightening of top bolts (left) and view from underside of top track showing the square plate washer used in all bolted connections (right)

As illustrated in Figure 3.13, a 1" (25.4 mm) thick spacer plate above and below the wall allowed the sheathing to rotate or displace relative to the framing without disruption during loading. The spacer plates were 3-1/2" (89 mm) wide such that they only supported the steel framing of the wall. This detail allowed for measurement of the shear capacity of the wall assembly alone without the introduction of additional resistance due to the restraint of the sheathing.

The loading beam (Figure 3.13) consists of several components: firstly, a hollow structural section (HSS 89 x 89 x 6.4 mm) welded to a 1" (25.4 mm) thick end plate which, in turn, is connected to a swivel joint and then to the load cell. The 1" (25.4 mm) thick aluminum spacer plate is connected at its two ends with bolts to the HSS section. Two C75 x 7 sections are also connected on each side of the HSS section with 1/2" (12.7 mm) threaded rods. These C-sections act as spacers so that the lateral braces can provide guidance to limit the out-of-plane movement of the loading beam. At the locations where the lateral braces come in contact with the C-section spacers, Teflon pads are glued on the two surfaces to limit friction. The lateral braces (Figure 3.15) are also made up of HSS sections welded to threaded rods which connect to the test frame.

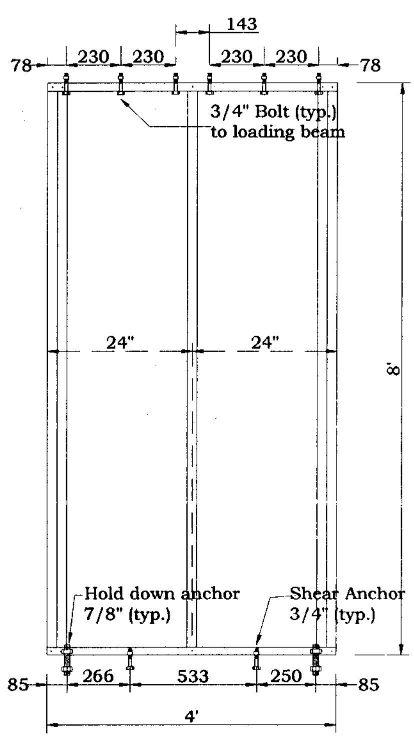

Figure 3.16: Anchorage to test frame for wall specimens

The arrangement of shear anchors, hold-downs, and top bolts for the 43 wall specimens is shown in Figure 3.16. The location of the shear anchors and hold-downs at the bottom of the wall was limited by the arrangement of pre-drilled holes in the test frame. This caused the layout to be non-symmetric with respect to the centre line of the wall. The holes in the loading beam and the top track were symmetric with respect to the centre line of the wall.

3.4.1 Monotonic Tests

All monotonic tests followed an identical test protocol to that used by Serrette *et al. (1996b)*. This protocol was used in order to simulate a "static" type loading such as often assumed to occur in the case of wind loads on a building. The unidirectional displacement at the top of the wall was constant at a rate of 7.5 mm per minute starting from the zero position. The zero position of the wall was defined as the point, after the wall had been installed and fully fastened to the test frame, where the load on the wall was close to zero. This means that slight adjustments had to be made to the position of the loading beam and actuator once all the bolts had been tightened and prior to the commencement of the test. The test continued until a sudden or significant drop in load carrying capacity was recognized.

In order to evaluate the permanent set at 12.5 mm (0.5% of wall height) and 38 mm (1.5% of wall height), the walls were unloaded to zero load once these displacements had been attained. A typical wall resistance vs. deflection curve for a monotonic test (Test 7A) is shown in Figure 3.17. It should be noted that Figure 3.17 represents corrected displacement values. The correction procedure is outlined in Section 3.6.

3.4.2 Reversed Cyclic Tests

The first four wall configurations (tests 1-A,B,C; 2; 3-A,B,C; 4-A,B,C) of the 109-wall test program were carried out in order to evaluate the feasibility of using the

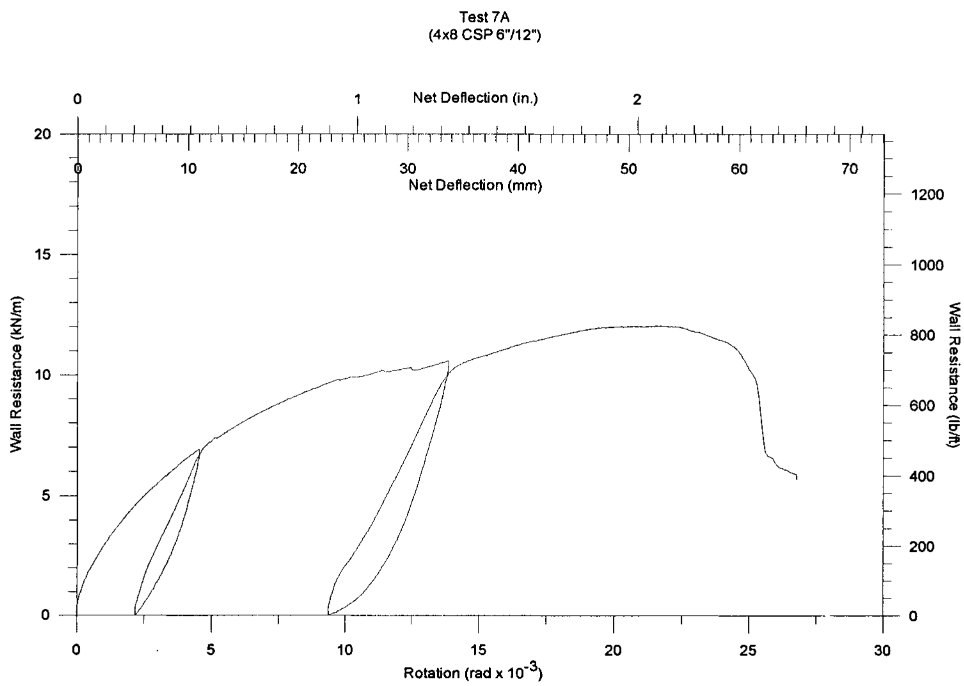

Figure 3.17: Typical wall resistance vs. deflection curve for a monotonic test

Sequential Phase Displacement (SPD) protocol *(SEAOSC, 1997)* or the CUREE (Consortium of Universities for Research in Earthquake Engineering) *(Krawinkler et al., 2000)* ordinary ground motions reversed cyclic loading protocols. These ten tests, as well as the task of determining which cyclic protocol would be used for the remainder of the cyclic tests, fall under the scope of the research performed by Boudreault *(2004)*.

The CUREE protocol for ordinary ground motions was chosen to be more suitable for the testing of steel frame / wood panel shear wall specimens. This protocol is based on the results of nonlinear dynamic time history analyses of structures constructed of wood frame shear walls, and hence, in comparison to the SPD protocol, is more representative of the expected demand to be imposed on this type of building component during an earthquake. The protocol was also developed with the notion that multiple earthquakes may occur during the lifetime of the structure and subjects components to ordinary ground motions (not near-fault) whose probability of exceedance in 50 years is 10 %. It should be noted that many data interpretation techniques (such as the equivalent energy elastic-plastic (EEEP) model presented in Chapter 4) are not dependant upon the loading protocol imposed on the specimen, however, the loading protocol can have a great effect on the design values obtained from the test results. This being said, it is important that the loading protocol reflects as much as possible the expected demand in a design level earthquake that may actually occur during the lifetime of a structure.

It should be stated that the loading history for the CUREE ordinary ground motions protocol is based upon the ultimate deformation capacity of monotonic tests. The monotonic deformation capacity, Δ_m, is a post-peak deflection defined as the position at which the wall resistance is reduced to 80 % of the maximum (peak) resistance (Figure 3.18). The above-mentioned displacement values are all based on corrected values (Section 3.6). In order to define the maximum deflection that the wall will sustain during a reversed cyclic test, a certain fraction (*i.e.* $\gamma\Delta_m$) of Δ_m is used as a reference deformation, Δ. The complete loading history is then based upon multiples of the reference deformation, Δ, which make up the initiation, primary, and trailing cycles. Throughout the loading protocol, one cycle (starting at zero displacement) is defined by an excursion to both a positive displacement and a negative displacement of equal value, finally returning to the zero position.

Small amplitude initiation cycles, typically well within the elastic range of the shear wall specimen (even though shear wall specimens display non-linear behaviour at very low amplitudes) make up the first six cycles of the loading protocol. During the time that the initiation cycles are being executed, the researcher can also check to make sure that everything is functioning correctly including the measuring devices and the data acquisition system. Subsequent to the initiation loading, the primary cycles are excursions to displacements that are larger than any of the preceding displacements, and hence usually cause the shear wall to enter into the non-linear range of behaviour. Trailing cycles follow the primary cycles and are defined by a displacement amplitude equal to 75 % of the amplitude of the preceding primary cycle. The loading history demonstrating the exact sequence of initiation, primary, and trailing cycles is defined in Table 3.3 and Figure 3.19.

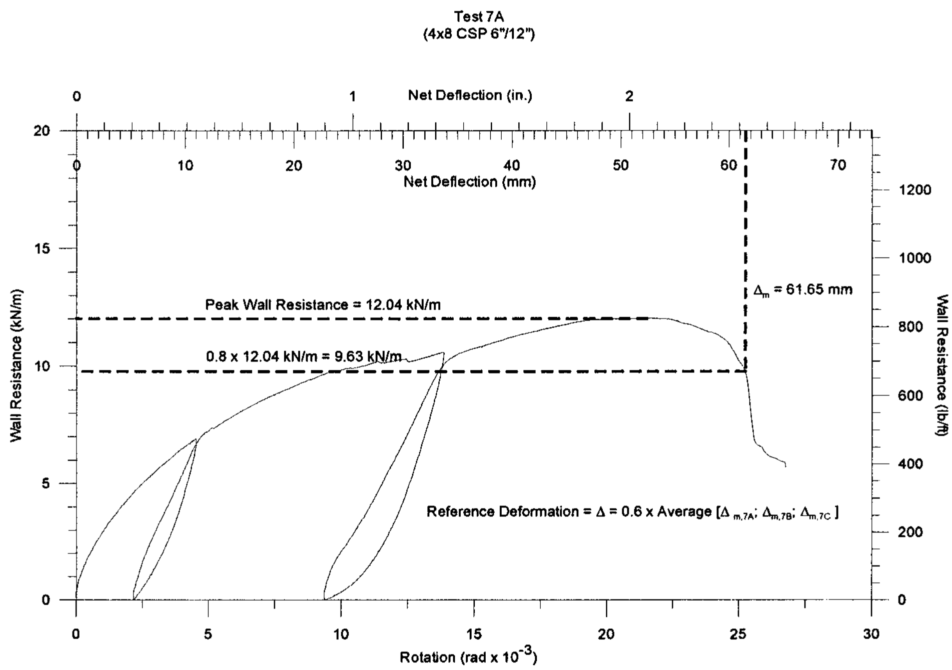

Figure 3.18: Determination of reference deformation (Δ) from monotonic test

Table 3.3: Displacement amplitudes following CUREE protocol for 4' x 8' (1220 x 2440 mm) shear wall tests 22-A,B,C with OSB sheathing and a screw schedule of 6"/12" (150/305 mm)

$\Delta = 0.6 * \Delta_m$	32.82	Screw Pattern:	6"/12"
		Sheathing:	OSB
	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	1.641	1.922	6
0.075 Δ	2.461	2.883	1
0.056 Δ	1.846	2.162	6
0.100 Δ	3.282	3.844	1
0.075 Δ	2.461	2.883	6
0.200 Δ	6.564	7.688	1
0.150 Δ	4.923	5.766	3
0.300 Δ	9.845	11.532	1
0.225 Δ	7.384	8.649	3
0.400 Δ	13.127	15.377	1
0.300 Δ	9.845	11.532	2
0.700 Δ	22.972	26.909	1
0.525 Δ	17.229	20.182	2
1.000 Δ	32.818	38.441	1
0.750 Δ	24.613	28.831	2
1.500 Δ	49.226	57.662	1
1.125 Δ	36.920	43.247	2
2.000 Δ	65.635	76.883	1
1.500 Δ	49.226	57.662	2

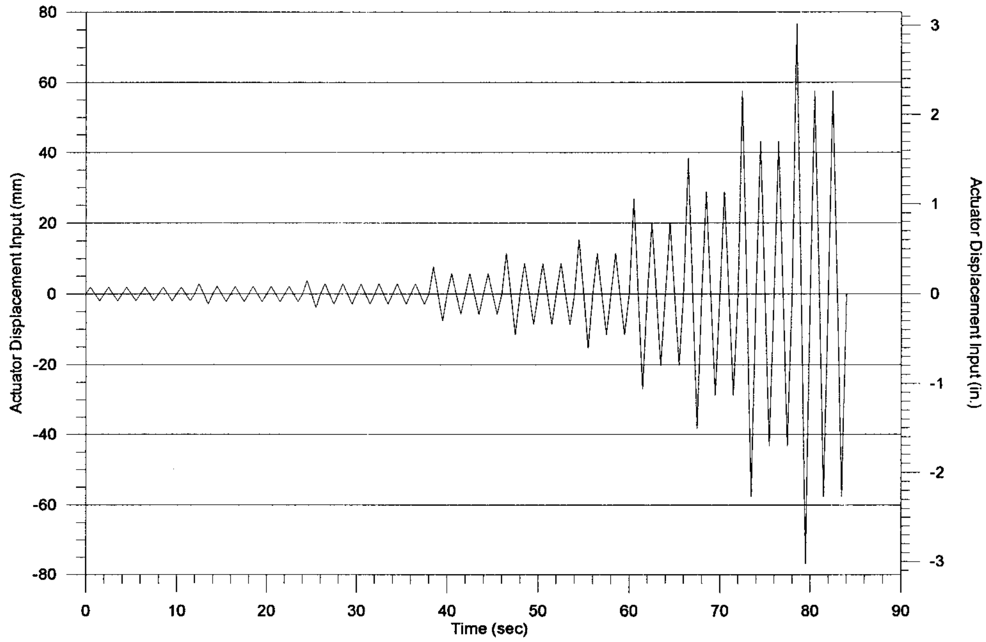

Figure 3.19: CUREE ordinary ground motions protocol for shear wall tests 22-A,B,C 4'x8' (1220 x 2440 mm) OSB 6"/12" (150/305 mm)

An average value of the monotonic deformation capacity (Δ_m) (maximum inelastic response) was found for each wall configuration from the results of the three nominally identical monotonic tests. This average value was then multiplied by γ = 0.6 to obtain Δ, the reference deformation, for the series of tests. The protocol for the three reversed cyclic tests with the same wall configuration was then based on the average reference displacement obtained, and each cycle with its target displacement, defined as a multiple of Δ, was determined. Because of corrections that are made to the test displacement data due to uplift and slip (Section 3.6) and because there always exists a small difference between the actuator input displacement and the actual top of wall displacement, it was deemed necessary to increase the actuator input displacements so that the wall specimen would be able to reach the target displacement (Table 3.3). From the monotonic tests, a target displacement could be matched with an actuator input displacement, and it would be this displacement which would be entered into the programmable computer system controlling the movement of the actuator. In all cases, a linear relationship existed between the actuator input displacement and the target displacement for the monotonic tests. The two variables were therefore plotted against each other and the slope of the line defined the multiplier used with the target displacement to obtain the actuator input displacement. Again, because three monotonic tests were conducted, an average value was used. This procedure was repeated for the seven different wall configurations under study in this thesis, which resulted in seven different CUREE reversed cyclic protocols (Appendix B).

In order to avoid excessive inertial effects due to the mass of the wall and certain components of the test frame such as the load cell, the frequency of the reversed cyclic tests following the CUREE ordinary ground motions protocol was kept at 0.5 Hz. For wall configuration 14, due to the limitations in the hydraulic oil supply and because the actuator input displacement was in excess of 100 mm, the protocol was slowed to 0.25 Hz for the last primary cycle and its trailing cycles (2.0Δ and 1.5Δ). As an example, the full protocol for wall configuration 22 (OSB 6"/12") (tests 22A, 22B, and 22C) is shown in Table 3.3 and Figure 3.19. It must be noted that a sine curve was used to connect the displacement amplitudes for the reversed cyclic protocol even though in Fig. 3.19 it appears as though straight line ramps were used. A typical reversed cyclic test corrected response curve is also shown in Figure 3.20.

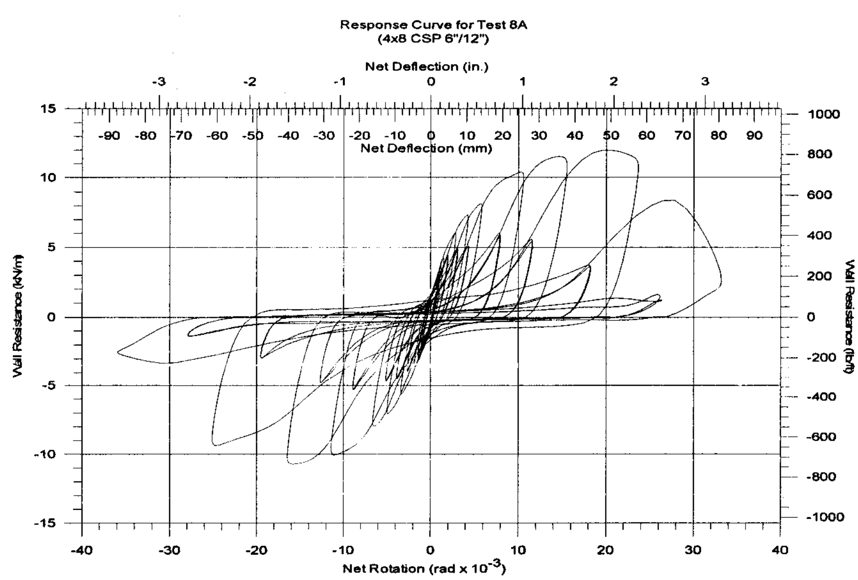

Figure 3.20: Typical wall resistance vs. deflection curve for a reversed cyclic test

3.5 Instrumentation and Data Acquisition

After the wall had been installed in the test frame, transducers (LVDT's) were attached to the test frame and the wall specimen such that all necessary displacements associated with the wall under load could be recorded. In total, twelve LVDT's and five load cells were used to monitor the performance of the 4' x 8' (1220 x 2440 mm) wall specimens.

Nine LVDT's (Figure 3.22) were connected directly to the wall specimen positioned to measure the slip at both ends of the wall, the uplift at both bottom corners of the wall, the top of wall in-plane lateral displacement, and the displacement of the sheathing relative to the light gauge steel framing (four LVDT's) (Figure 3.21). In addition to the nine LVDT's connected to the wall, the actuator contained a built-in LVDT which displayed readings of the actuator input displacements. Two more LVDT's were used to monitor the displacement of the lateral braces on both sides of the wall. These measurements were found to be negligible proving that the braces were stiff enough to restrict out-of-plane movement of the wall.

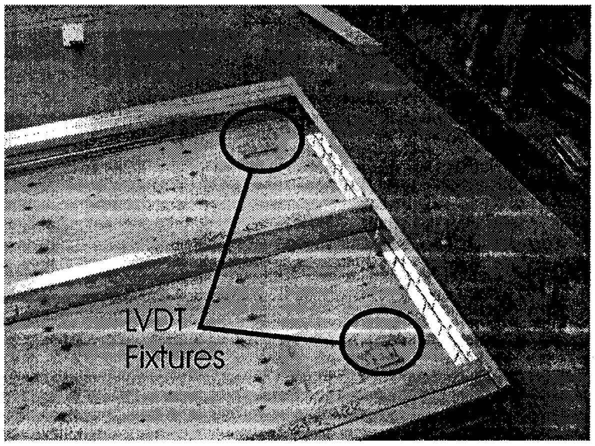

Figure 3.21: LVDT fixtures on the back of the wall specimen

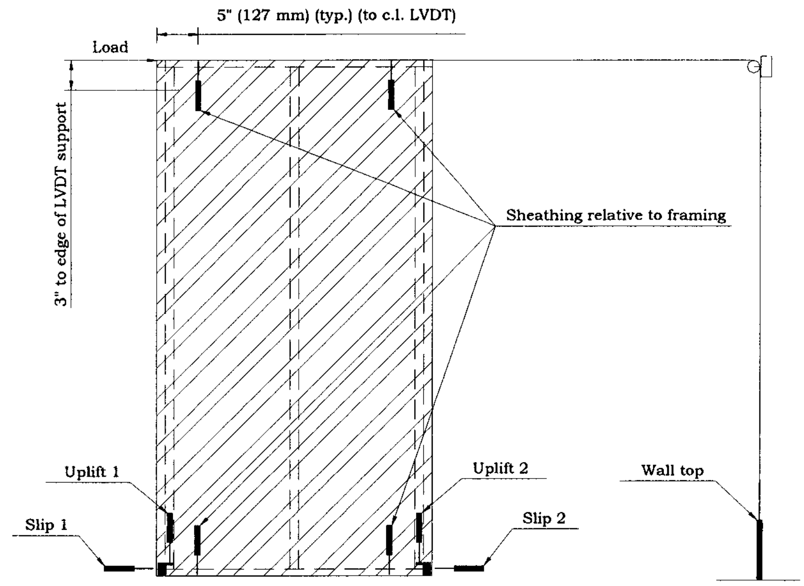

Figure 3.22: Positioning of nine LVDT's to measure and record wall displacements

The five load cells incorporated into the test set-up consisted of the main load cell connected to the loading beam (Figure 3.1), that measured the in-plane wall resistance, and four more load cells; two installed on the shear anchors and two installed on the hold-downs at the bottom of the wall. The shear anchor and hold-down load cells were used only for monitoring the axial load in the bolts when fastening the wall specimen to the test frame. During the test, the readings of the load cells were found to be erroneous and so the readings were not considered to be dependable.

An accelerometer was attached to the load cell assembly in order to measure the acceleration experienced by the load cell components and the top of the wall specimen during cyclic loading. The masses of the loading beam, the components connected to the loading beam, as well as the individual masses of the load cell and the swivel base of the load cell assembly were considered to add to the inertial effect on the wall. The weight of the top half of the wall was assumed to be negligible compared to the above-mentioned components.

All LVDT's were placed in a fashion as to measure the intended displacement as accurately as possible. One of the modifications that had to be made after reviewing the results of the preliminary testing program *(Branston et al., 2003)* dealt with the fashion in which the two LVDT's measuring uplift at each end of the wall would be positioned. For the data obtained from the preliminary testing program, a correction to the displacement values for rigid-body rotation using the uplift measurements (uplift 1, uplift 2) could not be made because it was found that the placement of the LVDT's caused the readings to be overly affected by the rotation of the wall, particularly the rotation of the chord studs themselves. They were not attached in a fashion which would allow them to measure the pure vertical movement of the wall at each end. It was decided that this attachment had to be modified for the main testing program.

In the preliminary testing program the uplift LVDT's were attached to the chord studs as shown in Figure 3.23. For the main testing program the uplift LVDT's were instead fixed on the test frame and the vertical movement of the chord stud was determined with the aid of a small seat angle screwed into the bottom track on the back side of the wall (Figure 3.23). This allowed for the vertical displacement of the chord stud to be measured directly without having the LVDT rotate with the movement of the wall.

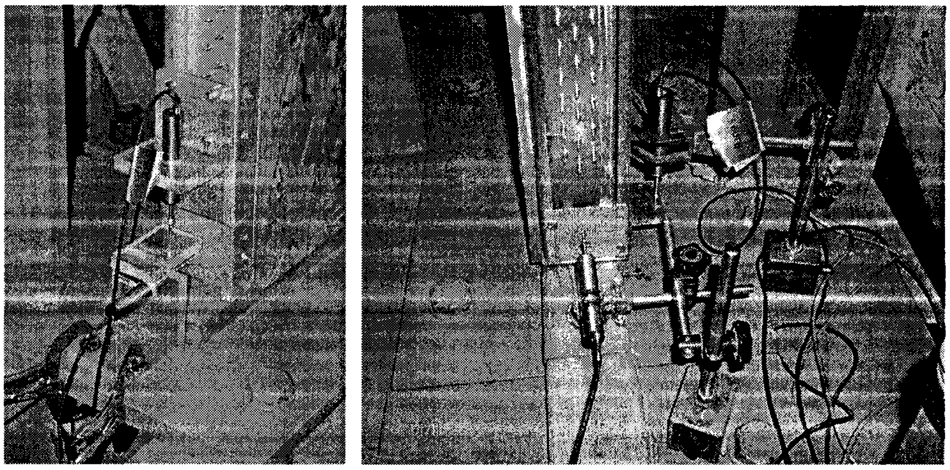

Figure 3.23: LVDT position for measuring uplift (upper LVDT) on the chord stud for preliminary testing program (left) and modification to attachment for main testing program (right)

Figure 3.23 also demonstrates the method by which the slip at both ends of the wall was measured with LVDT's. The configuration for this measurement remained unchanged from the preliminary testing program to the main testing program.

The top of wall lateral in-plane displacement was recorded using an LVDT with a range of ±100 mm. A steel piano wire was connected to the top corner of the shear wall specimen. This wire was also connected to the LVDT via a weight and pulley system as shown in Figures 3.22 and 3.24.

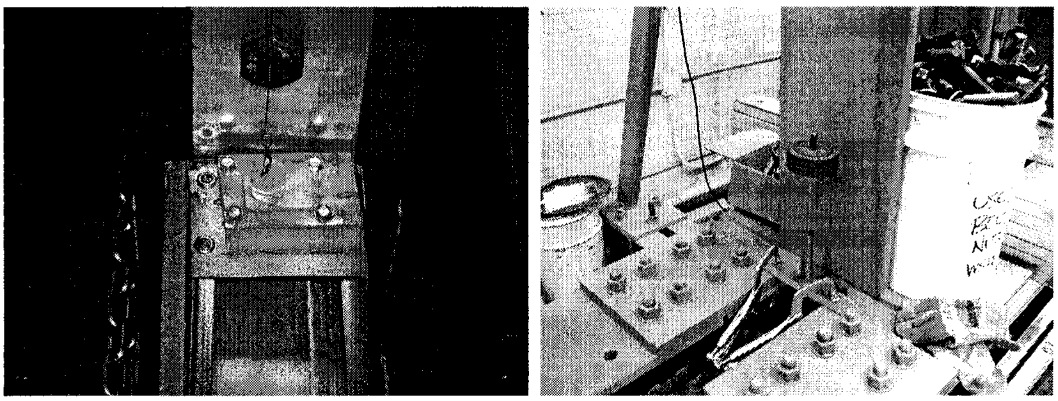

Figure 3.24: Steel wire attachment at top corner of wall (left) and attachment to LVDT at base of column (right)

All LVDT and load cells, as well as the accelerometer, were connected to Vishay Model 5100B scanners containing both high level and strain gauge cards. Data was recorded using the Vishay System 5000 StrainSmart software. For all monotonic tests, data was monitored at 50 scans per second and recorded at 2 scans per second, whereas for all reversed cyclic tests, data was both monitored and recorded at 50 scans per second.

3.6 DATA REDUCTION

Once the data had been obtained from the tests, some modifications had to be made to the top of the wall lateral displacement in order to obtain the "true" (net) deformations of the shear wall. The response curves for all 43 tests can be found in Branston *et al. (2004)*. The corrected response curves plot the wall resistance as a function of the net lateral displacement or net rotation of the wall. All top of wall displacement (for both monotonic and cyclic tests) results were corrected for slip at the base of the wall (rigid body translation) and uplift at both ends of the wall (rigid body rotation). An average of the slip measured at the two corners of the wall and the horizontal displacement caused by the rotation of the wall were subtracted from the total measured top of wall lateral displacement. Net deflection is defined by Equation (3-2) as:

$$\Delta_{net} = \Delta_{walltop} - \left[\left(\frac{\Delta_{baseslip1} + \Delta_{baseslip2}}{2}\right)\right] - \left[\left(\Delta_{uplift1} - \Delta_{uplift2}\right) \times \frac{H}{L}\right] \qquad (3\text{-}2)$$

where,

Δ_{net} = Net lateral in-plane displacement at the top of the wall

$\Delta_{wall\ top}$ = Total measured wall-top displacement

$\Delta_{base\ slip\ 1,2}$ = Measured slip at ends 1 and 2 of the wall specimen

$\Delta_{uplift\ 1,2}$ = Measured uplift at ends 1 and 2 of the wall specimen

H = Height of the wall specimen (8' = 2440 mm)

L = Length of the wall specimen (4' = 1220 mm)

and the rotation of the wall is defined by Equation (3-3) as:

$$\theta_{net} = \frac{\Delta_{net}}{H} \qquad (3\text{-}3)$$

where,

θ_{net} = Net rotation of the wall specimen, [radians]

Δ_{net} = as calculated using Eq. (3-2)

A loaded shear wall specimen in its deformed configuration is shown in Figure 3.25. It must be noted that the orientation of the arrows indicated on the figure for the various measured displacements, excluding rotations, denote a positive direction.

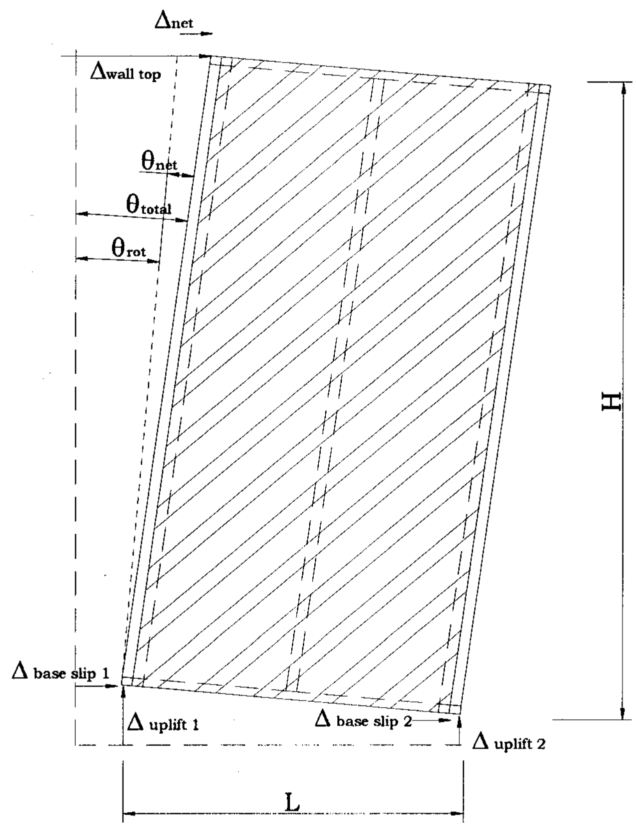

Figure 3.25: Shear wall specimen in its deformed configuration

Wall resistance, which was measured directly by the load cell in-line and in-plane with the top of the wall, is in most cases reported as a shear flow (shear force per unit length). In order to obtain the shear flow from the directly measured shear force, the following conversion is made with Equation (3-4):

$$S = \frac{F}{L} \tag{3-4}$$

where,

S = Wall resistance, [force per unit length]

F = In-plane lateral resistance measured by load cell, [force]

In the case of the reversed cyclic tests, acceleration was measured and recorded using an accelerometer attached to the load cell assembly and a correction to the lateral resistance obtained from the load cell was made for inertial effects (Equation (3-5)). A mass of 200 kg (1.96 kN), which included an allowance for the bolts, threaded rods, *etc.*, was used for the calculation of the force due to the inertial effect. For all data points, the inertial effect only slightly reduced the load measured by the load cell since the test protocol was executed at a reasonable frequency.

$$S' = S \pm \left(\frac{a \times g \times m}{1000 \times L} \right) \tag{3-5}$$

where,

S' = Wall resistance (corrected for inertia), [force per unit length]

S = Wall resistance as calculated by Eq. (3-4)

a = acceleration as measured by accelerometer, [g]

g = acceleration due to gravity (9.81 m/s^2)

m = mass (200 kg)

3.6.1 General Test Results

An overview of the direct results obtained from the 43 test specimens presented in Table 3.2 is shown in Tables 3.4 (monotonic tests) and 3.5 (reversed cyclic tests). Note that all corrections to displacement values and loads have been applied as described in the previous Section. A more detailed discussion of the interpretation of test data is presented in Chapter 4. The response curves for all tests along with wall resistance and displacement time histories for reversed cyclic tests can be found in Branston *et al. (2004)*.

Table 3.4: Test results for monotonic tests

Test	Panel Type	Fastener Schedule	Maximum Wall Resistance (S_u) kN/m	Displacement at $0.4S_u$ ($\Delta_{net,\,0.4u}$) mm	Displacement at S_u ($\Delta_{net,\,u}$) mm	Displacement at $0.8S_u$ ($\Delta_{net,\,0.8u}$) mm	Rotation at S_u ($\theta_{net,\,u}$) rad	Rotation at $0.8S_u$ ($\theta_{net,\,0.8u}$) rad	Energy Dissipation, E Joules
7A	CSP	6"/12"	12.0	5.7	52.9	61.7	0.0217	0.0253	706
7B	CSP	6"/12"	12.6	5.6	46.2	69.6	0.0189	0.0285	857
7C	CSP	6"/12"	13.6	6.6	52.9	70.2	0.0217	0.0288	912
AVERAGE	**CSP**	**6"/12"**	**12.7**	**6.0**	**50.7**	**67.1**	**0.0208**	**0.0275**	**825**
9A	CSP	3"/12"	27.2	9.1	64.6	78.0	0.0265	0.0320	1971
9B	CSP	3"/12"	23.5	9.4	58.4	64.1	0.0239	0.0263	1336
9C	CSP	3"/12"	24.7	9.2	59.9	68.0	0.0245	0.0279	1519
AVERAGE	**CSP**	**3"/12"**	**25.1**	**9.2**	**61.0**	**70.0**	**0.0250**	**0.0287**	**1609**
11A	DFP	6"/12"	15.8	7.8	57.5	74.0	0.0236	0.0303	1081
11B	DFP	6"/12"	16.9	7.3	56.7	70.9	0.0232	0.0291	1090
11C	DFP	6"/12"	15.3	7.3	50.1	64.0	0.0205	0.0262	910
AVERAGE	**DFP**	**6"/12"**	**16.0**	**7.5**	**54.8**	**69.6**	**0.0224**	**0.0285**	**1027**
13A[1]	DFP	3"/12"	28.0	9.0	56.2	60.6	0.0230	0.0248	1500
13B[1]	DFP	3"/12"	30.8	9.1	59.2	63.7	0.0242	0.0261	1663
13C[1]	DFP	3"/12"	30.4	9.3	59.4	63.6	0.0243	0.0261	1638
AVERAGE	**DFP**	**3"/12"**	**29.7**	**9.1**	**58.2**	**62.6**	**0.0239**	**0.0257**	**1600**
21A	OSB	6"/12"	13.4	3.4	39.2	54.9	0.0161	0.0225	747
21B	OSB	6"/12"	13.1	3.6	39.9	47.0	0.0164	0.0192	606
21C	OSB	6"/12"	13.3	3.9	44.2	62.4	0.0181	0.0256	830
AVERAGE	**OSB**	**6"/12"**	**13.2**	**3.7**	**41.1**	**54.7**	**0.0168**	**0.0224**	**727**
23A	OSB	4"/12"	19.1	5.0	38.3	51.8	0.0157	0.0212	977
23B	OSB	4"/12"	20.3	4.8	41.2	49.7	0.0169	0.0204	978
23C	OSB	4"/12"	18.5	3.9	39.2	46.9	0.0160	0.0192	860
AVERAGE	**OSB**	**4"/12"**	**19.3**	**4.6**	**39.5**	**49.5**	**0.0162**	**0.0203**	**938**
25A	OSB	3"/12"	23.7	5.1	38.0	42.6	0.0156	0.0175	932
25B	OSB	3"/12"	22.2	7.2	41.9	49.8	0.0172	0.0204	1016
25C	OSB	3"/12"	24.7	5.8	42.1	48.1	0.0173	0.0197	1110
AVERAGE	**OSB**	**3"/12"**	**23.5**	**6.0**	**40.7**	**46.8**	**0.0167**	**0.0192**	**1019**

[1]Tests 13-A,B,C governed by compression chord local buckling

Table 3.5: Test results for reversed cyclic tests

Test	Panel Type	Fastener Schedule	Maximum Wall Resistance (S'_{u+}) (positive cycle) kN/m	Displacement at S'_{u+} ($\Delta_{net, u+}$) mm	Rotation at S'_{u+} ($\theta_{net, u+}$) rad	Maximum Wall Resistance (S'_{u-}) (negative cycle) kN/m	Displacement at S'_{u-} ($\Delta_{net, u-}$) mm	Rotation at S'_{u-} ($\theta_{net, u-}$) rad	Energy Dissipation, E Joules
8A	CSP	6"/12"	12.0	50.1	0.0205	-10.7	-38.5	-0.0158	3977
8B	CSP	6"/12"	11.9	51.2	0.0210	-11.0	-38.8	-0.0159	3846
8C	CSP	6"/12"	11.8	50.4	0.0207	-10.2	-37.0	-0.0152	3848
AVERAGE	**CSP**	**6"/12"**	**11.9**	**50.6**	**0.0207**	**-10.6**	**-38.1**	**-0.0156**	**3890**
10A	CSP	3"/12"	26.1	54.8	0.0224	-23.3	-42.0	-0.0172	7012
10B	CSP	3"/12"	26.9	56.0	0.0229	-23.4	-43.1	-0.0177	7510
10C	CSP	3"/12"	25.5	51.5	0.0211	-22.6	-42.2	-0.0173	6314
AVERAGE	**CSP**	**3"/12"**	**26.2**	**54.1**	**0.0222**	**-23.1**	**-42.4**	**-0.0174**	**6946**
12A	DFP	6"/12"	13.5	49.7	0.0204	-12.2	-39.1	-0.0160	4216
12B	DFP	6"/12"	16.0	52.4	0.0215	-14.2	-40.1	-0.0164	4909
12C	DFP	6"/12"	14.4	52.3	0.0214	-13.7	-40.5	-0.0166	4348
AVERAGE	**DFP**	**6"/12"**	**14.6**	**51.5**	**0.0211**	**-13.4**	**-39.9**	**-0.0163**	**4491**
14A	DFP	3"/12"	31.0	54.9	0.0225	-27.9	-59.4	-0.0243	7948
14B	DFP	3"/12"	29.0	51.5	0.0211	-23.6	-36.9	-0.0151	6525
14C	DFP	3"/12"	29.5	55.1	0.0226	-27.6	-57.9	-0.0237	7998
14D	DFP	3"/12"	29.1	52.1	0.0214	-25.8	-54.1	-0.0222	7520
AVERAGE	**DFP**	**3"/12"**	**29.7**	**53.4**	**0.0219**	**-26.2**	**-52.1**	**-0.0213**	**7498**
22A	OSB	6"/12"	11.7	42.3	0.0174	-10.3	-29.1	-0.0119	3055
22B	OSB	6"/12"	11.9	44.0	0.0180	-10.8	-30.6	-0.0126	3338
22C	OSB	6"/12"	11.5	39.6	0.0162	-10.4	-30.7	-0.0126	2853
AVERAGE	**OSB**	**6"/12"**	**11.7**	**42.0**	**0.0172**	**-10.5**	**-30.2**	**-0.0124**	**3082**
24A	OSB	4"/12"	17.0	30.1	0.0123	-15.9	-27.4	-0.0112	3645
24B	OSB	4"/12"	17.4	44.4	0.0182	-15.5	-29.3	-0.0120	4224
24C	OSB	4"/12"	17.2	37.3	0.0153	-15.9	-28.6	-0.0117	3732
AVERAGE	**OSB**	**4"/12"**	**17.2**	**37.3**	**0.0153**	**-15.7**	**-28.4**	**-0.0116**	**3867**
26A	OSB	3"/12"	24.0	38.7	0.0159	-22.8	-26.4	-0.0108	4476
26B	OSB	3"/12"	22.6	29.5	0.0121	-21.7	-24.7	-0.0101	4201
26C	OSB	3"/12"	23.9	45.6	0.0187	-22.7	-42.9	-0.0176	5600
AVERAGE	**OSB**	**3"/12"**	**23.5**	**37.9**	**0.0155**	**-22.4**	**-31.3**	**-0.0128**	**4759**

3.6.2 Energy Dissipation

Energy dissipation values are shown in Tables 3.4 and 3.5 for both the monotonic and reversed cyclic tests respectively. Energy is defined as the component of force acting through a displacement in a given direction. When many displacements (data points) exist, it is possible to define the total or cumulative energy (*E*) dissipated by the wall specimen as the sum of all contributions of the product of a small increment in displacement and the average wall resistance (in kN). In terms of a plot of wall resistance (kN) versus net deflection (mm), the total dissipated energy (*E*, in Joules) during the test is represented by the area under the load-displacement curve. In design, the shear wall is considered as the sacrificial element (fuse) and is expected to dissipate the energy transferred to the building from the earthquake. A designer allows for a significant amount of the energy of the earthquake to be absorbed by this fuse element, while the remainder of the building retains its structural integrity to prevent loss of life.

For both monotonic and reversed cyclic tests, the data points were used to obtain the change in energy for each increment (Eq. (3-6)) as well as the total cumulative energy as defined by Equation (3-7):

$$\Delta E_i = \frac{F_i + F_{i-1}}{2} \times \left(\Delta_{net,i} - \Delta_{net,i-1}\right) \tag{3-6}$$

$$E = \sum \Delta E_i \tag{3-7}$$

where,

ΔE_i = Change in energy between data points *(i)* and *(i-1)*

$F_{i,\ i-1}$ = Wall resistance (corrected) at data points *(i)* and *(i-1)*, [force]

$\Delta_{net,\ i,\ i-1}$ = as calculated using Eq. (3-2)

E = Total cumulative energy

For monotonic tests, the wall was considered to have failed when the load diminished to 80 % of the peak load ($0.8F_{peak}$) reached during the test. The energy was therefore taken to be the area under the wall resistance (kN) versus net deflection curve up to the point of failure for all specimens. The calculation for energy dissipation (hatched area) in a monotonic wall specimen is depicted in Figure 3.26. The unloading portions at Δ = 12.5 mm and Δ = 38 mm of the wall resistance vs. net deflection curve were not considered in the energy calculation.

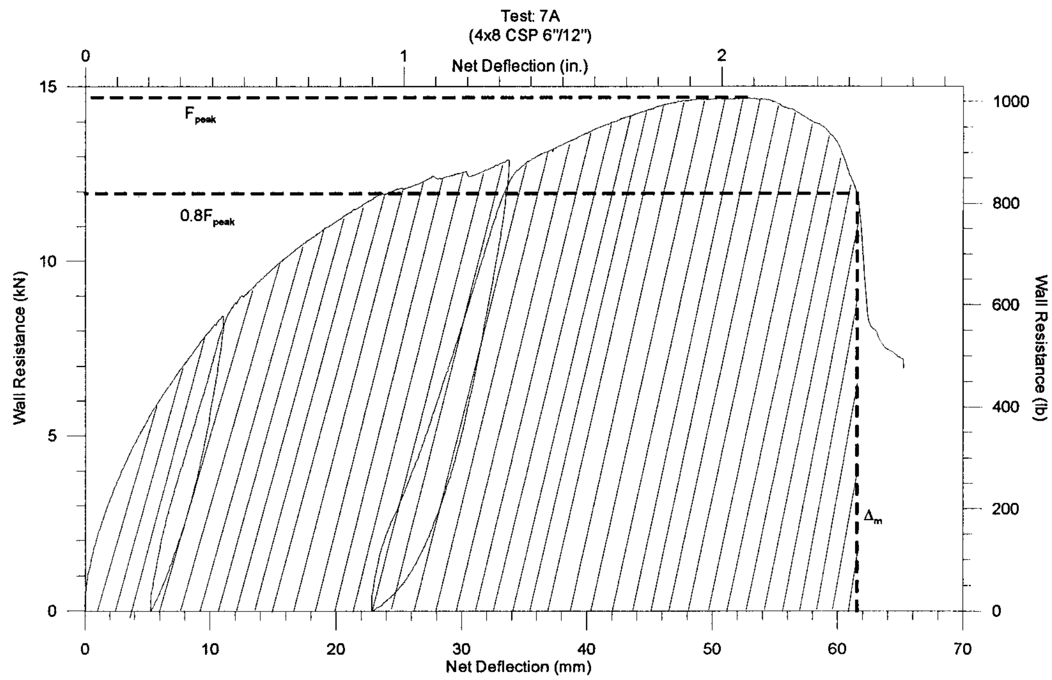

Figure 3.26: Energy dissipation for a monotonic shear wall specimen

For reversed cyclic tests, the energy dissipated by the wall specimen is defined as the area enclosed by the hysteretic loops. Equations (3-6) and (3-7) can be applied directly to calculate energy in a reversed cyclic wall specimen. For a given positive cycle, during the loading stage, because the i_{th} displacement is always larger than the $i\text{-}1_{th}$ displacement, the incremental energy will be a positive value. On the unloading arm of the cycle, the opposite will occur and so this incremental energy will subtract from the total cumulative energy and the remaining energy will represent only that portion enclosed by the hysteretic loop. A similar occurrence exists for the negative cycles. Energy time histories for all reversed cyclic tests are presented in Branston *et al. (2004)*.

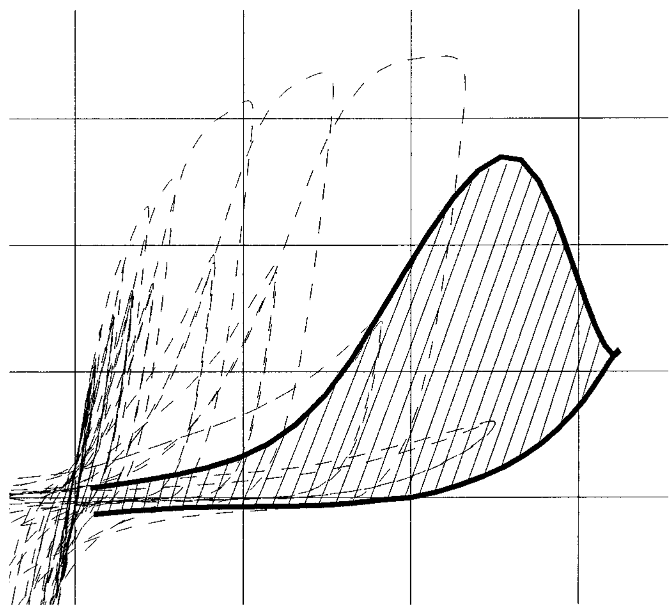

Figure 3.27: Energy dissipation for a reversed cyclic test is represented by the area enclosed by the hysteretic loops (single example loop shown in bold)

3.7 OBSERVED MODES OF FAILURE

In general, the failure of most wall specimens was due to the deterioration or the complete loss of the connection between the sheathing panel and the light gauge steel framing. The failure modes for the wood to steel connections can be classified into four main categories as follows:

1. Pull-through sheathing (PT) (Figures 3.28 and 3.32)
   Tilting of the screw during the monotonic tests and rocking of the screw during reversed cyclic tests resulted in the screw head being forced into the wood sheathing panel and the eventual pull-through of the screw head. When pull-through occurred, there was no significant damage to the edge of

the panel; because of the rocking and tilting, the screw hole became enlarged and allowed for the passage of the screw head.

2. Partial pull-through (PPT) (Figure 3.29)
   Partial pull-through occurred in the same manner as above, however, the screw head did not fully pull-through the sheathing. This condition was exhibited when the screw head remained embedded within the thickness of the panel. Further loading or cycling would have caused the screw head to fully pull-through the sheathing.

3. Tear-out of sheathing (TO) (Figure 3.30)
   This type of failure could only occur around the perimeter of the panel where the screws were installed at 1/2" (12.7 mm) from the edge of the panel. With significant loading during a test, because the sheathing moves independently of the steel framing, the sheathing panel may fail locally in bearing in the vicinity of the fastener. When this failure occurs, the screw head does not pull-through the screw hole, but instead tears out of the side of the panel.

4. Wood bearing failure (WB) (Figure 3.29)
   This failure type took place only in wall specimens that were sheathed with plywood. This condition is characterized by a ply or several plies failing while the others remained intact around the edge of the panel. Further loading or cycling would have probably led to a complete tear-out at the edge of the panel.

Combinations of the above-mentioned modes did also exist. In no case did a screw pull out of the flange of the steel studs or tracks. The hold-downs, the hold-down anchors, the shear anchors, and the tracks did not suffer any type of permanent damage nor did the steel to steel framing connections. In some cases where the sheathing screw penetrated through two layers of steel (at the track to stud connection location), a shear failure of the screw took place. This resulted in the screw being sheared with the screw head remaining in the sheathing and the shank of the screw remaining in the track / stud. This type of failure was mainly due to the two layers of steel providing more resistance to tilting of the screw. Because the screw remains perpendicular to the sheathing, it is subject to a much higher shear force. In the single steel layer case, when the screw tilts it is mainly loaded in direct tension, for which the screw material is much stronger than in shear. When the screws were subject to reversed cyclic loading, fatigue may also have added to deterioration in strength.

In general, the overall performance of the wall was governed by the sheathing to framing connections. A decline in capacity could be attributed to a complete side or the top or bottom of the panel being torn away or pulled away from the steel framing with the connections becoming no longer useful. The field fasteners (interior of panel) rarely exhibited any type of damage. At times, after failure had occurred over a large number of fasteners, the studs would act as small beams bending about the weak axis, to transfer the load from the closest useful connection to either the top or bottom track of the wall. As shown in Figure 3.33, since the bottom corner connections are no longer useful, the shear force must be transferred from connection 2-1 (Fig. 3.33) through the chord now acting in flexure about its weak axis (Figure 3.34). The shear force would eventually make its way to the shear anchor through the bottom track in compression. When this happened, local buckling would occur in the flanges and return lips of the studs. This usually took place after the peak load had been reached and hence was not considered to be a governing failure mode in the context of the laboratory tests. In reality, the loss of a group of sheathing connections could cause significant problems if a gravity load were in place. The compression chord studs would now be unsupported, and hence have a decreased compression capacity based on the unbraced length, as well as being subjected to a weak axis moment.

In the case of series 13, on the other hand, severe chord compression buckling/crushing took place, which governed the lateral capacity of the wall (Figure 3.31). This wall configuration is characterized by Douglas Fir Plywood (DFP) sheathing with a screw spacing of 3" (76.2 mm) around the perimeter of the panel. Because of the dense fastener schedule and the increased bearing resistance of DFP adding to the overall strength of the shear wall, large compression forces developed in the back-to-back chord studs, which caused their failure. End-chord compression buckling is an unfavourable governing failure mode for lateral force resisting shear walls because, in almost all cases, in addition to resisting a lateral load, the wall also supports gravity loads. The possibility exists that when the compression chord buckles, the wall system would no longer be able to support the gravity loads, which may lead to a possible collapse in part of the structure. This exceptional circumstance will be dealt with in Chapter 4: Interpretation of Test Results and Prescriptive Design, however, it should be noted that the matching reversed cyclic tests (14-A,B,C,D) did not experience compression chord failure, rather the sheathing connections controlled the behaviour as for all other wall configurations leading one to conclude that a possible rate of loading effect was present.

Test observation sheets for all 43 wall specimens, which contain failure modes for each fastener, can be found in Branston *et al. (2004)*. The failure modes are also illustrated in Figures 3.28 to 3.33.

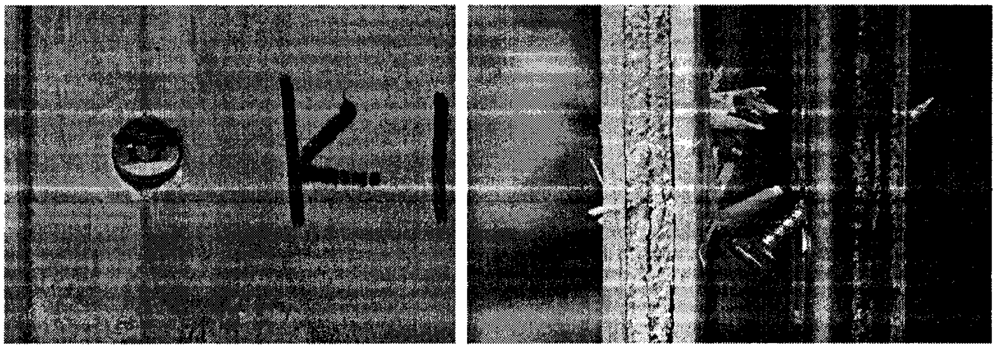

Figure 3.28: Tilting and rocking of the screw causes the head to become embedded in the sheathing panel (left); Pull-through failure (right)

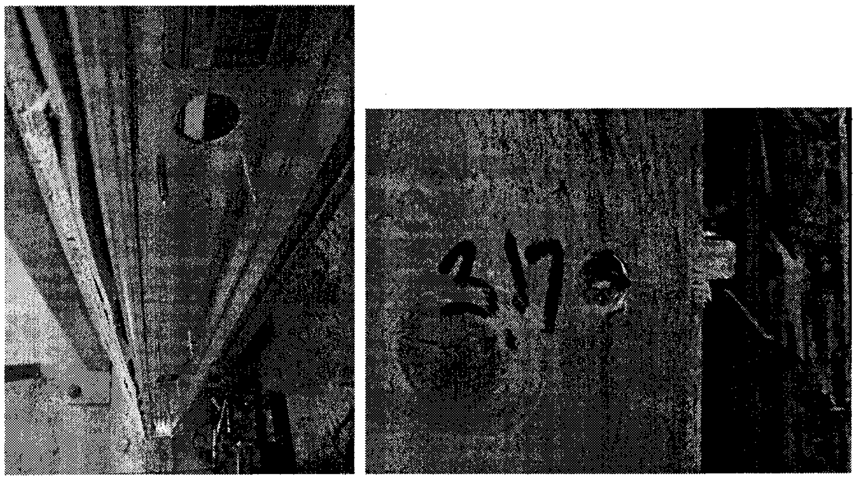

Figure 3.29: Partial pull-through failure of complete side of shear wall (left); Combination of pull-through and wood bearing failures (right)

Figure 3.30: Tear-out failure at corner of plywood panel (left); Tear-out failure at corner of OSB panel (right)

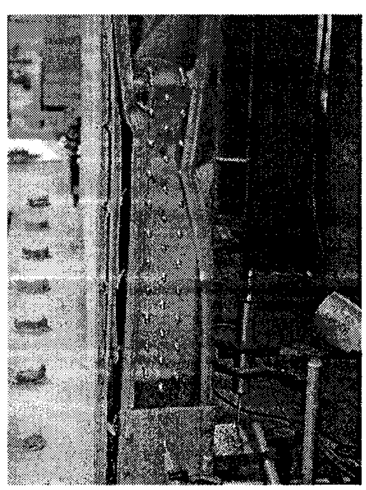

Figure 3.31: Compression chord buckling in Test 13B

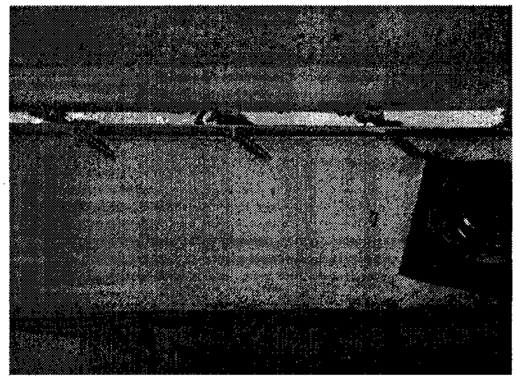

Figure 3.32: Screw tilting and pull-through along bottom track

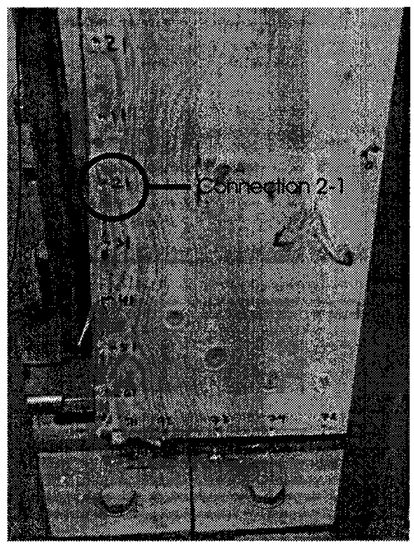

Figure 3.33: Failure of connections on bottom corner of wall

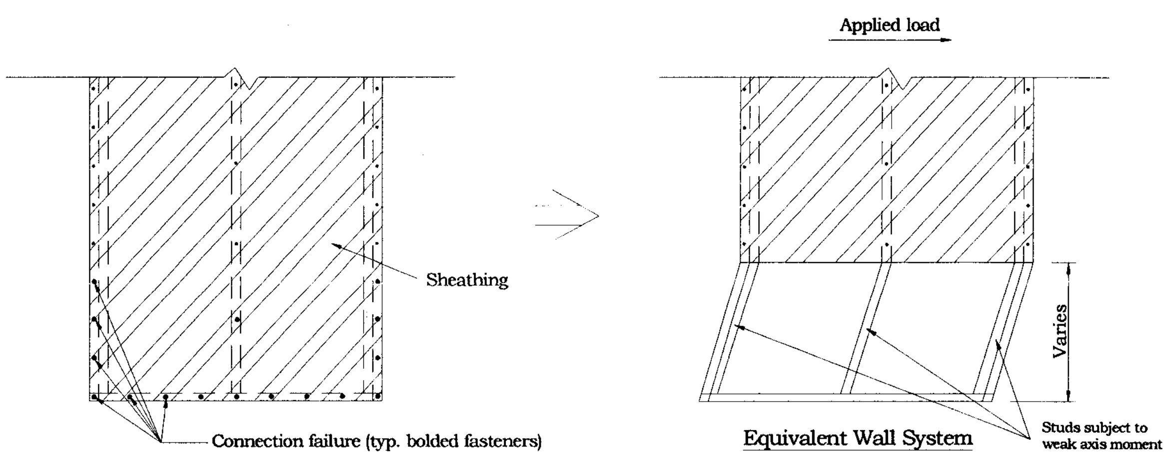

Figure 3.34: Loss of connection on bottom edge of wall

3.8 Ancillary Testing of Materials

As part of an ancillary testing program, the average material properties of both the light gauge steel studs and tracks and the sheathing panels used in the 43-wall test program were measured. In total, six steel coupons were tested according to ASTM A370 *(2002)* requirements. All steel coupon tension tests were conducted at a cross-head rate of 0.5 mm per minute in the elastic range, which was increased to a rate of 4 mm per minute beyond the yield point. Data was acquired at a rate of 2 scans per second for all specimens. Three replicate coupons were tested for both the ASTM A653 *(2002)* 1.12 mm thick steel studs and track. A 50 mm gauge length extensometer was used to measure the extension of the coupon, and hence, determine the respective Young's Modulus of the studs and track. All steel coupons were taken from the web of the respective components in the rolling direction. Upon completion of the coupon tests, the zinc coating on the steel was removed with a 25 % hydrochloric acid (HCl) solution to obtain the base metal thickness. This thickness was used to determine the material properties. As shown in Table 3.6, in all cases the measured yield strength exceeded the specified minimum strength (230 MPa) by a significant amount (8 – 21 %). The steel exhibited a sharp yielding behaviour with a yield plateau before strain hardening occurred prior to failure. The North American Specification for Cold-Formed Steel Members *(AISI, 2002b)* requires that $F_u/F_y \geq 1.08$ and that the elongation over a 50 mm gauge length be at least 10 %. All steel coupons tested met these requirements.

Table 3.6: Measured material properties of individual shear wall components

Coupon	Specimen	Member	Base Metal Thickness (mm)	Yield Stress (Fy - MPa)	Ultimate Stress (Fu - MPa)	Fu/Fy	Modulus of elasticity (E - MPa)	% Elong.
AVG	1.12mm 230 MPa	Stud	1.09	250.9	335.2	1.34	197667	38.5%
AVG	1.12mm 230 MPa	Track	1.08	272.1	343.7	1.26	203667	41.6%
Coupon	Specimen	Orientation	Specimen Thickness (mm)		Ultimate Shear Strength (MPa)		Shear Modulus (G - MPa)	Rigidity (N/mm)
AVG	12.5mm DFP		12.55		5.00		825	10371
AVG	12.5mm CSP		11.56		4.44		497	5738
AVG	11mm OSB		11.15		9.09		925	10303

The shear properties for all three wood sheathing panels are also reported in Table 3.6. Wood specimens were tested in shear following ASTM D1037 (edgewise shear) (1999). One half of the specimens were prepared with the long dimension parallel and the other half with the long dimension perpendicular to the long dimension of the 4' (1220 mm) x 8' (2440 mm) panel in order to evaluate the directional properties, however, it was found that the shear properties were not dependant upon the direction of testing. Average values are reported in the table, with full testing and specimen details found in Boudreault (2004).

Chapter 4 Interpretation of Test Results and Prescriptive Design

This chapter focuses primarily on the interpretation of the test data from the 43-specimen main testing program to obtain nominal strengths for use in a limit states design format, as is followed in Part 4 of the National Building Code of Canada *(NRCC, 1995, 2004)*. The general limit states formulation for ultimate loads and resistance, as well as the development of the resistance factor is detailed in Chapter 5. It should be noted that despite this chapter being limited to the 43 wall specimens that the author was solely responsible for, the data for all 109 specimens *(Boudreault, 2004; Branston et al., 2004; Chen, 2004)* included in the overall research investigation is analyzed in an identical manner to that described in Sections 4.3 and 4.4.

Over the years, design codes containing resistances and resistance equations for structural members have been developed based on test results. The specified strengths quoted in design codes are usually obtained from a lower percentile of the distribution of a strength random variable, *i.e.* ultimate strength *(Foschi, 2000)*. Design strength can also be based on the yield capacity of a test specimen. As an example, the tensile capacity of a structural steel tension member designed according to CSA S16.1 *(2001)* is a function of the yield strength of the gross cross-section. The failure mode that would result from yielding is usually ductile, which is much more favourable than a brittle failure mode, particularly in the case of seismic resistant design. The general resistance equation for gross cross-section yielding of a structural steel tension member is given by Equation (4-1). The member is designed so that the factored tensile resistance exceeds the factored load effects.

$$T_r = \Phi A_g F_y \tag{4-1}$$

where,

T_r = Factored tensile resistance of member, [force]

Φ = Resistance factor

A_g = Gross cross-sectional area of member

F_y = Yield stress of material

Equation (4-1) incorporates F_y, the yield stress or specified strength of steel, which for most sharp yielding steels is quite easily determined.

In the case of light frame shear walls, the specified or yield strength of the system is not as readily calculated or even identified. Laterally loaded wall systems exhibit a highly non-linear load-deflection response, and so the definition of a yield point is not apparent. For this reason, numerous data interpretation techniques to determine the yield point of a non-linear system, as well as several other design-related parameters, have been studied. Following this, the technique considered to be most appropriate was selected and then applied to interpret the data of the main shear wall testing program. The final design values are presented for the monotonic tests and both positive and negative cycles of the reversed cyclic tests for 43 wall specimens.

4.1 Existing Methods of Establishing Design Parameters from Test Results

Many data interpretation techniques exist in order to simplify the complicated nature of highly non-linear structural systems under testing conditions. These data interpretation techniques are utilized mainly to allow for design values to be deduced from test results. When testing shear walls subject to lateral load, the design values of interest include an equivalent elastic wall stiffness, yield wall resistance and ductility. Similar to the approach outlined by Equation (4-1), the yield wall resistance can be multiplied by a resistance factor to obtain a factored wall capacity for design. This procedure is detailed in Chapter 5. Wall stiffness is used in deflection calculations in order to verify that the service deflection and inelastic drift limitations set out by the respected building code are satisfied. System ductility is the main factor in determining the force modification factors (*R* factors) associated with seismic design *(Boudreault, 2004)*.

Because of the early onset of non-linear behaviour observed in both monotonic and reversed cyclic tests of the main testing program on light gauge steel frame / wood panel shear walls, it was decided that the most suitable data interpretation technique had to be selected in order to obtain reasonable design values. In order to complete this task, numerous techniques were explored and are outlined below. It should be noted that all of the data interpretation techniques described herein are applied to data that has already been corrected for uplift and slip (displacement) and for inertial effects (wall resistance – reversed cyclic tests). The recommendation presented in Section 4.3 is based on findings by Park *(1989)* and Foliente *(1996)* (Sections 4.1.10 and 4.1.11).

4.1.1 ICBO ES AC130 *(2002)*

These criteria were assembled by the International Conference of Building Officials Evaluation Service (ICBO ES) in order to provide guidelines for the acceptance of lateral loads for wood shear panels which differ from those configurations currently outlined in US model building codes. The US model building codes considered consist of the 1997 Uniform Building Code (UBC) *(ICBO, 1997)* and the 2000 International Building Code (IBC) *(ICC, 2000)*. Panels qualified under these criteria may also exceed the maximum allowable aspect ratios for wood frame shear walls given in the UBC and IBC, but these criteria do not address the issue of vertical (gravity) loads in combination with lateral loads on shear walls.

It is suggested that in order to comply with the acceptance criteria, reversed cyclic shear wall tests should be conducted following the sequential phase displacement (SPD) protocol *(SEAOSC, 1997)*, but, considered as an acceptable alternative by

ICBO, the CUREE ordinary ground motions protocol *(Krawinkler et al., 2000)* could be substituted. This data interpretation method would therefore apply to the reversed cyclic tests carried out in the main test program of this body of research.

The methodology outlined in AC130 is based on either a drift limit or an ultimate load limit. In general, the design load for the test sample is the lesser of the loads determined from the test data based on either the drift limit or the ultimate load limit.

<u>Drift limit</u>

The maximum inelastic response displacement (Δ_m) is defined as the displacement corresponding to the maximum wall resistance attained on the backbone curve ($\Delta_{net,u}$) of the test data but shall not be greater than the building code prescribed inelastic drift limit ($\Delta_{net,2.5\%}$). Using this maximum inelastic response displacement, the strength design level response displacement (Δ_y) is obtained by dividing the maximum inelastic response displacement by the seismic force modification factor (*R* factor). The force on the backbone curve corresponding to the design level displacement is defined as the strength level design resistance (S_y). Relating these guidelines to the 2.5 % of the storey height inelastic drift limit and the ductility related (R_d) and overstrength related (R_o) force modification factors in the upcoming edition of the National Building Code of Canada *(NRCC, 2004)* while being consistent with the variables already defined in this body of research, we obtain the following:

$$\Delta_m = \Delta_{net,2.5\%} \text{ or } \Delta_m = \Delta_{net,u}\text{, whichever is smaller} \tag{4-2}$$

$$\Delta_y = \frac{\Delta_m}{R_d R_o} \tag{4-3}$$

$$S_y = S_{@\Delta_y} \tag{4-4}$$

where all variables are as defined above.

Ultimate load limit

The acceptance criterion defines the ultimate load limit based on allowable stress design (ASD). The allowable load based on the tested shear wall is derived by dividing the ultimate wall resistance attained by a factor of safety of 2.0. For load and resistance factor design (LRFD), the strength level design resistance may not exceed the allowable load by a factor greater than 1.4.

This guideline was developed for acceptance of new configurations of wood frame / wood panel shear walls for which the force modification factors are already known. It would be difficult to utilize ICBO ES AC130 for light gauge steel frame / wood panel shear walls fabricated in Canada since the force modification factors have yet to be defined. The main goal in employing a data interpretation methodology is to determine the ductility related design parameters from the test data. In addition, ICBO ES AC130 does not provide guidance as to the determination of a wall stiffness to be used in deflection or drift calculations. It could be assumed, however, that a model elastic wall stiffness would be defined as the secant stiffness intersecting the origin and the design level displacement ordinate on the wall resistance vs. deflection backbone response curve.

4.1.2 Politehnica University of Timisoara, Romania *(ECCS, 1985; Fülöp and Dubina, 2002, 2003; Kawai et al., 1997)*

A total of 15 tests were carried out by Fülöp and Dubina on 12' (3600 mm) long light gauge steel frame shear walls sheathed with both corrugated sheet steel and OSB. The walls were tested under both monotonic and reversed cyclic protocols. For the cyclic tests both the initial and stabilized backbone curves were used for analysis. The results were analyzed using two methods in which the actual behaviour was modeled with an elastic-plastic bi-linear curve.

Method I (Figure 4.1)

Elastic stiffness (k_e) is defined as the secant stiffness through the origin of the wall resistance vs. displacement plot and the point corresponding to a load level of $0.4S_u$ ($S_{0.4u}$). This defines the elastic portion of the elastic-plastic bi-linear curve. A second straight line is then constructed tangent to the curve with a slope of $k_2 = 0.1k_e$. The intersection between the two lines defines the yield point, and the plastic portion of the curve is then extended horizontally from the yield point to the point of failure when the line intersects the downloading branch of the experimental curve. This definition of yield point is based on the European Convention for Constructional Steelwork (ECCS) *(1985)* recommendation.

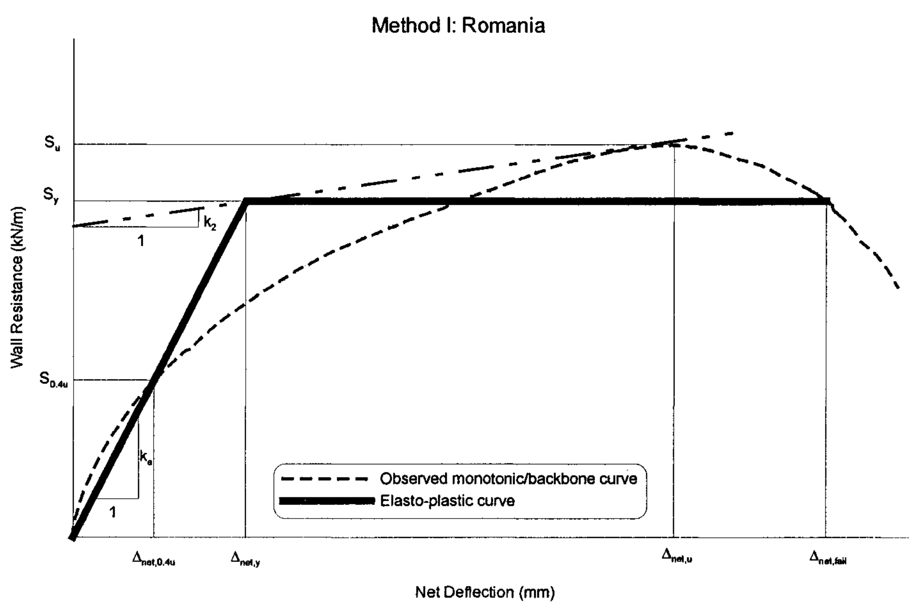

Figure 4.1: Method I, Romania for data interpretation
(ECCS, 1985; Fülöp and Dubina, 2002, 2003)

Method II (Figure 4.2)

This method was developed by Kawai *et al. (1997)*. The elastic stiffness (k_e) is taken as the secant stiffness through the origin and the point on the wall resistance vs. displacement plot corresponding to an interstorey drift of 1/400 (0.25 % of the storey height) ($\Delta_{net,0.25\%}$). The horizontal line depicting the plastic portion of the elastic-plastic curve is then adjusted so as to equate the areas under the experimental curve and the representative model (A_1 = A_2). Failure of the wall specimen is defined as the point of intersection between the horizontal plastic portion of the curve and the downloading branch of the experimental curve. For allowable stress design (ASD), the allowable strength is taken as the lesser of the wall resistance corresponding to an interstorey drift of 1/300 (0.33 %) ($S_{0.33\%}$) and 2/3 S_u.

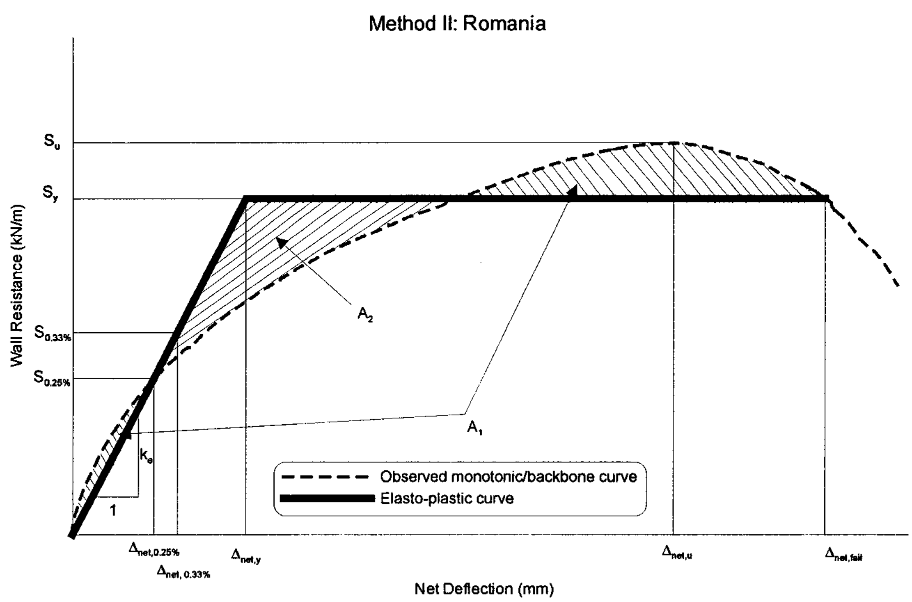

Figure 4.2: Method II, Romania for data interpretation
(Fülöp and Dubina, 2002, 2003; Kawai et al., 1997)

Fülöp and Dubina report that Method I tends to produce a lower yield limit with higher ductility values than Method II. Since both these methods are based on an equivalent elastic-plastic curve, the ductility related force modification factors as well as the yield wall resistance and initial or elastic stiffness are very easily defined from the data interpretation model.

4.1.3 Serrette: Phase I *(Serrette et al., 1996a, 1996b; Serrette, 1997)*

Serrette *et al. (1996b)* performed a total of 42 tests on light gauge steel frame / wood panel shear walls subject to both monotonic and reversed cyclic protocols as part of an applied research program sponsored by the American Iron and Steel Institute (AISI). The reversed cyclic tests were conducted according to the Sequential Phase Displacement (SPD) protocol (0.67 Hz) adopted by SEAOSC *(1997)*.

In terms of the interpretation of monotonic test data, modified final results were not presented. Maximum load capacity (S_u) was reported for each monotonic wall specimen, and this capacity was the ultimate wall resistance reached during the test. The corresponding displacement was also reported ($\Delta_{net,u}$). It is suggested, however, that wall stiffness and deflection should be addressed based on a limiting inelastic interstorey deflection. In terms of the 1994 Uniform Building Code (UBC) *(ICBO, 1994)*, the limiting inelastic drift may be obtained by amplifying an elastic displacement by $3R_w/8$. The elastic deflection can be taken as 1/200 (or 0.5 %) of the storey height. A comparison between the ultimate wall resistance obtained during the test and the nominal wall resistance at the limiting inelastic drift can then be made. The smaller of the two wall resistances would define a design maximum load. This method of interpreting monotonic test data does not, however, address the issue of wall yield resistance and so an "elastic" stiffness cannot be defined to ensure that elastic deflection limitations are satisfied. Since the design maximum load defined above could be significantly larger than the yield wall resistance as defined with, say, ICBO ES AC130 *(2002)* or the Romania Methods *(ECCS, 1985; Fülöp and Dubina, 2002, 2003; Kawai et al., 1997)*, the corresponding resistance factor would have to be significantly lower to obtain a comparable level of reliability / factor of safety for design.

In terms of reversed cyclic test data, the strength degradation associated with subjecting a shear wall specimen to several cycles at a given displacement was recognized. The strength degradation phenomenon was incorporated into the data interpretation technique by assigning the nominal load capacity of the specimen the value of the peak load on the lowest loop at the last set of stable hysteretic loops for both the positive and negative cycles of the protocol. Because the SPD protocol can subject the wall specimen to four cycles (in one direction) at a given displacement amplitude, significant strength (and stiffness) degradation can occur. The hysteretic loops were defined as unstable when the difference in wall resistance between consecutive cycles exceeds 5 % (Figure 4.3). The first excursion to a prescribed displacement was not included in the definition for stable hysteretic loops since the SPD protocol prescribes a series of degradation cycles following this first cycle.

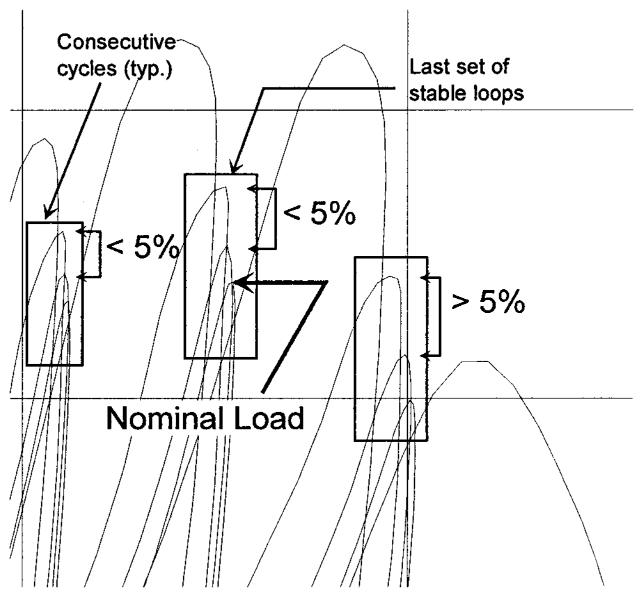

Figure 4.3: Serrette Phase I: Nominal load definition for SPD protocol *(Serrette et al., 1996b)*

Although this method is well defined for the SPD protocol, it does not truly apply to the CUREE Ordinary Ground Motions protocol used for the main testing program described in this body of research because of the lack of repeated cycles at a given displacement level (Table 3.3 and Figure 3.19). In addition, even though the nominal load is defined by the strength degradation characteristics, this data interpretation technique does not address the issue of deflection, or drift limitations, nor does it provide guidelines for defining a wall stiffness or any ductility related factors.

Serrette *(1997, 1998; Serrette et al., 1996a, 1996b)* recognized the need for a standardized method of deducing design values from test results and provided several alternative methods of doing so. For reversed cyclic tests, it is suggested to base a design wall resistance on a value which demarcates a change in wall stiffness (where hysteretic pinching becomes noticeable), or to consider the backbone curve as a static curve and use the methods described above for monotonic tests. In addition, Serrette indicated that a more detailed energy based approach is needed to properly characterize the behaviour and performance of light gauge steel frame / wood panel shear walls *(Serrette, 1998)*.

4.1.4 Serrette: Phase II *(Serrette et al., 1997a)*

In Phase II of the research performed by the Light Gauge Steel Research Group at Santa Clara University, 44 shear wall specimens were tested (28 cyclic tests, 16 monotonic tests). These tests were done in order to provide design values for alternative wall configurations not listed in the 1997 UBC *(ICBO, 1997)*, but to be included in the 2000 IBC *(ICC, 2000)*. Similar to Phase I *(Serrette et al., 1996b)* the SPD protocol was used for the reversed cyclic tests, however, at a higher frequency (1.0 Hz).

Phase II monotonic test data was interpreted as described above in Phase I of the research project. For the reversed cyclic test data, on the other hand, a slightly different data interpretation technique was used where the nominal load was based on the value of the peak wall resistance on the second loop (and not the last loop) at the last set of stable hysteretic loops (Figure 4.3). In employing this method, the same problems described above exist since the CUREE protocol was used for the main testing program of this research project.

4.1.5 Serrette: Phase III *(Serrette et al., 2002)*

Phase III of the research program taken on by the Light Gauge Steel Research Group consisted of a 20-specimen test program (10 reversed cyclic tests, 10 monotonic tests). These tests were performed, again, to expand on the suite of wall configurations provided in the U.S. model building codes (UBC, IBC) *(ICBO, 1997; ICC, 2000)* at the time.

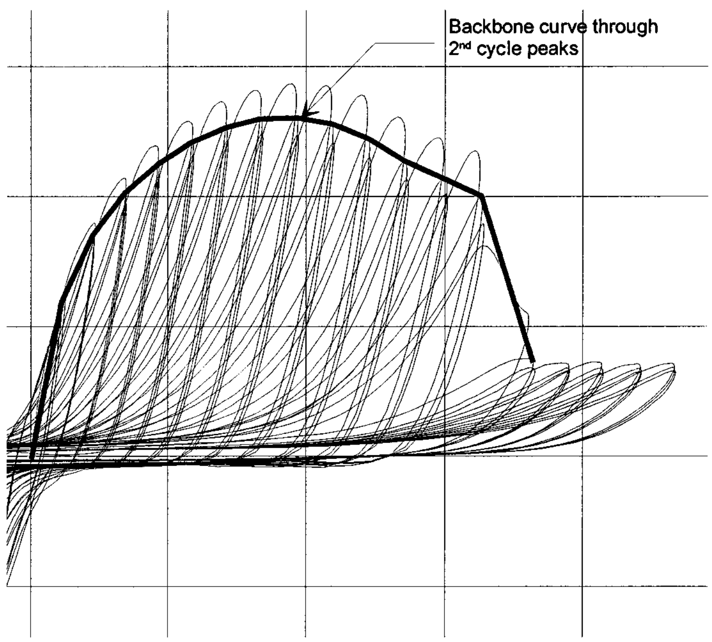

Figure 4.4: Second cycle backbone curve (positive cycles only) for SPD protocol without degradation cycles *(Serrette et al., 2002)*

The reversed cyclic protocol used for this series of tests differed from the SPD protocol used in Phase I and Phase II in that there were no degradation cycles following the primary excursion to each target displacement *(SEAOSC, 1997)*. The protocol therefore only consisted of three consecutive cycles at each target displacement with each subsequent series of cycles increasing in amplitude by a small amount. In order to interpret the cyclic test data, Serrette constructed a backbone curve through the peak wall resistance attained on each loop corresponding to the second cycle at each series of target displacement amplitudes (Figure 4.4). The maximum wall resistance attained on the second-cycle backbone curve was then reported.

4.1.6 CoLA / UCI Methods *(CoLA-UCI, 2001; Freund, 2001; Larsen, 2000; Shah, 2001; Smith, 2001)*

Following the 1994 Northridge earthquake (California, USA), the Federal Emergency Management Agency (FEMA) and the California Office of Emergency Services (OES) provided funding to the City of Los Angeles (CoLA) and the University of California at Irvine (UCI) to conduct and analyse a series of light gauge steel frame and wood frame shear wall tests. In total, 36 wall configurations were tested, with three wall specimens tested per group to explore the effects of dynamic or reversed cyclic loading on light-framed walls. The test program consisted only of reversed cyclic tests following the SPD protocol with degrading cycles *(SEAOSC, 1997)* at a frequency of 0.5 Hz. The reversed cyclic test protocol was similar to that used by Serrette in Phase I *(Serrette et al., 1996a, 1996b; Serrette, 1997)* (Section 4.1.3) and Phase II *(Serrette et al., 1997a)* (Section 4.1.4), however, at a slower frequency.

For the interpretation of the reversed cyclic tests, UCI generally followed the recommendations provided by SEAOSC *(1997)* and defined two backbone curves. The first backbone curve or envelope was traced through the peak loads at each displacement amplitude for the first excursion to a given displacement. The last backbone curve or stabilized curve was traced through the peak wall resistances on the loops which correspond to the last excursion to a given displacement amplitude (fourth cycle for the SPD protocol) (Figure 4.5). Using these two backbone curves, a bi-linear representation of the test data was then constructed for both the positive and negative cycles of the reversed cyclic test data, and this bi-linear representation demarcates two limit states. The yield limit state (YLS) is defined as the displacement and corresponding wall resistance level on the envelope curve where the load difference between the envelope curve and the stabilized curve is equal to 5 % (Figure 4.5). The wall resistance at the YLS is termed the nominal strength. The second limit state, the strength limit state (SLS), is defined as the peak wall resistance and corresponding wall displacement on the envelope curve. The bi-linear curve was constructed through the origin to the YLS and on to the SLS (Figure 4.5). This definition is consistent with the recommendations provided by SEAOSC *(1997)*.

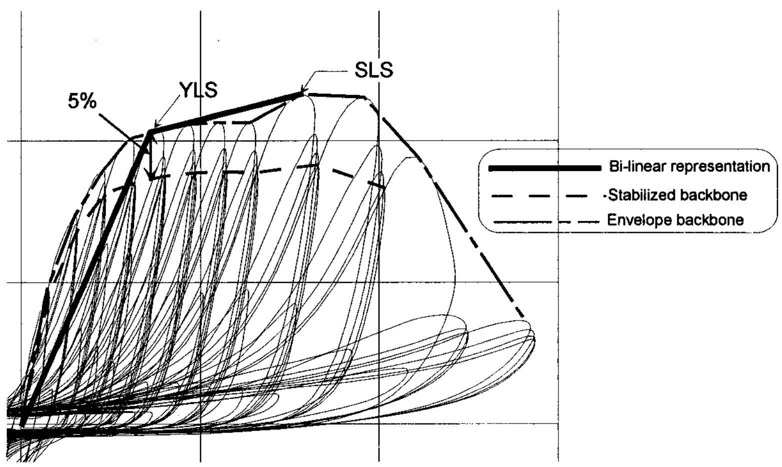

Figure 4.5: Definition of YLS and SLS for SPD protocol with degrading cycles

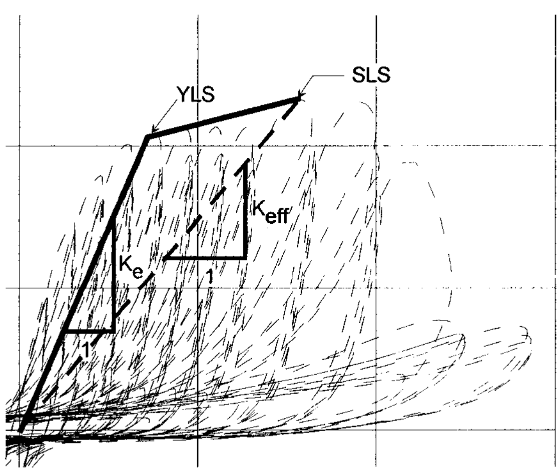

Figure 4.6: Definition of elastic and effective stiffness for UCI representation

Because the point where the wall resistance on the stabilized backbone curve equalled 95 % of that on the envelope curve rarely coincided with one of the existing data points, it was necessary to interpolate between data points to obtain the exact location of this occurrence. The various researchers on the CoLA-UCI project employed two methods to perform the interpolation. Some of the data interpretation was done using a linear interpolation between data points (Figure 4.5) *(CoLA-UCI, 2001; Freund, 2001)*, however, Larsen *(2000)*, Shah *(2001)*, and Smith *(2001)* made use of a quadratic polynomial fit between data points in order to obtain a better representation of the actual test data.

UCI considered wall stiffness in terms of an initial (elastic) stiffness as well as an effective stiffness. The initial stiffness is defined as the secant stiffness through the origin and the YLS (the slope of the elastic portion of the bi-linear representation), whereas the effective stiffness is taken as the secant stiffness through the origin and the SLS (Figure 4.6). Expressed as ratios, the initial and effective shear stiffnesses can be defined as follows:

$$K_e = \frac{F_{YLS}}{\Delta_{YLS}} \tag{4-5}$$

$$K_{eff} = \frac{F_{SLS}}{\Delta_{SLS}} \tag{4-6}$$

where,

K_e = Initial (elastic) shear stiffness, [force per unit length]

K_{eff} = Effective shear stiffness, [force per unit length]

F_{YLS} = Wall resistance at yield limit state (YLS), [force]

F_{SLS} = Wall resistance at strength limit state (SLS), [force]

Δ_{YLS} = Wall deflection at yield limit state (YLS)

Δ_{SLS} = Wall deflection at strength limit state (SLS)

In addition to defining a nominal wall resistance, an ultimate wall resistance, and an initial and effective shear stiffness, the CoLA / UCI research group also defines and reports numerous other design parameters of interest including ductility, drift capacity, and overstrength. Considering the wall to have failed when the ultimate wall resistance was reached, the ductility of the wall system was defined as a ratio of the wall deflection at the SLS to the wall deflection at the YLS ($\Delta_{SLS}/\ \Delta_{YLS}$). The drift capacity was calculated as the interstorey drift (Δ_{SLS}/H), where H is the height of the shear wall, and the overstrength factor was calculated as the ratio of the wall resistance at failure (SLS) to the wall resistance at yield (YLS) ($F_{SLS}/\ F_{YLS}$). The CoLA / UCI group did not modify their design values to account for an inelastic drift limitation prescribed by a respected building code, however, it should be noted that the drift capacities found for each wall group fell below the inelastic drift limitation of 2.5 % *(ICBO, 1997)*.

Research conducted by Shah *(2001)* and Smith *(2001)* also contained an investigation into other methods for determining the yield limit state of tested shear walls as well as other design parameters of interest. The other methods explored were the NAHB Method and the Dick Method and are reviewed in Sections 4.1.7 and 4.1.8, respectively.

4.1.7 NAHB Method *(Shah, 2001; Smith, 2001)*

This method was created by Jay Crandell of the National Association of Home Builders (NAHB) with the aid of research conducted as part of the CUREE-Caltech Woodframe Project. Crandell developed an empirical non-linear curve as a fit to load-deflection data from numerous wood frame shear wall tests. Crandell's empirical curve is defined as follows:

$$\Delta = 2.7\sqrt{a}\left(\frac{F}{F_u}\right)^{2.8}\left(\frac{H}{8}\right) \qquad (4\text{-}7)$$

where,

Δ = Lateral deflection of shear wall, [inches]

a = Shear wall aspect ratio, length/height

F = Force, [lbs.]

F_u = Ultimate shear wall capacity, [lbs.]

H = Height of shear wall, [feet]

Using equal energy principles, a bi-linear curve is constructed so that the area below this curve is equal to the area under the Crandell curve up to a displacement of $\Delta_{net,u}$, or, as shown in Figure 4.7, $A_1 = A_2$. The wall's shear yield resistance, F_y, is determined by dividing the ultimate wall resistance, F_u, by a factor of 1.5. The wall's shear yield displacement, $\beta\Delta_{net,y}$, is a fraction of the displacement ($\Delta_{net,y}$) on the Crandell plot corresponding to F_y. β is taken as 0.6. The slope of the elastic portion of the bi-linear curve denotes the elastic stiffness, K_e, and the semi-plastic portion of the bi-linear curve is constructed through the yield point and the point on the Crandell curve corresponding to the ultimate wall resistance.

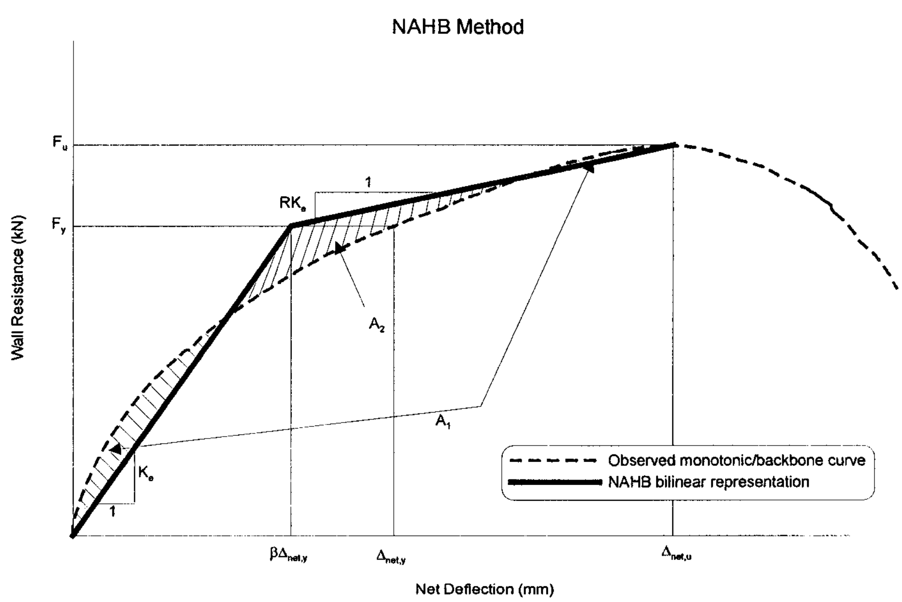

Figure 4.7: NAHB method to determine yield wall resistance *(Smith, 2001)*

As an alternative to using the Crandell curve to represent the actual test data, the observed monotonic and reversed cyclic backbone curves can be used in the analysis *(Smith, 2001)*. In performing this analysis however, even though the design parameters of interest can be easily derived, *i.e.* yield wall resistance, ultimate wall resistance, elastic stiffness, ductility, *etc.*, the yield resistance is simply determined by dividing the ultimate wall resistance by a factor of 1.5. In addition, if the actual test data is used instead of the Crandell approximation, then the β factor would have to be adjusted so that the equal energy concept is maintained.

4.1.8 Dick Method *(Smith, 2001)*

This method was suggested by Graeme Dick, Chair of the SEAOSC Test Committee 1998 – 2001. The purpose of this method is to provide a conservative estimate of the yield wall resistance of a shear wall when used in conjunction with the CoLA / UCI SEAOSC *(CoLA-UCI, 2001; Freund, 2001; Larsen, 2000; Shah, 2001; Smith, 2001)* method described above. In general, both methods would be applied to either a monotonic or reversed cyclic wall resistance vs. displacement test data plot, and the lower yield capacity would denote the yield wall resistance of the shear wall configuration.

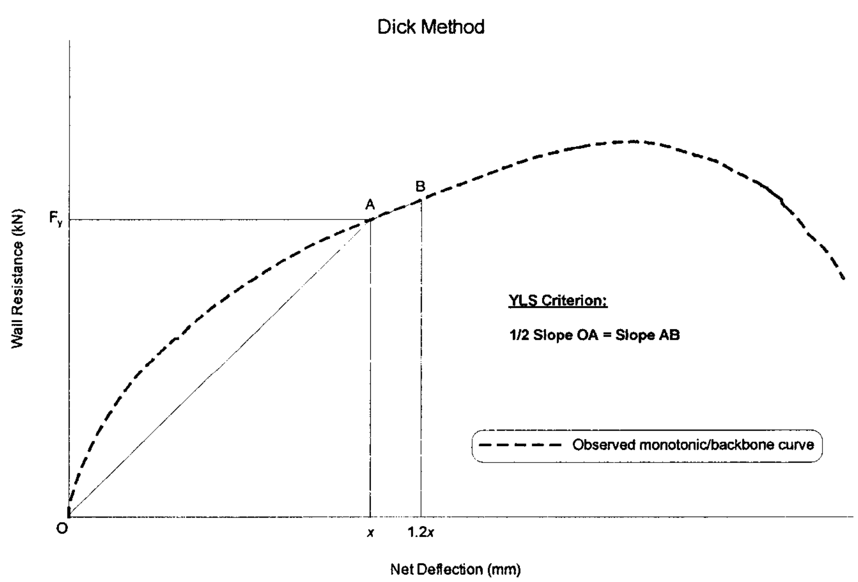

Figure 4.8: Dick method to determine yield wall resistance *(Smith, 2001)*

The Dick method makes use of the slope (stiffness) of the monotonic or backbone test curve and defines the yield limit state as the point on the curve at which the slope satisfies a certain criteria as shown in Figure 4.8. The process begins by assuming the YLS to be at a displacement x (Point A). Another point (Point B) is chosen such that the value of the displacement at Point B is 1.2 times greater than the value of the displacement at Point A ($1.2x$). At this point, both slopes OA and AB can be found. This can be done for various displacement values along the curve and the YLS is determined (as Point A) once the criterion is satisfied (1/2 slope OA = slope AB).

The Dick method and the CoLA / UCI method complement each other well since the Dick method accounts for stiffness degradation while the CoLA / UCI method accounts for strength degradation. When used in conjunction, the lower of the two values would lead to a conservative design accounting for the most important characteristics of reversed cyclic testing.

Although not explicitly stated, it can be assumed that the same definition would exist for the SLS *(CoLA-UCI, 2001; Freund, 2001; Larsen, 2000; Shah, 2001; Smith, 2001)*, and therefore an initial (elastic) and effective shear wall stiffness could be defined. Other design parameters such as ductility, drift capacity, and overstrength could also be found as described in Section 4.1.6.

Despite the conservativeness of the method in considering the strength and stiffness degradation associated with light frame shear walls subject to reversed cyclic loading it was originally intended to be used in conjunction with the SEAOSC SPD protocol *(1997)* and data analysis procedure *(CoLA-UCI, 2001; Freund, 2001; Larsen, 2000; Shah, 2001; Smith, 2001)* which does not apply to test data obtained while making use of the CUREE protocol.

4.1.9 ISO Method *(ISO 16670, 2000)*

This Standard provides a procedure for the cyclic testing of timber joints in order to create a reasonable representation of actual earthquake demand imposed on structures. The cyclic test protocol is developed from the monotonic test curve, where the displacement amplitudes are fractions of the ultimate displacement obtained during the monotonic test ($\Delta_{net,fail}$). ISO 16670 defines the ultimate displacement as the displacement when the specimen fractures ($\Delta_{net,fail\ case\ a}$) (case a), or the displacement at a load level of 80 % (post-peak) of ultimate ($\Delta_{net,fail\ case\ b} = \Delta_{net,0.8u}$) (case b), whichever occurs first (Figure 4.9). The reversed cyclic test protocol subjects the wall to three consecutive cycles at a given displacement before increasing the displacement amplitude (no decreasing cycles exist).

This Standard does not provide guidance as to the interpretation of the test data to obtain design parameters such as yield resistance, ductility, *etc.*, and so would not be of use in this research program. Guidance is provided in order to construct the first, second, and third backbone curves through each of the peak wall resistances reached on the first, second, and third excursion to a given displacement amplitude and it is stated that other data interpretation techniques may be used to determine the parameters of interest. The concept of first, second, and third backbone curves is, however, not applicable with the CUREE protocol since these consecutive cycles do not exist.

In order to interpret the data for stiffness, the following formulation is given:

$$K = \frac{0.3 \times F_u}{\Delta_{net,0.4u} - \Delta_{net,0.1u}} \qquad (4\text{-}8)$$

where the parameters are as defined in Figure 4.9.

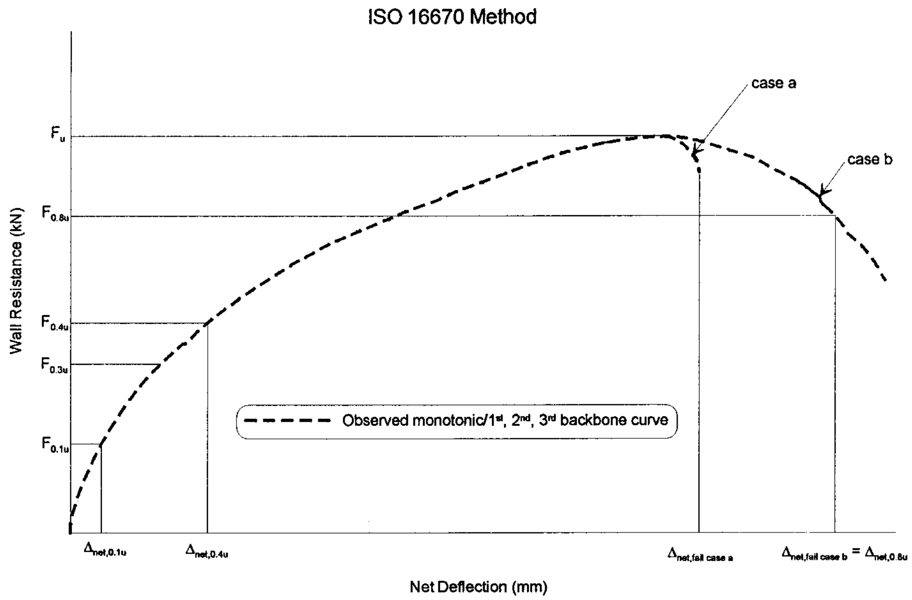

Figure 4.9: ISO 16670 method to determine ultimate displacement and stiffness *(ISO 16670, 2000)*

4.1.10 Park Methods *(Park, 1989)*

Park proposes several methods to determine a yield wall resistance as well as an ultimate (maximum available) deformation from a wall resistance vs. deflection test curve (either monotonic or a reversed cyclic backbone curve) with the aim of realistically approximating the available ductility demand of a tested structure. As shown in Figure 4.10, four alternative definitions for yield displacement are given:

a) The displacement when yielding is first noticed in the system.
b) The yield displacement is found at the intersection of a horizontal line intersecting the peak wall resistance and a straight line with the same initial stiffness as the system.
c) The yield displacement represented by the elastic-plastic system where the plastic portion (horizontal) intersects the peak wall resistance and energies are equated.
d) The yield displacement represented by the elastic-plastic system where the plastic portion (horizontal) intersects the peak wall resistance and the

elastic portion intersects the test curve at first yield of $0.75S_u$, whichever is less.

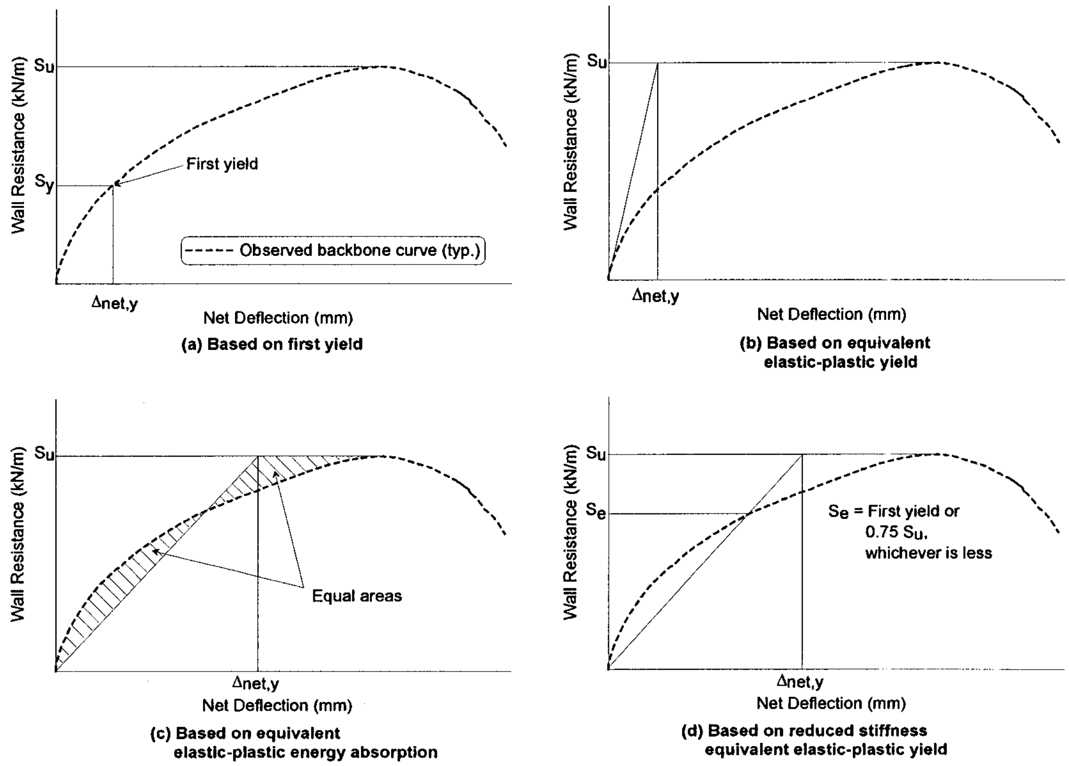

Figure 4.10: Alternative definitions for yield displacement *(Park, 1989)*

Because method (d) (Figure 4.10) is applicable to various materials including concrete, masonry, steel and timber, Park recommends that this definition be used to derive a yield displacement from test data. Definition (d) (as well as (a) (Figure 4.10)), however, incorporates a subjective characterization of a "first yield". In the case of shear walls, both framed with light gauge steel and lumber, the test data exhibits non-linear behaviour from the onset of lateral loading, and so a point of first yield would be very difficult to classify. Definitions (b) and (c) (Figure 4.10) would be applicable to shear walls, however, these definitions do not attempt to identify a yield wall resistance, and so the yielding plateau of the elastic / perfectly plastic bi-linear curve coincides with the ultimate wall resistance. This would not provide a necessary level of safety for design.

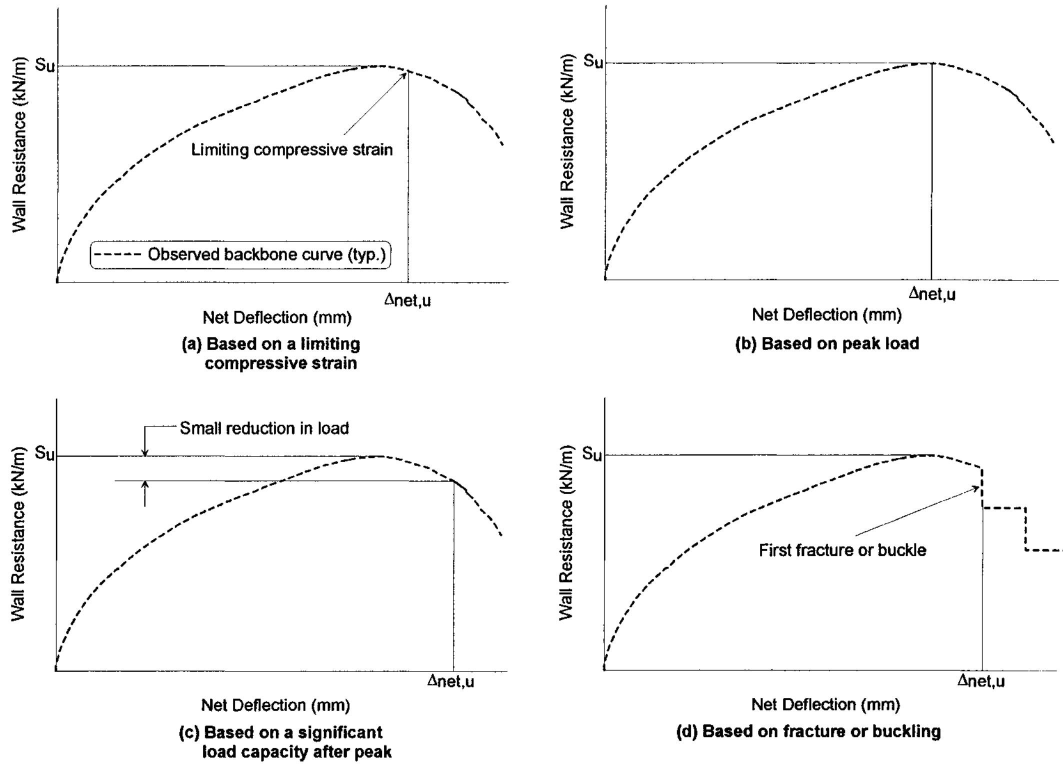

Figure 4.11: Alternative definitions for maximum available (ultimate) displacement *(Park, 1989)*

Of Park's definitions for maximum available useful capacity (at the ultimate lateral wall displacement), cases (b) and (c) (Figure 4.11) are most applicable and suitable for light-framed shear walls sheathed with wood panelling. These structural systems tend to fail in a gradual manner exhibiting an ability to undergo significant deformation while still carrying a substantial load. As well, these shear walls are able to perform reasonably well beyond the peak wall resistance without a significant or sudden reduction in strength. For these reasons, definition (c) (Figure 4.11) is considered to be most representative of light gauge steel framed / wood panel shear wall behaviour and will ultimately be used in the interpretation and analysis of all test data resulting from the 109-specimen main testing program *(Boudreault, 2004; Chen, 2004)* and all future research that forms part of the overall project. The small reduction in load that will be used is 20 % (Section 4.3). In terms of defining the elastic portion of the bi-linear curve, an approach similar to that depicted in Figure 4.10 (d) will be used, however, a value of $0.4S_u$ will be specified rather than $0.75S_u$ (see Section 4.4).

Zhao *(2002)* performed an analysis on previous wood frame and light gauge steel frame shear wall tests that had been carried out by Serrette *(Serrette et al., 1996b; Serrette et al., 1997a)* and the CoLA / UCI research group *(CoLA-UCI, 2001; Freund, 2001; Larsen, 2000; Shah, 2001; Smith, 2001)*. The main objective of the analysis was to provide a conservative estimate of the ductility related force modification factors (R values) associated with seismic design according to several building codes. In order to perform the analysis, the ductility (μ) of the tested wall specimens had to be determined since the R values are directly dependant upon the ability of the wall system to continue to resist load beyond the yield wall resistance *(Zhao, 2002; Boudreault, 2004)*. Zhao used definition (b) (Figure 4.10) *(Park, 1989)* to define the elastic portion of the bi-linear representation of the backbone curve. This portion therefore had the same initial stiffness as the original test data. The plastic portion of the curve was then constructed horizontally through the peak wall resistance obtained during the test (Figure 4.11 – (b)), and, by doing so, a lower bound on the system ductility would be obtained, however, this representation overlooks the post-peak deformation capacity of shear walls. An upper bound on the system ductility could be estimated by placing the horizontal plastic portion of the bi-linear curve at a load level which is 20 % lower than the ultimate wall resistance (Figure 4.11 – (c)). In both cases, and as defined in Figure 4.11, the ultimate wall displacement is defined at the point of intersection between the downloading branch of the test curve and the horizontal line depicting the plastic portion of the bi-linear representation (Figure 4.12).

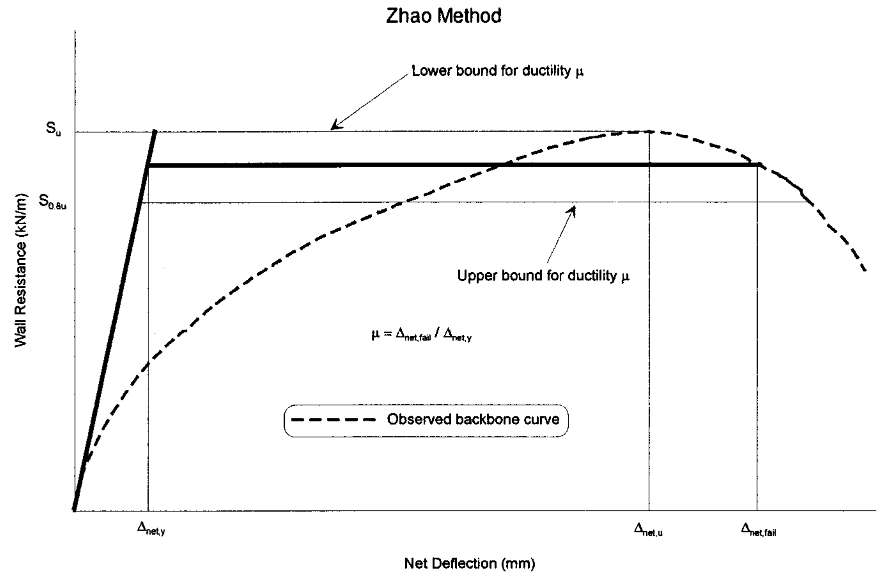

Figure 4.12: Zhao method to determine ductility related design parameters *(Zhao, 2002)*

4.1.11 Foliente Methods *(Foliente, 1996)*

In addition to reviewing the methods proposed by Park *(1989)*, Foliente comments on the need for a widely accepted method of determining a yield point from a highly non-linear force-displacement relationship. This choice of yield point will directly affect the resulting measure of ductility of the system.

Figure 4.13 depicts the methods proposed by Foliente. Method (a) (Figure 4.13) relies on an arbitrary increase of the displacement corresponding to $0.4S_u$ ($\Delta_{net,0.4u}$) by a factor of 1.25, while method (b) (Figure 4.13) depends on the definition of the unit elastic or initial stiffness of the system (k_e). Method (c) (Figure 4.13) utilizes the initial stiffness of the system and the intersection of the lines representing the initial stiffness and a stiffness which is equal to 1/6 of the initial stiffness and tangent to the curve denotes the yield point. Method (d) (Figure 4.13) is essentially based on the equivalent energy concept where a bi-linear relationship is fit to represent the same amount of energy as dissipated by the system before failure. This method of data interpretation in discussed in detail in Section 4.3.

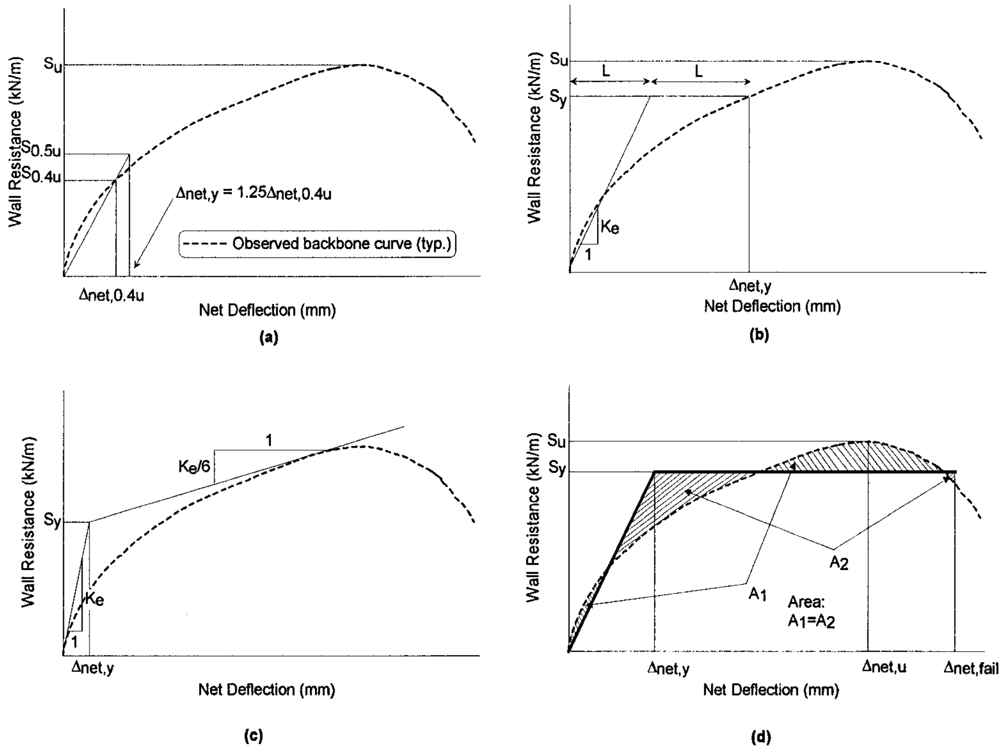

Figure 4.13: Alternative definitions for yield displacement *(Foliente, 1996)*

4.1.12 UK Method *(Griffiths and Wickens, 1996a, 1996b)*

This data interpretation method is linked to a testing protocol and test procedure that incorporates vertical (gravity) loading in addition to the conventional lateral load testing of light frame shear walls. Wall specimens are tested according to a monotonic testing protocol, however, stiffness cycles (four cycles) are included at the beginning of the test in order to simulate a repeated wind loading during the lifetime of the structure. All tests completed for this study were comprised of shear walls constructed with timber framing.

According to Section 6.1 of British Standard BS 5268 *(1988)* the wall specimen is loaded at a static rate to a limiting deflection of 1/500 of the storey height (0.002*H*). The wall is then unloaded and three more cycles are then executed to the same displacement. The wall stiffness was defined as the secant stiffness joining the origin to the lowest of the cycled wall resistances at the limiting deflection. The safe wall resistance (allowable) was then predicted by multiplying the lowest of the cycled wall resistances at the limiting deflection by a factor of 1.25. This wall resistance was termed the test racking stiffness load. It was assumed that this load would provide a safe working racking resistance providing an adequate factor of safety against failure while limiting the elastic interstorey deflection to $0.003H$. The factor of 1.25 is based on the assumption that a 1 in 50 year wind load is 1.25 times as large as a 1 in 12.5 year wind load. A 1 in 12.5 year wind load would occur four times during a structural life of 50 years and hence the four stiffness cycles which make up the BS 5268 *(1988)* protocol. After the four stiffness cycles of the test protocol were completed, the wall was loaded until failure and the test racking strength load was derived from the ultimate wall resistance reduced by a factor of safety between 1.6 – 2.4 depending on the type of sheathing used. Upon obtaining the test racking stiffness load and the test racking strength load, reduction factors ranging between 0.8 and 1.0 are applied to the test results; the average test value for stiffness loads and the minimum test value for strength loads. The applied reduction factors account for the variability in test results where the full wall resistance obtained may be used if five or more tests are performed and a twenty per cent reduction is implemented if only one test is performed for a given wall configuration.

In 1996 the BS EN 594 *(1996b)* became the governing test method for timber shear walls in the UK and Europe. In general, the overall test procedure changed, however, because BS EN 594 does not provide guidelines for the treatment of test data for design, the 1996 version of BS 5268 *(1996a)* incorporated both the new test procedure as well as the reduction method yielding a design wall resistance. Because the BS EN 594 test procedure differs slightly from the BS 5268 test procedure, the racking stiffness load (obtained in a similar manner as described above) is derived by multiplying the observed stiffness by $0.002H$ to estimate the equivalent of the wall resistance at a deformation of $0.002H$ for a wall subject to the BS 5268 test protocol. From then on, the data reduction procedure is identical to that described above. In general, the test racking stiffness load (R_1) and the test racking strength load (R_2) are defined as follows:

$$R_1 = K_e \times 0.002 \times H \times 1.25 \times K_{109} \tag{4-9}$$

$$R_2 = F_u \times K_{109} \tag{4-10}$$

where,

R_1 = Test racking stiffness load, [force]

R_2 = Test racking strength load, [force]

K_e = Elastic stiffness, [force/length]

H = Wall height

K_{109} = Statistical factor to account for number of test replicates ranging from 0.8 for one test to 1.0 for five tests

F_u = Ultimate wall resistance from monotonic test, [force]

For each wall configuration, the design load is taken as the lesser of:

$$R_d = R_{1,ave} \tag{4-11}$$

$$R_d = \frac{R_{2,\min}}{F.S.} \tag{4-12}$$

where,

R_d = Test racking design load

$R_{1,ave}$ = Average test racking stiffness load for given wall configuration

$R_{2,min}$ = Average test racking strength load for given wall configuration

$F.S.$ = Factor of safety ranging from 1.6 to 2.4 dependant upon sheathing

Despite this data reduction method being quite comprehensive in that it covers the major concerns in design such as a design wall resistance, stiffness, as well as a drift limitation, it is closely tied to the British Standards Institute *(1988, 1996a, 1996b)* test procedures and does not address reversed cyclic test protocols. This data reduction method would therefore not be applicable to a wall specimen tested under the CUREE protocol.

4.1.13 Generic Methods *(Adams, 1965; DFPA, 1948; McCreless and Tarpy, 1978; Tarpy, 1980; Tarpy and Girard, 1982; Tarpy and Hauenstein, 1978)*

Numerous dated research programs employed a generic method of data interpretation in which the ultimate resistance attained during the test was converted to a working load by dividing the ultimate resistance by a conservative safety factor. The Douglas Fir Plywood Association (DFPA) *(1948)* (later re-named the American Plywood Association (APA)) was one of the first organizations to perform connection tests that isolated the plywood sheathing to framing stud connection typical of a light frame shear wall. In all, to account for variation in test data, seasoning of the wood members, rate of loading effects, possible load reversal, and to assure that the working load always remained below the estimated proportional limit at approximately 25 % or 30 % of the ultimate load, a factor of slightly less than five was suggested.

In 1965, the APA *(Adams, 1965)* undertook a shear wall research program in which 39 wood frame shear walls with wood sheathing were tested under the monotonic load controlled ASTM E72 *(1961)* test method. Tabulated results show recommended design shear values which were obtained by dividing the ultimate wall resistance by a load factor greater than three, typically in the range of 3.3 – 5.0.

In the fall of 1978, as part of a research program sponsored by the American Iron and Steel Institute (AISI), 18 full-scale shear wall tests composed of light gauge steel framing and gypsum wallboard sheathing were conducted at Vanderbilt University *(Tarpy and Hauenstein, 1978)* according to ASTM test method E564 *(1976)*. This undertaking was one of the first attempts to provide the necessary information for obtaining code approval of light gauge steel stud shear walls to be used as a building component in the United States. A damage threshold level was observed during the tests to be a subjective estimate of the load level at which tearing of the gypsum wallboard paper occurred. The shear stiffness of the tested walls was defined to be the secant stiffness through the origin of the wall resistance vs. deflection graph and the deflection on the curve corresponding to a load level of $0.33S_u$ ($\Delta_{net,0.33u}$). For design purposes, the allowable load for the tested shear wall specimens was defined as the ultimate wall resistance (S_u) divided by a safety factor of 2.0. It was found that this value represented a design load level below that of the damage threshold load level and a corresponding deflection less than the allowable code limit of $L/240$. In subsequent reports initiated by Tarpy at Vanderbilt University *(McCreless and Tarpy, 1978; Tarpy, 1980; Tarpy and Girard, 1982)*, the same data interpretation techniques were applied to a vast selection of shear wall configurations. In a more recent research program sponsored by the AISI undertaken by Klippstein and Tarpy *(1992)*, 17 different wall types were tested for a total of 54 shear wall specimens. This data was interpreted in the same manner as described above for all other Tarpy investigations, however, an additional limitation was placed on the ultimate wall resistance obtained from the test. The ultimate wall resistance was defined as the smaller of the maximum resistance attained during the test and the wall resistance producing a wall deflection of 0.5" (12.7 mm) for a shear wall 8' (2440 mm) in height. The wall resistance obtained was termed the nominal resistance and a safety factor of 2.5 was applied to obtain the allowable shear strength.

4.2 Design Parameters from Existing Shear Wall Tests Incorporated into Current Codes and Associated Design Procedures

The shear wall lateral capacity values listed in current building codes are based primarily on laboratory testing. In the United States, shear wall values are available in tables for both light gauge steel frame shear walls *(AISI, 1998, 2002c; ICC, 2000; ICBO, 1997)* and wood frame shear walls *(AWC, 1996; ICC, 2000; ICBO, 1994, 1997)*. In Canada, shear wall values are only available for wood frame shear walls *(CSA O86, 2001)*.

Light Gauge Steel Frame Shear Walls (USA)

Beginning with the 1997 edition of the Uniform Building Code (UBC) *(ICBO, 1997)*, nominal shear wall values have been included in the US model building codes for walls constructed with light gauge steel framing. Because of ongoing research, the 1998 edition of the AISI Shear Wall Design Guide *(AISI, 1998)* includes all values given in the 1997 UBC and has expanded upon the configurations available for use by designers. These new wall configuration values were proposed and have since been included in the 2000 edition of the International Building Code (IBC) *(ICC, 2000)*. The 2002 draft version of the AISI Standard for Cold-Formed Steel Framing – Design Provisions – Lateral Resistance *(AISI, 2002c)* contains the most up-to-date listing of shear values for walls framed with light gauge steel. The values have remained unchanged from the 2000 IBC, however, the draft standard imposes some conservative restrictions on aspect ratio for certain wall configurations and offers some advice on designing perforated shear walls.

Since the 2000 IBC contains all current nominal shear values that are available to designers in the United States, only this document will be discussed hereafter. Chapter 22 of the 2000 IBC (Section 2211) includes wind and seismic requirements for light-framed cold-formed steel walls. Three tables are available for use by designers: the nominal shear value used to establish the allowable shear values or design shear values are given in Table 2211.1(1) or 2211.1(2) for wind loads or Table 2211.1(3) for seismic loads *(ICC, 2000)*. All nominal values are as found by Serrette in Phase I *(Serrette et al., 1996b)* and Phase II *(Serrette et al., 1997a)* of the research conducted at Santa Clara University as per the respective data interpretation technique described in Sections 4.1.3 and 4.1.4. Values for wind loads are based on monotonic tests, whereas values for seismic loads are based on reversed cyclic tests, and each value quoted in the code is an average value of two replicate tests performed. A reproduction of the design tables included in Chapter 22 of IBC 2000 *(ICC, 2000)* follows (Tables 4.1 – 4.3) and each value is linked with the actual test performed by Serrette in 1996 or 1997. All tests are given in parentheses: underlined tests indicate tests performed in 1997 *(Serrette et al., 1997a)* and non-underlined tests indicate tests performed in 1996 *(Serrette et al., 1996b)*.

Table 4.1: Nominal Shear Values[1] for Wind Forces in Pounds per Foot (plf) for Shear Walls Framed with Cold-Formed Steel Studs (USA) (adapted Table 2211.1(1) 2000 IBC *(ICC, 2000)*)

Assembly Description	Maximum Height/Length	Fastener Spacing at Panel Edges[4] (inches)				Maximum Framing Spacing
		6	4	3	2	
15/32-inch Structural 1 Sheathing (4-ply) one side	2:1	1065[3] (1A6/7)	-	-	-	24 inches o.c.
7/16-inch Rated Sheathing (OSB) one side	2:1	910[3] (1A2/3)	1410 (1D3/4)	1735 (1D5/6)	1910 (1D7/8)	24 inches o.c.
7/16-inch Rated Sheathing (OSB) one side **perpendicular** to framing	2:1	1020[3] (1E1/2)	-	-	-	24 inches o.c.
7/16-inch Rated Sheathing (OSB) one side	4:1[2]	-	1025 (5/6)	1425 (INTERP.)	1825 (7/8)	24 inches o.c.
0.018-inch Steel sheet, one side	2:1	485 (15/16)	-	-	-	24 inches o.c.
0.027-inch Steel sheet, one side	4:1[2]	-	1000 (13/14)	-	-	24 inches o.c.

1 inch = 25.4 mm, 1 pound per foot (plf) = 14.5939 N/m
[1]Nominal shear values shall be multiplied by the resistance factor Φ to determine design strength or divided by the safety factor Ω to determine allowable shear values.
[2]AISI 2002 draft Standard *(AISI, 2002c)* limits aspect ratio (H/L) to 2:1, however, permits the use of shear walls with aspect ratio not exceeding 4:1 provided the nominal shear strength is multiplied by 2L/H.
[3]Where fully blocked gypsum board is applied to the opposite of this assembly with screw spacing at 7 inches o.c. edge and 7 inches o.c. field, these nominal values are permitted to be increased by 30 per cent.
[4]Screws in the field of the panel shall be installed 12 inches o.c.

Table 4.2: Nominal Shear Values[1] for Wind Forces in Pounds per Foot (plf) for Shear Walls Framed with Cold-Formed Steel Studs and Faced with Gypsum Board (USA) (adapted Table 2211.1(2) 2000 IBC *(ICC, 2000)*)

Assembly Description	Maximum Height/Length	Orientation	Fastener Spacing		Nominal Shear Value (plf)
			Edge	Field	
1/2-inch gypsum board on **both sides** of wall; Studs maximum 24 inches o.c.	2:1	Gypsum board applied perp. to framing with strap blocking behind the horizontal joint and with solid blocking between the first two end studs	7	7	585 (2A1/3)
			4	4	850 (2A2/4)

1 inch = 25.4 mm, 1 pound per foot (plf) = 14.5939 N/m
[1]Nominal shear values shall be multiplied by the resistance factor Φ to determine design strength or divided by the safety factor Ω to determine allowable shear values.

Table 4.3: Nominal Shear Values[1,4] for Seismic Forces in Pounds per Foot (plf) for Shear Walls Framed with Cold-Formed Steel Studs (USA) (adapted Table 2211.1(3) 2000 IBC *(ICC, 2000)*)

Assembly Description	Maximum Height/Length	Fastener Spacing at Panel Edges[3] (inches)				Maximum Framing Spacing
		6	4	3	2	
15/32-inch Structural 1 Sheathing (4-ply) one side	2:1[2]	780 (PLY1/2)	990 (PLY3/4)	1465 (PLY5/6)	1625 (PLY7/8)	24 inches o.c.
15/32-inch Structural 1 Sheathing (4-ply) one side end studs 0.043-inch min. thickness	2:1	-	-	1775 (A1/2)	2190 (A3/4)	24 inches o.c.
15/32-inch Structural 1 Sheathing (4-ply) one side all studs (and track) 0.043-inch min. thickness	2:1	890 (B1/2)	1330 (INTERP.)	1775[5] (A1/2)	2190[5] (A3/4)	24 inches o.c.
7/16-inch Rated Sheathing (OSB) one side	2:1[2]	700 (OSB1/2)	915 (OSB3/4)	1275 (OSB5/6)	1625 (OSB7/8)	24 inches o.c.
7/16-inch Rated Sheathing (OSB) one side end studs 0.043-inch min. thickness	2:1	-	-	1520 (A5/6)	2060 (A7/8)	24 inches o.c.
0.018-inch Steel sheet, one side	2:1	390 (D1/2)	-	-	-	24 inches o.c.
0.027-inch Steel sheet, one side	2:1[2]	-	1000 (F1/2)	1085 (INTERP.)	1170 (F3/4)	24 inches o.c.

1 inch = 25.4 mm, 1 pound per foot (plf) = 14.5939 N/m

[1]Nominal shear values shall be multiplied by the resistance factor Φ to determine design strength or divided by the safety factor Ω to determine allowable shear values. Nominal shear values shall not be increased for material applied on both sides of the wall.

[2]AISI 2002 draft Standard *(AISI, 2002c)* limits aspect ratio (H/L) to 2:1, however, permits the use of shear walls with aspect ratio not exceeding 4:1 provided the nominal shear strength is multiplied by 2L/H.

[3]Screws in the field of the panel shall be installed 12 inches o.c.

[4]Nominal shear values for seismic loads are based on the average nominal load resulting from positive and negative cycles of the shear wall test.

[5]These values are considered as lower bounds since tests A1/2 and A3/4 were conducted on assemblies containing 0.043" thick end studs only.

For walls constructed in an identical manner to the test specimens tested by Serrette *(Serrette et al., 1996b, 1997a)* including walls consisting of back-to-back chord studs and track and studs of minimum thickness 0.033" (0.84 mm) unless otherwise specified in the tables, an allowable shear value can be determined by dividing the nominal shear value listed in Tables 4.1 – 4.3 by a factor of safety (Ω) of 2.5 for allowable stress design (ASD). Where load and resistance factor design (LRFD) is used, the design shear can be determined by multiplying the nominal shear value listed in Tables 4.1 – 4.3 by a resistance factor (Φ) of 0.55. Earlier codes (1997 UBC) used a divisor of 3.0 for wind loads (resistance factor of 0.45) and 2.5 for seismic loads (resistance factor of 0.55). Wood structural panel sheathing and sheet steel sheathing may be applied either parallel or perpendicular to the steel framing while retaining the same nominal strength value, however, all edges must be fully blocked, and gypsum wallboard sheathing must be applied perpendicular to the framing members in accordance with the provisions outlined in Table 4.2. In no case may a nominal shear value be increased for duration of load effects as is usually permitted for wood and, unless otherwise specified (note 3, Table 4.1), nominal shear values may not be increased for sheathing installed on both sides of a wall.

Wood Frame Shear Walls (USA)

As is the case for light gauge steel frame shear walls, the US model building codes offer shear values for wood frame shear walls in tabular format. Instead of presenting nominal values, however, allowable shear values are listed for use with allowable stress design (ASD). Since research has been ongoing over the years, the 2000 edition of the IBC *(ICC, 2000)* contains all construction configurations and allowable shears outlined in the 1997 UBC *(ICBO, 1997)* as well as additional wall configurations. For this reason, only the 2000 IBC will be discussed hereafter.

Chapter 23 of the 2000 IBC (Section 2305 and 2306) includes wind and seismic requirements for wood frame shear walls and diaphragms. Table 2306.4.1 *(ICC, 2000)* lists allowable shear values for fully blocked wood structural panel shear walls for wind and seismic loading. The same values exist for both wind and seismic loads since the testing conducted to produce these values was based solely upon monotonic loading. Since most of the allowable shear values included in the current code are results of tests performed as early as 1953 by the American Plywood Association (APA) *(Skaggs and Rose, 1996)*, a generic method (Section 4.1.13) in which a load factor or safety factor was applied to the peak wall resistance obtained during the test was used to obtain recommended design shears. A "rounded" design (allowable) shear was chosen in order to obtain a safety factor of approximately 3.0. In 1955 shear wall design values were recognized in the UBC *(Skaggs and Rose, 1996)*. Further research was performed by Adams in 1965 *(Adams, 1965)*. The safety factors between ultimate wall resistance and recommended design shears chosen by Adams fall into the range of 3.3 – 5.0. With the research conducted by Adams, the first separate table for design values for wood frame shear walls was incorporated into the 1967 edition of the UBC *(Tissell, 1989)*. Because the wind loads determined by ASCE 7 *(1998)* have increased and engineers now have a better understanding of structural performance under wind loads, the allowable shear capacities for wood frame shear walls listed in Table 2306.4.1 of IBC 2000 may be increased by 40 % for wind design only *(Ghosh and Chittenden, 2001)*. It is estimated that a safety factor of 2.0 is sufficient to safeguard wood shear walls against wind loads, and hence the 40 % increase is conservative considering that for most wall configurations, the inherent safety factor falls well above 3.0.

Because of extensive damage to wood frame buildings observed after the Northridge (CA, USA, January 17, 1994) earthquake, the City of Los Angeles, Department of Building and Safety, imposed a 25 % reduction on allowable shear values until further research into the behaviour of shear walls under reversed cyclic loading was conducted. The Northridge earthquake, a major seismic event the likes of which could also occur along the West Coast of BC, resulted in US $40 billion in property damage to wood frame construction, reduced 48 000 wood frame housing units to an uninhabitable status, and was responsible for 25 fatalities, 24 of which were caused by damage to wood frame buildings *(Krawinkler et al., 2000)*. In 1994, SEAOSC began developing the SPD reversed cyclic protocol for shear walls in order to provide a realistic simulation of earthquake loads. The protocol was reviewed over a 2-1/2 year period and a finalized version was published in 1997 *(SEAOSC, 1997)*. Using this protocol, numerous research programs have been initiated *(Rose, 1998; Dinehart and Shenton, 1998b; CoLA-UCI, 2001)* to study the effect of a reversed cyclic loading history on wood frame shear walls, and, in general, results indicate that a lower strength is achieved. It has been argued, however, that the SPD protocol demands far too much energy dissipation from the wall specimen and so the actual strength of the wall is underestimated while the system's available ductility is overestimated *(Karacabeyli et al., 1999)*.

Since load and resistance factor design (LRFD) for wood frame shear walls is not covered in the 1997 UBC or the 2000 IBC, a separate design document must be used in the United States. The LRFD Manual for Engineered Wood Construction along with the Supplement for Structural-Use Panels *(AWC, 1996)* contains the Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction *(AF&PA/ASCE 16, 1995)* and provides guidance for the prescriptive design of wood frame shear walls subject to both wind and seismic loads. The basic equation governing the design of shear walls to resist lateral loads given in the Standard *(AF&PA/ASCE 16, 1995)* is as follows:

$$D_u \leq \lambda \phi_z D' \tag{4-13}$$

$$D' = D \times \left(C_1 C_2 \ldots C_n\right) \tag{4-14}$$

where,

D_u = Applied force due to factored loads, [force per unit length]

λ = Time effect factor for loading condition = 1.0 for lateral force design (wind or seismic)

Φ_z = Resistance factor for shear walls limited by fastener strength = 0.65

D' = Adjusted shear wall resistance, [force per unit length]

D = Reference resistance

C_i = Applicable adjustment factors

Table 5.4 of the Supplement for Structural-Use Panels *(AWC, 1996)* lists adjusted factored (Φ_z = 0.65, λ = 1.0) shear resistances for wood frame shear walls. The values are applicable if dry conditions exist (C_M = 1.0), the shear wall is used in a normal temperature range (C_t = 1.0), the material is untreated (C_{pt} = C_{rt} = 1.0), and the wall is subject to short-term lateral loading (λ = 1.0). For shear walls that do not meet all of the above requirements, the tabulated values can be modified by the appropriate adjustment factors as necessary.

Because the Standard for LRFD for Engineered Wood Construction was developed in 1995, the design table (Table 5.4 – Supplement for Structural-Use Panels *(AWC, 1996)*) does not contain the vast array of wall configurations such as found in the 2000 IBC, however, the values from the 1994 UBC were directly converted to LRFD values by applying a factor of 1.3 to the ASD values. This factor is the same as the load factor for wind loads in the Standard *(AF&PA/ASCE 16, 1995)*. It must be noted, however, that since the LRFD values are directly determined from the ASD values given in the UBC, they too are solely based on monotonic testing of shear walls dating back to the early 1950s. Since the tabulated values can be applied for seismic loading as well, it can also be argued that these values are not conservative enough to account for the strength and stiffness degradation associated with reversed cyclic loading.

Wood Frame Shear Walls (Canada)

Beginning with the 1989 edition of CSA O86 *(CSA O86-M89, 1989)*, specified strengths for wood frame shear walls have been included in the Canadian Wood Standard for use by designers. These values are applicable for full-height shear wall segments of aspect ratio 3.5:1 or less subject to wind or seismic forces consistent with the limit states design procedure respected in Canada. Table 9.5.1A of the current edition of the Engineering Design in Wood Standard *(CSA O86, 2001)* lists the specified shear strength for wood frame shear walls based on the thickness of the sheathing panel. In terms of panel products available for use, this includes Douglas Fir plywood *(CSA O121, 1978)*, Canadian Softwood plywood *(CSA O151, 1978)*, Types 1 and 2 Design Rated OSB *(CSA O452, 1994)*, and OSB and waferboard meeting CSA O437 *(1993)*. In addition, the current wood standard (2001) allows for Construction Sheathing OSB in accordance with CSA O325 *(1992)* to be used, after a suite of tests *(He et al., 1997; Karacabeyli and Lum, 1999; Ni and Karacabeyli, 1998; Rose, 1998; Skaggs, 1995)* conducted on OSB panels revealed that it had similar strength characteristics to plywood panels. Because CSA O325 OSB is a performance rated panel which is designated based on its end-use, an equivalency table is included in the standard (Table 9.5.3 – CSA O86-01) to allow the user to equate the end-use panel mark to a tabulated panel thickness.

The specified strengths currently given in CSA O86 *(2001)* are derived from the ASD values listed in the 1997 UBC *(ICBO, 1997)* for Sheathing grade APA plywood. The derivation is based on a soft-conversion from ASD to limit states design (LSD) similar to that implemented in the conversion of ASD values to LRFD values, as follows:

$$\alpha(ASD) = \phi(K_D)v_d \tag{4-15}$$

where,

α = Load factor for wind load according to NBCC 1995 *(NRCC, 1995)* = 1.5

ASD = Allowable stress design value from 1997 UBC *(ICBO, 1997)* for sheathing grade plywood, [force per unit length]

Φ = Resistance factor for wood frame shear walls = 0.7

K_D = Load duration modification factor = 1.15 for short term loads

v_d = Specified shear strength, [force per unit length]

and, by rearranging terms,

$$v_d = \frac{\alpha(ASD)}{\phi(K_D)} = \frac{1.5(ASD)}{0.7(1.15)} = 1.863(ASD) \tag{4-16}$$

The specified shear strength values listed in Table 9.5.1A of CSA O86 *(2001)* can therefore be derived by applying a factor of 1.863 to the ASD values given in the 1997 UBC for sheathing grade plywood of equivalent nominal thickness. The values were calibrated to wind design because there were, and continue to be, significant changes to earthquake design loads. The factored shear resistances, however, are applicable for both wind and seismic design in Canada. In order to determine the factored shear resistance of a nailed shear wall with wood framing, the following procedure can be applied:

$$V_r = \sum V_{rs} \tag{4-17}$$

$$V_{rs} = \phi v_d K_D K_{SF} J_{ub} J_{sp} J_{hd} J_n L_w \tag{4-18}$$

where,

V_r = Total factored shear resistance of wall assembly, [force]

V_{rs} = Factored shear resistance of individual full-height wall segments with aspect ratio not exceeding 3.5:1, [force]

K_{SF} = Service condition factor for lateral loads on nails

J_{ub} = Strength adjustment factor for unblocked shear walls

J_{sp} = Species factor for framing material

J_{hd} = Hold-down effect factor for shear wall segment

J_n = Modification factor for nail diameter

L_w = Length of shear wall segment

CSA O86 *(2001)* allows for shear walls to be constructed with sheathing panels on both sides of the wall. When wood-based panels or gypsum wallboard panels are applied on opposite sides of the wall, the capacities can be summed for that particular shear wall segment. Where multiple layers exist on one side of a shear wall segment, the capacity must be determined using the innermost panel unless a wood structural panel is applied over 12.7 mm or 15.9 mm thick gypsum wallboard. With the latter case, the capacity of the wood-based panel may be considered.

4.3 Equivalent Energy Concept and the EEEP Model to Determine Design Parameters

Shear walls subject to lateral loads exhibit very complex and highly non-linear behaviour from the onset of loading. In order to evaluate specific design properties such as yield force, stiffness, ductility, energy dissipation capacity, *etc.*, it is important that test data is interpreted according to a methodology that accounts for the wall behaviour and whose use can be adequately justified. It is suggested that the Equivalent Energy Elastic-Plastic (EEEP) model be implemented in this body of research and for all other shear wall test data (both monotonic and reversed cyclic). Treating the data in the same manner, so as to develop a convention for the interpretation of test results, will allow for the development of a bank of data comprising of a wide range of construction configurations. Test data from other sources, both past and future, when incorporated, would make available a more comprehensive set of shear wall design parameters.

As stated in Chapter 3, not only is it important to choose a loading protocol for testing that best represents the design level loading that may occur during the lifetime of a structure, it is also important to implement a data interpretation model that closely represents the actual performance of the shear wall so that realistic design values can be obtained. Vital information on possible failure modes and performance characteristics can be obtained from laboratory testing, which otherwise may not be observed through analytical models. It is therefore important that when treating the data, this fundamental information not be overlooked.

For the interpretation of all monotonic and reversed cyclic tests included in the main testing program the equivalent energy elastic-elastic model was used to represent the behaviour of light gauge steel frame / wood panel shear walls observed during the tests. The concept of equivalent energy was first proposed by Park *(1989)*, presented in a modified form by Foliente *(1996)*, and applied to the interpretation of test data in an extensive wood frame and steel frame shear wall research program initiated at Virginia Polytechnic Institute and State University by various researchers *(Dolan and Heine, 1997a, 1997b, 1997c; Dolan and Johnson, 1997a, 1997b; Heine, 1997; Johnson, 1997; Johnson and Dolan, 1996; Salenikovich and Dolan, 1999b; Salenikovich et al., 2000a, 2000b).* After exploring the many data interpretation techniques outlined in Section 4.1, it was decided that the EEEP model best represented the behaviour of light gauge steel frame / wood panel shear walls subject to both monotonic and reversed cyclic loads. The model results in a load-deflection curve that can easily be defined and constructed, yet still provides a realistic depiction of the data obtained from laboratory testing. Moreover, the EEEP model can be used with any reversed cyclic loading protocol; as well it recognizes the post-peak deformation capacity by taking into account the energy dissipated by the test specimen up to failure *(Park, 1989)*. The EEEP method has also been commonly applied in the analysis of test data for other types of structural systems, *i.e.* masonry, concrete, steel, *etc.*, *(Park, 1989)*. Furthermore, the EEEP approach is consistent with Serrette's *(1998)* recommendations that a detailed energy-based methodology should be implemented for data interpretation.

The equivalent energy elastic-elastic (EEEP) model is based on the notion that the energy dissipated by the wall specimen during a monotonic or reversed cyclic test be equivalent to the energy which is represented by the bi-linear model curve. This representation demonstrates the performance of an ideal perfectly elastic / plastic shear wall system while dissipating an equivalent amount of energy. For simplicity, this curve is chosen to be bi-linear, which depicts linear elastic behaviour of the system until the yield point and perfectly plastic behaviour until failure. This simplification allows for the design parameters to be easily identifiable.

As described in Chapter 3, the unloading portions of the monotonic test data were not considered in the energy calculations (Figures 3.26 and 4.14). In the case of reversed cyclic tests, an outer envelope of the cyclic behaviour (backbone curve) was constructed, and then used to obtain the design parameters. Note that the actual energy dissipated by the wall specimen during a reversed cyclic test is as defined by Equations (3-6) and (3-7) and as shown by Figure 3.27, however, this calculated energy is significantly greater than that represented by the area under an envelope curve since the hysteresis loops overlap. This design assumption, whereby the repeated energy dissipation ability of the shear wall from cycle to cycle is ignored, will lead to a conservative estimate of the yield point of the system, *i.e.* a lower design capacity.

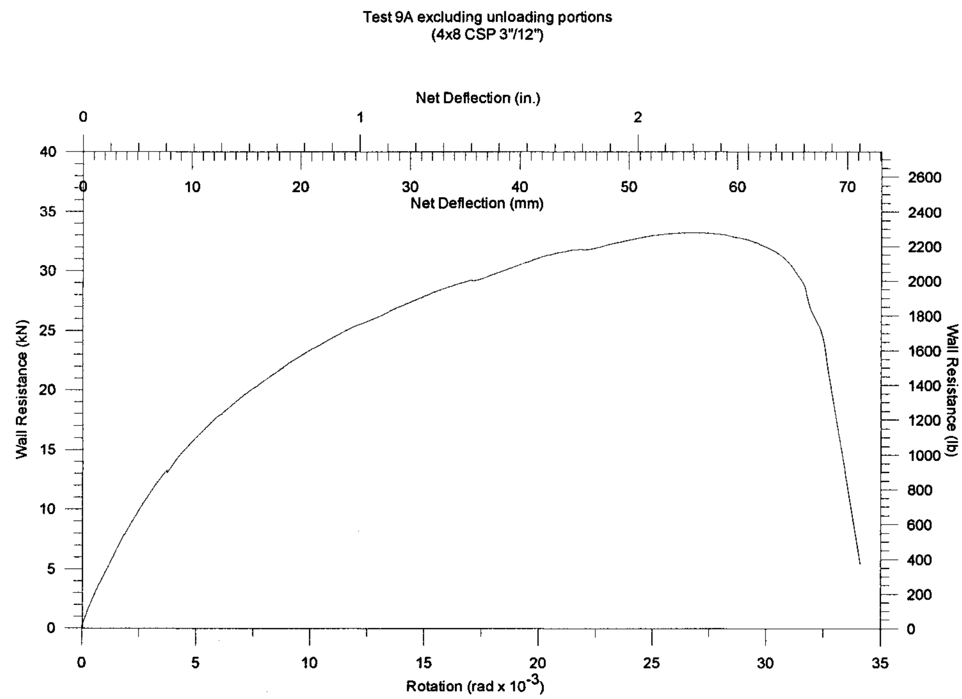

Figure 4.14: Monotonic test data excluding unloading portions of protocol (Test 9A)

Many different definitions for the backbone curve exist. If the Sequential Phase Displacement (SPD) protocol as suggested by SEAOSC *(1997)* is used to conduct reversed cyclic tests on shear wall specimens, two of the possible envelope curves are defined as follows: i) The initial envelope best-fit curve which accommodates the peak loads from the first cycle of each displacement amplitude (Figure 4.5). This backbone curve envelopes the whole hysteresis curve including all peaks and resembles the load path exhibited during a monotonic test. ii) The stabilized curve is defined as the best-fit curve which accommodates the peak loads from the last (fourth) cycle of each displacement amplitude (Figure 4.5). It is argued in a discussion by Karacabeyli *et al.* *(1999)* in response to research carried out by

Dinehart and Shenton *(1998b)* that the ultimate load capacity of a shear wall under earthquake loading would be similar to the load path outlined by the initial envelope and not that outlined by the stabilized envelope. It has been found that the SPD protocol demands far too much energy dissipating capability from the wall specimen, and this causes failure modes (nail rupture in wood frame shear walls) that are not consistent with past performance during shake-table testing *(Karacabeyli and Ceccotti, 1998)* and in reconnaissance surveys of timber structures in the US *(Ficcadenti et al., 1995)* after design level earthquakes. Karacabeyli and Ceccotti *(1998)* performed pseudodynamic (> 1 Hz harmonic excitation) tests on wood frame shear walls using historical earthquake records and measured the ultimate test resistances to be larger than those found by employing the initial envelope curve with the SPD protocol and even larger than the ultimate resistance obtained in a monotonic test. In view of the aforementioned findings and as suggested by CUREE in Ryan *et al. (2001)*, the backbone curve incorporated into this body of research consists of a curve which accommodates the peak resistance attained and / or the resistance attained at the maximum displacement for each primary cycle (Figure 4.15) of the CUREE ordinary ground motions reversed cyclic protocol as described in Section 3.4.2. According to this definition, the backbone curve could pass through two points for one given cycle (the peak resistance and the maximum displacement). In most cases, a curve passing through one of the above-mentioned points for each primary cycle provided a smooth envelope of the actual hysteretic behaviour.

A Microsoft Excel Macro *(Boudreault, 2004)* was implemented in order to ease the procedure outlined above for both the monotonic and reversed cyclic tests. The raw test data was first copied into the Macro and the unloading portions of the monotonic tests were removed for the energy calculation. Also, for both monotonic and cyclic tests, because the data acquisition system always began collecting data before the actuator began to displace, the unwanted data at the beginning of the test was removed manually. By inputting the wall length, the Macro was conditioned to correct all displacements (Eq. (3-2)), perform the calculations to obtain the wall rotation (Eq. (3-3)), wall resistance (Eq. (3-4)), corrected wall resistance for acceleration in the case of reversed cyclic tests (Eq. (3-5)) and plot the backbone curve.

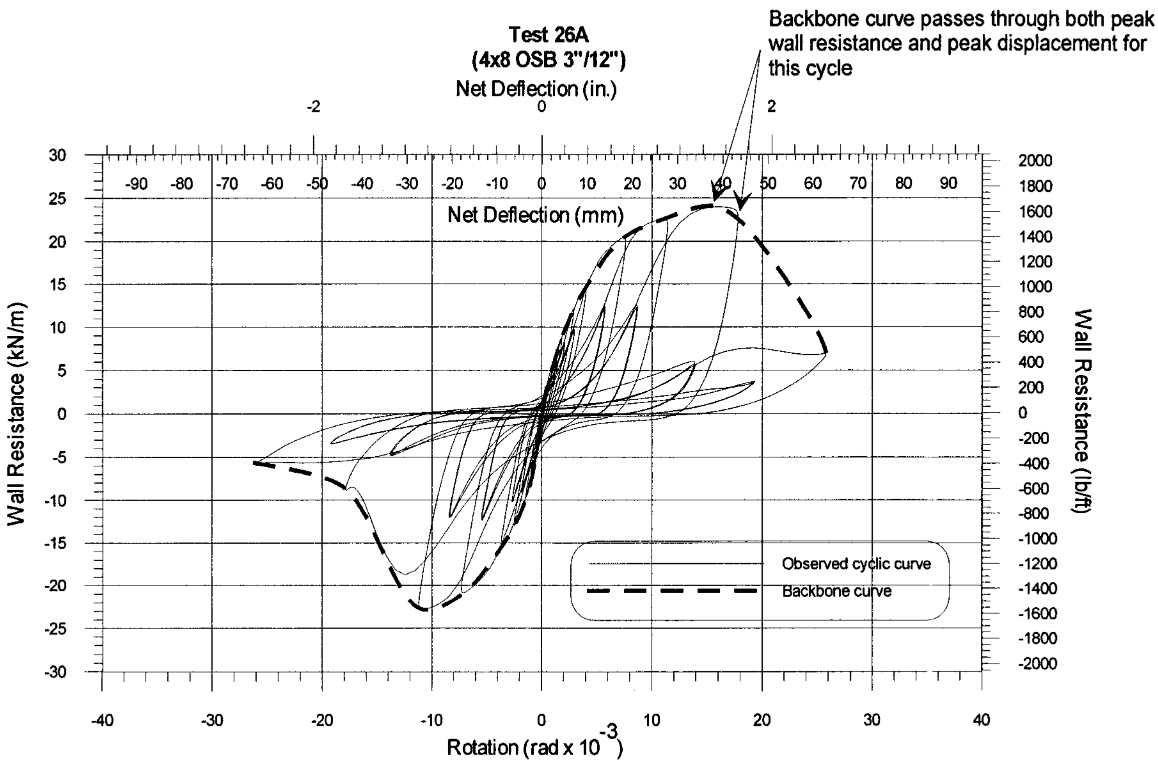

Figure 4.15: Hysteretic test data enveloped by backbone curve (Test 26A)

Once the backbone curve was constructed for each reversed cyclic test (for both positive and negative displacement cycles), the EEEP curve was created based on the equivalent energy approach (Figure 4.16). The monotonic curve (excluding the unloading portions) (Figure 4.14) can also be interpreted as a "backbone" curve and analyzed using the EEEP methodology.

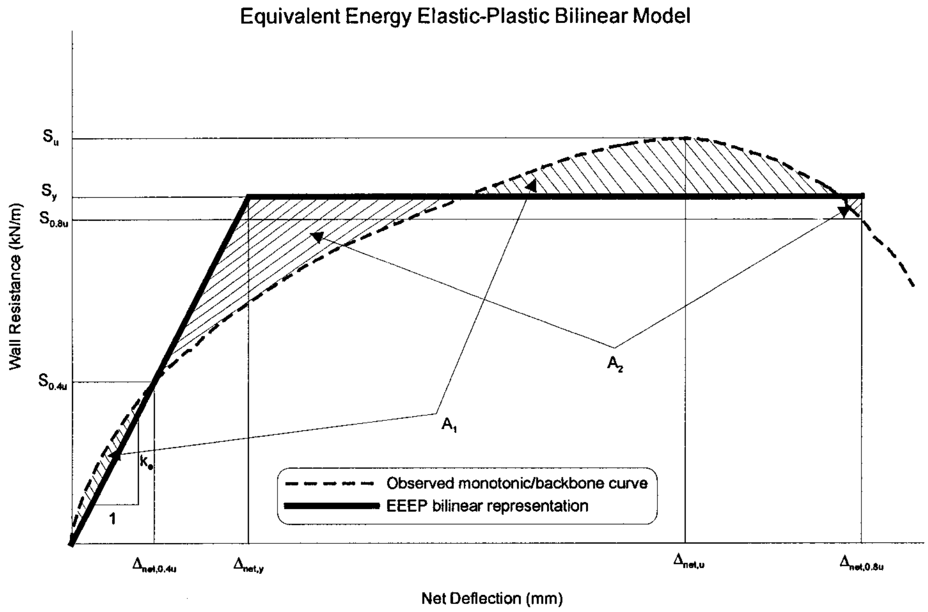

Figure 4.16: EEEP model *(Park, 1989; Salenikovich et al., 2000b)*

As shown in Figure 4.16, the EEEP curve is constructed by first determining three main parameters from the monotonic corrected test curve or the backbone curve for the reversed cyclic tests. These parameters include the maximum wall resistance attained (S_u), and the two points on the test curve corresponding to $0.4S_u$ and $0.8S_u$ (post-peak), as well as all matching displacements ($\Delta_{net,u}$; $\Delta_{net,0.4u}$; $\Delta_{net,0.8u}$). Using a search, the Macro was able to determine these points. In addition, the Macro performed a step-by-step energy integration according to Eqs. (3-6) and (3-7) for the monotonic test curves and the backbone curves up to failure ($\Delta_{net,0.8u}$). This post-peak load level is considered to be the limit of the useful capacity of the shear wall and determines the failure point of the specimen. In the case of the reversed cyclic tests, because only a limited number of data points formed the backbone curve, a polynomial trendline (known function) was used to replicate the backbone curve and give a more accurate result when step-by-step integration was used to calculate the area under the curve (a third order, fourth order, fifth order, or sixth order polynomial was used depending on fit). A straight line passing through the origin and ($0.4S_u$, $\Delta_{net,0.4u}$) defines the elastic portion of the bi-linear EEEP curve. A reasonable estimate of the maximum service load level is commonly chosen to be 40 % of the ultimate resistance. Many of the definitions for interpretation techniques included in Section 4.1, in addition to the proposed ASTM *(1995)* standard for cyclic tests of mechanical connections, use the 40 % of ultimate level to define the elastic stiffness *(Salenikovich et al., 2000b)*. In Figure 4.10 (d), Park *(1989)* utilizes a factor of 0.75 applied to the ultimate wall resistance to define the secant elastic stiffness of the system. As outlined in Section 4.4, for light gauge steel frame / wood panel shear walls subject to wind loads, it was deemed more appropriate to employ a factor of 0.40 since this also represents an approximate service load level. This elastic portion has a slope equal to the unit elastic stiffness (k_e) on the wall resistance (force per unit length) vs. deflection response plot (Figure 4.16) or the elastic stiffness (K_e) on the wall resistance (force) vs. deflection response plot. The horizontal line depicting the plastic portion of the EEEP curve is positioned so that the area bounded by the EEEP curve, the x-axis, and the limiting displacement ($\Delta_{net,0.8u}$) is equal to the area below the observed test curve (A) (calculated by step-by-step integration). This amounts to equating the two oppositely hatched areas ($A_1 = A_2$) shown in Figure 4.16. A relationship for the wall resistance at yield (S_y) can be derived as follows:

Defining the area under the EEEP curve,

$$A_{EEEP} = \frac{S_y \times \Delta_{net,y}}{2} + \left[S_y \times \left(\Delta_{net,0.8u} - \Delta_{net,y}\right)\right] \tag{4-19}$$

and enforcing the equal energy concept (set A_{EEEP} equal to the area under the response curve, A):

$$\frac{S_y \times \Delta_{net,y}}{2} + \left[S_y \times \left(\Delta_{net,0.8u} - \Delta_{net,y}\right)\right] = A \tag{4-20}$$

$$S_y\left[\frac{\Delta_{net,y}}{2} + \left(\Delta_{net,0.8u} - \Delta_{net,y}\right)\right] = A \tag{4-21}$$

From the definition of unit elastic stiffness,

$$\Delta_{net,y} = \frac{S_y}{k_e} \tag{4-22}$$

we can substitute into Eq. (4-21),

$$S_y\left[\frac{S_y}{2\times k_e}+\left(\Delta_{net,0.8u}-\frac{S_y}{k_e}\right)\right]=A \tag{4-23}$$

from which we obtain a quadratic relationship:

$$-\left(\frac{S_y^{\ 2}}{2\times k_e}\right)+\left(\Delta_{net,0.8u}\times S_y\right)-A=0 \tag{4-24}$$

We can apply the quadratic formula to obtain S_y:

$$S_y=\frac{-\Delta_{net,0.8u}\pm\sqrt{\Delta_{net,0.8u}^{\ \ 2}-\dfrac{2A}{k_e}}}{-\dfrac{1}{k_e}} \tag{4-25}$$

where,

$$k_e=\frac{0.4\times S_u}{\Delta_{net,0.4u}} \tag{4-26}$$

S_y = Wall resistance at yield, [force per unit length]

S_u = Ultimate wall resistance, [force per unit length]

A = Calculated area under monotonic response curve or backbone curve up to failure ($\Delta_{net,0.8u}$), [force]

k_e = Unit elastic stiffness, [force per length per wall length]

$\Delta_{net,0.8u}$ = Displacement corresponding to a post-peak wall resistance of $0.8S_u$

$\Delta_{net,y}$ = Yield displacement corresponding to S_y

Equation (4-25) was used by the Macro to calculate the yield wall resistance. The plus / minus exists in Equation (4-25) for the reversed cyclic tests. On positive cycles, the plus sign is used in the formulation (S_{y+}), and the opposite occurs with the negative cycles (S_{y-}). An EEEP plot was then constructed by the Macro and all values of interest were returned in tabular format. A complete description of the Microsoft Excel Macro used to analyse the data for the 109-specimen main test program is contained in Boudreault *(2004)*.

For the reversed cyclic tests, it was observed that at larger amplitudes of the CUREE protocol, the shear wall specimen usually failed on a positive cycle, and, when returning to complete the negative cycle at the same magnitude displacement amplitude, had a significantly reduced capacity. For this reason, it was decided that separate backbone curves, trendlines and EEEP curves would be constructed for the positive and negative cycles. Tables 4.5 and 4.6 report all design values for reversed cyclic tests for the positive cycles and the negative cycles respectively. Typical shear wall responses including backbone and EEEP curves are shown for a monotonic and reversed cyclic test in Figures 4.17 and 4.18, respectively. Branston *et al. (2004)* contains all response curves with superimposed backbone and EEEP curves for all tests forming part of the research program. In addition, this stand-alone document *(Branston et al., 2004)* of test data and results includes a summary table of the key design parameters, *i.e.* yield wall resistance, ductility, ultimate wall resistance, elastic stiffness, *etc.*, for each test conducted *(Boudreault, 2004; Chen, 2004)*.

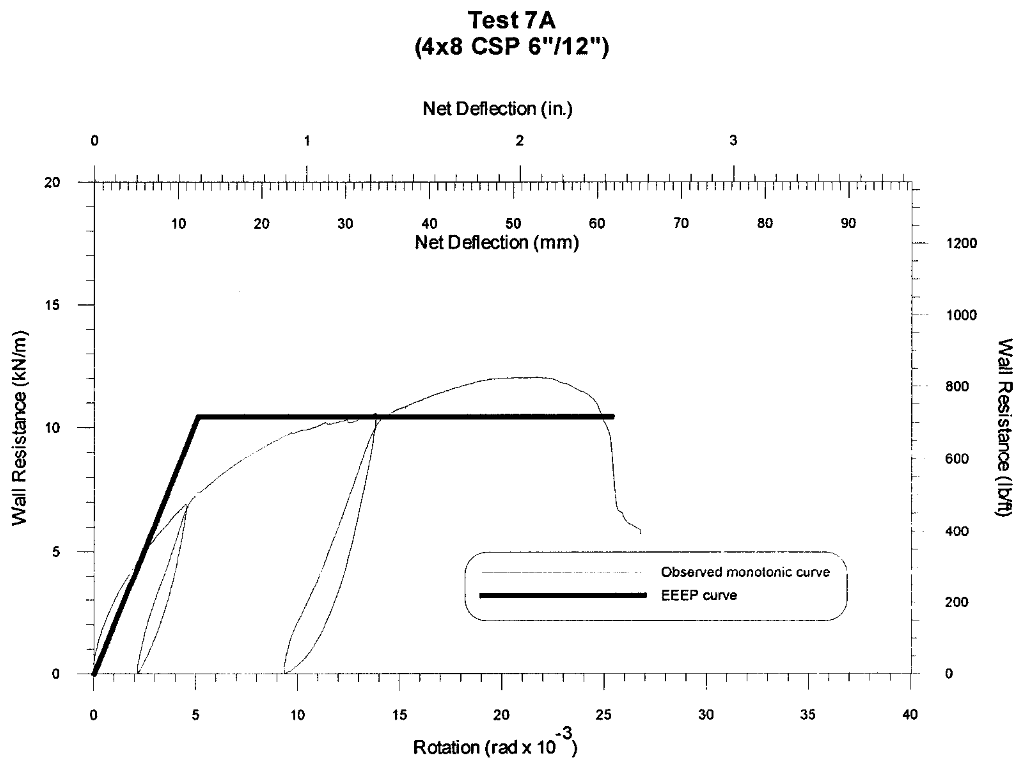

Figure 4.17: EEEP analysis for monotonic test 7A

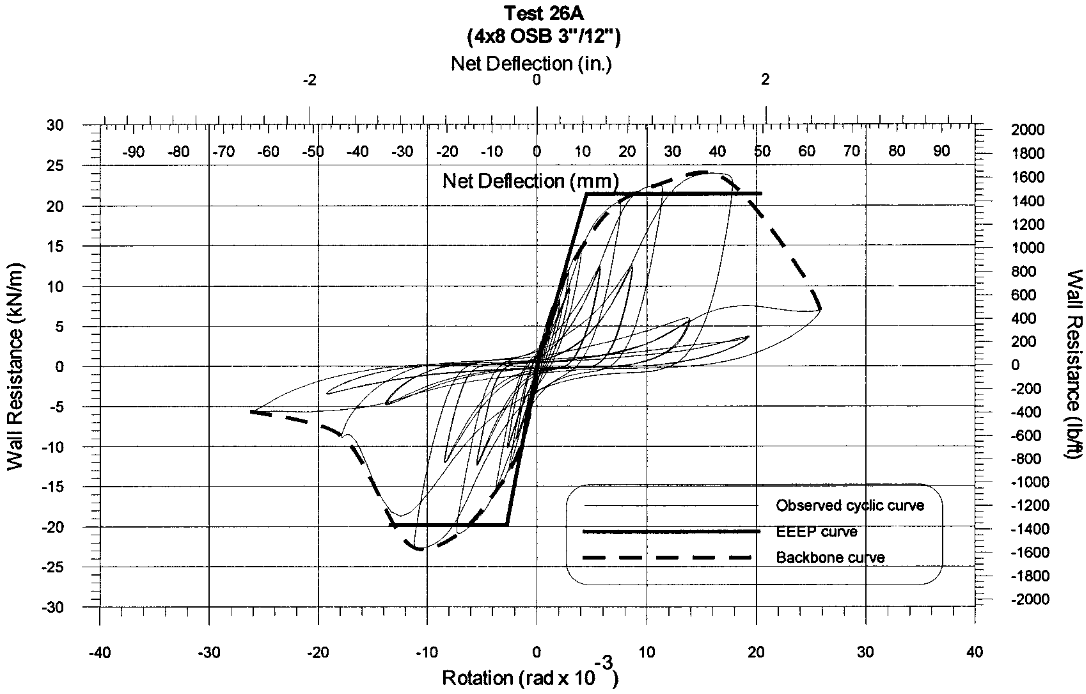

Figure 4.18: EEEP analysis for cyclic test 26A

Other design parameters of interest can also be derived once the specimen's yield resistance is determined with Equation (4-25).

$$K_e = \frac{S_y \times L}{\Delta_{net,y}} \tag{4-27}$$

$$\mu = \frac{\Delta_{net,0.8u}}{\Delta_{net,y}} \tag{4-28}$$

where,

K_e = Elastic stiffness, [force per unit length]

L = Length of the wall specimen (4' = 1220 mm)

μ = Ductility

Because light gauge steel frame / wood panel shear walls subject to lateral load are highly redundant systems, *i.e.* there are many connections between the sheathing panel and framing, the ductility of the wall specimens was defined as an ultimate ductility ratio *(Salenikovich et al., 2000b)* which is a measure of the ability of the system to carry load through plastic deformation beyond its ultimate load carrying capacity. The ductility was therefore calculated according to Equation (4-28) as a ratio between the displacement at failure and the displacement at yield. Ductility is an important performance indicator for any lateral force resisting system because it demonstrates the component's ability to withstand load while dissipating the energy of an earthquake through inelastic deformations.

4.4 DESIGN PARAMETERS RESPECTING NBCC 2005 DRIFT LIMIT

4.4.1 Serviceability Deflection Limitation

The draft version of the 2005 National Building Code of Canada *(NRCC, 2004)* requires that structural components be checked for serviceability limit states under the effect of service loads to ensure that both structural and non-structural elements will not be damaged in everyday use. Light gauge steel frame / wood panel shear walls are usually sheathed with gypsum wallboard as an interior finish. The total drift per storey under service wind loads is limited to 1/500 of the storey height, or 0.2 % in order to prevent cracking of the brittle interior finish *(NRCC, 2004)*. For a shear wall which is 8' (2440 mm) in height, this translates into an interstorey drift limit of 4.9 mm. This measure was used to gauge the serviceability performance of light gauge steel frame / wood panel shear walls subjected to the monotonic loading protocol. In order to obtain an estimate of the deflection of the tested light gauge steel frame / wood panel shear walls at a service load level, the following methodology can be applied. According to the existing NBCC 1995 *(NRCC, 1995)*,

$$\alpha_w W \propto \Phi S_y \tag{4-29}$$

$$W \propto \frac{\Phi S_y}{\alpha_w} \tag{4-30}$$

where,

α_w = Load factor for wind loads = 1.5

W = Specified wind load according the NBCC 1995

Φ = Resistance factor

S_y = Wall resistance at yield

By implementing the EEEP formulation on the test data and observing the results, it was concluded that S_y/S_u was generally in the range of 0.85. In addition, assuming that the resistance factor for light gauge steel frame / wood panel shear walls will be approximately 0.7 (Chapter 5) as is incorporated into the Engineering Design in Wood Standard *(CSA O86, 2001)* for wood frame shear walls, we can substitute into Eq. (4-30),

$$W \propto \frac{0.7 \times 0.85 \times S_u}{1.5} = 0.4 S_u \tag{4-31}$$

where,

S_u = Ultimate wall resistance from test data

The above formulation demonstrates that the 40 % of ultimate is a reasonable estimate of a wind service load level. The results of the data analysis (Table 4.4) demonstrated that the 21 wall specimens (monotonic tests) tested under the scope of this research performed reasonably well at the serviceability load level for wind loads. In most cases (10 tests), the net deflection at the top of the shear wall was limited to approximately 5 mm at a load level of $0.4S_u$, and in no case did the net deflection exceed 10 mm (Table 4.4). For the walls that were tested outside of this body of this research *(Chen, 2004)*, typically the 2' (610 mm) long walls resulted in the highest service level displacements. Additional research into the service level performance of light gauge steel frame / wood panel shear walls is necessary to properly address the use of a wind load based drift limit on design. Given the current uncertainty that exists with respect to the service limit state of these walls, design values given in Table 4.4 are not in any way limited by a serviceability (service level drift) criterion and were not adjusted based on the code prescribed limit. In design, the following calculation to determine the elastic drift under service loads would be carried out,

$$\Delta_e = \frac{W}{K_e} \tag{4-32}$$

where,

Δ_e = Elastic drift of shear wall

W = Service wind load

K_e = Elastic stiffness, [force per unit length], from Table 4.4

The designer would then compare the computed elastic deflection to the drift limit of 4.9 mm (for an 8' (2440 mm) storey height). A stiffer wall would be chosen if the criterion is not met.

4.4.2 Inelastic Interstorey Drift Limitation

The draft 2005 National Building Code of Canada *(NRCC, 2004)* requires that for seismic design, lateral deflections obtained from a linear elastic analysis be multiplied by R_dR_o/I_E to estimate the actual inelastic response of the system. R_d is the ductility related force modification factor, R_o is the overstrength related force modification factor, and I_E is the earthquake importance factor of the structure. For most structures intended for normal use, the importance factor can be taken as unity. The determination of the force modification factors falls under the scope of research conducted by Boudreault *(2004)*.

The largest interstorey deflection at any level based on the inelastic lateral displacements due to seismic loading is limited to 2.5 % of the storey height, or 0.025 h_s *(NRCC, 2004)*. This translates into an inelastic interstorey drift limit of 61 mm for an 8' (2440 mm) high shear wall. There are two cases (Figures 4.19 and 4.20) where the design of a light gauge steel frame / wood panel shear wall would be influenced by the inelastic drift limit of 61 mm: Case I: 61 mm < $\Delta_{net,u}$ (Figure

4.19) and Case II: $\Delta_{net,u}$ < 61 mm < $\Delta_{net,0.8u}$ (Figure 4.20). A third case also exists in which the failure displacement of the test specimen at $S_{0.8u}$ (post-peak) is below the seismic drift limit. In this situation, a restriction on the design capacity was not necessary and no modification to the EEEP curve procedure detailed in Figure 4.16 was utilized.

Case I: 61 mm < $\Delta_{net,u}$

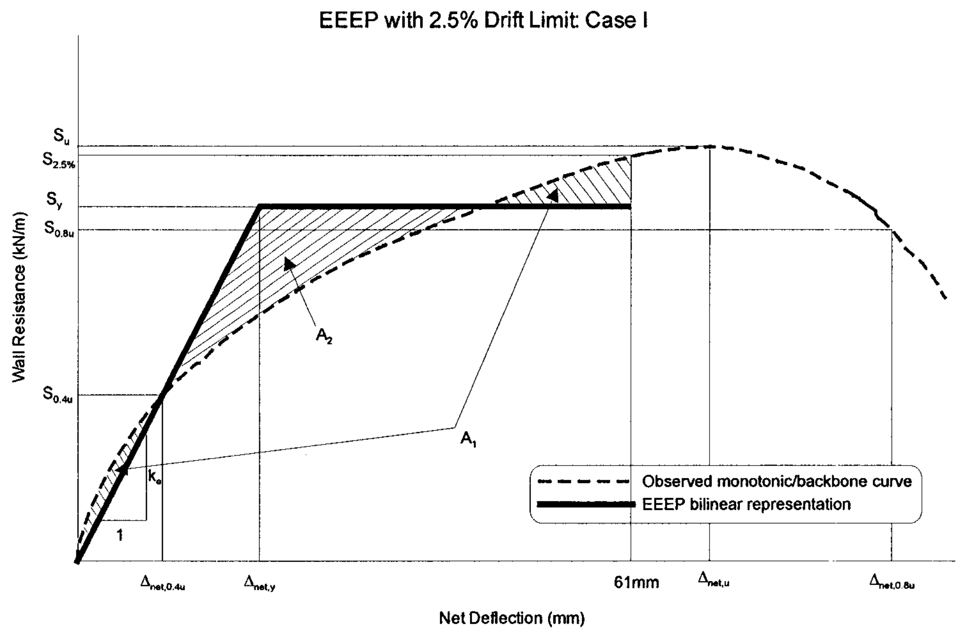

Figure 4.19: EEEP design curve with imposed 2.5 % drift limit (Case I)

In Case I, the 2.5 % inelastic drift limit (61 mm) governs the capacity of the wall. For stability reasons, the shear wall specimen is considered to have failed when it reaches this inelastic drift limit. Unlike the serviceability deflection criteria explained above for wind loads, the seismic requirement is an ultimate limit state which must be respected in order to preserve the structural integrity of the overall building during and after a design level seismic event. In this case, the area under the backbone curve was calculated up to the displacement of 61 mm. The elastic portion of the bi-linear EEEP curve was not affected by the imposed drift limit (it is still based on the secant stiffness through $0.4S_u$); however, the horizontal plastic portion is adjusted so as to equate areas A_1 and A_2. The imposed drift limit has the effect of slightly decreasing the yield wall resistance, S_y, as well as decreasing the ductility of the system, μ, compared with an approach in which no drift limit is imposed. Additionally, the force modification factors *(Boudreault, 2004)* that are to be recommended for the design of these walls are dependant upon this reduced ductility and yield wall resistance since they are based on this same EEEP approach and include the seismic inelastic drift limit. A designer would be able to use the given design capacity with confidence since, after calculating an elastic drift, and amplifying it by R_dR_o, it would still fall under the inelastic drift limit of 2.5 % of the storey height. For consistency, this drift limit was also applied to all monotonic test data; however, reliance on this inelastic shear capacity during wind loading is typically not necessary. Of the 43 tests performed under this body of research, only one test (monotonic Test 9A) required the implementation of a Case I approach. Typically, with the 2' (610 mm) walls *(Chen, 2004)* being much more flexible than the 4' (1220 mm) walls, the higher aspect ratio walls were almost all limited by the seismic inelastic drift limit.

Case II: $\Delta_{net,u} < 61\text{mm} < \Delta_{net,0.8u}$

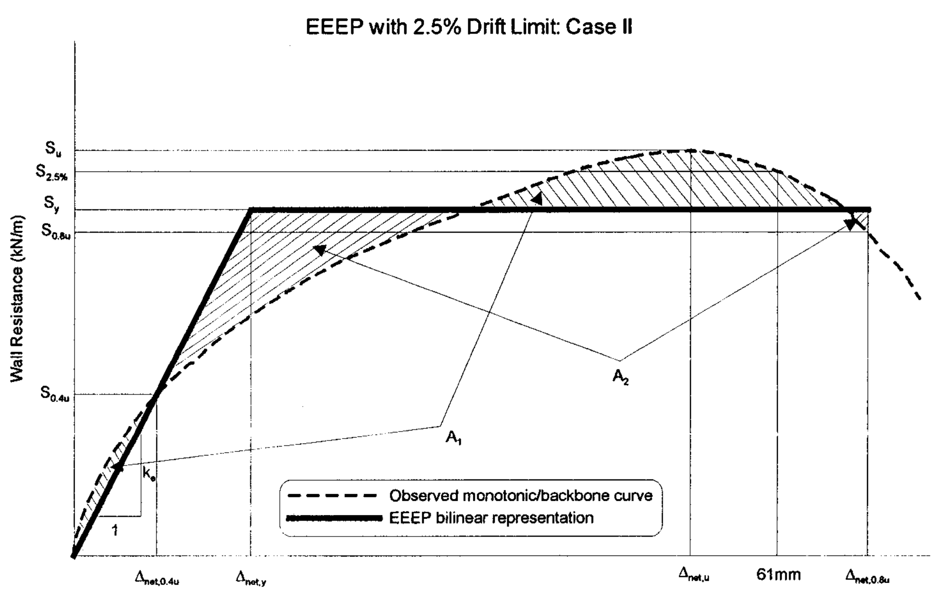

Figure 4.20: EEEP design curve with imposed 2.5 % drift limit (Case II)

In Case II, the 2.5 % inelastic drift limit was considered not to affect the design values given herein. Design values were determined by employing the usual EEEP model with failure defined at $0.8S_u$ and the elastic stiffness defined as the secant stiffness through the origin and a load level of $0.4S_u$. In Case I, the wall specimens did not reach their maximum capacity before reaching the inelastic drift limit. In Case II, the walls were able to reach their maximum capacity before reaching the inelastic drift limit and so it was assumed that the yield wall resistance would also be attained. A designer would have to ensure that the stability of the wall is maintained by applying the $0.025h_s$ verification. In order to make sure that the inelastic drift limit for seismic design of 2.5 % of the storey height is respected, a designer would calculate elastic displacements in much the same way as shown by Equation (4-32) based on an elastic base shear and the elastic stiffness given. The elastic drift would then be amplified by R_dR_o (assuming I_E = 1) to obtain an estimated inelastic response of the system. This inelastic drift could then be compared with the limit of 61 mm (for an 8' = 2440 mm high shear wall), and, if found to be below this value, then the wall would benefit slightly in terms of its design strength and ductility. The wall would be able to attain its maximum shear capacity prior to reaching the drift limit, and hence would be able to develop the shear yield resistance as well. In an attempt to simplify the data interpretation procedure, use of the drift limit was not considered necessary in this situation. It is possible that the wall will displace past the 2.5 % drift limit during large excursions into the inelastic zone; however, the designer would be able to gauge the expected wall behaviour based on the inter-storey drift limit and select a stiffer wall configuration if required.

Design values resulting from monotonic tests and from reversed cyclic tests are shown in Tables 4.4, 4.5, and 4.6. For monotonic tests, deflections at a load level of $0.4S_u$ are also shown so that the designer would be able to have a good estimate of the deflections that are expected to occur at service wind loading. It must be noted that nominal yield wall resistances, S_y, have been listed which must be multiplied by an appropriate resistance factor to obtain the factored resistance of the shear wall for design. In addition, the values listed below are for lateral loading only; that is, there is no compensation applied to the design values for the possibility of compression chord failure under combined lateral and gravity loading. The designer should be aware that the compression chord local buckling failure mode does exist (Test 13A, 13B, 13C) and that it may control the maximum applied lateral load in the presence of gravity loads. Further research is required to determine the behaviour of light gauge steel frame shear walls under combined gravity and lateral loading A complete determination of the resistance factor for design is outlined in Chapter 5 and the complete design procedure according to limit states design is also summarized.

Table 4.4: Design values resulting from monotonic tests

Test	Panel Type	Fastener Schedule	Yield Load (S_y) kN/m	Displacement at $0.4S_u$ ($\Delta_{net, 0.4u}$) mm	Displacement at S_y ($\Delta_{net, y}$) mm	Elastic Stiffness (K_e) kN/mm	Rotation at S_y ($\theta_{net, y}$) rad	Ductility μ	Energy Dissipation, E Joules
7A	CSP	6"/12"	10.4	5.7	12.4	1.03	0.0051	4.98	706
7B	CSP	6"/12"	11.1	5.6	12.3	1.10	0.0050	5.68	857
7C	CSP	6"/12"	11.9	6.6	14.3	1.01	0.0059	4.89	912
AVERAGE	**CSP**	**6"/12"**	**11.1**	6.0	**13.0**	**1.05**	**0.0053**	**5.18**	**825**
9A[1]	CSP	3"/12"	22.7	9.1	18.9	1.46	0.0077	3.23	1423
9B	CSP	3"/12"	20.3	9.4	20.2	1.22	0.0083	3.17	1336
9C	CSP	3"/12"	21.5	9.2	20.1	1.31	0.0082	3.39	1519
AVERAGE	**CSP**	**3"/12"**	**21.5**	9.2	**19.7**	**1.33**	**0.0081**	**3.26**	**1426**
11A	DFP	6"/12"	13.5	7.8	16.6	0.99	0.0068	4.47	1081
11B	DFP	6"/12"	14.1	7.3	15.3	1.13	0.0063	4.65	1090
11C	DFP	6"/12"	13.3	7.3	15.8	1.03	0.0065	4.04	910
AVERAGE	**DFP**	**6"/12"**	**13.6**	7.5	**15.9**	**1.05**	**0.0065**	**4.39**	**1027**
13A	DFP	3"/12"	24.2	9.0	19.4	1.52	0.0079	3.13	1500
13B	DFP	3"/12"	25.1	9.1	18.4	1.66	0.0076	3.45	1663
13C	DFP	3"/12"	24.8	9.3	19.0	1.60	0.0078	3.35	1638
AVERAGE	**DFP**	**3"/12"**	**24.7**	9.1	**18.9**	**1.59**	**0.0078**	**3.31**	**1600**
21A	OSB	6"/12"	12.0	3.4	7.6	1.91	0.0031	7.18	747
21B	OSB	6"/12"	11.6	3.6	8.0	1.77	0.0033	5.88	606
21C	OSB	6"/12"	11.7	3.9	8.6	1.66	0.0035	7.22	830
AVERAGE	**OSB**	**6"/12"**	**11.8**	3.7	**8.1**	**1.78**	**0.0033**	**6.76**	**727**
23A	OSB	4"/12"	17.4	5.0	11.3	1.87	0.0046	4.57	977
23B	OSB	4"/12"	18.1	4.8	10.7	2.06	0.0044	4.65	978
23C	OSB	4"/12"	16.6	3.9	8.7	2.33	0.0035	5.42	860
AVERAGE	**OSB**	**4"/12"**	**17.3**	4.6	**10.2**	**2.09**	**0.0042**	**4.88**	**938**
25A	OSB	3"/12"	20.6	5.1	11.0	2.29	0.0045	3.88	932
25B	OSB	3"/12"	20.0	7.2	16.1	1.51	0.0066	3.10	1016
25C	OSB	3"/12"	21.9	5.8	12.8	2.08	0.0053	3.75	1110
AVERAGE	**OSB**	**3"/12"**	**20.8**	6.0	**13.3**	**1.96**	**0.0054**	**3.58**	**1019**

[1] Test 9A capacity governed by 2.5% inelastic drift limit (Case I)

Table 4.5: Design values resulting from reversed cyclic tests (positive cycles)

Test	Panel Type	Fastener Schedule	Yield Load (S_{y+}) kN/m	Displacement at S_{y+} ($\Delta_{net,\ y+}$) mm	Elastic Stiffness (K_{e+}) kN/mm	Rotation at S_{y+} ($\theta_{net,\ y+}$) rad	Ductility μ	Energy Dissipation[1] E Joules
8A	CSP	6"/12"	10.6	10.0	1.30	0.0041	6.47	770
8B	CSP	6"/12"	10.4	10.3	1.24	0.0042	6.29	754
8C	CSP	6"/12"	10.4	10.1	1.25	0.0042	6.46	764
AVERAGE	**CSP**	**6"/12"**	**10.5**	**10.1**	**1.26**	**0.0041**	**6.41**	**763**
10A	CSP	3"/12"	22.9	15.6	1.79	0.0064	4.33	1659
10B	CSP	3"/12"	23.3	20.1	1.41	0.0082	3.39	1647
10C	CSP	3"/12"	21.6	16.3	1.61	0.0067	3.64	1348
AVERAGE	**CSP**	**3"/12"**	**22.6**	**17.3**	**1.61**	**0.0071**	**3.79**	**1551**
12A	DFP	6"/12"	11.8	10.3	1.41	0.0042	6.01	817
12B	DFP	6"/12"	13.7	14.4	1.16	0.0059	4.54	967
12C	DFP	6"/12"	12.4	14.6	1.03	0.0060	4.50	885
AVERAGE	**DFP**	**6"/12"**	**12.6**	**13.1**	**1.20**	**0.0054**	**5.02**	**890**
14A	DFP	3"/12"	26.6	18.7	1.74	0.0076	3.58	1860
14B	DFP	3"/12"	24.8	20.7	1.46	0.0085	3.08	1616
14C	DFP	3"/12"	25.8	18.4	1.72	0.0075	4.16	2113
14D	DFP	3"/12"	25.1	19.4	1.58	0.0080	3.23	1624
AVERAGE	**DFP**	**3"/12"**	**25.6**	**19.3**	**1.62**	**0.0079**	**3.51**	**1803**
22A	OSB	6"/12"	10.4	7.1	1.78	0.0029	7.77	659
22B	OSB	6"/12"	11.1	7.9	1.71	0.0032	7.59	754
22C	OSB	6"/12"	10.6	7.4	1.75	0.0030	6.77	596
AVERAGE	**OSB**	**6"/12"**	**10.7**	**7.5**	**1.75**	**0.0031**	**7.38**	**669**
24A	OSB	4"/12"	15.0	8.1	2.25	0.0033	4.43	583
24B	OSB	4"/12"	15.9	8.0	2.43	0.0033	6.32	894
24C	OSB	4"/12"	15.7	8.2	2.34	0.0034	5.40	767
AVERAGE	**OSB**	**4"/12"**	**15.5**	**8.1**	**2.34**	**0.0033**	**5.38**	**748**
26A	OSB	3"/12"	21.5	11.0	2.39	0.0045	4.51	1149
26B	OSB	3"/12"	20.2	9.6	2.56	0.0039	4.33	905
26C	OSB	3"/12"	20.9	11.1	2.28	0.0046	4.80	1219
AVERAGE	**OSB**	**3"/12"**	**20.8**	**10.6**	**2.41**	**0.0043**	**4.55**	**1091**

[1] Energy calculation based on area below backbone curve

Table 4.6: Design values resulting from reversed cyclic tests (negative cycles)

Test	Panel Type	Fastener Schedule	Yield Load (S_{y-}) kN/m	Displacement at S_{y-} ($\Delta_{net,\,y-}$) mm	Elastic Stiffness (K_{e-}) kN/mm	Rotation at S_{y-} ($\theta_{net,\,y-}$) rad	Ductility μ	Energy Dissipation[1] E Joules
8A	CSP	6"/12"	-9.8	-12.6	0.95	-0.0052	5.00	679
8B	CSP	6"/12"	-9.7	-11.5	1.03	-0.0047	4.41	533
8C	CSP	6"/12"	-9.4	-12.9	0.89	-0.0053	4.75	629
AVERAGE	**CSP**	**6"/12"**	**-9.6**	**-12.3**	**0.96**	**-0.0051**	**4.72**	**614**
10A	CSP	3"/12"	-20.7	-17.3	1.46	-0.0071	3.29	1219
10B	CSP	3"/12"	-20.7	-16.1	1.57	-0.0066	3.78	1329
10C	CSP	3"/12"	-20.8	-22.0	1.15	-0.0090	2.15	918
AVERAGE	**CSP**	**3"/12"**	**-20.7**	**-18.5**	**1.39**	**-0.0076**	**3.07**	**1155**
12A	DFP	6"/12"	-10.8	-11.2	1.17	-0.0046	5.51	740
12B	DFP	6"/12"	-12.8	-12.8	1.22	-0.0053	4.93	887
12C	DFP	6"/12"	-11.5	-9.7	1.45	-0.0040	4.95	603
AVERAGE	**DFP**	**6"/12"**	**-11.7**	**-11.2**	**1.28**	**-0.0046**	**5.13**	**744**
14A	DFP	3"/12"	-24.6	-17.0	1.77	-0.0070	4.07	1826
14B	DFP	3"/12"	-19.7	-12.1	1.98	-0.0050	3.78	950
14C	DFP	3"/12"	-24.1	-17.0	1.73	-0.0070	4.11	1809
14D	DFP	3"/12"	-23.4	-15.9	1.80	-0.0065	4.24	1689
AVERAGE	**DFP**	**3"/12"**	**-23.0**	**-15.5**	**1.82**	**-0.0063**	**4.05**	**1569**
22A	OSB	6"/12"	-9.7	-8.6	1.36	-0.0035	5.82	540
22B	OSB	6"/12"	-9.8	-7.5	1.59	-0.0031	6.88	577
22C	OSB	6"/12"	-9.4	-8.5	1.34	-0.0035	4.88	427
AVERAGE	**OSB**	**6"/12"**	**-9.6**	**-8.2**	**1.43**	**-0.0034**	**5.86**	**515**
24A	OSB	4"/12"	-15.2	-9.3	1.99	-0.0038	3.74	561
24B	OSB	4"/12"	-14.5	-8.4	2.10	-0.0034	5.98	811
24C	OSB	4"/12"	-14.4	-8.6	2.04	-0.0035	4.15	550
AVERAGE	**OSB**	**4"/12"**	**-14.7**	**-8.8**	**2.04**	**-0.0036**	**4.62**	**640**
26A	OSB	3"/12"	-19.8	-6.7	3.59	-0.0028	4.86	708
26B	OSB	3"/12"	-19.7	-10.0	2.40	-0.0041	3.14	637
26C	OSB	3"/12"	-20.8	-9.9	2.57	-0.0040	5.18	1170
AVERAGE	**OSB**	**3"/12"**	**-20.1**	**-8.9**	**2.85**	**-0.0036**	**4.39**	**839**

[1] **Energy calculation based on area below backbone curve**

CHAPTER 5 DEVELOPMENT OF A LIMIT STATES DESIGN PROCEDURE

In addition to the design parameters for the 43 wall specimens listed in Chapter 4, this Chapter incorporates data from companion bodies of research *(Boudreault, 2004; Chen, 2004)*, which was interpreted according to the same methodology as presented in Sections 4.3 and 4.4 (EEEP model with imposed drift limits). Using the design values for yield wall resistance (S_y) (nominal strength) *(Branston et al., 2004)*, this Chapter presents the calibration of the resistance factor for light gauge steel frame / wood panel shear walls and outlines the overall prescriptive design procedure to be followed consistent with the upcoming 2005 edition of the National Building Code of Canada *(NRCC, 2004)*.

5.1 RESISTANCE FACTOR CALIBRATION FOR THE DRAFT NBCC 2005

The basis of limit states design in which the factored resistance of a structural member, element, connection, or assembly shall be greater than the effect of the factored loads, is outlined in Eq. (5-1):

$$\Phi R \geq \sum \alpha S \tag{5-1}$$

where,

Φ = Resistance factor

R = Resistance of structural member (nominal strength)

α = Load factor (representing an overall load factor for a combination of loads)

S = Effect of some particular combination of loads (specified)

Denoting X as the difference between the nominal strength and specified load value for every combination of load and resistance,

$$X = R - S \tag{5-2}$$

and plotting the frequency distribution of $\ln\left(\frac{R}{S}\right)$ (Figure 5.1),

$$\ln X = \ln R - \ln S = \ln\left(\frac{R}{S}\right) \tag{5-3}$$

we can define the reliability or safety index, β_o, as:

$$\beta_O = \frac{\ln\left(\frac{\bar{R}}{\bar{S}}\right)}{\sigma_{\ln\left(\frac{R}{S}\right)}} \tag{5-4}$$

which provides a relative measure of the separation between the mean value of $\ln\left(\frac{R}{S}\right)$ and the point of failure of the element under consideration. In Eq. (5-4), the variables are defined as:

β_o = Reliability/safety index representing the number of standard deviations from 0 to the mean value (Figure 5.1)

$\sigma_{\ln\left(\frac{R}{S}\right)}$ = Standard deviation

$\bar{R}$ = Mean value of resistance

$\bar{S}$ = Mean value of load effect

Defining the variance (σ^2) with respect to the individual variance on R and S,

$$\sigma_{\ln\left(\frac{R}{S}\right)}^{\;2} \cong \frac{\partial \ln\left(\frac{\bar{R}}{\bar{S}}\right)^2}{\partial R}\sigma_R^{\;2} + \frac{\partial \ln\left(\frac{\bar{R}}{\bar{S}}\right)^2}{\partial S}\sigma_S^{\;2} = \frac{\sigma_R^{\;2}}{\bar{R}^2} + \frac{\sigma_S^{\;2}}{\bar{S}^2} = V_R^{\;2} + V_S^{\;2} \tag{5-5}$$

where,

V_R = Coefficient of variation of the assembly resistance

V_S = Coefficient of variation of the load effect

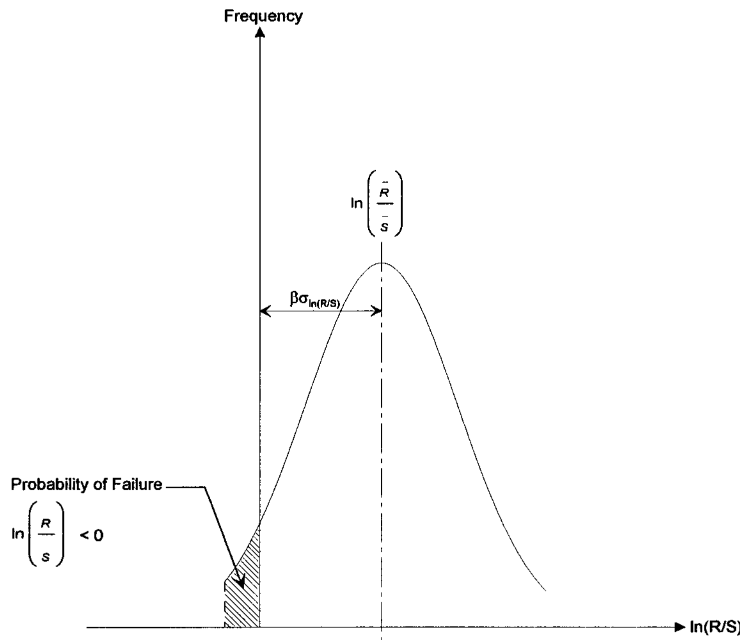

Figure 5.1: Frequency distribution for ln(*R*/*S*) depicting β_o value *(Kennedy and Gad Aly, 1980)*

We can substitute Eq. (5-5) into Eq. (5-4) to obtain,

$$\beta_o\sqrt{V_R^{\ 2}+V_S^{\ 2}} = \ln\left(\frac{\bar{R}}{\bar{S}}\right) \tag{5-6}$$

$$\left(\frac{\bar{R}}{\bar{S}}\right) = e^{\beta_o\sqrt{V_R^{\ 2}+V_S^{\ 2}}} \tag{5-7}$$

$$\bar{S} = \left(\frac{\bar{R}}{e^{\beta_o\sqrt{V_R^{\ 2}+V_S^{\ 2}}}}\right) \tag{5-8}$$

Making use of the relation,

$$S = \bar{S}\frac{S}{\bar{S}} \tag{5-9}$$

and substituting Eq. (5-9) into Eq. (5-1),

$$\Phi R = \alpha\frac{S}{\bar{S}}\bar{R}e^{-\beta_o\sqrt{V_R^{\ 2}+V_S^{\ 2}}} \tag{5-10}$$

$$\Phi = \alpha \frac{S}{\bar{S}} \frac{\bar{R}}{R} e^{-\beta_o \sqrt{V_R^2 + V_S^2}} \quad (5\text{-}11)$$

$$\Phi = \frac{\alpha}{\bar{S}/S} \frac{\bar{R}}{R} e^{-\beta_o \sqrt{V_R^2 + V_S^2}} \quad (5\text{-}12)$$

Let,

$$C_\Phi = \frac{\alpha}{\bar{S}/S} \quad (5\text{-}13)$$

$$M_m F_m P_m = \frac{\bar{R}}{R} \quad (5\text{-}14)$$

$$V_M^2 + V_F^2 + C_P V_P^2 = V_R^2 \quad (5\text{-}15)$$

and substituting these relations into Eq. (5-12), one obtains,

$$\Phi = C_\Phi (M_m F_m P_m) e^{-\beta_o \sqrt{V_M^2 + V_F^2 + C_P V_P^2 + V_S^2}} \quad (5\text{-}16)$$

where,

C_Φ = Calibration coefficient

M_m = Mean value of material factor for type of component involved

F_m = Mean value of fabrication factor for type of component involved

P_m = Mean value of professional factor for tested component

V_M = Coefficient of variation of material factor

V_F = Coefficient of variation of fabrication factor

V_P = Coefficient of variation of professional factor

C_P = Correction factor for sample size = $(1+1/n)m/(m-2)$ for $n \geq 4$, and 5.7 for n=3

m = Degrees of freedom = $n - 1$

n = number of tests (sample size)

e = Natural logarithmic base = 2.718...

Eq. (5-16) is the same as that shown in the North American Specification for the Design of Cold-Formed Steel Structural Members *(AISI, 2002a)*. This

Specification supplies provisions to determine the structural performance of assemblies, members, or connections in which not fewer than three identical specimens are tested and the result of any individual test does not differ from the average value of all tests by more than 15 %. The CSA Guidelines for the Development of Limit States Design *(CSA S408, 1981)* also presents the derivation in a comparable manner. A similar procedure was used by the Steel Deck Institute (SDI) when calibrating the resistance factor for the design of steel deck diaphragms *(SDI, 1981)*. The procedure used by the SDI, however, did not include any factors to account for the mean-to-nominal load effect factor (calibration coefficient (C_Φ), nor the variation in the load effect (V_S). The coefficient of variation of the professional factor was included, however, an amplification factor was not included to correct for the sample size (C_P). The equation outlined by Eq. (5-16) is therefore more refined in nature due, in part, to research conducted by Pekoz and Hall *(1988)* and Tsai *(1992)*, accounting both for the variability of the load effects as well as the testing sample size. It should be noted that as the testing sample size grows, the correction factor, C_P, approaches unity thereby rendering the correction factor negligible.

The various parameters which are grouped to make up the ratio of the mean-to-nominal resistance (Eq. (5-14)) are based on a statistical analysis of the material properties (M_m) and fabrication or dimensional properties (F_m) of the members that form the structural assembly, and the professional properties that are dependant upon the tests conducted. Furthermore, given that the 2005 NBCC *(NRCC, 2004)* will require a capacity based design approach for seismic loading, the selected fuse (failure) element / member in a shear wall was chosen to be the connection between the wood sheathing and the steel framing. These connections failed in a ductile mode for all but three of the tests that were carried out (Section 3.7). Hence, the material properties and dimensional properties of the entire wall assembly are highly dependant upon the strength and thickness of the plywood or OSB sheathing. Table F1 of the North American Specification *(AISI, 2002a)* provides statistical data for the determination of the resistance factor, particularly values for M_m, V_M, F_m, and V_F. The values are listed for many types of steel members and connections, however, not for light gauge steel frame / wood panel shear walls subject to lateral loading. The values for the material factor and the fabrication factor are typically in the range of 1.00 – 1.10 and account for variability in the material such as the ratio of the average actual strength of a material to the specified nominal strength. The fabrication factor would account for the ratio of the actual measured dimensions to the specified nominal dimensions for a large database of test specimens. A conservative approach would be to take both factors (M_m, F_m) equal to unity, *i.e.* no oversize or overstrength would be considered, thereby negating their effect on the calculation of the resistance factor.

The commentary to CSA O86 *(CWC, 2001)* does comment on the nominal strength values for structural-use panels and indicates that the fifth percentile exclusion values were calculated based on the mean value of the observed strength values and the coefficient of variation. The resulting fifth percentile values were then quoted as the nominal or specified strength to be used for design. Considering this, since the nominal strength values are derived based on the lower end of the normal frequency distribution curve (lower strength values), it was decided that a small amount of overstrength would be acceptable in the calculation of the resistance factor for light gauge steel frame / wood panel shear walls. A 5 % increase was chosen to be reasonable to account for this overstrength in the sheathing material, and so a value M_m = 1.05 was used in the calibration. For the fabrication factor, a conservative value of F_m = 1.00 was used for the calibration since it was assumed that given a large number of panel specimens, some thicknesses would be greater than the nominal thickness while some would be lesser, thereby causing the mean-to-nominal ratio to be in the vicinity of 1.00. This distribution on both sides of the nominal thickness would be expected due to the nature of the thickness tolerances specified in the product and performance standards for structural-use panels in Canada. For example, a thickness tolerance of -0.5 mm to +1.0 mm (compared to nominal) is specified in both the performance standard for OSB panels *(CSA O325,* *1992)* and the product standard for DFP *(CSA O121, 1978)*. The commentary to CSA O86 *(CWC, 2001)* also contains a summary of research into the reliability of structural-use panels including plywood and OSB reported by the Council of Forest Industries of British Columbia (COFI) *(Parasin, 1988)*, Foschi *(1992)* and Karacabeyli and Lum *(1999)*. It was found that for plywood the strength distribution followed a normal distribution, while for OSB, a two-parameter Weibull distribution was more appropriate. Both distributions resulted in a coefficient of variation of approximately 15 %. Taking this information into account, and basing the strength values on both material properties and dimensional properties, the coefficients of variation for the material factor (V_M) and the fabrication factor (V_F) were taken to be 0.11 and 0.10, respectively. When combined, these would provide for a total coefficient of variation of approximately 15 %, which would match the findings of the studies referred to above.

In terms of the professional factor (P_m) and the corresponding coefficient of variation (V_P), all test data, including the 43 wall specimens detailed in this thesis, as well as those tested by Boudreault *(2004)* and Chen *(2004)* for the main testing program, was gathered and analyzed. It was decided that the 2' (610 mm) long shear walls would not be included in the calibration of the resistance factor, nor would they be included as available design values for use by designers. The 2' (610 mm) long light gauge steel frame / wood panel shear walls were found to be too flexible to carry the specified design value at a similar displacement level as the 4' (1220 mm) and 8' (2440 mm) long walls *(Chen, 2004)*. This is of primary concern when segments of shear walls separated by perforations to accommodate for windows or doors are connected in sequence. Since the wall will displace as a unit and it is likely that each shear wall will be subject to an equal displacement level, the walls must be capable of developing the specified capacity at similar displacement levels for the design approach to be applicable.

Shear wall nominal capacities (S_y) were compiled for all 4' (1220 mm) and 8' (2440 mm) long walls (not including 1-D,E,F) tested both monotonically and cyclically in the main testing program. In total, nominal values for 78 tests were analyzed and included in the calibration of the resistance factor. Since the CUREE reversed cyclic protocol for ordinary ground motions produces results that are very similar to those revealed by a monotonic test for an identical wall configuration (Figure 5.2), it was decided that the results for the monotonic tests and the reversed cyclic tests would be combined to produce a minimum of six nominal shear values for each wall configuration. Typical of the CUREE reversed cyclic protocol is a non-symmetric resulting hysteresis plot in which the negative cycles do not reach a similar load level as the positive cycles. This is due to the wall first failing on a large excursion to a displacement amplitude on the positive side, and upon returning to the equivalent negative amplitude, the wall's capacity is severely reduced. It was therefore decided that an average value of the positive cycle nominal capacity (S_{y+}) and the negative cycle nominal capacity (S_{y-}) would provide a conservative estimate of the overall cyclic nominal capacity of the wall, however, the positive cycle nominal capacity alone was also considered for the calibration of the resistance factor to provide a level of comparison.

The monotonic and reversed cyclic nominal capacities were then combined to produce an average value for the nominal shear capacity ($S_{y,avg}$) of a given wall configuration with equal weight given to the monotonic and cyclic results (50 % each) as demonstrated by Equations (5-17) and (5-18). This weighting was important when the average of the positive (S_{y+}) and negative (S_{y-}) nominal cyclic capacities were averaged with the monotonic nominal capacities (Eq. (5-18)) or when more than 3 monotonic or cyclic tests were run for a specific wall configuration, *e.g.* tests 14-A,B,C,D.

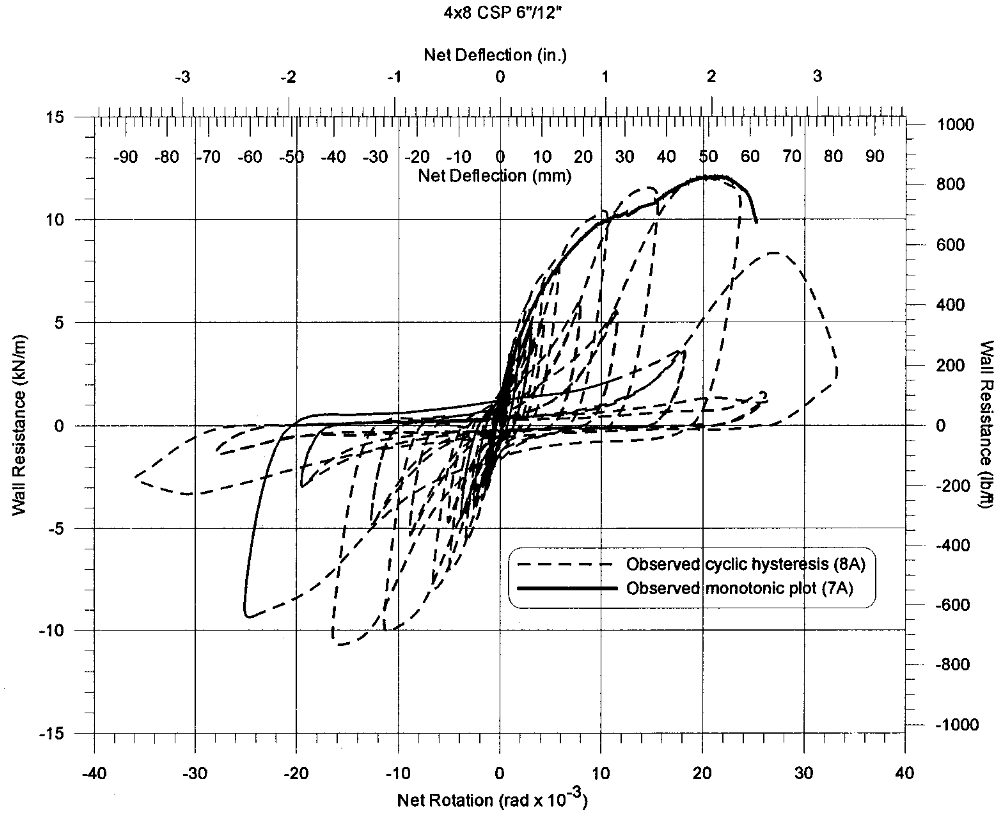

Figure 5.2: Superimposed monotonic and reversed cyclic results for 4' x 8' (1220 x 2440 mm) CSP 6"/12" (150/305 mm) (monotonic shown for positive cycles only)

Considering the monotonic test results combined with the positive cycle nominal capacity only,

$$S_{y,avg} = \frac{S_{y,mono,avg} + S_{y+,avg}}{2} \tag{5-17}$$

or considering the average of the positive and negative cycle nominal capacities combined with the monotonic test results,

$$S_{y,avg} = \frac{S_{y,mono,avg} + \dfrac{S_{y+,avg} + S_{y-,avg}}{2}}{2} \tag{5-18}$$

where $S_{y,mono,avg}$, $S_{y+,avg}$, $S_{y-,avg}$ are the average values for each wall configuration (minimum three tests) for the monotonic, positive cycle, and negative cycle tests respectively.

For each of the 12 wall configurations, the individual test results were compared to the average nominal result ($S_{y,avg}$) (minimum of six tests) to produce a ratio of $S_y/S_{y,avg}$, similar to a test-to-predicted ratio when a predicting equation is available. Additionally, the frequency distributions of $S_y/S_{y,avg}$ for the various groupings considered, *i.e.* all tests, 3" (75 mm) screw spacing, 4" (100 mm) screw spacing, *etc.*, generally followed a normal distribution (Figure 5.3). This was the case when considering the positive cycles alone or the average of the positive and negative cycles, and hence the mean (P_m) and coefficient of variation (V_P) of the various distributions were calculated as shown by Equations (5-19) and (5-20).

$$P_m = \frac{\sum_{i=1}^{n}\left(S_y / S_{y,avg}\right)_i}{n} \tag{5-19}$$

$$V_P = \frac{\sigma}{P_m} \tag{5-20}$$

where,

$$\sigma^2 = \frac{1}{n-1}\sum_{i=1}^{n}\left[\left(S_y / S_{y,avg}\right)_i - P_m\right]^2 \tag{5-21}$$

where,

n = number of test results included in configuration grouping considered

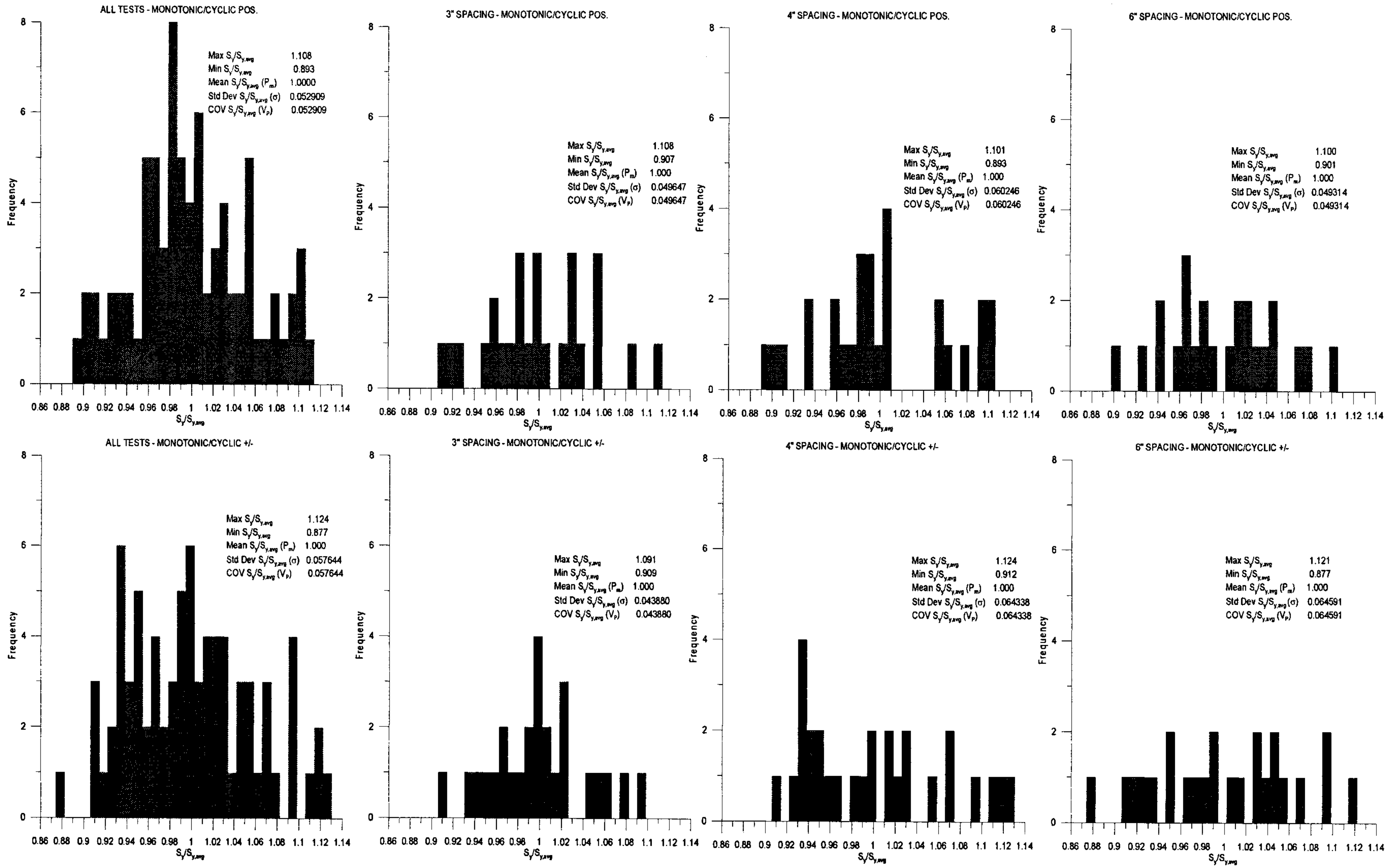

Figure 5.3: Frequency distributions for $S_y/S_{y,avg}$ for the wall configuration groupings considered

5.1.1 Calibration for Draft 2005 NBCC Wind Loads

The statistics of loads and load effects have been researched and documented by many researchers *(Allen, 1975)*, however, in the area of wind loads, Ellingwood *(Ellingwood et al., 1980; Ellingwood and Tekie, 1999)* and Bartlett *(Bartlett et al., 2003)* have taken a leading role in the research development. Ellingwood *(Ellingwood et al., 1980)* developed the wind load statistics based on the 50 year maximum wind speed, consistent with the draft version of the 2005 NBCC *(NRCC, 2004)* (the 1 in 50 year value, which has a return period of 50 years, will replace the 1 in 30 year value used with the 1995 NBCC for wind and snow loads *(NRCC, 1995)*). Ellingwood based his procedure on a generic wind load formula which, besides nomenclature differences, is identical to that given in the draft 2005 NBCC as shown by Equation (5-22):

$$p = I_w q_{1/50} C_e C_g C_p \tag{5-22}$$

where,

p = Specified static pressure on surface of building, either acting towards the surface or as a suction directed away from the surface

I_w = Importance factor for wind load = 1.0 for normal buildings

$q_{1/50}$ = 1 in 50 year reference velocity pressure

C_e = Exposure factor

C_g = Gust effect factor

C_p = External pressure coefficient

and, in addition,

$$q_{1/50} = \frac{1}{2} \rho V_{1/50}^{\ 2} \tag{5-23}$$

where,

ρ = Density of air = 1.2929 kg/m^3 for dry air at 0°C and standard atmosphere

$V_{1/50}$ = 1 in 50 year wind velocity

Because the 50 year return period velocity enters Eq. (5-22) as a squared value, its statistics will have the largest impact on the overall statistics of the wind pressure. Ellingwood fit an Extreme Value Type I probability distribution to the extreme annual wind speeds for seven sites in the United States. These sites included Baltimore, Detroit, St-Louis, Austin, Tucson, Rochester, and Sacramento. It was found that these sites provided appropriate coverage for the range of values available for over one hundred sites, and a vast geographical representation was achieved. After deriving the mean maximum 50 year wind speed from the extreme annual wind speed and combining the means and coefficients of variation for C_e, C_g, C_p, and $V_{1/50}$, a Type I distribution was fitted to the wind load data and a composite set of statistical estimates was drawn implying that:

$$\frac{\bar{p}}{p} = 0.78 \tag{5-24}$$

$$V_S = 0.37 \tag{5-25}$$

Bartlett *(Bartlett et al., 2003)* reported on the calibration of load factors for the draft 2005 NBCC *(NRCC, 2004)*. The first of a set of companion papers discusses the load effects and load statistics for dead load, live load, snow load, and wind load that have been adopted for calibration. Bartlett also considered wind loads using 50 year values consistent with the draft version of the new building code of Canada. Similar to Ellingwood *(Ellingwood et al., 1980)*, Bartlett derives the 50 year maximum wind speed from the extreme annual values at various sites. Bartlett bases his statistics, however, on sites in Canada. Wind velocity was considered as a random variable and fitted with a Gumbel distribution for which the mean-to-nominal ratio of the 50 year maximum wind speed can be derived from the coefficient of variation of the maximum annual wind speed as shown by Eq. (5-26):

$$\frac{\bar{V}_{1/50}}{V_{1/50}} = \frac{1 + 3.050COV_A}{1 + 2.592COV_A} \tag{5-26}$$

where,

$\frac{\bar{V}_{1/50}}{V_{1/50}}$ = Mean-to-nominal ratio of 50 year maximum wind speed

COV_A = Coefficient of variation of maximum annual wind speed

Annual maximum wind velocity data for 311 sites across Canada were considered in the derivation of 50 year wind speeds. The coefficient of variation of annual wind speed varied from 0.028 to a maximum of 0.394, with an overall mean of 0.134. The wind data for Halifax (COV_A = 0.150) represented a coefficient of variation slightly larger than the mean for the 311 sites. Deriving the mean-to-nominal ratio of the 50 year maximum wind speed using Eq. (5-26), one obtains:

$$\frac{\bar{V}_{1/50}}{V_{1/50}} = \frac{1+3.050(0.150)}{1+2.592(0.150)} = 1.049 \tag{5-27}$$

with a $COV_{1/50} = 0.103$.

According to Equations (5-22) and (5-23), the wind velocity must first be converted to a pressure and then transformed by the exposure factor (C_e), the gust effect factor (C_g) and the pressure coefficient (C_p) to obtain the wind load effect on the surface of a building. Bartlett proposes bias values (mean-to-nominal ratios) for the exposure coefficient and the combined gust and pressure coefficient (C_gC_p) to be 0.80 and 0.85, respectively. The corresponding coefficients of variation are 0.16 and 0.15, respectively. Assuming that all factors are independent random variables and that the constants inherent in Eq. (5-22) have negligible variation, we can compute, as an example since its statistical values are similar to those for all 311 sites across Canada, the overall mean-to-nominal ratio of the wind load in Halifax to be:

$$\frac{\bar{p}}{p} = \frac{\bar{C}_e}{C_e}\frac{\bar{C}_g\,\bar{C}_p}{C_gC_p}\left(\frac{\bar{V}_{1/50}}{V_{1/50}}\right)^2 = 0.80\times0.85\times1.049^2 = 0.748 \tag{5-28}$$

and the overall coefficient of variation for the 50 year mean-to-nominal wind load to be:

$$V_S = \sqrt{0.16^2 + 0.15^2 + 0.103^2} = 0.242 \tag{5-29}$$

Comparing these values to those obtained in the analysis by Ellingwood *(Ellingwood et al., 1980)*, in terms of determining the resistance factor according to Eq. (5-16), a less conservative approach would occur despite the similarity in the mean-to-nominal ratio of the wind load. In physical terms, the lower coefficient of variation implies much less variation in the wind load and less risk.

For the subsequent calibration of the resistance factor for light gauge steel frame / wood panel shear walls subject to wind loads, a conservative approach is taken in which a value of 0.76 is used for the mean-to-nominal ratio of the wind load, along with a coefficient of variation of 0.37 (V_S). Using Eq. (5-13) to compute the calibration coefficient (C_Φ) we obtain:

$$C_\Phi = \frac{\alpha}{\bar{S}/S} = \frac{1.4}{0.76} = 1.842 \tag{5-30}$$

in which 1.4 is the proposed load factor for wind loads in the draft 2005 NBCC *(NRCC, 2004)*.

The reliability factor (β_o) (Figure 5.1) was chosen to be 2.5 for the subsequent calibration. The reliability index characterizes the probability of failure while considering both the resistance and load effects. The Commentary on the 2001 North American Cold-Formed Steel Specification *(AISI, 2002b)* suggests a target reliability index of between 2.5 – 4.0. The higher end reliability is required for connections, in which failure must be avoided at all costs. During the analysis of test results from the main testing program and while deriving design values from raw corrected data, it was found that, on average, a 15 % reduction was consistently apparent between the ultimate wall resistance attained during the test and the final design (yield) load level obtained through the EEEP approach. For this reason, because a measurable overstrength is already built-in to the nominal design, a reliability index of 2.5 was considered adequate for the calibration of the resistance factor of the wall systems.

Table 5.1 lists the various factors and their values incorporated into Eq. (5-16) to obtain a resistance factor for the different wall groupings that were considered. It can be deduced that the grouping of walls does not have a significant effect on the calculated resistance factor nor does the use of only positive cycles or positive / negative average cycles. Based on these results, a resistance factor (Φ) of 0.7 is recommended in the design of light gauge steel frame / wood panel shear walls subject to wind loads calculated as per the upcoming version of the NBCC *(NRCC, 2004)*, with nominal capacities as derived in this body of research and constructed in an identical manner to the wall specimens tested as part of this research program.

Table 5.1: Resistance factor calibration for wind loads

	α	S_m/S	C_Φ	M_m	F_m	P_m	βo	V_M	V_F	V_S	n	C_P	V_P	Φ
ALL TESTS - MONO/CYCLIC +/-	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	78	1.040	0.0576	**0.706**
3 INCH - MONO/CYCLIC +/-	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	26	1.129	0.0439	**0.709**
4 INCH - MONO/CYCLIC +/-	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	28	1.119	0.0643	**0.704**
6 INCH - MONO/CYCLIC +/-	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	24	1.141	0.0646	**0.703**
ALL TESTS - MONO/CYCLIC POS.	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	78	1.040	0.0529	**0.707**
3 INCH - MONO/CYCLIC POS.	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	26	1.129	0.0496	**0.708**
4 INCH - MONO/CYCLIC POS.	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	28	1.119	0.0602	**0.705**
6 INCH - MONO/CYCLIC POS.	1.4	0.76	1.842	1.05	1.00	1.00	2.50	0.11	0.10	0.37	24	1.141	0.0493	**0.708**

5.1.2 Calibration for Draft 2005 NBCC Seismic Loads

The draft version of the 2005 NBCC *(NRCC, 2004)* proposes a method for calculating the minimum base shear for use with the equivalent static force procedure for normal structures as follows:

$$V = \frac{S_{(T_a)} M_v I_E W}{R_d R_o} \tag{5-31}$$

where,

V = Minimum lateral earthquake force at base of structure

$S_{(T_a)}$ = Spectral response acceleration (function of the period of the structure and site location)

T_a = Fundamental lateral period of vibration

M_v = Factor to account for higher mode effects

I_E = Importance factor = 1.0 for normal buildings

W = Seismic weight of the structure

R_d = Ductility related force modification factor

R_o = Overstrength related force modification factor

The determination of the numerical values for the force modification factors for light gauge steel frame / wood panel shear walls is described in Boudreault *(2004)*, however, the formulation for R_o as stated in Mitchell *et al. (2003)* is as follows:

$$R_o = R_{size} \times R_{\Phi} \times R_{yield} \times \ldots . \tag{5-32}$$

where,

$$R_{\Phi} = \frac{1}{\Phi} \tag{5-33}$$

It therefore follows that R_o is directly proportional to $1/\Phi$, making the equivalent static earthquake base shear (V) directly proportional to Φ. Consequently, when the factored wall resistance is equated to lateral base shear (earthquake load factor = 1.0) consistent with limit states design, both sides of the equation contain the resistance factor, hence the actual numerical value of Φ is of no consequence in seismic design. In addition, Mitchell *et al. (2003)* states that the R_{Φ} is included in Eq. (5-32) because in earthquake resistant design, structures are, after all, being designed for such a rare event with a return period of 2500 years (probability of exceedance of 2 % in 50 years). Under these conditions, it is normal to use unfactored or nominal resistances in design. For this reason, and to be consistent, the value of the resistance factor recommended remains 0.7 for the seismic design of light gauge steel frame / wood panel shear walls, identical to that specified for wind loads.

5.2 RECOMMENDED PRESCRIPTIVE DESIGN APPROACH FOR LIGHT GAUGE STEEL FRAME / WOOD PANEL SHEAR WALLS

As mentioned previously monotonic values were averaged with the average positive cycle / negative cycle cyclic resistances according to Eq. (5-18) to produce a conservative estimate ($S_{y,avg}$) of the shear wall resistance for each wall configuration. Reversed cyclic tests were performed in addition to monotonic tests in order to justify the similarity between loading capacities and to provide for a simulation of seismic activity. It was also decided that 2' x 8' (610 x 2440 mm) shear wall segments would not be included in the resistance factor calibration because of excessive deformation under loading *(Chen, 2004)*, and so for design purposes, the use of values will be limited to shear walls 4' (1220 mm) or 8' (2440 mm) in length by 8' (2440 mm) in height, or a limiting aspect ratio of 2:1 (height:length). The Engineering Design in Wood Standard *(CSA O86, 2001)* limits the applicability of listed nominal strength values to wood frame shear walls of a maximum height-to-length ratio of 3.5:1. The draft version of the Shear Wall Design Guide *(AISI, 2002c)* recommends a reduction factor of $2L/H$ applied to the listed nominal capacity for shear walls of aspect ratio greater than 2:1, but not exceeding 4:1. In order to evaluate the performance of shear walls of varying height-to-length ratios constructed in accordance with this body of research, further testing on shear walls of various aspect ratios must be completed, *i.e.* 3' x 8' (915 mm x 2440 mm) walls. The main testing program consisted of monotonic and reversed cyclic tests (CUREE protocol) on 78 shear wall specimens of aspect ratio 2:1 or less and the average nominal strengths ($S_{y,avg}$) are listed in Table 5.2 for the various wall configurations.

Table 5.2: $S_{y,avg}$ values for shear wall specimens of aspect ratio 2:1 or less

Specimen	Sheathing Type	Sheathing Thickness (mm)	Fastener Schedule (in.)[1]	$S_{y,avg}$ (kN/m)	
				4'x8' wall (1220x2440mm)	8'x8' wall (2440x2440mm)
9-A,B,C; 10-A,B,C 33[2]-A,B,C;34[2]-A,B,C,D	CSP	12.5	3"/12"	**21.6**	**22.5**
13-A,B,C; 14-A,B,C,D	DFP	12.5	3"/12"	**24.5**	-
25-A,B,C; 26-A,B,C	OSB	11.0	3"/12"	**20.6**	-
1[3]-A,B,C; 4[3]-A,B,C 31[2]-A,B,C,D,E,F; 32[2]-A,B,C	CSP	12.5	4"/12"	**14.4**	**17.3**
5[3]-A,B,C,D; 6[3]-A,B,C	DFP	12.5	4"/12"	**19.1**	-
23-A,B,C; 24-A,B,C	OSB	11.0	4"/12"	**16.2**	-
7-A,B,C; 8-A,B,C 29[2]-A,B,C; 30[2]-A,B,C	CSP	12.5	6"/12"	**10.6**	**11.7**
11-A,B,C; 12-A,B,C	DFP	12.5	6"/12"	**12.9**	-
21-A,B,C; 22-A,B,C	OSB	11.0	6"/12"	**11.0**	-

[1]1" = 25.4 mm
[2]*(Chen, 2004)*
[3]*(Boudreault, 2004)*

From Table 5.2, it is apparent that the added strength of Douglas Fir Plywood (DFP) *(CSA O121, 1978)* contributes significantly to the overall shear wall lateral resistance, since in all cases the wall resistance obtained with Canadian Softwood Plywood (CSP) *(CSA O151, 1978)* sheathing and with Oriented Strand Board (OSB) *(CSA O325, 1992)* falls below the wall resistance obtained with DFP sheathing. For each perimeter screw spacing, a conservative estimate of the wall resistance independent of the sheathing type could be based on the lowest strength sheathing configuration in design. The only 1:1 aspect ratio walls tested were sheathed with CSP (33-A,B,C; 34-A,B,C,D; 31-A,B,C,D,E,F; 32-A,B,C; 29-A,B,C; 30-A,B,C) *(Chen, 2004)*, however, it can be assumed that a similar scenario would exist for walls sheathed with DFP and OSB, where the wall resistance (normalized by the wall length) is slightly increased due to the lower aspect ratio. This is apparent for all cases in which an 8' x 8' (2440 x 2440 mm) wall was tested with CSP sheathing. For design purposes, a lower bound of the nominal wall resistance could be estimated based on the 4' x 8' (1220 x 2440 mm) wall resistances.

In terms of wall stiffness, a similar approach is used to obtain the design recommendations. Monotonic values were averaged with the average positive cycle / negative cycle cyclic resistances as outlined by Eq. (5-18) to produce a conservative estimate of the unit wall stiffness ($k_{e,avg}$) (wall stiffness per unit length of wall). An exception to this approach exists for the 4' x 8' (1220 x 2440 mm) walls sheathed with CSP and tested monotonically (1-A,B,C) *(Boudreault, 2004)* as they provided unusually low outlying wall stiffnesses. As indicated by Note 4 in Table 5.3, the average of the positive and negative cycle and additional monotonic tests 1-E,F elastic stiffnesses were used. As shown in Table 5.3, the results are quite similar to those obtained for the nominal strength values. It is apparent that walls sheathed with OSB are the stiffest, and so, for each perimeter screw spacing, a conservative estimate of the unit wall stiffness independent of the sheathing type could be based on the lowest stiffness configuration in design. It is also apparent that the 1:1 aspect ratio walls that were tested were consistently stiffer than the 2:1 aspect ratio walls, and so, in design, a lower bound of the specified unit wall stiffness could be estimated based on the 4' x 8' (1220 x 2440 mm) wall stiffnesses.

Table 5.3: Average unit elastic stiffness ($k_{e,avg}$) (per meter wall length) values for shear wall specimens of aspect ratio 2:1 or less

Specimen	Sheathing Type	Sheathing Thickness (mm)	Fastener Schedule (in.)[1]	$k_{e,avg}$ (kN/mm/m wall length)	
				4'x8' wall (1220x2440mm)	8'x8' wall (2440x2440mm)
9-A,B,C; 10-A,B,C 33[2]-A,B,C;34[2]-A,B,C,D	CSP	12.5	3"/12"	1.16	1.27
13-A,B,C; 14-A,B,C,D	DFP	12.5	3"/12"	1.36	-
25-A,B,C; 26-A,B,C	OSB	11.0	3"/12"	1.88	-
1[3]-E,F; 4[3]-A,B,C 31[2]-A,B,C,D,E,F; 32[2]-A,B,C	CSP	12.5	4"/12"	0.97[4]	1.08
5[3]-A,B,C,D; 6[3]-A,B,C	DFP	12.5	4"/12"	1.08	-
23-A,B,C; 24-A,B,C	OSB	11.0	4"/12"	1.75	-
7-A,B,C; 8-A,B,C 29[2]-A,B,C; 30[2]-A,B,C	CSP	12.5	6"/12"	0.88	0.97
11-A,B,C; 12-A,B,C	DFP	12.5	6"/12"	0.94	-
21-A,B,C; 22-A,B,C	OSB	11.0	6"/12"	1.38	-

[1]1" = 25.4 mm
[2]*(Chen, 2004)*
[3]*(Boudreault, 2004)*
[4]This value is based on the average of the positive / negative cycles of the reversed cyclic tests and additional tests 1-E,F *(Boudreault, 2004)* since it was found that the monotonic tests (1-A,B,C) provided inconsistent results. The actual value of the unit elastic stiffness found considering both monotonic (1-A,B,C) and cyclic tests was 0.85 kN/mm/m.

Based on the above findings, tables for use in design can be formulated. Two recommendations are presented: i) Table 5.4 contains nominal strength and unit elastic stiffness values for light gauge steel frame / wood panel shear walls independent of the sheathing type but dependant on the perimeter screw spacing, and ii) Table 5.5 contains nominal strength and unit elastic stiffness values for light gauge steel frame / wood panel shear walls dependant upon sheathing type and perimeter screw spacing. Table 5.4 is a simplification of Table 5.5 and is recommended for use in design because of its concise and conservative nature, however, Table 5.5 provides a more detailed breakdown of wall configurations in order to more accurately match a particular construction configuration. This may be useful when it comes to estimating the overstrength inherent in design for seismic related capacity based design issues, or to obtain a more accurate estimate of the actual shear wall deformation or deflection.

It must be noted that the tabulated resistances do not account for gravity loading in combination with lateral loading. The designer must be aware that the compression buckling / local buckling failure may exist in the chord studs, and therefore these studs must be designed to resist the expected forces in order to preserve the overall structural integrity of the building. This failure mode may control the maximum applied lateral load (such as with Tests 13-A,B,C) and could lead to overall structural failures when gravity loads are present. In order to safeguard against compression chord buckling the designer should verify that the back-to-back chord studs are able to carry the load expected due to the factored loads on the wall assembly in wind design. Studies to quantify the effect of gravity loading on shear wall lateral performance are ongoing. For earthquake resistant design, a capacity based design approach should be used as outlined in Section 5.4. It must also be noted that the hold-down anchors in addition to the shear anchors at the bottom of the wall should be designed to resist the expected uplift and shear forces, respectively, accounting for the capacity of the wall in the case of seismic loading.

Table 5.4: Nominal shear strength, S_y (kN/m), and unit elastic stiffness, k_e (kN/mm/m), for light gauge steel frame / wood panel shear walls independent of sheathing material

Minimum nominal Panel thickness (mm)	Screw spacing at panel edges (mm)					
	75		100		150	
	S_y (kN/m)	k_e (kN/mm/m)	S_y (kN/m)	k_e (kN/mm/m)	S_y (kN/m)	k_e (kN/mm/m)
11.0 mm OSB or 12.5 mm Plywood	20.6	1.16	14.4	0.97[10]	10.6	0.88

Notes:

(1) $\Phi = 0.7$ to obtain factored resistance for design.

(2) Full-height shear wall segments of maximum aspect ratio 2:1 shall be included in resistance calculations. Increases of nominal strength for sheathing installed on both sides of the wall shall not be permitted.

(3) Tabulated values are applicable for dry service conditions (sheathing panels) and short-term load duration ($K_D = 1.15$) such as wind or earthquake loading. For shear walls under permanent loading, tabulated values must be multiplied by 0.565; and under standard term loads, tabulated values must be multiplied by 0.870.

(4) Back-to-back chord studs connected by two No. 10-16 x 3/4" (19.1 mm) screws at 12" (305 mm) o.c. equipped with industry standard hold-downs must be used for all shear wall segments with intermediate studs spaced at a maximum of 24" (610 mm) o.c. For 8' (2440 mm) long shear walls, back-to-back studs are also used at the centre of the wall to facilitate the use of a 1/2" (12.7 mm) edge spacing.

(5) All panel edges shall be fully blocked with edge fasteners installed at not less than 1/2" (12 mm) from the panel edge and fasteners along intermediate supports shall be spaced at 305 mm o.c. Sheathing panels must be installed vertically with strength axis parallel to framing members.

(6) Minimum No.8 x 1/2" (12.7 mm) framing and No. 8 x 1-1/2" (38.1 mm) sheathing screws shall be used.

(7) ASTM A653 Grade 230 MPa of minimum uncoated base metal thickness 1.12 mm steel shall be used throughout.

(8) Studs: 3-5.8" (92.1 mm) web, 1-5/8" (41.3 mm) flange, 1/2" (12.7 mm) return lip.
Tracks: 3-5/8" (92.1 mm) web, 1-3/16" (30.2 mm) flange.

(9) Plywood: CSA O151 or CSA O121.
OSB: CSA O325 minimum end use 1R24/2F16/W24.

(10) This value is based on the average of the positive / negative cycles of the reversed cyclic tests and additional tests 1-E,F *(Boudreault, 2004)* since it was found that the monotonic tests (1-A,B,C) provided inconsistent results. The actual value of the unit elastic stiffness found considering both monotonic (1-A,B,C) and cyclic tests was 0.85 kN/mm/m.

(11) The above values are for lateral loading only. It must be noted that the compression chord failure chord may exist, particularly when gravity loads exist in combination with lateral loads, and the compression chord must be designed to account for these loads.

Table 5.5: Nominal shear strength, S_y (kN/m), and unit elastic stiffness, k_e (kN/mm/m), for light gauge steel frame / wood panel shear walls dependent on sheathing material

Minimum nominal Panel thickness (mm) and Grade	**Screw spacing at panel edges (mm)**					
	75		**100**		**150**	
	S_y **(kN/m)**	k_e **(kN/mm/m)**	S_y **(kN/m)**	k_e **(kN/mm/m)**	S_y **(kN/m)**	k_e **(kN/mm/m)**
12.5 mm Canadian Softwood Plywood (CSP) CSA O151	21.6	1.16	14.4	0.97[9]	10.6	0.88
12.5 mm Douglas Fir Plywood (DFP) CSA O121	24.5	1.36	19.1	1.08	12.9	0.94
11.0 mm Oriented Strand Board (OSB) CSA O325 1R24/2F16/W24	20.6	1.88	16.2	1.75	11.0	1.38

Notes:

(1) $\Phi = 0.7$ to obtain factored resistance for design.

(2) Full-height shear wall segments of maximum aspect ratio 2:1 shall be included in resistance calculations. Increases of nominal strength for sheathing installed on both sides of the wall shall not be permitted.

(3) Tabulated values are applicable for dry service conditions (sheathing panels) and short-term load duration ($K_D = 1.15$) such as wind or earthquake loading. For shear walls under permanent loading, tabulated values must be multiplied by 0.565; and under standard term loads, tabulated values must be multiplied by 0.870.

(4) Back-to-back chord studs connected by two No. 10-16 x 3/4" (19.1 mm) screws at 12" (305 mm) o.c. equipped with industry standard hold-downs must be used for all shear wall segments with intermediate studs spaced at a maximum of 24" (610 mm) o.c. For 8' (2440 mm) long shear walls, back-to-back studs are also used at the centre of the wall to facilitate the use of a 1/2" (12.7 mm) edge spacing.

(5) All panel edges shall be fully blocked with edge fasteners installed at not less than 1/2" (12 mm) from the panel edge and fasteners along intermediate supports shall be spaced at 305 mm o.c. Sheathing panels must be installed vertically with strength axis parallel to framing members.

(6) Minimum No.8 x 1/2" (12.7 mm) framing and No. 8 x 1-1/2" (38.1 mm) sheathing screws shall be used.

(7) ASTM A653 Grade 230 MPa of minimum uncoated base metal thickness 1.12 mm steel shall be used throughout.

(8) Studs: 3-5.8" (92.1 mm) web, 1-5/8" (41.3 mm) flange, 1/2" (12.7 mm) return lip.
Tracks: 3-5/8" (92.1 mm) web, 1-3/16" (30.2 mm) flange.

(9) This value is based on the average of the positive / negative cycles of the reversed cyclic tests and additional tests 1-E,F *(Boudreault, 2004)* since it was found that the monotonic tests (1-A,B,C) provided inconsistent results. The actual value of the unit elastic stiffness found considering both monotonic (1-A,B,C) and cyclic tests was 0.85 kN/mm/m.

(10) The above values are for lateral loading only. It must be noted that the compression chord failure chord may exist, particularly when gravity loads exist in combination with lateral loads, and the compression chord must be designed to account for these loads.

The factored shear resistance of a shear wall may equal the sum of the individual full-height (storey height) shear wall segments of aspect ratio less than or equal to 2:1 (height:length). Taking these considerations into account, the factored shear resistance of light gauge steel frame / wood panel shear walls constructed in accordance with this body of research and the notes accompanying Tables 5.4 and 5.5 shall be determined as follows:

$$S_r = \sum S_{rs} \tag{5-34}$$

where,

$$S_{rs} = \Phi S_y K'_D L \tag{5-35}$$

S_r = Factored shear resistance of shear wall, [kN]

S_{rs} = Factored shear resistance of shear wall segment, [kN]

$\Phi = 0.7$

S_y = Nominal shear strength for shear wall segment in accordance with Table 5.4 or 5.4, [kN/m]

K'_D = Load duration factor (calibrated for short term loading)

= 1.0 for short term loading

= 0.565 for permanent loading

= 0.870 for standard loading

L = Length of shear wall segment parallel to direction of load, [m]

Because light gauge steel frame / wood panel shear wall behaviour is highly dependant upon the bearing and pull-through strength of the wood panel, it was decided that the load duration factor (K'_D) should be included in Eq. (5-35) to account for the influence of the duration of the applied load on wood strength. Load duration research was carried out in an experimental program undertaken by the U.S. Forest Products Laboratory in Madison, Wisconsin during the 1940s, and a curve was developed to describe the effects of load duration on wood strength *(Wood, 1960)*. In general, wood products exhibit an increased resistance to short-term loads. This approach is consistent with the Canadian Engineering in Wood Design Standard *(CSA O86, 2001)*. Furthermore, the nominal values listed in

Tables 5.4 and 5.5 are applicable for dry conditions only. Not only do wet conditions, *i.e.* an increase in equilibrium moisture content, present a problem for the steel members, but it also leads to a reduction in capacity of wood members. The tests that were carried out as part of this experimental investigation were conducted on specimens constructed with sheathing panels in a dry condition (< 12 % moisture content) and the loading protocols were assumed to provide a short-term loading to the wall specimen (Chapter 3).

5.3 Factor of Safety for Design Levels

Tables 5.6 and 5.7 present the factor of safety incorporated into design for wind loads when using the values tabulated in Table 5.5 and a resistance factor (Φ) of 0.7 in Eq. (5-35). Table 5.6 compares the design level resistance for each monotonic test to the factored resistance for the same wall configuration, and Table 5.7 provides similar results for the reversed cyclic tests. The factor of safety can be found as per Eq. (5-36) and is demonstrated in Figure 5.4. It should be noted that these factors of safety are inherent in the design for lateral loading only and do not include the effects of gravity loads in combination with lateral loads. For monotonic tests, the factor of safety falls in the range of 1.54 – 2.24, with a mean value of 1.77, a standard deviation of 0.15, and a coefficient of variation of 9 %. It is important to note, however, that this factor of safety is representative only for limit states design where factored loads are compared to factored resistances. In order to gain a better understanding, the factor of safety can be multiplied by the load factor for wind loads (1.4) *(NRCC, 2004)* to obtain an equivalent safety factor in working or allowable stress design (ASD). This computation would produce a safety factor of 2.5. In comparison to previous research that has followed an ASD approach, this factor of safety is consistent with the design guideline provided in the 2000 IBC *(ICC, 2000)* for light gauge steel frame shear walls where the ultimate wall resistance is reduced by a factor of 2.5 to obtain an allowable capacity. Moreover, the 2000 IBC handbook *(Ghosh and Chittenden,*

2001) suggests that a factor of safety of 2.0 is sufficient to safeguard wood shear walls against failure when subjected to wind loading.

$$F.S. = \frac{S_u}{S_r} \tag{5-36}$$

where,

$F.S.$ = Factor of safety for design (limit states design)

S_u = Ultimate wall resistance observed during test

S_r = Factored wall resistance (Φ=0.7)

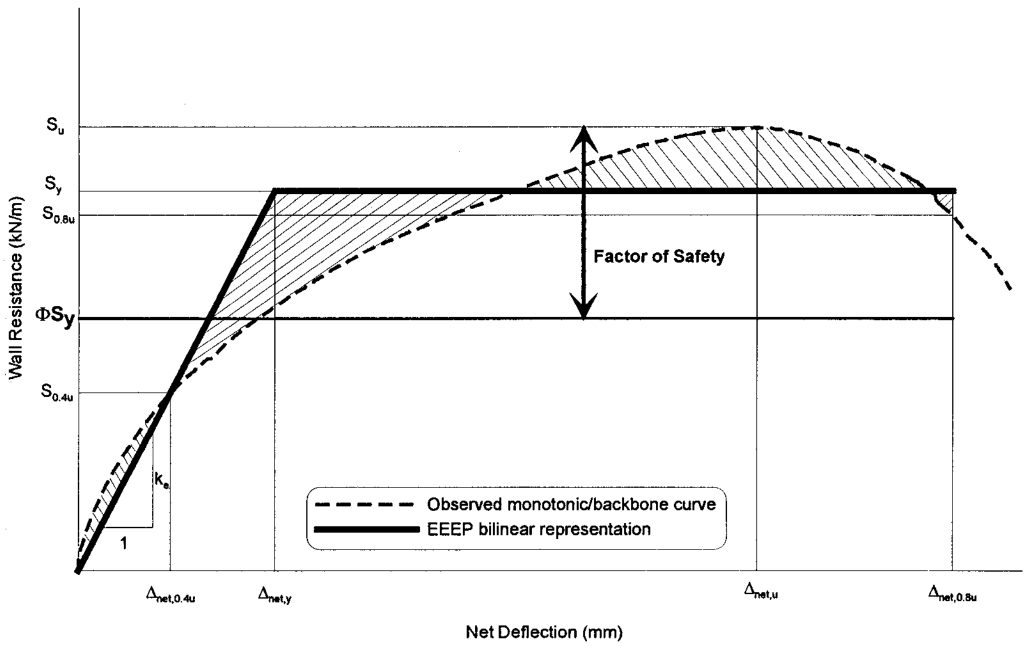

Figure 5.4: Factor of safety inherent in limit states design

Table 5.6: Factor of safety inherent in design for monotonic test values

Test	Panel Type	Fastener Schedule	Ultimate Resistance (S_u) kN/m	Yield Load (S_y) kN/m (Table 5.5)	Factored Resistance (S_r) Φ=0.7	Factor of Safety (LSD) S_u/S_r	Factor of Safety (ASD) S_u/S_r * 1.4
1A	CSP	4"/12"	15.9	14.4	10.1	1.58	2.21
1B	CSP	4"/12"	17.1	14.4	10.1	1.70	2.37
1C	CSP	4"/12"	16.8	14.4	10.1	1.66	2.33
AVERAGE	**CSP**	**4"/12"**	**16.6**	**14.4**	**10.1**	**1.65**	**2.31**
5A	DFP	4"/12"	21.1	19.1	13.4	1.58	2.21
5B	DFP	4"/12"	25.7	19.1	13.4	1.92	2.69
5C	DFP	4"/12"	23.9	19.1	13.4	1.79	2.50
5D	DFP	4"/12"	24.5	19.1	13.4	1.83	2.56
AVERAGE	**DFP**	**4"/12"**	**23.8**	**19.1**	**13.4**	**1.78**	**2.49**
7A	CSP	6"/12"	12.0	10.6	7.4	1.62	2.27
7B	CSP	6"/12"	12.6	10.6	7.4	1.69	2.37
7C	CSP	6"/12"	13.6	10.6	7.4	1.84	2.57
AVERAGE	**CSP**	**6"/12"**	**12.7**	**10.6**	**7.4**	**1.72**	**2.40**
9A	CSP	3"/12"	27.2	21.6	15.1	1.80	2.52
9B	CSP	3"/12"	23.5	21.6	15.1	1.55	2.17
9C	CSP	3"/12"	24.7	21.6	15.1	1.63	2.29
AVERAGE	**CSP**	**3"/12"**	**25.1**	**21.6**	**15.1**	**1.66**	**2.33**
11A	DFP	6"/12"	15.8	12.9	9.0	1.74	2.44
11B	DFP	6"/12"	16.9	12.9	9.0	1.87	2.62
11C	DFP	6"/12"	15.3	12.9	9.0	1.70	2.38
AVERAGE	**DFP**	**6"/12"**	**16.0**	**12.9**	**9.0**	**1.77**	**2.48**
13A	DFP	3"/12"	28.0	24.5	17.2	1.63	2.28
13B	DFP	3"/12"	30.8	24.5	17.2	1.79	2.51
13C	DFP	3"/12"	30.4	24.5	17.2	1.77	2.48
AVERAGE	**DFP**	**3"/12"**	**29.7**	**24.5**	**17.2**	**1.73**	**2.43**
21A	OSB	6"/12"	13.4	11.0	7.7	1.74	2.44
21B	OSB	6"/12"	13.1	11.0	7.7	1.70	2.38
21C	OSB	6"/12"	13.3	11.0	7.7	1.72	2.41
AVERAGE	**OSB**	**6"/12"**	**13.2**	**11.0**	**7.7**	**1.72**	**2.41**
23A	OSB	4"/12"	19.1	16.2	11.3	1.68	2.35
23B	OSB	4"/12"	20.3	16.2	11.3	1.79	2.51
23C	OSB	4"/12"	18.5	16.2	11.3	1.63	2.28
AVERAGE	**OSB**	**4"/12"**	**19.3**	**16.2**	**11.3**	**1.70**	**2.38**
25A	OSB	3"/12"	23.7	20.6	14.4	1.64	2.30
25B	OSB	3"/12"	22.2	20.6	14.4	1.54	2.16
25C	OSB	3"/12"	24.7	20.6	14.4	1.71	2.39
AVERAGE	**OSB**	**3"/12"**	**23.5**	**20.6**	**14.4**	**1.63**	**2.28**
29A	CSP	6"/12"	13.6	10.6	7.4	1.84	2.57
29B	CSP	6"/12"	13.8	10.6	7.4	1.85	2.60
29C	CSP	6"/12"	13.3	10.6	7.4	1.80	2.51
AVERAGE	**CSP**	**6"/12"**	**13.6**	**10.6**	**7.4**	**1.83**	**2.56**
31A	CSP	4"/12"	21.9	14.4	10.1	2.17	3.03
31B	CSP	4"/12"	18.8	14.4	10.1	1.86	2.61
31C	CSP	4"/12"	19.8	14.4	10.1	1.96	2.75
31D	CSP	4"/12"	19.2	14.4	10.1	1.91	2.67
31E	CSP	4"/12"	22.6	14.4	10.1	2.24	3.14
31F	CSP	4"/12"	21.0	14.4	10.1	2.08	2.91
AVERAGE	**CSP**	**4"/12"**	**20.5**	**14.4**	**10.1**	**2.04**	**2.85**
33A	CSP	3"/12"	26.1	21.6	15.1	1.72	2.41
33B	CSP	3"/12"	27.4	21.6	15.1	1.81	2.54
33C	CSP	3"/12"	25.6	21.6	15.1	1.69	2.37
AVERAGE	**CSP**	**3"/12"**	**26.4**	**21.6**	**15.1**	**1.74**	**2.44**
					AVERAGE	**1.77**	**2.48**
					STD. DEV.	**0.15**	**0.22**
					CoV	**0.0869**	**0.0869**

Table 5.7: Factor of safety inherent in design for cyclic test values

Test	Panel Type	Fastener Schedule	Ultimate Resistance (S_u) kN/m	Yield Load (S_y) kN/m (Table 5.5)	Factored Resistance (S_r) Φ=0.7	Factor of Safety (LSD) S_u/S_r	Factor of Safety (ASD) S_u/S_r * 1.4
4A	CSP	4"/12"	16.1	14.4	10.1	1.60	2.24
4B	CSP	4"/12"	17.6	14.4	10.1	1.75	2.45
4C	CSP	4"/12"	18.7	14.4	10.1	1.85	2.59
AVERAGE	**CSP**	**4"/12"**	**17.5**	**14.4**	**10.1**	**1.73**	**2.43**
6A	DFP	4"/12"	22.6	19.1	13.4	1.69	2.36
6B	DFP	4"/12"	22.9	19.1	13.4	1.72	2.40
6C	DFP	4"/12"	22.3	19.1	13.4	1.66	2.33
AVERAGE	**DFP**	**4"/12"**	**22.6**	**19.1**	**13.4**	**1.69**	**2.36**
8A	CSP	6"/12"	12.0	10.6	7.4	1.61	2.26
8B	CSP	6"/12"	11.9	10.6	7.4	1.61	2.25
8C	CSP	6"/12"	11.8	10.6	7.4	1.59	2.22
AVERAGE	**CSP**	**6"/12"**	**11.9**	**10.6**	**7.4**	**1.60**	**2.24**
10A	CSP	3"/12"	26.1	21.6	15.1	1.72	2.41
10B	CSP	3"/12"	26.9	21.6	15.1	1.78	2.49
10C	CSP	3"/12"	25.5	21.6	15.1	1.68	2.36
AVERAGE	**CSP**	**3"/12"**	**26.2**	**21.6**	**15.1**	**1.73**	**2.42**
12A	DFP	6"/12"	13.5	12.9	9.0	1.50	2.10
12B	DFP	6"/12"	16.0	12.9	9.0	1.77	2.48
12C	DFP	6"/12"	14.4	12.9	9.0	1.59	2.23
AVERAGE	**DFP**	**6"/12"**	**14.6**	**12.9**	**9.0**	**1.62**	**2.27**
14A	DFP	3"/12"	31.0	24.5	17.2	1.81	2.53
14B	DFP	3"/12"	29.0	24.5	17.2	1.69	2.37
14C	DFP	3"/12"	29.5	24.5	17.2	1.72	2.41
14D	DFP	3"/12"	29.1	24.5	17.2	1.70	2.38
AVERAGE	**DFP**	**3"/12"**	**29.7**	**24.5**	**17.2**	**1.73**	**2.42**
22A	OSB	6"/12"	11.7	11.0	7.7	1.52	2.13
22B	OSB	6"/12"	11.9	11.0	7.7	1.54	2.16
22C	OSB	6"/12"	11.5	11.0	7.7	1.49	2.09
AVERAGE	**OSB**	**6"/12"**	**11.7**	**11.0**	**7.7**	**1.52**	**2.12**
24A	OSB	4"/12"	17.0	16.2	11.3	1.50	2.10
24B	OSB	4"/12"	17.4	16.2	11.3	1.54	2.15
24C	OSB	4"/12"	17.2	16.2	11.3	1.52	2.13
AVERAGE	**OSB**	**4"/12"**	**17.2**	**16.2**	**11.3**	**1.52**	**2.13**
26A	OSB	3"/12"	24.0	20.6	14.4	1.66	2.33
26B	OSB	3"/12"	22.6	20.6	14.4	1.56	2.19
26C	OSB	3"/12"	23.9	20.6	14.4	1.65	2.32
AVERAGE	**OSB**	**3"/12"**	**23.5**	**20.6**	**14.4**	**1.63**	**2.28**
30A	CSP	6"/12"	13.5	10.6	7.4	1.82	2.55
30B	CSP	6"/12"	13.1	10.6	7.4	1.76	2.46
30C	CSP	6"/12"	13.4	10.6	7.4	1.80	2.52
AVERAGE	**CSP**	**6"/12"**	**13.3**	**10.6**	**7.4**	**1.79**	**2.51**
32A	CSP	4"/12"	20.0	14.4	10.1	1.98	2.78
32B	CSP	4"/12"	20.7	14.4	10.1	2.06	2.88
32C	CSP	4"/12"	20.4	14.4	10.1	2.02	2.83
AVERAGE	**CSP**	**4"/12"**	**20.4**	**14.4**	**10.1**	**2.02**	**2.83**
34A	CSP	3"/12"	26.8	21.6	15.1	1.77	2.48
34B	CSP	3"/12"	29.1	21.6	15.1	1.93	2.70
34C	CSP	3"/12"	28.0	21.6	15.1	1.85	2.60
34D	CSP	3"/12"	30.5	21.6	15.1	2.02	2.82
AVERAGE	**CSP**	**3"/12"**	**28.6**	**21.6**	**15.1**	**1.89**	**2.65**
					AVERAGE	**1.71**	**2.40**
					STD. DEV.	**0.15**	**0.22**
					CoV	**0.0898**	**0.0898**

The factor of safety for limit states design for the reversed cyclic tests (Table 5.7) falls in the range of 1.49 – 2.06 with a mean value of 1.71, a standard deviation of 0.15, and coefficient of variation of 9 %. When amplified by the load factor of 1.4, the factor of safety comparable to allowable stress design has a mean value of 2.4, and, as mentioned above, this value falls in the range of what is suggested in the 2000 IBC *(ICC, 2000)* and the 2000 IBC Handbook *(Ghosh and Chittenden, 2001)*. Furthermore, wind loads according to the draft 2005 NBCC *(NRCC, 2004)* are now based on a return period of 50 years providing an added factor of safety when compared to wind loads based on the previous versions of the NBCC *(NRCC, 1995)* (1 in 30 year return period).

The factor of safety for the reversed cyclic tests was determined with respect to the average ultimate wall resistance represented by the positive cycles only (S_{u+}). In all tests conducted for this research program, it was found that the ultimate wall resistance on the positive cycle was larger than the ultimate wall resistance on the corresponding displacement cycle. Since the positive cycles were executed prior to the negative cycles in the test protocol sequence, it was deemed that when pushed to failure, the walls would actually reach this larger capacity value rather than the average of the two values.

It should be apparent that the factor of safety is only significant in the case of wind design. Earthquake resistant design is somewhat different in that the structure is expected to go well into the inelastic range to provide the necessary ductility and energy absorption if R_d and R_o values greater than 1 are used. In simple terms, the earthquake is able to provide the full elastic base shear as an applied load; the level of loading is actually limited by the capacity of the lateral load carrying shear wall, which is designed as the weakest link in the lateral load carrying path. It is therefore not possible to define a factor of safety that is similar to that described above for wind loading, when it comes to a capacity based design approach with R_d and R_o greater than 1. Rather, the expected seismic performance of the shear wall is related to the ductility of the system; that is its ability to carry load without severe degradation through repeated inelastic loading cycles. The capacity based design approach is discussed in detail in the following Section.

5.4 CAPACITY BASED DESIGN

As mentioned in the above Section, current earthquake resistant design requirements follow an approach in which certain elements in the seismic force resisting system of a structure are designed to be the "energy dissipators". In a structure which makes use of light gauge steel frame / wood panel shear walls as lateral force resisting elements, the shear walls are designed to be the fuse elements which fail in a conventional sense. More specifically, at the "Life Safety" design level, the wood sheathing-to-steel framing connections of the shear wall would be relied on as they fail in a ductile fashion to dissipate energy due to seismic excitation. The capacity based design approach also stipulates that all other "connector" elements in the lateral load carrying path must be designed to withstand the expected or probable capacity of the fuse element while taking into account any overstrength which may exist. In general, by doing so, an engineer avoids having to design all of the structure to be ductile, and may concentrate their efforts on providing the necessary ductility in the fuse element. In the case of a light frame shear wall, the connector elements include the chord studs, intermediate studs, hold-downs, anchors, tracks, *etc.*, and these elements are designed to remain elastic while the sheathing to framing connections fail in a ductile manner.

The wood sheathing-to-steel connectors were chosen as the fuse element of the shear wall in order to preserve the capacity of the gravity load carrying system, *i.e.* the intermediate and chord studs, during and after an earthquake in compliance with the main goal of preventing loss of life in earthquake resistance design. The wood sheathing-to-steel connections are able to perform in a ductile manner because, as the screw rotates back and forth during reversed cyclic action, the wood surrounding the screw head crushes thereby allowing the shear wall to enter into the plastic region of behaviour. In order to design the stud elements it is necessary to estimate the probable capacity of the shear wall. This can be achieved by applying an overstrength factor to the nominal resistance specified (S_y). The overstrength factor can be found by dividing the ultimate wall resistance observed during testing by the nominal wall resistance as represented by Eq. (5-37) and Figure 5.5. Initial selection of the shear wall to resist the expected NBCC base shear should be based on a factored resistance, *i.e.* the overstrength factor should not be included during wall selection. The probable capacity is only used to estimate the forces in other connecting elements around the wall.

$$overstrength = \frac{S_u}{S_y} \tag{5-37}$$

where,

S_u = Ultimate wall resistance observed during test

S_y = Nominal yield wall resistance, Table 5.4 or 5.5

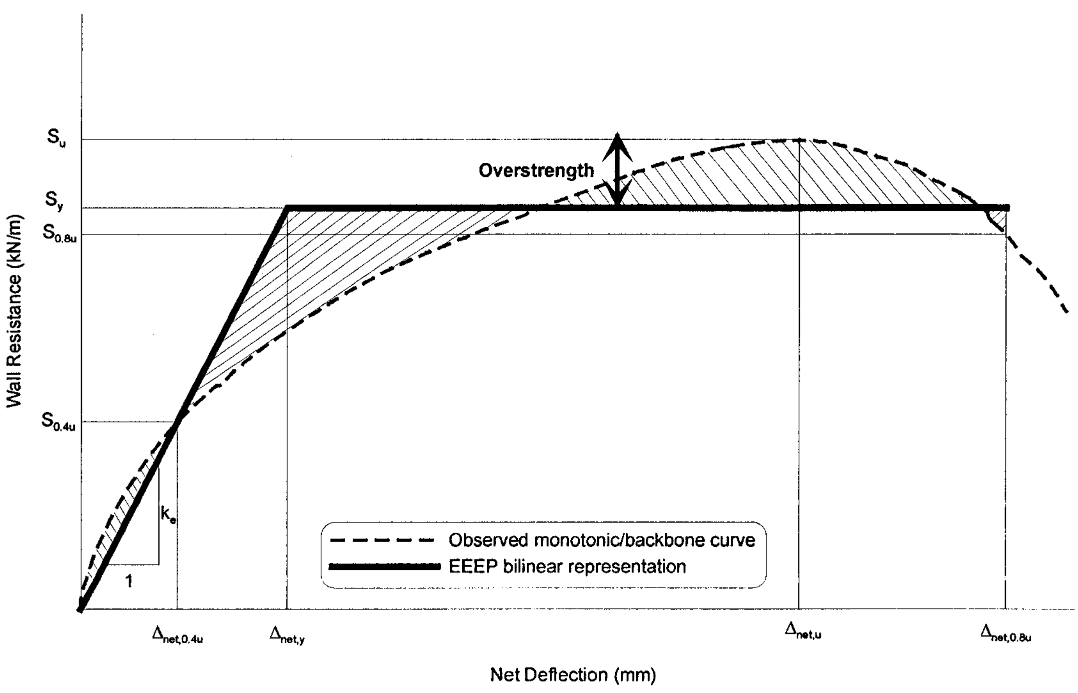

Figure 5.5: Overstrength inherent in design

Table 5.8: Overstrength inherent in design for monotonic test values

Test	Panel Type	Fastener Schedule	Ultimate Resistance (S_u) kN/m	Yield Load (S_y) kN/m (Table 5.5)	Overstrength S_u/S_y
1A	CSP	4"/12"	15.9	14.4	1.11
1B	CSP	4"/12"	17.1	14.4	1.19
1C	CSP	4"/12"	16.8	14.4	1.16
AVERAGE	**CSP**	**4"/12"**	**16.6**	**14.4**	**1.15**
5A	DFP	4"/12"	21.1	19.1	1.10
5B	DFP	4"/12"	25.7	19.1	1.34
5C	DFP	4"/12"	23.9	19.1	1.25
5D	DFP	4"/12"	24.5	19.1	1.28
AVERAGE	**DFP**	**4"/12"**	**23.8**	**19.1**	**1.25**
7A	CSP	6"/12"	12.0	10.6	1.13
7B	CSP	6"/12"	12.6	10.6	1.19
7C	CSP	6"/12"	13.6	10.6	1.28
AVERAGE	**CSP**	**6"/12"**	**12.7**	**10.6**	**1.20**
9A	CSP	3"/12"	27.2	21.6	1.26
9B	CSP	3"/12"	23.5	21.6	1.09
9C	CSP	3"/12"	24.7	21.6	1.14
AVERAGE	**CSP**	**3"/12"**	**25.1**	**21.6**	**1.16**
11A	DFP	6"/12"	15.8	12.9	1.22
11B	DFP	6"/12"	16.9	12.9	1.31
11C	DFP	6"/12"	15.3	12.9	1.19
AVERAGE	**DFP**	**6"/12"**	**16.0**	**12.9**	**1.24**
13A	DFP	3"/12"	28.0	24.5	1.14
13B	DFP	3"/12"	30.8	24.5	1.26
13C	DFP	3"/12"	30.4	24.5	1.24
AVERAGE	**DFP**	**3"/12"**	**29.7**	**24.5**	**1.21**
21A	OSB	6"/12"	13.4	11.0	1.22
21B	OSB	6"/12"	13.1	11.0	1.19
21C	OSB	6"/12"	13.3	11.0	1.20
AVERAGE	**OSB**	**6"/12"**	**13.2**	**11.0**	**1.20**
23A	OSB	4"/12"	19.1	16.2	1.18
23B	OSB	4"/12"	20.3	16.2	1.25
23C	OSB	4"/12"	18.5	16.2	1.14
AVERAGE	**OSB**	**4"/12"**	**19.3**	**16.2**	**1.19**
25A	OSB	3"/12"	23.7	20.6	1.15
25B	OSB	3"/12"	22.2	20.6	1.08
25C	OSB	3"/12"	24.7	20.6	1.20
AVERAGE	**OSB**	**3"/12"**	**23.5**	**20.6**	**1.14**
29A	CSP	6"/12"	13.6	10.6	1.29
29B	CSP	6"/12"	13.8	10.6	1.30
29C	CSP	6"/12"	13.3	10.6	1.26
AVERAGE	**CSP**	**6"/12"**	**13.6**	**10.6**	**1.28**
31A	CSP	4"/12"	21.9	14.4	1.52
31B	CSP	4"/12"	18.8	14.4	1.30
31C	CSP	4"/12"	19.8	14.4	1.38
31D	CSP	4"/12"	19.2	14.4	1.33
31E	CSP	4"/12"	22.6	14.4	1.57
31F	CSP	4"/12"	21.0	14.4	1.46
AVERAGE	**CSP**	**4"/12"**	**20.5**	**14.4**	**1.43**
33A	CSP	3"/12"	26.1	21.6	1.21
33B	CSP	3"/12"	27.4	21.6	1.27
33C	CSP	3"/12"	25.6	21.6	1.19
AVERAGE	**CSP**	**3"/12"**	**26.4**	**21.6**	**1.22**
				AVERAGE	**1.24**
				STD. DEV.	**0.11**
				CoV	**0.0869**

Table 5.9: Overstrength inherent in design for cyclic test values

Test	Panel Type	Fastener Schedule	Ultimate Resistance (S_{u+}) kN/m	Yield Load (S_y) kN/m (Table 5.5)	Overstrength S_u/S_y
4A	CSP	4"/12"	16.1	14.4	1.12
4B	CSP	4"/12"	17.6	14.4	1.22
4C	CSP	4"/12"	18.7	14.4	1.30
AVERAGE	**CSP**	**4"/12"**	**17.5**	**14.4**	**1.21**
6A	DFP	4"/12"	22.6	19.1	1.18
6B	DFP	4"/12"	22.9	19.1	1.20
6C	DFP	4"/12"	22.3	19.1	1.17
AVERAGE	**DFP**	**4"/12"**	**22.6**	**19.1**	**1.18**
8A	CSP	6"/12"	12.0	10.6	1.13
8B	CSP	6"/12"	11.9	10.6	1.13
8C	CSP	6"/12"	11.8	10.6	1.11
AVERAGE	**CSP**	**6"/12"**	**11.9**	**10.6**	**1.12**
10A	CSP	3"/12"	26.1	21.6	1.21
10B	CSP	3"/12"	26.9	21.6	1.25
10C	CSP	3"/12"	25.5	21.6	1.18
AVERAGE	**CSP**	**3"/12"**	**26.2**	**21.6**	**1.21**
12A	DFP	6"/12"	13.5	12.9	1.05
12B	DFP	6"/12"	16.0	12.9	1.24
12C	DFP	6"/12"	14.4	12.9	1.12
AVERAGE	**DFP**	**6"/12"**	**14.6**	**12.9**	**1.13**
14A	DFP	3"/12"	31.0	24.5	1.26
14B	DFP	3"/12"	29.0	24.5	1.18
14C	DFP	3"/12"	29.5	24.5	1.21
14D	DFP	3"/12"	29.1	24.5	1.19
AVERAGE	**DFP**	**3"/12"**	**29.7**	**24.5**	**1.21**
22A	OSB	6"/12"	11.7	11.0	1.06
22B	OSB	6"/12"	11.9	11.0	1.08
22C	OSB	6"/12"	11.5	11.0	1.04
AVERAGE	**OSB**	**6"/12"**	**11.7**	**11.0**	**1.06**
24A	OSB	4"/12"	17.0	16.2	1.05
24B	OSB	4"/12"	17.4	16.2	1.08
24C	OSB	4"/12"	17.2	16.2	1.06
AVERAGE	**OSB**	**4"/12"**	**17.2**	**16.2**	**1.06**
26A	OSB	3"/12"	24.0	20.6	1.16
26B	OSB	3"/12"	22.6	20.6	1.09
26C	OSB	3"/12"	23.9	20.6	1.16
AVERAGE	**OSB**	**3"/12"**	**23.5**	**20.6**	**1.14**
30A	CSP	6"/12"	13.5	10.6	1.27
30B	CSP	6"/12"	13.1	10.6	1.23
30C	CSP	6"/12"	13.4	10.6	1.26
AVERAGE	**CSP**	**6"/12"**	**13.3**	**10.6**	**1.26**
32A	CSP	4"/12"	20.0	14.4	1.39
32B	CSP	4"/12"	20.7	14.4	1.44
32C	CSP	4"/12"	20.4	14.4	1.42
AVERAGE	**CSP**	**4"/12"**	**20.4**	**14.4**	**1.41**
34A	CSP	3"/12"	26.8	21.6	1.24
34B	CSP	3"/12"	29.1	21.6	1.35
34C	CSP	3"/12"	28.0	21.6	1.30
34D	CSP	3"/12"	30.5	21.6	1.41
AVERAGE	**CSP**	**3"/12"**	**28.6**	**21.6**	**1.32**
				AVERAGE	**1.20**
				STD. DEV.	**0.11**
				CoV	**0.0898**

For monotonic tests, the overstrength factor falls in the range of 1.08 – 1.57, with a mean value of 1.24, a standard deviation of 0.11 and a coefficient of variation of 9 %. As was the case with the factor of safety, the overstrength factor was determined with respect to the average ultimate wall resistance represented by the positive cycles only (S_{u+}) in reversed cyclic tests and falls in the range of 1.04 – 1.44, with a mean value of 1.20, a standard deviation of 0.11 and a coefficient of variation of 9 %. When incorporating the capacity based design approach, it is suggested that the designer apply an overstrength factor of 1.2 to determine a probable or expected resistance of the wall. All connector elements would then be designed to perform adequately under this expected shear wall force. To be more accurate, a designer could also consult Tables 5.8 and 5.9 and apply average overstrength factors to the nominal yield wall resistance based on a specific individual wall configuration. It must be noted, however, consistent with the design approach outlined in this Chapter, the overstrength factor applies only to walls of a maximum aspect ratio of 2:1.

CHAPTER 6 CONCLUSIONS AND RECOMMENDATIONS

6.1 CONCLUSIONS

The preliminary section of this thesis presents a literature review that consists of an evaluation and analysis of: i) Existing wood frame and light gauge steel frame shear wall testing programs undertaken both in and outside North America. ii) Various standards for structural-use panels for both Canada and the United States. iii) Existing methods of establishing design parameters from shear wall test results. And iv) design parameters incorporated into current codes and prescriptive standards for both light gauge steel frame and wood frame shear walls in the United States and wood frame shear walls in Canada.

The focus of the thesis is the main shear wall testing program, which consisted of monotonic and reversed cyclic tests on 43 light gauge steel frame / wood panel shear wall specimens. These test results form part of a larger 109-wall testing program *(Boudreault, 2004; Chen, 2004)*. The author was responsible for tests on 4' x 8' (1220 mm x 2440 mm) walls (seven wall configurations) constructed of 1.12 mm thick steel framing members of 230 MPa grade. The walls were sheathed with 12.5 mm Canadian Softwood Plywood (CSP) *(CSA O151, 1978)*, 12.5 mm Douglas Fir Plywood (DFP) *(CSA O121, 1978)* or 11 mm Oriented Strand Board (OSB) *(CSA O325, 1992)* and fastened to the steel framing members at a spacing of 3" (75 mm), 4" (100 mm) or 6" (150 mm) around the perimeter of the panel. All field fasteners were spaced at 12" (305 mm).

The test results were interpreted according to the equivalent energy elastic-plastic (EEEP) method to produce design values for all wall configurations considered while abiding by stiffness and drift limitations according to the upcoming draft version of the National Building Code of Canada *(NRCC, 2004)*. The calibration of a resistance factor was performed and a limit states design approach for light gauge steel frame / wood panel shear walls was presented for both wind and seismic loads. Finally, recommendations for future research and testing were provided in order to expand on the suite of engineering guidelines developed for light gauge steel frame / wood panel shear walls used in Canada.

From the findings of the literature review, the following conclusions were drawn:

1) Performance rated oriented strand board (OSB) panels graded according to CSA O325 *(1992)* and PS2-92 *(1992)* can be used interchangeably in shear wall applications provided they are stamped with equivalent end-use and span rating designations. The performance requirements for CSA O325 and for PS2-92 rated sheathing panels are almost identical. However, PS2-92 Structural I panels have added requirements that are not covered in CSA O325 and should be considered of superior strength to CSA graded OSB panels.
2) Plywood panels manufactured under APA PS1 *(1995)* and Canadian Standards CSA O121 (DFP) *(1978)* and CSA O151 (CSP) *(1978)* differ considerably in their species make-up. Wood behaviour varies greatly with species and when considering the behaviour of light gauge steel frame / wood panel shear walls, the wood-to-frame connections have a direct impact on performance. At this time it is suggested that a designer not interchangeably use US and Canadian plywood in a shear wall assembly assuming that a similar design capacity will exist.
3) The equivalent energy elastic-plastic (EEEP) model for the interpretation of test data and the determination of a representative yield point is most suitable for light gauge steel frame / wood panel shear walls. This data interpretation technique can be used irrespective of the reversed cyclic protocol used to test the walls and it considers the post-peak deformation capacity of light frame shear walls, an important characteristic because of their highly non-linear load-deformation behaviour. In addition, this bi-linear model provides a sound basis for determining other key design

parameters related to ductility. It is suggested that this data interpretation model be used for all subsequent treatment of light gauge steel frame / wood panel shear wall test data.

From the interpretation of all test data pertaining to the 109-wall main testing program *(Branston et al., 2004)*, the following conclusions are drawn:

1) A strength (yield wall resistance) and stiffness (elastic stiffness) design value can be assigned to a given wall configuration based on the average of the monotonic and reversed cyclic test results. These design values are given in Tables 5.4 and 5.5 for all wall configurations of maximum aspect ratio of 2:1 tested under the 109-specimen program. The design values are directly applicable for walls constructed in an identical manner to those tested, used in dry conditions and subject to short-term lateral loading.
2) A resistance factor (Φ) of 0.7 is suitable for use in a limit states design format for both wind and seismic loads consistent with the load factors set out by the upcoming NBCC *(NRCC, 2004)*. This resistance factor was found to provide the necessary level of reliability and safety and takes into account the variability of the loading scenarios in addition to the variability in the wall specimens and test data itself. This resistance factor can be applied to a nominal strength presented in either Table 5.4 or Table 5.5 following Equations (5-34) and (5-35), which are reproduced as Eqs. (6-1) and (6-2) below, to obtain a factored shear resistance for a shear wall of a certain configuration. It was found that walls 2' (610 mm) in length by 8' (2440 mm) in height were too flexible and did not provide the necessary capacity at similar displacement levels as the 4' (1220 mm) and 8' (2440 mm) long walls. At this time the specified nominal capacities given in Tables 5.4 and 5.5 are therefore limited to shear walls of aspect ratio 2:1 or less.

$$S_r = \sum S_{rs} \qquad (6\text{-}1)$$

$$S_{rs} = \Phi S_y K'_D L \tag{6-2}$$

3) An overstrength of approximately 1.2 was found to exist between the nominal yield values determined for the shear walls and their measured ultimate shear capacities. The inherent added strength must be accounted for when using the nominal shear strength in a capacity based design approach for seismic loads.

6.2 RECOMMENDATIONS FOR FURTHER STUDY

Because the nominal strength and stiffness values for several wall configurations studied in this body of research are based on physical testing of full-size light gauge steel frame / wood panel shear wall specimens, their applicability is limited to future walls constructed in an identical manner. To obtain a wider range of construction configurations available for use by future designers, builders, and engineers, more testing must be completed on light gauge steel frame shear walls. These testing programs could include wall specimens containing:

- Plywood (CSP or DFP) and OSB sheathing of other thicknesses common to the construction industry. 9.5 mm, 11.0 mm, and 12.5 mm panel thicknesses are all common in today's Canadian construction practice. In addition, connection tests to determine the pull-through resistance of screw connections in Canadian plywood should be performed so that, together with lateral resistance capacities *(Okasha, 2004)*, a link can be drawn between the connection capacity in CSP and DFP, and the APA PS1 *(1995)* plywood values provided in APA E830C *(1995)*. For conservative design, a suite of light gauge steel frame / APA PS1 plywood sheathed shear wall tests should be conducted to ensure that bottom line design values are provided for use in the industry when all types of North American plywood are available to designers and constructors.

- Light gauge steel framing members of varying thicknesses. All steel used in this scope of research was of 1.12 mm thickness; most of the shear wall research conducted in previous years in the United States was on walls with 0.84 mm thick framing members. Thinner and thicker steel members also exist (0.69 mm and 1.37 mm) and should be tested in shear walls to ensure that the thickness of the studs does not affect the ductile failure mode of the assembly.
- Sheathing oriented with its strength axis perpendicular to the vertical studs with and without blocking at panel edges. It is assumed that shear walls constructed with the panels placed vertically exhibit a lower-bound strength and stiffness, however, this assumption should be verified by testing.
- Intermediate stud spacing variations. It is assumed that stud spacings of 16" (406 mm) and 12" (305 mm) would provide for a higher strength and stiffness.

In addition to the variations in wall configuration described above, it would also be important to modify the test set-up and procedure to allow for testing to be conducted on light gauge steel frame / wood panel shear walls subject to both lateral and gravity loads simultaneously. It is anticipated that overall shear wall capacity will be reduced since compression chord buckling may be a governing failure mode. It is important that the loss of the gravity load carrying system is avoided during an earthquake. Furthermore, an investigation with respect to the wind service limit state drift limit should be completed to properly quantify the displacement level expected under these loading conditions.

This body of research recommends that the equivalent energy elastic-plastic (EEEP) model be used for all data interpretation involving light gauge steel frame / wood panel shear wall tests. That being said, past research that has been completed in the United States should also be analyzed according this methodology to improve on the basis of information available to users in Canada. The literature review, in addition to the stated recommendations for future research, should provide the needed information to draw a link between panel products manufactured and / or graded according to Canadian and American standards.

Future research should also be geared towards extending the single-storey shear wall segment information contained within to a multi-storey shear wall and eventually, the complete building structure. In order to accomplish this, an emphasis should be placed on the effect of interstorey connections on the transfer of forces and system ductility as well as a non-linear time history analysis and a shake-table testing program of full-scale building models under representative earthquake records. In designing the full-scale building model, a capacity based design approach should be utilized to ensure that the ductile failure modes of the fuse elements are observed while all connector elements and gravity load carrying systems remain intact.

REFERENCES

Adams, N.R. (1965). "APA Research Report 105: Plywood Shear Walls", Technical Services Division, American Plywood Association, Tacoma, WA, USA.

Allen, D.E. (1975). "Limit States Design – A Probabilistic Study", *Canadian Journal of Civil Engineering*, Vol. 2, Iss. 1, 36 – 49.

American Forest & Paper Association (AF&PA), AF&PA/ASCE 16-95 (1995). "Standard for Load and Resistance Factor Design (LRFD) for Engineered Wood Construction", American Society of Civil Engineers, New York, NY, USA.

American Iron and Steel Institute (1998). "Shear Wall Design Guide", Publication RG-9804, American Iron and Steel Institute, Washington, DC, USA.

American Iron and Steel Institute (2002a). "2001 Edition of the North American Specification for the Design of Cold-Formed Steel Structural Members", American Iron and Steel Institute and Canadian Standards Association, Washington, DC, USA.

American Iron and Steel Institute (2002b). "Commentary on the 2001 Edition of the North American Specification for the Design of Cold-Formed Steel Structural Members", American Iron and Steel Institute and Canadian Standards Association, Washington, DC, USA.

American Iron and Steel Institute (2002c). "Standard for Cold-Formed Steel Framing Design Provisions Lateral Resistance", Feb. 2002 Draft, Washington, DC, USA.

American Iron and Steel Institute (2004). "The New Steel. Sustainable, World Leader in Recycling", www.steel.org, Washington, DC, USA.

American Plywood Association – the Engineered Wood Association, E830C (1995). "Fastener Loads for Plywood - Screws", Technical Note E830C, Tacoma, WA, USA.

American Plywood Association – the Engineered Wood Association, PRP-108 (2001). "Performance Standards and Qualification Policy for Structural-Use Panels", Tacoma, WA, USA.

American Plywood Association – the Engineered Wood Association, PS1 (1995). "Voluntary Product Standard, Construction and Industrial Plywood", Tacoma, WA, USA.

American Plywood Association – the Engineered Wood Association, PS2 (1992). "Performance Standard for Wood-Based Structural-Use Panels", Tacoma, WA, USA.

American Society for Testing and Materials (1995). "Proposed Standard Method for Dynamic Properties of Connections Assembled with Mechanical Fasteners", 4th Draft, Philadelphia, PA, USA.

American Society for Testing and Materials, A307 (2003). "Standard Specification for Carbon Steel Bolts and Studs, 60 000 psi Tensile Strength", West Conshohocken, PA, USA.

American Society for Testing and Materials, A325 (2002). "Standard Specification for Structural Bolts, Steel, Heat Treated 120/105 ksi Minimum Tensile Strength", West Conshohocken, PA, USA.

American Society for Testing and Materials, A370 (2002). "Standard Test Methods and Definitions for Mechanical Testing of Steel Products", West Conshohocken, PA, USA.

American Society for Testing and Materials, A653 (2002). "Standard Specification for Steel Sheet, Zinc-Coated (Galvanized) or Zinc-Iron Allow-Coated (Galvannealed) by the Hot-Dip Process", West Conshohocken, PA, USA.

American Society for Testing and Materials, D1037 (1999). "Standard Test Methods for Evaluating Properties of Wood-Base Fiber and Particle Panel Materials – Edgewise Shear", West Conshohocken, PA, USA.

American Society for Testing and Materials, E72 (1961). "Standard Methods of Conducting Strength Tests of Panels for Building Construction", West Conshohocken, PA, USA.

American Society for Testing and Materials, E564 (1976). "Standard Method of Static Load Test for Shear Resistance of Framed Walls for Buildings", West Conshohocken, PA, USA.

American Society of Civil Engineers, ASCE 7-98 (1998). "ASCE Standard Minimum Design Loads for Buildings and Other Structures", American Society of Civil Engineers, New York, NY, USA.

American Wood Council (1996). "Load and Resistance Factor Design Manual for Engineered Wood Construction *(inc. Supplement – Structural-Use Panels)*", 1996 Edition, American Forest & Paper Association, Washington, DC, USA.

Atherton, G.H. (1983). "Ultimate strength of structural particleboard diaphragms", *Forest Products Journal*, Vol. 33, No. 5, 22 – 26.

Bartlett, F.M., Hong, H.P., Zhou, W. (2003). "Load factor calibration for the proposed 2005 edition of the National Building Code of Canada: Statistics of loads and load effects", *Canadian Journal of Civil Engineering*, Vol. 30, No. 2, 429 – 439.

Boudreault, F.A. (2004). "Seismic Analysis of Steel Frame / Wood Panel Shear Walls", Master's Thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, QC, Canada.

Bracci, J.M., Jones, A. (1998). "Performance of bolted wood-to-concrete connections and bolted connections in plywood shear walls", *Proc., Structural Engineering Worldwide*, Paper No. T207-2, Elsevier Science, New York, NY, USA.

Branston, A.E., Boudreault, F.A., Chen, C.Y., Rogers, C.A. (2004). "Light Gauge Steel Frame / Wood Panel Shear Wall Test Data: Summer 2003", Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, QC, Canada.

Branston, A., Boudreault, F., Rogers, C.A. (2003). "Testing of Steel Frame / Wood Panel Shear Walls (Match tests of existing shear wall experiments): Progress Report", Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, QC, Canada.

British Standards Institution (BSI), BS 5268 (1988). "Structural use of timber, code of practice for timber frame walls, dwellings not exceeding three storeys", Section 6.1, London, England.

British Standards Institution (BSI), BS 5268 (1996a). "Structural use of timber, code of practice for timber frame walls, dwellings not exceeding four storeys", Section 6.1, London, England.

British Standards Institution (BSI), BS EN 594 (1996b). "Timber structures – Test methods – Racking strength and stiffness of timber frame wall panels", London, England.

Canadian Plywood Association (CANPLY) (1999). "Plywood Handbook", North Vancouver, BC, Canada.

Canadian Standards Association, 086-M89 (1989). "Engineering Design in Wood", Toronto, ON, Canada.

Canadian Standards Association, O86 (2001). "Engineering Design in Wood", Toronto, ON, Canada.

Canadian Standards Association, O121 (1978). "Douglas Fir Plywood", Rexdale, ON, Canada.

Canadian Standards Association, O151 (1978). “Canadian Softwood Plywood”, Rexdale, ON, Canada.

Canadian Standards Association, O153 (1978). “Poplar Plywood”, Rexdale, ON, Canada.

Canadian Standards Association, O325 (1992). “Construction Sheathing”, Rexdale, ON, Canada.

Canadian Standards Association, O437 (1993). “OSB and Waferboard”, Rexdale, ON, Canada.

Canadian Standards Association, O452 (1994). “Design Rated OSB”, Rexdale, ON, Canada.

Canadian Standards Association, S16.1 (1994). "Limit States Design of Steel Structures", Etobicoke, ON, Canada.

Canadian Standards Association, S16.1 (2001). "Limit States Design of Steel Structures", Etobicoke, ON, Canada.

Canadian Standards Association, S408 (1981). “Guidelines for the Development of Limit States Design”, Rexdale, ON, Canada.

Canadian Wood Council (2001). "Wood Design Manual 2001", Canadian Wood Council, Nepean, ON, Canada.

Canadian Wood Council (2002). “Introduction to Wood Design: A learning guide to complement the Wood Design Manual”, Canadian Wood Council, Nepean, ON, Canada.

Carney, J.M. (1975). “Bibliography on wood and plywood diaphragms”, *Journal of the Structural Division,* ASCE, Vol. 101, No. 11, 2423 – 2436.

Chen, C.Y. (2004). “Testing and Performance of Steel Frame / Wood Panel Shear Walls, Master’s Thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, QC, Canada.

Cheung, C.K., Itani, R.Y., Polensek, A. (1988). “Characteristics of wood diaphragms: Experimental and parametric studies”, *Wood Fiber Science*, Vol. 20, No. 4, 438 – 456.

CoLA-UCI (2001). “Report of a Testing Program of Light-Framed Walls with Wood-Sheathed Shear Panels”, Final Report to the City of Los Angeles Department of Building and Safety, Light Frame Test Committee, Subcommittee of Research Committee, Department of Civil & Environmental Engineering, University of California, Irvine, CA, USA.

Deam, B.L., Dean, J.A., Buchanan, A.H. (1991). "Full scale testing of 3-story plywood shearwalls", *Proc., Pacific Conference on Earthquake Engineering.*

Delmhorst Instrument Co. (2003). Delmhorst Moisture Meters. www.delmhorst.com

Diekmann, E.F. (1997). "Design and Design Code Issues in the Design of Diaphragms and Shearwalls", *In: Earthquake Performance and Safety of Timber Structures, G.C. Foliente (Ed.)*, Forest Products Society, Madison, WI, USA.

Dinehart, D.W., Shenton III, H.W. (1998a). "Comparison of the response of timber shear walls with and without passive dampers", *Proc., Structural Engineering Worldwide*, Paper No. T207-5, Elsevier Science, New York, NY, USA.

Dinehart, D.W., Shenton III, H.W. (1998b). "Comparison of Static and Dynamic Response of Timber Shear Walls", *Journal of Structural Engineering,* ASCE, Vol. 124, No. 6, 686 – 695.

Dinehart, D.W., Shenton III, H.W., Elliott, T.E. (1999). "The dynamic response of wood-frame shear walls with viscoelastic dampers", *Earthquake Spectra*, Vol. 15, No. 1, 67 – 86.

Dolan, J.D., Heine, C.P. (1997a). "Monotonic Tests of Wood-Frame Shear Walls with Various Openings and Base Restraint Configurations", Report No. TE-1997-001 submitted to the NAHB Research Center, Blacksburg, VA, USA.

Dolan, J.D., Heine, C.P. (1997b). "Sequential Phased Displacement Cyclic Tests of Wood-Frame Shear Walls with Various Openings and Base Restraint Configurations", Report No. TE-1997-002 submitted to the NAHB Research Center, Blacksburg, VA, USA.

Dolan, J.D., Heine, C.P. (1997c). "Sequential Phased Displacement Tests of Wood-Frame Shear Walls with Corners", Report No. TE-1997-003 submitted to the NAHB Research Center, Blacksburg, VA, USA.

Dolan, J.D., Heine, C.P. (1998). "Cyclic response of light-framed shear walls with openings", *Proc., Structural Engineering Worldwide*, Paper No. T207-3, Elsevier Science, New York, NY, USA.

Dolan, J.D., Johnson, A.C. (1997a). "Monotonic Tests of Long Shear Walls with Openings", Report No. TE-1996-001 submitted to the American Forest & Paper Association, Blacksburg, VA, USA.

Dolan, J.D., Johnson, A.C. (1997b). "Cyclic Tests of Long Shear Walls with Openings", Report No. TE-1996-002 submitted to the American Forest & Paper Association, Blacksburg, VA, USA.

Dolan, J.D., Madsen, B. (1992). "Monotonic and cyclic tests of timber shear walls", *Canadian Journal of Civil Engineering*, Vol. 19, No. 3, 115 – 422.

Douglas Fir Plywood Association (1948). "Technical Data on Douglas Fir Plywood for Engineers and Architects, Section 6: The Lateral Bearing Strength of Nailed Plywood Joints", DFPA.

Durham, J., Lam, F., Prion, G.L. (2001). "Seismic resistance of wood shear walls with large OSB panels", *Journal of Structural Engineering*, ASCE, Vol. 127, No. 12, 1460 – 1466.

Ellingwood, B.R., Galambos, T.V., MacGregor, J.G., Cornell, C.A. (1980). "Development of a probability based load criterion for American National Standard A58", NBS Special Publication 577, National Bureau of Standards, U.S. Department of Commerce, Washington, DC, USA.

Ellingwood, B.R., Tekie, P.B., (1999). "Wind Load Statistics for Probability-Based Structural Design", *Journal of Structural Engineering,* ASCE, April 1999, Vol. 125, No.4, 453 – 463.

Enjily, V., Griffiths, R.D. (1996). "The Racking Resistance of Large Wall Panels", *Proc., International Wood Engineering Conference*, New Orleans, LA, USA, Vol. 2, 321 – 328.

European Convention for Constructional Steelwork (ECCS) (1985). "Recommended Testing Procedure for Assessing the Behaviour of Structural Steel Elements under Cyclic Loads", September 1985.

Falk, R.H., Itani, R.Y. (1987). "Dynamic characteristics of wood and gypsum diaphragms", *Journal of Structural Engineering*, ASCE, Vol. 113, No. 6, 1357 – 1370.

Ficcadenti, S.J., Castle, T.A., Kazanjy, R. (1995). "Laboratory testing of as built timber diaphragm to shear wall connections", *Proc., 64th Annual SEAOC Convention*, Sacramento, CA, USA, 373 – 388.

Ficcadenti, S.J., Steiner, M., Pardoen, G., Kazanjy, R. (1998). "Cyclic load testing of wood-framed, plywood sheathed shear walls using ASTM E564 and three loading sequences", *Proc., Sixth U.S. National Conference on Earthquake Engineering.*

Filiatrault, A. (2001). "Woodframe Project: Testing and Analysis Literature Reviews", Report W-03, CUREE/Caltech Woodframe Project. Consortium of Universities for Research in Earthquake Engineering (CUREE), Richmond, CA, USA.

Foliente, G.C. (1996). "Issues in Seismic Performance Testing and Evaluation of Timber Structural Systems", *Proc., International Wood Engineering Conference*, New Orleans, LA, USA, Vol. 1, 29 – 36.

Foschi, R.O. (1992). "Reliability-Based Performance Factors for OSB", Report prepared for the Structural Board Association (SBA), Toronto, ON, Canada.

Foschi, R.O. (2000). "Reliability Applications in Wood Design", *Progress in Structural Engineering Materials*, John Wiley & Sons, Ltd., Vol. 2.

Freund, E. (2001). "Performance Comparison of Plywood vs. OSB Shear Walls", Master's Thesis, Department of Civil Engineering, University of California, Irvine, CA, USA.

Fülöp, L.A., Dubina, D. (2002). "Performance of Shear Wall Systems in Seismic Resistant Steel Buildings, Part I: Experimental Results for Wall Panels", Faculty of Civil Engineering and Architecture, Department of Steel Structures and Structural Mechanics, Politehnica University of Timisoara, Timisoara, Romania.

Fülöp, L.A., Dubina, D. (2003). "Performance of wall-stud cold-formed shear panels under monotonic and cyclic loading, Part I: Experimental research", *Thin-Walled Structures, Elsevier Science Ltd.*, Faculty of Civil Engineering and Architecture, Department of Steel Structures and Structural Mechanics, Politehnica University of Timisoara, Timisoara, Romania.

Gad, E.F., Chandler, A.M., Duffield, C.F., Hutchinson, G.L. (1999a). "Earthquake Ductility and Over-Strength in Residential Structures", *Structural Engineering and Mechanics*, Vol. 8, No. 4, 361 – 382.

Gad, E.F., Chandler, A.M., Duffield, C.F., Stark, G. (1999b). "Lateral Behaviour of Plasterboard-clad Residential Steel Frames", *Journal of Structural Engineering,* ASCE, Vol. 125, No. 1, 32 – 39.

Gad, E.F., Duffield, C.F. (1997). "Interaction Between Brick Veneer Walls and Domestic Framed Structures when Subjected to Earthquakes", *Proc., Fifteenth Australian Conference on the Mechanics of Structures and Materials*, Melbourne Victoria, Australia, 323 – 329.

Gad, E.F., Duffield, C.F. (2000). "Lateral Behaviour of Light Framed Walls in Residential Structures", *Proc., Twelfth World Conference on Earthquake Engineering*, Auckland, New Zealand, Paper 1663.

Gad, E.F., Duffield, C.F., Chandler, A.M., Stark, G. (1998). "Testing of Cold-Formed Steel Framed Domestic Structures", *Proc., Eleventh European Conference on Earthquake Engineering*, Paris, France.

Gad, E.F., Duffield, C.F., Hutchinson, G.L., Mansell, D.S., Stark, G. (1999c). "Lateral Performance of Cold-Formed Steel-Framed Domestic Structures", *Engineering Structures,* Elsevier Science Ltd., Vol. 21, 83 – 95.

Ghosh, S.K., Chittenden, R. (2001). "2000 IBC Handbook – Structural Provisions", International Conference of Building Officials, Whittier, CA, USA.

Griffiths, R.D., Wickens, H.G. (1996a). "The Derivation of Design Data from UK Timber Frame Wall Racking Tests", *Proc., International Wood Engineering Conference*, New Orleans, LA, USA, Vol. 4, 3 – 9.

Griffiths, R.D., Wickens, H.G. (1996b). "Timber Frame Walls: Design for Racking Resistance", *Proceedings of the International Wood Engineering Conference*, New Orleans, LA, USA, Vol. 2, 37 – 44.

He, M., Lam, F., Prion, H. (1997). "Lateral Resistance of Shear Walls with Regular Size OSB Panels According to CSA 0325 and CSA 0437", Report to the Structural Board Association (SBA), Department of Wood Science, University of British Columbia, Vancouver, BC, Canada.

He, M., Lam, F., Prion, G.L. (1998). "Influence of cyclic test protocol on performance of wood-based shear walls", *Canadian Journal of Civil Engineering*, Vol. 25, No. 3, 539 – 550.

He, M., Magnusson, H., Lam, F., Prion, H.G.L. (1999). "Cyclic performance of perforated wood shear walls with oversize OSB panels", *Journal of Structural Engineering*, ASCE, Vol. 125, No. 1, 10 – 18.

Heine, C.P. (1997). "Effect of Overturning Restraint on the Performance of Fully Sheathed and Perforated Timber Framed Shear Walls", Master's Thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA, USA.

Higgins, C. (2001). "Hysteretic dampers for wood frame shear walls", *Proc., 2001 Structures Congress.*

International Code Council (2000). "International Building Code 2000", 3rd Printing, Falls Church, VA, USA.

International Conference of Building Officials (1994). "Uniform Building Code – ICBO", Whittier, CA, USA

International Conference of Building Officials (1997). "Uniform Building Code – ICBO", Whittier, CA, USA.

International Conference of Building Officials Evaluation Service, Inc., ICBO ES AC130 (2002). "Acceptance Criteria for Prefabricated Wood Shear Panels, AC130" *Effective October 1, 2002*, Whittier, CA, USA.

International Organization for Standardization, ISO 16670 (2000). "Timber structures – Joints made with mechanical fasteners – Quasi-static reversed-cyclic test method – *Draft International Standard*", ISO TC 165.

Johnson, A.C. (1997). "Monotonic and Cyclic Performance of Long Shear Walls with Openings", Master's Thesis, Virginia Polytechnic Institute and State University, Blacksburg, VA, USA.

Johnson, A.C., Dolan, J.D. (1996). "Performance of Long Shear Walls with Openings", *Proc., International Wood Engineering Conference*, New Orleans, LA, USA, Vol. 2, 337 – 344.

Kamiya, F., Sugimoto, K., Mii, N. (1996). "Pseudo dynamic test of sheathed wood walls", *Proc., International Wood Engineering Conference*, Vol. 2, 187 – 194.

Karacabeyli, E., Ceccotti, A. (1996). "Test results on the lateral resistance of nailed shear walls", *Proc., International Wood Engineering Conference*, Vol. 2, 179 – 186.

Karacabeyli, E., Ceccotti, A. (1998). "Nailed wood-frame shear walls for seismic loads: Test results and design considerations", *Proc., Structural Engineers World Congress*, San Francisco, CA, USA.

Karacabeyli, E., Dolan, J.D., Ceccotti, A., Ni, C. (1999). "Comparison of Static and Dynamic Response of Timber Shear Walls – Discussion", *Journal of Structural Engineering,* ASCE, July 1999, 796 – 797.

Karacabeyli, E., Lum, C. (1999). "CSA-0325 OSB Design Values: Strength and Stiffness Capacities (Final Report)", Forintek Canada Corp, Confidential report prepared for the Structural Board Association (SBA), Toronto, ON, Canada.

Karacabeyli, E., Stiemer, S., Ni, C. (2001). "MIDPLY shearwall system", *A Structural Engineering Odyssey, Proc., 2001 Structural Congress and Exposition*, ASCE, Reston, VA, USA.

Kawai, Y., Kanno, R., Hanya, K. (1997). "Cyclic Shear Resistance of Light-Gauge Steel Framed Walls", *ASCE Structures Conference,* Poland.

Kawai, N. (1998). "Pseudo dynamic tests on shear walls", *Proc., Fifth World Conference on Timber Engineering*, 412 – 419.

Kennedy, D.J.L., Gad Aly, M. (1980). "Limit States Design of Steel Structures – Performance Factors", *Canadian Journal of Civil Engineering*, Vol. 7, 45 – 77.

Kesik, T.J., Lio, M. (1997). "Canadian Wood-Frame House Construction", Canadian Mortgage and Housing Corporation (CMHC), Canada.

Klippstein, K.H., Tarpy, T.S. (1992). "Shear Resistance of Walls with Steel Studs", Report CF 92-2, A Research Report sponsored by the American Iron and Steel Institute, March 1992.

Krawinkler, H., Parisi, F., Ibarra, L., Ayoub, A., Medina, R. (2000). "Development of a Testing Protocol for Woodframe Structures", Report W-02 covering Task 1.3.2, CUREE/Caltech Woodframe Project. Consortium of Universities for Research in Earthquake Engineering (CUREE), Richmond, CA, USA.

Lam, F., Prion, H.G.L., He, M. (1997). "Lateral resistance of wood shear walls with large sheathing panels", *Journal of Structural Engineering*, ASCE, Vol. 123, No. 12, 1666 – 1673.

Larsen, D.M. (2000). "Experimental Cyclic Tests of Timber Shear Walls Utilizing Plywood and Gypsum Wallboard Sheathing", Master's Thesis, Department of Civil Engineering, University of California, Irvine, CA, USA.

Leiva-Arevena, L. (1996). "Behaviour of timber-framed shear walls subjected to reversed cyclic lateral loading", *Proc., International Wood Engineering Conference*, Vol. 2, 201 – 206.

Madison's Report (2004). Madison's Canadian Lumber Reporter. www.madisonsreport.com

McCreless, C.S. (1977). "Shear Resistance Tests of Steel-Stud Wall Panels", Master's Thesis, Department of Civil Engineering, Vanderbilt University, Nashville, TN, USA.

McCreless, C.S., Tarpy, T.S. (1978). "Experimental Investigation of Steel Stud Shear Wall Diaphragms", *Proc., Fourth International Specialty Conference on Cold-Formed Steel Structures*, St-Louis, MO, USA, 647 – 672.

Mitchell, D., Tremblay, R., Karacabeyli, E., Paultre, P., Saatcioglu, M., Anderson, D.L. (2003). "Seismic force modification factors for the proposed 2005 edition of the National Building Code of Canada", *Canadian Journal of Civil Engineering*, Vol. 30, No. 2, 308 – 327.

National Association of Home Builders Research Center (NAHB) (1997). "Monotonic Tests of Cold-Formed Steel Shear Walls with Openings", Report prepared for the American Iron and Steel Institute, the U.S. Department of Housing and Urban Development and the National Association of Home Builders, NAHB Research Center Inc., Upper Marlboro, MD, USA.

National Research Council of Canada (1995). "National Building Code of Canada 1995 *(inc. Structural Commentaries Part 4)*", 11th edition, Ottawa, ON, Canada.

National Research Council of Canada (2004). "National Building Code of Canada 2004", 12th edition Draft, Ottawa, ON, Canada.

Nelson, E.L., Wheat, D.L., Fowler, D.W. (1985). "Structural behaviour of wood shear walls assemblies", *Journal of Structural Engineering*, ASCE, Vol. 111, No. 3, 654 – 666.

Ni, C., Karacabeyli, E. (1998). "CSA-O325 OSB Design Values: SBA Shear Wall Tests (Final Report)", Prepared for the Structural Board Association (SBA), Forintek Project No. 1778, Forintek Canada Corp., Vancouver, BC, Canada.

Okasha, A. (2004). "Evaluation of Connection Performance for Steel Frame / Wood Panel Shear Walls", M.Eng. Project Report, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, QC, Canada.

Pacific Coast Building Officials Conference (1967). "Uniform Building Code", Long Beach, CA, USA.

Parasin, A.V. (1988). "Structural Reliability Analysis of Plywood", Council of Forest Industries of British Columbia (COFI), Report 144, Vancouver, BC, Canada.

Park, R. (1989). "Evaluation of Ductility of Structures and Structural Assemblages from Laboratory Testing", *Bulletin of the New Zealand National Society for Earthquake Engineering,* Vol. 22, No. 3, 155 – 166.

Patton-Mallory, M., Wolfe, R.W. (1985). "Light-frame shear wall length and opening effects", *Journal of Structural Engineering*, ASCE, Vol. 111, No. 10, 2227 – 2239.

Pekoz, T.B., Hall, W.B. (1988). "Probabilistic Evaluation of Test Results", *Proc., Ninth International Specialty Conference on Cold-Formed Steel Structures*, University of Missouri-Rolla, Rolla, MO, USA.

Peterson, J. (1983). "Bibliography on lumber and wood panel diaphragms", *Journal of Structural Engineering*, ASCE, Vol. 109, No. 12, 2838 – 2852.

Polensek, A., Schimel, B.D. (1991). "Dynamic properties of light-frame wood subsystems", *Journal of Structural Engineering*, ASCE, Vol. 117, No. 4, 1079 – 1095.

Porter, M.L. (1987). "Sequential Phased Displacement (SPD) Procedure for TCCMAR Testing", *Proc., Third Meeting of the Joint Technical Coordinating Committee on Masonry Research*, U.S. – Japan Coordinated Earthquake Research Program, Tomamu, Japan.

Rainer, J.H., Karacabeyli, E. (2000a). "Ensuring Good Seismic Performance with Platform-Frame Wood Housing", *Construction Technology Update No. 45*, National Research Council of Canada (NRCC), Ottawa, ON, Canada.

Rainer, J.H., Karacabeyli, E. (2000b). "Performance of Wood-Frame Construction in Earthquakes", *Proc., Twelfth World Conference on Earthquake Engineering*, Report No. 2454, Auckland, New Zealand, .

Rose, J.D., Keith, E.L. (1997). "Wood Structural Panel Shear Walls with Gypsum Wallboard and Window/Door Openings", APA Research Report 157, Tacoma, WA, USA.

Rose, J.D. (1998). "Preliminary Testing of Wood Structural Panel Shear Walls Under Cyclic (Reversed) Loading", APA Research Report 158, Tacoma, WA, USA.

Ryan, T.J., Fridley, K.J., Pollock, D.G., Itani, R.Y. (2001). "Inter-Story Shear Transfer in Woodframe Buildings", Report W-22 covering Task 1.4.8.2, CUREE/Caltech Woodframe Project. Consortium of Universities for Research in Earthquake Engineering (CUREE), Richmond, CA, USA.

Salenikovich, A.J., Dolan, J.D. (1999a). "Effects of Aspect Ratio and Overturning Restraint on Performance of Light-Frame Shear Walls under Monotonic and Reverse Cyclic Loading", *Proc., Pacific Timber Engineering Conference*, Rotorua, NZ.

Salenikovich, A.J., Dolan, J.D. (1999b). "Monotonic and Cyclic Tests of Long Steel-Frame Shear Walls with Openings", Report No. TE-1999-001 submitted to the American Iron & Steel Institute, Blacksburg, VA, USA.

Salenikovich, A.J., Dolan, J.D., Easterling, W.S. (2000a). "Racking Performance of Long Steel-Frame Shear Walls", *Proc., Fifteenth International Specialty Conference on Cold-Formed Steel Structures*, St-Louis, MO, USA, 471 – 480.

Salenikovich, A.J., Dolan, J.D., Loferski, J.R., Easterling, W.S., Woeste, F., White, M.W. (2000b). "The Racking Performance of Light-Frame Shear Walls", PhD. Dissertation, Department of Wood Science and Forest Products, Virginia Polytechnic Institute and State University, Blacksburg, Virginia, USA.

Schmid, B.L., Neilsen, M., Linderman, R.R. (1994). "Narrow plywood shear panels", *Earthquake Spectra*, Vol. 10, No. 3, 569 – 588.

Serrette, R. (1997). "Behaviour of Cyclically Loaded Light Gauge Steel Framed Shear Walls", *Building to Last: Proc., Fifteenth Structures Congress*, Portland, OR, USA.

Serrette, R.L. (1998). "Seismic Design of Light Gauge Steel Structures: A Discussion", *Proc., Fourteenth International Specialty Conference on Cold-Formed Steel Structures*, St-Louis, MO, USA, 471 – 480.

Serrette, R., Encalada, J., Hall, G., Matchen, B, Nguyen, H., Williams, A. (1997a). "Additional Shear Wall Values for Light Weight Steel Framing", Report No. LGSRG-1-97, Light Gauge Steel Research Group, Department of Civil Engineering, Santa Clara University, Santa Clara, CA, USA.

Serrette, R.L., Encalada, J., Juadines, M., Nguyen, H. (1997b). "Static Racking Behaviour of Plywood, OSB, Gypsum, and FiberBond Walls with Metal Framing", *Journal of Structural Engineering,* ASCE, Vol. 123, No. 8, 1079 – 1086.

Serrette, R., Hall, G., Nguyen, H. (1996a). "Dynamic Performance of Light Gauge Steel Framed Shear Walls", *Proc., Thirteenth International Specialty Conference on Cold-Formed Steel Structures*, St-Louis, MO, USA, 487 – 498.

Serrette, R., Morgan, K.A., Sorhouet, M.A. (2002). "Performance of Cold-Formed Steel-Framed Shear Walls: Alternative Configurations", Report No. LGSRG-06-02, Light Gauge Steel Research Group, Department of Civil Engineering, Santa Clara University, Santa Clara, CA, USA.

Serrette, R., Nguyen, H., Hall, G. (1996b). "Shear Wall Values for Light Weight Steel Framing", Report No. LGSRG-3-96, Light Gauge Steel Research Group, Department of Civil Engineering, Santa Clara University, Santa Clara, CA, USA.

Serrette, R., Ogunfunmi, K. (1996). "Shear Resistance of Gypsum-Sheathed Light-Gauge Steel Stud Walls", *Journal of Structural Engineering,* ASCE, Vol. 122, No. 4, 383 – 389.

Shah, N.N. (2001). "Shear Resistance of Oriented Strand Board and Plywood-Sheathed, Light-Gauge Steel and Wood-Framed Stud Walls", Master's Thesis, Department of Civil Engineering, University of California, Irvine, CA, USA.

Shepherd, R., Allred, B.A., (1998). "Lateral load resistance of narrow plywood shear walls", *Proc., Sixth U.S. National Conference on Earthquake Engineering*, pp. 12.

Shenton III, H.W., Dinehart, D.W., Elliott, T.E. (1998). "Stiffness and energy degradation of wood frame shear walls", *Canadian Journal of Civil Engineering*, Vol. 25, No. 3, 412 – 423.

Shipp, J.G., Erickson, T.W., Rhodebeck, M. (2000). "Plywood shearwalls: Cyclical testing gives new design insight", *Structural Engineering*, July, 34 – 37.

Simpson Strong-Tie Co., Inc. (2001). "Light Gauge Steel Construction Connectors", Dublin, CA, USA.

Skaggs, T.D. (1995). "Summary of OSB Sheathed Shear Wall and Small Specimen Static Bending Tests", A Report to CSA 086 Panel Subcommittee, APA Report APA95-24, APA Technical Services Division, Tacoma, WA, USA.

Skaggs, T.D., Rose, J.D. (1996). "Cyclic Load Testing of Wood Structural Panel Shear Walls", *Proc., International Wood Engineering Conference*, New Orleans, LA, USA, Vol. 2, 195 – 200.

Smith, A.M. (2001). "Exploring a Yield Limit State for Timber Shear Walls", Master's Thesis, Department of Civil Engineering, University of California, Irvine, CA, USA.

Soltis, L.A., Patton-Mallory, M. (1986). "Strength and ductility of sheathed walls", *Proc., Eighth European Conference on Earthquake Engineering*, Vol. 4, 57 – 63.

Steel Deck Institute, Inc. (1981). "Steel Deck Institute Diaphragm Design Manual", First Edition, Canton, OH, USA.

Steel Recycling Institute (2003). www.recycle-steel.org

Stewart, W.G., Dean, J.A., Carr, A.J. (1988). "The earthquake behaviour of plywood sheathed shearwalls", *Proc., International Conference on Timber Engineering*, 248 – 261.

Structural Board Association (SBA) (2001). "OSB: Performance by Design, OSB in Wood Frame Construction, Canadian Edition 2001", Toronto (Willowdale), ON, Canada.

Structural Engineers Association of Southern California (1997). “Standard Method of Cyclic (Reversed) Load Test for Shear Resistance of Framed Walls for Buildings”, Whittier, CA, USA.

SuperDrive (2003). Grabber SuperDrive Construction Products. www.superdrive.info

Tarpy, T.S. (1980). “Shear Resistance of Steel-Stud Walls Panels”, *Proc., Fifth International Specialty Conference on Cold-Formed Steel Structures*, St-Louis, MO, USA, 331 – 348.

Tarpy, T.S., Girard, J.D. (1982). “Shear Resistance of Steel-Stud Wall Panels”, *Proc., Sixth International Specialty Conference on Cold-Formed Steel Structures*, St-Louis, MO, USA, 449 – 465.

Tarpy, T.S., Hauenstein, S.F. (1978). “Effect of Construction Details on Shear Resistance of Steel-Stud Wall Panels”, Project No. 1201-412 sponsored by the AISI, Department of Civil Engineering, Vanderbilt University, Nashville, TN, USA.

Tarpy, T.S., McBrearty, A.R. (1978). “Shear Resistance of Steel-Stud Wall Panels with Large Aspect Ratios”, Report No. CE-USS-2, Department of Civil Engineering, Vanderbilt University, Nashville, TN, USA.

Tarpy, T.S., McCreless, C.S. (1976). “Shear Resistance Tests on Steel-Stud Wall Panels”, Department of Civil Engineering, Vanderbilt University, Nashville, TN, USA.

Tissell, J.R. (1989). “Panel-Sheathed Shear Walls – Past and Future”, *In: Structural Design, Analysis and Testing. Proc., Sessions Related to Design, Analysis, and Testing,* ASCE Structures Congress, American Society of Civil Engineers, New York, NY, USA, 124 – 133.

Tissell, J.R. (1993). “Wood Structural Panel Shear Walls”, Report No. 154, APA – The Engineered Wood Association, Tacoma, WA, USA.

Trayer, G.W. (1929) revised 1947. “The Rigidity and Strength of Frame Walls”, Forest Products Laboratory, USA.

Tsai, M. (1992). “Reliability Models of Load Testing”, PhD. Dissertation, Dept. of Aeronautical and Astronautical Engineering, University of Illinois at Urbana-Champaign, IL, USA.

van de Lindt, J.W. (2004). “Evolution of Wood Shear Wall Testing, Modeling, and Reliability Analysis: Bibliography”, *Practice Periodical on Structural Design and Construction*, ASCE, February 2004, 44 – 53.

Waite, T.J. (2000). “Steel-Frame House Construction *(NAHB Research Center)*”, Craftsman Book Company, Carlsbad, CA, USA.

Wood, L. (1960). “Relation of Strength of Wood to Duration of Load”, U.S. Dept. of Agriculture, Forest Products Lab, Report No. 1916, Madison, WI, USA.

Yamaguchi, N., Minowa, C. (1998). “Dynamic performance of wooden bearing walls by shaking table test”, *Proc., Fifth World Conference on Timber Engineering*, 26 – 33.

Zacher, E.G., Gray, R.G. (1989). “Lessons learned from dynamic tests of shear panels”, *Structures Congress 1989: Structural design, analysis and testing*, ASCE, Vol. 3, 134 – 142.

Zhao, Y. (2001). “Cyclic Performance of Cold-Formed Steel Stud Shear Walls”, Master’s Thesis, Department of Civil Engineering and Applied Mechanics, McGill University, Montreal, QC, Canada.

APPENDIX 'A' EXISTING STEEL FRAME SHEAR WALL TEST PROGRAMS

Table A.1: Additional details on Serrette *et al.* *(2002)* test program

Test No.	Framing[1]	Sheathing Type	Sheathing Screw Spacing	Sheathing Screw Size	Test Protocol
1 and 2[2]	0.054" (1.37 mm) stud and track, Grade 50 ksi (345 MPa)	7/16" (11 mm) OSB one side of wall	2"/12"	No. 8	Reversed Cyclic
3 and 4[2]	0.068" (1.73 mm) stud and track, Grade 50 ksi (345 MPa)	7/16" (11 mm) OSB one side of wall	2"/12"	No. 10	Reversed Cyclic
6 and 7[2]	0.054" (1.37 mm) stud and track, Grade 50 ksi (345 MPa)	7/16" (11 mm) OSB both sides of wall	2"/12"	No. 8	Reversed Cyclic
8 and 9[2]	0.068" (1.73 mm) stud and track, Grade 50 ksi (345 MPa)	7/16" (11 mm) OSB both sides of wall	2"/12"	No. 10	Reversed Cyclic
10 and 11[3]	0.033" (0.84 mm) stud and track, Grade 33 ksi (230 MPa)	0.027" (0.69 mm) sheet steel[1] one side of wall[4], Grade 33 ksi (230 MPa)	2"/12"	No. 8	Reversed Cyclic
12 and 13[3]	0.033" (0.84 mm) stud and track, Grade 33 ksi (230 MPa)	1/2" (12.7 mm) Gypsum Wallboard[5] one side of wall, perp. to framing No blocking	4"/4"	No. 6	Monotonic
14 and 15[3]	0.033" (0.84 mm) stud and track, Grade 33 ksi (230 MPa)	1/2" (12.7 mm) Gypsum Wallboard[5] one side of wall, perp. to framing No blocking	7"/7"	No. 6	Monotonic
16 and 17[3]	0.033" (0.84 mm) stud and track, Grade 33 ksi (230 MPa)	1/2" (12.7 mm) Gypsum Wallboard[5] one side of wall, perp. to framing No blocking	8"/12"	No. 6	Monotonic
18 and 19[3]	0.033" (0.84 mm) stud and track, Grade 33 ksi (230 MPa)	1/2" (12.7 mm) Gypsum Wallboard[5] one side of wall, perp. to framing 2" (50 mm) strap at joint	4"/12"	No. 6	Monotonic
20 and 21[3]	0.033" (0.84 mm) stud and track, Grade 33 ksi (230 MPa)	1/2" (12.7 mm) Gypsum Wallboard[5] one side of wall, perp. to framing No blocking	4"/12"	No. 6	Monotonic

[1]ASTM A653 or A792 or A875 steel
[2]Simpson S/HD10 hold-downs used with 33 No. 10 screws, No. 8 mod. truss head framing screws
[3]Simpson S/HD15 hold-downs used with 48 No. 10 screws, No. 10 pancake head framing screws
[4]1.5" (38 mm) lap joint at mid-height
[5]Type X ASTM C36
Wall configurations 1 – 11 4' x 8' walls; wall configurations 12 – 21 8' x 8' walls
1" = 25.4 mm

APPENDIX 'B' REVERSED CYCLIC TEST PROTOCOLS

Table B.1: CUREE cyclic protocol for tests 8-A,B,C

$\Delta=0.6*\Delta_m$	40.26	Screw Pattern:	6"/12"
		Sheathing:	CSP

	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	2.013	2.451	6
0.075 Δ	3.019	3.613	1
0.056 Δ	2.264	2.742	6
0.100 Δ	4.026	4.763	1
0.075 Δ	3.019	3.613	6
0.200 Δ	8.051	9.418	1
0.150 Δ	6.038	7.082	3
0.300 Δ	12.077	14.167	1
0.225 Δ	9.058	10.635	3
0.400 Δ	16.102	18.898	1
0.300 Δ	12.077	14.167	2
0.700 Δ	28.179	32.627	1
0.525 Δ	21.134	24.661	2
1.000 Δ	40.256	46.460	1
0.750 Δ	30.192	34.862	2
1.500 Δ	60.383	69.385	1
1.125 Δ	45.288	52.401	2
2.000 Δ	80.511	94.475	1
1.500 Δ	60.383	69.385	2

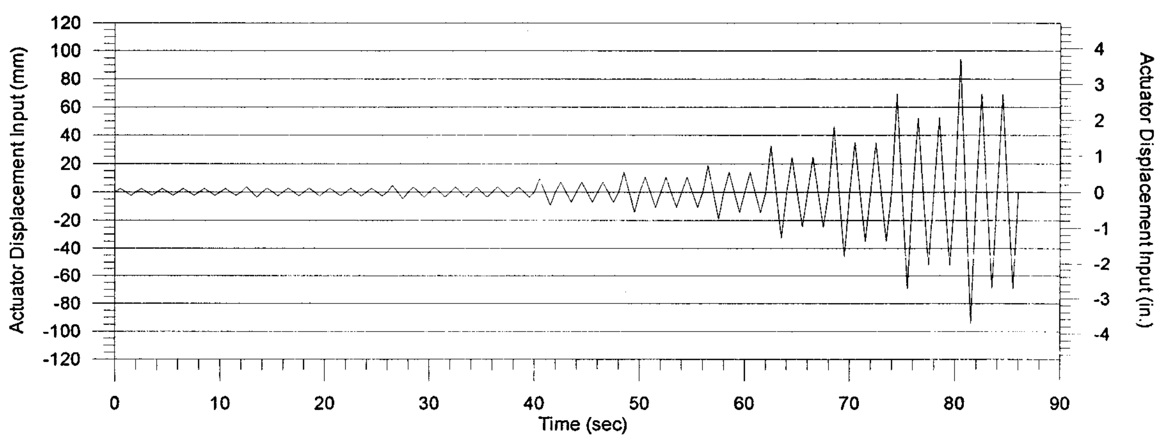

Figure B.1: CUREE cyclic protocol for tests 8-A,B,C

Table B.2: CUREE cyclic protocol for tests 10-A,B,C

$\Delta=0.6*\Delta_m$	41.79	Screw Pattern: 3"/12"	
		Sheathing: CSP	

	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	2.090	2.770	6
0.075 Δ	3.134	4.123	1
0.056 Δ	2.351	3.108	6
0.100 Δ	4.179	5.550	1
0.075 Δ	3.134	4.123	6
0.200 Δ	8.358	11.372	1
0.150 Δ	6.269	8.450	3
0.300 Δ	12.537	17.097	1
0.225 Δ	9.403	12.841	3
0.400 Δ	16.716	23.115	1
0.300 Δ	12.537	17.097	2
0.700 Δ	29.253	39.225	1
0.525 Δ	21.940	29.740	2
1.000 Δ	41.790	56.002	1
0.750 Δ	31.343	42.109	2
1.500 Δ	62.686	83.348	1
1.125 Δ	47.014	62.706	2
2.000 Δ	83.581	103.112	1
1.500 Δ	62.686	83.348	2

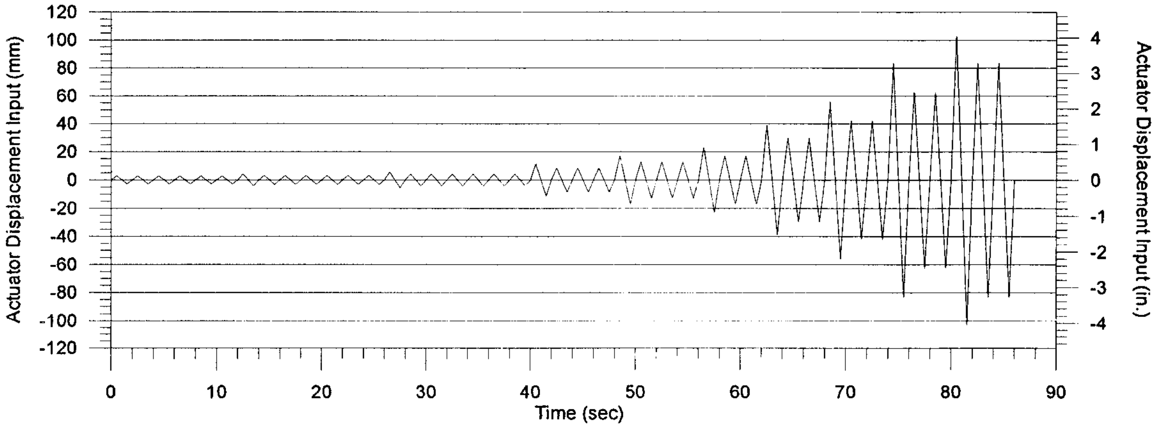

Figure B.2: CUREE cyclic protocol for tests 10-A,B,C

Table B.3: CUREE cyclic protocol for tests 12-A,B,C

$\Delta=0.6*\Delta_m$	41.66	Screw Pattern: 6"/12"	
		Sheathing: DFP	

	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	2.083	2.383	6
0.075 Δ	3.125	3.551	1
0.056 Δ	2.343	2.676	6
0.100 Δ	4.166	4.704	1
0.075 Δ	3.125	3.551	6
0.200 Δ	8.332	9.712	1
0.150 Δ	6.249	7.178	3
0.300 Δ	12.498	14.602	1
0.225 Δ	9.374	10.923	3
0.400 Δ	16.665	19.492	1
0.300 Δ	12.498	14.602	2
0.700 Δ	29.163	33.954	1
0.525 Δ	21.872	25.611	2
1.000 Δ	41.661	48.648	1
0.750 Δ	31.246	36.318	2
1.500 Δ	62.492	72.965	1
1.125 Δ	46.869	54.912	2
2.000 Δ	83.323	94.475	1
1.500 Δ	62.492	72.965	2

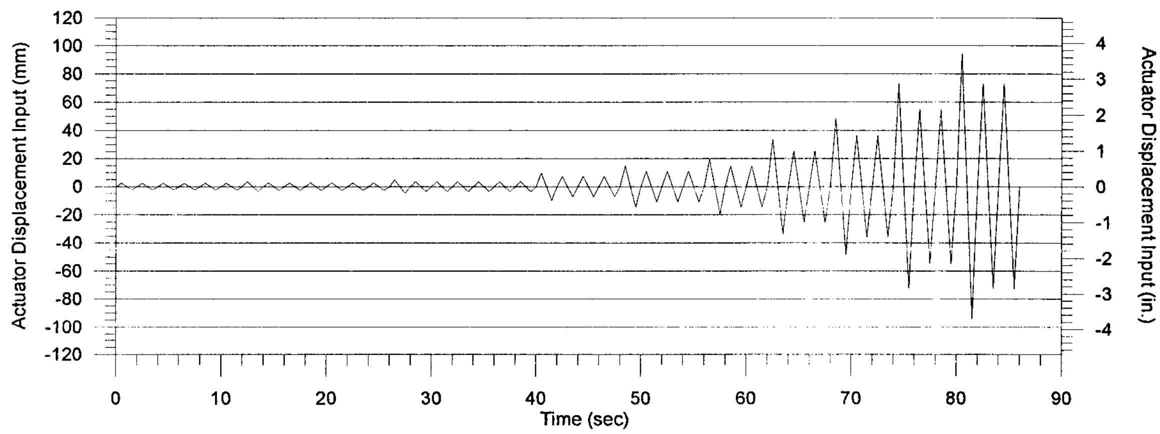

Figure B.3: CUREE cyclic protocol for tests 12-A,B,C

Table B.4: CUREE cyclic protocol for tests 14-A,B,C,D

$\Delta=0.6*\Delta_m$	37.51	Screw Pattern: 3"/12"	
		Sheathing: DFP	
	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	1.875	2.505	6
0.075 Δ	2.813	3.757	1
0.056 Δ	2.110	2.818	6
0.100 Δ	3.751	5.010	1
0.075 Δ	2.813	3.757	6
0.200 Δ	7.501	10.020	1
0.150 Δ	5.626	7.515	3
0.300 Δ	11.252	15.029	1
0.225 Δ	8.439	11.272	3
0.400 Δ	15.002	20.039	1
0.300 Δ	11.252	15.029	2
0.700 Δ	26.254	35.069	1
0.525 Δ	19.691	26.302	2
1.000 Δ	37.506	50.098	1
0.750 Δ	28.130	37.574	2
1.500 Δ	56.259	75.147	1
1.125 Δ	42.194	56.361	2
2.000 Δ	75.012	100.197	1
1.500 Δ	56.259	75.147	2

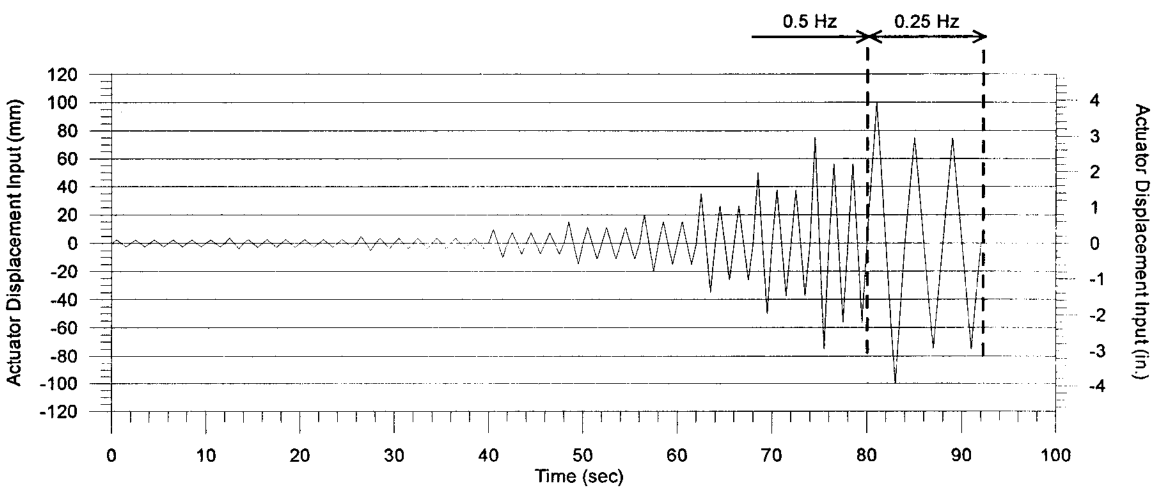

Figure B.4: CUREE cyclic protocol for tests 14-A,B,C,D

Table B.5: CUREE cyclic protocol for tests 22-A,B,C

$\Delta=0.6*\Delta_m$	32.82	Screw Pattern: 6"/12"	
		Sheathing: OSB	
	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	1.641	1.922	6
0.075 Δ	2.461	2.883	1
0.056 Δ	1.846	2.162	6
0.100 Δ	3.282	3.844	1
0.075 Δ	2.461	2.883	6
0.200 Δ	6.564	7.688	1
0.150 Δ	4.923	5.766	3
0.300 Δ	9.845	11.532	1
0.225 Δ	7.384	8.649	3
0.400 Δ	13.127	15.377	1
0.300 Δ	9.845	11.532	2
0.700 Δ	22.972	26.909	1
0.525 Δ	17.229	20.182	2
1.000 Δ	32.818	38.441	1
0.750 Δ	24.613	28.831	2
1.500 Δ	49.226	57.662	1
1.125 Δ	36.920	43.247	2
2.000 Δ	65.635	76.883	1
1.500 Δ	49.226	57.662	2

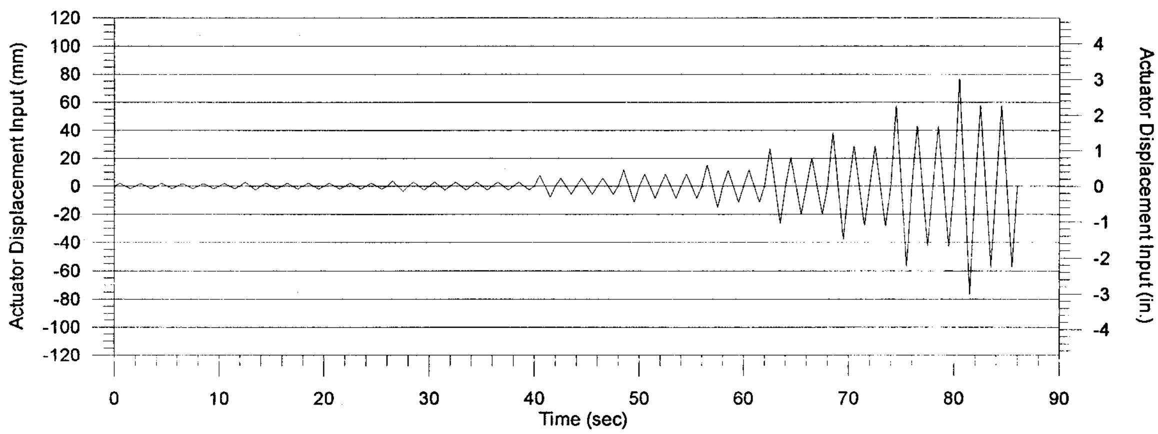

Figure B.5: CUREE cyclic protocol for tests 22-A,B,C

Table B.6: CUREE cyclic protocol for tests 24-A,B,C

$\Delta=0.6*\Delta_m$	29.64	Screw Pattern: 4"/12"	
		Sheathing: OSB	
	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	1.482	1.862	6
0.075 Δ	2.223	2.794	1
0.056 Δ	1.667	2.095	6
0.100 Δ	2.964	3.725	1
0.075 Δ	2.223	2.794	6
0.200 Δ	5.928	7.450	1
0.150 Δ	4.446	5.587	3
0.300 Δ	8.892	11.174	1
0.225 Δ	6.669	8.381	3
0.400 Δ	11.856	14.899	1
0.300 Δ	8.892	11.174	2
0.700 Δ	20.748	26.074	1
0.525 Δ	15.561	19.555	2
1.000 Δ	29.640	37.248	1
0.750 Δ	22.230	27.936	2
1.500 Δ	44.459	55.872	1
1.125 Δ	33.345	41.904	2
2.000 Δ	59.279	74.496	1
1.500 Δ	44.459	55.872	2

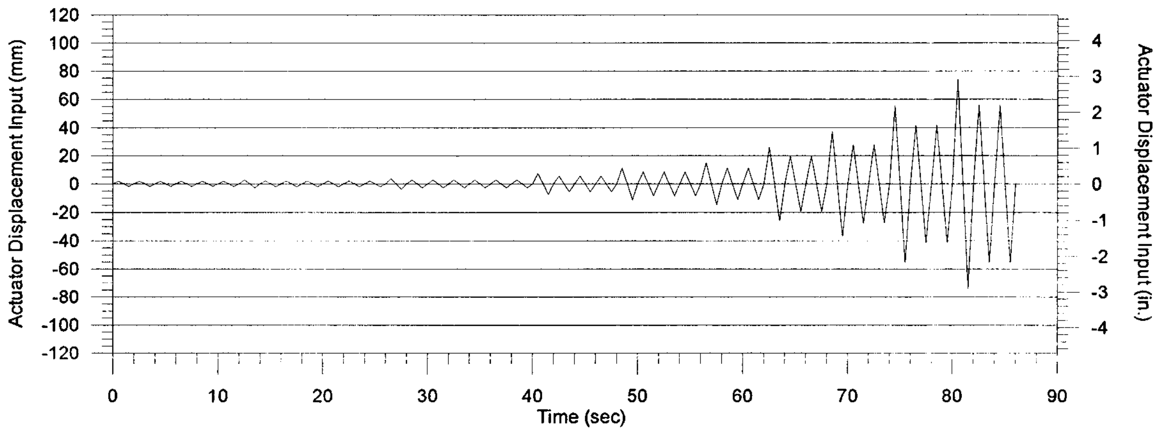

Figure B.6: CUREE cyclic protocol for tests 24-A,B,C

Table B.7: CUREE cyclic protocol for tests 26-A,B,C

$\Delta=0.6*\Delta_m$	28.07	Screw Pattern: 3"/12"	
		Sheathing: OSB	
	Target (corr.)	Actuator Input	
Displ.	mm	mm	No. Of cycles
0.050 Δ	1.403	1.927	6
0.075 Δ	2.105	2.891	1
0.056 Δ	1.579	2.168	6
0.100 Δ	2.807	3.855	1
0.075 Δ	2.105	2.891	6
0.200 Δ	5.614	7.709	1
0.150 Δ	4.210	5.782	3
0.300 Δ	8.420	11.564	1
0.225 Δ	6.315	8.673	3
0.400 Δ	11.227	15.419	1
0.300 Δ	8.420	11.564	2
0.700 Δ	19.647	26.983	1
0.525 Δ	14.736	20.237	2
1.000 Δ	28.068	38.547	1
0.750 Δ	21.051	28.911	2
1.500 Δ	42.102	57.821	1
1.125 Δ	31.576	43.366	2
2.000 Δ	56.136	77.095	1
1.500 Δ	42.102	57.821	2

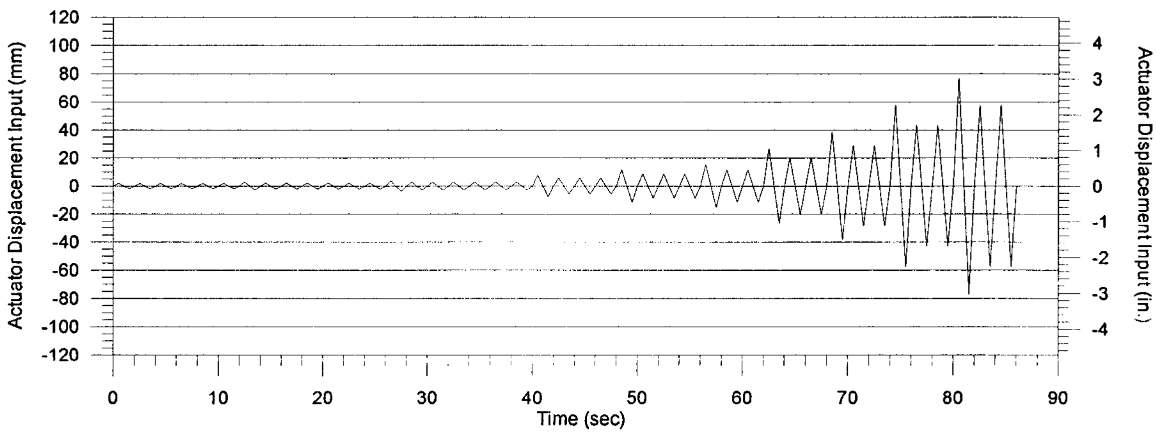

Figure B.7: CUREE cyclic protocol for tests 26-A,B,C